MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

JENKINS BRIDGE
CRAM STREET OVER GREAT WORKS STREAM
BRADLEY, MAINE

STATE OF MAINE
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Penobscot County
PIN 16687.00

Soils Report No. 2009-34
Bridge No. 3365

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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for the replacement of Jenkins Bridge over the Great Works Stream in Bradley, Maine. The proposed replacement bridge will be twin 10-foot high by 15-foot wide concrete box culverts constructed in stages. The bridge will be widened to 24 feet with 10-foot travel lanes and 2-foot shoulders, as well as accommodation for guardrail, to address MaineDOTBridge Design Guide (BDG) standards for local roads. The total span length will be on the order of 38 feet and there are no vertical or horizontal alignment changes planned. The design and construction recommendations below are discussed in greater detail in Section 7.0 Foundation Considerations and Recommendations.

Box Culvert Design and Construction – The concrete box culverts will be supplier-designed and the design shall consider all relevant strength, service and extreme limit states and load combinations in accordance with the AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, with 2009 Interims (herein referred to as LRFD). The culverts will be constructed in general conformance with BDG Section 8, Buried Structures, and Special Provision 534, Precast Structural Concrete Arches, Box Culverts. A copy of the special provision is presented in Appendix D, Special Provision. The box culvert designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

The soil envelope bedding and backfill shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill. Bedding and/or backfill should be placed in lifts 6 to 8 inches thick loose measure and compacted to manufacturer's specifications, but in no case shall the bedding and/or backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

Culvert Headwall Design - Culvert headwalls should consider all relevant LRFD strength and service limit states and load combinations and be designed to resist and/or absorb lateral earth loads, vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culverts.

Culvert headwall sections that are fixed to the box culverts to resist movement should be designed for earth pressure using an at-rest earth pressure coefficient, K_0, of 0.5. Headwall sections that are independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a, equal to 0.31. This assumes level backslope. The earth pressure coefficient may change if backslope conditions are different.

The bearing resistance for the wall footing shall be checked for the extreme and service limit states with a resistance factor of 1.0. The wall must be designed so that the factored bearing resistance, in conjunction with the depth of scour resulting from the design flood, provides adequate resistance to support the factored strength limit state loads. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor, φ, of 0.65. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated scour depth.

The designer shall apply a sliding resistance factor, φ_τ, of 0.80 to the nominal sliding resistance of cast-in-place concrete spread footings on soil. For footings on soil, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth (1/4) of the footing dimensions, in either direction. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.45 (tan 24 degrees) at the foundation soil to footing concrete interface.

Bearing Resistance for Box Culverts and Spread Footings – The factored bearing resistance at the strength limit state for box culverts on compacted fill or native glacial till should not exceed 6.0 ksf. The factored bearing resistance for footings 2 to 4 feet wide should not exceed 3.5 ksf. Based on presumptive bearing resistance values, a factored bearing resistance of 6.0 ksf may be used when analyzing box bottom slabs or wall footings for the service limit state and for preliminary footing sizing. In no instance shall the bearing stress exceed the nominal resistance of the footing concrete, which may be taken as $0.3f'c$. The minimum footing size is 2 feet wide regardless of the applied bearing pressure or bearing material.

Settlement – We estimate that settlement as a result of fill replacement and minor embankment fill extensions over natural soils will be negligible. We have estimated that the total settlement of a prepared subgrade consisting of compacted fill or native glacial till will be on the order of ½-inch or less for box culverts or footings. In both cases, this settlement is acceptable and will occur during construction. Post-construction settlement will be negligible.

Scour Protection – The box culverts will be fitted with concrete headwalls and inlet and outlet seepage cutoff walls below the culvert, all to provide scour protection. We recommend that the bridge approach slopes be armored with a 3-foot thick layer of riprap up and down alignment beyond the headwall. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot thick layer of bedding material conforming to Standard Specification 703.19, Granular Borrow for Underwater Backfill. Riprap shall meet the requirements of Section 703.26, Plain and Hand Laid Riprap of Special Provision 703, Aggregates. The riprap slope protection should be constructed no steeper than a maximum 1.75:1 (H:V) extending from the edge of roadway down to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation.

Frost Protection – Foundations placed on granular soils shall be founded a minimum of 4.5 feet below finish exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on soil and not those founded on bedrock.

Seismic Design Considerations – Since these buried structures do not cross active faults, no seismic analysis is required.

Construction Considerations –

Excavation

- Construction of the new concrete box culverts will require staged construction and soil excavation. Earth support systems may be required.
- Protect the excavated subgrade from exposure to water and unnecessary construction traffic. Remove and replace water-softened, disturbed, or rutted subgrade soil with compacted gravel borrow.

Dewatering

- Control groundwater and surface water infiltration to permit construction in-the-dry.
- Temporary ditches, French drains, pumping from sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment may be needed to divert groundwater if significant seepage is encountered during excavation.

Reuse of Excavated Soil and Bedrock

- Do not use excavated existing subbase aggregate or approach fill soil for pavement structure construction or to re-base shoulders. Excavated subbase sand and gravel or granular fill may be used as fill below subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met.

Embankment Fill Areas

- Bench existing fill slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, where new fill slope extensions are constructed over existing slopes.

Erosion Control

- Use MaineDOT Best Management Practices February 2008 to minimize erosion of fine-grained soils found on the project site.

1.0 INTRODUCTION

The Maine Department of Transportation (MaineDOT) plans to replace Jenkins Bridge carrying Cram Street over Great Works Stream in the Town of Bradley, Penobscot County, Maine. We show the project location on Sheet 1, Site Location Map, appended to this report. We conducted subsurface investigations at the bridge site to develop geotechnical recommendations for the bridge replacement. This report summarizes our findings, discusses our evaluation of the subsurface conditions and presents our geotechnical recommendations for design and construction of the bridge foundations.

The existing twin 17-foot structural plate pipe arch culverts were built in 1971 by the Town of Bradley. Consequently, there are no existing bridge plans in MaineDOT archives. From maintenance records, the twin pipes cover a roadway length of about 39 feet. The outboard embankment slopes between the pipe arches consists of concreted rip rap placed in the mid-eighties to minimize erosion of the rip rap under high water conditions. The bridge had a sufficiency rating of 42.2 in 2008.

MaineDOT is proposing twin, 10-foot high by 15-foot wide, concrete box culverts to replace the existing twin plate arch structures. The new bridge will be on the same horizontal and vertical alignment. The new bridge will have a rail-to-rail width of approximately 24 feet. Current plans include construction of concrete culvert headwalls and armoring the approach embankments with riprap.

2.0 GEOLOGIC SETTING

The Maine Geologic Survey "Surficial Geology of Orono Quadrangle, Maine, Open-file No. 81-6" (1981) indicates that surficial soils in the vicinity of Jenkins Bridge consist of Presumpscot Formation sands, silt, and clays with nearby soil unit contacts with glacial till deposits which consist of heterogeneous mixtures of sand, silt, clay and stones. The latter are the predominant soils at the site based on our subsurface explorations.

According to the "Bedrock Geologic Map of Maine" (1985), the bedrock at the Jenkins Bridge site consists of Silurian-Ordovician, calcareous sandstone, interbedded sandstone and impure limestone of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

We investigated subsurface conditions at the site by drilling two test borings, BB-BGWS-101 and BB-BGWS-102, conducted by the MaineDOT drill crew on August 4 and 5, 2009. The borings were terminated with bedrock cores. The boring locations and soil profile are shown on Sheet 2, Boring Location and Interpretive Subsurface Profile. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on Sheet 3, Boring Logs, and in Appendix A, Boring Logs, provided at the end of this report.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field and laboratory testing requirements. A Haley & Aldrich geologist logged the subsurface conditions encountered on the field logs. The field crew tied down the boring locations by taping distances to adjacent site features.

We used solid stem auger and cased wash boring techniques to conduct the borings. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The standard penetration resistances, or N-values, discussed in this report are corrected for average hammer energy transfer. We compute the corrected or, N_{60}-values, by applying an average hammer energy transfer factor of 0.84 to the raw field N-values obtained with the MaineDOT drill rig. Bedrock was cored using an NQ-2 core barrel producing a 2.0-inch diameter rock core.

4.0 LABORATORY TESTING

We conducted a laboratory soil testing program on selected samples recovered from the test borings to evaluate soil classification, material reuse, and subgrade soil properties. Laboratory testing consisted of eleven (11) standard grain size analyses with natural water contents tests. We present results of laboratory testing in Appendix B, Laboratory Test Data. The AASHTO and Unified Soil Classification System (USCS) soil classifications and water content data are also presented on the boring logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

Regional surficial geology maps show that the bridge site is situated in an area of marine sediment deposits. However, the bridge is situated in an ancient downcut stream channel and floodplain 50 to 100 feet wide up and down stream, respectively. Thus we did not observe any marine sediments at the boring locations adjacent to the bridge. Those soils have likely been eroded away down to the glacial till surface approximately at the stream level.

The bridge itself is situated at the end of short fill extensions built into the Great Works Stream flood plain. The approach embankment soil up and down station from the existing culverts is predominantly granular fill overlying approximately 27 to 32 feet of glacial till. The glacial till overlies bedrock at both boring locations. We observed metamorphic schist bedrock at both boring locations. We present a profile depicting the generalized soil stratigraphy at the bridge site on Sheet 2, Boring Location Plan and Interpretive Subsurface Profile, provided at the end of this report. A summary description of the subsurface conditions follows.

5.1 Granular Fill

We encountered granular fill to a depth of approximately 11.5 and 15.1 feet below ground surface (bgs) in BB-BGWS-101 and BB-BGWS-102, respectively. The granular fill consists of fine to coarse sandy gravel with trace to little silt or fine to coarse sand with trace to some gravel and trace to little silt. The SPT N_{60}-values in the granular fill ranged from 11 to 39 blows per foot (bpf) indicating that the unit is medium dense to dense in consistency.

The granular fill samples had water contents ranging between approximately 3 and 13 percent. Grain size analyses conducted on selected samples of the fill soils indicate that the soils are classified as A-1-a, A-1-b, and A-2-4 by the AASHTO Classification System and GW-GM, SP-SM, SW-SM, and SM under the Unified Soil Classification System.

5.2 Glacial Till

The glacial till found in the borings generally comprised of silt with some sand and trace to little gravel with occasional cobbles or sandy silt with trace to little gravel and occasional cobbles. The thickness of this soil unit ranged between approximately 27 to 32 feet. SPT N_{60}-values ranged from 20 to 60 bpf, indicating the fine-grained subunits are very stiff to hard and the sandy deposits are medium dense to dense in consistency. We generally observed the glacial till unit over bedrock in each of the borings.

The glacial till samples had water contents ranging between approximately 9 and 16 percent. Grain size analyses conducted on selected samples of the till soils indicate that the soils are classified as A-4 by the AASHTO Classification System and SM and ML under the Unified Soil Classification System.

5.3 Bedrock

We encountered bedrock at approximate depths of 43.4 and 42.1 feet bgs at BB-BGWS-101 and BB-BGWS-102, respectively. Locally, the bedrock is mapped as calcareous sandstone, interbedded sandstone and impure limestone of the Vassalboro Formation. Visual identification of rock cores indicates that the bedrock at both borings is grey, metamorphic, fine-grained schist that is moderately hard to hard, slightly weathered, and slightly fractured. We determined that the rock quality designation (RQD) of the bedrock ranged from 60 to 92 percent which correlates to a fair to excellent rock mass quality. The table below summarizes the top of bedrock elevations at the boring locations:

Substructure	Boring	Station	Depth to Bedrock (feet bgs)	Elevation of Bedrock Surface (feet)
Abutment No. 1	BB-BGWS-101	3+10.6, 6.8 LT	43.4	58.1
Abutment No. 2	BB-BGWS-102	3+58.4, 9.6 RT	42.1	59.1

Bedrock Depth and Elevation at the Boring Locations

5.4 Groundwater

We observed groundwater levels at approximate depths of 11.5 and 6.8 feet bgs at BB-BGWS-101 and BB-BGWS-102, respectively. However, the groundwater level will fluctuate with seasonal changes, runoff, and adjacent construction activities.

For a more detailed description of the subsurface conditions, please refer to Appendix A, Boring Logs attached to this report.

6.0 FOUNDATION ALTERNATIVES

The project team considered three alternate replacement designs: 1) steel girder on H-pile supported integral abutments; 2) structural plate pipe arches; and 3) concrete box culverts. The project team selected alternate No. 3, concrete box culvert, for the replacement structure. The following section presents geotechnical design recommendations for the concrete box culvert alternate.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The design team has selected twin concrete box culverts to replace the bridge at the Bradley site. The proposed replacement bridge will consist of twin 10-foot high by 15-foot wide concrete box culverts. The new bridge will be on the same horizontal and vertical alignment as the existing bridge. The new bridge will have a rail-to-rail width of approximately 24 feet. The design methodology used in the following evaluation is referenced from the AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, with 2009 Interims. See Appendix C, Calculations, for supporting documentation for the design parameters discussed below.

7.1 Box Culvert Design and Construction

Precast concrete boxes are detailed on the contract plans with only basic layout and required hydraulic opening so that the contractor may choose among available proprietary products. The manufacturer is responsible for the design of the structure in accordance with Special Provision 534, Precast Structural Concrete Arches, Box Culverts, which includes determination of the wall thickness, haunch thickness and reinforcement. The loading specified for the structure should be Modified HL-93 Strength 1, in which the HL-93 wheel loads aer increased by a factor of 1.25. The designer should use Soil Type 4 as presented in Section 3.6, Earth Loads, of the BDG to design earth loads from the soil envelope. The Soil Type properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

The concrete box culverts will be supplier-designed in accordance with LRFD specifications. The culverts should be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Article 3.4.1, and LRFD Section 12. The culverts will be constructed in general conformance with BDG Section 8, Buried Structures, and Special Provision 534, Precast Structural Concrete Arches, Box Culverts. The soil envelope bedding and backfill shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill, except that the maximum particle size shall be limited to 4 inches. We recommend a bedding layer 12 inches thick. Bedding and/or backfill should be placed in lifts 6 to 8 inches thick loose measure and compacted to manufacturer's specifications, but in no case shall the bedding and/or backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density. The leveling course below the box culvert shall consist of 12 inches of

7.2 Culvert Headwall Design

Culvert headwalls are essentially retaining walls and should 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 and 11.6. The headwalls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culverts. The wall shall also be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to 2.0 feet of soil.

Culvert headwall sections that are fixed to the box culverts to resist movement should be designed using an at-rest earth pressure coefficient, K_0, of 0.5. Headwall sections that are independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a, equal to 0.31. This assumes level backslope. The earth pressure coefficient may change if backslope conditions are different.

The bearing resistance for the wall footing shall be checked for the extreme and service limit state with a resistance factor of 1.0. The wall must be designed so that the factored bearing resistance, in conjunction with the depth of scour resulting from the design flood, provides adequate resistance to support the factored strength limit state loads. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor, φ, of 0.65. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated scour depth.

The designer shall apply a sliding resistance factor, φ_τ, of 0.80 to the nominal sliding resistance of cast-in-place concrete spread footings on soil. This resistance factor, φ_τ, may be increased to 0.90 for precast headwall footings on soil. For footings on soil, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth (1/4) of the footing dimensions, in either direction. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.45 (tan 24 degrees) at the foundation soil to footing concrete interface. The recommended sliding frictional coefficient is based on LRFD Table 3.11.5.3-1.

7.3 Bearing Resistance for Box Culverts and Spread Footings

The factored bearing resistance at the strength limit state for box culverts on compacted fill or native glacial till should not exceed 6.0 ksf. The factored bearing resistance for wall footings 2 to 4 feet wide should not exceed 3.5 ksf. For wall foundations on soil, the designer may assume the stress distribution to be a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used when analyzing box bottom slabs or wall footing for the service limit state and for preliminary footing sizing, as allowed in LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the footing concrete, which may be taken as $0.3f'c$. The minimum footing size is 2 feet wide regardless of the applied bearing pressure or bearing material.

7.4 Settlement

We have evaluated the potential settlement at the Bradley bridge site. MaineDOT currently does not plan horizontal or vertical alignment changes. Consequently, we estimate that settlement as a result of fill replacement and minor embankment fill extensions over natural soils will be negligible.

We have estimated that the total settlement of a prepared subgrade consisting of compacted fill or native glacial till will be on the order of ½-inch or less. If foundation footings are needed, we estimate that settlement of footings constructed on compacted fill or native glacial till will be on the order of ½-inch or less. In each case, this settlement is acceptable and will occur during construction. Post-construction settlement will be negligible.

7.5 Scour Protection

The box culverts will be fitted with concrete headwalls and inlet and outlet section seepage cutoff walls below the culvert, all to provide scour protection per BDG 8.3.1. We recommend that the bridge approach slopes be armored with a 3-foot thick layer of riprap up and down alignment beyond the headwall. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot thick layer of bedding material conforming to Standard Specification 703.19, Granular Borrow for Underwater Backfill and as shown in Standard Detail 610(02). Riprap shall meet the requirements of Section 703.26, Plain and Hand Laid Riprap of Special Provision 703, Aggregates. The riprap slope protection should be constructed no steeper than a maximum 1.75:1 (H:V) extending from the edge of roadway down to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation.

7.6 Frost Protection

We have evaluated the potential frost depth at the Bradley bridge site. Based on State of Maine frost depth maps, MaineDOT Bridge Design Guide (BDG) Figure 5-1, the site has a design-freezing index of approximately 1730 F-degree days. This correlates to a frost depth of 5.2 feet. We also considered Modberg frost depth projections. The results of the Modberg frost depth model indicate a potential frost depth of 4.1 feet. Consequently, we recommend that any spread footing or leveling pads constructed at the site be founded a minimum of 4.5 feet below finished exterior grade for frost protection. This minimum embedment applies only to foundations constructed on soil and not those founded on bedrock.

7.7 Seismic Design Considerations

In accordance with LRFD Article 12.6.1, Loading, earthquake loading should only be considered where buried structures cross active faults. Since there are no known active faults in Maine, no seismic analysis is required.

7.8 Construction Considerations

7.8.1 Excavation

Construction of the new twin concrete box culverts will require soil excavation. Earth support systems may be required. The fill and native glacial till soils at the site will be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. We recommend that the contractor protect any subgrade from exposure to water and any unnecessary construction traffic. If disturbance and rutting occur, we recommend that the contractor remove and replace the disturbed materials and replace with compacted gravel borrow. If the subgrade soil contains cobbles or boulders, we recommend that the contractor remove any cobbles and boulders larger than 6 inches in diameter. After excavating to the subgrade level, the contractor should proof-roll the surface to identify weak soil areas.

If encountered, unsuitable soils should also be excavated from the subgrade to a depth of one foot and replaced with compacted gravel borrow. Gravel borrow should conform to MaineDOT Standard Specification 703.20, Gravel Borrow. The gravel borrow should be compacted to 95 percent of the Modified Proctor maximum dry density (AASHTO T-180).

7.8.2 Dewatering

The existing fill and native glacial till soils within the project area are both poorly drained and moderately to highly frost susceptible. In some locations, these soil units may be saturated and significant water seepage may be encountered during excavation. The groundwater may be trapped in layers and lenses of coarse-grained soil overlying glacial till sediments. We anticipate that this seepage will be temporary but there may be localized sloughing and near-surface instability of some soil slopes.

The contractor should control groundwater and surface water infiltration to permit construction in-the-dry. We recommend that the contractor use temporary ditches, sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert groundwater if significant seepage is encountered during construction. We also recommend using French drains daylighted to nearby ditches if significant seepage is encountered in the subgrade along the construction areas. If the amount of seepage is significant, we anticipate that pumping from sumps will likely be needed to control the water.

7.8.3 Reuse of Excavated Soil and Bedrock

The project plans call for excavation of the existing approach areas to achieve planned grades. In the process, the contractor will excavate both the existing subbase gravel, and subgrade fill soils. We do not recommend using the excavated subbase aggregate to re-base the bridge approaches. Excavated subbase and subgrade sand and gravel may be used as fill below subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met.

We do not recommend using any glacial till soil excavation as fill beneath the pavement structure. This soil may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. Contractors should expect that, prior to placement and compaction, it may be necessary to spread out and dry portions of the glacial till soils that are excessively moist. This soil may also be used for dressing slopes, but only below the bottom elevation of the shoulder subbase gravel.

7.8.4 Embankment Fill Areas

The current project plans require construction of fill extensions along the bridge approaches. The plans indicate that the side slopes will constructed to 1.75:1 (H:V) grades and will be armored with riprap. We recommend benching the existing fill slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, where new fill slope extensions are constructed over existing slopes in preparation for construction of the riprap layer.

7.8.5 Erosion Control Recommendations

The fine-grained soils along the project are susceptible to erosion. We recommend using appropriate erosion control measures during construction as described in the MaineDOT Best Management Practices February 2008 guidelines to minimize erosion of the fine-grained soils at the site.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for specific application to the replacement of the Jenkins Bridge over Great Works Stream in Bradley, Maine. We have prepared the report in accordance with generally accepted soil and foundation engineering practices. No other intended use or warranty is expressed or 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 completed at discrete locations on the project site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

REFERENCES

AASHTO, (2007), LRFD Bridge Design Specifications, Fourth Edition, with 2009 Interims, AASHTO, Washington, D.C.

Bowles, Joseph E. (1996), Foundation Analysis and Design, Fifth Edition, McGraw-Hill, New York, NY.

MaineDOT, (2003), Bridge Design Guide, MaineDOT Bridge Program, Augusta, ME with various Interims.

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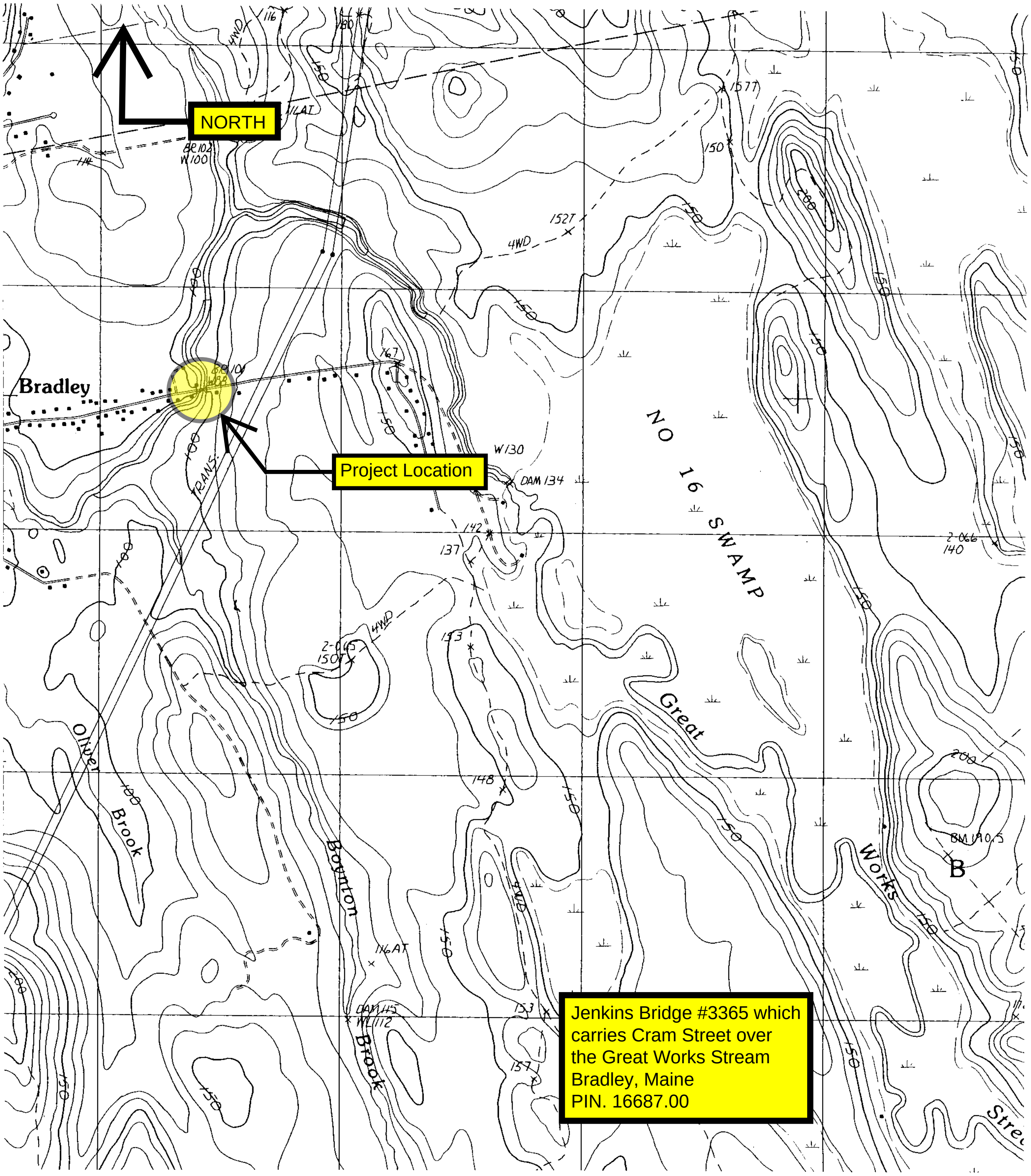
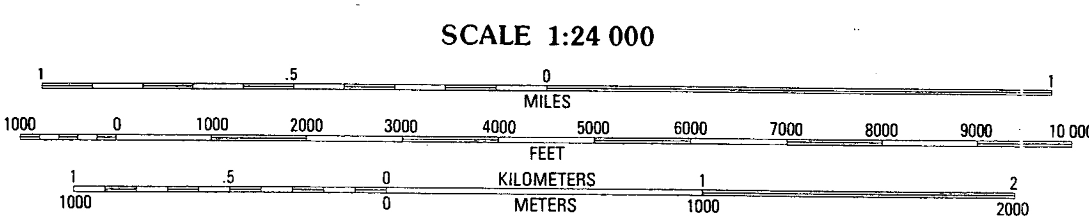


OTTER CHAIN PONDS QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)



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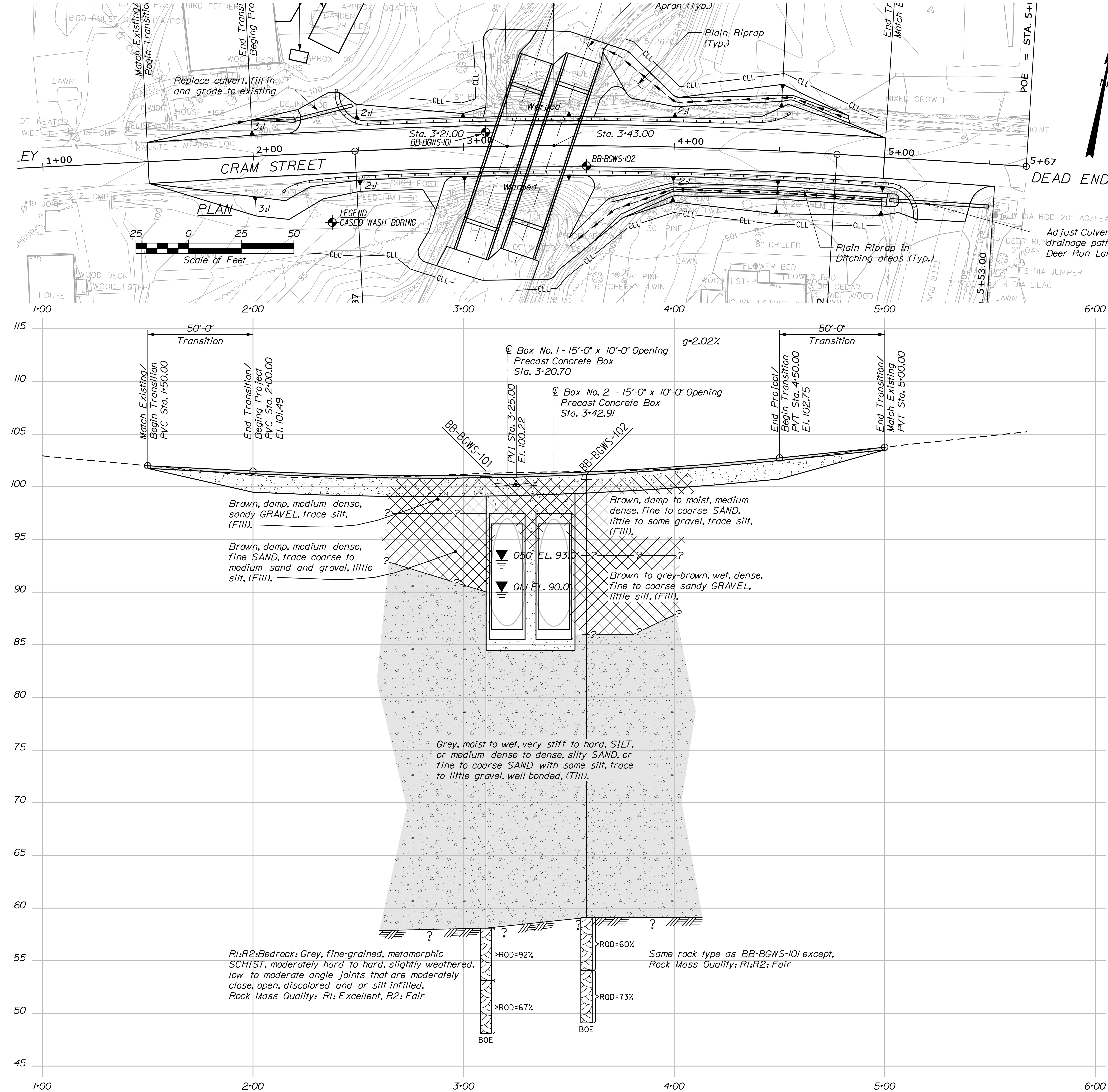

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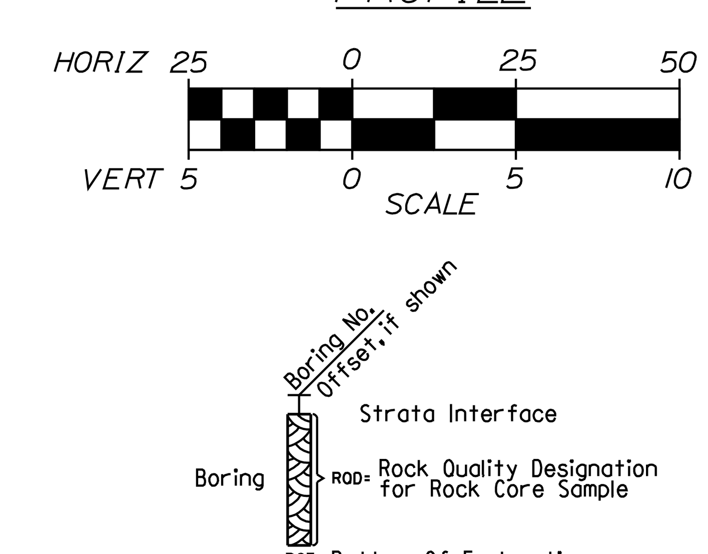

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1668(700)X

PIN 16687.00

BRIDGE PLANS

BRIDGE NO. 3365

PROJ. MANAGER	Steve Bodge	
DESIGN-DETAILED	M.MOREAU	
CHECKED-REVIEWED	T.WHITE	NOV 2009
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

JENKINS BRIDGE
GREAT WORKS STREAM
BRADLEY PENOBSCOT COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\msta\007_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:12/28/2009

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

Project: Jenkins Bridge #3365 which carries Cram Street over the
Location: Bradley, Maine

Boring No.: BB-BGWS-101
PIN: 16687.00

Driller:	MaineDOT	Elevation (ft.)	101.5	Auger ID/OD:	Hollow Stem
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Babcock (H&A)	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/5/09-8/5/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+10.6, 6.8 Lt.	Casing ID/OD:	HW	Water Level*:	11.5' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	101.00	ASPHALT. 0.50	
	1D	24/10	1.00 - 3.00	8/8/8/7	16	22			Brown, damp, medium dense, fine to coarse sandy GRAVEL, trace silt, (Fill).	G#246283 A-1-a, GW-GM WC=2.7%
5	2D	24/16	5.00 - 7.00	6/5/3/7	8	11			Brown, damp, medium dense, fine SAND, trace fine gravel, trace coarse to medium sand, little silt, (Fill).	G#246284 A-2-4, SP-SM WC=5.2%
10	3D	24/12	10.00 - 12.00	4/4/4/17	8	11	27			
							25	90.00	11.50 Brown to grey, moist to wet, medium dense, coarse to fine SAND, some silt, little coarse to fine gravel, occasional bonding in bottom 6" of recovery, (Glacial Till).	
							101			
							186			
							277		Pushed Cobble from 14.9-15.4' bgs.	
15	4D	24/16	15.40 - 17.40	9/23/20/29	43	60	129		Grey, moist, hard, SILT, some coarse to fine sand, little coarse to fine gravel, well bonded. (Till)	G#246285 A-4, ML WC=9.3%
							150			
							a165 bWA		a165=120(4")+45(8") bCasing Refusal at 17.4' bgs, washed ahead to 19.6' bgs.	
							80			
	5D	24/10	19.60 - 21.60	14/14/13/13	27	38	~~96~~ WASH		Grey, wet, dense, coarse to fine SAND, some silt, little coarse to fine gravel, well bonded. (Till)	
20							AHEAD			
25	6D	24/8	25.00 - 27.00	13/9/9/9	18	25			Grey, wet, very stiff, SILT, some coarse to fine sand, little coarse to fine gravel, well bonded. (Till)	G#246286 A-4, ML WC=10.2%
30	7D	24/12	30.00 - 32.00	11/13/8/9	21	29			Similar to above.	
35	8D	24/14	35.00 - 37.00	14/18/13/31	31	43			Grey, wet, dense, coarse to fine silty SAND, trace coarse to fine gravel, well bonded, occasional stratification. (Till)	G#246287 A-4, SM WC=10.8%
							cNQ		Cobble from 37.2-37.8' bgs. cAdvanced borehole from 37.2-42.7' bgs with NQ Barrel. No recovery, cored through 0.6' cobble and grey Glacial Till similar to 8D.	
40										
	9D	8.4/6	42.70 - 43.40	27/100(2.4")	---				Grey, wet, very dense, coarse to fine SAND, some silt, little coarse to fine gravel, well bonded, occasional stratification. (Till)	
	R1	60/60	43.40 - 48.40	RQD = 92%			NQ-2	58.10	43.40 Top of Bedrock at Elev. 58.1' R1:Bedrock: Grey, fine grained, metamorphic SCHIST, hard, slightly weathered, low angle joints, moderately close, open, joints are slightly weathered to decomposed, silt infilled, one silt seam 1 cm thick at 46.0', quartzose veins throughout. [Vassalboro Formation] Rock Quality Mass: Excellent R1:Core Times (min:sec) 43.4-44.4' (2:45) 44.4-45.4' (2:16) 45.4-46.4' (1:45) 46.4-47.4' (2:00) 47.4-48.4' (2:36) 100% Recovery	
45										
	R2	60/59	48.40 - 53.40	RQD = 67%				53.10	48.40 R2:Bedrock: Same rock type, except slightly to moderately weathered, moderately angled joints, moderately close, open, joints are slightly to highly weathered with silt infilling, one highly weathered zone at 52.1' to 52.7'. Rock Quality Mass: Fair R2:Core Times (min:sec) 48.4-49.4' (2:01) 49.4-50.4' (2:10) 50.4-51.4' (2:00) 51.4-52.4' (2:24) 52.4-53.4' (2:31) 98% Recovery	
50										
								48.10	53.40 Bottom of Exploration at 53.40 feet below ground surface.	
55										
60										
65										
70										
75										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-BGWS-101

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

Project: Jenkins Bridge #3365 which carries Cram Street over the
Location: Bradley, Maine

Boring No.: BB-BGWS-102
PIN: 16687.00

Driller:	MaineDOT	Elevation (ft.)	101.2	Auger ID/OD:	Hollow Stem
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Babcock (H&A)	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/4/09-8/4/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+58.4, 9.6 Rt.	Casing ID/OD:	HW	Water Level*:	6.8' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	100.70	ASPHALT. 0.50	
	1D	24/12	1.00 - 3.00	7/7/6/7	13	18			Brown, damp, medium dense, coarse to fine SAND, some gravel, trace silt, (Fill).	G#246288 A-1-b, SW-SM WC=4.0%
5	2D	24/6	5.00 - 7.00	3/12/5/4	17	24			Brown, damp to moist, medium dense, coarse to fine SAND, little gravel, trace silt, (Fill).	
10	3D	24/6	10.00 - 12.00	15/14/4/3	28	39	20		Brown to grey-brown, wet, dense, coarse to fine sandy GRAVEL, little silt, (Fill).	G#246289 A-1-b, SM WC=12.8%
							15			
							15			
							20			
							112		Cobble from 14.2-15.1' bgs.	
15								86.10	15.10	
	4D	24/18	15.40 - 17.40	5/6/8/15	14	20	104		Grey, moist, very stiff, SILT, some fine to coarse sand, trace gravel, well bonded, (Glacial Till).	G#246290 A-4, ML WC=10.7%
							163			
							160		Washed ahead to 20.0' bgs.	
							150			
							213			
20	5D	24/12	20.00 - 22.00	8/6/8/9	14	20	WASH		Grey, moist, medium dense, fine to coarse sandy SILT, little gravel, well bonded. (Till)	G#246291 A-4, SM WC=10.3%
							AHEAD			
25	6D	24/12	25.00 - 27.00	24/19/19/8	38	53			Grey, moist, hard, SILT, some coarse to fine sand, trace gravel, well bonded, occasionally stratified, (Till).	
30	7D	24/16	30.00 - 32.00	12/21/15/13	36	50			Grey, wet, hard, fine to coarse sandy SILT, trace coarse to fine gravel, well bonded, occasional stratification, (Glacial Till).	G#246292 A-4, ML WC=11.0%
35	8D	24/14	35.00 - 37.00	15/20/12/17	32	45			Grey, wet, hard, SILT, some fine sand, trace coarse to medium sand, trace coarse to fine gravel, well bonded, occasional stratification, (Till).	
40	9D	24/16	40.00 - 42.00	13/19/15/16	34	48			Similar to above.	G#246293 A-4, ML WC=15.9%
	R1	60/57	42.10 - 47.10	RQD = 60%			NQ-2	59.10	42.10 Top of Bedrock at Elev. 59.1' R1:Bedrock: Grey, fine-grained metamorphic SCHIST, moderately hard, slightly weathered, steep angle joints moderately close, open and discolored to silt infilled, one silt seam 1 cm thick at 42.4', low angle joints moderately close, open and discolored to slightly weathered, zones of probable lost recovery 42.1-42.6' and 46.0', noted drill action change probably associated with moderately to highly weathered zone. [Vassalboro Formation] Rock Mass Quality: Fair R1:Core Times (min:sec) 42.1-43.1' (4:05) 43.1-44.1' (3:09) 44.1-45.1' (2:30) 45.1-46.1' (2:30) 46.1-47.1' (2:00) 95% Recovery	
45										
	R2	60/58	47.10 - 52.10	RQD = 73%				54.10	47.10 R2:Bedrock: Same rock type, low angle joints moderately close, open, silt infilled. Rock Quality Mass: Fair R2:Core Times (min:sec) 47.1-48.1' (2:30) 48.1-49.1' (2:43) 49.1-50.1' (1:52) 50.1-51.1' (2:00) 51.1-52.1' (2:43) 97% Recovery	
50										
								49.10	52.10 Bottom of Exploration at 52.10 feet below ground surface.	
55										
60										
65										
70										
75										

Remarks:

Water depth may not represent actual depth.

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.

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Boring No.: BB-BGWS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1668(700)X

BRIDGE NO. 3365   PIN 16687.00   BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	Steve Bodge		
DESIGN-DETAILED	M.MOREAU	T.WHITE	NOV 2009
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	-----	-----	-----
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----		-----
REVISIONS 2	-----		-----
REVISIONS 3	-----		-----
REVISIONS 4	-----		-----
FIELD CHANGES	-----		-----

SIGNATURE

P.E. NUMBER

DATE

JENKINS BRIDGE
GREAT WORKS STREAM
BRADLEY   PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Jenkins Bridge #3365 which carries Cram Street over the Great Works Stream
Location: Bradley, Maine

Boring No.: BB-BGWS-101
PIN: 16687.00

Driller:	MaineDOT	**Elevation (ft.)**	101.5	**Auger ID/OD:**	Hollow Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Babcock (H&A)	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/5/09-8/5/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+10.6, 6.8 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.5' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	101.00		ASPHALT. 0.50	
	1D	24/10	1.00 - 3.00	8/8/8/7	16	22				Brown, damp, medium dense, fine to coarse sandy GRAVEL, trace silt, (Fill).	G#246283 A-1-a, GW-GM WC=2.7%
5	2D	24/16	5.00 - 7.00	6/5/3/7	8	11				Brown, damp, medium dense, fine SAND, trace fine gravel, trace coarse to medium sand, little silt, (Fill).	G#246284 A-2-4, SP-SM WC=5.2%
10	3D	24/12	10.00 - 12.00	4/4/4/17	8	11	27				
							25	90.00		11.50 Brown to grey, moist to wet, medium dense, coarse to fine SAND, some silt, little coarse to fine gravel, occasional bonding in bottom 6" of recovery, (Glacial Till).	
							101				
							186				
							277			Pushed Cobble from 14.9-15.4' bgs.	
15	4D	24/16	15.40 - 17.40	9/23/20/29	43	60	129			Grey, moist, hard, SILT, some coarse to fine sand, little coarse to fine gravel, well bonded. (Till)	G#246285 A-4, ML WC=9.3%
							150				
							a165 bWA			a165=120(4")+45(8") bCasing Refusal at 17.4' bgs, washed ahead to 19.6' bgs.	
							80				
	5D	24/10	19.60 - 21.60	14/14/13/13	27	38	96				
20							WASH AHEAD			Grey, wet, dense, coarse to fine SAND, some silt, little coarse to fine gravel, well bonded. (Till)	
25											

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Jenkins Bridge #3365 which carries Cram Street over the Great Works Stream **Location:** Bradley, Maine	**Boring No.:** BB-BGWS-101 **PIN:** 16687.00

Driller:	MaineDOT	**Elevation (ft.)**	101.5	**Auger ID/OD:**	Hollow Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Babcock (H&A)	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/5/09-8/5/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+10.6, 6.8 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.5' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/8	25.00 - 27.00	13/9/9/9	18	25				Grey, wet, very stiff, SILT, some coarse to fine sand, little coarse to fine gravel, well bonded. (Till)	G#246286 A-4, ML WC=10.2%
30	7D	24/12	30.00 - 32.00	11/13/8/9	21	29				Similar to above.	
35	8D	24/14	35.00 - 37.00	14/18/13/31	31	43				Grey, wet, dense, coarse to fine silty SAND, trace coarse to fine gravel, well bonded, occasional stratification. (Till)	G#246287 A-4, SM WC=10.8%
							cNQ			Cobble from 37.2-37.8' bgs. cAdvanced borehole from 37.2-42.7' bgs with NQ Barrel. No recovery, cored through 0.6' cobble and grey Glacial Till similar to 8D.	
40											
	9D	8.4/6	42.70 - 43.40	27/100(2.4")	---					Grey, wet, very dense, coarse to fine SAND, some silt, little coarse to fine gravel, well bonded, occasional stratification. (Till) 43.40	
	R1	60/60	43.40 - 48.40	RQD = 92%			NQ-2	58.10		Top of Bedrock at Elev. 58.1' R1:Bedrock: Grey, fine grained, metamorphic SCHIST, hard, slightly weathered, low angle joints, moderately close, open, joints are slightly weathered to decomposed, silt infilled, one silt seam 1 cm thick at 46.0', quartzose veins throughout. [Vassalboro Formation] Rock Quality Mass: Excellent R1:Core Times (min:sec) 43.4-44.4' (2:45) 44.4-45.4' (2:16) 45.4-46.4' (1:45) 46.4-47.4' (2:00) 47.4-48.4' (2:36) 100% Recovery	
45											
	R2	60/59	48.40 - 53.40	RQD = 67%				53.10		48.40	
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Jenkins Bridge #3365 which carries Cram Street over the Great Works Stream	**Boring No.:**	BB-BGWS-101		
Location:	Bradley, Maine	**PIN:**	16687.00		
Driller:	MaineDOT	**Elevation (ft.)**	101.5	**Auger ID/OD:**	Hollow Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Babcock (H&A)	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/5/09-8/5/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+10.6, 6.8 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.5' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										R2:Bedrock: Same rock type, except slightly to moderately weathered, moderately angled joints, moderately close, open, joints are slightly to highly weathered with silt infilling, one highly weathered zone at 52.1' to 52.7'. Rock Quality Mass: Fair R2:Core Times (min:sec) 48.4-49.4' (2:01) 49.4-50.4' (2:10) 50.4-51.4' (2:00) 51.4-52.4' (2:24) 52.4-53.4' (2:31) 98% Recovery	
								48.10			
										53.40	
										Bottom of Exploration at 53.40 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Jenkins Bridge #3365 which carries Cram Street over the Great Works Stream
Location: Bradley, Maine

Boring No.: BB-BGWS-102
PIN: 16687.00

Driller:	MaineDOT	**Elevation (ft.)**	101.2	**Auger ID/OD:**	Hollow Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Babcock (H&A)	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/4/09-8/4/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+58.4, 9.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	6.8' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	100.70		ASPHALT. 0.50	
	1D	24/12	1.00 - 3.00	7/7/6/7	13	18				Brown, damp, medium dense, coarse to fine SAND, some gravel, trace silt, (Fill).	G#246288 A-1-b, SW-SM WC=4.0%
5	2D	24/6	5.00 - 7.00	3/12/5/4	17	24				Brown, damp to moist, medium dense, coarse to fine SAND, little gravel, trace silt, (Fill).	
10	3D	24/6	10.00 - 12.00	15/14/4/3	28	39	20			Brown to grey-brown, wet, dense, coarse to fine sandy GRAVEL, little silt, (Fill).	G#246289 A-1-b, SM WC=12.8%
							15				
							15				
							20				
							112			Cobble from 14.2-15.1' bgs.	
15	4D	24/18	15.40 - 17.40	5/6/8/15	14	20	104	86.10		15.10 Grey, moist, very stiff, SILT, some fine to coarse sand, trace gravel, well bonded, (Glacial Till).	G#246290 A-4, ML WC=10.7%
							163				
							160			Washed ahead to 20.0' bgs.	
							150				
							213				
20	5D	24/12	20.00 - 22.00	8/6/8/9	14	20	WASH			Grey, moist, medium dense, fine to coarse sandy SILT, little gravel, well bonded. (Till)	G#246291 A-4, SM WC=10.3%
							AHEAD				
25											

Remarks:

Water depth may not represent actual depth.

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: Jenkins Bridge #3365 which carries Cram Street over the Great Works Stream
Location: Bradley, Maine

Boring No.: BB-BGWS-102
PIN: 16687.00

Driller:	MaineDOT	**Elevation (ft.)**	101.2	**Auger ID/OD:**	Hollow Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Babcock (H&A)	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/4/09-8/4/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+58.4, 9.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	6.8' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.00 - 27.00	24/19/19/8	38	53				Grey, moist, hard, SILT, some coarse to fine sand, trace gravel, well bonded, occasionally stratified, (Till).	
30	7D	24/16	30.00 - 32.00	12/21/15/13	36	50				Grey, wet, hard, fine to coarse sandy SILT, trace coarse to fine gravel, well bonded, occasional stratification, (Glacial Till).	G#246292 A-4, ML WC=11.0%
35	8D	24/14	35.00 - 37.00	15/20/12/17	32	45				Grey, wet, hard, SILT, some fine sand, trace coarse to medium sand, trace coarse to fine gravel, well bonded, occasional stratification, (Till).	
40	9D	24/16	40.00 - 42.00	13/19/15/16	34	48				Similar to above.	G#246293 A-4, ML WC=15.9%
	R1	60/57	42.10 - 47.10	RQD = 60%			NQ-2	59.10		42.10 Top of Bedrock at Elev. 59.1' R1:Bedrock: Grey, fine-grained metamorphic SCHIST, moderately hard, slightly weathered, steep angle joints moderately close, open and discolored to silt infilled, one silt seam 1 cm thick at 42.4', low angle joints moderately close, open and discolored to slightly weathered, zones of probable lost recovery 42.1-42.6' and 46.0', noted drill action change probably associated with moderately to highly weathered zone. [Vassalboro Formation] Rock Mass Quality: Fair R1:Core Times (min:sec) 42.1-43.1' (4:05) 43.1-44.1' (3:09) 44.1-45.1' (2:30) 45.1-46.1' (2:30) 46.1-47.1' (2:00) 95% Recovery	
45											
	R2	60/58	47.10 - 52.10	RQD = 73%				54.10		47.10	
50											

Remarks:

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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: Jenkins Bridge #3365 which carries Cram Street over the Great Works Stream Location: Bradley, Maine	Boring No.: BB-BGWS-102 PIN: 16687.00

Driller:	MaineDOT	Elevation (ft.)	101.2	Auger ID/OD:	Hollow Stem
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Babcock (H&A)	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/4/09-8/4/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+58.4, 9.6 Rt.	Casing ID/OD:	HW	Water Level*:	6.8' bgs.
Hammer Efficiency Factor:	0.84	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50								49.10		R2:Bedrock: Same rock type, low angle joints moderately close, open, silt infilled. Rock Quality Mass: Fair R2:Core Times (min:sec) 47.1-48.1' (2:30) 48.1-49.1' (2:43) 49.1-50.1' (1:52) 50.1-51.1' (2:00) 51.1-52.1' (2:43) 97% Recovery 52.10 **Bottom of Exploration at 52.10 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

Water depth may not represent actual depth.

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.

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	

Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Bradley Project Number: 16687.00

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-BGWS-101, 1D	3+10.6	6.8 Lt.	1.0-3.0	246283	1	2.7			GW-GM	A-1-a	0
BB-BGWS-101, 2D	3+10.6	6.8 Lt.	5.0-7.0	246284	1	5.2			SP-SM	A-2-4	0
BB-BGWS-101, 4D	3+10.6	6.8 Lt.	15.4-17.4	246285	1	9.3			ML	A-4	IV
BB-BGWS-101, 6D	3+10.6	6.8 Lt.	25.0-27.0	246286	1	10.2			ML	A-4	IV
BB-BGWS-101, 8D	3+10.6	6.8 Lt.	35.0-37.0	246287	1	10.8			SM	A-4	III
BB-BGWS-102, 1D	3+58.4	9.6 Rt.	1.0-3.0	246288	2	4.0			SW-SM	A-1-b	0
BB-BGWS-102, 3D	3+58.4	9.6 Rt.	10.0-12.0	246289	2	12.8			SM	A-1-b	II
BB-BGWS-102, 4D	3+58.4	9.6 Rt.	15.4-17.4	246290	2	10.7			ML	A-4	IV
BB-BGWS-102, 5D	3+58.4	9.6 Rt.	20.0-22.0	246291	2	10.3			SM	A-4	III
BB-BGWS-102, 7D	3+58.4	9.6 Rt.	30.0-32.0	246292	2	11.0			ML	A-4	IV
BB-BGWS-102, 9D	3+58.4	9.6 Rt.	40.0-42.0	246293	2	15.9			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 MaineDOT and Corps of Engineers Classification Systems.

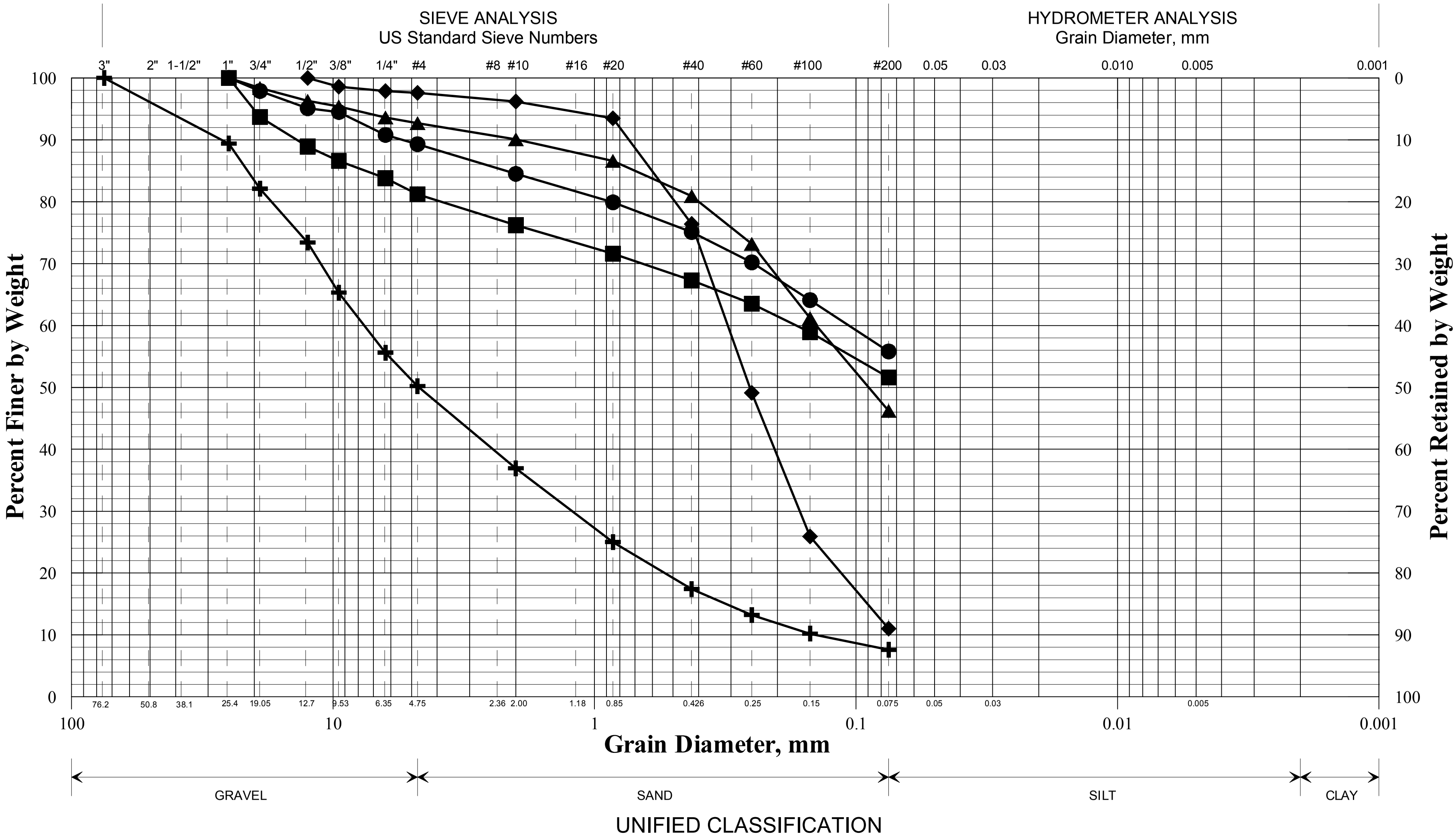
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

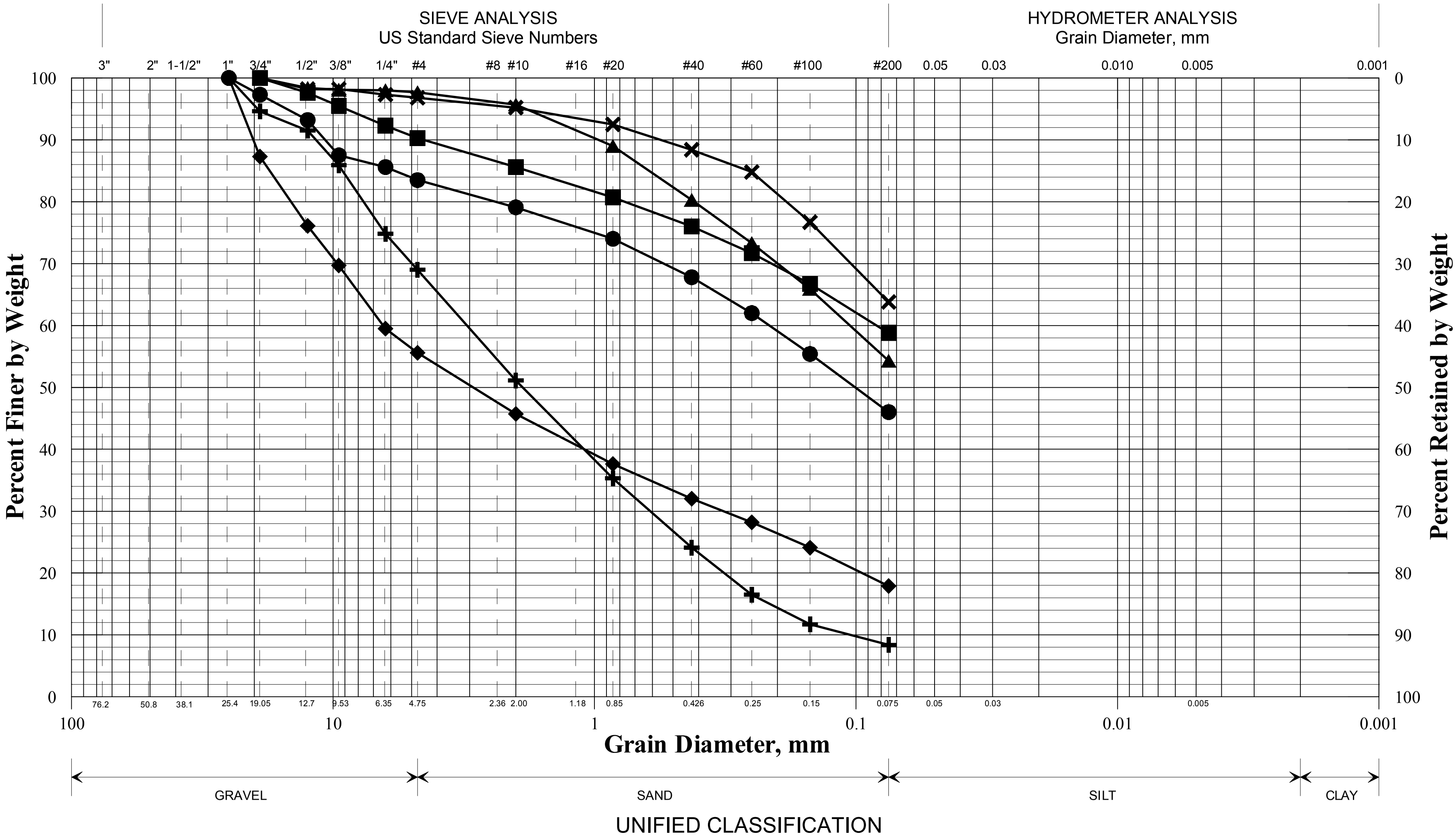


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BGWS-101/1D	3+10.6	6.8 LT	1.0-3.0	Sandy GRAVEL, trace silt.	2.7			
◆	BB-BGWS-101/2D	3+10.6	6.8 LT	5.0-7.0	SAND, little silt, trace gravel.	5.2			
■	BB-BGWS-101/4D	3+10.6	6.8 LT	15.4-17.4	SILT, some sand, little gravel.	9.3			
●	BB-BGWS-101/6D	3+10.6	6.8 LT	25.0-27.0	SILT, some sand, little gravel.	10.2			
▲	BB-BGWS-101/8D	3+10.6	6.8 LT	35.0-37.0	Silty SAND, trace gravel.	10.8			
✖									

PIN
016687.00
Town
Bradley
Reported by/Date
WHITE, TERRY A 9/11/2009

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BGWS-102/1D	3+58.4	9.6 RT	1.0-3.0	SAND, some gravel, trace silt.	4.0			
◆	BB-BGWS-102/3D	3+58.4	9.6 RT	10.0-12.0	Sandy GRAVEL, little silt.	12.8			
■	BB-BGWS-102/4D	3+58.4	9.6 RT	15.4-17.4	SILT, some sand, trace gravel.	10.7			
●	BB-BGWS-102/5D	3+58.4	9.6 RT	20.0-22.0	Sandy SILT, little gravel.	10.3			
▲	BB-BGWS-102/7D	3+58.4	9.6 RT	30.0-32.0	Sandy SILT, trace gravel.	11.0			
✖	BB-BGWS-102/9D	3+58.4	9.6 RT	40.0-42.0	SILT, some sand, trace gravel.	15.9			

PIN	
016687.00	
Town	
Bradley	
Reported by/Date	
WHITE, TERRY A	9/11/2009

SHEET 2

Appendix C

Calculations

HEADWALL ACTIVE EARTH PRESSURE:

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

Either Rankine or Coulomb may be used for long-heeled cantilever walls where the failure surface is uninterrupted by the top of the wall stem. In general, use Rankine though.

Soil angle of internal friction: $\phi := 32\text{deg}$

Slope angle of backfill soil from horizontal: $\beta := 0\text{deg}$

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

$K_a = 0.31$

FROST PROTECTION

Method 1:

From the Maine Design Freezing Index Map:
DFI = 1730 degree-days
Site has Fine Gained Native Soils With Wn = 10% to 15%

From the 2003 Bridge Design Guide Table 5-1:

$\text{Frost\_depth} := [0.3\cdot(64.0\text{in} - 62.2\text{in}) + 62.2\text{in}]$

$\text{Frost\_depth} = 62.74\cdot\text{in}$

$\text{Frost\_depth} = 5.23\cdot\text{ft}$

Method 2:

```
                    -----------------------
                    --- ModBerg Results ---
                    -----------------------

Project Location: Orono, Maine

Air Design Freezing Index        =  1588 F-days
N-Factor                         =  0.70
Surface Design Freezing Index    =  1112 F-days
Mean Annual Temperature          =  43.5 deg F
Design Length of Freezing Season =  132 days

---------------------------------------------------------
Layer
#:Type           t    w%     d    Cf  Cu   Kf   Ku     L
---------------------------------------------------------
1-Asphalt       4.0    .1 140.0   28  28    .9   .9      0
2-Fine         45.2 10.0 120.0   26  32   1.0   .9  1,728
---------------------------------------------------------

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

*********************************************************
  Total Depth of Frost Penetration = 4.10 ft = 49.2 in.
*********************************************************
```

Use 4.5 feet

BEARING RESISTANCE ON COMPACTED FILL SOILS:

Consider this for use with Box Culverts and Headwalls.

SERVICE LIMIT STATE:

LRFD Table C10.6.2.6.1-1, (Based on NAVFAC DM 7.2) - "Presumptive Bearing Resistances for Spread Footing Foundations at the Service Limit State"

Bearing Material	Consistency in Place	Bearing Resistance (kips per sq. foot)	Recommended Value
Coarse to Medium sand, little gravel	Very dense	8 to 12	8 ksf
	Medium dense to dense	4 to 8	6 ksf
	Loose	2 to 4	3 ksf

Recommend **6.0 ksf** to control settlements for **Service Limit State** analyses and for preliminary footing sizing.

STRENGTH LIMIT STATE:

Nominal and Factored Bearing Resistance for spread footings on fill soils at the Strength Limit State:

Assumptions:

1. Footings will be embedded 4.5 feet for frost protection.

$$D_f := 4.5ft$$

2. Assumed parameters for soils:
   Assume granular fill

   Moist unit weight: $\gamma_m := 125pcf$

   Saturated unit weight: $\gamma_{sat} := 130pcf$

   Soil angle of internal friction: $\phi_{ns} := 32$

   Undrained shear strength (cohesion): $c_{ns} := 0psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w := 0 \cdot ft$

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

Effective Stress at the footing bearing level: $q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.3 \cdot ksf$

Look at several footing widths:

$$B := \begin{pmatrix} 2 \\ 4 \\ 15 \end{pmatrix} ft$$

Terzaghi Shape Factors from Table 4-1, p. 220
For strip footing:

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 32 deg   Bowles 5th Ed. Table 4-4 pg. 223

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

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

$$q_{nom} := c_{ns} \cdot N_c \cdot s_c + q_{eff\_str} \cdot N_q + 0.5(\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{nom} = \begin{pmatrix} 8.5 \\ 10 \\ 18.2 \end{pmatrix} \cdot ksf$$

Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-32: $\phi_b := 0.45$

$$q_{fac} := q_{nom} \cdot \phi_b$$

$$q_{fac} = \begin{pmatrix} 3.8 \\ 4.5 \\ 8.2 \end{pmatrix} \cdot ksf$$

Recommend **Strength Limit State** Factored Bearing Resistance of **3.5 ksf** for footings 2 to 4 feet wide.

Nominal and Factored Bearing Resistance for box culvert on fill soils at the Strength Limit State:

Assumptions:

1. Box Culvert will be embedded 2.0 feet for frost protection.

$D_f := 2.0ft$

2. Assumed parameters for soils:
   Assume granular fill

Moist unit weight: $\gamma_m := 125pcf$

Saturated unit weight: $\gamma_{sat} := 130pcf$

Soil angle of internal friction: $\phi_{ns} := 32$

Undrained shear strength (cohesion): $c_{ns} := 0psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w := 0 \cdot ft$

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

Effective Stress at the footing bearing level: $q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.14 \cdot ksf$

Look at several footing widths:

$B := 15ft$

Terzaghi Shape Factors from Table 4-1, p. 220
For strip footing: $s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 32 deg   Bowles 5th Ed. Table 4-4 pg. 223

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

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

$$q_{nom} := c_{ns} \cdot N_c \cdot s_c + q_{eff\_str} \cdot N_q + 0.5 (\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{nom} = 14.3 \cdot ksf$$

Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-32: $\phi_b := 0.45$

$$q_{fac} := q_{nom} \cdot \phi_b$$

$$q_{fac} = 6.4 \cdot ksf$$

Recommend **Strength Limit State** Factored Bearing Resistance of **6.0 ksf** for the box culverts.

SETTLEMENT ANALYSIS:

Estimate Settlement for Headwall Footing On Soil Using Hough Method:
Ref. LRFD Section 10.6.2.4.2, pg. 10-49

Assumptions:
B = 2 ft
Assume 10 feet of fill at wall footing location
Soil thickness below footing is 20 feet
Use N1 of 30 (assumed corrected N_{60} value for very dense till or compacted fill)
I Influence factors from LRFD Figure 10.6.2.4.1-1, pg. 10-49
Bearing Capacity Indices (C') from LRFD Figure 10.6.2.4.2-1, pg. 10-52

$N1 := 30 \qquad C' := 105$

$$z := \begin{pmatrix} 1 \\ 4 \\ 12 \end{pmatrix} \qquad I := \begin{pmatrix} 0.85 \\ 0.3 \\ 0.1 \end{pmatrix}$$

$\sigma_o := (135pcf - 62.4pcf) \cdot 4.5ft$

$\Delta\sigma_v := 10ft \cdot 125pcf \cdot I$

$$H := \begin{pmatrix} 2 \\ 4 \\ 12 \end{pmatrix} \cdot ft \qquad \Delta\sigma_v = \begin{pmatrix} 1.06 \\ 0.38 \\ 0.13 \end{pmatrix} \cdot ksf$$

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_v}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix} 0.14368 \\ 0.15177 \\ 0.19296 \end{pmatrix} \cdot in$$

$\Delta H_{TOTAL} := 0.14 \cdot in + 0.15 \cdot in + 0.19 \cdot in$

$\Delta H_{TOTAL} = 0.48 \cdot in$

OK, Say 1/2 inch or less settlement below footing on soil.

<u>Appendix D</u>

Special Provision

SPECIAL PROVISION
SECTION 534
PRECAST STRUCTURAL CONCRETE
(Precast Structural Concrete Arches, Box Culverts)

534.10 Description The Contractor shall design, manufacture, furnish, and install elements, precast structural concrete structures, arches, or box culverts and associated wings, headwalls, and appurtenances, in accordance with the contract documents.

534.20 Materials Structural precast elements for the arch or box culvert and associated precast elements shall meet the requirements of the following Subsection:

Structural Precast Concrete Units	712.061

Grout, concrete patching material, and geotextiles shall be one of the products listed on the Department's list of prequalified materials, unless otherwise approved by the Department.

Box culvert bedding and backfill material shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill, with the additional requirement that the maximum particle size be limited to 4 inches.

534.30 Design Requirements The Contractor shall design the precast structural concrete structure in accordance with the AASHTO Standard Specifications for Highway Bridges, current edition. The design live load shall be as follows: *modified HL-93 Strength I for LRFD method. *(modify HL-93 by increasing all wheel loads by a factor of 1.25)

The Contractor shall submit design calculations and shop drawings for the precast structure to the Department for approval. A Registered Professional Engineer, licensed in accordance with State of Maine laws, shall sign and seal all design calculations and drawings. The Contractor shall submit a bridge rating on the Department's Standard Bridge Rating Summary Sheet with the design calculations. Drawings shall conform with Section 105.7 - Working Drawings.

The Contractor shall submit the following items for review by the Resident at least ten working days prior to production:

A) The name and location of the manufacturer.
B) Method of manufacture and material certificates.
C) Description of method of handling, storing, transporting, and erecting the members.
D) Shop Drawings with the following minimum details:

   1) Fully dimensioned views showing the geometry of the members, including all projections, recesses, notches, openings, block outs, and keyways.
   2) Details and bending schedules of reinforcing steel including the size, spacing, and location. Reinforcing provided under lifting devices shall be shown in detail.
   3) Details and locations of all items to be embedded.

4) Total mass (weight) of each member.

534.40 Construction Requirements The applicable provisions of Subsection 535.10 - Forms and Casting Beds and Subsection 535.20 – Finishing Concrete and Repairing Defects shall be met.

Manufacture of Precast Units The internal dimensions shall not vary by more than 1 percent from the design dimensions or 38 mm [1 ½ in], whichever is less. The haunch dimensions shall not vary by more than 19 mm [¾ in] from the design dimension. The dimension of the legs shall not vary by more than 6 mm [¼ in] from the dimension shown on the approved shop drawings.

The slab and wall thickness shall not be less than the design thickness by more than 6 mm [¼ in]. A thickness greater than the design thickness shall not be cause for rejection.

Variations in laying lengths of two opposite surfaces shall not be more than 15 mm [⅝ in] in any section, except where beveled ends for laying of curves are specified.

The under-run in length of any section shall not be more than 12 mm [½ in].

The cover of concrete over the outside circumferential reinforcement shall be 50 mm [2 in] minimum. The concrete cover over the inside reinforcement shall be 38 mm [1 ½ in] minimum. The clear distance of the end of circumferential wires shall not be less than 25 mm [1 in] or more than 50 mm [2 in] from the end of the sections. Reinforcement shall be single or multiple layers of welded wire fabric or a single layer of deformed billet steel bars.

Welded wire fabric shall meet the space requirements and contain sufficient longitudinal wires extending through the section to maintain the shape and position of the reinforcement. Longitudinal distribution reinforcement may be welded wire fabric or deformed billet steel bars which meet the spacing requirements. The ends of the longitudinal distribution reinforcement shall be not more than 75 mm [3 in] from the ends of the sections.

The inside circumferential reinforcing steel for the haunch radii or fillet shall be bent to match the radii or fillets of the forms.

Tension splices in the reinforcement will not be permitted. For splices other than tension splices, the overlap shall be a minimum of 300 mm [12 in] for welded wire fabric or billet steel bars. The spacing center to center of the circumferential wires in a wire fabric sheet shall be not less than 50 mm [2 in] or more than 100 mm [4 in]. For the wire fabric, the spacing center to center of the longitudinal wires shall not be more than 200 mm [8 in]. The spacing center to center of the longitudinal distribution steel for either line of reinforcing in the top slab shall be not more than 375 mm [15 in].

The members shall be free of fractures. The ends of the members shall be normal to the walls and centerline of the section, within the limits of variation provided, except where beveled ends are specified. The surfaces of the members shall be a smooth steel form or troweled surface finish, unless a form liner is specified. The ends and interior of the assembled structure shall make a continuous line of members with a smooth interior surface.

Defects which may cause rejection of precast units include the following:

1) Any discontinuity (crack or rock pocket etc.) of the concrete which could allow moisture to reach the reinforcing steel.
2) Rock pockets or honeycomb over 4000 mm² [6 in²] in area or over 25 mm [1 in] deep.
3) Edge or corner breakage exceeding 300 mm [12 in] in length or 25 mm [1 in] in depth.
4) Extensive fine hair cracks or checks.
5) Any other defect that clearly and substantially impacts the quality, durability, or maintainability of the structure as measured by accepted industry standards.

The Contractor shall store and transport members in a manner to prevent cracking or damage. The Contractor shall not place precast members in an upright position until a compressive strength of at least 30 MPa [4350 psi] is attained.

<u>Installation of Precast Units</u> The Contractor shall not ship precast members until sufficient strength has been attained to withstand shipping, handling and erection stresses without cracking, deformation, or spalling (but in no case less than 30 MPa [4350 psi].

The Contractor shall set precast members on 12 mm [½ in] neoprene pads during shipment to prevent damage to the section legs. The Contractor shall repair any damage to precast members resulting from shipping or handling by saw cutting a minimum of 12 mm [½ in] deep around the perimeter of the damaged area and placing a polymer-modified cementitious patching material.

When footings are required, the Contractor shall install the precast members on concrete footings that have reached a compressive strength of at least 20 MPa [2900 psi]. The Contractor shall construct the completed footing surface to the lines and grades shown on the plans. When checked with a 3 m [10 ft] straightedge, the surface shall not vary more than 6 mm [¼ in] in 3 meters [10 ft]. The footing keyway shall be filled with a non-shrink flowable cementitious grout with a design compressive strength of at least 35 MPa [5075 psi].

The Contractor shall fill holes that were cast in the units for handling, with either Portland cement mortar, or with precast plugs secured with Portland cement mortar or other approved adhesive. The Contractor shall completely fill the exterior face of joints between precast members with an approved material and cover with a minimum 300 mm [12 in] wide joint wrap. The surface shall be free of dirt and deleterious materials before applying the filler material and joint wrap. The Contractor shall install the external wrap in one continuous piece over each member joint, taking care to keep the joint wrap in place during backfilling. The Contractor shall seal the joints between the end unit and attached elements with a non-woven geotextile. The Contractor shall install and tighten the bolts fastening the connection plate(s) between the elements that are designed to be fastened together as designated by the manufacturer. Final assembly shall be approved by the manufacturer's representative prior to backfilling.

The Contractor shall place and compact the bedding material as shown on the plans prior to lifting and setting the box culvert sections. The Contractor shall backfill the structure in accordance with the manufacturer's instructions and the Contract Documents. The Contractor shall uniformly distribute backfill material in layers of not more than 200 mm [8 in] depth, loose measure, and thoroughly compact each layer using approved compactors before successive layers are placed. The Contractor shall compact the Granular Borrow bedding and backfill in accordance with Section 203.12 - Construction of Earth Embankment with Moisture and Density Control, except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180, Method C or D. The Contractor shall place and compact backfill without disturbance or displacement of the wall units, keeping the fill at approximately the same elevation on both sides of the structure. Whenever a compaction test fails, the Contractor shall not place additional backfill over the area until the lift is re-compacted and a passing test achieved.

The Contractor shall use hand-operated compactors within 1.5 m [5 ft] of the precast structure as well as over the top until it is covered with at least 300 mm [12 in] of backfill. Equipment in excess of 11 Mg [12 ton] shall not use the structure until a minimum of 600 mm [24 in] of backfill cover is in place and compacted.

<u>534.50 Method of Measurement</u> The Department will measure Precast Structural Concrete Arch or Box Culvert for payment per Lump Sum each, complete in place and accepted.

<u>534.60 Basis of Payment</u> The Department will pay for the accepted quantity of Precast Structural Concrete Arch or Box Culvert at the Contract Lump Sum price, such payment being full compensation for all labor, equipment, materials, professional services, and incidentals for furnishing and installing the precast concrete elements and accessories. Falsework, reinforcing steel, jointing tape, grout, cast-in-place concrete fill or grout fill for anchorage of precast wings and/or other appurtenances is incidental to the Lump Sum pay item. Cast-in-place concrete, reinforcing steel in cast-in-place elements, excavation, backfill material, and membrane waterproofing will be measured and paid for separately under the provided Contract pay items. Pay adjustments for quality level will not be made for precast concrete.

Payment will be made under:

Pay Item	Pay Unit
534.71 Precast Concrete Box Culvert	Lump Sum

MaineDOT

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Soils Report 2009-34 Addendum #1

To:	Pam Heatherly, SEA Consultants
Cc:	Nate Benoit, Project Manager
Author:	Laura Krusinski
Subject:	Replacement of Jenkins Bridge Addendum to Soils Report 2009-34
Doc Type:	24
Date:	March 24, 2010
Bridge #:	3365
Route:	Cram Street
PIN:	16687.00
Town:	Bradley

1.0 BACKGROUND

The MaineDOT Bridge Program originally planned to replace Jenkins Bridge in Bradley, Maine, which consists of twin 17-foot structural plate pipe arch culverts, with twin concrete boxes. A geotechnical report summarizing the subsurface investigation conducted at the site and geotechnical design recommendations for the precast concrete box alternative was published and is available as Soils Report 2009-34, dated 23 December 2010. See Soil Report 2009-34 for specific information regarding the subsurface investigation, laboratory testing and detailed subsurface conditions at the site.

Subsequently, the Jenkins Bridge site has been selected as a location to install a 32-foot span, composite tubular arch bridge structure developed by the University of Maine's Advanced Engineering Wood & Composites Center (AEWC) in Orono, Maine.

AEWC's tubular arches are made of Fiber Reinforced Polymer (FRP) composite materials. The carbon fiber tubes are inflated off-site and infused with resin. After hardening, the tubes are transported to the bridge site, lowered into place and filled with concrete. The tubular arches are covered with a corrugated, FRP composite deck material and backfill is placed over the structure.

The purpose of this Addendum #1 is to provide geotechnical design recommendations for a tubular, composite arch bridge structure supported on cast-in-place concrete pile caps on driven H-piles.

The design of the FRP tubular arches and associated headwalls is the responsibility of the AEWC and will be supplied to the designer and Contractor prior to construction of the structure.

2.0 FOUNDATION ALTERNATIVES

The following foundation alternatives were considered for proposed arch bridge:

- spread footings founded on seals cast on native soil,
- reinforced concrete arch stem walls/pile caps supported on H-piles or pipe piles driven to bedrock

Due to the depth of overburden at the site, the use of driven piles to support the arches is recommended. For the purposes of this addendum it is assumed that driven H-piles will be used to support the arch bridge structure. If during final design, it is determined that the use of pipe piles is necessary, pipe pile resistances will be developed and provided to the designer.

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

This section provides geotechnical design recommendations for H-pile supported arch stem walls/pile caps.

3.1 Site Conditions

The bridge site was investigated by drilling two test borings in August of 2009. The boring locations and generalized soil profile are shown on the Boring Location Plan and Interpretive Subsurface Profile included at the end of the addendum.

Bedrock was encountered and cored at approximate depths of 43.4 and 42.1 feet below ground surface (bgs) at borings BB-BGWS-101 and BB-BGWS-102, respectively. Boring logs are included in Soils Report 2009-34. The bedrock was identified as grey, fine-grained schist that is moderately hard, slightly weathered and slightly fractured. The RQD of the bedrock was determined to range from 60 to 92 percent, correlating to a rock mass quality of fair to excellent.

The table below summarizes approximate top of bedrock elevations at the exploration locations.

Proposed Substructure	Boring	Station	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
Pile Cap No. 1	BB-BGWS-101	3+10.6	43.4	58.1
Pile Cap No. 2	BB-BGWS-102	3+58.4	42.1	59.1

Approximate Elevation of Bedrock Surface at Exploration Locations

Site conditions are presented in detail in Soils Report 2009-34.

3.2 Driven H-Pile Design

H-piles should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 14x73, 14x89, or 14x117 depending on the factored design axial and lateral loads. Piles should be 50 ksi, Grade A572 steel. The piles should be oriented for strong axis bending. Piles should be fitted with driving pile points to protect the tips and improve penetration. Piles may be plumb, battered or a combination of both.

Pile lengths at the proposed arch stem wall/pile caps, considering a nominal 18 inch pile embedment in the pile cap, will range from approximately 25 to 26 feet. This data is summarized in the table below:

Proposed Structure	Approximate Bedrock Elevation (feet)	Estimated Arch Stem Wall/Pile Cap Bottom Elevation (feet)	Estimated Pile Embedment in Abutment (feet)	Estimated Pile Lengths after cut-off (feet)
Pile Cap No. 1	58.1	82.0	2.0	26
Pile Cap No. 2	59.1	82.0	2.0	25

Estimated Pile Lengths for Plumb Piles

The center-to-center pile spacing should not be less than 30 inches or 2.5 to 3 times the pile diameter. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9 inches. The tops of the piles should project at least 18 inches into the pile cap.

H-piles shall be designed at the strength limit states considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 3.2.1 below.

The design of H-piles at the service limit state shall consider tolerable horizontal movement of the piles, and overall stability of the pile group and displacements considering changes in foundation conditions due to scour at the design flood event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

Since the H-piles will be subjected to lateral loading, piles should be analyzed for combined axial and flexure as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2

3.2.1 Strength Limit State Design

For preliminary analyses, the H-piles were assumed fully embedded, and the column slenderness factor, λ was taken as 0. The factored structural axial compressive resistances of the four proposed H-pile sections presented in this report were calculated using a resistance factor, Φ_c, of 0.60 and a λ of 0. It is the responsibility of the Structural Designer to recalculate λ for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and recalculate structural resistances accordingly.

For the portion of the pile which is theoretically in pure compression, i.e. below the point of fixity, the factored structural axial resistances of four H-pile sections were calculated using a resistance factor, Φ_c, of 0.60. The factored structural axial resistance may be controlled by the combined axial and flexural resistance of the pile. This analysis is the responsibility of the Structural Designer.

The nominal and factored axial geotechnical resistance in the strength limit state was calculated using the Canadian Geotechnical Society method for end bearing on bedrock, Tomlinson for side friction in cohesive soils, and Nordlund for side friction in cohesionless soils. A resistance factor, φ_{stat}, of 0.45, was applied assuming nominal pile resistances are verified during construction with dynamic pile tests with signal matching. The calculated factored geotechnical resistances of four (4) H-pile sections are provided in the table, below.

Drivability analyses of the four (4) proposed H-pile sections were conducted. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be no more that 45 ksi. The resistance factor for a single pile in axial compression when a dynamic test is performed given in LRFD Table 10.5.5.2.3-1 is φ_{dyn} = 0.65. LRFD 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. When a pile group is nonredundant, i.e., there are less than five (5) piles, LRFD Article 10.5.5.2.3 dictates a 20 percent reduction of the resistance factor value of 0.65. The factored pile resistances provided in this report assume a minimum five-pile group, and therefore are factored by resistance factor, φ_{dyn}, of 0.65.

For the strength limit state, the calculated factored axial compressive structural, geotechnical and drivability resistances of four (4) proposed H-piles sections are summarized in the table below. Supporting calculations can be found at the end of this addendum.

	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance Φ_c=0.60 λ= 0	Geotechnical Resistance φ_{stat} = 0.45	Drivability Resistance φ_{dyn} = 0.65	Governing Pile Resistance
HP 12 x 53	465	107	273	273
HP 14 x 73	642	141	364	364
HP 14 x 89	783	167	429	429
HP 14 x 117	1032	212	435	435

Factored Axial Compressive Resistances for H-Pile Sections for Strength Limit State Design

LRFD Article 10.7.3.2.3 states that the nominal compressive resistance of piles driven to hard rock is controlled by the structural limit state. However, the calculated factored axial drivability resistance is less than the calculated factored axial structural resistance, and local experience supports the estimated factored resistance from the drivability analyses. Therefore, the recommended governing resistance for pile design should be the factored drivability resistance provided in the table above.

Since the abutment piles will be modeled with a fixed pile head in the arch pile cap and subjected to lateral and axial loads, bending moments and displacement, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15. An L-Pile® analysis is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, moments and pile head displacements applied. The resistance for the piles should be determined for compliance with the interaction equation. The upper portion of the pile is defined per LRFD Figure C6.15.2-1 as that portion of the pile above the point of second inflection in the moment vs. pile depth curve, or at the lowest point of zero deflection. For strength limit state load combinations, resistance factors of 0.70 for axial resistance (Φ_c) and 1.0 for flexural resistance (Φ_f) should be applied to the combined axial and flexural resistance of the pile in the interaction equation. The resistance of the pile in the lower zone need only be checked against axial load, but only if the piles are fully fixed.

3.2.2 Service and Extreme Limit State Design

The design of piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and the consequences of changed foundation conditions resulting from scour at the design flow event. For the service limit states, a resistance factor of 1.0 should be used for the calculation of structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.2. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65.

The extreme limit state design shall include a determination that there is adequate nominal foundation resistance remaining after scour due to the check flood to resist the unfactored extreme limit state load combination with a resistance factor of 1.0.

The calculated factored axial structural, geotechnical and drivability resistances of four (4) H-pile sections were calculated for the service and extreme limit states and are provided in the table below. Supporting documentation is provided at the end of this addendum.

	Service and Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance, assuming λ= 0	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
HP 12 x 53	775	239	420	420
HP 14 x 73	1070	313	560	560
HP 14 x 89	1305	371	660	660
HP 14 x 117	1720	471	669	669

Factored Axial Pile Resistance for H-Piles Sections for Service and Extreme Limit State Design

LRFD Article 10.7.3.2.3 states that the nominal compressive resistance of piles driven to hard rock is controlled by the structural limit state. However, the calculated factored axial drivability resistance is less than the calculated factored axial structural resistance, and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the governing resistance used in design be the factored drivability resistance in the table above.

3.2.3 Lateral Pile Resistance

Lateral loads may be reacted by plumb or battered piles. We recommend the designer perform a series of lateral pile resistance analyses to evaluate pile top deflections and bending stresses under strength limit state design lateral loads using L-Pile® software or FB-Pier software. Similar software for analyzing pile response under lateral loads where the nonlinear soil behavior is modeled using soil resistance (p-y) curves may be used. These analyses should take into consideration pile batter, if any. There is not a performance criteria at this time for allowable lateral displacements at the pile head, therefore, the designer should consider performing lateral pile analyses to determine maximum factored lateral loads permissible based on the allowable displacement criteria. Furthermore, the designer should evaluate the associated pile stresses under factored lateral loads.

Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in the tables below. In general, the model developed should emulate the soil at the site by using the soil layers (referenced in the tables below by elevations) and appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed. It is recommended that the analyses be conducted assuming a fixed pile-head boundary condition.

Soil Layer	Approximate Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)
Sand and gravel (Fill)	87 – 100	Above	0.0694 (120)
Silt and sand (Glacial Till)	58 – 87	Below	0.0336 (58)

Soil Layer	k_s (lb/in^3)	Cohesion (lb/in^2)	E_{50} for clays	Friction Angle
Sand and Gravel (Fill)	90	-	-	32°
Silt and sand (Glacial Till)	500	3000	-	36°

Soil Parameters for Generation of Soil-Resistance (p-y) Curves

3.2.4 Driven Pile Resistance and Pile Quality Control

Contract documents should require the contractor to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each substructure. 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. Restrikes will be not be required as part of the pile field quality control program.

With this level of quality control, the ultimate 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, φ_{dyn}, of 0.65 provided that a minimum of three piles out of the total number of piles driven at the project site are dynamically tested, in accordance with LRFD Tables 10.5.5.2.3-1 and -3. LRFD Article 10.5.5.2.3 further specifies that the resistance factor, φ_{dyn}, of 0.65 be reduced by 20 percent when applied to nonredundant pile groups, i.e. pile groups with less than five (5) piles. Although a resistance factor, φ_{dyn}, of 0.65 cannot be justified where only two dynamic pile load tests are planned, a pile resistance factor of 0.65 is used in the pile analyses because past practice has been to perform one dynamic pile test at each abutment at conventional, single span integral bridges.

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 $0.90\varphi_{da}\ F_y$, where φ_{da} is equal to 1.0, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 5 to 15 blows per inch (bpi) based on MaineDOT criteria or less than 10 bpi based on FHWA criteria. 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.

3.3 Arch Footing/Pile Cap Design

Arch stem walls/pile caps shall be designed for all relevant strength, service and extreme limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications 4th Edition, 2007, with 2008 and 2009 interims (LRFD) Articles 3.4.1, 11.5.5., and 12.5. Arch pile caps shall be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and lateral thrust forces transferred through the bridge arches. The design of arch pile caps at the strength limit state shall consider pile stability and reinforced-concrete structural design.

A resistance factor of 1.0 shall be used to assess arch pile cap design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65.

Extreme limit state design of the pile cap supported on H-piles shall include pile structural resistance, pile geotechnical resistance and pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for arch wall and pile cap backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, γ = 125 pcf.

Calculation of passive earth pressures for resisting lateral thrust forces from the arch should assume a Rankine passive earth pressure coefficient, K_p, of 3.25, anticipating the arch pile caps experience small movements. Should the ratio of lateral pile cap movement to the pile cap stem wall height (y/H) exceed 0.005, then the calculation of passive earth pressure should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. Use a resistance factor for passive rest earth pressures (φ_{ep}) mobilized to resist lateral sliding forces, of 0.50 per LRFD Table 10.5.5.2.2-1. For designing the arch pile cap reinforcing steel to resist passive earth pressures, use a maximum load factor (γ_{EH}) of 1.50.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. The live load surcharge on arch stem walls/pile caps may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Arch Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
>=20	2.0

Equivalent Height of Soil for Estimating Live Load Surcharge on Arch Footings

Arch foundations shall include a drainage system behind the arch stem wall/pile cap to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of the arches, arch footing 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.

3.4 Scour and Riprap

Grain size analyses were performed on three (3) soil samples taken from the upper glacial till deposit encountered in BB-BGWS-101 and BB-BGWS-102, for the purpose of generating grain size curves for determining parameters to be used in scour analyses. The samples were assumed to be similar in nature to the silty glacial till soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50% passing, D_{50} = 0.07 mm
- Average diameter of particle at 95% passing, D_{95} = 14 mm
- Soil Classification: AASHTO Soil Type: A-4

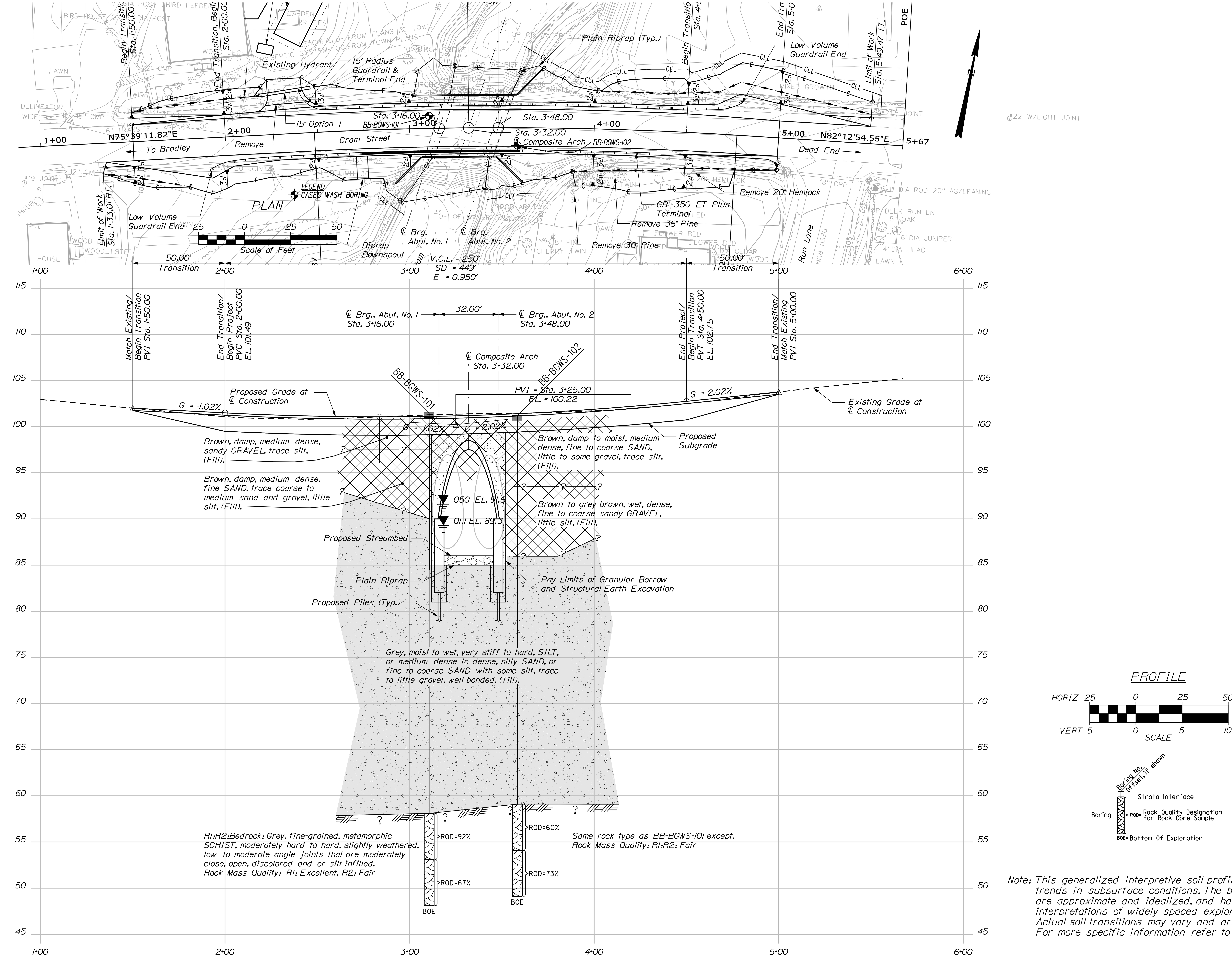
The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour with respect to factored strength limit state loads. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design flood.

Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of arch footings and wingwalls. Stone riprap shall conform to item number 703.26 of Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class 1 nonwoven erosion control geotextile per Standard Details 610(02) through 610(04). Riprap shall be 3 feet thick.

Attachments:
Boring Location Plan and Interpretive Subsurface Profile
Calculations – Driven H-Pile Resistances

Prepared by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1668(700)X

PIN 16687.00

BRIDGE PLANS

BRIDGE NO. 3365

	BY	DATE
PROJ. MANAGER	Steve Bodge	
DESIGN-DETAILED	M.MOREAU	T.WHITE	NOV 2009
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

JENKINS BRIDGE
GREAT WORKS STREAM
BRADLEY PENOBSCOT COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Bedrock Properties at the Site

RQD from bedrock cores
92 to 67% in BB-BGWS-101
60 to 73% in BB-BGWS-102

Rock Type: Schist - Fair to excellent rock mass quality

ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1);

uniaxial compressive strength = Co= 1400 to 21,000 psi - use **15,000 psi** for design (AASHTO TABLE 4.4.8.1.2 B

Pile Properties

Use the following piles: 12x53, 14x73, 14x89, 14x117

$$A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.89 \\ 198.356 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$$

Nominal and Factored Structural Compressive Resistance of HP piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock
Use LRFD Equation 6.9.2.1-1

Normalized column slenderness factor, λ, in equation 6.9.4.1-1 is assumed to be zero since the unbraced length is zero.

$$F_y := 50 \cdot ksi$$

$$\lambda := 0$$

Nominal Axial Structural Resistance

From LRFD 6.9.4.1-1

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

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

Factored Axial Structural Resistance of single H pile

Resistance factor or H-pile in compression, no damage anticipated, LRFD 6.5.4.2

$\phi_c := 0.6$

Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 $P_r := \phi_c \cdot P_n$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 465 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot kip$$

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Table 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example www.fhwa.gov/bridge/lrfd/fhwanhi04041_steel.pdf

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

Schist $q_{u\_1} := 15000 \cdot psi$

Spacing of discontinuities $s_d := 6 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $t_d := \frac{1}{64} \cdot in$

Pile width is b - matrix $D := b$

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 0 \cdot ft$

Diameter of socket: $D_s := 12 \cdot in$

Depth factor $dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$ and dd < 3

$dd = 1$ OK

Ksp

$$K_{sp} := \frac{3 + \frac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \frac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.262 \\ 0.256 \\ 0.255 \\ 0.255 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd \qquad q_{p\_1} = \begin{pmatrix} 1698 \\ 1656 \\ 1655 \\ 1652 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, Rp - *Extreme Limit States and Service Limit States*

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)} \qquad R_{p\_1} = \begin{pmatrix} 183 \\ 246 \\ 300 \\ 395 \end{pmatrix} \cdot kip$$

Factored axial geotechnical tip compressive resistance - *Strength Limit States*

Resistance factor, based on Single Pile in Axial Compression - Static Analysis Methods

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

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1} \qquad R_{r\_p1} = \begin{pmatrix} 82 \\ 111 \\ 135 \\ 178 \end{pmatrix} \cdot kip$$

Factored Axial Geotechnical Side Resistance - *Strength Limit States*

Resistance factor, based on Singel Pile in Axial Compression - Static Analysis Methods (ref: AASHTO LRFD Table 10.5.5.2.3-1

$\phi_{stat} := 0.45$

Driven Analysis

Side friction analyses were done using the soil profile at BB-BGWS-101 and BB-BGWS-102, and the pile skin/shaft friction values were lower at BB-BGWS-101, therefore govern the design.

$$\boxed{R_{side\_1} := \begin{pmatrix} 56 \\ 67 \\ 71 \\ 76 \end{pmatrix} \cdot kip}$$

Factored Geotechnical Side Resistance (Rr)

$R_{r\_side} := \phi_{stat} \cdot R_{side\_1}$

$$\boxed{R_{r\_side} = \begin{pmatrix} 25 \\ 30 \\ 32 \\ 34 \end{pmatrix} \cdot kip}$$

Percentage of Shaft Resistance from DRIVEN for use in GRLWEAP

Based on first interation of drivability analyses, and using the shaft pile friction estimates from Driven using soil profile at BB-BGWS-101:
12 x 53 - 13% side friction
14 x 73 - 10%
14 x 89 - 9%
14 x 117 - 10%

Total Nominal Axial Geotechnical Resistance - *Service & Extreme Limit States*

Resistance factor, reference: AASHTO LRFD 10.5.51 and LRFD 10.5.5.3

$\phi_{service} := 1.0$

$R_n := \left(R_{p\_1} + R_{side\_1}\right) \cdot \phi_{service}$

$$\boxed{R_n = \begin{pmatrix} 239 \\ 313 \\ 371 \\ 471 \end{pmatrix} \cdot kip}$$

Total Factored Axial Geotechnical Resistance - *Strength Limit States*

$R_t := R_{r\_p1} + R_{r\_side}$

$$R_t = \begin{pmatrix} 107 \\ 141 \\ 167 \\ 212 \end{pmatrix} \cdot kip$$

Drivability Analyses

Ref: LRFD Article 10.7.8

For steel piles in compression or tension, driving stresses are limited to 90% of fy

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

$\sigma_{dr} := 0.90 \cdot 50 \cdot (ksi) \cdot \phi_{da}$

$\sigma_{dr} = 45 \cdot ksi$ driving stress cannot exceed 45 ksi

Compute the resistance that can be achieved in a drivablity analysis:

The resistance that must be achieved in a drivablity analysis will be the maximum factored pile load divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 page 10-38 gives resistance factor for dynamic test,

$\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium variablity. Past practice has been to test only 1 to 2 piles at small bridge sites. But, if it is anticipated that the pile group would be nonredundant, i.e. there will be < five piles - reduce Φ by 20%.

$\phi_{dyn\_red} := 0.65 \cdot .80$ $\phi_{dyn\_red} = 0.52$

Therefore, use 0.65

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 09-Mar-2010
12 x 53 soils at -101 with 13% side — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	43.24	1.63	5.9	7.81	15.41
410.0	44.12	1.78	6.1	7.91	15.64
420.0	45.01	1.94	6.3	8.01	15.83
440.0	46.58	2.34	6.8	8.18	16.16

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	26.00	ft
Pile Penetration	24.00	ft
Pile Top Area	15.50	in2

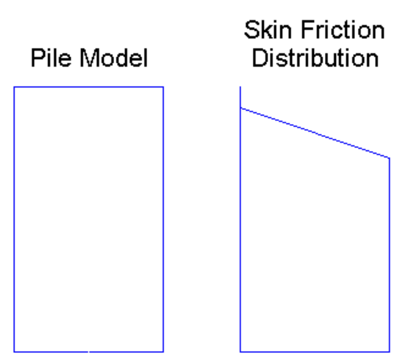
Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.12}{45.01 - 44.12}\right)\cdot(420\cdot kip - 410\cdot kip) + 410\cdot kip$$

$$R_{ndr} = 419.9\cdot kip$$

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 273\cdot kip}$$



Res. Shaft = 13 %
(Constant Res. Shaft)

Pile Size is 14 x 73

The 14x 73 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 73 Soil Profile at BB-BGWS-101

09-Mar-2010
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	36.22	1.46	4.8	8.26	16.39
500.0	41.88	2.98	6.4	9.06	17.61
520.0	42.97	3.13	6.8	9.24	17.93
540.0	44.02	3.22	7.2	9.41	18.19
560.0	45.04	3.25	7.6	9.57	18.45
580.0	45.97	3.34	8.1	9.71	18.63
600.0	46.88	3.47	8.6	9.86	18.85

Limiting driving stress to 45 ksi:

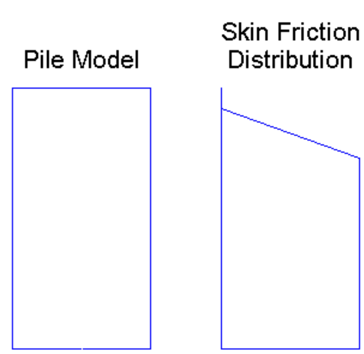
$R_{ndr} := 560 \cdot kip$

$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 364 \cdot kip}$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	26.00	ft
Pile Penetration	24.00	ft
Pile Top Area	21.40	in2



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 89

The 14 x 89 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 09-Mar-2010
14 x 89 Soil profile at BB-BGWS-101 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	32.79	1.47	4.8	8.19	15.94
620.0	43.25	3.74	8.8	9.69	18.01
640.0	44.08	3.99	9.3	9.83	18.20
660.0	44.92	4.13	9.8	9.97	18.51
740.0	47.96	4.51	12.5	10.49	19.66
750.0	48.37	4.48	12.8	10.58	19.86
760.0	48.71	4.44	13.2	10.64	19.99
780.0	49.56	4.24	13.8	10.81	20.36

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	26.00	ft
Pile Penetration	24.00	ft
Pile Top Area	26.10	in2

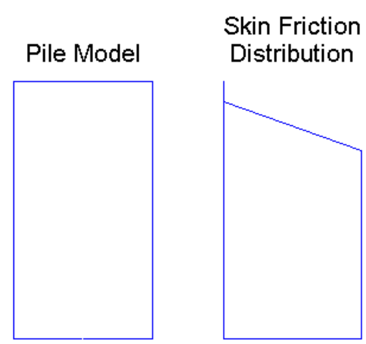
Limiting stress to 45 ksi:

$R_{ndr} := 660 \cdot kip$

$R_{ndr} = 660 \cdot kip$

$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$R_{fdr} = 429 \cdot kip$



Res. Shaft = 9 %
(Constant Res. Shaft)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a D 36-32 at Fuel Setting 2 and a 2.7 kip helmet, at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 09-Mar-2010
14x117 fuel set 2 =1305 psi BB-BGWS-101 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	37.36	0.08	3.2	7.47	29.29
650.0	44.24	0.89	4.2	8.05	30.74
700.0	46.20	1.13	4.6	8.26	31.20
780.0	49.13	1.93	5.3	8.52	31.56
800.0	49.74	1.98	5.5	8.58	31.73

$$R_{ndr} := \left(\frac{45 - 44.24}{46.20 - 44.22}\right)\cdot(700\cdot kip - 650\cdot kip) + 650\cdot kip$$

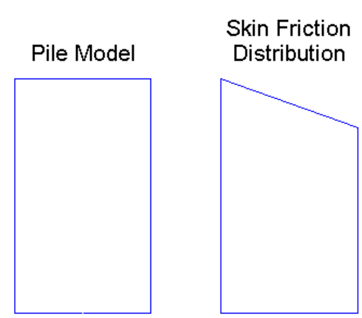
$$R_{ndr} = 669.2\cdot kip$$

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 435\cdot kip}$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	24.00	ft
Pile Penetration	24.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 10 %
(Constant Res. Shaft)

MaineDOT

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

Laura Krusinski, PE
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Soils Report 2009-34 Addendum #2

To:	Pam Heatherly, SEA Consultants
Cc:	Nate Benoit, Project Manager
Author:	Laura Krusinski
Subject:	Replacement of Jenkins Bridge Addendum 2 to Soils Report 2009-34
Doc Type:	24
Date:	March 31, 2010
Bridge #:	3365
Route:	Cram Street
PIN:	16687.00
Town:	Bradley

1.0 BACKGROUND

The MaineDOT Bridge Program originally planned to replace Jenkins Bridge in Bradley, Maine, which consists of twin 17-foot structural plate pipe arch culverts, with twin concrete boxes. A geotechnical report summarizing the subsurface investigation conducted at the site and geotechnical design recommendations for the precast concrete box alternative was published and is available as Soils Report 2009-34, dated 23 December 2010. See Soil Report 2009-34 for specific information regarding the subsurface investigation, laboratory testing and detailed subsurface conditions at the site.

Subsequently, the Jenkins Bridge site has been selected as a location to install a 32-foot span, composite tubular arch bridge structure developed by the University of Maine's Advanced Engineering Wood & Composites Center (AEWC) in Orono, Maine.

The purpose of this Addendum #2 is to provide geotechnical design recommendations for Precast Concrete Modular Gravity (PCMG) retaining walls.

2.0 PRECAST CONCRETE MODULAR GRAVITY WALLS

Precast Concrete Modular Gravity (PCMG) walls may be used to retain approach fills. In general, PCMG wingwalls should be used only at stream crossings where the flow velocities are low, and the potential for severe ice or wave action is low. PCMG walls should also only be used above the ordinary mean high water elevation (Q1.1). The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. PCMG walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Retaining Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic: 0 feet	Distance from wall pressure surface to edge of traffic: >=1 foot
5	5.0	2.0
10	3.5	2.0
>=20	2.0	2.0

Equivalent Height of Soil for Estimating Live Load Surcharge on Walls

The bearing resistance for PCMG walls founded on a 6 by 12 inch (minimum) leveling slab and compacted granular bedding material or glacial till shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases 8 feet wide or less and 6.5 ksf for bases 10 to 14 feet wide. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state. The vertical stress may be calculated assuming a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1.

For the lowest PCMG unit on granular bedding material, the eccentricity of factored loads at the strength limit state shall not exceed one-fourth (1/4) of the footing dimensions, in either direction.

The bearing resistance for the bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, the PCMG wall units should be designed so that the factored bearing resistance, in conjunction with the depth of scour determined for the design flood for scour provides adequate resistance to support the factored strength limit state loads with appropriate resistance factors. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated design scour depth. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65

Failure by sliding shall be investigated by the wall designer-supplier. A sliding resistance factor, φ_T, of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on concrete fill or granular borrow. A sliding resistance factor of 0.90 shall be applied to the nominal sliding resistance of soil within the precast concrete units on granular bedding soils. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.46 (0.80·tan 30°) at the bedding soil to concrete unit interfaces, a maximum frictional coefficient of 0.58 (tan 30°) at foundation soil to soil-infill interfaces, and a maximum friction coefficient of 0.56 (0.80·tan 35°) at interfaces between concrete units and concrete fill. Recommended values of sliding frictional coefficients are based on LRFD Articles 11.11.4.2 and 10.6.3.4 and Table 10.5.5.2.2-1.

Retaining wall foundations placed on granular soils shall be embedded a minimum of 4.5 feet below finish exterior grade for frost protection.

Calculations – PCMG Walls Bearing Resistance

Prepared by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Analysis : Bearing Resistance of PCMG walls on granular bedding material

Assumptions

1. Base of footing founded with 4 feet embedment for frost (conservative, 4.5 feet is recommended.)

2. Assumed parameters for compacted granular backfill
   Saturated unit weight = 130 pcf (Bowles Table 3-4; Holtz, Kovacs, Table 2-1 1981)
   Dry unit weight = 125 pcf
   ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
   ϕ and SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
   ϕ = 32 degrees (Bowles Tables 3-4 and 2-6).
   Su= undrained shear strength (c) 0 psf

3. Method used: Terzaghi, use strip equations since L>B

PCMG Footing Width and Depth

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft \qquad D_f := 4.0 \cdot ft \qquad D_w := 0 \cdot ft \qquad \gamma_w := 62.4 \cdot pcf$$

Foundation Soil (Granular Fill)

$$\gamma 1_{sat} := 130 \cdot pcf$$

$$\gamma 1_d := 125 \cdot pcf$$

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

$$c_1 := 0 \cdot psf$$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Method**:** LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures* , Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Coarse to medium sand, little gravel	Very dense	8 to 12	8
	Medium dense to dense	4 to 8	6
	Loose	2 to 6	4

Recommend 6 ksf, to limit settlement to 1.0 inch for Service Limit State analyses and for preliminary footing sizing.

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for strip footing (Bowles 5th Ed., pg 220)

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

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223)

$N_c := 35.47$ $\qquad$ $N_q := 23.2$ $\qquad$ $N_\gamma := 22$

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

$$q := D_f \cdot (\gamma 1_{sat} - \gamma_w) \qquad q = 0.27 \cdot ksf$$

$$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_n = \begin{pmatrix} 10.7 \\ 12.2 \\ 13.7 \\ 15.2 \\ 16.7 \\ 18.2 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.45$

$q_r := q_n \cdot \varphi_b$

$q_r = \begin{pmatrix} 4.8 \\ 5.5 \\ 6.2 \\ 6.8 \\ 7.5 \\ 8.2 \end{pmatrix} \cdot ksf$ **for** $B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 5 ksf for footings 8 feet wide or smaller, on compacted granular fill.
Recommend a bearing resistance of 6.5 ksf for footings 10 to 14 feet wide or greater.