MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MAIN STREET BRIDGE
OVER EAST BRANCH SEBASTICOOK RIVER
NEWPORT, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Main Street Bridge which carries State Route 2/100 over East Branch Sebasticook River in Newport, Maine. Main Street Bridge was built in 1930 and consists of two independent bridges (east bridge and west bridge) separated by an earth causeway. The proposed replacement east bridge will be a 120-foot single span, fully integral bridge. The proposed structure will have a centerline approximately matching the existing bridge centerline. The roadway profile will be raised approximately 2 feet along the west bridge approach. The shortened span of the replacement bridge will require filling in that portion of the riverbed between the existing east abutment and the third river pier. The west bridge will be removed and the canal it spans filled in. The following design recommendations are discussed in detail in this report.

Integral Pile Design - The piles should be end bearing and driven to the required resistance on, or within, bedrock. Piles may be HP 12x53, 14x73, 14x89, or 14x117 depending on the factored design pile loads. Piles should be 50 ksi, Grade A572 steel. Driven piles should be fitted with driving points to protect the tips, and improve penetration

H-piles shall be designed for all relevant strength, service and extreme limit state load groups. The structural resistance check should include checking axial, lateral and flexural resistance.

The maximum factored axial pile load should not exceed the calculated factored drivability pile resistances provided in this report. An L-Pile® analysis is recommended to evaluate the combined axial compression and flexure, with factored axial loads, moments and pile head displacements applied. As the proposed integral H-piles will be modeled as fully fixed at the pile head, the resistance of the piles should be evaluated for structural compliance with the interaction equation.

Driven Pile Quality Control - The contractor is required to perform a wave equation analysis of the proposed pile-hammer system. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. The ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, φ_{dyn}, of 0.65. The maximum factored pile load and the resistance factor should be shown on the plans.

Integral Stub Abutment Design - Integral abutment sections shall be designed for all relevant strength, service and extreme limit states. Integral abutment sections and wingwall sections that are integral with the abutment shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure calculated using a passive pressure coefficient, K_p, of 7.33, calculated using Coulomb Theory. A load factor for passive earth pressure, γ_{EH}, of 1.5 should be applied.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Developing full passive pressure requires displacements of the abutment or wingwall on the order of 2 to 5 percent of the abutment or wingwall height. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction. In general, wall friction acts downward in the passive case, and increase passive pressure as considered in the Coulomb Theory. All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. The approach slab should be positively attached to the integral abutment.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required for the abutments and wingwalls if an approach slab is not specified. If a structural approach slab is specified, some reduction of the surcharge loads is permitted.

Scour and Riprap - For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments and wingwalls should be armored with 3 feet of riprap. Riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1-foot thick layer of bedding material.

Settlement – There are three (3) areas where settlement is of concern on this project. The existing west bridge spans a canal which will be filled in with 13 feet of fill. The grade of the roadway approach to Abutment No. 1 will be raised approximately 2 feet. The portion of streambed below the fourth span of the east bridge will be filled in with 13 feet of fill. Settlements due to elastic compression of the foundation soils in these areas of concern were calculated and are provided in Table 1 below. Settlement of the granular foundation soils will be elastic and occur primarily during construction. Any settlement of the bridge abutments will be due to elastic settlement of the bedrock or piles, which is assumed to occur during construction and will be negligible.

Location	Estimated Elastic Settlement (inches)
West bridge over canal (approx. Sta. 6+00) with 13 feet of new fill	1.3
Abutment No. 1 approach (approx. Sta 7+25) with 2 feet of new fill	0.5
Riverbank between the existing pier 3 and east abutment (approx. Sta. 9+00) with 13 feet of fill	1.5

Table 1. Estimated Settlement

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Frost Protection - Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection. Any foundations placed on granular fill soils should be founded a minimum of 7.0 feet below finished exterior grade for frost protection.

Seismic Design Considerations – Seismic analysis is not required for single-span bridges, regardless of seismic zone, however superstructure connections and bridge seat dimensions shall be satisfied.

Construction Considerations – Construction of the pile foundations and abutments will require soil excavation and bridge substructure removal. Construction activities may require cofferdams and earth support systems. The existing west abutment wingwall and third river pier will obstruct installation of piles. Removal of all or some of the existing substructures will be necessary. The pile foundation area may require placement and compaction of granular fill up to the abutment subgrade level.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Main Street Bridge which carries State Route 2/100 over the East Branch Sebasticook River, in Newport, Maine. This report presents the soils information obtained at the site during the subsurface investigations, foundation recommendations and geotechnical design parameters for replacement bridge foundations.

Main Street Bridge was built in 1930 and consists of two independent bridges (east bridge and west bridge) separated by an earth causeway. The bridges incorporate several split stone with masonry substructures from the steel truss bridges that the 1930 bridge replaced. The shorter west bridge spans a canal that acts as an overflow channel and is a 38-foot single span concrete T-beam superstructure on stone and masonry abutments. The east bridge is a 4-span concrete T-beam superstructure with a total length of 162 feet. Abutment 1 of the east bridge is a stacked stone and masonry abutment on timber cribbing and Abutment 2 of the east bridge is mass concrete with a spread footing on soil. Piers 1 and 3 of the east bridge are mass concrete on spread footings bearing on soil. Pier 2 of the east bridge consists of a pre-1930 stacked stone and masonry pier on timber cribbing, with concrete extension and cap. A concrete dam located immediately downstream was partially removed in 2002.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports for the east bridge indicate substructure distress in areas in the form of large cracks in the pier breastwalls, heavy deterioration of the pier noses, deep scaling and heavy cracking in the east abutment, and mortar missing between the dry laid granite stone of the west abutment. 2008 MaineDOT Bridge Maintenance inspection reports assign the substructure a condition rating of 5 – fair, and the channel protection a rating of 8 – bank protected. The bridge has Bridge Sufficiency Rating of 58.3.

The MaineDOT Bridge Program is currently proposing a replacement structure for the east bridge consisting of a 120-foot single-span welded steel plate girder integral bridge founded on pile-supported integral abutments. The west bridge will be entirely removed and the canal filled in. The proposed replacement structure will have a centerline approximately matching the existing bridge centerline. The roadway profile will be raised approximately 2 feet along the west approach to the bridge, and the shortened span of the replacement bridge will require filling in the fourth span of the existing bridge.

2.0 GEOLOGIC SETTING

Main Street Bridge located on State Routes 2 and 100 in Newport, Maine crosses the East Branch Sebasticook River as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey "Surficial Geology of Pittsfield Quadrangle, Maine, Open-file No. 86-35" (1986) indicates that the surfical soils at the Main Street Bridge site consist of predominantly glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay and stone. The unit was deposited directly by glacial ice, and commonly conforms to the bedrock surface.

According to the Bedrock Geologic Map of Maine, Maine Geologic Survey, 1985, the site is underlain by bedrock of the Vassalboro Formation, which consists of interbedded calcareous sandstone and impure limestone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling three (3) test borings. The borings were terminated with bedrock cores. Test borings BB-NSR-101, BB-NSR-102 and BB-NSR-201 were all drilled through the bridge deck and into the streambed. Test boring BB-NSR-102 was drilled approximately 6 feet west of the proposed Abutment No 2. Test boring BB-NSR-201 was drilled approximately 4 feet west of proposed Abutment No. 1 centerline. Test boring BB-NSR-101 was drilled approximately 40 feet east of the proposed Abutment No. 1 centerline of bearing The boring locations are shown on Sheet 2 - Boring Location Plan, found at the end of this report.

Borings BB-NSR-101 and BB-NSR-102 were drilled on June 3 and 4, 2008 and boring BB-NSR-201 on June 15 and 25, 2009, using the MaineDOT drill rig. The borings were drilled using cased wash boring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance.

The MaineDOT drill rig is equipped with a Central Mine Equipment (CME) automatic hammer. The hammer was calibrated by MaineDOT in August of 2007 and subsequently in February of 2009 and was found to deliver approximately 30 percent, and subsequently in 2009, 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying average energy transfer factors of 0.77 or 0.84 to the raw field N-values. These hammer efficiency factors, 0.77 and 0.84, and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in two borings using an NQ-2 or BX core barrel and the Rock Quality Designation (RQD) of the core was calculated for the NQ cores. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Geotechnical Team Member or a New England Transportation Technical Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by taping to site features after completion of the drilling program.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs, found at the end of this report.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of seven (7) standard grain size analyses, two (2) grain size analyses with hydrometer, nine (9) natural water contents and one (1) Atterberg Limits test. The results of soil laboratory tests are included as Appendix B - Laboratory Data, at the end of this report. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at test borings BB-NSR-101, BB-NSR-102 and BB-NSR-201 generally consisted of river bottom sediments and alluvium underlain by glacial till and metamorphic bedrock. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is shown on Sheet 3 – Interpretive Subsurface Profile, found at the end of this report. The boring logs are provided in Appendix A – Boring Logs. A brief summary description of the strata encountered follows:

5.1 Interbedded River Bottom Sediments and Alluvium

A layer of interbedded river bottom sediments and alluvial soils was encountered in borings. The encountered layer is approximately 3.5 to 9.65 feet thick. The deposit generally consisted of brown and dark brown, damp to wet, silty sand, sandy gravel, gravel and gravelly sand, with minor portions of organic silt and root fibers, and dark brown, wet, sandy organic silt with slight odor. Isolated boulders, cobbles and wood fragments were encountered in BB-NSR-101 and BB–NRS-201.

Corrected SPT N-values in the unit ranged from 6 to greater than 50 blows per foot (bpf), indicating a soil that is loose to very dense in consistency.

Grain size analyses were conducted on two (2) samples from the river bottom sediments and alluvial unit. Grain size analyses resulted in the soil being classified as A-1-a under the AASHTO Soil Classification System and SM and GP-GM under the Unified Soil Classification System. Measured natural water contents of samples tested ranged from approximately 12 to 18 percent.

5.2 Alluvium

An alluvial deposit was encountered below the interbedded river bottom deposits and alluvium deposit. The encountered thickness of the unit was approximately 2.35 to 9 feet thick. The lower alluvial unit consisted of grey, moist to wet, gravelly sand, sand with some gravel, and sandy gravel, with trace to some silt. Occasional cobbles were noted in the alluvium deposit in BB-NSR-201.

Corrected SPT N-values in alluvium ranged from 18 to greater than 50 bpf, indicating that the soil deposit is medium dense to very dense in consistency.

Grain size analyses were conducted on three (3) samples from the alluvial unit. Grain size analyses resulted in the soil being classified as an A-1-a and A-1-b under the AASHTO Soil Classification System and SM, SW-SM and GM under the Unified Soil Classification System. The natural water contents were approximately 11 and 17 percent.

5.3 Glacial Till

Glacial till was encountered underlying the alluvium in the borings. The encountered thickness of the deposit was approximately 6 to 14.7 feet at the boring locations. The glacial till generally consisted of grey to brown, moist to wet, silty sand and sand, some silt, little to some gravel, trace clay, and olive-grey, damp to wet, silty sand and sand, with lesser portions of gravel and clay; and olive-grey, mottled, damp to wet, gravelly silt and sandy silt, some sand, little clay.

Corrected SPT N-values in the glacial till unit were greater than 50 bpf with the exception of one SPT N-value of 32 bpf. This indicates a soil of generally very dense consistency, but some subunits are dense in consistency.

Grain size analyses were conducted on four (4) samples from the glacial till unit. Grain size analyses resulted in the soil being classified as an A-2-4, and A-4 under the AASHTO Soil Classification System and SC-SM, SM and CL-ML under the Unified Soil Classification System. The natural water contents ranged from approximately 9 to 11 percent.

One Atterberg Limits test on a sample from the deposit determined the moisture content was approximately 11 percent and plastic limit was 17. The natural water content did not exceed the liquid limit of 22 or the plastic limit. The calculated value of liquidity index for the soil tested was 11.16, meaning the soil is heavily preconsolidated.

5.4 Bedrock

Bedrock was encountered and cored beginning at a depths ranging from approximately 18 feet below ground surface (bgs) and approximate Elevation 167.60 feet in boring BB-NSR-101 to a depth of approximately 25 feet bgs and approximate Elevation 163.6 feet in boring BB-NSR-102.

The bedrock at the site is identified as grey to dark grey, fine grained, metasedimentary hornfels, moderately hard, moderately weathered to fresh, no foliation to foliated at steep angles, tight, weathered and stained surfaces, with occasional weathered zones, fractured zones and quartz seams. The RQD of the bedrock was determined to range from 26 to 96 percent, correlating to a rock mass quality of very poor to excellent.

Table 2 summarizes approximate top of bedrock elevations at the exploration locations.

Proposed Substructure	Boring	Station	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
Abutment 1	BB-NSR-201	7+47.8	27.2	165.3
none	BB-NSR-101	7+92.6	18.0	167.6
Abutment 2	BB-NSR-102	8+60.9	25.0	163.6

Table 2. Approximate Elevation of Bedrock Surface at Exploration Locations

5.5 Groundwater

The water level in boring BB-NSR-101 was consistent with the river level elevation. The groundwater level in BB-NSR-102 was inferred to be at a depth of approximately 2 feet bgs or approximately Elevation 187 feet. The groundwater level in BB-NSR-201 ranged from approximately 9 to 16 feet bgs. Groundwater levels will fluctuate with seasonal changes, runoff, and adjacent construction activities.

6.0 FOUNDATION ALTERNATIVES

The following foundations were considered for the replacement bridge substructures and evaluated for practicality and effectiveness during preliminary design:

- Full height, cantilever-type concrete abutments supported on pile groups driven to bedrock.
- Integral abutments supported on piles driven to bedrock.

The MaineDOT Bridge Program Preliminary Design Report proposes a replacement bridge consisting of a 120-foot single-span welded steel plate girder integral bridge founded on H-pile supported abutments. This report addresses this selected foundation alternative. The west bridge will be removed and the canal filled in.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

This section provides geotechnical design recommendations for pile-supported integral abutments.

7.1 General - Integral Abutment Founded on Driven H-piles

A 120-foot span integral structure will require an estimated girder depth of approximately 5 feet and abutment breastwall height of approximately 6 feet. This results in a depth of approximately 11 feet to accommodate the superstructure and stub abutment. The depth to bedrock below the existing roadway elevation is approximately 36 feet in the vicinity of Abutment No. 1 and approximately 38 feet in the vicinity of Abutment No. 2. The substructure design, considering pile embedment in the abutment, will require pile lengths of approximately 32 to 34 feet. This data is summarized in Table 3.

Proposed Structure	Approximate Bedrock Elevation (feet)	Estimated Pile Cap Elevation (feet)	Estimated Pile Embedment in Abutment (feet)	Estimated Pile Lengths after cut-off (feet)
Abutment No. 1	165.30	191.5	6.0	32
Abutment No. 2	163.60	191.0	6.0	34

Table 3. Estimated Pile Lengths after cut-off

7.2 Integral Pile Design

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

H-piles shall be designed at the strength limit states considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 7.2.1 below.

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

7.2.1 Strength Limit State Design

The nominal compressive resistance (P_n) in the structural limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. For preliminary analyses, the H-piles were assumed fully embedded, and the column slenderness factor, λ, was taken as 0. The factored structural axial compressive resistances of the four proposed H-pile sections presented in this report were calculated using a resistance factor, ϕ_c, of 0.60 and a λ of 0. It is the responsibility of the Structural Designer to recalculate λ for the upper and lower portions of the H-pile based on unbraced length and K-values from project specific L-Pile® analyses and recalculate structural resistances.

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

The nominal and factored axial geotechnical resistance in the strength limit state was calculated using the Canadian Geotechnical Society method and a resistance factor, φ_{stat}, of 0.45. The calculated factored geotechnical resistances of four (4) H-pile sections are provided in Table 4, below.

Drivability analyses of the four (4) proposed H-pile sections were conducted. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be no more that 45 ksi. The resistance factor for a single pile in axial compression when a dynamic test is performed given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn} = 0.65. LRFD Table 10.5.5.2.3-3 requires that no less than three to four dynamic tests be conducted for sites with low to medium variability. When a pile group is nonredundant, i.e., there are less than five (5) piles, LRFD Article 10.5.5.2.3 dictates a 20 percent reduction of the resistance factor value of 0.65. The factored pile resistances provided in this report assume a five-pile group, and therefore are factored by resistance factor, ϕ_{dyn}, of 0.65.

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

	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance ϕ_c=0.60 $\lambda = 0$	Geotechnical Resistance $\varphi_{stat} = 0.45$	Drivablity Resistance $\varphi_{stat} = 0.65$	Governing Pile Resistance
HP 12 x 53	465	47	285	285
HP 14 x 73	642	64	373	373
HP 14 x 89	783	78	413	413
HP 14 x 117	1032	103	465	465

Table 4. Factored Axial Compressive Resistances for H-Pile Sections for Strength Limit State Design

LRFD Article 10.7.3.2.3 states that the nominal compressive resistance of piles driven to hard rock is controlled by the structural limit state. However, the calculated factored axial drivablity resistance is less than the calculated factored axial structural resistance, and local experience supports the estimated factored resistance from the drivability analyses. Therefore, the recommended governing resistance for pile design should be the factored drivability resistance in Table 4.

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

7.2.2 Service and Extreme Limit State Design

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

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

The calculated factored axial structural, geotechnical and drivablity resistances of four (4) H-pile sections were calculated for the service and extreme limit states and are provided below in Table 5. Supporting documentation is provided in Appendix C – Calculations.

	Service and Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance, assuming $\lambda = 0$	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
HP 12 x 53	775	105	438	438
HP 14 x 73	1070	143	574	574
HP 14 x 89	1305	174	636	636
HP 14 x 117	1720	229	716	716

Table 5. Factored Axial Pile Resistance for H-Piles Sections for Service and Extreme Limit State Design

LRFD Article 10.7.3.2.3 states that the nominal compressive resistance of piles driven to hard rock is controlled by the structural limit state. However, the calculated factored axial drivability resistance is less than the calculated factored axial structural resistance, and local experience supports the estimated factored resistance from the drivablity analyses. Therefore, it is recommended that the governing resistance used in design be the factored drivability resistance in Table 5.

7.2.3 Driven Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock pile splices should not be permitted.

Contract documents should require the contractor to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each substructure. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. Restrikes will be not be required as part of the pile field quality control program.

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

Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than $0.90\phi_{da}$ F_y, where ϕ_{da} is equal to 1.0, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 5 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.3 Integral Stub Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states specified in LRFD Articles 3.4.1 and 11.5.5. The design of abutments at the strength limit state shall consider pile group failure and structural reinforced concrete failure. Strength limit state shall also consider changed in foundation conditions and pile group resistance after scour due to the design flood. The design of cantilevered, in-line wingwalls at the strength limit state shall consider structural reinforced concrete failure.

A resistance factor of 1.0 shall be used for abutment design at the service limit state, including: settlement, excessive horizontal movement and movement resulting from scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65

Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

The Designer may assume Soil Type 4 MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for backfill material. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf and a soil-concrete friction coefficient of 0.45. Cast-in-place integral abutment and wingwall sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure. The Coulomb passive earth pressure coefficient, K_p, of 7.33 is recommended. Developing full passive pressure requires displacements of the abutment on the order of 2 to 5 percent of the abutment height. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction, or designing using a reduced

Coulomb passive earth pressure coefficient, but in no case should the passive earth pressure case be less than the Rankine passive earth pressure coefficient, K_p, of 3.3.

A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, γ_{EH}=1.50.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the abutments and walls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads is permitted per LRFD Article 3.11.6.5. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) taken from Table 6 below:

Abutment Height (feet)	H_{eq} (feet)
5	4.0
10	3.0
>=20	2.0

Table 6. Equivalent Height of Soil for Estimating Live Load Surcharge

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. The approach slab should be positively attached to the integral abutment.

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

Slopes in front of pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile and not exceed 1.75H:1V.

7.4 Scour and Riprap

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

In general, for scour protection, any footings which are constructed on soil deposits should be embedded at least 2 feet below the design scour depth and armored with 3 feet of riprap for scour protection. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

For scour protection, bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the MaineDOT BDG. Stone riprap shall conform to item number 703.26 Plain and Hand Laid Riprap of the Standard Specification and be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. The riprap section shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material conforming to item number 703.19, of the Standard Specification. Riprap may be placed at the toes of abutments, wingwalls and retaining walls, as required.

7.5 Settlement

There are three (3) areas where settlement is of concern on this project. The existing west bridge spans a canal which will be filled in with 13 feet of soil. The grade of approach roadway to Abutment No. 1 will be raised approximately 2 feet. The riverbank below the existing fourth bridge span of the east bridge, between the existing pier 3 and abutment 2, will be filled in with 13 feet of soil. Settlements due to elastic compression of the soils in these areas of concern were calculated and are provided in Table 7 below. Settlement of the granular foundation soils will be elastic and occur primarily during construction. Supporting calculations are provided in Appendix C – Calculations.

Location	Estimated Elastic Settlement (inches)
West bridge over canal (approx. Sta. 6+00) with 13 feet of fill	1.3
Abutment 1 approach (approx. Sta 7+25) with 2 feet of new fill	0.5
Riverbank between the existing pier 3 and east abutment (approx. Sta. 9+00) with 13 feet of fill	1.5

Table 7. Estimated Settlement

Any settlement of the proposed bridge abutments will be due to elastic settlement of the bedrock or piles, which is assumed to occur during construction and will be negligible.

7.6 Frost Protection

Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG.

It is anticipated that return wingwalls at the corners of the abutments will be straight extension wings. However, should any walls be founded on spread footings on compacted granular borrow, the foundations should be designed with an appropriate embedment for frost protection. According to the MaineDOT BDG, Newport, Maine has a design freezing index of approximately 1800 F-degree days. A water content of 15% from laboratory data was used for coarse grained granular fill soil above the water table. These components correlate to a frost depth of approximately 6.9 feet. Modberg, a computer program, developed by U.S. Army Cold Regions Research and Engineering Laboratory, was used to check the calculated maximum depth of frost penetration. The calculated depth of frost according to the Modberg solution, which is based on the Modified Berggren Equation, is 7.3 feet.

It is recommended that foundations placed on granular fill soil should be founded a minimum of 7.0 feet below finished exterior grade for frost protection.

7.7 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2, seismic analysis is not required for single-span bridges, regardless of seismic zone. Main Street Bridge is not on the National Highway System, and is therefore not classified as functional important. Furthermore, the bridge is not classified as a major structure, since the bridge construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and bridge seat dimensions shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD Manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak ground acceleration coefficient (PGA) = 0.072g
- Design spectral acceleration coefficient at the 0.2-second period, S_{DS} = 0.248g
- Design spectral acceleration coefficient at the1.0-second period, S_{D1} = 0.110g
- Site Class D (site soils with an average blow count between 5 and 50 bpf or an undrained shear strength between 1000 and 2000 psf)
- Seismic Zone 1 (based on a $S_{D1} \leq 0.15$g)

7.8 Construction Considerations

Construction of the abutments will require soil excavation and excavation of the existing substructures. Construction activities may require cofferdams and earth support systems. Portions of existing abutments, retaining walls and piers that are left in place may obstruct installation of piles. Removal of all of the existing substructures may be necessary, in particular at the proposed pile locations at Abutment No. 1 and Abutment No. 2. This may also necessitate the replacement of excavated backfill soils and old substructure locations with compacted granular fill before pile driving can commence.

In some locations, the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The contractor should control groundwater, surface water infiltration, and soil erosion.

Using the excavated native soils as structural backfill should not be permitted, and may only be used as common borrow in accordance with MaineDOT Standard Specifications Sections 203 and 703.

The contractor will have to excavate the existing subbase gravel and the subgrade fill soils. These materials should not be used to re-base the new bridge approaches, but excavated subbase sand and gravel may be used as fill below subgrade level in fill embankments provided all other requirements of MaineDOT Standard Specifications Sections 203 and 703 are met.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Main Street Bridge in Newport, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-101
PIN: 15625.00

Driller:	MaineDOT	**Elevation (ft.)**	185.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/08; 11:00-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+92.6, 9.6 Rt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	Stream Elev.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/3	0.00 - 2.00	10/8/2/2	10	13	47			Dark brown, moist, medium dense, silty fine to coarse SAND, little roots and fibers, slightly organic, two 1" rock fragments, (River Bottom Sediments).	
							33				
	2D	24/7	2.00 - 4.00	9/22/20/14	42	54	46			Dark brown, wet, very dense, fine angular GRAVEL, some fine to coarse SAND, trace organic silt, few rock fragments, (Alluvium with Riverbottom Sediments).	G#209975 A-1-a, GP-GM WC=11.8%
							604				
							19			Hit wood at 4.0' bgs. Wood in wash water from 4.0-5.0' bgs.	
5	3D	24/3	5.00 - 7.00	3/2/3/8	5	6	6			Brown, wet, loose, medium to coarse SAND, some wood, little silt, trace clay and fine angular gravel. Telescoped NW Casing into HW Casing at 5.0' bgs.	
							26				
							25				
	R1	24/17	8.25 - 10.25	RQD = N/A%			[a]31	177.35		[a]31 blows for 3", then 220 blows after coring. 8.25	
							NQ-2 134	175.95		R1: Granite BOULDER 1.4' thick. R1:Core Times (min:sec) 8.25-9.25' (3:19) 9.25-10.25' (2:38)	
10							CORE 77			9.65	
	4D/AB	24/19	11.00 - 13.00	7/16/27/36	43	55	102	173.60		(4D/A) 11.0-12.0' bgs. Grey, moist, very dense, angular fine to coarse gravelly SAND, trace silt, (Alluvium). Changed to brown in wash water at 11'3". 12.00	G#210011 A-1-a, SW-SM WC=10.7%
							142			(4D/B) 12.0-13.0' bgs. Olive-brown, damp to moist, silty fine SAND, some medium sand, little clay, little fine angular gravel, some staining. (Till).	
							255				
							215				
15	5D	24/20	15.00 - 17.00	24/33/43/36	76	98	[b]RC			[b]Roller Coned ahead to 18.0' bgs. Brown, moist, very dense, fine to coarse SAND, some silt, little fine to coarse angular gravel, trace clay, well bonded, some staining/ oxidation. (Till).	G#210012 A-2-4, SC-SM WC=9.4%
	R2	60/53	18.00 - 23.00	RQD = 26%			NQ-2 CORE	167.60		18.00 Top of Bedrock at Elev. 167.6 ft. R2: Bedrock: Grey, fine grained, metasedimentary (HORNFELS), moderately hard to hard, moderately weathered, cleaves along folliation at steep angles, tight, weathered and stained surfaces; moderately fractured and weathered zone 3.6-4.2', no foliation or cleavage in upper 10", pegmatite veins in lower 6". Vassalboro Formation. Rock Mass Quality: Poor. R2:Core Times (min:sec) 18.0-19.0' (3:08) 19.0-20.0' (2:26) 20.0-21.0' (2:40) 21.0-22.0' (2:31) 22.0-23.0' (2:40) 88% Recovery	
20											
								162.60		23.00	
25											

Remarks:

Large cobble moved to side at Ground Surface.
16.8' from Bridge Deck to Ground.

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-NSR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Main Street Bridge #2501 over East Branch Sabasticook River	**Location:**	Newport, Maine	**Boring No.:**	BB-NSR-101
				PIN:	15625.00
Driller:	MaineDOT	**Elevation (ft.)**	185.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/08; 11:00-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+92.6, 9.6 Rt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	Stream Elev.
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										**Bottom of Exploration at 23.00 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

Large cobble moved to side at Ground Surface.
16.8' from Bridge Deck to Ground.

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: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-102
PIN: 15625.00

Driller:	MaineDOT	**Elevation (ft.)**	188.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/3/08-6/4/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+60.9, 10.9 Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	2.0' 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D/AB	24/7	0.00 - 2.00	11/10/3/3	13	17	8		(1D/A) Brown, damp, medium dense, sandy GRAVEL. (1D/B) Grey, damp, angular coarse GRAVEL (broken rock fragments), little fine to medium sand.	
							14			
	2D	24/8	2.00 - 4.00	2/2/4/3	6	8	5	186.60	2.00 Dark brown, wet, loose, fine sandy organic SILT, grading to very dark brown, moist, fine to coarse SAND, trace gravel, slight odor, (River Bottom Sediment and Alluvium).	
							8			
							50		Washing out to 5.0' bgs, hit rock fragment at 4.5' bgs.	
5	3D	24/8	5.00 - 7.00	12/17/20/12	37	47	59	183.60	5.00 Dark olive-grey, saturated, dense, angular fine to coarse gravelly SAND, little silt, (gravel is broken rock fragments), slight odor, (River Bottom Sediment and Alluvium).	G#209970 A-1-a, SM WC=18.4%
							93			
							77			
							70			
							100	179.60	9.00	
10	4D	24/10	10.00 - 12.00	15/18/24/20	42	54	81		Grey, moist, very dense, fine to coarse SAND, some fine angular gravel, little silt, well sorted, (Alluvium). Telescoped NW Casing into HW Casing at 11.1' bgs.	G#209971 A-1-b, SM WC=10.6%
							23			
							65			
							71			
							109	174.60	14.00	
15	5D/BA	24/12	15.00 - 17.00	27/34/43/32	77	99	71		(5D/B) Olive grey, damp to moist, very dense, silty SAND, trace angular gravel, some staining, (Glacial Till). (5D/A) Olive grey, damp, very dense, fine to coarse SAND, some silt, little fine angular gravel and weathered rock fragments, (Glacial Till).	G#209973 A-4, SM WC=11.1% G#209972 A-2-4, SM WC=10.1%
							95			
							140			
							172			
							265			
20	6D/AB	24/18	20.00 - 22.00	33/53/52/49	105	135	63		(6D/A) Olive grey and brown, mottled, damp, very dense, gravelly SILT, gravel fine to coarse, angular, including weathered rock fragments, some fine sand, little clay, (Glacial Till).	
							70		Roller Coned ahead to 23.8' bgs., hit something hard, roller coned ahead to 25.0' bgs.	
							79		(6D/B) Brown, moist, very dense, silty fine to coarse SAND, some fine to coarse angular gravel, trace clay. (Glacial Till).	
							88			
25									NW Casing to 24.5' bgs.	

Remarks:

13.9' from Bridge Deck to Ground.

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: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-102
PIN: 15625.00

Driller:	MaineDOT	**Elevation (ft.)**	188.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/3/08-6/4/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+60.9, 10.9 Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	2.0' 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	R1	60/57.6	25.00 - 30.00	RQD = 96%			NQ-2 CORE	163.60		25.00 Top of Bedrock at Elev. 163.6 ft. R1: Bedrock: Dark grey, fine grained, metasedimentary (HORNFELS) moderately hard, fresh, occasional quartz veins, no foliation, drill breaks along quartz veins, one open seam 8" from top, upper 8" quartz disolved, some vuggy seams. Vassalboro Formation. Rock Mass Quality: Excellent. R1:Core Times (min:sec) 25.0-26.0' (10:40) 26.0-27.0' (5:00) 27.0-28.0' (4:25) 28.0-29.0' (5:15) 29.0-30.0' (6:27) 96% Recovery	
30	R2	58.8/58.8	30.00 - 34.90	RQD = 86%						R2: Bedrock: Same as R1, only less fractured, fractures along quartz veins, surfaces stained with some oxidation, drill breaks along quartz veins. Vassalboro Formation. Rock Mass Quality: Good. R2:Core Times (min:sec) 30.0-31.0' (6:30) 31.0-32.0' (5:50) 32.0-33.0' (3:30) 33.0-34.0' (4:05) 34.0-34.9' (2:33) 100 % Recovery	
35								153.70		34.90 **Bottom of Exploration at 34.90 feet below ground surface.**	
40											
45											
50											

Remarks:

13.9' from Bridge Deck to Ground.

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: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-201
PIN: 15625.00

Driller:	MaineDOT	**Elevation (ft.)**	192.5	**Auger ID/OD:**	N/A
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:	6/15/09, 6/25/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	7+47.8, 12.9 Rt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	16.0'-6/15, 9.0' 6/25 bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	7.2/5	0.00 - 0.60	1/50(1.2")	---		2			Brown, wet, very dense, gravelly fine to coarse SAND, some silt, roots, cobbles.	
							50			Boulder from 1.4-2.5' bgs. Roller Coned ahead to 5.0' bgs.	
							15				
							OPEN HOLE	189.00		3.50	
5	2D	24/10	5.00 - 7.00	6/11/16/11	27	38	15			Dark grey, wet, dense, sandy GRAVEL, little silt, occasional cobble, slight odor. Switched to NW casing. Roller Coned ahead to 9.0' bgs.	G#212332 A-1-a, GM WC=16.6%
							81				
							142				
							171				
	3D	24/1	9.00 - 11.00	3/7/6/7	13	18	150			Similar to above, except medium dense.	
10							44				
							59				
							64	180.00		12.50	
							51				
	4D	24/8	14.00 - 16.00	19/13/10/18	23	32	37			Olive-grey, wet, dense, silty fine to coarse SAND, some gravel, (Till).	
15							76				
							88				
							125				
							146				
	5D	18/16	19.00 - 20.50	42/33/52	85	119	56			Olive-grey, wet, very dense, fine to coarse sandy SILT, some fine to coarse gravel, blocky Roller Coned ahead to 24.0' bgs.	
20							73				
							99				
							130				
							166	169.50		23.00	
	6D	24/17	24.00 - 26.00	21/31/29/30	60	84	14			Brown, moist, very dense, SAND, some silt, some gravel, little clay, (Till).	G#212333 A-4, CL-ML
25											

Remarks:

11.0' from Bridge Deck to Ground.
Bridge Deck Concrete 12" thick.
Used BX Core Barrel, casing was bent to much to get NQ-2 Core Barrel down hole.

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: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-201
PIN: 15625.00

Driller:	MaineDOT	**Elevation (ft.)**	192.5	**Auger ID/OD:**	N/A
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:	6/15/09, 6/25/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	7+47.8, 12.9 Rt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	16.0'-6/15, 9.0' 6/25 bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							RC			Roller Coned ahead from 25.0-27.2' bgd.	WC=11.2% LL=22 PL=17 PI=5
	R1	44.4/34	27.20 - 30.90				BX	165.30		27.20 Top of Bedrock at Elev. 165.3'. R1:Bedrock: Grey, fine grained, metasedimentary (HORNFELS) with quartz veins, moderately hard, moderately weathered. Joint breaks at close spacing. Vassalboro Formation. Rock Mass Quality: very poor, based on an estimated NQ RQD of 21%. R1:Core Times (min:sec) 27.2-28.2' (4:30) 28.2-29.2' (3:00) 29.2-30.2' (3:30) 30.2-30.9' (4:12) 76% Recovery Core Blocked	
30	R2	60/60	30.90 - 35.90							R2: Bedrock: Same as R1, except fresh, joint set moderately close to close. Rock Mass Quality: good, based on an estimated NQ RQD of 68%. R2:Core Times (min:sec) 30.9-31.9' (4:25) 31.9-32.9' (4:00) 32.9-33.9' (4:12) 33.9-34.9' (4:35) 34.9-35.9' (4:50) 100% Recovery Could not get Core Barrel back down, casing bent.	
35								156.60		35.90 **Bottom of Exploration at 35.90 feet below ground surface.**	
40											
45											
50											

Remarks:

11.0' from Bridge Deck to Ground.
Bridge Deck Concrete 12" thick.
Used BX Core Barrel, casing was bent to much to get NQ-2 Core Barrel down hole.

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

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Newport   Project Number: 15625.00

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-NSR-201, 2D	7+47.8	12.9 Rt.	5.0-7.0	212332	3	16.6			GM	A-1-a	I
BB-NSR-201, 6D	7+47.8	12.9 Rt.	24.0-26.0	212333	3	11.2	22	5	CL-ML	A-4	III
BB-NSR-101, 2D	7+92.6	9.6 Rt.	2.0-4.0	209975	1	11.8			GP-GM	A-1-a	0
BB-NSR-101, 4D/A	7+92.6	9.6 Rt.	11.0-12.0	210011	1	10.7			SW-SM	A-1-a	0
BB-NSR-101, 5D	7+92.6	9.6 Rt.	15.0-17.0	210012	1	9.4			SC-SM	A-2-4	III
BB-NSR-102, 3D	8+60.9	10.9 Lt.	5.0-7.0	209970	2	18.4			SM	A-1-a	II
BB-NSR-102, 4D	8+60.9	10.9 Lt.	10.0-12.0	209971	2	10.6			SM	A-1-b	II
BB-NSR-102, 5D/B	8+60.9	10.9 Lt.	15.0-17.0	209973	2	11.1			SM	A-4	III
BB-NSR-102, 5D/A	8+60.9	10.9 Lt.	15.0-17.0	209972	2	10.1			SM	A-2-4	II

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)

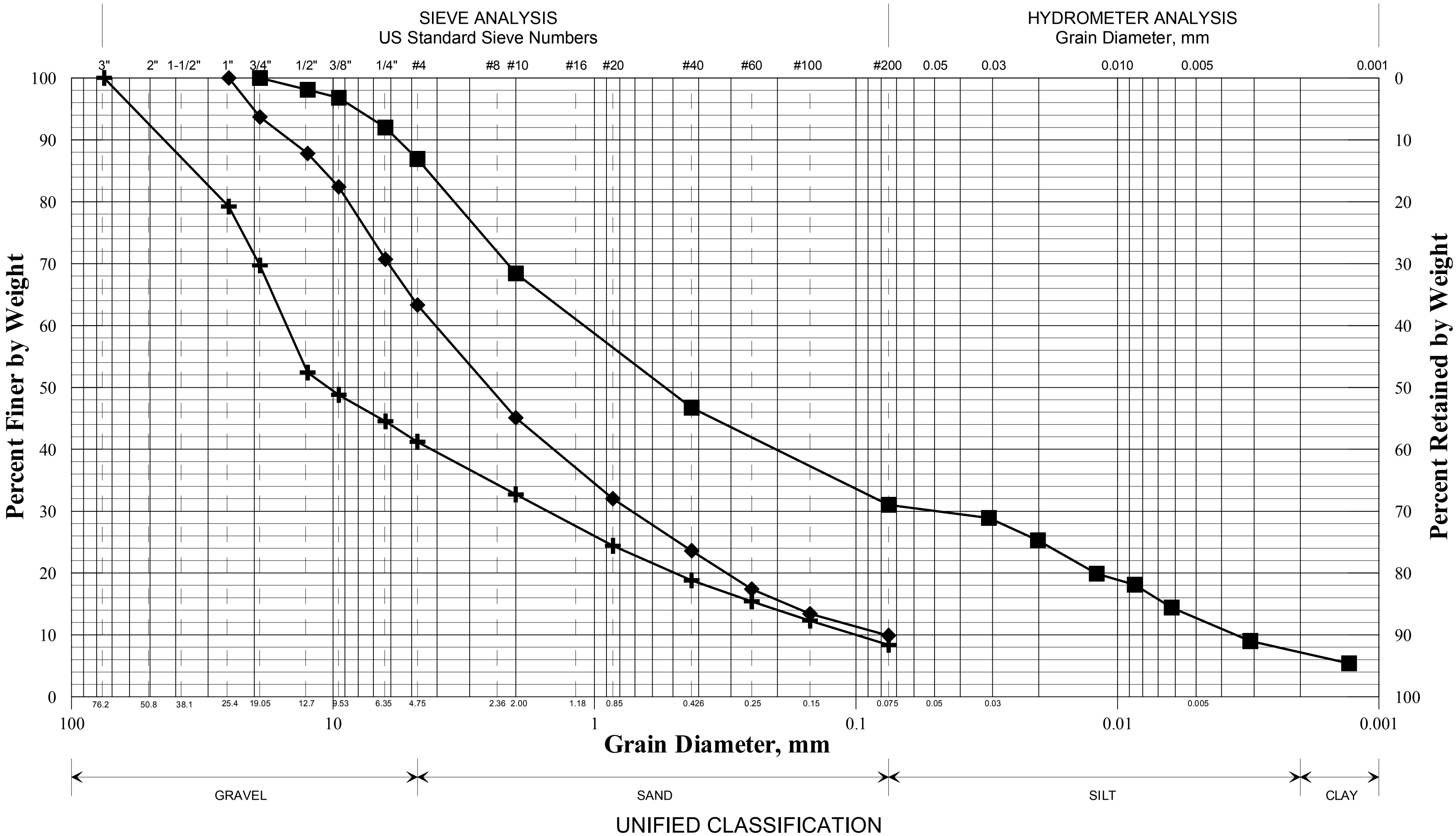
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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



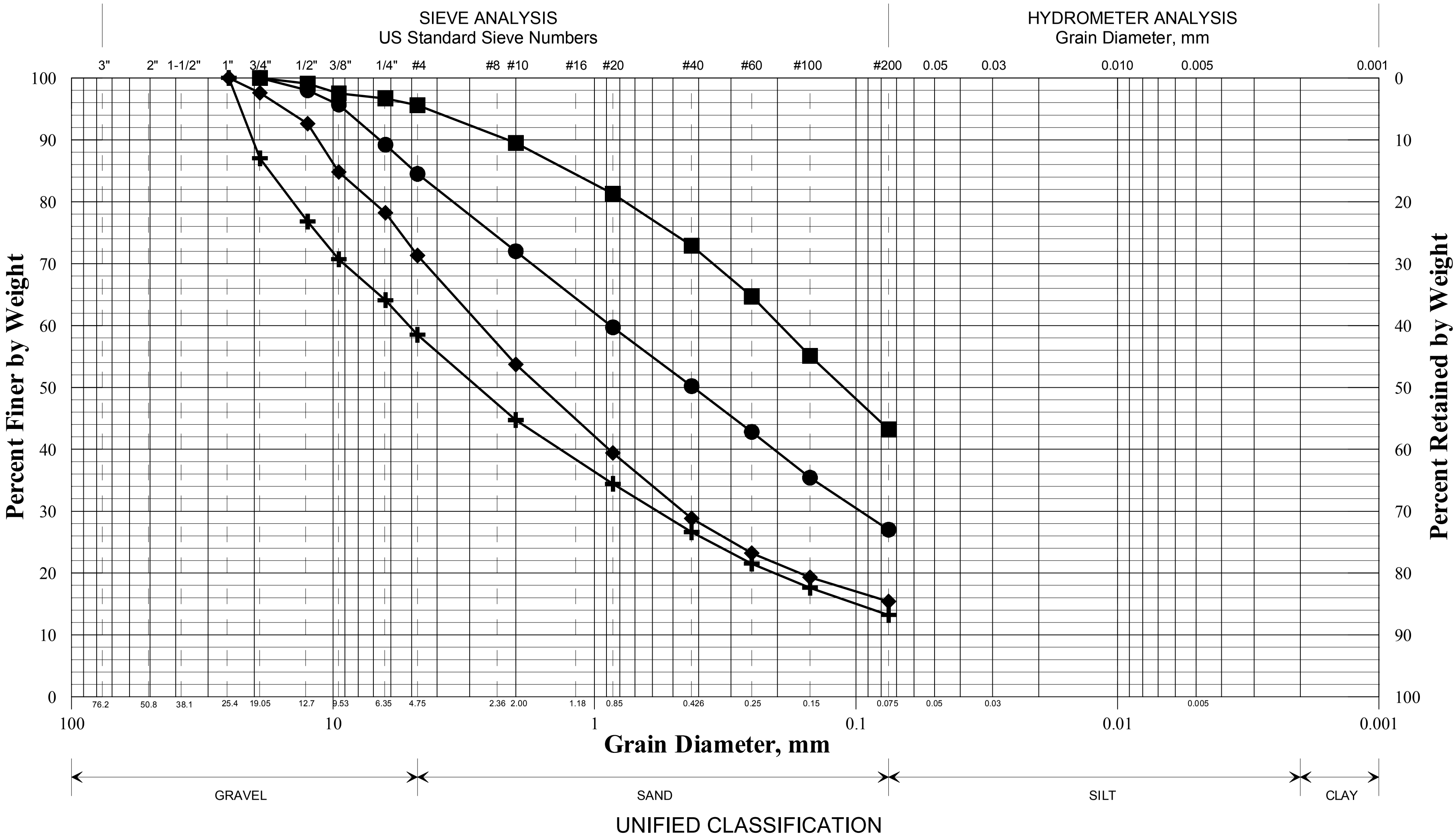
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NSR-101/2D	7+92.6	9.6 RT	2.0-4.0	GRAVEL, some sand, trace silt.	11.8			
◆	BB-NSR-101/4D/A	7+92.6	9.6 RT	11.0-13.0	Gravelly SAND, trace silt.	10.7			
■	BB-NSR-101/5D	7+92.6	9.6 RT	15.0-17.0	SAND, some silt, little gravel, trace clay.	9.4			
●									
▲									
✖									

PIN	
015625.00	
Town	
Newport	
Reported by/Date	
WHITE, TERRY A	9/8/2008

SHEET 1

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE

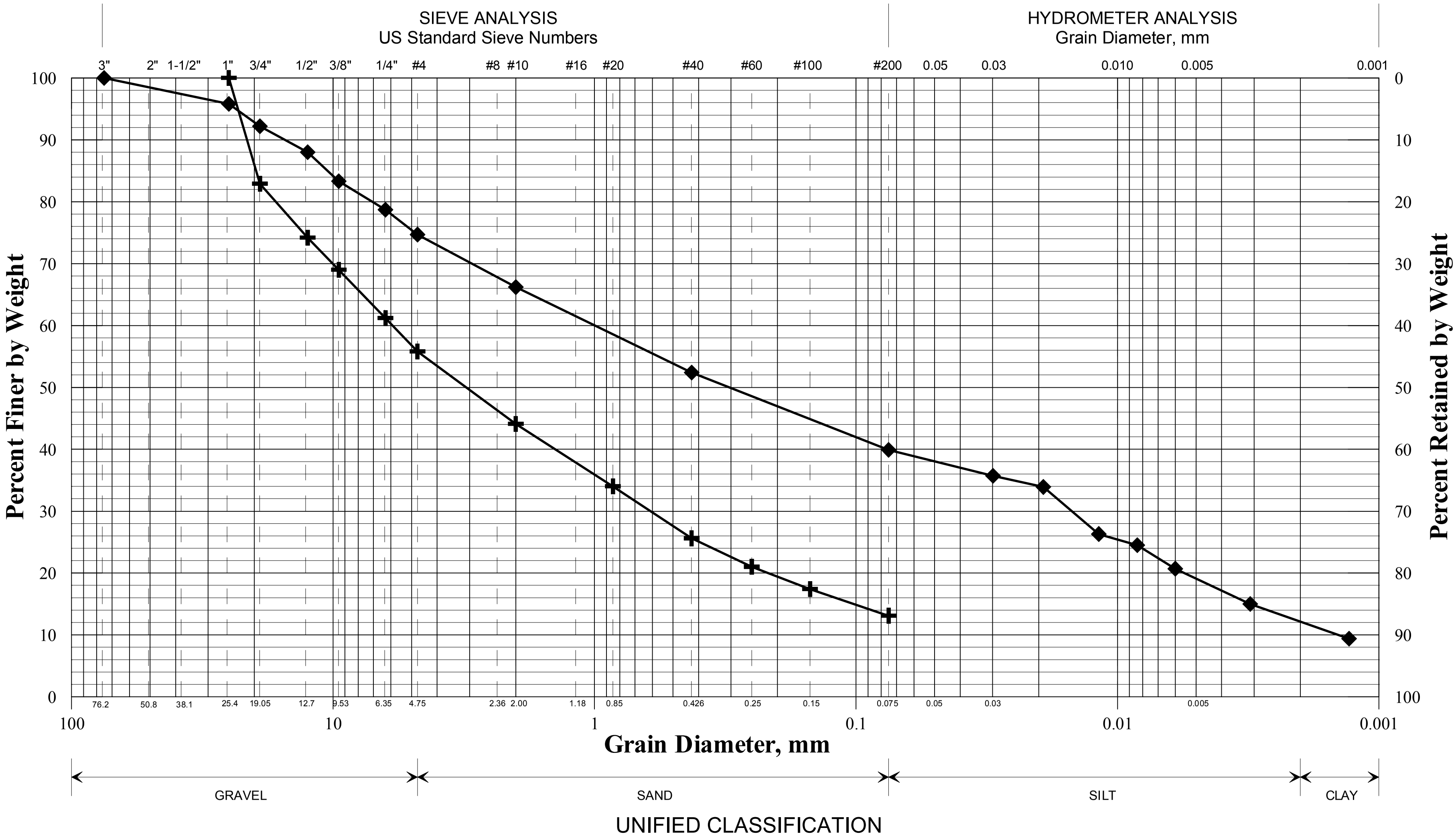


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NSR-102/3D	8+60.9	10.9 LT	5.0-7.0	Gravelly SAND, little silt.	18.4			
◆	BB-NSR-102/4D	8+60.9	10.9 LT	10.0-12.0	SAND, some gravel, little silt.	10.6			
■	BB-NSR-102/5D/B	8+60.9	10.9 LT	15.0-17.0	Silty SAND, trace gravel.	11.1			
●	BB-NSR-102/5D/A	8+60.9	10.9 LT	15.0-17.0	SAND, some silt, little gravel.	10.1			
▲									
✖									

PIN
015625.00
Town
Newport
Reported by/Date
WHITE, TERRY A 9/8/2008

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NSR-201/2D	7+47.8	12.9 RT	5.0-7.0	Sandy GRAVEL, little silt.	16.6			
◆	BB-NSR-201/6D	7+47.8	12.9 RT	24.0-26.0	SAND, some silt, some gravel, little clay.	11.2	22	17	5
■									
●									
▲									
✖									

PIN
015625.00
Town
Newport
Reported by/Date
WHITE, TERRY A 8/13/2009

SHEET 3

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
212333	**BB-NSR-201/6D**	**GEOTECHNICAL (DISTURBED)**	**6/25/2009**	**7/24/2009**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **7+47.8** Offset, ft: **12.9 RT** Dbfg, ft: **24.0-26.0**

PIN: **015625.00** Town: **Newport** Sampler: **GIGUERE, ERVIN M**

TEST RESULTS

Sieve Analysis (T-88)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**95.8**
¾ in. [19.0 mm]	**92.2**
½ in. [12.5 mm]	**88.0**
⅜ in. [9.5 mm]	**83.3**
¼ in. [6.3 mm]	**78.7**
No. 4 [4.75 mm]	**74.7**
No. 10 [2.00 mm]	**66.2**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**52.4**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**39.9**
[0.0299 mm]	**35.7**
[0.0192 mm]	**33.9**
[0.0118 mm]	**26.3**
[0.0084 mm]	**24.5**
[0.0060 mm]	**20.7**
[0.0031 mm]	**15.0**
[0.0013 mm]	**9.4**

Wash Method

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89)
22

Plastic Limit (T 90)
17

Plasticity Index (T 90)
5

Specific Gravity, Corrected to 20°C (T 100)
2.75

Loss on Ignition (T 267)
Loss, % H2O, %

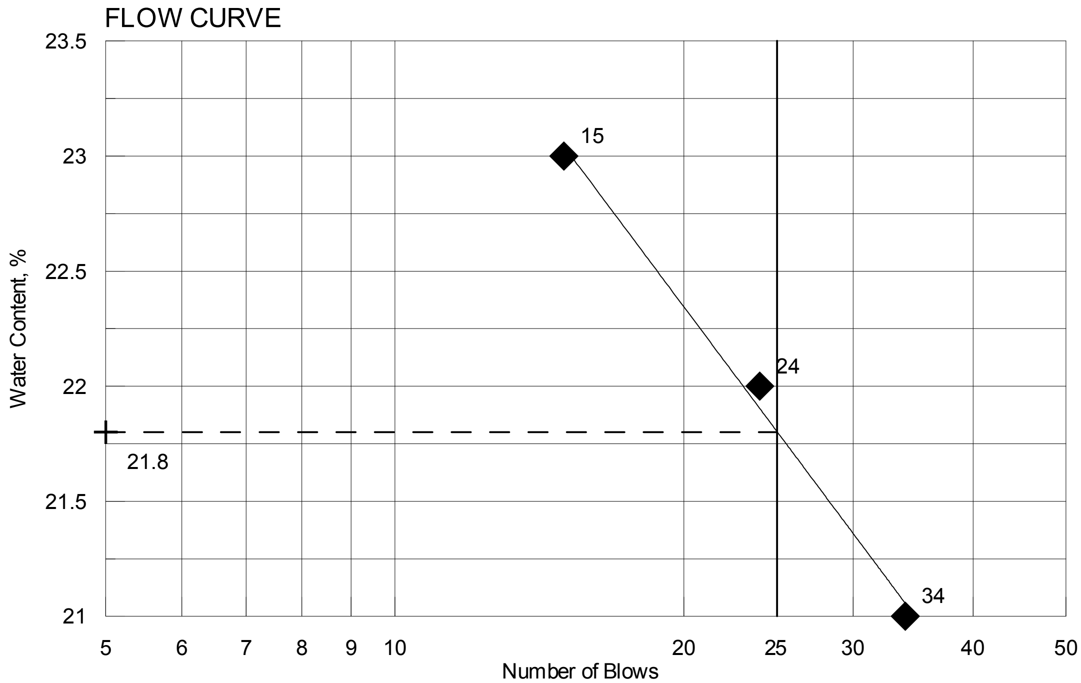
Water Content (T 265), %
11.2

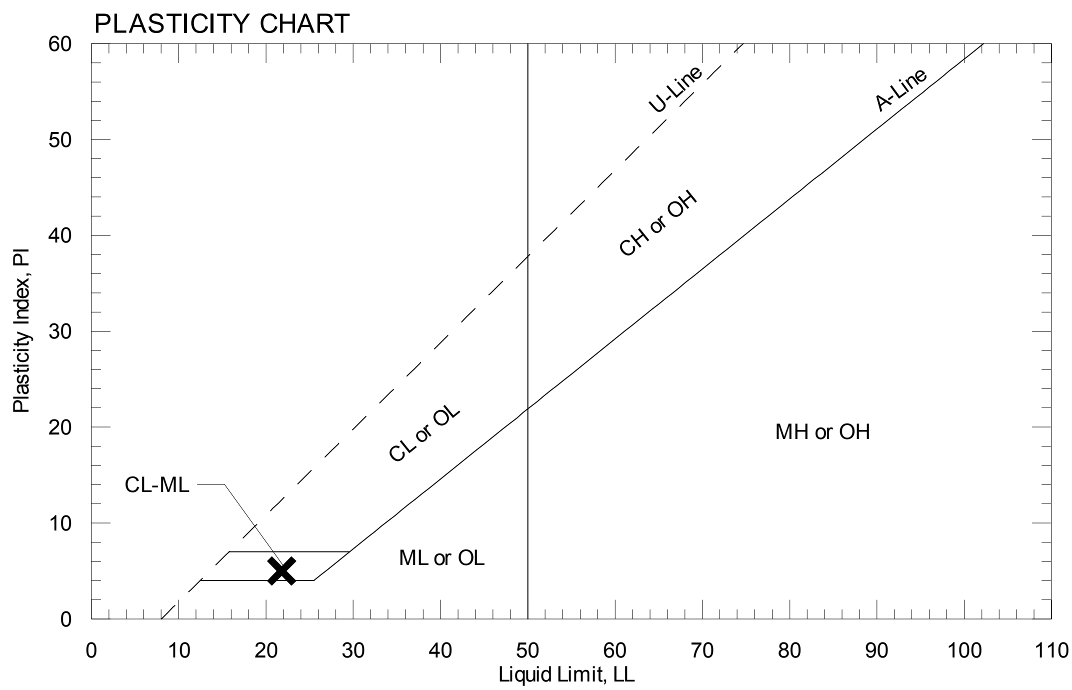
Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

TOWN	Newport	Reference No.	212333
PIN	015625.00	Water Content, %	11.2
Sampled	6/25/2009	Plastic Limit	17
Boring No./Sample No.	BB-NSR-201/6D	Liquid Limit	22
Station	7+47.8	Plasticity Index	5
Depth	24.0-26.0	Tested By	BBURR





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

Reported by: **FOGG, BRIAN** Date Reported: **8/4/2009**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

Bedrock Properties at the Site

RQD of bedrock cores
26% in BB-NSR-101
BX core in BB-NSR-201 (if NQ, 21% to 68%)
96% to 86% in BB-NSR-102

Rock Type: Metasedimentary (Hornfels)

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

uniaxial compressive strength = Co= 1400 to 21,000 psi - use **10,000 psi** for design AASHTO TABLE 4.4.8.1.2.B

Pile Properties

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

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

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

Nominal and Factored Structural Compressive Resistance of HP piles

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

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

$$F_y := 50 \cdot ksi$$

$$\lambda := 0$$

Nominal Axial Structural Resistance

From LRFD 6.9.4.1-1

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

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

Factored Axial Structural Resistance of single H pile

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

$\phi_c := 0.6$

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

Factored structural compressive resistance, Pr

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

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Table 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example in FHWA-NHI-05-094.

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

Hornfels $q_{u\_1} := 10000 \cdot psi$

Spacing of discontinuities $s_d := 4 \cdot in$

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

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

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

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

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

$dd = 1$ OK

Ksp

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

$$K_{sp} = \begin{pmatrix} 0.226 \\ 0.222 \\ 0.222 \\ 0.222 \end{pmatrix}$$

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

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$$

$$q_{p\_1} = \begin{pmatrix} 977 \\ 960 \\ 959 \\ 958 \end{pmatrix} \cdot ksf$$

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

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$$

$$R_{p\_1} = \begin{pmatrix} 105 \\ 143 \\ 174 \\ 229 \end{pmatrix} \cdot kip$$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Candadian Geotechnical Society method

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

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$$

$$R_{r\_p1} = \begin{pmatrix} 47 \\ 64 \\ 78 \\ 103 \end{pmatrix} \cdot kip$$

Drivability Analysis

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

$\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium variablity. Only 2 piles will be tested, and the pile group would be nonredundant, i.e. less than five piles. Therefore reduce Φ by 20%.

$\phi_{dyn\_red} := 0.65 \cdot 0.8$ $\phi_{dyn\_red} = 0.52$

Pile Size is 12 x 53

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

State of Maine Dept. Of Transportation — 21-Jul-2009
Newport 12 x 53 fuel set reduced — GRLWEAP (TM) Version 2003

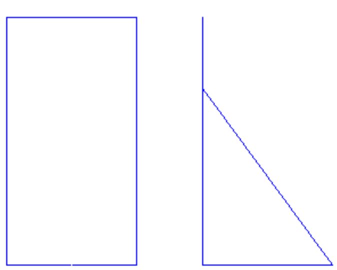
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	19.70	0.16	1.1	4.92	12.82
200.0	26.81	0.45	2.7	5.90	12.28
300.0	34.39	2.97	4.4	6.62	13.35
350.0	37.71	3.58	5.6	6.97	14.01
400.0	42.25	4.00	6.9	7.47	15.11
450.0	45.84	4.43	9.0	7.81	15.73
500.0	49.11	4.83	11.7	8.21	16.56

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	25.00	ft
Pile Top Area	15.50	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 30 %
(Constant Res. Shaft)

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 42.25}{45.84 - 42.25}\right)\cdot(450\cdot kip - 400\cdot kip) + 400\cdot kip$$

$$R_{ndr} = 438.3\cdot kip$$

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

$$\boxed{R_{fdr} = 228\cdot kip}$$

For a resistance factor for dynamic test of 0.65:

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

$$\boxed{R_{fdr} = 285\cdot kip}$$

Pile Size is 14 x 74

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

State of Maine Dept. Of Transportation
14 x 73 fuel set 9 ft str

21-Jul-2009
GRLWEAP (TM) Version 2003

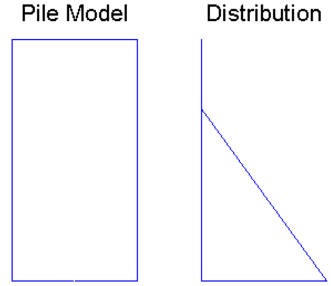
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	19.46	0.07	1.0	5.51	15.43
300.0	29.82	1.21	3.9	7.23	14.87
400.0	35.77	3.08	5.8	7.87	15.92
500.0	41.38	4.11	8.3	8.71	17.54
550.0	43.49	4.41	10.4	8.99	17.99
600.0	45.66	5.00	12.9	9.35	18.64
700.0	49.59	6.05	20.6	10.05	20.22

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.060	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	25.00	ft
Pile Top Area	21.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 30 %
(Constant Res. Shaft)

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 43.49}{46.66 - 43.49}\right)\cdot(600\cdot kip - 550\cdot kip) + 550\cdot kip$$

$$R_{ndr} = 574\cdot kip$$

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

$$\boxed{R_{fdr} = 298\cdot kip}$$

For a resistance factor for dynamic test of 0.65:

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

$$\boxed{R_{fdr} = 373\cdot kip}$$

Pile Size is 14 x 89

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

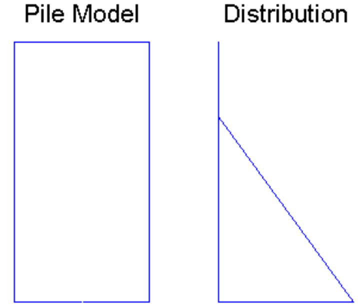
State of Maine Dept. Of Transportation — 21-Jul-2009
14 x 89 Delmag 19-42 helmet 2.7 kip — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	19.67	0.35	0.9	5.93	17.68
300.0	27.96	0.80	3.5	7.78	16.30
610.0	43.95	3.65	9.0	9.89	20.09
630.0	44.77	3.86	9.6	10.02	20.38
640.0	45.16	3.99	9.9	10.08	20.55
665.0	46.16	4.26	10.8	10.23	20.93
700.0	47.43	4.62	12.2	10.43	21.39

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	25.00	ft
Pile Top Area	26.10	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 30 %
(Constant Res. Shaft)

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.77}{45.16 - 44.77}\right)\cdot(640\cdot kip - 630\cdot kip) + 630\cdot kip$$

$$R_{ndr} = 635.9\cdot kip$$

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

$$\boxed{R_{fdr} = 331\cdot kip}$$

For a resistance factor for dynamic test of 0.65:

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

$$\boxed{R_{fdr} = 413\cdot kip}$$

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a D 36-32 at Fuel Setting 3 and a 2.7 kip helmet, at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 21-Jul-2009
14 x 117 fuel setting 3 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	16.16	0.59	0.5	4.18	32.67
300.0	26.34	0.17	1.9	6.01	26.48
500.0	35.50	0.90	3.6	6.76	26.29
650.0	42.27	1.79	4.9	7.38	28.22
700.0	44.32	2.53	5.5	7.60	28.91
750.0	46.44	2.68	6.1	7.84	29.76

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	25.00	ft
Pile Top Area	34.40	in2

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.32}{46.44 - 44.32}\right)\cdot(750\cdot kip - 700\cdot kip) + 700\cdot kip$$

$$R_{ndr} = 716\cdot kip$$

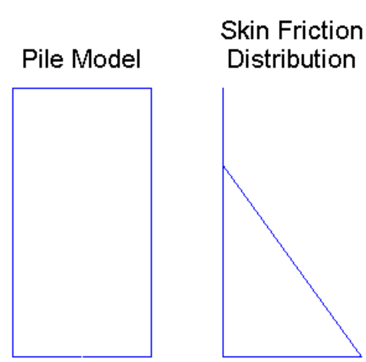
$$R_{fdr} := R_{ndr}\cdot\phi_{dyn\_red}$$

$$\boxed{R_{fdr} = 372\cdot kip}$$

For a resistance factor for dynamic test of 0.65:

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

$$\boxed{R_{fdr} = 465\cdot kip}$$



Res. Shaft = 30 %
(Constant Res. Shaft)

Calibration back to ASD - Structural Capacity

Geotechnical design capacity shall not exceed the pile structural allowable design load , based on allowable steel stress for integral piles, use 50 ksi steel, therefore 0.25Fy is the allowable stress.

For 50 ksi steel $F_y := 50 \cdot ksi$ $\sigma_a := \frac{F_y}{4}$ $Q_{all} := \sigma_a \cdot A_s$

$$Q_{all} = \begin{pmatrix} 194 \\ 268 \\ 326 \\ 430 \end{pmatrix} \cdot kip$$

50 ksi steel piles driven to 2.25 times the structural capacity

$Q_{ult} := Q_{all} \cdot 2.25$

$$Q_{ult} = \begin{pmatrix} 436 \\ 602 \\ 734 \\ 968 \end{pmatrix} \cdot kip$$

Assume the above equals the nominal geotechnical capacity

Factored resistance = 2.25 times the structural capacity times a resistance factor of 0.65

$R_{factored} := Q_{ult} \cdot 0.65$

$$R_{factored} = \begin{pmatrix} 283 \\ 391 \\ 477 \\ 629 \end{pmatrix} \cdot kip$$

Calculation of Elastic Settlement due to 13 of fill from filling in existing west bridge - Soil profile based on strata encountered at BB-NSR-201

Soil properties & groundwater conditons; unit weight per LRFD 3.5.1-1

$\gamma_t := 120 \cdot pcf$ $\gamma_w := 62.4 \cdot pcf$ $\gamma' := \gamma_t - \gamma_w$ $\gamma' = 57.6 \cdot pcf$ $D_w := 9 \cdot ft$

N values already corrected for hammer efficiency

$$N := \begin{pmatrix} 38 \\ 18 \\ 32 \\ 119 \\ 84 \end{pmatrix}$$

Drained friction angles per LRFD 10.4.6.2.4-1

N160	ϕ
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Soil Profile at BB-NSR-201

First Layer- alluvium and riverbottom sediments
0-3.5 feet, gravelly sand, some silt, roots cobbles. H=3 feet
Second Stratum - alluvium
3.5 - 12.5 feet, sandy gravel, little silt, occassional cobble (alluvium). H=9 ft
Third Stratum - glacial till
12.5 to 27 feet bgs - H=15 ft
silty sand, some gravel
sandy silt, some gravel
sand, some silt, some gravel, little clay

Settlement Computation for Cohesionless Soils

Reference : FHWA Soils and Foundation Workshop Manual NHI-06-088, 2006

Existing Vertical Overburden Stress and Change in vertical stress due to 13 feet of filling in the west bridge.

See last sheet for STRESS output for change in stress.

Break soil profile into six layers:

Layer 1 - 3 feet of alluvium w/ riverbottom sediments, 120 pcf, 32 degrees
Layer 2 - 4 feet of alluvium, 125 pcf, 36 degrees
Layer 3 - 5 feet of alluvium, 120 pcf 30 degrees
Layer 4 - 5 feet of till, 120 pcf, 32 degrees
Layer 5 - 5 feet of till, 125 pcf, 38 degrees
Layer 6 - 5 feet of till, 125 pcf, 38 degrees

The change in stresses below are at the center of each layer

$$\Delta\sigma z := \begin{pmatrix} 1624.86 \\ 1620.19 \\ 1596.01 \\ 1541.11 \\ 1464.58 \\ 1377.79 \end{pmatrix} \cdot psf$$

Layer 1

No Field SPT (bpf) use N=15 $\quad N1 := 15$

If SPT at 1-3 feet $\quad \sigma_2 := 2 \cdot ft \cdot 120 \cdot pcf \qquad \sigma_2 = 240 \cdot psf \qquad$ at 2 ft bgs

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_2 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_2}\right)$$ Should not exceed 2.0

$$CN_2 = 1.711$$

$$Ncor1 := CN_2 \cdot N1$$

$$Ncor1 = 25.662$$

FHWA NHI-06-088, Figure 7-7, Curve for Well graded fine to medium silty SAND

Bearing Capacity Index $C_2 := 70$

Layer $H_2 := 3 \cdot ft$

Effective overburden stress at midpoint of layer $\sigma'_2 := 1.5 \cdot ft \cdot 120 \cdot pcf$ $\sigma'_2 = 180 \cdot psf$

Do not use a σv less than 200 psf

$\sigma'_2 := 200 \cdot psf$

Settlement

$$\Delta H_2 := \overrightarrow{\left[H_2 \cdot \frac{1}{C_2} \cdot \log \left[\frac{(\sigma'_2) + \Delta\sigma z_0}{\sigma'_2} \right] \right]}$$

$\Delta H_2 = 0.494 \cdot in$

Layer 2

Field SPT (bpf) $N_0 = 38$ at 6 ft bgs

Overburden pressure at SPT elevation $\sigma_3 := 3.0 \cdot ft \cdot 120 \cdot pcf + 3 \cdot ft \cdot 125 \cdot pcf$

$\sigma_3 = 735 \cdot psf$

N - value correction for overburdent per LRFD 10.4.6.2.4

$CN_3 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_3}\right)$ Should not exceed 2.0

$CN_3 = 1.337$

$Ncor1 := CN_3 \cdot N_0$

$Ncor1 = 50.789$

NHI-08-088, Figure 7-7, Curve for Well graded silty SAND & GRAVEL

Bearing Capacity Index $C_3 := 173$

Layer $H_3 := 4 \cdot ft$

Effective overburden stress at midpoint of layer $\sigma'_3 := 1.5 \cdot ft \cdot 120 \cdot pcf + 1.5 \cdot ft \cdot 120 \cdot pcf + 2 \cdot ft \cdot 125 \cdot pcf$

$\sigma'_3 = 610 \cdot psf$

Settlement

$$\Delta H_3 := \overrightarrow{\left[H_3 \cdot \frac{1}{C_3} \cdot \log\left[\frac{(\sigma'_3) + \Delta\sigma z_1}{\sigma'_3}\right]\right]}$$

$\Delta H_3 = 0.156 \cdot in$

Layer 3

Field SPT (bpf) $N_1 = 18$

Note : groundwater at the middle of this 5 foot thick layer (at a depth of 9.0 ft)

Overburden pressure at SPT elevation
(SPT from 9-11' - use 10 ft) $\sigma_4 := \sigma_3 + 1 \cdot ft \cdot 125 \cdot pcf + 2.0 \cdot ft \cdot 120 \cdot pcf + 1 \cdot ft \cdot (120 \cdot pcf - 62.4 \cdot pcf)$

$\sigma_4 = 1157.6 \cdot psf$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_4 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_4}\right)$$ Should not exceed 2.0

$CN_4 = 1.185$

$Ncor1 := CN_4 \cdot N_1$

$Ncor1 = 21.324$

NHI-06-088, Figure 7-7, Curve for Well graded silty SAND & GRAVEL

Bearing Capacity Index $C_4 := 77$

Layer $H_4 := 5 \cdot ft$

Effective overburden stress at midpoint of layer (9.5 feet bgs)

$\sigma'_4 := \sigma'_3 + 2 \cdot ft \cdot 125 \cdot pcf + 2 \cdot ft \cdot 120 \cdot pcf + 0.5 \cdot ft \cdot (120 \cdot pcf - 62.4 \cdot pcf)$

$\sigma'_4 = 1128.8 \cdot psf$

Settlement

$$\Delta H_4 := \overrightarrow{\left[H_4 \cdot \frac{1}{C_4} \cdot \log\left[\frac{(\sigma'_4) + \Delta\sigma z_2}{\sigma'_4}\right]\right]}$$

$$\Delta H_4 = 0.298 \cdot in$$

Layer 4

Field SPT (bpf) $N_2 = 32$ SPT from 14-16, use 15 ft

Overburden pressure at SPT elevation $\sigma_5 := \sigma_4 + 2 \cdot ft \cdot (120 \cdot pcf - \gamma_w) + 3 \cdot ft \cdot ((120 \cdot pcf - \gamma_w))$

$$\sigma_5 = 1445.6 \cdot psf$$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_5 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_5}\right)$$

Should not exceed 2.0

$$CN_5 = 1.11$$

$$Ncor1 := CN_5 \cdot N_2$$

$$Ncor1 = 35.531$$

FHWA NHI-06-088 Figure 7-7, Curve for silty SAND

Bearing Capacity Index $C_5 := 87$

Layer $H_5 := 5 \cdot ft$

Effective overburdent stress at midpoint of layer

$$\sigma'_5 := \sigma'_4 + [2.5 \cdot ft \cdot (120 \cdot pcf - \gamma_w) + 2.5 \cdot ft \cdot ((120 \cdot pcf - \gamma_w))]$$

$$\sigma'_5 = 1416.8 \cdot psf$$

Settlement

$$\Delta H_5 := \overrightarrow{\left[H_5 \cdot \frac{1}{C_5} \cdot \log\left[\frac{(\sigma'_5) + \Delta\sigma z_3}{\sigma'_5}\right]\right]}$$

$$\Delta H_5 = 0.22 \cdot in$$

Layer 5

Field SPT (bpf) from 19-21 ft bgs) $N_3 = 119$ use d=20 ft for calculation below

Overburden pressure at SPT elevation $\sigma_6 := \sigma_5 + 2.0\cdot ft\cdot(120\cdot pcf - \gamma_w) + 3\cdot ft\cdot(125\cdot pcf - \gamma_w)$

$\sigma_6 = 1748.6\cdot psf$

N - value correction for overburden per LRFD 10.4.6.2.4

$CN_6 := 0.77\cdot \log\left(\frac{40\cdot ksf}{\sigma_6}\right)$ Should not exceed 2.0

$CN_6 = 1.047$

$Ncor1 := CN_6\cdot N_3$

$Ncor1 = 124.559$

Figure 7-7 Curve for inorganic SILT

Bearing Capacity Index $C_6 := 160$

Layer $H_6 := 5\cdot ft$

Effective overburdent stress at midpoint of layer d=19.5 ft bgs

$\sigma'_6 := \sigma'_5 + [2.5\cdot ft\cdot(120\cdot pcf - \gamma_w) + 2.5\cdot ft\cdot((125\cdot pcf - \gamma_w))]$

$\sigma'_6 = 1717.3\cdot psf$

Settlement

$$\Delta H_6 := \overrightarrow{\left[H_6\cdot\frac{1}{C_6}\cdot\log\left[\frac{(\sigma'_6) + \Delta\sigma z_4}{\sigma'_6}\right]\right]}$$

$\Delta H_6 = 0.1\cdot in$

Layer 6

Field SPT (bpf) $N_4 = 84$ at d = 25 ft bgs

Overburden pressure at SPT elevation $\sigma_7 := \sigma_6 + 2.\cdot ft\cdot(125\cdot pcf - \gamma_w) + 3\cdot ft\cdot((125\cdot pcf - \gamma_w))$

$$\sigma_7 = 2061.6 \cdot psf$$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_7 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_7}\right)$$ Should not exceed 2.0

$$CN_7 = 0.992$$

$$Ncor1 := CN_7 \cdot N_4$$

$$Ncor1 = 83.298$$

NHI-06-088 Figure 7-7, Curve for silty SAND

Bearing Capacity Index $C_7 := 200$

Layer $H_7 := 5 \cdot ft$

Effective overburdent stress at midpoint of layer

d = 24.4 ft bgs

$$\sigma'_7 := \sigma'_6 + \left[2.5 \cdot ft \cdot \left(125 \cdot pcf - \gamma_w\right) + 2.5 \cdot ft \cdot \left(\left(125 \cdot pcf - \gamma_w\right)\right)\right]$$

$$\sigma'_7 = 2030.3 \cdot psf$$

Settlement

$$\Delta H_7 := \overrightarrow{\left[H_7 \cdot \frac{1}{C_7} \cdot \log\left[\frac{\left(\sigma'_7\right) + \Delta\sigma z_5}{\sigma'_7}\right]\right]}$$

$$\Delta H_7 = 0.067 \cdot in$$

Total Elastic Settlement

$$\Delta H_T := \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5 + \Delta H_6 + \Delta H_7$$

$$\boxed{\Delta H_T = 1.337 \cdot in}$$

$\text{Load} := 13.0\cdot\text{ft}\cdot 125\cdot\text{pcf}$ $\qquad$ $\text{Load} = 1625\cdot\text{psf}$

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Stress due to filling in existing west bridge with 13–ft high fill embankment
at Station 6+00

³ Embank. slope a = 25.00(ft)
³ Embank. width b = 72.00(ft)
³ p load/unit area = 1625.00(psf)

³ INCREMENT OF STRESSES FOR Z-DIRECTION
³ X = 50.00(ft)

	Z (ft)	Vertical Stress (psf)		
³	1.00	1624.96	³	
³	1.50	1624.86	³	Layer 1
³	2.00	1624.68	³	
³	2.50	1624.37	³	
³	3.00	1623.92	³	
³	3.50	1623.30	³	
³	4.00	1622.49	³	
³	4.50	1621.45	³	
³	5.00	1620.19	³	Layer 2
³	5.50	1618.66	³	
³	6.00	1616.87	³	
³	6.50	1614.79	³	
³	7.00	1612.43	³	
³	7.50	1609.76	³	
³	8.00	1606.79	³	
³	8.50	1603.50	³	
³	9.00	1599.91	³	
³	9.50	1596.01	³	Layer 3
³	10.00	1591.79	³	
³	10.50	1587.28	³	
³	11.00	1582.46	³	
³	11.50	1577.35	³	
³	12.00	1571.96	³	
³	12.50	1566.30	³	
³	13.00	1560.37	³	
³	13.50	1554.19	³	
³	14.00	1547.77	³	
³	14.50	1541.11	³	Layer 4
³	15.00	1534.24	³	
³	15.50	1527.16	³	
³	16.00	1519.89	³	
³	16.50	1512.43	³	
³	17.00	1504.81	³	
³	17.50	1497.03	³	
³	18.00	1489.11	³	
³	18.50	1481.05	³	
³	19.00	1472.87	³	
³	19.50	1464.58	³	Layer 5
³	20.00	1456.19	³	
³	20.50	1447.72	³	
³	21.00	1439.16	³	
³	21.50	1430.53	³	
³	22.00	1421.85	³	
³	22.50	1413.11	³	
³	23.00	1404.32	³	
³	23.50	1395.50	³	
³	24.00	1386.66	³	
³	24.50	1377.79	³	Layer 6
³	25.00	1368.91	³	
³	25.50	1360.02	³	

Calculation of Elastic Settlement due to raise in grade of 2 ft at Abutment 1 - Soil profile based on strata encountered at BB-NSR-201

Soil properties & groundwater conditons; unit weight per LRFD 3.5.1-1

$\gamma_t := 120 \cdot pcf \qquad \gamma_w := 62.4 \cdot pcf \qquad \gamma' := \gamma_t - \gamma_w \qquad \gamma' = 57.6 \cdot pcf \qquad D_w := 19 \cdot ft$

N values already corrected for hammer efficiency

$$N := \begin{pmatrix} 38 \\ 18 \\ 32 \\ 119 \\ 84 \end{pmatrix}$$

Drained friction angles per LRFD 10.4.6.2.4-1

N160	ϕ
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Soil Profile at BB-NSR-201

Existing Approach Fills - not sampled in Boring BB-NRS-201
assume 10 feet of silty sand, some gravel
First Layer- alluvium and riverbottom sediments
0-3.5 feet, gravelly sand, some silt, roots cobbles. H=3 feet
Second Stratum - alluvium
3.5 - 12.5 feet, sandy gravel, little silt, occassional cobble (alluvium). H=9 ft
Third Stratum - glacial till
12.5 to 27 feet bgs - H=15 ft
silty sand, some gravel
sandy silt, some gravel
sand, some silt, some gravel, little clay

Settlement Computation for Cohesionless Soils

Reference : FHWA Soils and Foundation Workshop Manual NHI-06-088, 2006

Existing Vertical Overburden Stress and Change in vertical stress due to 2 foot raise in bridge approach embankment at Abutment 1

See last sheet for STRESS output for change in stress.

Break soil profile into seven layers:

Layer 1 - 10 feet of existing fill, 120 pcf, 32 degrees
Layer 2 - 3 feet of alluvium w/ riverbottom sediments, 120 pcf, 32 degrees
Layer 3 - 4 feet of alluvium, 125 pcf, 36 degrees
Layer 4 - 5 feet of alluvium, 120 pcf 30 degrees
Layer 5 - 5 feet of till, 120 pcf, 32 degrees
Layer 6 - 5 feet of till, 125 pcf, 38 degrees
Layer 7 - 5 feet of till, 125 pcf, 38 degrees

The change in stresses below are at the center of each layer

$$\Delta\sigma z := \begin{pmatrix} 239.58 \\ 235.6 \\ 231.27 \\ 223.70 \\ 213.48 \\ 202.29 \\ 190.89 \end{pmatrix} \cdot psf$$

z (depth) of midpoint
(feet)

5
11.5
15
19.5
24.5
29.5
34.5

<u>Layer 1</u>

Overburden pressure at midpoint of 10-ft of fill $\sigma'_o := 120 \cdot pcf \cdot 5 \cdot ft$

$\sigma'_o = 600 \cdot psf$

No SPT information, assume corrected N value of 15 bpf and 32 degrees

Curve for "Well graded fine to medium silty SAND"

Based on Figure 7-7 of FHWA NHI-06-088:

Bearing Capacity Index $C_1 := 50$

Layer $H_1 := 10 \cdot ft$

Settlement

$$\Delta H_1 := \overrightarrow{\left[H_1 \cdot \frac{1}{C_1} \cdot \log\left[\frac{(\sigma'_o) + \Delta\sigma z_0}{\sigma'_o}\right]\right]}$$

$$\Delta H_1 = 0.35 \cdot in$$

Layer 2

Boring BB-NSR-201 was drilled in front of abutment 1 - will need to add overburdern pressure of existing 10 feet of approach fill to all calcluations of overburden pressure but **not** overburden calculations associated with correction of SPT for overburden pressure.

No Field SPT (bpf) use N=15 $N1 := 15$

If SPT at 1-3 feet $\sigma_2 := 2 \cdot ft \cdot 120 \cdot pcf$ at 2 ft bgs

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_2 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_2}\right)$$

Should not exceed 2.0

$$CN_2 = 1.711$$

$$Ncor1 := CN_2 \cdot N1$$

$$Ncor1 = 25.662$$

FHWA NHI-06-088, Figure 7-7, Curve for Well graded fine to medium silty SAND

Bearing Capacity Index $C_2 := 70$

Layer $H_2 := 3 \cdot ft$

Effective overburden stress at midpoint of layer

at depth of 11.5 ft bgs

$$\sigma'_2 := 120 \cdot pcf \cdot 10 \cdot ft + 1.5 \cdot ft \cdot 120 \cdot pcf$$

$$\sigma'_2 = 1380 \cdot psf$$

Settlement

$$\Delta H_2 := \overrightarrow{\left[H_2 \cdot \frac{1}{C_2} \cdot \log\left[\frac{(\sigma'_2) + \Delta\sigma z_1}{\sigma'_2} \right] \right]}$$

$$\Delta H_2 = 0.035 \cdot in$$

Layer 3

Field SPT (bpf) $N_0 = 38$

Overburden pressure at SPT elevation $\sigma_3 := 3.0 \cdot ft \cdot 120 \cdot pcf + 3 \cdot ft \cdot 125 \cdot pcf$ 6 ft bgs

$\sigma_3 = 735 \cdot psf$

N - value correction for overburdent per LRFD 10.4.6.2.4

$$CN_3 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_3} \right)$$ Should not exceed 2.0

$$CN_3 = 1.337$$

$$Ncor1 := CN_3 \cdot N_0$$

$$Ncor1 = 50.789$$

NHI-08-088, Figure 7-7, Curve for Well graded silty SAND & GRAVEL

Bearing Capacity Index $C_3 := 173$

Layer $H_3 := 4 \cdot ft$

Effective overburden stress at midpoint of layer $\sigma'_3 := \sigma'_2 + 1.5 \cdot ft \cdot 120 \cdot pcf + 2 \cdot ft \cdot 125 \cdot pcf$

z = 15 ft

$\sigma'_3 = 1810 \cdot psf$

Settlement

$$\Delta H_3 := \overrightarrow{\left[H_3 \cdot \frac{1}{C_3} \cdot \log\left[\frac{(\sigma'_3) + \Delta\sigma z_2}{\sigma'_3} \right] \right]}$$

$$\Delta H_3 = 0.014 \cdot in$$

Layer 4

Field SPT (bpf) d = 10 ft bgs $N_1 = 18$

Note : groundwater at the middle of this 5 foot thick layer

Overburden pressure at SPT elevation $\sigma_4 := \sigma_3 + 1\cdot ft\cdot 125\cdot pcf + 2\cdot ft\cdot 120\cdot pcf + 1\cdot ft\cdot\left[120(pcf) - \gamma_w\right]$

$\sigma_4 = 1157.6\cdot psf$

N - value correction for overburden per LRFD 10.4.6.2.4

$CN_4 := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_4}\right)$ Should not exceed 2.0

$CN_4 = 1.185$

$Ncor1 := CN_4\cdot N_1$

$Ncor1 = 21.324$

NHI-06-088, Figure 7-7, Curve for Well graded silty SAND & GRAVEL

Bearing Capacity Index $C_4 := 77$

Layer $H_4 := 5\cdot ft$

Effective overburdent stress at midpoint of layer z = 19.5 ft

$\sigma'_4 := \sigma'_3 + 2\cdot ft\cdot 125\cdot pcf + 2\cdot ft\cdot 120\cdot pcf + 0.5\cdot ft\cdot\left(120\cdot pcf - \gamma_w\right)$

$\sigma'_4 = 2328.8\cdot psf$

Settlement

$$\Delta H_4 := \overrightarrow{\left[H_4\cdot\frac{1}{C_4}\cdot\log\left[\frac{(\sigma'_4) + \Delta\sigma z_3}{\sigma'_4}\right]\right]}$$

$\Delta H_4 = 0.031\cdot in$

Layer 5

Field SPT (bpf) $N_2 = 32$ at 15 ft bgs (SPT 14-16 ft)

Overburden pressure at SPT elevation $\sigma_5 := \sigma_4 + 2\cdot ft\cdot(120\cdot pcf - \gamma_w) + 3.0\cdot ft\cdot((120\cdot pcf - \gamma_w))$

$\sigma_5 = 1445.6\cdot psf$

N - value correction for overburden per LRFD 10.4.6.2.4

$CN_5 := 0.77\cdot \log\left(\frac{40\cdot ksf}{\sigma_5}\right)$ Should not exceed 2.0

$CN_5 = 1.11$

$Ncor1 := CN_5\cdot N_2$

$Ncor1 = 35.531$

FHWA NHI-06-088 Figure 7-7, Curve for silty SAND

Bearing Capacity Index $C_5 := 87$

Layer $H_5 := 5\cdot ft$

Effective overburdent stress at midpoint of layer

$\sigma'_5 := \sigma'_4 + [2.5\cdot ft\cdot(120\cdot pcf - \gamma_w) + 2.5\cdot ft\cdot((120\cdot pcf - \gamma_w))]$

$\sigma'_5 = 2616.8\cdot psf$

Settlement

$$\Delta H_5 := \overrightarrow{\left[H_5\cdot\frac{1}{C_5}\cdot\log\left[\frac{(\sigma'_5) + \Delta\sigma z_4}{\sigma'_5}\right]\right]}$$

$\Delta H_5 = 0.023\cdot in$

Layer 6

Field SPT (bpf) $N_3 = 119$ SPT from 19-20.5 use z = 20 ft

Overburden pressure at SPT elevation

$$\sigma_6 := \sigma_5 + 2.0 \cdot ft \cdot (120 \cdot pcf - \gamma_w) + 3 \cdot ft \cdot (125 \cdot pcf - \gamma_w)$$

$$\sigma_6 = 1748.6 \cdot psf$$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_6 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_6}\right)$$

Should not exceed 2.0

$$CN_6 = 1.047$$

$$Ncor1 := CN_6 \cdot N_3$$

$$Ncor1 = 124.559$$

Figure 7-7 Curve for inorganic SILT

Bearing Capacity Index $C_6 := 160$

Layer $H_6 := 5 \cdot ft$

Effective overburdent stress at midpoint of layer

$$\sigma'_6 := \sigma'_5 + \left[2.5 \cdot ft \cdot (120 \cdot pcf - \gamma_w) + 2.5 \cdot ft \cdot ((125 \cdot pcf - \gamma_w))\right]$$

$$\sigma'_6 = 2917.3 \cdot psf$$

Settlement

$$\Delta H_6 := \overrightarrow{\left[H_6 \cdot \frac{1}{C_6} \cdot \log\left[\frac{(\sigma'_6) + \Delta\sigma z_5}{\sigma'_6}\right]\right]}$$

$$\Delta H_6 = 0.011 \cdot in$$

Layer 7

Field SPT (bpf) $N_4 = 84$

Overburden pressure at SPT elevation SPT from 24-25 ft bgs, use z = 25 ft

$$\sigma_7 := \sigma_6 + 2 \cdot ft \cdot (125 \cdot pcf - \gamma_w) + 3 \cdot ft \cdot ((125 \cdot pcf - \gamma_w))$$

$$\sigma_7 = 2061.6 \cdot psf$$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_7 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_7}\right)$$

Should not exceed 2.0

$$CN_7 = 0.992$$

$$Ncor1 := CN_7 \cdot N_4$$

$$Ncor1 = 83.298$$

NHI-06-088 Figure 7-7, Curve for silty SAND

Bearing Capacity Index $C_7 := 200$

Layer $H_7 := 5 \cdot ft$

Effective overburdent stress at midpoint of layer

$$\sigma'_7 := \sigma'_6 + \left[2.5 \cdot ft \cdot \left(125 \cdot pcf - \gamma_w\right) + 2.5 \cdot ft \cdot \left(\left(125 \cdot pcf - \gamma_w\right)\right)\right]$$

$$\sigma'_7 = 3230.3 \cdot psf$$

Settlement

$$\Delta H_7 := \overrightarrow{\left[H_7 \cdot \frac{1}{C_7} \cdot \log\left[\frac{(\sigma'_7) + \Delta\sigma z_6}{\sigma'_7}\right]\right]}$$

$$\Delta H_7 = 0.007 \cdot in$$

Total Elastic Settlement

$$\Delta H_T := \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5 + \Delta H_6 + \Delta H_7$$

$$\boxed{\Delta H_T = 0.473 \cdot in}$$

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

³ Project Name : Newport Main St. Br. Client: MaineDOT Bridge
³ Project Number : 15625.00 Project Manager : D. Anderson
³ Date: 09/09/09 Computed by : LK

³ Embank. slope a = 35.00(ft)
³ Embank. width b = 85.00(ft)
³ p load/unit area = 240.00(psf)
³

INCREMENT OF STRESSES FOR Z-DIRECTION - **STATION 7+50**
³ X = 55.00(ft)
³

Z (ft)	Vertical Stress (psf)	
1.00	240.00	
2.00	239.97	
3.00	239.91	
4.00	239.78	
5.00	239.58	Layer 1, z=5' (Fill)
6.00	239.29	
7.00	238.89	
8.00	238.38	
9.00	237.75	
10.00	236.99	Ground Surface for overburden correction of SPT values
11.00	236.10	
12.00	235.08	Layer 2, z=11.5'
13.00	233.93	
14.00	232.66	
15.00	231.27	Layer 3, z=15'
16.00	229.76	
17.00	228.15	
18.00	226.44	
19.00	224.64	
20.00	222.76	Layer 4, z=19.5' (Groundwater at middle of Layer 4)
21.00	220.80	
22.00	218.78	
23.00	216.70	
24.00	214.57	
25.00	212.40	Layer 5, z=24.5
26.00	210.19	
27.00	207.96	
28.00	205.70	
29.00	203.43	
30.00	201.15	Layer 6, z=29.5
31.00	198.87	
32.00	196.58	
33.00	194.30	
34.00	192.02	
35.00	189.76	Layer 7, z=34.5'
36.00	187.50	
37.00	185.27	

Calculation of Elastic Settlement due to filling in between exisiting pier 3 and Abutment 2 - Soil profile based on strata encountered at BB-NSR-102

Soil Properties & Groundwater conditions- unit weight per LRFD 3.5.1-1

$\gamma_t := 120 \cdot pcf$ $\quad \gamma_w := 62.4 \cdot pcf$ $\quad \gamma' := \gamma_t - \gamma_w$ $\quad \gamma' = 57.6 \cdot pcf$ $\quad D_w := 2 \cdot ft$

N values already corrected for hammer efficiency

$$N := \begin{pmatrix} 17 \\ 8 \\ 47 \\ 54 \\ 99 \\ 135 \end{pmatrix}$$

Drained friction angles per LRFD 10.4.6.2.4-1

N160	ϕ
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Soil Profile at BB-NSR-102:

Proposed Approach Fills
13 feet of granular borrow

First Layer- alluvium and riverbottom sediments
0-2 feet, gravel N=17
3 feet, silt to sand N=8
4 feet, gravelly sand, little silt N=47

Second Stratum - alluvium
5 feet, sand, some gravel, little silt, N=54

Third Stratum - glacial till
6 feet - silty sand, tr. gravel to sand, some silt, little gravel N=99
5 ft - gravelly silt and silty sand some gravel, N=135

Settlement Computation for Cohesionless Soils

Reference : FHWA Soils and Foundation Workshop Manual NHI-06-088, 2006

Existing Vertical Overburden Stress and Change in vertical stress due to 13 feet of fill between existing pier 3 and existing abutment 2

See last sheet for STRESS output for change in stress.

Break soil profile into six layers.

Layer 1 - 2 feet of alluvium with riverbottom sediments, 120 pcf, 32 degrees
Layer 2 - 3 feet of alluvium w/ riverbottom sediments, 115 pcf, 27 degrees
Layer 3 - 4 feet of alluvium w/ riverbottom sediments, 120 pcf, 32 degrees
Layer 4 - 5 feet of alluvium, 125 pcf 34 degrees
Layer 5 - 6 feet of till, 125 pcf, 34 degrees
Layer 6 - 5 feet of till, 125 pcf, 34 degrees

The change in stresses below are at the center of each layer

$$\Delta\sigma z := \begin{pmatrix} 1559.96 \\ 1558.14 \\ 1547.06 \\ 1511.56 \\ 1440.4 \\ 1352.44 \end{pmatrix} \cdot psf$$

z - direction, depth (ft)

1.0
3.5
7.0
11.5
17
22.5

Layer 1

Overburden presure for overburden correction of SPT N-value $\sigma'_1 := 120 \cdot pcf \cdot 1 \cdot ft$

Field SPT (bpf) $N_0 = 17$ at z = 1 ft

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_1 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma'_1}\right)$$ Should not exceed 2.0

$$CN_1 = 1.943$$

$$Ncor1 := CN_1 \cdot N_0$$

$$Ncor1 = 33.024$$

FHWA NHI-06-088 Figure 7-7, Curve for SAND and GRAVEL

Bearing Capacity Index $C_1 := 110$

Layer $H_1 := 2 \cdot ft$

Settlement

since σ' is < 200 psf, override σ'1 with 200 psf per FHWA NHI-06-088 page 7-16

$\sigma'_1 := 200 \cdot psf$

$$\Delta H_1 := \overrightarrow{\left[H_1 \cdot \frac{1}{C_1} \cdot \log\left[\frac{(\sigma'_1) + \Delta\sigma z_0}{\sigma'_1} \right] \right]}$$

$\Delta H_1 = 0.206 \cdot in$

Layer 2

Field SPT (bpf) $N_1 = 8$ at SPT interval 2-4 ft, use z=3 ft

Overburden presure for overburden correction of SPT N-value

$$\sigma'_2 := 2 \cdot ft \cdot 120 \cdot pcf + 1 \cdot ft \cdot (115 \cdot pcf - \gamma_w)$$

$$\sigma'_2 = 292.6 \cdot psf$$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_2 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma'_2} \right)$$

Should not exceed 2.0

$$CN_2 = 1.645$$

$$Ncor1 := CN_2 \cdot N_1$$

$$Ncor1 = 13.156$$

FHWA NHI-06-088 Figure 7-7, Curve for SILT

Bearing Capacity Index $C_2 := 32$

Layer $H_2 := 3 \cdot ft$

Effective overburden stress at midpoint of layer z = 3.5 ft

$$\sigma'_2 := 2 \cdot ft \cdot 120 \cdot pcf + 1.5 \cdot ft \cdot (115 \cdot pcf - \gamma_w)$$

$$\sigma'_2 = 318.9 \cdot psf$$

Settlement

$$\Delta H_2 := \overrightarrow{\left[H_2 \cdot \frac{1}{C_2} \cdot \log\left[\frac{(\sigma'_2) + \Delta\sigma z_1}{\sigma'_2}\right]\right]}$$

$$\Delta H_2 = 0.866 \cdot in$$

Layer 3

Field SPT (bpf) $N_2 = 47$ at SPT interval 5-7 ft, z=6 ft bgs

Overburden pressure at SPT elevation

$$\sigma_3 := (\sigma'_2) + 1.5 \cdot ft \cdot (115 \cdot pcf - \gamma_w) + 1 \cdot ft \cdot (120 \cdot pcf - \gamma_w)$$

$$\sigma_3 = 455.4 \cdot psf$$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_3 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_3}\right)$$ Should not exceed 2.0

$$CN_3 = 1.497$$

$$Ncor1 := CN_3 \cdot N_2$$

$$Ncor1 = 70.341$$

FHWA NHI-06-088 Figure 7-7 - Curve for Well graded silty SAND & GRAVEL

Bearing Capacity Index $C_3 := 265$

Layer $H_3 := 4 \cdot ft$

Effective overburden stress at midpoint of layer

$$\sigma'_3 := \sigma'_2 + 1.5 \cdot ft \cdot (115 \cdot pcf - \gamma_w) + 2 \cdot ft \cdot (120 \cdot pcf - \gamma_w)$$

$$\sigma'_3 = 513 \cdot psf$$

Settlement

$$\Delta H_3 := \overrightarrow{\left[H_3 \cdot \frac{1}{C_3} \cdot \log\left[\frac{(\sigma'_3) + \Delta\sigma z_2}{\sigma'_3}\right]\right]}$$

$\Delta H_3 = 0.109 \cdot in$

Layer 4

Field SPT (bpf) $N_3 = 54$ SPT interval 10-12 ft, use z = 11 ft

Overburden pressure at SPT elevation (11'bgs) $\sigma_4 := \sigma'_3 + 2 \cdot ft \cdot (120 \cdot pcf - \gamma_w) + 2 \cdot ft \cdot (125 \cdot pcf - \gamma_w)$

$\sigma_4 = 753.4 \cdot psf$

N - value correction for overburdent per LRFD 10.4.6.2.4 Should not exceed 2.0

$$CN_4 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_4}\right)$$

$CN_4 = 1.328$

$Ncor1 := CN_4 \cdot N_3$

$Ncor1 = 71.727$

NHI-06-088 Figure 7.7, Curve for well graded fine to coarse SAND

Bearing Capacity Index $C_4 := 210$

Layer $H_4 := 5 \cdot ft$

Effective overburden stress at midpoint of layer $\sigma'_4 := \sigma'_3 + 2 \cdot ft \cdot (120 \cdot pcf - \gamma_w) + 2.5 \cdot ft \cdot (125 \cdot pcf - \gamma_w)$

$\sigma'_4 = 784.7 \cdot psf$

Settlement

$$\Delta H_4 := \overrightarrow{\left[H_4 \cdot \frac{1}{C_4} \cdot \log\left[\frac{(\sigma'_4) + \Delta\sigma z_3}{\sigma'_4}\right]\right]}$$

$\Delta H_4 = 0.133 \cdot in$

Layer 5

Field SPT (bpf) $N_4 = 99$ at z=16 ft bgs

Overburden pressure at SPT elevation $\sigma_5 := \sigma'_4 + 2.5\cdot ft\cdot(125\cdot pcf - \gamma_w) + 2\cdot ft\cdot((125\cdot pcf - \gamma_w))$

$\sigma_5 = 1066.4\cdot psf$

N - value correction for overburdent per LRFD 10.4.6.2.4

$$CN_5 := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_5}\right)$$ Should not exceed 2.0

$CN_5 = 1.212$

$Ncor1 := CN_5\cdot N_4$

$Ncor1 = 119.997$

NHI-06-088 Figure 7-7 Curve for fine to medium silty SAND

Bearing Capacity Index $C_5 := 250$

Layer $H_5 := 6\cdot ft$

Effective overburdent stress at midpoint of layer $\sigma'_5 := \sigma'_4 + 2.5\cdot ft\cdot(125\cdot pcf - \gamma_w) + 3\cdot ft\cdot(125\cdot pcf - \gamma_w)$

$\sigma'_5 = 1129\cdot psf$

Settlement

$$\Delta H_5 := \overrightarrow{\left[H_5\cdot\frac{1}{C_5}\cdot\log\left[\frac{(\sigma'_5) + \Delta\sigma z_4}{\sigma'_5}\right]\right]}$$

$\Delta H_5 = 0.103\cdot in$

Layer 6

Field SPT (bpf) $N_5 = 135$ at z = 21 ft

Overburden pressure at SPT elevation $\sigma_6 := (\sigma'_5) + 3\cdot ft\cdot(125\cdot pcf - \gamma_w) + 1\cdot ft\cdot(125\cdot pcf - \gamma_w)$

$\sigma_6 = 1379.4\cdot psf$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_6 := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_6}\right)$$ Should not exceed 2.0

$$CN_6 = 1.126$$

$$Ncor1 := CN_6 \cdot N_5$$

$$Ncor1 = 152.013$$

FHWA NHI-06-088 Figure 7-7 Curve for inorganic SILT

Bearing Capacity Index $C_6 := 160$

Layer $H_6 := 5 \cdot ft$

Effective overburden stress at midpoint of layer

$$\sigma'_6 := \sigma'_5 + 3 \cdot ft \cdot (125 \cdot pcf - \gamma_w) + 2.5 \cdot ft \cdot (125 \cdot pcf - \gamma_w)$$

$$\sigma'_6 = 1473.3 \cdot psf$$

Settlement

$$\Delta H_6 := \overrightarrow{\left[H_6 \cdot \frac{1}{C_6} \cdot \log\left[\frac{(\sigma'_6) + \Delta\sigma z_5}{\sigma'_6}\right]\right]}$$

$$\Delta H_6 = 0.106 \cdot in$$

Total Elastic settlement

$$\Delta H_T := \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5 + \Delta H_6$$

$$\boxed{\Delta H_T = 1.524 \cdot in}$$

Stress Computer Ouput

³ Project Name: Newport Main St. Br. Client : MaineDOT Bridge
³ Project Number: 15625.00 Project Manager : D. Anderson
³ Date : 09/16/09 Computed by : LK

For Settlement Analysis for Filling in between Pier 1-Abutment 2

³ Embank. slope a = 29.00(ft)
³ Embank. width b = 76.00(ft)
³ p load/unit area = 1560.00(psf)

³ INCREMENT OF STRESSES FOR Z-DIRECTION
³ X = 55.00(ft)

Z (ft)	Vertical Stress Component (psf)	
1.00	1559.96	Layer 1, t=2', γ=120, Φ=32
2.00	1559.66	
3.00	1558.88	
4.00	1557.40	Layer 2, t=3', γ=115, Φ=27
5.00	1555.02	
6.00	1551.61	
7.00	1547.06	Layer 3, t=4', γ=120, Φ=34
8.00	1541.32	
9.00	1534.34	
10.00	1526.15	
11.00	1516.79	
12.00	1506.32	Layer 4, t=5', γ=125, Φ=34
13.00	1494.81	
14.00	1482.37	
15.00	1469.08	
16.00	1455.06	
17.00	1440.40	Layer 5, t=6', γ=125, Φ=36
18.00	1425.20	
19.00	1409.56	
20.00	1393.55	
21.00	1377.26	
22.00	1360.76	
23.00	1344.12	Layer 6, t=5', γ=125, Φ=36
24.00	1327.39	
25.00	1310.63	

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

From Design Freezing Index Map:
Newport, Maine
DFI = 1800 degree-days

Case I - Soils at elevation of possible footings of WC=15% and coarse-grained

Interpolate between frost depth of 90.1 for WC=10% at 1800 DFI and 74.5 inches for WC=20%

Depth of Frost Penetration =

$$d := \frac{90.1 - 74.5}{2} \cdot in + 74.5 \cdot in \qquad d = 82.3 \cdot in \qquad d = 6.858 \cdot ft$$

Method 2 - ModBerg Software

Newport lies on the same Design Freezing Index contour as Madision, Maine, BDG Fig. 5-1

Case 1 - coarse grained soils with water content of 15%

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

Project Location: Madison, Maine

Air Design Freezing Index       =  1847 F-days
N-Factor                        =  0.80
Surface Design Freezing Index   =  1478 F-days
Mean Annual Temperature         =  42.4 deg F
Design Length of Freezing Season =  136 days

---------------------------------------------------------
Layer
#:Type        t    w%     d    Cf  Cu   Kf   Ku    L
---------------------------------------------------------
1-Coarse    87.7 15.0 125.0  31  40   2.9  1.8  2,700
---------------------------------------------------------

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

***********************************************************
  Total Depth of Frost Penetration = 7.30 ft = 87.7 in.
***********************************************************
```

Recommendation: use 7.0 feet for for design for foundations not founded on bedrock

Output Calculations and Ground Motion Maps

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04953
Zip Code Latitude = 44.837100
Zip Code Longitude = -069.272500
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.072	PGA - Site Class B
0.2	0.155	Ss - Site Class B
1.0	0.046	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04953
Zip Code Latitude = 44.837100
Zip Code Longitude = -069.272500
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.116	As - Site Class D
0.2	0.248	SDs - Site Class D
1.0	0.110	SD1 - Site Class D

Abutment and Wingwall Active Earth Pressure

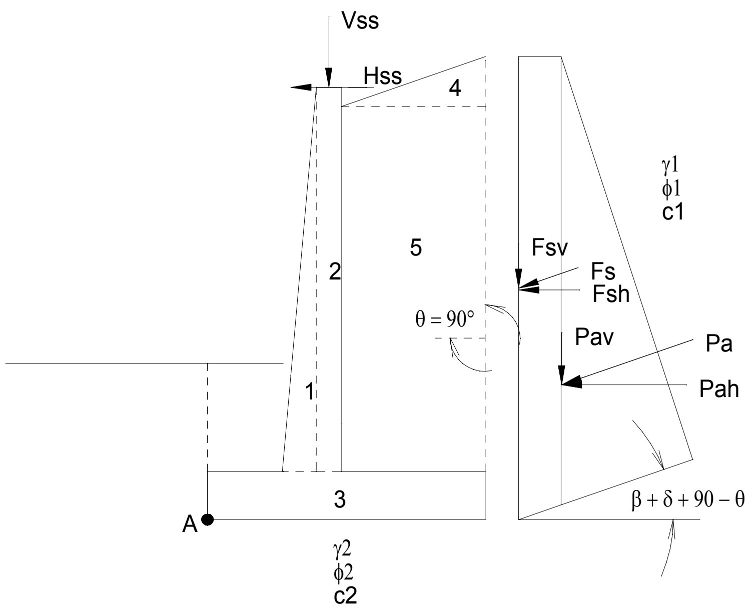
Backfill engineering strength parameters

Soil Type 4 Properties from Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot \text{pcf}$

Internal friction angle $\phi_1 := 32 \cdot \text{deg}$

Cohesion $c_1 := 0 \cdot \text{psf}$



Active Earth Pressure - Rankine Theory

Either Rankine or Coulomb may be used for **long heeled** cantilever walls, where the failure surface is uninterupted by the top of the wall stem. In general, use Rankine though. The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot \text{deg} - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.307$$

- For a sloped backfill

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot \text{deg}$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.307$$

- Pa is oriented at an angle of β to the vertical plane

Coulomb Theory

In general, for cases where the back face of the wall interferes with the development of a full sliding surface in the backfill, as assumed by Rankine Theory, use Coulomb.

- Coulomb theory applies for gravity, semigravity and prefab modular walls with steep back faces
- Coulomb theory also applies to concrete cantilever walls with short heels where the sliding surface in restricted by the top of wall - the wedge of soil does not move.
- Interface friction is considered in Coulomb.

Angle of back face of wall to the horizontal, θ :

$$\theta := 90 \cdot deg$$

Friction angle between fill and wall, δ :

Per LRFD Table 3.11.5.3-1, for "Clean sand, silty sand-gravel mixture, single-size hard rock fill against Formed or precast concrete" δ = 17 to 22 degrees; select 20 degrees.

$\delta := 20 \cdot deg$ for a gravity shaped wall where the interface friction is between soil and concrete

to $\delta := 24 \cdot deg$ per BDG Table 3-3

Per LRFD Figure C3.11.5.3-1, for a cantilever wall where the sliding surface is a plane from the footing heel to the top of the wall, δ=1/3 to 2/3 Φ

$$\delta := \frac{2}{3} \cdot \phi_1$$

$$\delta = 21.333 \cdot deg$$

(If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb)

$$K_{ac} := \frac{\sin(\theta + \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2} \qquad K_{ac} = 0.275$$

Orientation of Coulomb Pa

- In the case of gravity shaped walls and prefab walls, Pa is oriented δ degrees up from a perpendicular line to the backface.
- In the case of short heeled cantilever walls where the top of the wall interferes with the failure surface, Pa is oriented at an angle of $\phi/3$ to $2/3^*\phi$ to the normal of a vertical line extending up from the heel of the wall

Passive Earth Pressure - Rankine Theory

Bowles does not recommend use of Rankine method for Kp when B>0.

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot \text{deg}$

$$K_{pslope} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}$$

$K_{pslope} = 3.255$

Pp is oriented at an angle of β to the vertical plane

Passive Earth Pressure - Coulomb Theory

For cases where the back face of the wall interferes with the development of a full sliding surface in the backfill, as assumed by Rankine Theory.

- Coulomb theory applies for gravity, semigravity and prefab modular walls with steep back faces
- Coulomb theory also applies to concrete cantilever walls with short heels where the sliding surface in restricted by the top of wall - the wedge of soil does not move.

Interface friction is considered in Coulomb.

For a smooth vertical wall with horizontal backfill $\delta = \beta = 0$ and θ = 90 degrees (refer: Bowles, 5th edition, pag 596

θ = Angle of back face of wall to the horizontal

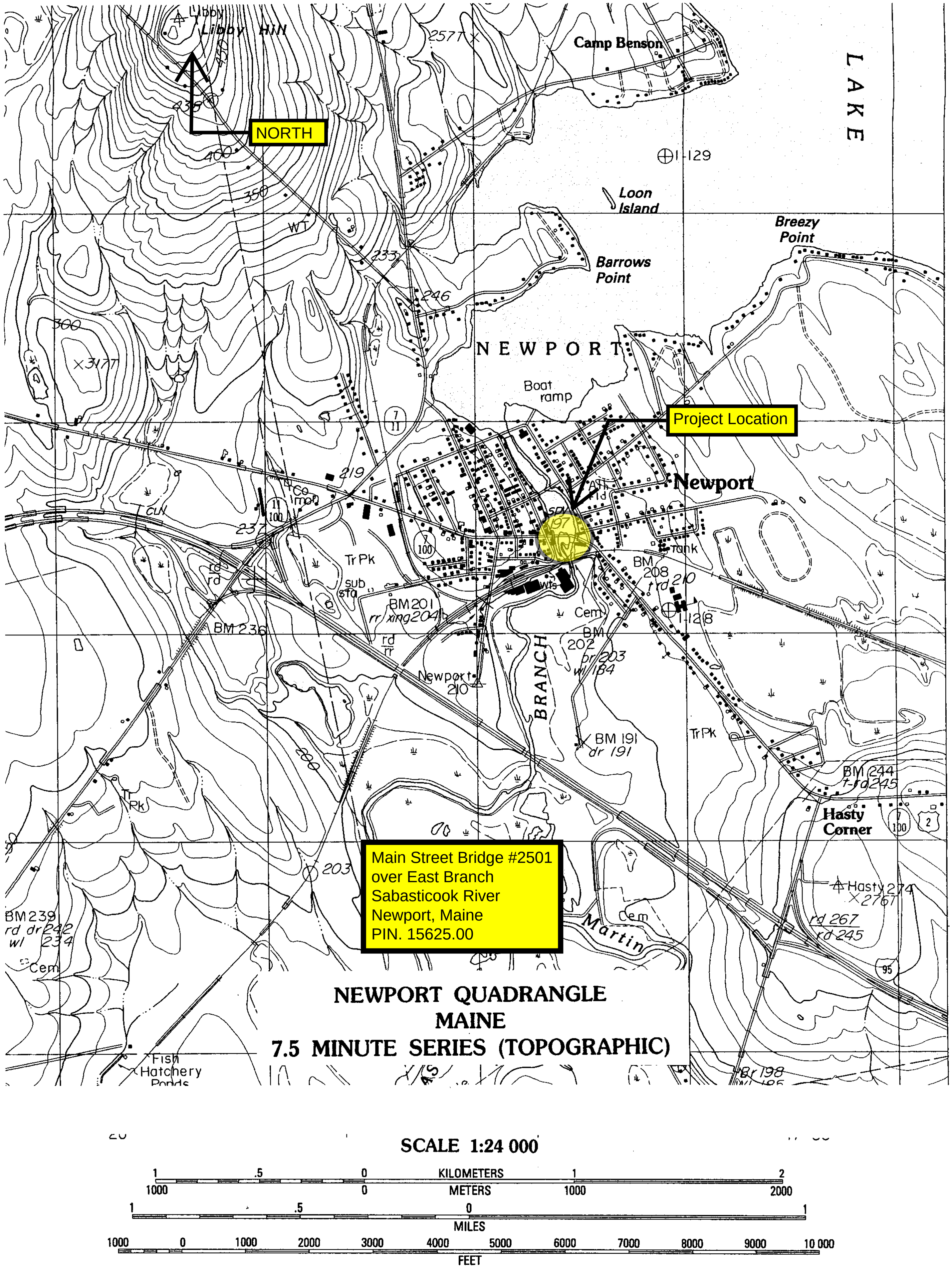
$\theta := 90 \cdot \text{deg}$

δ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

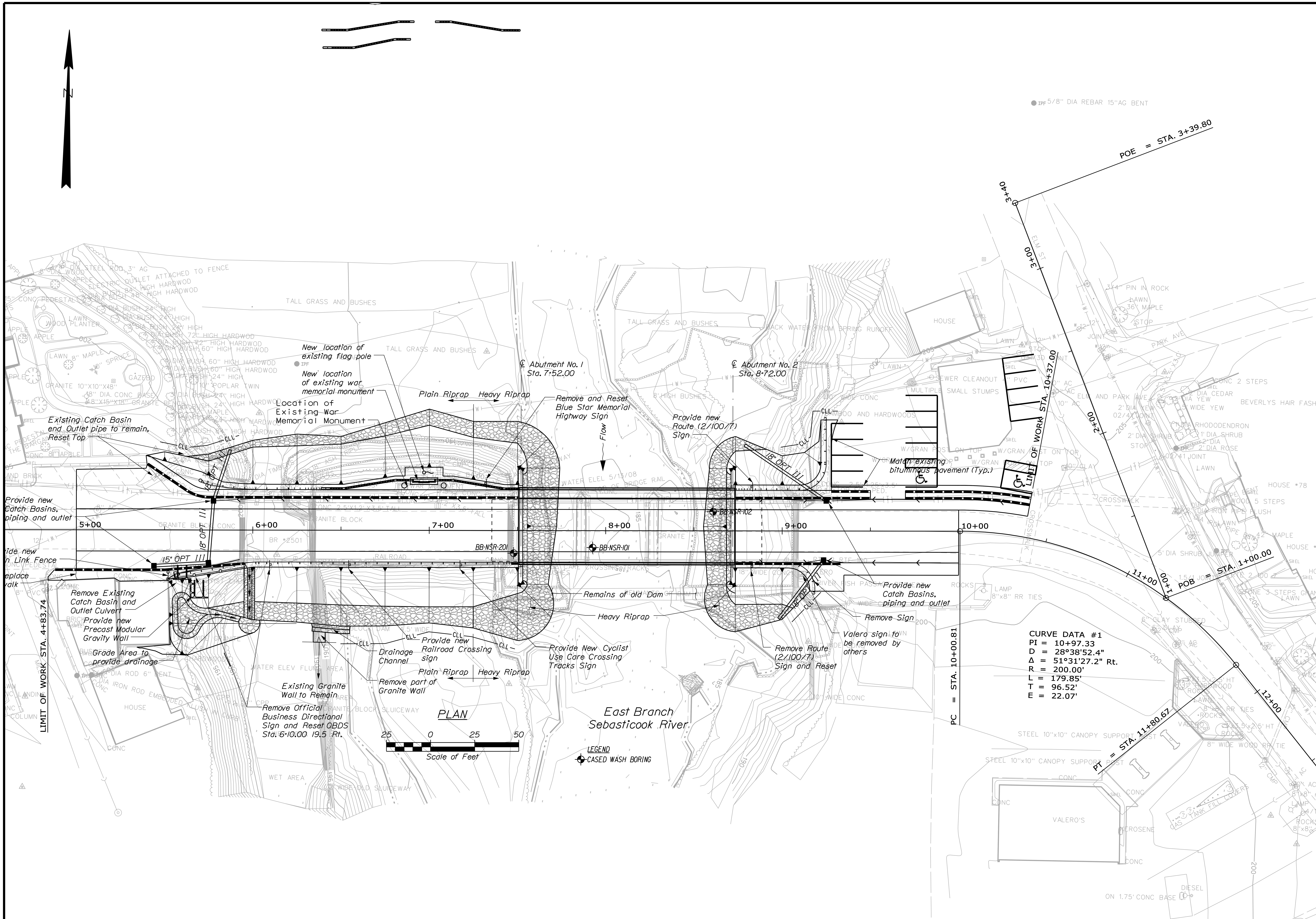
$\delta := \frac{2}{3} \cdot \phi_1$ $\quad$ $\delta = 0.372$

$$K_{pc} := \frac{\sin(\theta - \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 + \beta)}{\sin(\theta + \delta) \cdot \sin(\theta + \beta)}}\right)^2} \qquad K_{pc} = 7.333$$

Sheets


NORTH
Project Location
Main Street Bridge #2501
over East Branch
Sabasticook River
Newport, Maine
PIN. 15625.00
Camp Benson
LAKE
Loon Island
Breezy Point
Barrows Point
NEWPORT
Boat ramp
Newport
Hasty Corner
BRANCH
Martin
NEWPORT QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)
Fish Hatchery Ponds
SCALE 1:24 000
KILOMETERS
METERS
MILES
FEET

Filename: ...\00\geotech\msta\006_BLP1.dgn   Division: GEOTECH   Username: terry.white   Date:10/19/2009



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

15625.00

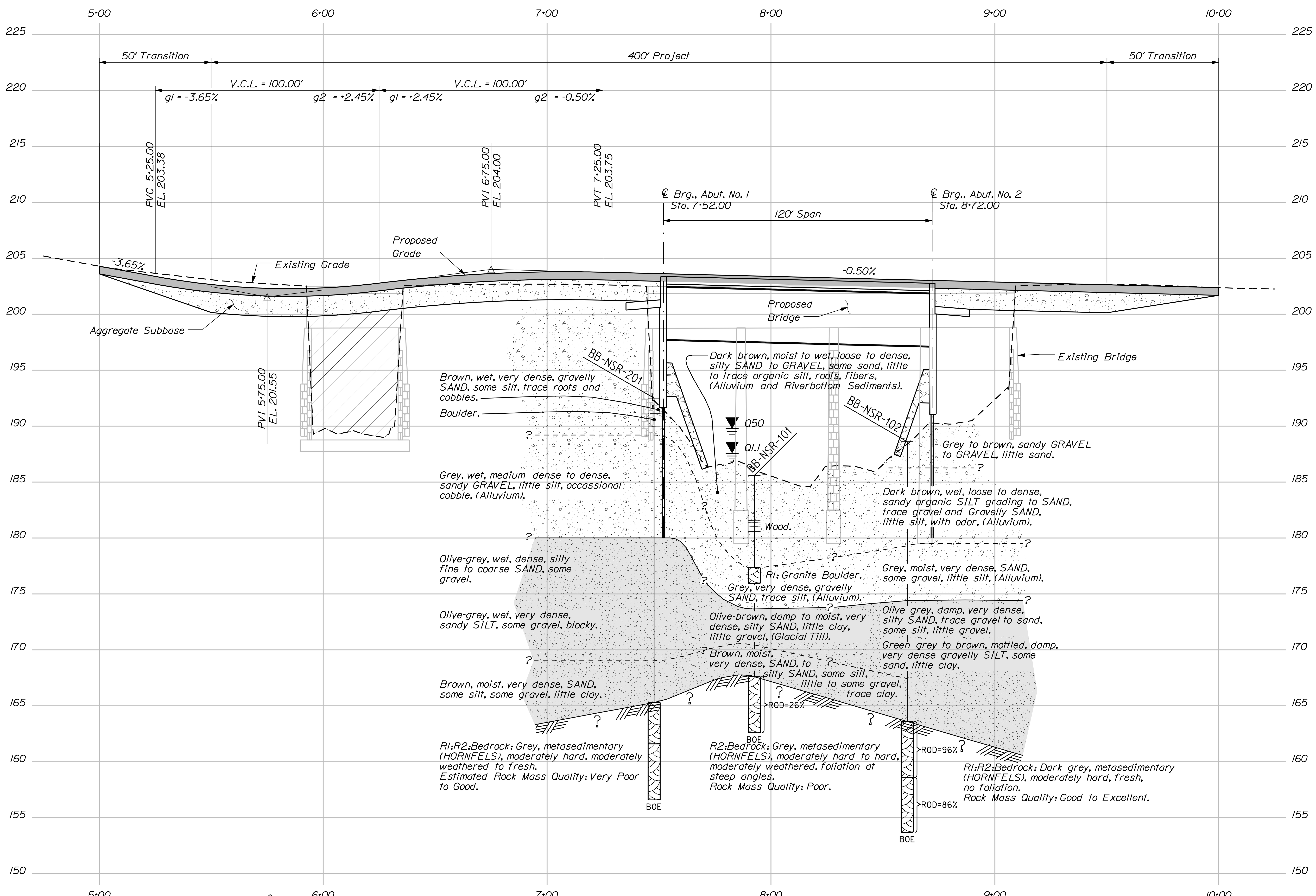
BRIDGE NO. 2501 | PIN 15625.00 | BRIDGE PLANS

	BY	DATE	SIGNATURE
PROJ. MANAGER			
DESIGN-DETAILED	L. KRUSINSKI		
CHECKED-REVIEWED	T. WHITE	JULY 2008	
DESIGN2-DETAILED2			P.E. NUMBER
DESIGN3-DETAILED3			
REVISIONS 1			DATE
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MAIN STREET BRIDGE
EAST BRANCH SEBASTICOOK RIVER
NEWPORT   PENOBSCOT COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4



Boring No.
Offset, if shown
Strata Interface
Boring
ROD= Rock Quality Designation for Rock Core Sample
BOE= Bottom Of Exploration

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.

Filename: ...\00\GEOTECH\MSTA\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:10/20/2009

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

15625.00

BRIDGE NO. 2501 PIN 15625.00 BRIDGE PLANS

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

SIGNATURE
P.E. NUMBER
DATE

MAIN STREET BRIDGE
EAST BRANCH SEBASTICOOK RIVER
NEWPORT PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:10/19/2009

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

Project: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-201
PIN: 15625.00

Driller:	MaineDOT	Elevation (ft.)	192.5	Auger ID/OD:	N/A
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:	6/15/09, 6/25/09	Drilling Method:	Cased Wash Boring	Core Barrel:	BX
Boring Location:	7+47.8, 12.9 Rt.	Casing ID/OD:	NW & HW	Water Level*:	16.0'-6/15, 9.0' 6/25 bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	7.2/5	0.00 - 0.60	1/50(1.2")	---		2			Brown, wet, very dense, gravelly fine to coarse SAND, some silt, roots, cobbles.	
							50			Boulder from 1.4-2.5' bgs. Roller Coned ahead to 5.0' bgs.	
							15				
							OPEN HOLE	189.00		3.50	
5	2D	24/10	5.00 - 7.00	6/11/16/11	27	38	15			Dark grey, wet, dense, sandy GRAVEL, little silt, occasional cobble, slight odor. Switched to NW casing. Roller Coned ahead to 9.0' bgs.	G#212332 A-1-a, GM WC=16.6%
							81				
							142				
							171				
	3D	24/1	9.00 - 11.00	3/7/6/7	13	18	150			Similar to above, except medium dense.	
10							44				
							59				
							64	180.00		12.50	
							51				
	4D	24/8	14.00 - 16.00	19/13/10/18	23	32	37			Olive-grey, wet, dense, silty fine to coarse SAND, some gravel, (Till).	
15							76				
							88				
							125				
							146				
	5D	18/16	19.00 - 20.50	42/33/52	85	119	56			Olive-grey, wet, very dense, fine to coarse sandy SILT, some fine to coarse gravel, blocky Roller Coned ahead to 24.0' bgs.	
20							73				
							99				
							130				
							166	169.50		23.00	
	6D	24/17	24.00 - 26.00	21/31/29/30	60	84	14			Brown, moist, very dense, SAND, some silt, some gravel, little clay, (Till).	G#212333 A-4, CL-ML WC=11.2% LL=22 PL=17 PI=5
25							RC			Roller Coned ahead from 25.0-27.2' bgd.	
	R1	44.4/34	27.20 - 30.90				BX	165.30		27.20 Top of Bedrock at Elev. 165.3'. R1:Bedrock: Grey, fine grained, metasedimentary (HORNFELS) with quartz veins, moderately hard, moderately weathered. Joint breaks at close spacing. Vassalboro Formation. Rock Mass Quality: very poor, based on an estimated NQ RQD of 21%. R1:Core Times (min:sec) 27.2-28.2' (4:30) 28.2-29.2' (3:00) 29.2-30.2' (3:30) 30.2-30.9' (4:12) 76% Recovery Core Blocked	
30	R2	60/60	30.90 - 35.90							R2: Bedrock: Same as R1, except fresh, joint set moderately close to close. Rock Mass Quality: good, based on an estimated NQ RQD of 68%. R2:Core Times (min:sec) 30.9-31.9' (4:25) 31.9-32.9' (4:00) 32.9-33.9' (4:12) 33.9-34.9' (4:35) 34.9-35.9' (4:50) 100% Recovery Could not get Core Barrel back down, casing bent.	
35								156.60		35.90 Bottom of Exploration at 35.90 feet below ground surface.	
40											
45											
50											

Remarks:

11.0' from Bridge Deck to Ground.
Bridge Deck Concrete 12" thick.
Used BX Core Barrel, casing was bent to much to get NQ-2 Core Barrel down hole.

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-NSR-201

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

Project: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-101
PIN: 15625.00

Driller:	MaineDOT	Elevation (ft.)	185.6	Auger ID/OD:	N/A
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	L. Krusinski	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/4/08; 11:00-16:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+92.6, 9.6 Rt.	Casing ID/OD:	NW & HW	Water Level*:	Stream Elev.
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	1D	24/3	0.00 - 2.00	10/8/2/2	10	13	47			Dark brown, moist, medium dense, silty fine to coarse SAND, little roots and fibers, slightly organic, two 1" rock fragments, (River Bottom Sediments).	
							33				
	2D	24/7	2.00 - 4.00	9/22/20/14	42	54	46			Dark brown, wet, very dense, fine angular GRAVEL, some fine to coarse SAND, trace organic silt, few rock fragments, (Alluvium with Riverbottom Sediments).	G#209975 A-1-a, GP-GM WC=11.8%
							604				
							19			Hit wood at 4.0' bgs. Wood in wash water from 4.0-5.0' bgs.	
5	3D	24/3	5.00 - 7.00	3/2/3/8	5	6	6			Brown, wet, loose, medium to coarse SAND, some wood, little silt, trace clay and fine angular gravel. Telescoped NW Casing into HW Casing at 5.0' bgs.	
							26				
							25				
	R1	24/17	8.25 - 10.25	RQD = N/A%			a31 NQ-2	177.35		a31 blows for 3", then 220 blows after coring. 8.25 R1: Granite BOULDER 1.4' thick. R1:Core Times (min:sec) 8.25-9.25' (3:19) 9.25-10.25' (2:38)	
							134 CORE	175.95		9.65	
10							77				
	4D/AB	24/19	11.00 - 13.00	7/16/27/36	43	55	102			(4D/A) 11.0-12.0' bgs. Grey, moist, very dense, angular fine to coarse gravelly SAND, trace silt, (Alluvium). Changed to brown in wash water at 11'3".	G#210011 A-1-a, SW-SM WC=10.7%
							142	173.60		12.00 (4D/B) 12.0-13.0' bgs. Olive-brown, damp to moist, silty fine SAND, some medium sand, little clay, little fine angular gravel, some staining. (Till).	
							255				
							215				
15	5D	24/20	15.00 - 17.00	24/33/43/36	76	98	bRC			bRoller Coned ahead to 18.0' bgs. Brown, moist, very dense, fine to coarse SAND, some silt, little fine to coarse angular gravel, trace clay, well bonded, some staining/ oxidation. (Till).	G#210012 A-2-4, SC-SM WC=9.4%
	R2	60/53	18.00 - 23.00	RQD = 26%			NQ-2 CORE	167.60		18.00 Top of Bedrock at Elev. 167.6 ft. R2: Bedrock: Grey, fine grained, metasedimentary (HORNFELS), moderately hard to hard, moderately weathered, cleaves along folliation at steep angles, tight, weathered and stained surfaces; moderately fractured and weathered zone 3.6-4.2', no foliation or cleavage in upper 10", pegmatite veins in lower 6". Vassalboro Formation. Rock Mass Quality: Poor. R2:Core Times (min:sec) 18.0-19.0' (3:08) 19.0-20.0' (2:26) 20.0-21.0' (2:40) 21.0-22.0' (2:31) 22.0-23.0' (2:40) 88% Recovery	
20											
								162.60		23.00 Bottom of Exploration at 23.00 feet below ground surface.	
25											

Remarks:

Large cobble moved to side at Ground Surface.
16.8' from Bridge Deck to Ground.

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-NSR-101

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

Project: Main Street Bridge #2501 over East Branch Sabasticook River
Location: Newport, Maine

Boring No.: BB-NSR-102
PIN: 15625.00

Driller:	MaineDOT	Elevation (ft.)	188.6	Auger ID/OD:	N/A
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	L. Krusinski	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/3/08-6/4/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	8+60.9, 10.9 Lt.	Casing ID/OD:	NW & HW	Water Level*:	2.0' 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	1D/AB	24/7	0.00 - 2.00	11/10/3/3	13	17	8			(1D/A) Brown, damp, medium dense, sandy GRAVEL.	
							14			(1D/B) Grey, damp, angular coarse GRAVEL (broken rock fragments), little fine to medium sand.	
	2D	24/8	2.00 - 4.00	2/2/4/3	6	8	5	186.60		2.00 Dark brown, wet, loose, fine sandy organic SILT, grading to very dark brown, moist, fine to coarse SAND, trace gravel, slight odor, (River Bottom Sediment and Alluvium).	
							8				
							50			Washing out to 5.0' bgs, hit rock fragment at 4.5' bgs.	
5	3D	24/8	5.00 - 7.00	12/17/20/12	37	47	59	183.60		5.00 Dark olive-grey, saturated, dense, angular fine to coarse gravelly SAND, little silt, (gravel is broken rock fragments), slight odor, (River Bottom Sediment and Alluvium).	G#209970 A-1-a, SM WC=18.4%
							93				
							77				
							70				
							100	179.60		9.00	
10	4D	24/10	10.00 - 12.00	15/18/24/20	42	54	81			Grey, moist, very dense, fine to coarse SAND, some fine angular gravel, little silt, well sorted, (Alluvium). Telescoped NW Casing into HW Casing at 11.1' bgs.	G#209971 A-1-b, SM WC=10.6%
							23				
							65				
							71				
							109	174.60		14.00	
15	5D/BA	24/12	15.00 - 17.00	27/34/43/32	77	99	71			(5D/B) Olive grey, damp to moist, very dense, silty SAND, trace angular gravel, some staining, (Glacial Till). (5D/A) Olive grey, damp, very dense, fine to coarse SAND, some silt, little fine angular gravel and weathered rock fragments, (Glacial Till).	G#209973 A-4, SM WC=11.1% G#209972 A-2-4, SM WC=10.1%
							95				
							140				
							172				
							265				
20	6D/AB	24/18	20.00 - 22.00	33/53/52/49	105	135	63			(6D/A) Olive grey and brown, mottled, damp, very dense, gravelly SILT, gravel fine to coarse, angular, including weathered rock fragments, some fine sand, little clay, (Glacial Till). Roller Coned ahead to 23.8' bgs., hit something hard, roller coned ahead to 25.0' bgs. (6D/B) Brown, moist, very dense, silty fine to coarse SAND, some fine to coarse angular gravel, trace clay, (Glacial Till).	
							70				
							79				
							88				
										NW Casing to 24.5' bgs.	
25	R1	60/57.6	25.00 - 30.00	RQD = 96%			NQ-2 CORE	163.60		25.00 Top of Bedrock at Elev. 163.6 ft. R1: Bedrock: Dark grey, fine grained, metasedimentary (HORNFELS) moderately hard, fresh, occasional quartz veins, no foliation, drill breaks along quartz veins, one open seam 8" from top, upper 8" quartz disolved, some vuggy seams. Vassalboro Formation. Rock Mass Quality: Excellent. R1:Core Times (min:sec) 25.0-26.0' (10:40) 26.0-27.0' (5:00) 27.0-28.0' (4:25) 28.0-29.0' (5:15) 29.0-30.0' (6:27) 96% Recovery	
30	R2	58.8/58.8	30.00 - 34.90	RQD = 86%						R2: Bedrock: Same as R1, only less fractured, fractures along quartz veins, surfaces stained with some oxidation, drill breaks along quartz veins. Vassalboro Formation. Rock Mass Quality: Good. R2:Core Times (min:sec) 30.0-31.0' (6:30) 31.0-32.0' (5:50) 32.0-33.0' (3:30) 33.0-34.0' (4:05) 34.0-34.9' (2:33) 100 % Recovery	
35								153.70		34.90 Bottom of Exploration at 34.90 feet below ground surface.	
40											
45											
50											

Remarks:

13.9' from Bridge Deck to Ground.

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-NSR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
15625.00
BRIDGE NO. 2501
PIN 15625.00
BRIDGE PLANS

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

SIGNATURE
P.E. NUMBER
DATE

MAIN STREET BRIDGE
EAST BRANCH SEBASTICOOK RIVER
NEWPORT PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4