MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

DURGIN BRIDGE
KING ROAD OVER SABATTUS RIVER
LISBON, MAINE

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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for the replacement of the Durgin Bridge over the Sabattus River in Lisbon, Maine. The proposed replacement bridge will be single-span, approximately 90 feet long, steel I-beam superstructure founded on pile-supported integral abutments along the existing alignment. The design and construction recommendations below are discussed in greater detail in Section 7.0 Foundation Considerations and Recommendations.

Integral Abutment H-Piles – The abutments will be cast-in-place concrete stub abutments with “butterfly” return wings. The abutments will be supported on driven integral H-piles. The piles should be end bearing, driven to the required resistance on or within the bedrock. The piles should be oriented for weak axis bending. Driven piles should be fitted with driving points to protect the pile tips and improve penetration.

Piles will be 50 ksi, A572 steel H-piles. The factored structural resistance of the piles exceeds the factored static and drivability axial pile resistances. The drivability axial pile resistances from our analyses provide the best estimates of factored pile resistances. We recommend that the resistances from the drivability analyses be used for design. The contractor is required to perform a wave equation analysis and dynamic pile test. The nominal pile resistance that must be achieved in the wave equation analysis and dynamic testing is the maximum factored axial pile load divided by a resistance factor of $\varphi_{dyn} = 0.52$. The maximum factored pile load should be as shown on the plans. We present the design factored pile axial resistances in Section 7.1.1, Strength Limit State.

Integral Stub Abutment and Wingwalls – The integral abutments and wingwalls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, superstructure loads, creep, and temperature and shrinkage deformations of the superstructure. They shall be designed for all relevant service and strength limit states. Current plans include stub abutments with “butterfly” wingwalls. Thus, the designer should size the piles to account for the additional bending moment stress resulting from the cantilevered wingwall configuration.

Integral abutment and integral wingwall sections should be designed to resist passive earth pressure using a Coulomb earth pressure coefficient, K_p, equal to 6.89. Coulomb theory considers wall friction, which acts downward against the passive soil wedge and increases passive pressures. Developing full passive earth pressure requires displacements on the order of 2 to 5 percent of the abutment or wingwall height. Only if the calculated displacements are less than 0.5 percent of the wall or abutment height, may the designer consider using a Rankine earth pressure coefficient of 3.25, which assumes no wall friction. Wingwall sections that are independent of the abutment 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.

Scour Protection - Bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap. The riprap shall be underlain by a Class A erosion control geotextile and a 1-

foot thick layer of bedding material conforming to Item number 703.19, Granular Borrow for Underwater Backfill of the Standard Specification and as shown in Standard Detail 610(03). Riprap shall meet the requirements of Section 703.26, Plain and Hand Laid Riprap. For abutments and wingwalls, riprap shall extend 1.5 feet horizontally in front of walls before sloping down at a maximum 1.75H:1V slope to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation.

Settlement – The plans indicate a grade rise of about two feet. We estimate that settlement as a result of fill placement over existing fill and natural soils will be on the order of 1 inch or less. This considers removal of existing approach embankment walls 6 feet below finish roadway grade and fill placement outside and above the remaining walls to planned finish grade with 2:1 (H:V) outboard slopes.

Settlement of the bridge abutments will be limited to the axial compression of the piles which will occur as the bridge is constructed and will be negligible.

Frost Protection – Foundations placed on granular soils shall be founded a minimum of 5.5 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.

The existing dry laid granite walls must be removed down to a level at least 6 feet below the roadway shoulder finish grade elevation. This will help minimize differential frost heave of the approach pavement.

Seismic Design Considerations – In accordance with AASHTO LRFD Bridge Design Specifications, 4th Edition, with 2008 Interims (herein referred to as LRFD), Article 4.7.4.2, seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and bridge seat dimensions must satisfy LRFD Article 3.10.9 and 4.7.4.4, respectively.

Construction Considerations –

Excavation

- Construction of new abutment structures will require soil excavation. Earth support systems may be required.
- Remove the old u-shaped return walls a minimum of six feet below finish exterior grade in approach embankments. These walls will also likely have to be removed entirely at the new integral abutment location to accommodate pile installation. The existing stone abutment breastwall may be removed to accommodate riprap placement or they may remain and be buttressed with riprap up and downstream.
- 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, 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.

Installing Piles

- There is a potential that cobbles, boulders, timber cribbing, or quarried stone from old foundations and walls may obstruct pile driving operations at the proposed abutment locations. Obstructions may be cleared by conventional excavation methods, pre-drilling, or spudding. Alternative methods to clear obstructions may be used as approved by the Resident.

Reuse of Excavated Soil and Bedrock

- Do not use excavated existing subbase aggregate 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

MaineDOT plans to replace the Durgin Bridge carrying King Road over Sabattus River in the Town of Lisbon, Androscoggin 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 single-span bridge was built in 1947. The plans for that bridge indicate that the stone abutments predated the 1947 construction. The bridge constructed in 1947 simply capped the pre-existing mortared granite abutments with a new concrete abutment section. It is not known whether the pre-existing stone abutments were constructed over concrete footings or directly on soil. The existing span length is approximately 58 feet. The bridge had a sufficiency rating of 49.9 in 2008.

MaineDOT is proposing a replacement bridge that will be single-span, approximately 90 foot long, pile supported integral abutments with a steel I-beam superstructure and concrete deck. The new bridge will be on the same alignment as the existing bridge with a grade rise of approximately two feet. The new bridge will have an out-to-out width of approximately 32 feet. Current plans include armoring the approach and abutment fill embankments with riprap.

2.0 GEOLOGIC SETTING

The Maine Geologic Survey "Surficial Geology of Lisbon Falls North Quadrangle, Maine, Open-file No. 03-14" (2003) indicates that surficial soils in the vicinity of the Durgin Bridge consist of Presumpscot Formation sands, silt, and clays with nearby soil unit contacts with Marine Nearshore Deposits which consist of sand, gravel, and mud deposited in shallow marine environments. 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 Durgin Bridge site consists of Silurian-Ordovician, interbedded pelite and sandstone of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

We investigated subsurface conditions at the site by drilling two test borings, BB-LSS-101 and BB-LSS-102, conducted by the MaineDOT drill crew. The borings were terminated with bedrock cores. The boring locations and soil profile are shown on Sheet 2, Boring Location and Interpretive Subsurface Profile. The borings BB-LSS-101 and BB-LSS-102 were conducted on October 30 and December 18, 2008, respectively. 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 MaineDOT Certified Subsurface Inspector logged the subsurface conditions encountered on the field logs. The MaineDOT survey crew determined the boring location coordinates in the field when they collected the project survey data.

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 0.77 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 fourteen (14) standard grain size analyses with natural water contents and one loss on ignition test. 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. We typically found glaciomarine sands over bedrock. However, the bridge itself is situated at the end of short fill extensions built into the Sabattus River flood plain. Consequently, the soil behind the existing abutments is predominantly granular fill overlying approximately 30 to 32 feet of glaciomarine sand. At the BB-LSS-101 boring location, the sand was underlain by granite bedrock. At the BB-LSS-102 boring location, the sand was underlain by metamorphic gneiss bedrock. 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 9.0 and 9.5 feet below ground surface (bgs) in BB-LSS-101 and BB-LSS-102, respectively. Based on the boring logs, the fill layer is generally comprised of two subunits. The upper unit consists of fine to coarse sand with little to some gravel and little silt. The lower unit consists of fine to coarse sand with some silt to silty and trace gravel which also contained trace organics at the BB-LSS-102 location. The SPT N_{60}-values in the granular fill ranged from 4 to 21 blows per foot (bpf) indicating that the unit is very loose to medium dense in consistency.

The granular fill samples had water contents ranging between approximately 6 and 21 percent. Grain size analyses conducted on selected samples of the fill soils indicate that the soils are classified as A-1-b, A-2-4 and A-4 by the AASHTO Classification System and SM under the Unified Soil Classification System.

5.2 Marine Deposited Sediments

All of the soils beneath the fill layer were deposited in marine environments. The soil immediately beneath the fill soils are typically alluvial gravels to silts. The remainder of the soil sequence above bedrock consists of glaciomarine sandy silts and silty sands. The thickness of the combined alluvial and glaciomarine sediments ranged between 31.4 and 32.9 feet at BB-LSS-101 and BB-LSS-102, respectively.

The alluvial deposits consisted of loose gravel with some fine to coarse sand and some silt or stiff organic silt with trace fine sand. The glaciomarine soils are typically stiff to very stiff silt with some fine sand and trace gravel or fine to coarse sandy silt with trace gravel. SPT N_{60}-values ranged from 4 to 42 bpf, indicating these granular soils are very loose to dense and the silts are stiff to very stiff in consistency.

The marine sediments had water contents ranging between 19 and 30 percent. Grain size analyses conducted on selected samples indicate that the soils are classified as A-1-b, A-2-4, A-3 and A-4 by the AASHTO Classification System and GM, SM, SP, SP-SM and ML under the Unified Soil Classification System.

5.3 Bedrock

We encountered bedrock at approximate depths of 40.4 and 42.4 feet bgs at BB-LSS-101 and BB-LSS-102, respectively. Locally, the bedrock is mapped as the Vassalboro Formation which is made up of interbedded pelite and sandstone. Visual identification of rock cores indicates that the bedrock at BB-LSS-101 is a grey, fine to medium-grained granite, very hard and moderately fractured. We determined that the rock quality designation (RQD) of the bedrock ranged from 33 to 67 percent which correlates to a poor to fair rock mass quality. Visual identification of bedrock at BB-LSS-102 is a white and grey, fine to medium-grained gneiss, moderately hard and moderately fractured. We determined that the rock quality designation (RQD) of the bedrock ranged from 94 to 100 percent which correlates to a very good 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-LSS-102	6+13, 6.1 LT	42.4	144.6
Abutment No. 2	BB-LSS-101	7+05, 6.5 RT	40.4	144.6

Bedrock Depth and Elevation at the Boring Locations

5.4 Groundwater

We interpreted groundwater levels at the boring locations based on field observations. Groundwater occurred at approximate depths of 9.0 and 15.0 feet bgs at BB-LSS-101 and BB-LSS-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 four alternate replacement designs: 1) steel girder on H-pile supported integral abutments; 2) steel truss on H-pile supported stub abutments; 3) precast concrete voided slab with full height concrete cantilever abutments; and 4) precast, prestressed concrete box beam sections founded on H-pile supported integral abutments. The project team selected alternate No. 1, steel girder on H-pile supported integral abutments, for the replacement structure. The following section presents geotechnical design recommendations for precast, H-pile supported integral abutments and wingwalls.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The design team has selected a single-span, integral abutment structure to replace the bridge at the Grand Lake Stream site. The proposed replacement bridge will be approximately 90 feet long and consist of a steel girder with a cast in place concrete deck founded on H-pile supported integral abutments. The new bridge will be on the same alignment as the existing bridge with a grade rise of about two feet. The new bridge will have an out-to-out width of approximately 32 feet. The design methodology used in the following evaluation is referenced from the AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, with 2008 Interims.

The replacement bridge will be a “hybrid” integral structure using three H-piles. The piles will be capped with reinforced concrete. The steel bridge girders will be anchored to the cap and concrete will be placed around and above the anchored bridge girders.

7.1 Integral Abutment H-piles

The piles should be end bearing, driven to the required resistance on or within the bedrock, and oriented for weak axis bending (perpendicular to superstructure beams). Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the factored design axial loads. Foundation piles should consist of 50 ksi, Grade A572 steel H-piles fitted with driving points to protect the tips, improve penetration, and improve friction at the pile tip.

The contractor may estimate the required pile lengths based on the following data. The estimated pile length below does not include embedment in the pile cap (embedment can range from 2 to 6 feet) or lead length required for installation.

Location	Estimated Bottom of Pile Cap Elevation (feet)	Top of Bedrock Elevation (feet)	First Run RQD (%)	Estimated Pile Length (feet)[1]
Abutment 1 BB-GLS-102	179	145	100	34
Abutment 2 BB-GLS-101	178	145	67	33

[1] pile length does not include embedment in the pile cap (2 to 6 feet anticipated) or lead length required for installation

Estimated Pile Lengths for Piles Installed to Depth of Bedrock Surface

Typically, the designer will design the H-piles at the strength limit state considering the combined axial and flexural structural resistance of the piles, and the axial geotechnical resistance of the piles. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed below.

The design of H-piles at the service limit state should consider tolerable horizontal movement of the piles, and overall stability of the pile group. Since the abutment piles will be subjected to lateral loading, the pile should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2.

7.1.1 Strength Limit State

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. For preliminary analysis, the factored structural axial compressive resistances of the three proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 and column slenderness factor, λ, of 0. It is the responsibility of the designer to recalculate λ for the upper and lower portions of the H-pile based on unbraced lengths and an effective length factor (K) from project specific analyses and then recalculate the structural resistances.

The nominal geotechnical axial compressive resistance in the strength limit state was calculated using the Pell, Turner, Tomlinson method referenced in Tomlinson (1994). Since there are less than five piles in each substructure, they are deemed “non-redundant” in LRFD Article 10.5.5.2.3. Thus, the resistance factor from LRFD Table 10.5.5.2.3-1, φ_{stat}, of 0.45 must be reduced 20 percent in accordance with Article 10.5.5.2.3. Consequently, the factored geotechnical compressive resistances of the three proposed H-pile sections were calculated using a resistance factor φ_{stat}, of 0.36 for end bearing. We also used Driven 1.0 software (FHWA 2003) to estimate individual pile skin friction.

We also calculated the nominal geotechnical compressive resistance in a wave equation drivability analysis using GRLWEAP. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The resistance factor for a single pile in axial compression with the driving resistance established by a dynamic load test per LRFD Table 10.5.5.2.3-1 is $\varphi_{dyn} = 0.65$. Table 10.5.5.2.3-3 requires that no less than 3 or 4 dynamic tests be conducted for sites with low to medium variability. Since we typically perform only two tests per bridge, one per abutment, and the pile group is non-redundant, we have reduced this factor by 20 percent resulting in a resistance factor of $\varphi_{dyn} = 0.52$.

We present the factored axial compressive structural, geotechnical and drivability resistances for the four proposed H-pile sections in the table below. Supporting calculations are provided in Appendix C, Calculations. Based on our analysis, we recommend that the factored drivability resistance be used for strength limit state design.

H-Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Static Resistance	Drivability Resistance	Governing Pile Resistance
12 x 53	465	220	217	217
12 x 74	654	302	354	354
14 x 73	642	302	345	345
14 x 89	783	364	400	400
14 x 117	1032	471	422	422

Factored Axial Pile Resistances at the Strength Limit State

In accordance with LRFD Article 6.5.4.2 at the strength limit state, H-piles in compression and bending, the axial resistance factor $\phi_c = 0.7$ and the flexural resistance factor $\phi_f = 1.0$ shall be applied to the combined axial and flexural resistance of the pile in the interaction equation. For the strength limit state, the combined axial compression and flexure should be evaluated as shown in LRFD Article 6.9.2.2. The structural designer should evaluate the capacity of the pile in combined axial load and flexure when the loads and moments are calculated. Additional bending moments resulting from the abutment wingwalls must also be considered in design of the piles.

7.1.2 Service and Extreme Limit States

In accordance with LRFD Article 10.5.5, Resistance Factors, the resistance factors for the service and extreme limit states for structural and geotechnical pile resistances are 1.0. We present the factored axial compressive structural, geotechnical and drivability resistances for the four proposed H-pile sections at the service/extreme limit state in the table below. Supporting calculations are provided in Appendix C, Calculations. Based on our analysis, we recommend that the factored drivability resistance be used for service/extreme limit state design.

H-Pile Section	Service/Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Static Resistance	Drivability Resistance	Governing Pile Resistance
12 x 53	775	610	417	417
12 x 74	1090	840	681	681
14 x 73	1070	838	663	663
14 x 89	1305	1011	770	770
14 x 117	1720	1309	811	811

Factored Axial Pile Resistances at the Service/Extreme Limit State

7.1.3 Pile Resistance and Pile Quality Control

The contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. The nominal pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the maximum factored axial pile load divided by a resistance factor of 0.52. The maximum factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis, the dynamic test results, and as approved by the resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. The contractor should select a hammer that provides the required nominal resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the pile penetration is less than 0.5-inch in 10 consecutive blows.

7.1.4 L-Pile Analysis Parameters

We recommend that the structural designer use the following parameters in their L-Pile analysis. In general, the model should emulate the soil/structure conditions at the site by using four (4) layers (referenced bgs) with the following parameters:

Layer No	Depth (ft, bgs)	Water Table Condition	k_s (lb/in^3)	Effective Wt (lb/in^3)	Friction Angle (degrees)
1	0-3	Above	90	0.0694 (120 pcf)	32
2	3-10	Above	25	0.0666 (115 pcf)	30
3	10-15	Below	60	0.0307 (53 pcf)	30
4	15-41	Below	20	0.0307 (53 pcf)	30

The total model height should be 41 feet high (avg of 40 and 42 foot depth of borings). Considering this, and a roughly 10-foot tall stub abutment with 2-foot pile embedment, the pile length should be 33 feet. The designer should adjust this for the actual abutment height and embedment.

7.2 Integral Stub Abutments and Wingwalls

Integral stub abutments and wingwalls 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 11.6.1.3. The design of abutments and wingwalls at the strength limit state shall consider structural failure. Integral abutments and wingwalls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, superstructure loads, creep, and temperature and shrinkage deformations of the superstructure. Current plans include stub abutments with "butterfly" wingwalls. Thus, the designer should size the piles to account for the additional bending moment stress resulting from the cantilevered wingwall sections.

Integral abutment and integral wingwall sections should be designed to resist passive earth pressure using a Coulomb earth pressure coefficient, K_p, equal to 6.89. Coulomb theory considers wall friction, which acts downward against the passive soil wedge and increases passive pressures. Developing full passive earth pressure requires displacements on the order of 2 to 5 percent of the abutment or wingwall height. Only if the calculated displacements are less than 0.5 percent of the wall or abutment height, may the designer consider using a Rankine earth pressure coefficient of 3.25, which assumes no wall friction. Wingwall sections that are independent of the abutment 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.

To minimize water intrusion behind the abutment, the approach slab should connect directly to the abutment, and appropriate provisions should be made to provide for drainage for any entrapped water. Backfill that is within 10 feet of the abutments and wingwalls and side slope fill should conform to MaineDOT Standard Specification 709.19, Granular Borrow for Underwater Backfill. This material requires 10 percent or less material passing the No. 200 which will help minimize frost action behind the structure.

7.3 Scour Protection

The designer shall consider the consequences of changes in foundation conditions at the service and extreme limit states resulting from scour due to the design flood event. The extreme limit state shall determine that there is adequate foundation resistance to support the unfactored strength limit state loads with a resistance factor of 1.0, in accordance with AASHTO LRFD Article 10.5.2.1. Changes in foundation conditions shall be investigated at pile-supported abutments and wingwalls. Integral abutment piles rely on the stability of slopes to provide lateral support. Therefore scour protection and armoring of the 1.75H:1V slopes in front of the abutments and along the approach embankments is critical. For the Lisbon site, the designer has specified the use of riprap for scour protection. Refer to BDG Section 2.3.11 for additional information regarding scour design.

For abutments and wingwalls, the riprap shall extend 1.5 feet horizontally in front of the structure before sloping at maximum 1.75H:1V slope to the existing ground surface. The riprap shall be underlain by a Class A erosion control geotextile and a 1-foot thick layer of bedding material conforming to Item number 703.19, Granular Borrow for Underwater Backfill of the Standard Specification and as shown in Standard Detail 610(03). Riprap shall meet the requirements of Section 703.26, Plain and Hand Laid Riprap. The toe of the riprap layer shall be constructed 1 foot below the streambed elevation.

7.4 Settlement

The current bridge replacement plans include profile changes on the order a two foot grade rise. Thus, we estimate that settlement as a result of fill placement over existing fill and natural soils will be on the order of 1 inch or less. This considers removal of existing approach embankment walls 6 feet below finish roadway grade and fill placement outside and above the remaining walls to planned finish grade with 2:1 (H:V) outboard slopes.

We expect that any settlement of the bridge abutments will be limited to the axial compression of the piles which will occur as the bridge is constructed and will be negligible.

7.5 Frost Protection

We have evaluated the potential frost depth at the Lisbon 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 1440 F-degree days. This correlates to a frost depth of 5.5 feet. Consequently, we recommend that any spread footing or leveling pads constructed at the site be founded a minimum of 6.0 feet below finished exterior grade. This minimum embedment applies only to foundations constructed on soil and not those founded on bedrock. We recommend that integral abutments be embedded a minimum of 4 feet for frost protection as shown on Figure 5-2 of the MaineDOT BDG.

We also recommend that the existing dry laid granite walls be removed down to a level at least 6 feet below the roadway shoulder finish grade elevation. This will help minimize differential frost heave of the approach pavement.

7.6 Seismic Design Considerations

The Durgin Bridge is not classified as a major structure since construction costs will be less than $10 million dollars, nor is it on the National Highway System. Thus the bridge is not classified as functionally important or essential in the BDG or LRFD. In conformance with LRFD Article 4.7.4.2, seismic analysis is not required for single-span bridges, regardless of seismic zone. However, superstructure connections and bridge seat dimensions shall be satisfied per LRFD 3.10.9 and 4.7.4.4, respectively. Seismic earth loads do not need to be considered in bridge substructure design.

7.7 Construction Considerations

7.7.1 Installing Piles

There is a potential that cobbles, boulders, timber cribbing, or quarried stone from old foundations may obstruct pile driving operations at the proposed abutment locations. Obstructions may be cleared by conventional excavation methods, pre-drilling, or spudding. Alternative methods to clear obstruction may be used as approved by the Resident.

7.7.2 Excavation

Construction of the new abutment structures will require soil excavation. Earth support systems may be required. The fill and marine sediment 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).

7.7.3 Dewatering

The native fill and marine sediment 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 marine 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.

7.7.4 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 marine sediment 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 these 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.

7.7.5 Embankment Fill Areas

The current project plans require construction of fill extensions along the bridge approaches and in front of the abutments. 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.

7.7.6 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 Durgin Bridge over Sabattus River in Lisbon, 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

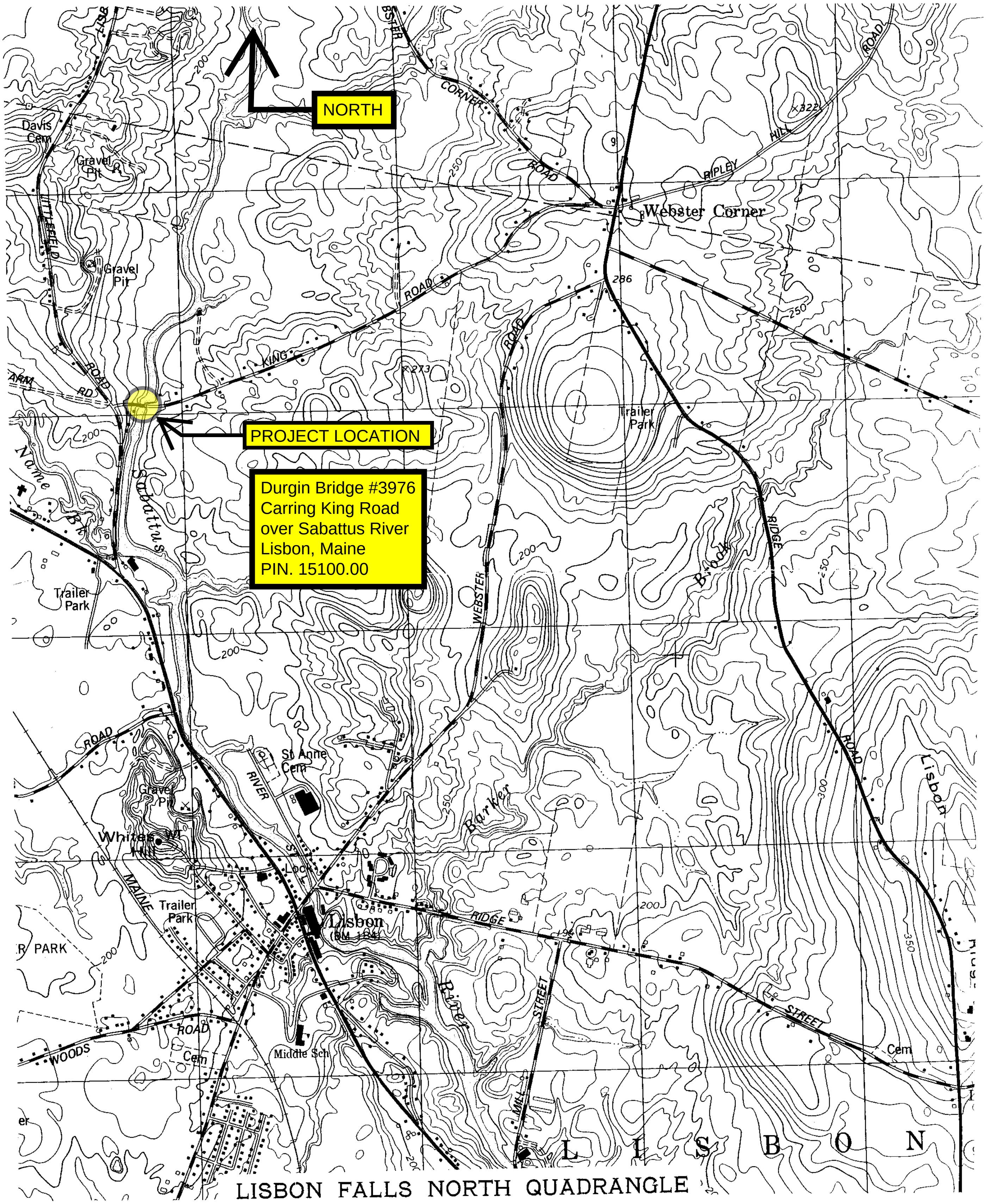
AASHTO, (2007), LRFD Bridge Design Specifications, Fourth Edition, with 2008 Interims, AASHTO, Washington, D.C.

Fang, Hsai-Yang (1991), Foundation Engineering Handbook, Second Edition, Van Nostrand Reinhold, New York, NY.

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

Tomlinson, M. J., (1994), Pile Design and Construction Practice, Fourth Edition, E & FN Spon, New York, NY.

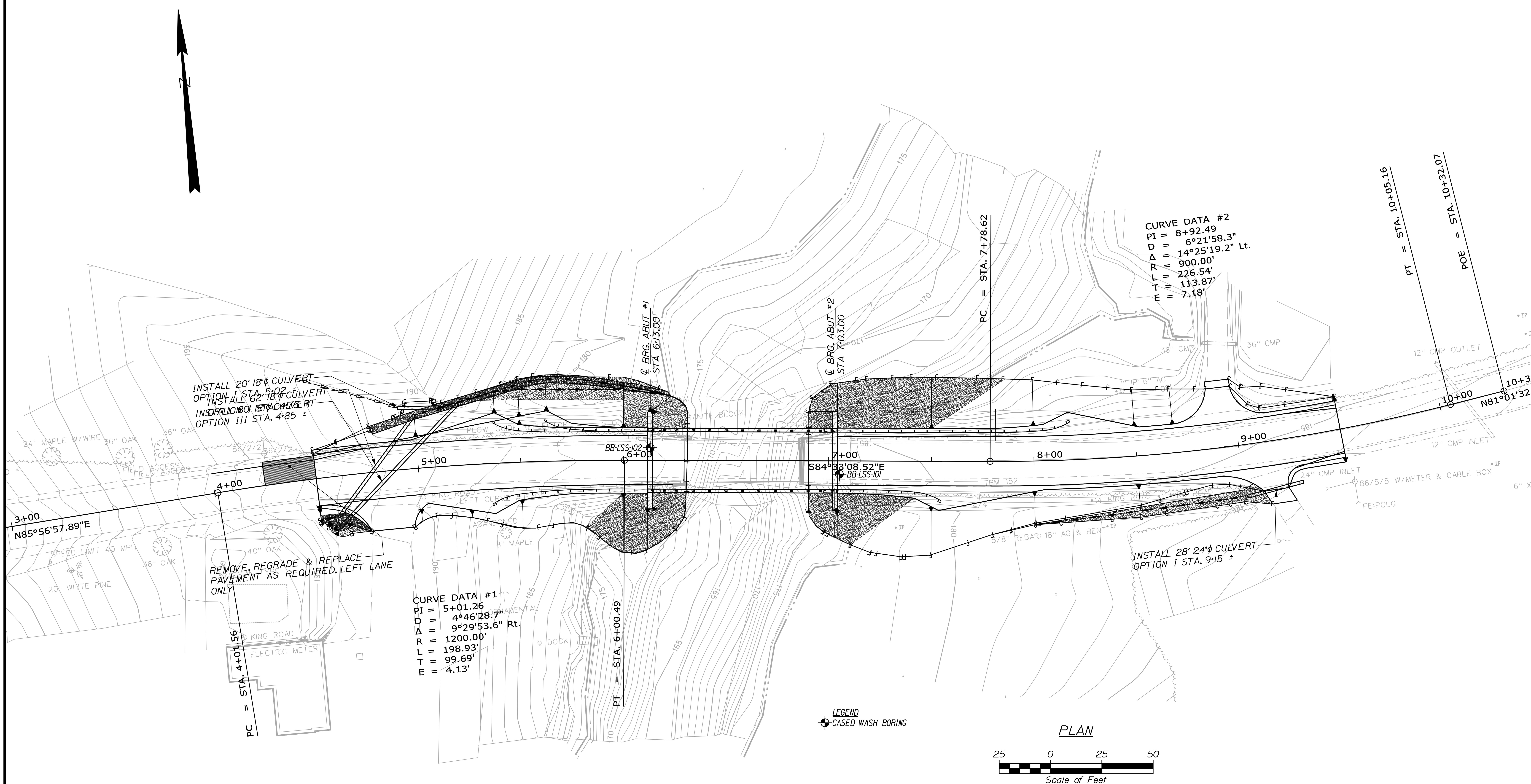
Sheets



LISBON FALLS NORTH QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)
SE/4 LEWISTON 15' QUADRANGLE

SCALE 1:24 000

Filename: ...\00\geotech\msta\004_BLP1.dgn   Division: GEOTECH   Username: terry.white   Date:6/3/2009



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A510(000)X

BRIDGE NO. 3976 | PIN 15100.00 | BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	T. WHITE	NOV 2008
CHECKED-REVIEWED	M. MOREAU	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

DURGIN BRIDGE
SABATTUS RIVER
LISBON   ANDROSCOGGIN COUNTY

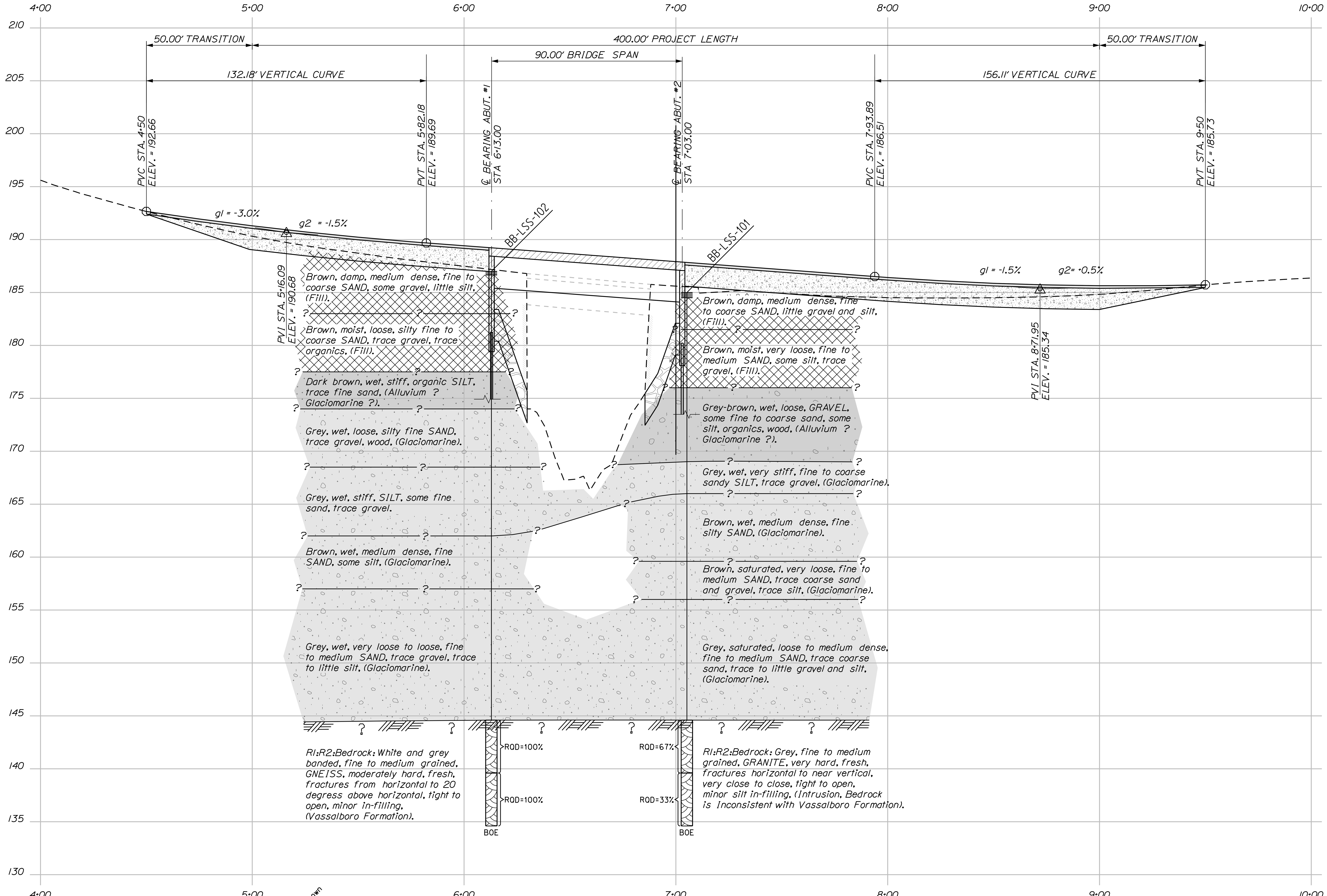
BORING LOCATION PLAN

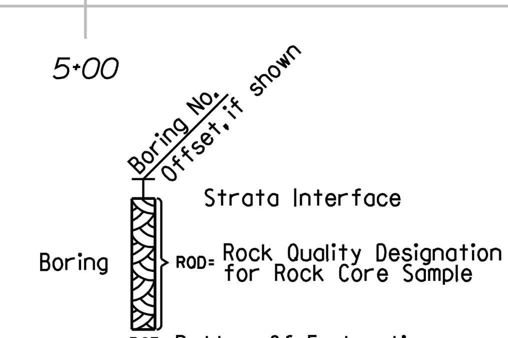
SHEET NUMBER

2

OF 4

Filename: ...\00\geotech\msta\005_ISP1.dgn   Division: GEOTECH   Username: terry.white   Date:6/3/2009





PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A510(000)X

PIN 15100.00

BRIDGE PLANS

BRIDGE NO. 3976

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	T. WHITE	NOV 2008
CHECKED-REVIEWED	M. MOREAU	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

DURGIN BRIDGE
SABATTUS RIVER
LISBON ANDROSCOGGIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\msta\006_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:6/3/2009

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Durgin Bridge #3976 Carrying King Road over the Sabattus River
Location: Lisbon, Maine

Boring No.: BB-LSS-102
PIN: 15100.00

Driller:	MaineDOT	Elevation (ft.)	187.0	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/18/08; 08:00-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	6+12.9, 6.1 Lt.	Casing ID/OD:	NW	Water Level*:	15.0' bgs.
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0							SSA
1	1D	24/19	1.00 - 3.00	7/9/7/7	16	21	
5	2D	24/20	5.00 - 7.00	2/2/2/7	4	5	
10	3D	24/14	10.00 - 12.00	5/12/21/7	33	42	53
11							33
12							40
13							72
14							44
15	4D	24/6	15.00 - 17.00	3/3/4/4	7	9	39
16							46
17							50
18							56
19							40
20	5D	24/11	20.00 - 22.00	3/4/3/4	7	9	49
21							48
22							46
23							43
24							41
25	6D	24/12	25.00 - 27.00	6/5/4/4	9	12	41
26							37
27							40
28							38
29							37
30	7D	24/18	30.00 - 32.00	2/2/3/3	5	6	47
31							46
32							47
33							46
34							47
35	8D	24/16	35.00 - 37.00	2/2/2/13	4	5	63
36							69
37							58
38							47
39							47
40	9D	24/14	40.00 - 42.00	WOR/WOR/3/3	3	4	65
41							70
42	R1	60/60	42.40 - 47.40	RQD = 100%			a100 NQ-2
47	R2	60/60	47.40 - 52.40	RQD = 94%			

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
186.65	0.35	PAVEMENT.	
183.00	4.00	Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt. (Fill).	G#175859 A-1-b, SM WC=5.5%
177.50	9.50	Brown, moist, loose, silty fine to coarse SAND, trace gravel, trace organics. (Fill)	G#175860 A-4, SM WC=20.0%
174.00	13.00	Dark brown, wet, very stiff, organic SILT, trace fine sand. (Alluvium? Glaciomarine?) Note, SPT blow count reflects wood interference. Wood layer from 10.7-11.4' bgs.	
168.50	18.50	Grey, wet, loose, silty fine SAND, trace gravel, wood. (Glaciomarine)	
162.00	25.00	Grey, wet, stiff, SILT, some fine sand, trace gravel. (Glaciomarine)	G#175861 A-4, ML WC=29.9%
157.00	30.00	Brown, wet, medium dense, fine SAND, some silt. (Glaciomarine)	G#175862 A-2-4, SM WC=30.1%
148.50	38.50	Grey, wet, loose, fine to medium SAND, trace silt. (Glaciomarine) Similar to above.	G#175863 A-3, SP-SM WC=23.2%
144.60	42.40	Grey, wet, very loose, fine to medium SAND, trace gravel, little silt. (Glaciomarine) a100 blows for 0.4'.	G#175864 A-2-4, SM WC=24.9%
134.60	52.40	Top of Bedrock at Elev. 144.6'. Bedrock: White and grey banded, fine to medium grained, biotite, quartz, amphibole, amazonite GNEISS, moderately hard, fresh, fractures from horizontal to 20 degrees above horizontal, tight to open, with minor silt in-filling. Joints appear to follow the biotite cleavage. [Vassalboro Formation] R1:Core Times (min:sec) 42.4-43.4' (2:27) 43.4-44.4' (2:42) 44.4-45.4' (2:08) 45.4-46.4' (2:42) 46.4-47.4' (4:02) 100% Recovery R2:Core Times (min:sec) 47.4-48.4' (4:36) 48.4-49.4' (2:52) 49.4-50.4' (2:35) 50.4-51.4' (2:46) 51.4-52.4' (3:08) 100% Recovery	

Bottom of Exploration at 52.40 feet below ground surface.

Remarks:

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

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

Page 1 of 1

Boring No.: BB-LSS-102

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Durgin Bridge #3976 Carrying King Road over the Sabattus River
Location: Lisbon, Maine

Boring No.: BB-LSS-101
PIN: 15100.00

Driller:	MaineDOT	Elevation (ft.)	185.0	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/30/08; 08:00-14:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+05.2, 6.5 Rt.	Casing ID/OD:	HW	Water Level*:	9.0' bgs.
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0							SSA
1	1D	24/18	1.00 - 3.00	8/9/5/5	14	18	
5	2D	24/20	5.00 - 7.00	2/1/2/2	3	4	14
6							18
7							24
8							46
9							27
10	3D	24/13	10.00 - 12.00	5/2/2/2	4	5	25
11							21
12							13
13							14
14							20
15	4D	24/14	15.00 - 17.00	2/5/17/9	22	28	37
16							63
17							115
18							111
19							77
20	5D	24/12	20.00 - 22.00	10/6/7/6	13	17	111
21							88
22							77
23							63
24							50
25	6D	24/12	25.00 - 27.00	9/2/1/2	3	4	OPEN HOLE
30	7D	24/10	30.00 - 32.00	8/3/1/1	4	5	
35	8D	24/16	35.00 - 37.00	10/6/4/4	10	13	
40	MD R1	4.8/0 60/60	40.00 - 40.40 40.40 - 45.40	50(4.8") RQD = 67%	---		NQ-2
45	R2	60/60	45.40 - 50.40	RQD = 33%			

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
184.50	0.50	PAVEMENT.	
181.50	3.50	Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	G#175851 A-1-b, SM WC=5.9%
176.00	9.00	Brown, moist, very loose, fine to medium SAND, trace gravel, some silt. (Fill)	G#175852 A-2-4, SM WC=20.5%
169.00	16.00	Grey-brown, wet, loose, GRAVEL, some fine to coarse sand, some silt, little organics with wood. (Alluvium? Glaciomarine?)	G#175853 A-1-b, GM WC=27.3% Loss Ignition Loss=5.5% H2O=39.0% G#175854 A-4, SM WC=28.8%
166.00	19.00	Grey, wet, very stiff, fine to coarse sandy SILT, trace gravel. (Glaciomarine)	
160.00	25.00	Brown, wet, medium dense, fine silty SAND. (Glaciomarine)	G#175855 A-4, SM wc=26.8%
156.00	29.00	Brown, saturated, very loose, fine to medium SAND, trace coarse sand and gravel, trace silt. (Glaciomarine)	G#175856 A-3, SP-SM WC=20.3%
144.60	40.40	Grey, saturated, loose to medium dense, fine to medium SAND, trace coarse sand, trace to little gravel and silt. (Glaciomarine) Failed sample attempt.	G#175857 A-3, SP WC=20.4% G#175858 A-2-4, SM WC=18.5%
134.60	50.40	Top of Bedrock at Elev. 144.6'. Bedrock: Grey, fine to medium grained, GRANITE, very hard, fresh, fractures from horizontal to near vertical, very close to close, tight to open, with minor silt in-filling. Some secondary iron pyrite mineralization oxidized into hematite with cubic crystal forms remaining betweeen 45.4' and 46.6'. [Bedrock Intrusion, Inconsistent with Vassalboro Formation] R1:Core Times (min:sec) 40.4-41.4' (2:56) 41.4-42.4' (2:49) 42.4-43.4' (2:39) 43.4-44.4' (2:29) 44.4-45.4' (2:12) 100% Recovery R2:Core Times (min:sec) 45.4-46.4' (3:00) 46.4-47.4' (3:24) 47.4-48.4' (3:37) 48.4-49.4' (2:43) 49.4-50.4' (2:45) 100% Recovery	

Bottom of Exploration at 50.40 feet below ground surface.

Remarks:

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

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

Page 1 of 1

Boring No.: BB-LSS-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A510(000)X

BRIDGE NO. 3976 | PIN 15100.00 | BRIDGE PLANS

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	M. MOREAU	T. WHITE	NOV 2008
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

LISBON
DURGIN BRIDGE
SABATTUS RIVER
ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER

4

OF 4

Appendix A

Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Durgin Bridge #3976 Carrying King Road over the Sabattus River **Location:** Lisbon, Maine	**Boring No.:** BB-LSS-101 **PIN:** 15100.00

Driller:	MaineDOT	**Elevation (ft.)**	185.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/30/08; 08:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+05.2, 6.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	184.50		PAVEMENT. 0.50	
	1D	24/18	1.00 - 3.00	8/9/5/5	14	18				Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	G#175851 A-1-b, SM WC=5.9%
								181.50		3.50	
5	2D	24/20	5.00 - 7.00	2/1/2/2	3	4	14			Brown, moist, very loose, fine to medium SAND, trace gravel, some silt. (Fill)	G#175852 A-2-4, SM WC=20.5%
							18				
							24				
							46				
							27	176.00		9.00	
10	3D	24/13	10.00 - 12.00	5/2/2/2	4	5	25			Grey-brown, wet, loose, GRAVEL, some fine to coarse sand, some silt, little organics with wood. (Alluvium? Glaciomarine?)	G#175853 A-1-b, GM WC=27.3% Loss Ignition Loss=5.5% H2O=39.0%
							21				
							13				
							14				
							20				
15	4D	24/14	15.00 - 17.00	2/5/17/9	22	28	37				G#175854 A-4, SM WC=28.8%
							63	169.00		16.00 Grey, wet, very stiff, fine to coarse sandy SILT, trace gravel. (Glaciomarine)	
							115				
							111				
							77	166.00		19.00	
20	5D	24/12	20.00 - 22.00	10/6/7/6	13	17	111			Brown, wet, medium dense, fine silty SAND. (Glaciomarine)	G#175855 A-4, SM wc=26.8%
							88				
							77				
							63				
							50				
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:	Durgin Bridge #3976 Carrying King Road over the Sabattus River	**Boring No.:**	BB-LSS-101		
Location:	Lisbon, Maine	**PIN:**	15100.00		
Driller:	MaineDOT	**Elevation (ft.)**	185.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/30/08; 08:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+05.2, 6.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.00 - 27.00	9/2/1/2	3	4	OPEN HOLE	160.00		25.00 Brown, saturated, very loose, fine to medium SAND, trace coarse sand and gravel, trace silt. (Glaciomarine)	G#175856 A-3, SP-SM WC=20.3%
								156.00		29.00	
30	7D	24/10	30.00 - 32.00	8/3/1/1	4	5				Grey, saturated, loose to medium dense, fine to medium SAND, trace coarse sand, trace to little gravel and silt. (Glaciomarine)	G#175857 A-3, SP WC=20.4%
35	8D	24/16	35.00 - 37.00	10/6/4/4	10	13					G#175858 A-2-4, SM WC=18.5%
40	MD	4.8/0	40.00 - 40.40	50(4.8")	---		NQ-2	144.60		Failed sample attempt. 40.40	
	R1	60/60	40.40 - 45.40	RQD = 67%						Top of Bedrock at Elev. 144.6'. Bedrock: Grey, fine to medium grained, GRANITE, very hard, fresh, fractures from horizontal to near vertical, very close to close, tight to open, with minor silt in-filling. Some secondary iron pyrite mineralization oxidized into hematite with cubic crystal forms remaining betweeen 45.4' and 46.6'. [Bedrock Intrusion, Inconsistent with Vassalboro Formation] R1:Core Times (min:sec) 40.4-41.4' (2:56) 41.4-42.4' (2:49) 42.4-43.4' (2:39) 43.4-44.4' (2:29) 44.4-45.4' (2:12) 100% Recovery	
45	R2	60/60	45.40 - 50.40	RQD = 33%						R2:Core Times (min:sec) 45.4-46.4' (3:00) 46.4-47.4' (3:24) 47.4-48.4' (3:37)	
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Durgin Bridge #3976 Carrying King Road over the Sabattus River
Location: Lisbon, Maine

Boring No.: BB-LSS-101
PIN: 15100.00

Driller:	MaineDOT	**Elevation (ft.)**	185.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/30/08; 08:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+05.2, 6.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							V	134.60		48.4-49.4' (2:43) 49.4-50.4' (2:45) 100% Recovery	
										50.40	
										Bottom of Exploration at 50.40 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: Durgin Bridge #3976 Carrying King Road over the Sabattus River
Location: Lisbon, Maine

Boring No.: BB-LSS-102
PIN: 15100.00

Driller:	MaineDOT	**Elevation (ft.)**	187.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/08; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+12.9, 6.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	15.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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							SSA	186.65		PAVEMENT. 0.35	
	1D	24/19	1.00 - 3.00	7/9/7/7	16	21				Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#175859 A-1-b, SM WC=5.5%
								183.00		4.00	
5	2D	24/20	5.00 - 7.00	2/2/2/7	4	5				Brown, moist, loose, silty fine to coarse SAND, trace gravel, trace organics. (Fill)	G#175860 A-4, SM WC=20.0%
								177.50		9.50	
10	3D	24/14	10.00 - 12.00	5/12/21/7	33	42	53			Dark brown, wet, very stiff, organic SILT, trace fine sand. (Alluvium? Glaciomarine?) Note, SPT blow count reflects wood interference. Wood layer from 10.7-11.4' bgs.	
							33				
							40	174.00		13.00	
							72				
							44				
15	4D	24/6	15.00 - 17.00	3/3/4/4	7	9	39			Grey, wet, loose, silty fine SAND, trace gravel, wood. (Glaciomarine)	
							46				
							50				
							56	168.50		18.50	
							40				
20	5D	24/11	20.00 - 22.00	3/4/3/4	7	9	49			Grey, wet, stiff, SILT, some fine sand, trace gravel. (Glaciomarine)	G#175861 A-4, ML WC=29.9%
							48				
							46				
							43				
25							41				

Remarks:

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* 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:	Durgin Bridge #3976 Carrying King Road over the Sabattus River	**Location:**	Lisbon, Maine	**Boring No.:**	BB-LSS-102
				PIN:	15100.00
Driller:	MaineDOT	**Elevation (ft.)**	187.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/08; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+12.9, 6.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	15.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.00 - 27.00	6/5/4/4	9	12	41	162.00		25.00 Brown, wet, medium dense, fine SAND, some silt. (Glaciomarine)	G#175862 A-2-4, SM WC=30.1%
							37				
							40				
							38				
							37				
30	7D	24/18	30.00 - 32.00	2/2/3/3	5	6	47	157.00		30.00 Grey, wet, loose, fine to medium SAND, trace silt. (Glaciomarine)	G#175863 A-3, SP-SM WC=23.2%
							46				
							47				
							46				
							47				
35	8D	24/16	35.00 - 37.00	2/2/2/13	4	5	63			Similar to above.	
							69				
							58				
							47	148.50		38.50	
							47				
40	9D	24/14	40.00 - 42.00	WOR/WOR/3/3	3	4	65			Grey, wet, very loose, fine to medium SAND, trace gravel, little silt. (Glaciomarine)	G#175864 A-2-4, SM WC=24.9%
							70			a100 blows for 0.4'.	
	R1	60/60	42.40 - 47.40	RQD = 100%			a100 NQ-2	144.60		42.40 Top of Bedrock at Elev. 144.6'. Bedrock: White and grey banded, fine to medium grained, biotite, quartz, amphibole, amazonite GNEISS, moderately hard, fresh, fractures from horizontal to 20 degrees above horizontal, tight to open, with minor silt in-filling. Joints appear to follow the biotite cleavage. [Vassalboro Formation]	
45										R1:Core Times (min:sec) 42.4-43.4' (2:27) 43.4-44.4' (2:42) 44.4-45.4' (2:08) 45.4-46.4' (2:42) 46.4-47.4' (4:02) 100% Recovery	
	R2	60/60	47.40 - 52.40	RQD = 94%							
50										R2:Core Times (min:sec)	

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: Durgin Bridge #3976 Carrying King Road over the Sabattus River Location: Lisbon, Maine	Boring No.: BB-LSS-102 PIN: 15100.00

Driller:	MaineDOT	**Elevation (ft.)**	187.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/08; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+12.9, 6.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	15.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										47.4-48.4' (4:36) 48.4-49.4' (2:52) 49.4-50.4' (2:35) 50.4-51.4' (2:46) 51.4-52.4' (3:08) 100% Recovery	
								134.60		52.40 **Bottom of Exploration at 52.40 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.

Boring No.: BB-LSS-102

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Lisbon Project Number: 15100.00

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-LSS-101, 1D	7+05.2	6.5 Rt.	1.0-3.0	175851	1	5.9			SM	A-1-b	II
BB-LSS-101, 2D	7+05.2	6.5 Rt.	5.0-7.0	175852	1	20.5			SM	A-2-4	II
BB-LSS-101, 3D	7+05.2	6.5 Rt.	10.0-12.0	175853	1	27.3			GM	A-1-b	I
BB-LSS-101, 4D	7+05.2	6.5 Rt.	15.0-17.0	175854	1	28.8			SM	A-4	III
BB-LSS-101, 5D	7+05.2	6.5 Rt.	20.0-22.0	175855	2	26.8			SM	A-4	III
BB-LSS-101, 6D	7+05.2	6.5 Rt.	25.0-27.0	175856	2	20.3			SP-SM	A-3	0
BB-LSS-101, 7D	7+05.2	6.5 Rt.	30.0-32.0	175857	2	20.4			SP	A-3	0
BB-LSS-101, 8D	7+05.2	6.5 Rt.	35.0-37.0	175858	2	18.5			SM	A-2-4	II
BB-LSS-102, 1D	6+12.9	6.1 Lt.	1.0-3.0	175859	3	5.5			SM	A-1-b	II
BB-LSS-102, 2D	6+12.9	6.1 Lt.	5.0-7.0	175860	3	20.0			SM	A-4	III
BB-LSS-102, 5D	6+12.9	6.1 Lt.	20.0-22.0	175861	3	29.9			ML	A-4	IV
BB-LSS-102, 6D	6+12.9	6.1 Lt.	25.0-27.0	175862	3	30.1			SM	A-2-4	II
BB-LSS-102, 7D	6+12.9	6.1 Lt.	30.0-32.0	175863	3	23.2			SP-SM	A-3	0
BB-LSS-102, 9D	6+12.9	6.1 Lt.	40.0-42.0	175864	3	24.9			SM	A-2-4	II
BB-LSS-101, 3D Loss on Ignition (T 267) Loss 5.5%, H2O 39.0%											

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

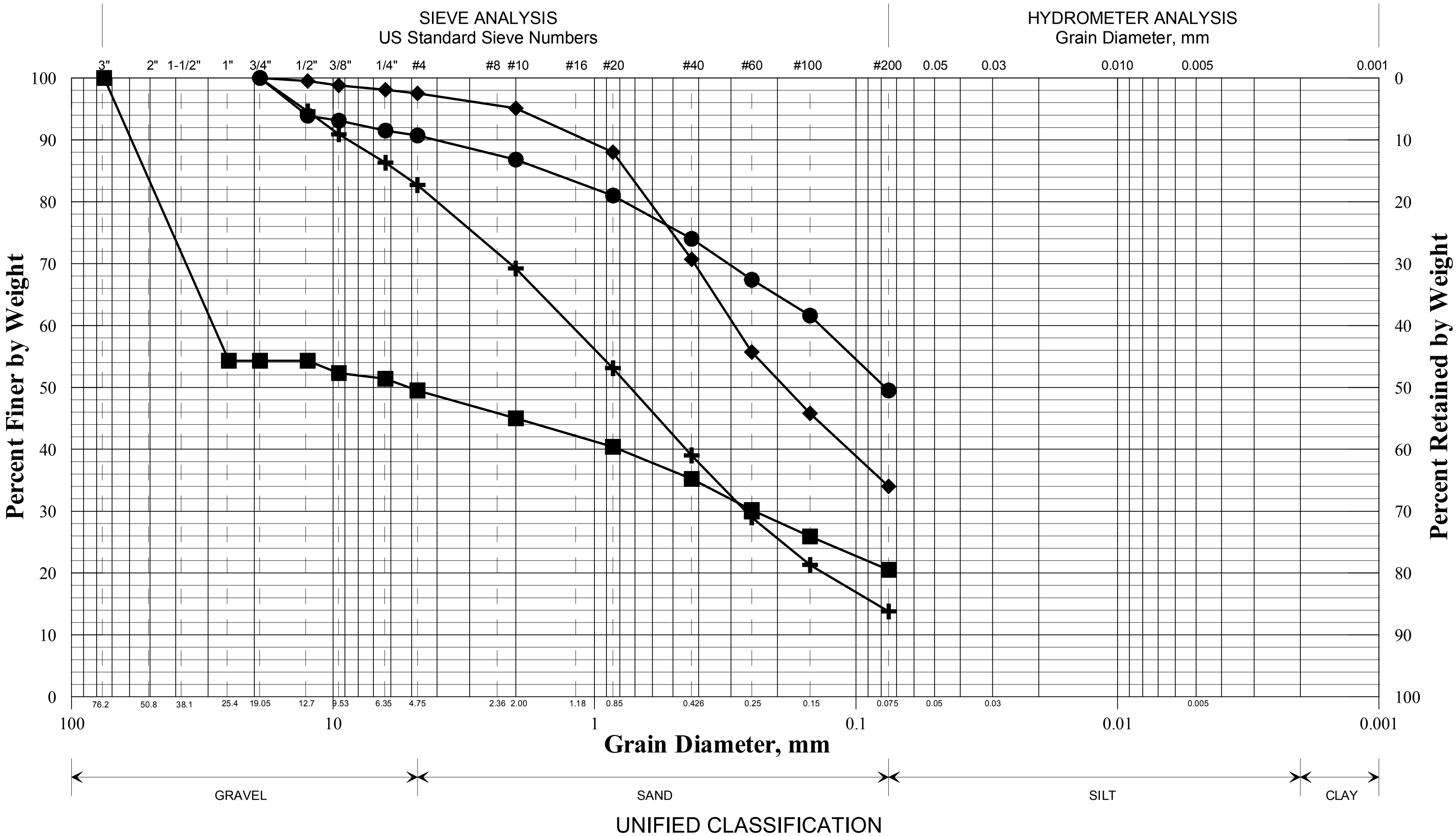
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-LSS-101/1D	7+05.2	6.5 RT	1.0-3.0	SAND, little gravel, little silt.	5.9			
◆	BB-LSS-101/2D	7+05.2	6.5 RT	5.0-7.0	SAND, some silt, trace gravel.	20.5			
■	BB-LSS-101/3D	7+05.2	6.5 RT	10.0-12.0	GRAVEL, some sand, some silt.	27.3			
●	BB-LSS-101/4D	7+05.2	6.5 RT	15.0-17.0	Sandy SILT, trace gravel.	28.8			
▲									
✖									

PIN
015100.00
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Lisbon
Reported by/Date
WHITE, TERRY A 2/2/2009

SHEET 1

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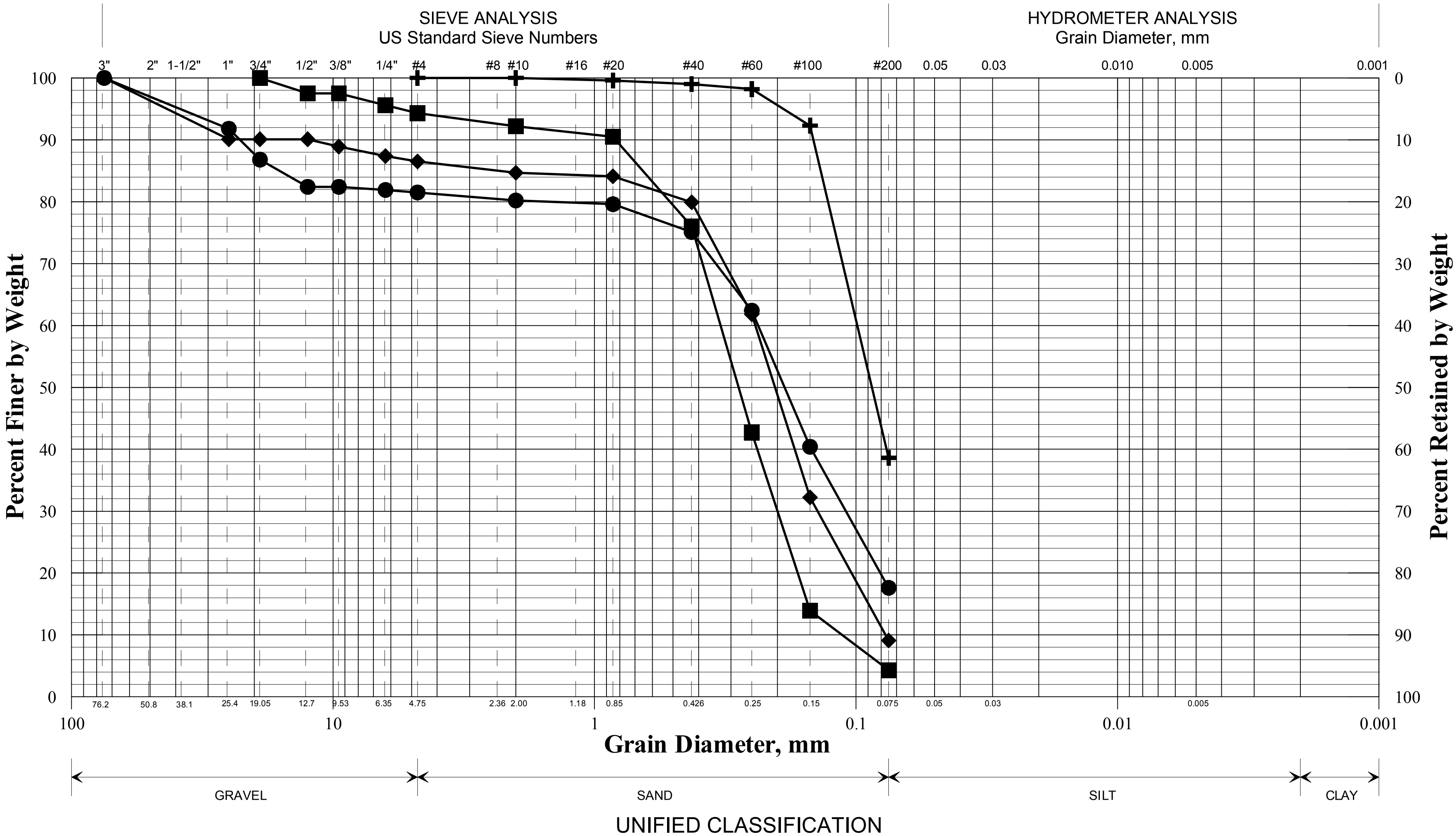
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LSS-101/5D	7+05.2	6.5 RT	20.0-22.0	Silty SAND.	26.8			
◆	BB-LSS-101/6D	7+05.2	6.5 RT	25.0-27.0	SAND, little gravel, trace silt.	20.3			
■	BB-LSS-101/7D	7+05.2	6.5 RT	30.0-32.0	SAND, trace gravel, trace silt.	20.4			
●	BB-LSS-101/8D	7+05.2	6.5 RT	35.0-37.0	SAND, little gravel, little silt.	18.5			
▲									
✖									

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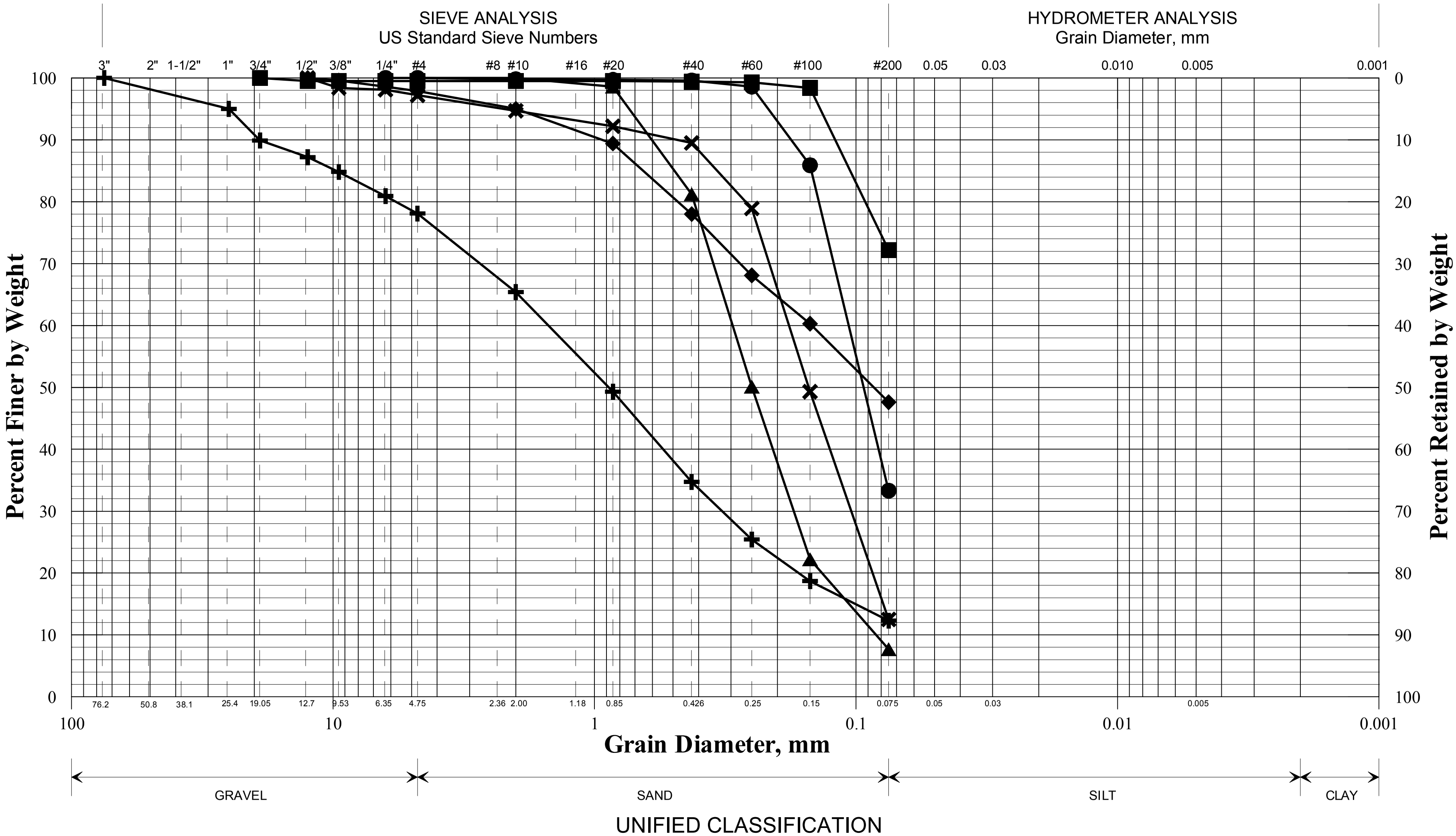
Lisbon
Reported by/Date
WHITE, TERRY A 2/2/2009

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LSS-102/1D	6+12.9	6.1 LT	1.0-3.0	SAND, some gravel, little silt.	5.5			
◆	BB-LSS-102/2D	6+12.9	6.1 LT	5.0-7.0	Silty SAND, trace gravel.	20.0			
■	BB-LSS-102/5D	6+12.9	6.1 LT	20.0-22.0	SILT, some sand, trace gravel.	29.9			
●	BB-LSS-102/6D	6+12.9	6.1 LT	25.0-27.0	SAND, some silt.	30.1			
▲	BB-LSS-102/7D	6+12.9	6.1 LT	30.0-32.0	SAND, trace silt.	23.2			
✖	BB-LSS-102/9D	6+12.9	6.1 LT	40.0-42.0	SAND, little silt, trace gravel.	24.9			

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SHEET 3

Appendix C

Calculations

FROST PROTECTION:

Reference: MaineDOT Bridge Design Guide, Design Freezing Index (DFI) Map and Depth of Frost Penetration Table 5-1.

Lisbon Maine
DFI = 1440 degree-days
Site has Coarse-Grained Soils and Project will Raise Grades.
Use Coarse-Grained for design With typical W_n = 20% in subgrade fills. Use W_n = 10%

From the 2003 Bridge Design Guide Table 5-1:

$$Frost\_depth := [0.4\cdot(67.9 - 65.5) + 65.5]in$$

$$Frost\_depth = 66.46\cdot in$$

$$Frost\_depth = 5.54\cdot ft$$

Use 5.5 feet

INTEGRAL ABUTMENT DRIVEN H-PILES:

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

1. STRUCTURAL AXIAL RESISTANCE OF INDIVIDUAL H-PILES

STRENGTH LIMIT STATE:

Look at the following piles:

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

Note: All matrices are set up in this order

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

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

Yield Strength: $F_y := 50ksi$

Nominal Compressive Resistance:

Nominal Compressive Resistance: $P_n = 0.66^{\lambda} * F_y * A_s$ eq. 6.9.4.1-1 pg. 6-73

Where λ = normalized column slenderness factor

$\lambda = (K\ell/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3 pg. 6-74

$\lambda := 0$ Where the unbraced length ℓ is 0

So: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

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

Factored Compressive Resistance:

Factor for piles in compression under good driving conditions:

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance for Strength Limit State:

$P_f = \phi_c * P_n$ eq. 6.9.2.1-1 pg. 6-71

$P_f := \phi_c \cdot P_n$

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

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

Strength Limit State
Factored Compressive Resistance

SERVICE/EXTREME LIMIT STATES:

Nominal Compressive Resistance:

Nominal Compressive Resistance: $P_n = 0.66^{\lambda} * F_y * A_s$ eq. 6.9.4.1-1 pg. 6-73

Where λ = normalized column slenderness factor

$\lambda = (K\ell / r_s \pi)2 * F_y / E$ eq. 6.9.4.1-3 pg. 6-74

$\lambda := 0$ Where the unbraced length ℓ is 0

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

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

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

Factored Compressive Resistance:

Resistance Factors for Service and Extreme Limit States:

From Articles 105.5.1 and 105.5.3 $\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

$P_f = \phi * P_n$ eq. 6.9.2.1-1 pg. 6-71

$P_f := \phi \cdot P_n$

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

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

Service and Extreme Limit State
Factored Compressive Resistance

2. GEOTECHNICAL AXIAL RESISTANCE OF INDIVIDUAL H-PILES FROM STATIC ANALYSIS

Assume piles will be end bearing on bedrock driven through overlying granular fill and till.

Bedrock Type: Granite and Gneiss
RQD ranges from 33% to 67% (Granite) and 94% to 100% (Gneiss)
ϕ = 27 to 34 deg (Gneiss) and 34 to 40 deg (Granite), Tomlinson 4th Ed. pg. 139
Use ϕ = 27 to 34 deg for design

Look at the following piles:

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

Note: All matrices are set up in this order

H-Pile Steel Area: Pile Depth: Pile Width:

$$A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.05 \\ 12.22 \\ 14.59 \\ 14.70 \\ 14.89 \end{pmatrix} \cdot in$$

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

Calculate pile box area:

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.95 \\ 148.23 \\ 198.57 \\ 203.30 \\ 211.59 \end{pmatrix} \cdot in^2$$

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

End bearing resistance of piles on bedrock:

REF: "Pile Design and Construction Practice," Tomlinson, 4th Ed., page 139.

Average compressive strength of rock core from
AASHTO Standard Specification for Highway Bridges, 17th Ed., 2002
Table 4.4.8.1.2B pg 64:

q_{uc} for Granite and Gneiss: Granite - 2,100 to 49,000 psi; Gneiss - 3,500 to 45,000 psi
Although some RQD values are low, rock jointing at this sight is tight with generally good core recovery indicating relatively intact rock

Assume $q_{uc} := 20000 \cdot psi$

Correct for wedge failure under strip footing:

for Nc multiply cNc by 1.25 - square piles
1.2 for circular piles

for Nγ multiply γNγ by 0.8 - square piles
0.7 for circular

For RQD 0-70 %
qc = 0.33 x Quc
c = 0.1 x Quc
ϕ = 30 deg

Tomlinson, PG. 139

For RQD 70-100 %
qc = 0.33 to 0.88 x Quc
c = 0.1 x Quc
ϕ = 30 to 60 deg

Max RQD = 67% at Abutment No.2, Use for Design. Therefore:
ϕ = 30
c = 0.1 x Quc Assume pile penetrates 1 inch into bedrock
qc = 0.33 x Quc

$Q_{uc} := q_{uc}$ $c := 0.1Q_{uc}$ $c = 2000 \cdot psi$

$D := 1in$

$B_{min} := 12in$

$\gamma := 145pcf$ $q_c := 0.33 \cdot Q_{uc}$ $q_c = 6600 \cdot psi$ Bedrock Unit Wt: Fang, p.95

$N_c := 13.86$ $N_q := 9.0$ $N_\gamma := 13.86$ Tomlinson Figure 4.35, p. 140

$$q_{ub} := 1.25 \cdot c \cdot N_c + \left(\gamma \cdot B_{min} \cdot \frac{N_\gamma}{2}\right) \cdot 0.8 + \gamma \cdot D \cdot N_q$$

$q_{ub} = 34.66 \cdot ksi$

Nominal Geotechnical Tip Resistance:

$R_{p\_nom} := q_{ub} \cdot A_s$

$$R_{p\_nom} = \begin{pmatrix} 537 \\ 756 \\ 742 \\ 905 \\ 1192 \end{pmatrix} \cdot kip$$

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

Factored Geotechnical Tip Resistance:

Resistance factor for Single Pile in Axial Compression End Bearing in Rock:

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1, pg. 10-38/39

For Lisbon Durgin Bridge, only 4 piles per abutment, so need to reduce ϕstat by 20%

$$\phi_{stat80\%} := \phi_{stat} \cdot 0.8$$

$$\phi_{stat80\%} = 0.36$$

$$R_{tipf} := R_{p\_nom} \cdot \phi_{stat80\%}$$

$$R_{tipf} = \begin{pmatrix} 193 \\ 272 \\ 267 \\ 326 \\ 429 \end{pmatrix} \cdot kip$$

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

Axial Geotechnical Skin Resistance of Single H-Piles:

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

DRIVEN 1.0

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\15100.DVN
Project Name: Lisbon Durgin Bridge
Project Date: 05/05/2009
Project Client: MaineDOT
Computed By: MJM
Project Manager: Jim Wentworth

PILE INFORMATION

Pile Type: H Pile - HP12X53
Top of Pile: 10.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	41.00 ft	10.00%	115.00 pcf	30.0/0.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
10.00 ft	0.00 Kips	0.22 Kips	0.22 Kips
10.01 ft	0.01 Kips	0.22 Kips	0.23 Kips
19.01 ft	14.94 Kips	0.31 Kips	15.25 Kips
28.01 ft	34.97 Kips	0.30 Kips	35.27 Kips
37.01 ft	60.09 Kips	0.29 Kips	60.38 Kips
40.99 ft	72.82 Kips	0.29 Kips	73.10 Kips

HP 12 x 74

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
10.00 ft	0.00 Kips	0.31 Kips	0.31 Kips
10.01 ft	0.02 Kips	0.31 Kips	0.32 Kips
19.01 ft	17.15 Kips	0.43 Kips	17.58 Kips
28.01 ft	40.13 Kips	0.43 Kips	40.56 Kips
37.01 ft	68.96 Kips	0.42 Kips	69.37 Kips
40.99 ft	83.57 Kips	0.41 Kips	83.98 Kips

HP 14 x 73

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
10.00 ft	0.00 Kips	0.30 Kips	0.30 Kips
10.01 ft	0.02 Kips	0.30 Kips	0.32 Kips
19.01 ft	19.73 Kips	0.42 Kips	20.15 Kips
28.01 ft	46.16 Kips	0.50 Kips	46.66 Kips
37.01 ft	79.32 Kips	0.47 Kips	79.79 Kips
40.99 ft	96.13 Kips	0.47 Kips	96.60 Kips

HP 14 x 89

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
10.00 ft	0.00 Kips	0.37 Kips	0.37 Kips
10.01 ft	0.02 Kips	0.37 Kips	0.39 Kips
19.01 ft	21.45 Kips	0.52 Kips	21.96 Kips
28.01 ft	50.18 Kips	0.61 Kips	50.79 Kips
37.01 ft	86.23 Kips	0.57 Kips	86.80 Kips
40.99 ft	104.51 Kips	0.57 Kips	105.08 Kips

HP 14 x 117

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
10.00 ft	0.00 Kips	0.48 Kips	0.48 Kips
10.01 ft	0.02 Kips	0.48 Kips	0.50 Kips
19.01 ft	24.02 Kips	0.68 Kips	24.70 Kips
28.01 ft	56.21 Kips	0.81 Kips	57.02 Kips
37.01 ft	96.59 Kips	0.76 Kips	97.35 Kips
40.99 ft	117.06 Kips	0.76 Kips	117.82 Kips

$$R_{skin} := \begin{pmatrix} 73 \\ 84 \\ 96 \\ 105 \\ 117 \end{pmatrix} kip$$

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

$$R_{sf} := R_{skin} \cdot \phi_{stat80\%} \qquad R_{sf} = \begin{pmatrix} 26 \\ 30 \\ 35 \\ 38 \\ 42 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Strength Limit State Factored Geotechnical Resistance, R_{gf}:

$$R_{gf} := R_{tipf} + R_{sf} \qquad R_{gf} = \begin{pmatrix} 220 \\ 302 \\ 302 \\ 363 \\ 471 \end{pmatrix} \cdot kip$$

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

Strength Limit State **Factored** Geotechnical Resistance, R_{gf}

SERVICE/EXTREME LIMIT STATES:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0
LRFD 10.5.5.1, pg. 10-30 and 10.5.5.3, pg. 10-43

$$\phi := 1.0$$

Nominal Geotechnical Tip Resistance, R_{p_nom}, as before:

$$R_{p\_nom} := q_{ub} \cdot A_s \qquad R_{p\_nom} = \begin{pmatrix} 537 \\ 756 \\ 742 \\ 905 \\ 1192 \end{pmatrix} \cdot kip$$

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

Skin Friction:

$$R_{skin} := \begin{pmatrix} 73 \\ 84 \\ 96 \\ 105 \\ 117 \end{pmatrix} kip$$

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

Service/Extreme Limit State Factored Geotechnical Resistance, R_g:

$$R_g := \left(R_{p\_nom} + R_{skin}\right) \cdot \phi \qquad R_g = \begin{pmatrix} 610 \\ 840 \\ 838 \\ 1010 \\ 1309 \end{pmatrix} \cdot kip$$

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

Service/Extreme Limit State **Factored** Geotechnical Resistance, R_g

3. GEOTECHNICAL AXIAL RESISTANCE OF INDIVIDUAL H-PILES FROM WAVE EQUATION ANALYSIS

Ref. LRFD Article 10.7.8 pg. 10-121

$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8.1)

$f_y := 50ksi$ yield strength of steel

$\phi_{da} := 1.0$ Resistance factor from LRFD Table 10.5.5.2.3-1 pg. 10-38/39
Pile Drivability Analysis, Steel Piles (Refers to Article 6.5.4.2, p. 6-28: $\phi da = 1.0$)

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

Compute resistance that must be achieved in a drivability analysis:

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

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

$\phi_{dyn} := 0.65$

Table 10.5.5.2.3-1 requires no less than 3 to 4 piles dynamically tested for a site with low to medium site variability. Additionally there are only 4 piles per substructure at this site. There will probably be only 2 piles tested per bridge - one per abutment will be requested. Therefore, reduce ϕ_{dyn} by 20%.

$\phi_{dyn80\%} := 0.65 \cdot 0.8$ $\phi_{dyn80\%} = 0.52$

Use GRLWeap to perform drivability analysis.

Limit Driving Stress to 45 ksi

Limit Blow Count to less than 15 bpi

HP 12 x 53

State of Maine Dept. Of Transportation — 05-May-2009

15100 LDB Drivability w Delmag D 19-42 — GRLWEAP (TM) Version 2003

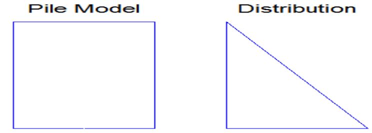
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
410.0	44.47	3.24	5.0	8.50	18.09
411.0	44.53	3.26	5.1	8.51	18.07
412.0	44.62	3.28	5.1	8.52	18.12
413.0	44.69	3.29	5.1	8.53	18.17
414.0	44.75	3.30	5.1	8.54	18.16
415.0	44.84	3.32	5.1	8.56	18.19
416.0	44.94	3.34	5.1	8.57	18.23
417.0	45.00	3.35	5.2	8.58	18.23
418.0	45.06	3.37	5.2	8.59	18.27
419.0	45.14	3.38	5.2	8.60	18.26

DELMAG D 19-42

Efficiency	0.800
Helmet	4.00 kips
Hammer Cushion	39129 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	30.00 ft
Pile Penetration	30.00 ft
Pile Top Area	15.50 in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

HP 12 x 74

State of Maine Dept. Of Transportation — 05-May-2009

15100 LDB Drivability w Delmag D 19-42 — GRLWEAP (TM) Version 2003

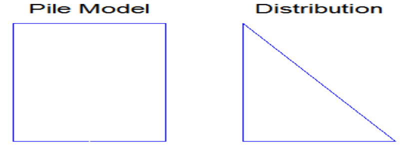
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
680.0	44.96	3.71	12.0	10.08	19.65
681.0	45.01	3.71	12.1	10.09	19.67
682.0	45.04	3.72	12.1	10.09	19.68
683.0	45.08	3.72	12.2	10.10	19.70
684.0	45.09	3.72	12.2	10.11	19.71
685.0	45.15	3.73	12.3	10.11	19.72
686.0	45.20	3.72	12.3	10.12	19.74
687.0	45.24	3.73	12.4	10.13	19.75
688.0	45.27	3.73	12.4	10.13	19.76
689.0	45.29	3.73	12.4	10.14	19.78

DELMAG D 19-42

Efficiency	0.800
Helmet	4.00 kips
Hammer Cushion	39129 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	30.00 ft
Pile Penetration	30.00 ft
Pile Top Area	21.80 in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

HP 14 x 73

State of Maine Dept. Of Transportation — 05-May-2009

15100 LDB Drivability w Delmag D 19-42 — GRLWEAP (TM) Version 2003

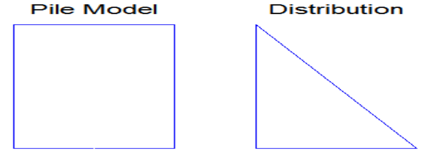
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
660.0	44.90	3.75	11.2	9.97	19.54
661.0	44.91	3.75	11.3	9.97	19.51
662.0	44.97	3.75	11.4	9.98	19.53
663.0	45.02	3.75	11.4	9.98	19.54
664.0	45.05	3.76	11.5	9.99	19.56
665.0	45.08	3.76	11.5	10.00	19.58
666.0	45.12	3.77	11.5	10.01	19.59
667.0	45.17	3.77	11.6	10.01	19.61
668.0	45.23	3.77	11.6	10.02	19.62
669.0	45.25	3.78	11.7	10.03	19.64

DELMAG D 19-42

Efficiency	0.800
Helmet	4.00 kips
Hammer Cushion	39129 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	30.00 ft
Pile Penetration	30.00 ft
Pile Top Area	21.40 in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

HP 14 x 89

State of Maine Dept. Of Transportation
15100 LDB Drivability w Delmag D 19-42

05-May-2009
GRLWEAP (TM) Version 2003

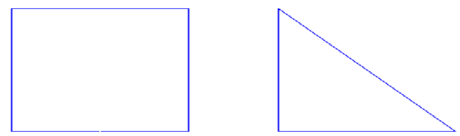
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
765.0	40.80	5.55	14.7	10.32	19.44
766.0	40.83	5.56	14.7	10.32	19.44
767.0	40.86	5.56	14.8	10.33	19.44
768.0	40.90	5.57	14.9	10.33	19.45
769.0	40.92	5.58	14.9	10.34	19.45
770.0	40.95	5.59	15.0	10.35	19.46
771.0	41.00	5.60	15.0	10.35	19.51
772.0	41.03	5.62	15.1	10.36	19.51
773.0	41.06	5.62	15.2	10.36	19.51
774.0	41.09	5.63	15.2	10.37	19.52

DELMAG D 19-42

Efficiency	0.800	
Helmet	4.00	kips
Hammer Cushion	39129	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	30.00	ft
Pile Penetration	30.00	ft
Pile Top Area	26.10	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

HP 14 x 117

State of Maine Dept. Of Transportation
15100 LDB Drivability w Delmag D 19-42

05-May-2009
GRLWEAP (TM) Version 2003

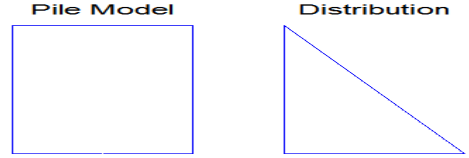
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
805.0	35.72	3.74	14.8	10.02	18.41
807.0	35.83	3.77	14.8	10.03	18.45
809.0	35.85	3.77	14.9	10.04	18.48
811.0	35.90	3.78	15.0	10.04	18.52
813.0	35.92	3.78	15.1	10.05	18.51
815.0	35.99	3.79	15.2	10.07	18.55
817.0	36.05	3.80	15.3	10.08	18.58
819.0	36.07	3.81	15.4	10.08	18.57
821.0	36.13	3.82	15.5	10.10	18.61
823.0	36.16	3.85	15.6	10.11	18.63

DELMAG D 19-42

Efficiency	0.800	
Helmet	4.00	kips
Hammer Cushion	39129	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	30.00	ft
Pile Penetration	30.00	ft
Pile Top Area	34.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

R_{driv} from GRLWeap Analysis:

$$R_{driv} := \begin{pmatrix} 417 \\ 681 \\ 663 \\ 770 \\ 811 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Strength Limit State Factored Geotechnical Resistance:

$R_{driv\_factored} := R_{driv} \cdot \phi_{dyn80\%}$

$R_{driv\_factored} = \begin{pmatrix} 217 \\ 354 \\ 345 \\ 400 \\ 422 \end{pmatrix} \cdot kip$

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

Strength Limit State **Factored** Drivability Resistance

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit State:

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$
LRFD 10.5.5.1, pg. 10-30 and 10.5.5.3, pg. 10-43

$\phi_{serv\_ext} := 1.0$

$R_{driv\_serv\_ext} := R_{driv} \cdot \phi_{serv\_ext}$

$R_{driv\_serv\_ext} = \begin{pmatrix} 417 \\ 681 \\ 663 \\ 770 \\ 811 \end{pmatrix} \cdot kip$

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

Service Limit State **Factored** Drivability Resistance

Factored Resistances from Static Analysis appear conservative. Recommend using Factored Resistances from Drivability Analysis.

ABUTMENT AND WINGWALL PASSIVE AND ACTIVE EARTH PRESSURES:

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

Angle of back face of wall: $\alpha := 90deg$

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

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

For walls, $\delta = \beta$ $\delta := \beta$

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

$K_a = 0.31$

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

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

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

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

$K_a = 0.31$

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

For gravity walls , semi-gravity walls, prefabricated modular walls, and cantilever walls and abutments with short heels where wall and backfill interface friction is considered, use Coulomb Theory

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

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1, pg. 3-74, δ ranges from 17 to 22 $\delta := 20\text{deg}$

Angle of backfill from horizontal: $\beta := 0\text{deg}$

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

$K_p = 6.89$

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

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

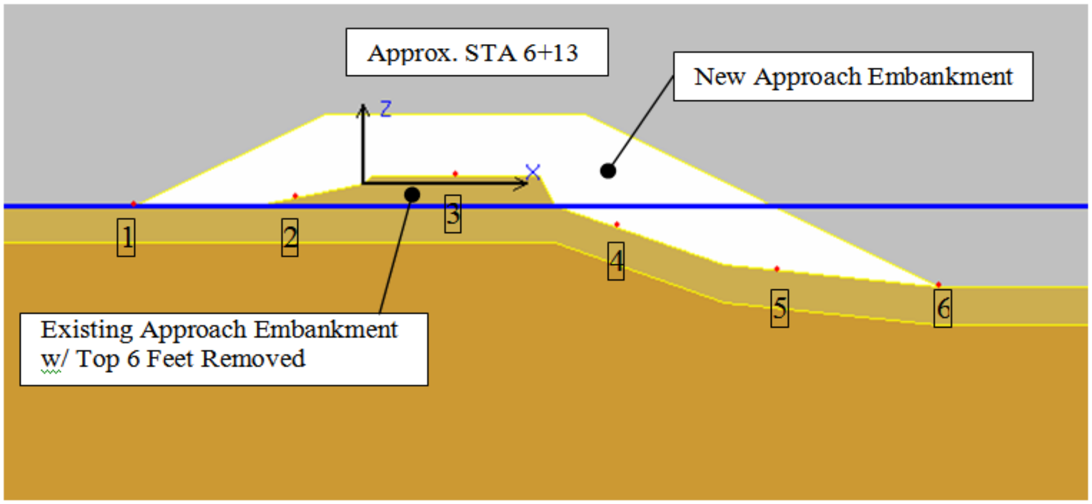
Slope angle of backfill soil from horizontal: $\beta := 0\text{deg}$

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

$K_{p\_rank} = 3.25$

APPROACH FILL SETTLEMENT:

Points 1 Through 6 Where Settlement Was Calculated Shown Below



FoSSA -- Foundation Stress & Settlement Analysis
Present Date/Time: Tue May 12 16:09:34 2009

Lisbon, Durgin Bridge
C:\FoSSA\15100 Lisbon Durgin Brg.F2S

Lisbon, Durgin Bridge

PROJECT IDENTIFICATION

Title: Lisbon, Durgin Bridge
Project Number: PIN 15100 -
Client:
Designer: Mike Moreau, PE
Station Number: 6+13

Description: Approximate STA 6+13 Abutment Location - Largest Fill

Company's information:

Name: MaineDOT
Street: 16 State House Station
Augusta, ME 04333-0016
Telephone #:
Fax #:
E-Mail:

Original file path and name: C:\FoSSA\15100 Lisbon Durgin Brg.F2S
Original date and time of creating this file: Tue May 12 12:23:08 2009

GEOMETRY: Analysis of a 2D geometry

FoSSA -- Foundation Stress & Settlement Analysis — Lisbon, Durgin Bridge

Present Date/Time: Tue May 12 16:15:21 2009 — C:\FoSSA\15100 Lisbon Durgin Brg.F2S

TABULTAED GEOMETRY INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z): (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	310.00	325.00	Silty Sand with Trace Organics
	2	326.00	325.00	
	3	340.00	328.00	
	4	341.00	334.80	
	5	363.00	334.80	
	6	365.00	325.00	
	7	387.00	317.00	
	8	415.00	314.00	
2	1	310.00	320.00	Silty Sand
	2	365.00	320.00	
	3	387.00	312.00	
	4	415.00	309.00	

FoSSA -- Foundation Stress & Settlement Analysis — Lisbon, Durgin Bridge

Present Date/Time: Wed May 13 08:35:31 2009 — C:\FoSSA\15100 Lisbon Durgin Brg.F2S

IMMEDIATE SETTLEMENT, Si

Node #	Settlement along section: X [ft.]	Y [ft.]	Layer #	Young's Modulus, E [lb/ft²]	Poisson's Ratio, μ	Si(k) [ft.]	Z initial [ft.]	Z final [ft.]	Total Settlement
1	310.00	0.00	1	200000	0.4000	-0.0019	325.00	325.00	0.02 in
			2	300000	0.3000	0.0038			
2	331.00	0.00	1	200000	0.4000	0.0128	326.07	326.01	0.69 in
			2	300000	0.3000	0.0445			
3	352.00	0.00	1	200000	0.4000	0.0140	329.00	328.94	0.72 in
			2	300000	0.3000	0.0464			
4	373.00	0.00	1	200000	0.4000	0.0186	322.09	322.01	0.93 in
			2	300000	0.3000	0.0588			
5	394.00	0.00	1	200000	0.4000	0.0126	316.25	316.21	0.46 in
			2	300000	0.3000	0.0256			
6	415.00	0.00	1	200000	0.4000	-0.0018	314.00	314.00	0.01 in
			2	300000	0.3000	0.0027			