MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CLAYHILL BRIDGE
OVER MOUSAM RIVER
STATE ROUTE 9
KENNEBUNK, MAINE

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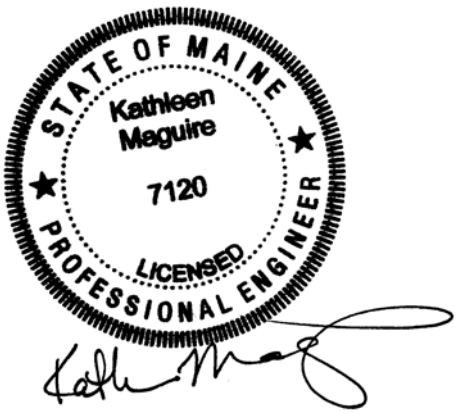

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of the Clayhill Bridge over the Mousam River in Kennebunk, Maine. The proposed bridge replacement will consist of three-span precast concrete superstructure founded on H-pile supported integral abutments and two pipe pile bent piers. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-Piles – The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. The H-piles shall be design for all relevant strength, service and extreme limit state load groups. The structural resistance check should include checking axial, lateral, and flexural resistance. 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.

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

Integral Stub Abutments – Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. In designing integral abutments for passive earth pressure, the Rankine earth pressure coefficient (K_p) of 3.25 is allowed if the displacement of the abutment is less than 2 percent of the abutment height. All abutment designs shall include a drainage system to intercept any water. The approach slab should be positively connected to the integral abutment. Additional lateral earth pressure due to construction surcharge or live load surcharge is required if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted.

Pipe Pile Pier Bents - Piles for the pier bents may consist of concrete filled pipe piles driven to bedrock. Pipe piles can be driven open-ended or closed-ended. The pipe piles shall be designed at the strength limit state considering the structural, geotechnical and drivability resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the pipe piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. A modified strength limit state analysis should be performed that includes the ice pressures specified in MaineDOT BDG Section 3.9.

Extreme limit state design checks for piers shall include pile geotechnical and structural failure by buckling and uplift with respect to extreme event loading combinations related to ice loads, vessel collision and certain hydraulic events. The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in MaineDOT BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading. All piles should be designed to achieve a fixed condition for the design scour event.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each pier. The first pile driven at each pier should be dynamically tested with a minimum 24-hour restrike test to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Scour and Riprap – The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. The riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material.

Settlement - The roadway profile will be raised approximately 1.85 ft at the abutments. Evaluation of the potential settlement due the placement of the proposed fill resulted in less than 1 inch of settlement. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

Frost Protection - Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection.

Seismic Design Considerations – Seismic analysis is required for multi-span bridges in Seismic Zone 2. The minimum analysis requirements for Seismic Effects are single mode elastic method/uniform load elastic method (SM/UL).

Construction Considerations – Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. 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. Using the excavated native soils as structural backfill should not be permitted. Materials excavated from the existing subbase and subgrade fill soils in approaches should not be used to re-base the new bridge approaches.

Cobbles were encountered within the fill at Abutment No. 2. A layer of wood was encountered in the area of Pier No. 2 (BB-KMR-103). There is potential for these obstructions to impact the pile driving and/or installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Clearing obstructions shall be specified as incidental to related pay items. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

All timber piling within the river shall be removed to 1 foot below river bed. There is a potential for the existing abutment and pier piles to interfere with the installation of the proposed piles. If the existing piles are encountered during pile installation they shall be removed by the Contractor.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of the Clayhill Bridge over Mousam River in Kennebunk, Maine. A subsurface investigation at the site has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing Clayhill Bridge carries Western Avenue (State Route 9) over Mousam River and was constructed in 1941. The bridge consists of a four-span structure with rolled steel beams and a concrete deck founded on timber pile supported abutments and timber pile bent piers. The bridge deck was replaced in 1964 and the pier caps were replaced in 1990. The structure has a total length of approximately 154 feet on a 30 degree skew. The 2010 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck and super structure are in fair condition (rating of 5) and the substructures are in poor condition (rating of 4). The Bridge Sufficiency Rating is 44.9. The structure has a scour critical rating of "8 – Stable Above Footing" meaning that the foundations have been determined to be stable for the assessed or calculated scour condition. The scour is determined to be above the top of the footings. Inspection records note that the bridge is in overall fair condition with moderate deterioration of elements. Notes state that at some point the westerly abutment bridge has settled as there is a pronounced dip in the bridge rail. The timber pile bents have moderate rotting and splitting with heavy infestation of marine borers in some piling. The timber abutment caps also have moderate splitting and cracking.

The MaineDOT Bridge Program is currently proposing a replacement structure consisting of a three-span, concrete superstructure with a sidewalk founded on H-pile supported integral abutments and pipe pile bent piers with approximately 700 feet of approach work. The overall length of the proposed replacement structure will be 168 feet. The proposed structure will have a skew of approximately 20 degrees. The proposed roadway profile will be raised approximately 1.85 feet at the abutments. The new roadway centerline will be located approximately 6.0 feet downstream of the existing centerline to minimize impacts to the adjoining Rachel Carson Wildlife Refuge property. Two-way traffic will be maintained during construction using a temporary bridge.

2.0 GEOLOGIC SETTING

The Clayhill Bridge in Kennebunk carries Western Avenue (State Route 9) over the Mousam River as shown on Sheet 1 - Location Map found at the end of this report.

According to the Wells Quadrangle, Maine Surficial Geology map published by the Maine Geological Survey Open File No. 99-104 (1999) the surficial soils in the vicinity of the site consist of wetland, swamp and marine regressive sand deposits. The wetland and swamp deposits generally consist of peat, muck, silt and sand and are deposited on flat terrain. These deposits are formed by accumulation of sediments and organic material in depressions and other poorly drained areas. The marine regressive sand deposits generally consist of

massive to stratified and cross-stratified, well sorted, brown and grey-brown sand. Marine regressive deposits generally have basal contact with silts and silt clays of the Presumpscot Formation. Marine regressive sands were deposited during the regressive phase of marine submergence.

According to the Bedrock Geologic Map of Maine published by the Maine Geological Survey (1985) the bedrock in the vicinity of the site consists of fine-grained, calcareous, feldspathic, sandstone of the Kittery Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling four (4) test borings. Test boring BB-KMR-101 was conducted behind the south abutment. Test borings BB-KMR-102 and BB-KMR-103 were conducted in the river near the proposed pier locations. Test boring BB-KMR-104 was conducted behind the north abutment. The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the soil stratigraphy across the site is shown on Sheet 3 – Interpretive Subsurface Profile found at the end of this report. The borings were drilled between October 18 and 21, 2010 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in March of 2010 and was found to deliver approximately 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 an average energy transfer factor of 0.84 to the raw field N-values. This hammer efficiency factor (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 the borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector or the geotechnical team member logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the exploration programs.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of fifteen (15) standard grain size analyses with water content and nineteen (19) grain size analyses with hydrometer and water content. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 4 – Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the borings generally consisted of sand fill, marine sand, glaciomarine silt and sand/gravel all underlain by bedrock. The exploration locations are shown on Sheet 2 - Boring Location Plan and an interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 – Interpretive Subsurface Profile both found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in the borings in detail:

5.1 Sand Fill

A layer of sand fill is present beneath the pavement at the abutment locations. The sand fill was brown, damp, fine to coarse sand with some to little gravel, little to trace silt, and occasional cobbles and brown, damp, gravelly, fine to coarse sand with trace silt and few cobbles. The thickness of the sand fill layer was approximately 9.0 feet in borings BB-KMS-101 and BB-KMS-104. Corrected SPT N-values in the sand fill ranged from 7 to 60 blows per foot (bpf) indicating that the fill is loose to very dense in consistency. Water contents from three (3) samples obtained within the sand fill layer range from approximately 4% to 9%. Three (3) grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-2-4 or A-1-b by the AASHTO Classification System and an SM by the Unified Soil Classification System.

5.2 Marine Regressive Sands

A deposit of marine regressive sand was encountered beneath the fill and in the river. The thickness of the deposit ranged from approximately 6.0 feet in boring BB-KMR-104 to approximately 41.0 feet thick boring BB-KMR-101. The deposit generally consisted of layers of:

- Brown to dark brown, wet, fine to coarse sand, with trace to little gravel, trace to little silt and trace clay;
- Grey-brown, grey and dark grey, wet, fine to coarse and fine to medium sand, with trace to some gravel, trace to some silt, trace clay, trace organics and occasional cobbles.

Corrected SPT N-values in the layer ranged from 3 to 76 bpf indicating that the soil is very loose to very dense in consistency. Water contents from thirteen (13) samples obtained

within the layer range from approximately 8% to 41%. Thirteen (13) grain size analyses conducted on samples from the layer indicate that the soil is classified as an A-3, A-2-4, A-4, or A-1-a by the AASHTO Classification System and a SP-SM, SC-SM, SW or SP by the Unified Soil Classification System.

5.3 Glaciomarine Silt

A layer of glaciomarine silt was encountered beneath the marine sand in all of the borings. The thickness of the layer ranged from approximately 5.5 feet in boring BB-KMR-101 to 33.8 feet in boring BB-KMR-103. The silt generally consisted of grey and dark brown, wet, silt and sandy silt with trace to some sand, trace to little clay and trace gravel. Corrected SPT N-values in the layer ranged from weight of hammer (WOH) to 4 bpf indicating that the soil is very soft to soft in consistency. Water contents from thirteen (13) samples obtained within the layer ranged from approximately 36% to 139%. Thirteen (13) grain size analysis conducted on samples from the layer indicate that the soil is classified as an A-4 or A-1-a by the AASHTO Classification System and as an ML by the Unified Soil Classification System.

5.4 Sand/Gravel

A layer of sand or gravel was encountered beneath the silt in three of the four borings. The thickness of the layer ranged from approximately 5.6 feet in boring BB-KMR-101 to 25.8 feet in boring BB-KMR-104. No sand or gravel was present beneath the silt layer in boring BB-KMR-103. The sand/gravel generally consisted of:

- grey brown, wet, fine to coarse sand with some gravel and trace to little silt;
- grey, wet sandy gravel, with trace silt;
- grey, wet, fine to coarse sand with trace to some gravel and trace to little silt;
- brown, wet, fine to coarse sand with little silt and trace gravel; and
- brown and grey , wet, gravelly fine to coarse sand with trace to little silt.

Corrected SPT N-values in the layer ranged from 6 to >50 bpf indicating that the soil is loose to very dense in consistency. Water contents from five (5) samples obtained within the layer ranged from approximately 11% to 15%. Five (5) grain size analysis conducted on samples from the layer indicate that the soil is classified as an A-1-a or A-1-b by the AASHTO Classification System and as an GW-GM, SW-SM, SP-SM or SM by the Unified Soil Classification System.

5.5 Bedrock

Bedrock was encountered and cored in all of the borings. The Table 5-1 summarizes the depths to bedrock corresponding elevations of the top of bedrock and RQD:

Boring Number	Depth to Bedrock	Bedrock Elevation	RQD
BB-KMR-101	61.1 feet	-51.0 feet	16 – 58%
BB-KMR-102	39.5 feet	-45.0 feet	70%
BB-KMR-103	42.8 feet	-50.2 feet	57%
BB-KMR-104	50.3 feet	-41.0 feet	63 – 67%

Table 5-1 - Summary of Bedrock Depths, Elevations and RQD

The bedrock is identified as black, fine grained, very hard, meta-siltstone, meta-sandstone and phyllite, fresh, with 60 to 80 degree bedding and scattered quartz veins and pyrite along the bedding planes. The rock quality designation (RQD) of the bedrock was determined to range from 16 to 70 percent indicating a rock mass quality of very poor to fair.

5.6 Groundwater

Groundwater was observed at a depth of approximately 9.0 to 9.5 feet below the existing ground surface in the abutment borings. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

The following foundation alternatives were considered for the bridge replacement:

- Cantilever-type abutments founded on spread footings on soil;
- Cantilever-type abutments on H-pile groups;
- Integral, driven H-pile supported stub abutments;
- Concrete filled pipe pile pier bents;
- Pier bents consisting of H-pile driven to bedrock with pipe pile encasement to depth of fixity;
- Pre-stressed concrete pile pier bents;
- Mass concrete piers supported on driven H-piles; and
- Drilled shaft supported piers.

After consideration of all of the alternatives, pile-supported integral abutments located behind the existing abutments and pipe pile pier bents were selected because they require minimal future maintenance. This report addresses only these foundation types.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for stub abutments founded on a single row of integral H-piles driven to bedrock and pipe pile pier bents driven to bedrock which have been identified as the optimal substructures for the project.

7.1 Integral Abutment H-Piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with pile tips to protect the tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on Table 7-1 below:

Location	Estimated Pile Cap Bottom Elevation	Approximate Depth to Bedrock From Ground Surface	Approximate Top of Rock Elevation	Estimated Pile Length (including 1 foot embedment into pile cap)
Abutment #1 BB-KMR-101	3.0 feet	61.1 feet	-51.0 feet	55 feet
Abutment #2 BB-KMR-104	3.0 feet	50.3 feet	-41.0 feet	45 feet

Table 7-1 – Estimated Pile Lengths for H-Piles

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

The H-piles shall be designed for the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 7.1.1 below. The H-piles shall also be checked for fixity and combined axial and flexure using LPile® software.

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and 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. The design and check floods for scour are defined in LRFD Articles 2.6.4.4.2 and 3.7.5.

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

7.1.1 Strength Limit State Design

The nominal compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. It is the responsibility of the structural engineer to recalculate the nominal structural compressive resistance (P_n) based on "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e". Preliminary estimates of the factored structural axial compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (good driving conditions) and an unbraced length (ℓ) of 48 inches (for scour) and an effective length factor (K) of 1.2.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the proposed H-pile sections were calculated using a resistance factor, φ_{stat}, of 0.45.

The drivability of the five (5) proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done, given in LRFD Table 10.5.5.2.3-1, is φ_{dyn}= 0.65.

For the strength limit state, the calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections are summarized in Table 7-2 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* ϕ_c=0.60	Geotechnical Resistance φ_{stat}=0.45	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
HP 12x53	451	279	291	291
HP 12x74	636	389	407	407
HP 14x73	629	347	406	406
HP 14x89	768	421	473	473
HP 14x117	1013	552	588	588

* based on preliminary assumption of ℓ=48" and K=1.2

Table 7-2 - Factored Axial Resistances for Abutment Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the

structural limit state. Even though the factored axial drivability resistance is less than the factored axial structural resistance, local experience supports the estimated factored resistance from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the factored drivability resistance shown in the last column of Table 7-2 above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2.

7.1.2 Service and Extreme Limit State Design

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and displacements considering changes in foundation conditions due to scour at the design flood event. 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.

For the service and extreme limit states resistance factors, φ, of 1.0 are recommended for structural and geotechnical pile resistances. It is the responsibility of the structural engineer to recalculate P_n based on refined elastic critical buckling resistance (P_e) evaluations.

For the service and extreme limit states, the calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections are summarized in Table 7-3 below. Supporting calculations are included in Appendix C-Calculations found at the end of this report.

Pile Section	Service and Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* ϕ=1.0	Geotechnical Resistance φ=1.0	Drivability Resistance φ=1.0	Governing Resistance
HP 12x53	752	620	448	448
HP 12x74	1059	865	626	626
HP 14x73	1049	771	625	625
HP 14x89	1280	937	728	728
HP 14x117	1688	1226	904	904

*based on preliminary assumption of ℓ=48" and K=1.2

Table 7-3 - Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the

structural limit state. The factored axial drivability resistance is less than the factored axial structural resistance and local experience supports the estimated factored resistance from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the service and extreme limit states should not exceed the factored drivability resistance shown in the last column of Table 7-3 above.

7.1.3 Driven Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each integral abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

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

7.2 Integral Stub Abutment Design

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

A resistance factor of $\phi = 1.0$ shall be used to assess 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 checks for abutments supported on piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf and a soil-concrete friction angle of 20 degrees. Cast-in-place integral abutment sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure state. The Coulomb passive earth pressure coefficient, K_p, of 6.89 is recommended. Developing full passive 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 not less than the Rankine passive earth pressure case using a Rankine passive earth pressure coefficient, K_p, of 3.25. 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 for the integral abutment backwall design.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 7-4 below:

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

Table 7-4 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

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 connected to the integral abutment.

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

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

7.3 Pipe Pile Pier Bents

Pile bent piers were selected for intermediate structure support. Piles for the pier bents may consist of concrete filled pipe piles driven to bedrock. Pipe piles with diameters ranging from 24 to 30 inches and wall thicknesses of 1/2 to 5/8 inch are recommended. Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with minimum yield strength of 45 ksi. Piles shall be filled with Class A concrete. Piles should have straight butt-welded seams. Spiral seams are not recommended because the welded surfaces are vulnerable to thin fusion bonded epoxy coatings, ice abrasion and bumping during construction. Any welds between pile segments should be ground down and blended smooth wit the pipe pile material. Pipe piles can be driven open-ended or closed-ended. Open ended piles should be equipped with a cutting shoe constructed from ASTM A148 grade 90/60 steel. Closed ended piles should be equipped with a conical point constructed from ASTM A148 grade 90/60 cast steel. Pipe pile pier bent piles should be end bearing and driven to the required nominal resistance on or within the bedrock.

Pipe piles shall be coated with a fusion bonded epoxy coating with a thickness of 18 to 20 mil and top coated in accordance with Special Provision 506. The fusion bonded epoxy protective coating shall be applied to a minimum of 10 feet below river bed or 2 feet below the total scour depth. The portion of the pipe pile to be embedded in the concrete pile cap shall not be top coated. Cathodic protection by aluminum alloy anodes shall be used in addition to fusion bonded epoxy protective coating.

Pile lengths at the proposed pier may be roughly estimated based on Table 7-5 below:

Location/ Boring	Pile Orientation	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length (including 1 foot embedment into pile cap)
Pier 1 BB-KMR-102	Plumb 4 in/ft Batter	5.8 feet	39.5 feet	-45.0 feet	52 feet 55 feet
Pier 2 BB-KMR-103	Plumb 4 in/ft Batter	5.8 feet	42.8 feet	-50.2 feet	57 feet 59 feet

Table 7-5 – Estimated Pile Lengths for Pipe Piles

This estimated pile length does not take into account the variability of the bedrock surface within the channel or the additional eight (8) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

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

The design of the piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the pier piles will be subjected to

lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2. A modified strength limit state analysis should be performed that includes the ice pressures specified in MaineDOT BDG Section 3.9 – Ice Loads.

Extreme limit state design checks for piers shall include pile geotechnical and structural failure by buckling and uplift with respect to extreme event loading combinations related to ice loads, vessel collision and certain hydraulic events. Resistance factors, φ, for the extreme limit state shall be taken as 1.0. The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in MaineDOT BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

7.3.1 Strength Limit State

The nominal compressive structural resistance (P_n) for piles in the strength limit state loaded in compression shall be as specified in LRFD Article 6.9.4.1 for non-composite members (H-pile) and Article 6.9.5.1 for composite members (pipe pile). The pipe piles have an unbraced length (ℓ) and require calculation of the λ-factor as specified in LRFD Article 6.9.5.1.

For the strength limit state, the factored axial compressive structural resistance of the pipe pile (P_r) shall be calculated using the resistance factors (ϕ_c) of 0.7 for pipe pile in good driving conditions as specified in LRFD Article 6.5.4.2. The proposed pier bent piles will have an unbraced pile length ranging from 26 to 29 feet.

Per LRFD Article 6.5.4.2, at the strength limit state, for pipe piles in compression and bending, the axial resistance factor ϕ_c=0.8 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined nominal axial and flexural resistance of the pile in the interaction equation, (LRFD Eq. 6.9.2.2-1 or -2) with flexural resistance determined as specified in LRFD 6.12. The factored structural resistance for pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural designer.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the eight (8) proposed pipe pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45 for end bearing piles on bedrock.

The drivability of the eight (8) proposed pipe pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 45 ksi steel, shall be less than 40 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65.

Factored axial compressive structural, geotechnical and drivability resistances for eight (8) pipe pile sections are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pipe Pile		Factored Resistance			
Diameter	Wall Thickness	Structural Resistance* (non-composite section) ϕ_c=0.70	Geotechnical Resistance φ_{stat}=0.45	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
24 inches	½ inch	616 kips	392 kips	481 kips	481 kips
26 inches	½ inch	704 kips	412 kips	525 kips	525 kips
28 inches	½ inch	791 kips	432 kips	569 kips	569 kips
30 inches	½ inch	877 kips	453 kips	621 kips	621 kips
24 inches	5/8 inch	814 kips	520 kips	633 kips	633 kips
26 inches	5/8 inch	931 kips	547 kips	641 kips	641 kips
28 inches	5/8 inch	1047 kips	574 kips	657 kips	657 kips
30 inches	5/8 inch	1162 kips	601 kips	674 kips	674 kips

* based on preliminary assumption of ℓ=25 feet and K=2.0

Table 7-6 - Factored Axial Resistances for Pipe Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. For all of the pile analyzed the factored axial geotechnical resistance is less than the factored axial structural resistance and the factored axial drivability resistance. Based on local experience, it is recommended that the maximum factored axial pile load used in design for the strength limit state not exceed the drivability resistances shown in the last column of Table 7-6 above.

7.3.2 Service Limit and Extreme Limit State Designs

Per LRFD Article 10.5.5.1 the ability of the pier piles to meet defection criteria at the service limit state shall be investigated using a resistance factor of 1.0. Per LRFD Article 10.5.5.3.3 the ability of the pier piles at the extreme limit state shall be investigated using a resistance factor of 1.0. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0.

The axial structural resistance of eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was investigated using a resistance factor of 1.0. The piles have an unbraced length and require calculation of the λ factor as specified in LRFD Article 6.9. The axial geotechnical compressive resistance of eight (8) proposed pipe pile sections and was calculated using Canadian Foundation Engineering Manual methods and a resistance factor of 1.0. The drivability of the eight (8) proposed pipe pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 45 ksi steel, shall be less than 40.5 ksi. The resistance factor for a single pile in axial compression for the service and extreme limit states of 1.0 was used.

The calculated factored axial structural, geotechnical and drivability resistances for the eight (8) pipe pile sections are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pipe Pile		Factored Resistance			
Diameter	Wall Thickness	Structural Resistance (non-composite section) ϕ=1.0	Geotechnical Resistance ϕ=1.0	Drivability Resistance ϕ=1.0	Governing Resistance
24 inches	½ inch	880 kips	871 kips	740 kips	740 kips
26 inches	½ inch	1005 kips	916 kips	807 kips	807 kips
28 inches	½ inch	1129 kips	961 kips	875 kips	875 kips
30 inches	½ inch	1253 kips	1006 kips	955 kips	955 kips
24 inches	5/8 inch	1162 kips	1155 kips	974 kips	974 kips
26 inches	5/8 inch	1330 kips	1215 kips	986 kips	986 kips
28 inches	5/8 inch	1496 kips	1275 kips	1010 kips	1010 kips
30 inches	5/8 inch	1660 kips	1335 kips	1037 kips	1037 kips

*based on preliminary assumption of ℓ=25 feet and K=2.0

Table 7-7 - Factored Axial Resistances for Pipe Piles at the Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The factored axial drivability resistance is less than the factored axial structural resistance and the factored axial geotechnical resistance. It is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the governing resistance shown in the last column of Table 7-7 above.

7.3.3 Estimated Effective Pile Lengths

Buckling stability of the piles shall be evaluated in accordance with the provisions in LRFD Articles 6.9, 6.12 and 6.15 using an effective pile length of the pile that accounts for the laterally unsupported length of the exposed pile extending through the air and/or water plus the embedment depth to pile fixity.

All piles should be designed to achieve a fixed condition for the design scour event. Preliminary depths to fixity for eight (8) proposed pipe pile sections were calculated, assuming only axial loading and without consideration of lateral loads, using the buckling methodology in LRFD Article 10.7.3.13.4. Table 7-8 below summarizes the estimated depths to fixity for the eight (8) proposed pile sections and the estimated design scour depth. The design scour depth provided by the Structural Designer was estimated to be less than 13 feet. For the purposes of the geotechnical calculations the effective length of the pile was assumed to be the length of pile above the river bed (approximately 13 feet) plus the depth to fixity calculated for each proposed pile section. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Outside Pipe Pile Diameter/ Wall thickness	Preliminary Estimates of Depth to Fixity w/ no lateral loads applied	Estimated Exposed Pile Length Due to Design Scour	Estimated Unsupported Length, ℓ (length in air and water)	Estimated Effective Length For Buckling Analysis (depth to fixity + scour + unsupported length)
24-in / ½ in	13.6 feet	13.0 feet	12.0 feet	38.6 feet
26-in / ½ in	14.4 feet	13.0 feet	12.0 feet	39.4 feet
28-in / ½ in	15.2 feet	13.0 feet	12.0 feet	40.2 feet
30-in / ½ in	16.0 feet	13.0 feet	12.0 feet	41.0 feet
24-in / ⅝ in	14.0 feet	13.0 feet	12.0 feet	39.0 feet
26-in / ⅝ in	14.8 feet	13.0 feet	12.0 feet	39.8 feet
28-in / ⅝ in	15.6 feet	13.0 feet	12.0 feet	40.6 feet
30-in / ⅝ in	16.4 feet	13.0 feet	12.0 feet	41.4 feet

Table 7-8 - Preliminary Estimates of Effective Pile Lengths

Due to the depth of the overburden at the site, the pile sections will all achieve a fixed condition under normal conditions (no scour) and the design scour event when they are driven to end bearing on bedrock.

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

7.3.4 Buckling and Combined Axial and Flexure

Pile group design shall consider loading effects due to combined axial and flexural loading, as outlined in LRFD Article 6.15. In designing piles for the bent group the effects of soil-structure interaction shall be considered in conformance with LRFD Article 10.7.3.12. The recommended design approach considers the non-linear response of soil with lateral displacement. Soil-structure interaction considering the non-linear response of soil can be modeled using LPile or FBPier software.

The factored structural resistances for pipe pile sections in combined axial compression and flexure and buckling analyses are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural engineer. For evaluating buckling and lateral stability in accordance with LRFD Article 10.7.3.13.4 use the effective pile lengths provided in Table 7-8.

7.3.5 Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock at the site, pile splices will be required. The location and number of pile splices shall be in conformance with MaineDOT Standard Specification 501 and be subject to the approval of the Resident.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with a minimum 24-hour restrike at each pier. The first pile driven at each pier should be dynamically tested (including 24-hour restrike) to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans per LRFD Article 3.6.5.2. Calculations for the pile resistance required by a drivability wave equation analysis are included the Appendix C- Calculations.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis, dynamic pile test measurements CAPWAPs and as approved by the Resident. Driving stresses in the pipe pile determined in the drivability analysis shall be less than 40.5 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 8 to 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.4 Scour and Riprap

Grain size analyses were performed on soil samples taken at the approximate streambed elevation to generate grain size curves for determining parameters to be used in scour analyses. The samples were assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 0.5 mm
- Average diameter of particle at 95 percent passing, D_{95} = 8 mm
- Soil Classification AASHTO Soil Type A-4 or A-2-4

The grain size curves are included in Appendix B- Laboratory Data found at the end of this report.

The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. 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 load.

For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Bridge approach slopes and slopes at wingwalls shall be armored with 3 feet of riprap. Stone riprap shall conform to item number 703.26 of MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "1" Erosion Control Geotextile per Standard Details 610(02) through 610(04).

7.5 Settlement

The roadway profile will be raised approximately 1.85 ft at the abutments. Evaluation of the potential settlement due the placement of the proposed fill resulted in less than 1 inch of settlement. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible. See Appendix C - Calculations at the end of this report for supporting documentation.

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.

In the event that any foundation is placed on granular subgrade soils, it should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1123 F-degree days. In a granular soil with a water content of approximately 10%, this correlates to a frost depth of approximately 5.0 feet. Any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. See Appendix C - Calculations at the end of this report for supporting documentation.

7.7 Seismic Design Considerations

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) = 0.093g
- Site Class E (soil profile with average N-value for the upper 100 feet of the soils and rock profile of less than 15 bpf)
- Acceleration coefficient (A_s) = 0.233
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.454g
- Design spectral acceleration coefficient at 1.0-second period (S_{D1}) = 0.156g
- Seismic Zone 2 (based on S_{D1} greater than 0.15g and less than 0.30g)

In conformance with LRFD Article 4.7.4.3, seismic analysis is required for multi-span bridges in Seismic Zone 2. The minimum analysis requirements for Seismic Effects are single mode elastic method/uniform load elastic method (SM/UL). Additional requirements for the determination of seismic design forces for foundations in Seismic Zone 2 are discussed in LRFD Article 3.10.9.3. According to Figure 2-2 of the MaineDOT BDG, the Clayhill Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million.

See Appendix C- Calculations at the end of this report for supporting documentation.

7.8 Construction Considerations

Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The removal of the existing structure may require the replacement of excavated soils with compacted granular fill prior to pile driving.

Cobbles were encountered within the fill at Abutment No. 2 (BB-KMR-104). A layer of wood was encountered in the area of Pier No. 2 (BB-KMR-103). There is potential for these obstructions to impact the pile driving and/or installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Clearing obstructions shall be specified as incidental to related pay items. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

All timber piling within the river shall be removed to 1 foot below river bed. Payment shall be considered incidental to bridge removal. There is a potential for the existing abutment and pier piles to interfere with the installation of the proposed piles. If the existing piles are encountered during pile installation they shall be removed by the Contractor to the Resident's satisfaction. This condition should be noted on the plans and the work should be considered incidental to pile installation.

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 during construction.

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

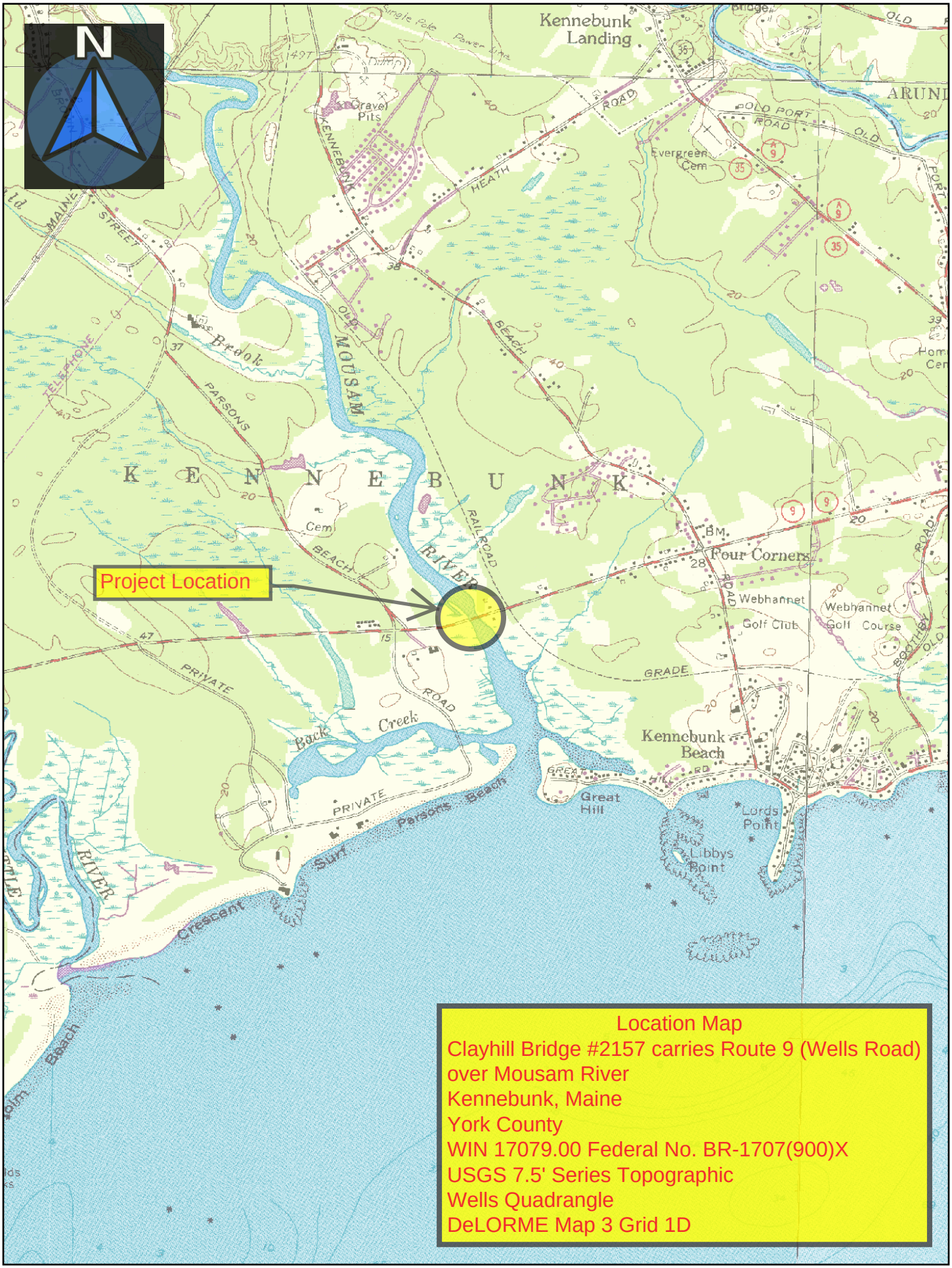
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 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 the Clayhill Bridge in Kennebunk in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations 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 to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

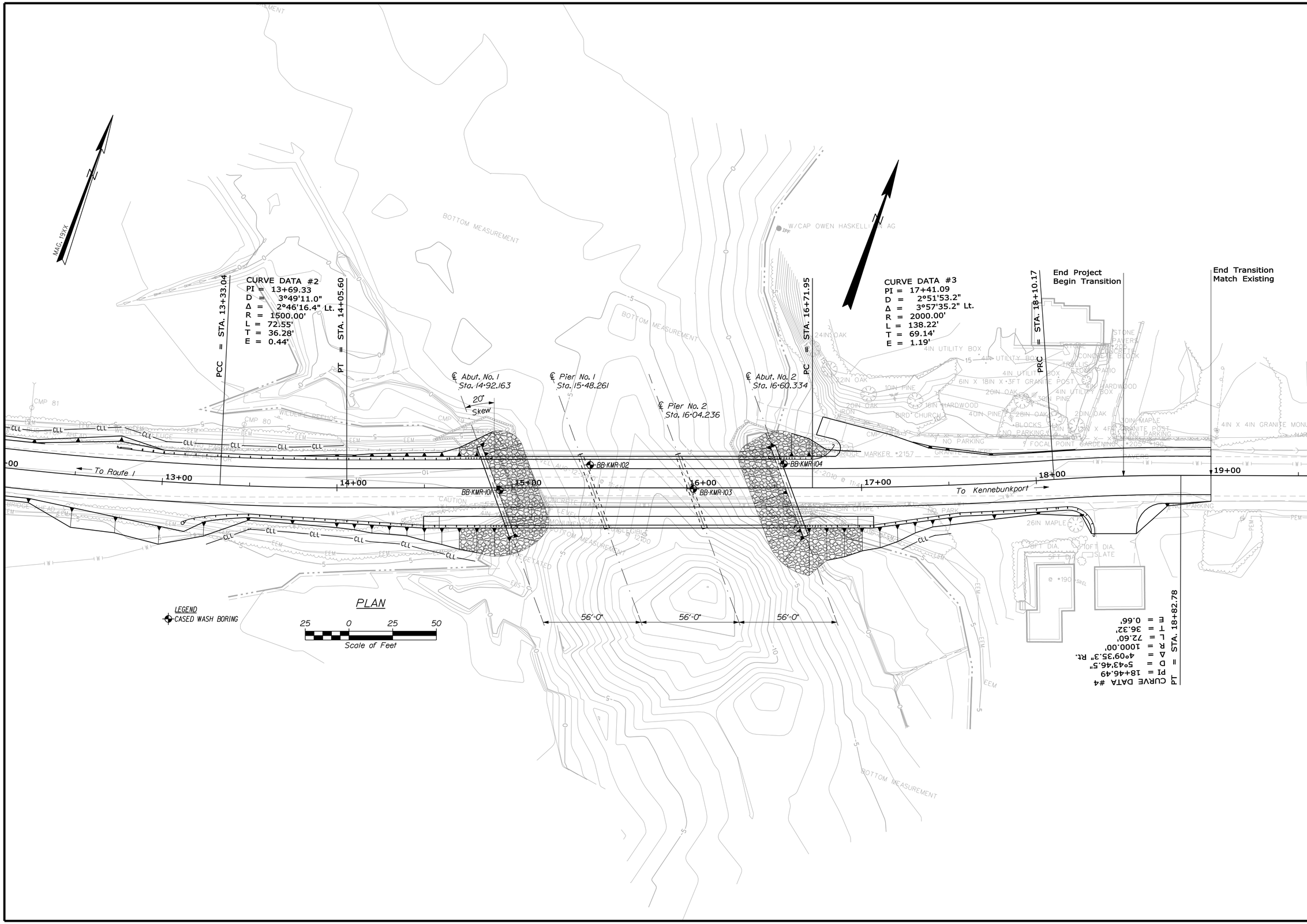
Sheets



Location Map
Clayhill Bridge #2157 carries Route 9 (Wells Road) over Mousam River
Kennebunk, Maine
York County
WIN 17079.00 Federal No. BR-1707(900)X
USGS 7.5' Series Topographic
Wells Quadrangle
DeLORME Map 3 Grid 1D

Map Scale 1:24000

The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch. Road names used on this map may not match official road names.



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1707(900)X

PIN 17079.00

BRIDGE NO. 2157

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	B.Condon	
DESIGN-DETAILED	K.MAGUIRE	OCT 2010
CHECKED-REVIEWED	------	------
DESIGN2-DETAILED2	------	------
DESIGN3-DETAILED3	------	------
REVISIONS 1	------	------
REVISIONS 2	------	------
REVISIONS 3	------	------
REVISIONS 4	------	------
FIELD CHANGES	------	------

SIGNATURE

P.E. NUMBER

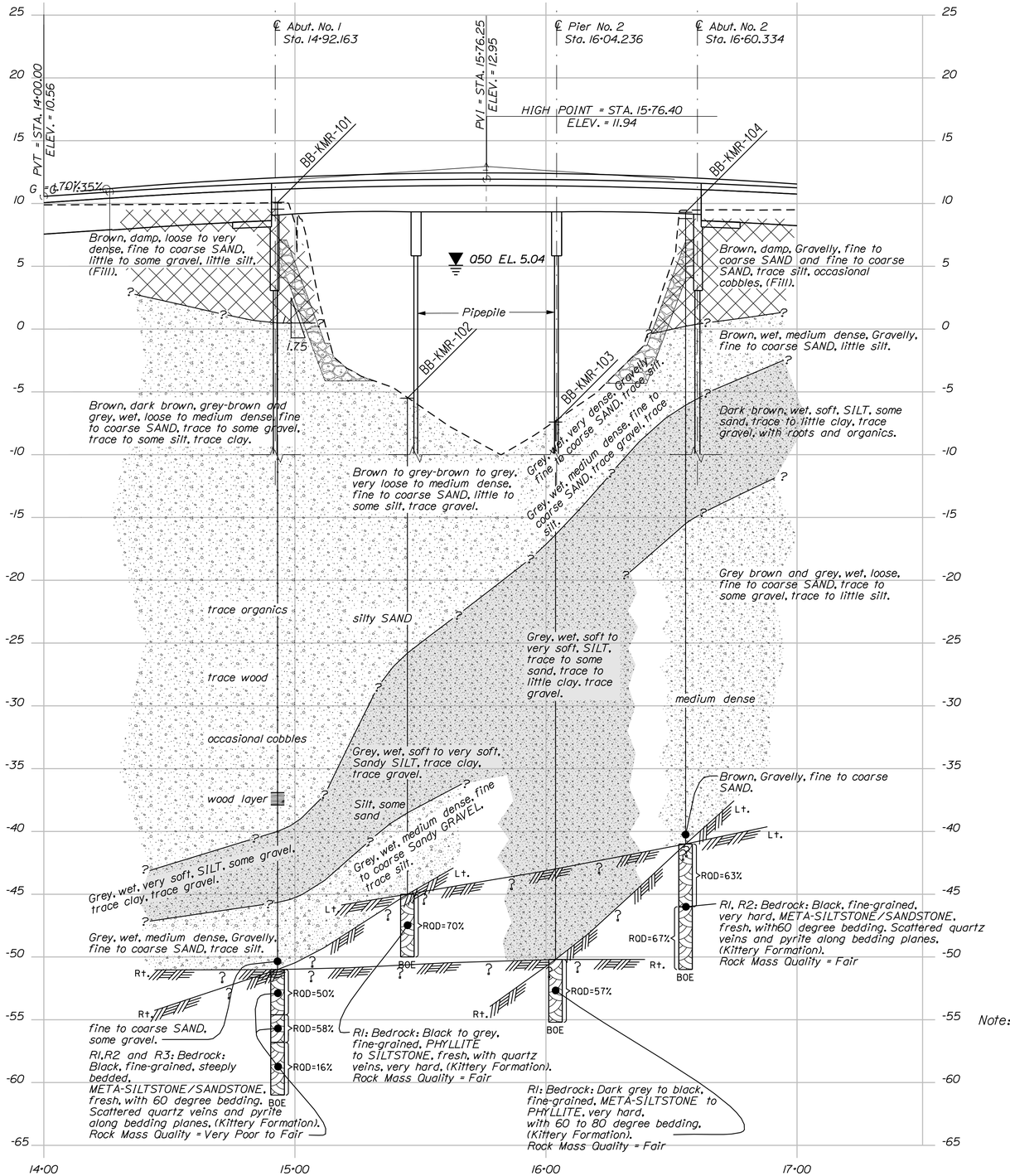
DATE

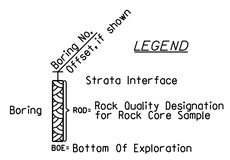
CLAYHILL BRIDGE
MOUSAM RIVER
KENNEBUNK YORK COUNTY

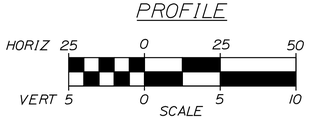
BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\00\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:8/2/2011







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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1707(900)X

PIN 17079.00

BRIDGE NO. 2157

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	B.Condon		
DESIGN-DETAILED	K.MAGUIRE	T.WHITE	OCT 2010
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	------	------	------
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

DATE

CLAYHILL BRIDGE
MOUSAM RIVER
KENNEBUNK YORK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:8/2/2011

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:8/2/2011

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

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-101 WIN: 17079.00

Driller:	MaineDOT	Elevation (ft.)	10.1	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/18/10-10/19/10	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+93.1, 1.4 ft Rt.	Casing ID/OD:	HW	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	9.55	Pavement —0.55	
	1D	24/13	1.00 - 3.00	12/19/24/13	43	60			Brown, damp, very dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#240001 A-1-b, SM WC=4.3%
5	2D	24/14	5.00 - 7.00	2/3/2/4	5	7	29		Brown, damp, loose, fine to medium SAND, little silt, little gravel, (Fill).	G#240002 A-2-4, SM WC=9.0%
							22			
							15			
							17			
							21	1.10	—9.00	
10	3D	24/18	10.00 - 12.00	2/3/3/6	6	8	17		Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	G#240003 A-3, SP-SM WC=15.8%
							23			
							38			
							60			
							134			
15	4D	24/16	15.50 - 17.50	9/3/11/11	14	20	59		Cobble from 15.0-15.3 ft bgs. Dark brown, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#240004 A-2-4, SP-SM WC=24.0%
							69			
							91			
							126			
							111			
20	5D	24/18	20.00 - 22.00	4/6/7/8	13	18	63		Grey-brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#240005 A-3, SP-SM WC=20.3%
							79			
							95			
							93			
							93			
25	6D	24/17	25.00 - 27.00	4/5/5/4	10	14	69		Grey-brown, wet, medium dense, fine to coarse SAND, little silt.	G#240006 A-2-4, SP-SM WC=8.3%
							82			
							83			
							84			
							93			
30	7D	24/16	30.00 - 32.00	2/2/2/2	4	6	87		Grey, wet, loose, fine to coarse SAND, some silt, trace clay, trace organics.	G#240007 A-2-4, SC-SM WC=28.1%
							98			
							99			
							106			
							132			
35	8D	24/18	35.00 - 37.00	2/2/2/5	4	6	114		Dark grey, wet, loose, fine to coarse SAND, little silt, trace gravel, trace wood.	G#240008 A-2-4, SP-SM WC=29.7%
							133			
							125			
							135			
							133			
40	9D	24/19	40.00 - 42.00	2/2/6/10	8	11	121		Grey, wet, medium dense, fine to coarse SAND, some silt, some gravel, trace clay, occasional cobbles.	G#240009 A-2-4, SC-SM WC=27.3%
							136			
							116			
							116			
							123			
45	10D	24/18	45.00 - 47.00	2/3/2/5	5	7	97		Grey, wet, loose, fine to medium SAND, some silt, trace clay.	G#240010 A-2-4, SC-SM WC=38.4%
							124			
							127		Wood in wash water from 47.0-48.0 ft bgs.	
							130			
							170			
50	11D	24/20	50.00 - 52.00	WOH/WOH/WOH/WOH	---		124	-39.90	—50.00 Grey, wet, very soft, SILT, some sand, trace clay, trace gravel.	G#240011 A-4, ML WC=40.5%
							168			
							166			
							167			
							168			
55	12D	24/5	55.00 - 57.00	3/7/13/14	20	28	167	-45.40	—55.50 Grey, wet, medium dense, Gravelly, fine to coarse SAND, trace silt.	
							189			
							189			
							246			
							246			
60	13D	7.2/4	60.50 - 61.10	16/30(1.2")	---		195		Grey-brown, wet, very dense, silty, fine to coarse SAND, some gravel, (Till). a100 blows for 0.1 ft.	
	R1	43.2/43.2	61.10 - 64.70	RQD = 50%			a100 NQ-2	-51.00	—61.10 Top of Bedrock at Elev. -51.0 ft. R1:Bedrock: Black, steeply bedded, very hard, limey, fine grained, META-SILTSTONE/SANDSTONE, fresh, with 60 degree bedding. Scattered quartz veins and pyrite along bedding planes. (Kittery Formation). Rock Mass Quality = Poor. R1:Core Times (min:sec) 61.1-62.1 ft (9:30) 62.1-63.1 ft (5:10) 63.1-64.1 ft (5:30) 64.1-64.7 ft (7:00) 100% Recovery Core Blocked	
	R2	26.4/15	64.70 - 66.90	RQD = 58%					R2:Bedrock: Similar to above. Rock Mass Quality = Fair. R2:Core Times (min:sec) 64.7-65.7 ft (7:41) 65.7-66.7 ft (7:30) 66.7-66.9 ft (3:00) 57% Recovery Core Blocked	
	R3	50.4/50.4	66.90 - 71.10	RQD = 16%				-61.00	R3:Bedrock: Similar to above. Rock Mass Quality = Very Poor. R3:Core Times (min:sec) 66.9-67.9 ft (5:10) 67.9-68.9 ft (9:05) 68.9-69.9 ft (6:55) 69.9-70.9 ft (7:10) 70.9-71.1 ft (2:30) 100% Recovery —71.10 Bottom of Exploration at 71.10 feet below ground surface.	
75										

Remarks:

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

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

Page 1 of 1 — Boring No.: BB-KMR-101

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

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-102 WIN: 17079.00

Driller:	MaineDOT	Elevation (ft.)	-5.5	Auger ID/OD:	N/A
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/20/10-10/21/10	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	15+44.8, 13.1 ft Lt.	Casing ID/OD:	NW	Water Level*:	River Boring, Tidal
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/1	0.00 - 2.00	2/2/2/6	4	6	2		Brown, wet, loose, fine to medium SAND, little silt, trace gravel, trace clay.	
							9			
							26			
							10	-8.50	—3.00	
							5			
5	1D	24/12	5.00 - 7.00	4/3/3/1	6	8	WOH		Grey-brown, wet, loose, fine to coarse SAND, little silt, trace gravel, trace clay.	G#240012 A-2-4, SC-SM WC=24.3%
							8			
							15			
							23	-13.50	—8.00	
							22			
10	2D	24/15	10.00 - 12.00	5/4/4/3	8	11	21		Grey, wet, medium dense, fine to medium SAND, some silt, trace clay, trace roots.	G#240013 A-4, SC-SM WC=27.5%
							15			
							31			
							43			
							34			
15	3D	24/20	15.00 - 17.00	WOH/WOH/2/3	2	3	35	-20.50	—15.00 Grey, wet, very loose, Silty, fine to medium SAND, trace clay, trace gravel, trace roots.	G#240014 A-4, SC-SM WC=41.1%
							36			
							51			
							49			
							51			
20	4D	24/18	20.00 - 22.00	WOH/WOH/2/4	2	3	63	-25.50	—20.00 Grey, wet, soft, Sandy SILT, trace clay, trace gravel.	G#240015 A-4, ML WC=39.3%
							62			
							73			
							89			
							102			
25	5D	24/22	25.00 - 27.00	WOH/WOH/2/4	2	3	105		Grey, wet, soft, Sandy SILT, trace clay.	G#240016 A-4, ML WC=40.3%
							73			
							76			
							76	-34.50	—29.00	
							77			
30	6D	24/20	30.50 - 32.50	WOH/WOH/WOH/WOH	---		111		Grey, wet, very soft, SILT, some sand, trace clay.	G#240017 A-4, ML WC=46.3%
							133			
							113			
							108	-38.50	—33.00	
							100			
35	7D	24/12	35.00 - 37.00	8/8/5/8	13	18	151		Grey, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt.	G#240018 A-1-a, GW-GM WC=10.9%
							142			
							84			
							111		a124 blows for 0.5 ft.	
	R1	60/60	39.50 - 44.50	RQD = 70%			a124 NQ-2	-45.00	—39.50 Top of Bedrock at Elev. -45.0 ft. R1:Bedrock: Black to grey, fine grained, PHYLLITE and SILTSTONE, fresh, with augite crystals and quartz veins, very hard, (Kittery Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 39.5-40.5 ft (4:05) 40.5-41.5 ft (4:50) 41.5-42.5 ft (4:40) 42.5-43.5 ft (4:30) 43.5-44.5 ft (3:45) 100% Recovery	
40										
45								-50.00	—44.50 Bottom of Exploration at 44.50 feet below ground surface.	
50										

Remarks:

Bridge Deck 0.4 ft Pavement, 0.5 ft Concrete.
20.0 ft 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-KMR-102

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

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-103 WIN: 17079.00

Driller:	MaineDOT	Elevation (ft.)	-7.4	Auger ID/OD:	N/A
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/21/10; 10:30-17:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	16+03.8, 0.6 ft Rt.	Casing ID/OD:	NW	Water Level*:	River Boring, Tidal
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/7	0.00 - 2.00	5/24/30/18	54	76	5		Grey, wet, very dense, Gravelly, fine to coarse SAND, trace silt.	G#240019 A-1-a, SW WC=15.0%
							34			
							14			
							14			
							16			
5	2D	24/12	5.00 - 7.00	4/4/5/3	9	13	2		Grey, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#240020 A-2-4, SP WC=19.0%
							8			
							9			
							13	-16.40	—9.00	
							22			
10	3D	24/16	10.00 - 12.00	WOH/WOH/WOH/WOH	---		10		Grey, wet, soft, fine to coarse, Sandy SILT, trace clay, trace gravel, trace organics, roots.	G#240021 A-4, ML WC=35.7%
							14			
							21			
							19			
							21			
15	4D	24/24	15.00 - 17.00	WOH/WOH/WOH/WOH	---		25		Grey, wet, very soft, SILT, little clay, trace sand.	G#240022 A-4, ML WC=58.0%
							25			
							33			
							34			
							43			
20	5D	24/22	20.00 - 22.00	WOH/1/2/3	3	4	65		Grey, wet, soft, SILT, some sand, trace clay, trace gravel, trace wood fiber pieces.	G#240023 A-4, ML WC=53.1%
							58			
							50			
							56			
							52			
25	6D	24/20	25.00 - 27.00	WOH/WOH/1/1	1	1	63		Grey, wet, very soft, SILT, little sand, trace clay, trace gravel.	G#240024 A-4, ML WC=67.9%
							67			
							105			
							93			
							101			
30	7D	24/22	30.00 - 32.00	1/2/1/1	3	4	99		Grey, wet, soft, SILT, some sand, trace clay.	G#240025 A-4, ML WC=47.1%
							82			
							63			
							74			
							65			
35	8D	24/18	35.00 - 37.00	WOH/WOH/WOH/WOH	---		88		Grey, wet, very soft, SILT, little sand, trace clay.	G#240047 A-4, ML WC=54.4%
							89			
							78			
							89			
							86			
40	9D	24/15	40.00 - 42.00	4/1/3/3	4	6	118		Grey, wet, medium stiff, SILT, some sand, trace gravel, trace clay.	G#240048 A-4, ML WC=36.4%
							119		a176 blows for 0.8 ft.	
	R1	60/60	42.80 - 47.80	RQD = 57%			a176 NQ-2	-50.20	—42.80 Top of Bedrock at Elev. -50.2 ft. R1:Bedrock: Dark grey to black, fine grained, laminated, META-SILTSTONE to PHYLLITE, fresh, with 60 to 80 degree bedding, very hard (Kittery Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 42.8-43.8 ft (5:00) 43.8-44.8 ft (3:20) 44.8-45.8 ft (3:40) 45.8-46.8 ft (3:00) 46.8-47.8 ft (2:50) 100% Recovery	
45										
								-55.20	—47.80 Bottom of Exploration at 47.80 feet below ground surface.	
50										

Remarks:

Bridge Deck 0.4 ft Pavement, 0.5 ft Concrete.
14.5 ft 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-KMR-103

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

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-104 WIN: 17079.00

Driller:	MaineDOT	Elevation (ft.)	9.3	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/19/10-10/20/10	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	16+55.8, 13.7 ft Lt.	Casing ID/OD:	NW	Water Level*:	Approx. 9.0 ft bgs
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	8.80	Pavement —0.50 Brown, damp, Gravelly, fine to coarse SAND, trace silt, few 0.3-0.7 ft dia. cobbles, (Fill).	
5	1D	24/12	5.00 - 7.00	10/17/16/11	33	46			Brown, damp, dense, fine to coarse SAND, some gravel, trace silt, occasional cobbes, (Fill).	G#240049 A-1-b, SM WC=4.8%
								0.30	—9.00	
10	2D	24/13	10.00 - 12.00	3/6/10/33	16	22	6		Brown, wet, medium dense, Gravelly, fine to coarse SAND, little silt.	
							24			
							55			
							20			
							16			
15	3D	24/18	15.00 - 17.00	5/2/1/2	3	4	27	-5.70	—15.00 Dark brown, wet, soft, SILT, some sand, little clay, with roots and organics.	G#240050 A-4, ML WC=139.4%
							37			
							38			
							40			
							41			
20	4D	24/20	20.00 - 22.00	3/1/2/2	3	4	61		Dark brown, wet, soft, SILT, some sand, trace clay, trace gravel.	G#240051 A-4, ML WC=116.4%
							51			
							54			
							52			
							49	-15.20	—24.50	
25	5D	24/6	25.50 - 27.50	4/2/2/3	4	6	130		Grey-brown, wet, loose, fine to coarse SAND, some gravel, little silt.	
							109			
							76			
							70			
							72			
30	6D	24/12	30.00 - 32.00	3/2/3/4	5	7	82		Grey, wet, loose, fine to coarse SAND, some gravel, trace silt.	G#240052 A-1-b, SW-SM WC=13.8%
							84			
							75			
							69			
							74			
35	7D	24/19	35.00 - 37.00	WOH/2/4/6	6	8	81		Grey, wet, loose, fine to coarse SAND, trace silt, trace gravel.	G#240053 A-1-b, SP-SM WC=14.8%
							93			
							85			
							86			
							86			
40	8D	24/15	40.00 - 42.00	3/3/5/5	8	11	82		Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#240054 A-1-b, SP-SM WC=13.6%
							86			
							117			
							113			
							120			
45	9D	24/20	45.00 - 47.00	2/7/11/10	18	25	110		Brown, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#240055 A-1-b, SM WC=14.9%
							108			
							122			
							162			
							178			
50	10D R1	3.6/3.6 60/60	50.00 - 50.30 50.30 - 55.30	50(3.6") RQD = 63%	---		a50 NQ-2	-41.00	a50 blows for 0.3 ft. Brown, wet, very dense, Gravelly, fine to coarse SAND, little silt. —50.30 Top of Bedrock at Elev. -41.0 ft. R1:Bedrock: Black, steeply bedded, fine grained, META-SILTSTONE/SANDSTONE, fresh, with 60 degree bedding, very hard, (Kittery Formation). Scattered quartz veins and pyrite along bedding planes. Rock Mass Quality = Fair. R1:Core Times (min:sec) 50.3-51.3 ft (5:32) 51.3-52.3 ft (3:50) 52.3-53.3 ft (3:10) 53.3-54.3 ft (3:00) 54.3-55.3 ft (3:10) 100% Recovery	
55	R2	60/56	55.30 - 60.30	RQD = 67%					R2:Bedrock: Simliar to above, with 30 to 60 degree bedding. Rock Mass Quality = Fair. R2:Core Times (min:sec) 55.5-56.5 ft (4:20) 56.5-57.5 ft (4:45) 57.5-58.5 ft (3:55) 58.5-59.5 ft (4:00) 59.5-60.5 ft (3:30) 93% Recovery	
60								-51.00	—60.30 Bottom of Exploration at 60.30 feet below ground surface.	
65										
70										
75										

Remarks:

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

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

Page 1 of 1 — Boring No.: BB-KMR-104

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BRIDGE PLANS

BR-1707(900)X

PIN 17079.00

BRIDGE NO. 2157

	BY	DATE
PROJ. MANAGER	B.Condon	
DESIGN-DETAILED	K.MAGUIRE	OCT 2010
CHECKED-REVIEWED	T.WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

CLAYHILL BRIDGE
MOUSAM RIVER
KENNEBUNK YORK COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-101
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	10.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/18/10-10/19/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+93.1, 1.4 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	9.55	Pavement (0.55)	
	1D	24/13	1.00 - 3.00	12/19/24/13	43	60			Brown, damp, very dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#240001 A-1-b, SM WC=4.3%
5	2D	24/14	5.00 - 7.00	2/3/2/4	5	7	29		Brown, damp, loose, fine to medium SAND, little silt, little gravel, (Fill).	G#240002 A-2-4, SM WC=9.0%
							22			
							15			
							17			
							21	1.10	(9.00)	
10	3D	24/18	10.00 - 12.00	2/3/3/6	6	8	17		Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	G#240003 A-3, SP-SM WC=15.8%
							23			
							38			
							60			
							134			
15	4D	24/16	15.50 - 17.50	9/3/11/11	14	20	59		Cobble from 15.0-15.3 ft bgs. Dark brown, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#240004 A-2-4, SP-SM WC=24.0%
							69			
							91			
							126			
							111			
20	5D	24/18	20.00 - 22.00	4/6/7/8	13	18	63		Grey-brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#240005 A-3, SP-SM WC=20.3%
							79			
							95			
							93			
25							93			

Remarks:

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

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

Boring No.: BB-KMR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-101
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	10.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/18/10-10/19/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+93.1, 1.4 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/17	25.00 - 27.00	4/5/5/4	10	14	69			Grey-brown, wet, medium dense, fine to coarse SAND, little silt.	G#240006 A-2-4, SP-SM WC=8.3%
							82				
							83				
							84				
							93				
30	7D	24/16	30.00 - 32.00	2/2/2/2	4	6	87			Grey, wet, loose, fine to coarse SAND, some silt, trace clay, trace organics.	G#240007 A-2-4, SC-SM WC=28.1%
							98				
							99				
							106				
							132				
35	8D	24/18	35.00 - 37.00	2/2/2/5	4	6	114			Dark grey, wet, loose, fine to coarse SAND, little silt, trace gravel, trace wood.	G#240008 A-2-4, SP-SM WC=29.7%
							133				
							125				
							135				
							133				
40	9D	24/19	40.00 - 42.00	2/2/6/10	8	11	121			Grey, wet, medium dense, fine to coarse SAND, some silt, some gravel, trace clay, occasional cobbles.	G#240009 A-2-4, SC-SM WC=27.3%
							136				
							116				
							116				
							123				
45	10D	24/18	45.00 - 47.00	2/3/2/5	5	7	97			Grey, wet, loose, fine to medium SAND, some silt, trace clay.	G#240010 A-2-4, SC-SM WC=38.4%
							124				
							127			Wood in wash water from 47.0-48.0 ft bgs.	
							130				
50							170				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-101
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	10.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/18/10-10/19/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+93.1, 1.4 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Insitu Field Vane Shear Strength (psf)
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q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/20	50.00 - 52.00	WOH/WOH/WOH/ WOH	---		124	-39.90	Grey, wet, very soft, SILT, some sand, trace clay, trace gravel. 50.00	G#240011 A-4, ML WC=40.5%
							168			
							166			
							167			
							168			
55	12D	24/5	55.00 - 57.00	3/7/13/14	20	28	167	-45.40	Grey, wet, medium dense, Gravelly, fine to coarse SAND, trace silt. 55.50	
							189			
							189			
							246			
							246			
60	13D	7.2/4	60.50 - 61.10	16/30(1.2")	---		195		Grey-brown, wet, very dense, silty, fine to coarse SAND, some gravel, (Till) .	
	R1	43.2/43.2	61.10 - 64.70	RQD = 50%			a100 NQ-2	-51.00	a100 blows for 0.1 ft. 61.10	
									Top of Bedrock at Elev. -51.0 ft. R1:Bedrock: Black, steeply bedded, very hard, limey, fine grained, META-SILTSTONE/SANDSTONE, fresh, with 60 degree bedding. Scattered quartz veins and pyrite along bedding planes, (Kittery Formation). Rock Mass Quality = Poor. R1:Core Times (min:sec) 61.1-62.1 ft (9:30) 62.1-63.1 ft (5:10) 63.1-64.1 ft (5:30) 64.1-64.7 ft (7:00) 100% Recovery Core Blocked	
	R2	26.4/15	64.70 - 66.90	RQD = 58%						
65										
	R3	50.4/50.4	66.90 - 71.10	RQD = 16%						
									R2:Bedrock: Similar to above. Rock Mass Quality = Fair. R2:Core Times (min:sec) 64.7-65.7 ft (7:41) 65.7-66.7 ft (7:30) 66.7-66.9 ft (3:00) 57% Recovery Core Blocked	
70								-61.00	R3:Bedrock: Similar to above. Rock Mass Quality = Very Poor. R3:Core Times (min:sec) 66.9-67.9 ft (5:10) 67.9-68.9 ft (9:05) 68.9-69.9 ft (6:55) 69.9-70.9 ft (7:10) 70.9-71.1 ft (2:30) 100% Recovery	
75										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-101
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	10.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/18/10-10/19/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+93.1, 1.4 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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.
75										71.10 **Bottom of Exploration at 71.10 feet below ground surface.**	
80											
85											
90											
95											
100											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-102
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	-5.5	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/20/10-10/21/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+44.8, 13.1 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	River Boring, Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
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G = Grain Size Analysis
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Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/1	0.00 - 2.00	2/2/2/6	4	6	2		Brown, wet, loose, fine to medium SAND, little silt, trace gravel, trace clay.	
							9			
							26			
							10	-8.50	3.00	
							5			
5	1D	24/12	5.00 - 7.00	4/3/3/1	6	8	WOH		Grey-brown, wet, loose, fine to coarse SAND, little silt, trace gravel, trace clay.	G#240012 A-2-4, SC-SM WC=24.3%
							8			
							15			
							23	-13.50	8.00	
							22			
10	2D	24/15	10.00 - 12.00	5/4/4/3	8	11	21		Grey, wet, medium dense, fine to medium SAND, some silt, trace clay, trace roots.	G#240013 A-4, SC-SM WC=27.5%
							15			
							31			
							43			
							34			
15	3D	24/20	15.00 - 17.00	WOH/WOH/2/3	2	3	35	-20.50	15.00 Grey, wet, very loose, Silty, fine to medium SAND, trace clay, trace gravel, trace roots.	G#240014 A-4, SC-SM WC=41.1%
							36			
							51			
							49			
							51			
20	4D	24/18	20.00 - 22.00	WOH/WOH/2/4	2	3	63	-25.50	20.00 Grey, wet, soft, Sandy SILT, trace clay, trace gravel.	G#240015 A-4, ML WC=39.3%
							62			
							73			
							89			
25							102			

Remarks:

Bridge Deck 0.4 ft Pavement, 0.5 ft Concrete.
20.0 ft 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-KMR-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-102
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	-5.5	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/20/10-10/21/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+44.8, 13.1 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	River Boring, Tidal
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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/22	25.00 - 27.00	WOH/WOH/2/4	2	3	105			Grey, wet, soft, Sandy SILT, trace clay.	G#240016 A-4, ML WC=40.3%
							73				
							76				
							76	-34.50		29.00	
							77				
30	6D	24/20	30.50 - 32.50	WOH/WOH/WOH/ WOH	---		111			Grey, wet, very soft, SILT, some sand, trace clay.	G#240017 A-4, ML WC=46.3%
							133				
							113	-38.50		33.00	
							108				
							100				
35	7D	24/12	35.00 - 37.00	8/8/5/8	13	18	151			Grey, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt.	G#240018 A-1-a, GW-GM WC=10.9%
							142				
							84				
							111				
	R1	60/60	39.50 - 44.50	RQD = 70%			[a]124 NQ-2	-45.00		[a]124 blows for 0.5 ft. 39.50	
40										Top of Bedrock at Elev. -45.0 ft. R1:Bedrock: Black to grey, fine grained, PHYLLITE and SILTSTONE, fresh, with augite crystals and quartz veins, very hard, (Kittery Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 39.5-40.5 ft (4:05) 40.5-41.5 ft (4:50) 41.5-42.5 ft (4:40) 42.5-43.5 ft (4:30) 43.5-44.5 ft (3:45) 100% Recovery	
								-50.00		44.50	
45										**Bottom of Exploration at 44.50 feet below ground surface.**	
50											

Remarks:

Bridge Deck 0.4 ft Pavement, 0.5 ft Concrete.
20.0 ft 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: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-103
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	-7.4	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/21/10; 10:30-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+03.8, 0.6 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	River Boring, Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/7	0.00 - 2.00	5/24/30/18	54	76	5		Grey, wet, very dense, Gravelly, fine to coarse SAND, trace silt.	G#240019 A-1-a, SW WC=15.0%
							34			
							14			
							14			
							16			
5	2D	24/12	5.00 - 7.00	4/4/5/3	9	13	2		Grey, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#240020 A-2-4, SP WC=19.0%
							8			
							9			
							13			
							22	-16.40	9.00	
10	3D	24/16	10.00 - 12.00	WOH/WOH/WOH/ WOH	---		10		Grey, wet, soft, fine to coarse, Sandy SILT, trace clay, trace gravel, trace organics, roots.	G#240021 A-4, ML WC=35.7%
							14			
							21			
							19			
							21			
15	4D	24/24	15.00 - 17.00	WOH/WOH/WOH/ WOH	---		25		Grey, wet, very soft, SILT, little clay, trace sand.	G#240022 A-4, ML WC=58.0%
							25			
							33			
							34			
							43			
20	5D	24/22	20.00 - 22.00	WOH/1/2/3	3	4	65		Grey, wet, soft, SILT, some sand, trace clay, trace gravel, trace wood fiber pieces.	G#240023 A-4, ML WC=53.1%
							58			
							50			
							56			
25							52			

Remarks:

Bridge Deck 0.4 ft Pavement, 0.5 ft Concrete.
14.5 ft from Bridge Deck to Ground.

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

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Boring No.: BB-KMR-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-103
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	-7.4	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/21/10; 10:30-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+03.8, 0.6 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	River Boring, Tidal
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/20	25.00 - 27.00	WOH/WOH/1/1	1	1	63		Grey, wet, very soft, SILT, little sand, trace clay, trace gravel.	G#240024 A-4, ML WC=67.9%
							67			
							105			
							93			
							101			
30	7D	24/22	30.00 - 32.00	1/2/1/1	3	4	99		Grey, wet, soft, SILT, some sand, trace clay.	G#240025 A-4, ML WC=47.1%
							82			
							63			
							74			
							65			
35	8D	24/18	35.00 - 37.00	WOH/WOH/WOH/ WOH	---		88		Grey, wet, very soft, SILT, little sand, trace clay.	G#240047 A-4, ML WC=54.4%
							89			
							78			
							89			
							86			
40	9D	24/15	40.00 - 42.00	4/1/3/3	4	6	118		Grey, wet, medium stiff, SILT, some sand, trace gravel, trace clay.	G#240048 A-4, ML WC=36.4%
							119			
	R1	60/60	42.80 - 47.80	RQD = 57%			[a]176 NQ-2	-50.20	[a]176 blows for 0.8 ft. 42.80	
									Top of Bedrock at Elev. -50.2 ft. R1:Bedrock: Dark grey to black, fine grained, laminated, META-SILTSTONE to PHYLLITE, fresh, with 60 to 80 degree bedding, very hard (Kittery Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 42.8-43.8 ft (5:00) 43.8-44.8 ft (3:20) 44.8-45.8 ft (3:40) 45.8-46.8 ft (3:00) 46.8-47.8 ft (2:50) 100% Recovery	
45								-55.20	47.80 **Bottom of Exploration at 47.80 feet below ground surface.**	
50										

Remarks:

Bridge Deck 0.4 ft Pavement, 0.5 ft Concrete.
14.5 ft from Bridge Deck to Ground.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-104
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	9.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/19/10-10/20/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+55.8, 13.7 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Approx. 9.0 ft 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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	8.80	Pavement (0.50)	
									Brown, damp, Gravelly, fine to coarse SAND, trace silt, few 0.3-0.7 ft dia. cobbles, (Fill).	
5	1D	24/12	5.00 - 7.00	10/17/16/11	33	46			Brown, damp, dense, fine to coarse SAND, some gravel, trace silt, occasional cobbes, (Fill).	G#240049 A-1-b, SM WC=4.8%
								0.30	(9.00)	
10	2D	24/13	10.00 - 12.00	3/6/10/33	16	22	6		Brown, wet, medium dense, Gravelly, fine to coarse SAND, little silt.	
							24			
							55			
							20			
							16			
15	3D	24/18	15.00 - 17.00	5/2/1/2	3	4	27	-5.70	(15.00) Dark brown, wet, soft, SILT, some sand, little clay, with roots and organics.	G#240050 A-4, ML WC=139.4%
							37			
							38			
							40			
							41			
20	4D	24/20	20.00 - 22.00	3/1/2/2	3	4	61		Dark brown, wet, soft, SILT, some sand, trace clay, trace gravel.	G#240051 A-4, ML WC=116.4%
							51			
							54			
							52			
25							49	-15.20	(24.50)	

Remarks:

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Boring No.: BB-KMR-104

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-104
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	9.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/19/10-10/20/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+55.8, 13.7 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Approx. 9.0 ft 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	5D	24/6	25.50 - 27.50	4/2/2/3	4	6	130			Grey-brown, wet, loose, fine to coarse SAND, some gravel, little silt.	
							109				
							76				
							70				
							72				
30	6D	24/12	30.00 - 32.00	3/2/3/4	5	7	82			Grey, wet, loose, fine to coarse SAND, some gravel, trace silt.	G#240052 A-1-b, SW-SM WC=13.8%
							84				
							75				
							69				
							74				
35	7D	24/19	35.00 - 37.00	WOH/2/4/6	6	8	81			Grey, wet, loose, fine to coarse SAND, trace silt, trace gravel.	G#240053 A-1-b, SP-SM WC=14.8%
							93				
							85				
							86				
							86				
40	8D	24/15	40.00 - 42.00	3/3/5/5	8	11	82			Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#240054 A-1-b, SP-SM WC=13.6%
							86				
							117				
							113				
							120				
45	9D	24/20	45.00 - 47.00	2/7/11/10	18	25	110			Brown, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#240055 A-1-b, SM WC=14.9%
							108				
							122				
							162				
50							178				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clayhill Bridge #2157 carries Route 9 over Mousam River.
Location: Kennebunk, Maine

Boring No.: BB-KMR-104
PIN: 17079.00

Driller:	MaineDOT	**Elevation (ft.)**	9.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/19/10-10/20/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+55.8, 13.7 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Approx. 9.0 ft bgs
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	3.6/3.6	50.00 - 50.30	50(3.6")	---		a50	-41.00		a50 blows for 0.3 ft.	
	R1	60/60	50.30 - 55.30	RQD = 63%			NQ-2			Brown, wet, very dense, Gravelly, fine to coarse SAND, little silt. 50.30	
										Top of Bedrock at Elev. -41.0 ft.	
										R1:Bedrock: Black, steeply bedded, fine grained, META-SILTSTONE/ SANDSTONE, fresh, with 60 degree bedding, very hard, (Kittery Formation). Scattered quartz veins and pyrite along bedding planes.	
										Rock Mass Quality = Fair.	
										R1:Core Times (min:sec)	
										50.3-51.3 ft (5:32)	
										51.3-52.3 ft (3:50)	
55	R2	60/56	55.30 - 60.30	RQD = 67%						52.3-53.3 ft (3:10)	
										53.3-54.3 ft (3:00)	
										54.3-55.3 ft (3:10) 100% Recovery	
										R2:Bedrock: Simliar to above, with 30 to 60 degree bedding.	
										Rock Mass Quality = Fair.	
										R2:Core Times (min:sec)	
										55.5-56.5 ft (4:20)	
										56.5-57.5 ft (4:45)	
										57.5-58.5 ft (3:55)	
										58.5-59.5 ft (4:00)	
60								-51.00		59.5-60.5 ft (3:30) 93% Recovery 60.30	
										Bottom of Exploration at 60.30 feet below ground surface.	
65											
70											
75											

Remarks:

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Kennebunk Project Number: 17079.00

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-KMR-101, 1D	14+93.1	1.4 Rt.	1.0-3.0	240001	1	4.3			SM	A-1-b	II
BB-KMR-101, 2D	14+93.1	1.4 Rt.	5.0-7.0	240002	1	9.0			SM	A-2-4	II
BB-KMR-101, 3D	14+93.1	1.4 Rt.	10.0-12.0	240003	1	15.8			SP-SM	A-3	0
BB-KMR-101, 4D	14+93.1	1.4 Rt.	15.5-17.5	240004	1	24.0			SP-SM	A-2-4	0
BB-KMR-101, 5D	14+93.1	1.4 Rt.	20.0-22.0	240005	1	20.3			SP-SM	A-3	0
BB-KMR-101, 6D	14+93.1	1.4 Rt.	25.0-27.0	240006	2	8.3			SP-SM	A-2-4	0
BB-KMR-101, 7D	14+93.1	1.4 Rt.	30.0-32.0	240007	2	28.1			SC-SM	A-2-4	II
BB-KMR-101, 8D	14+93.1	1.4 Rt.	35.0-37.0	240008	2	29.7			SP-SM	A-2-4	0
BB-KMR-101, 9D	14+93.1	1.4 Rt.	40.0-42.0	240009	2	27.3			SC-SM	A-2-4	II
BB-KMR-101, 10D	14+93.1	1.4 Rt.	45.0-47.0	240010	2	38.4			SC-SM	A-2-4	II
BB-KMR-101, 11D	14+93.1	1.4 Rt.	50.0-52.0	240011	2	40.5			ML	A-4	IV
BB-KMR-102, 1D	15+44.8	13.1 Lt.	5.0-7.0	240012	3	24.3			SC-SM	A-2-4	II
BB-KMR-102, 2D	15+44.8	13.1 Lt.	10.0-12.0	240013	3	27.5			SC-SM	A-4	III
BB-KMR-102, 3D	15+44.8	13.1 Lt.	15.0-17.0	240014	3	41.1			SC-SM	A-4	III
BB-KMR-102, 4D	15+44.8	13.1 Lt.	20.0-22.0	240015	4	39.3			ML	A-4	IV
BB-KMR-102, 5D	15+44.8	13.1 Lt.	25.0-27.0	240016	4	40.3			ML	A-4	IV
BB-KMR-102, 6D	15+44.8	13.1 Lt.	30.5-32.5	240017	4	46.3			ML	A-4	IV
BB-KMR-102, 7D	15+44.8	13.1 Lt.	35.0-37.0	240018	4	10.9			GW-GM	A-1-a	0
BB-KMR-103, 1D	16+03.8	0.6 Rt.	0.0-2.0	240019	5	15.0			SW	A-1-a	0
BB-KMR-103, 2D	16+03.8	0.6 Rt.	5.0-7.0	240020	5	19.0			SP	A-2-4	0
BB-KMR-103, 3D	16+03.8	0.6 Rt.	10.0-12.0	240021	5	35.7			ML	A-4	IV
BB-KMR-103, 4D	16+03.8	0.6 Rt.	15.0-17.0	240022	5	58.0			ML	A-4	IV
BB-KMR-103, 5D	16+03.8	0.6 Rt.	20.0-22.0	240023	6	53.1			ML	A-4	IV
BB-KMR-103, 6D	16+03.8	0.6 Rt.	25.0-27.0	240024	6	67.9			ML	A-4	IV
BB-KMR-103, 7D	16+03.8	0.6 Rt.	30.0-32.0	240025	6	47.1			ML	A-4	IV
BB-KMR-103, 8D	16+03.8	0.6 Rt.	35.0-37.0	240047	6	54.4			ML	A-4	IV
BB-KMR-103, 9D	16+03.8	0.6 Rt.	40.0-42.0	240048	6	36.4			ML	A-4	IV
BB-KMR-104, 1D	16+55.8	13.7 Lt.	5.0-7.0	240049	7	4.8			SM	A-1-b	II
BB-KMR-104, 3D	16+55.8	13.7 Lt.	15.0-17.0	240050	7	139			ML	A-4	IV
BB-KMR-104, 4D	16+55.8	13.7 Lt.	20.0-22.0	240051	7	116			ML	A-4	IV
BB-KMR-104, 6D	16+55.8	13.7 Lt.	30.0-32.0	240052	8	13.8			SW-SM	A-1-b	0
BB-KMR-104, 7D	16+55.8	13.7 Lt.	35.0-37.0	240053	8	14.8			SP-SM	A-1-b	0
BB-KMR-104, 8D	16+55.8	13.7 Lt.	40.0-42.0	240054	8	13.6			SP-SM	A-1-b	0
BB-KMR-104, 9D	16+55.8	13.7 Lt.	45.0-47.0	240055	8	14.9			SM	A-1-b	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

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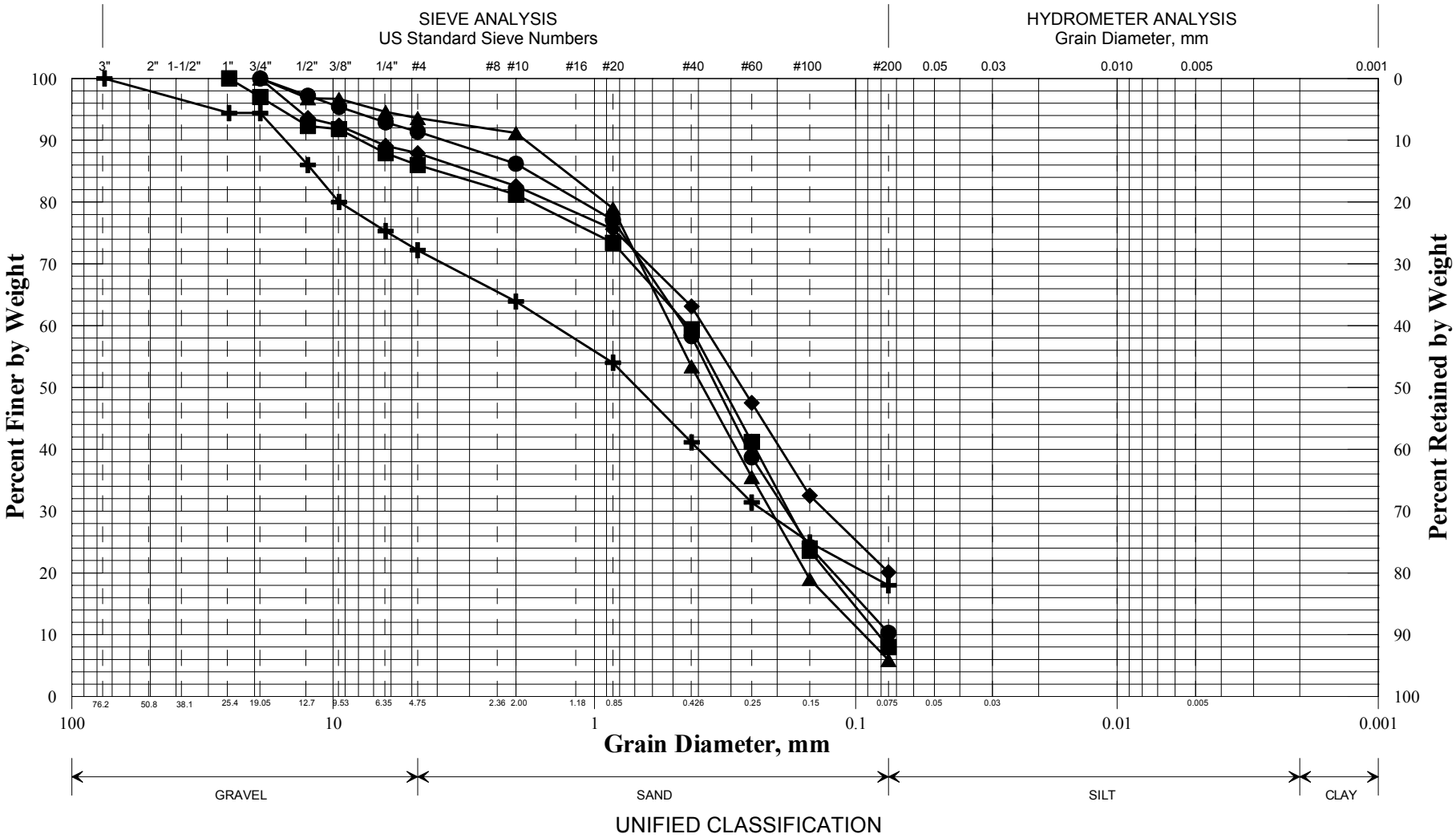
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-101/1D	14+93.1	1.4 RT	1.0-3.0	SAND, some gravel, little silt.	4.3			
◆	BB-KMR-101/2D	14+93.1	1.4 RT	5.0-7.0	SAND, little silt, little gravel.	9.0			
■	BB-KMR-101/3D	14+93.1	1.4 RT	10.0-12.0	SAND, little gravel, trace silt.	15.8			
●	BB-KMR-101/4D	14+93.1	1.4 RT	15.5-17.5	SAND, little silt, trace gravel.	24.0			
▲	BB-KMR-101/5D	14+93.1	1.4 RT	20.0-22.0	SAND, trace gravel, trace silt.	20.3			
✖									

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SHEET 1

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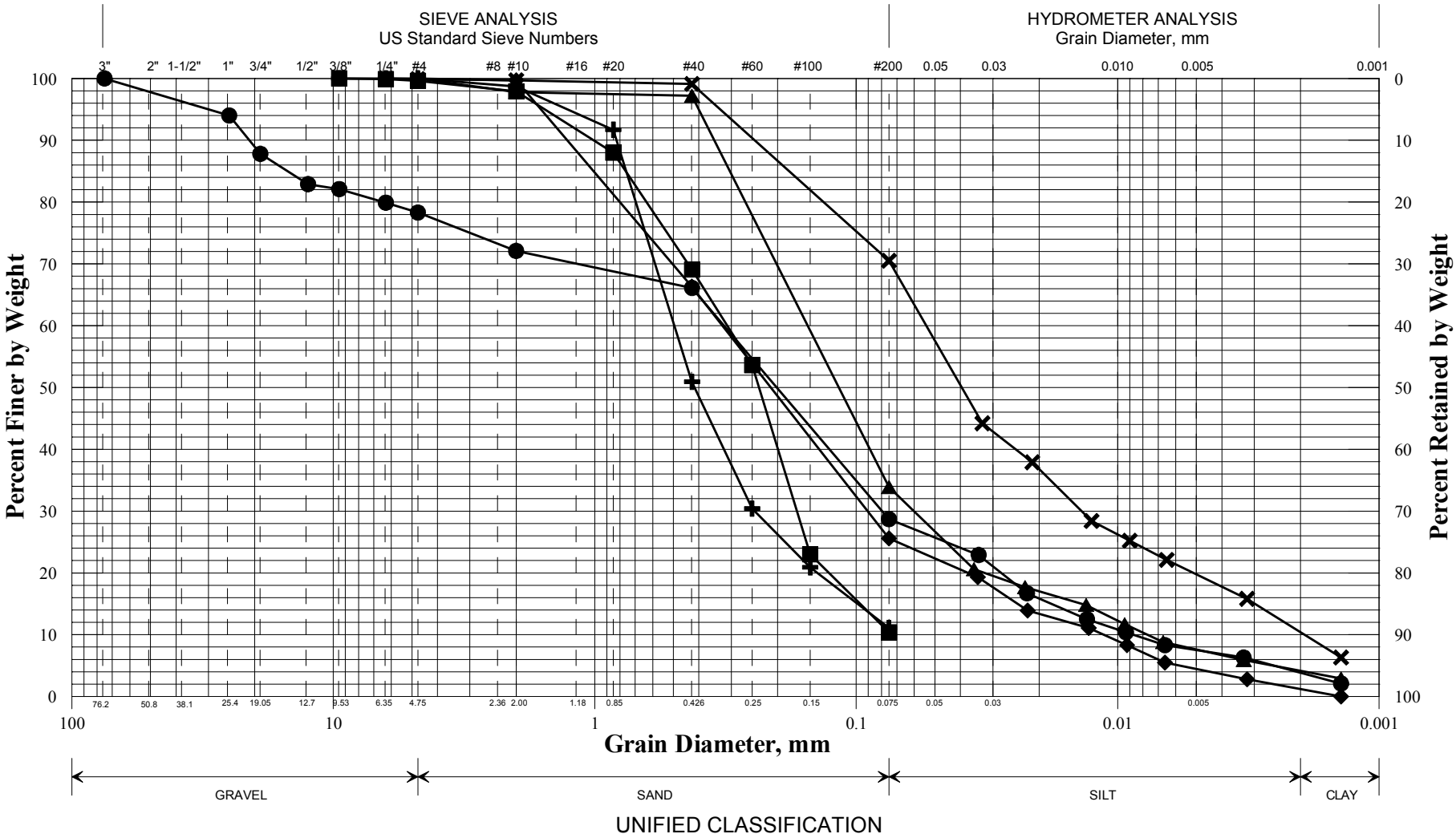
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-101/6D	14+93.1	1.4 RT	25.0-27.0	SAND, little silt.	8.3			
◆	BB-KMR-101/7D	14+93.1	1.4 RT	30.0-32.0	SAND, some silt, trace clay.	28.1			
■	BB-KMR-101/8D	14+93.1	1.4 RT	35.0-37.0	SAND, little silt, trace gravel.	29.7			
●	BB-KMR-101/9D	14+93.1	1.4 RT	40.0-42.0	SAND, some silt, some gravel, trace clay.	27.3			
▲	BB-KMR-101/10D	14+93.1	1.4 RT	45.0-47.0	SAND, some silt, trace clay.	38.4			
✕	BB-KMR-101/11D	14+93.1	1.4 RT	50.0-52.0	SILT, some sand, trace clay, trace gravel.	40.5			

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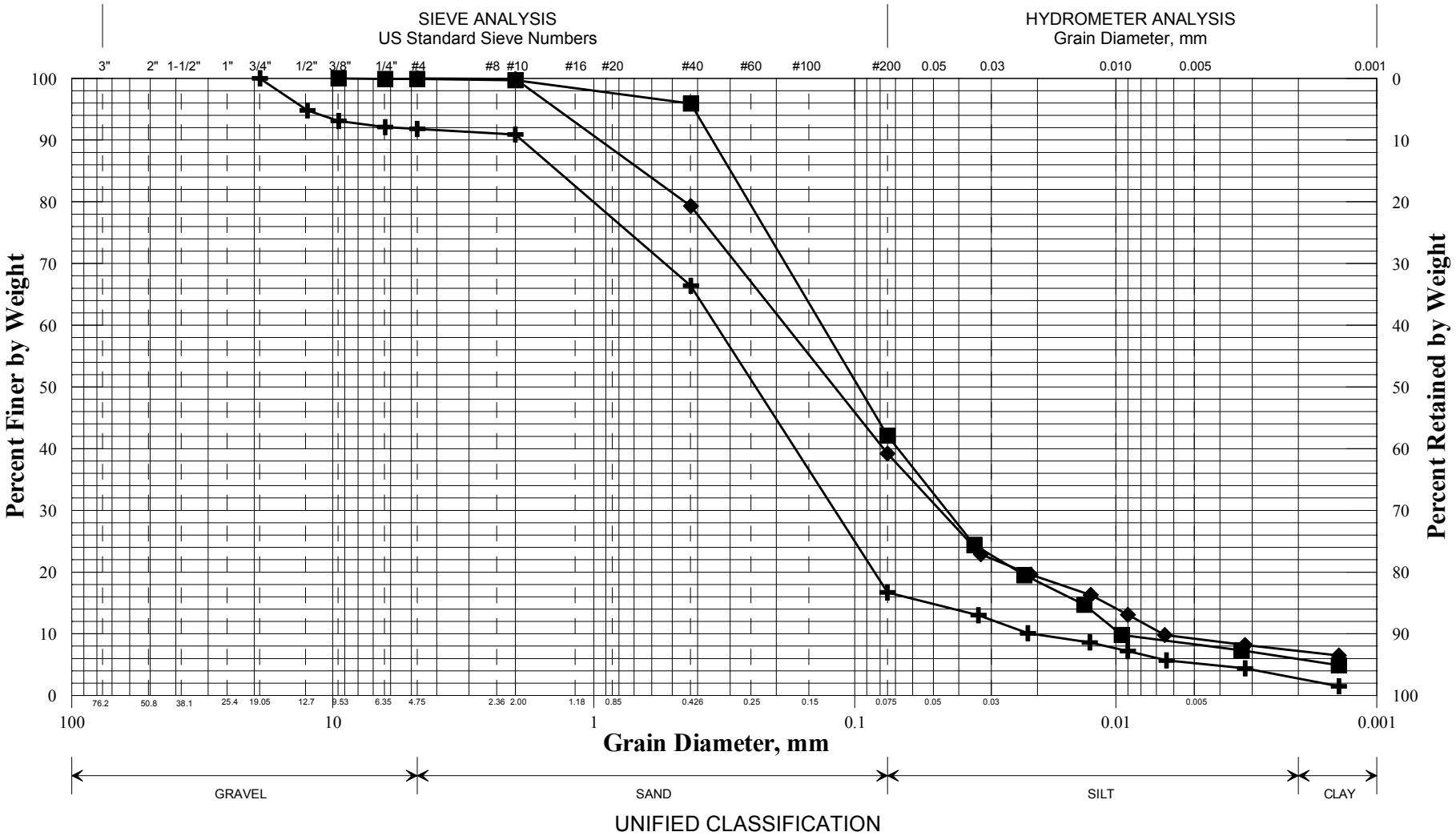

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-102/1D	15+44.8	13.1 LT	5.0-7.0	SAND, little silt, trace gravel, trace clay.	24.3			
◆	BB-KMR-102/2D	15+44.8	13.1 LT	10.0-12.0	SAND, some silt, trace clay.	27.5			
■	BB-KMR-102/3D	15+44.8	13.1 LT	15.0-17.0	Silty SAND, trace clay, trace gravel.	41.1			
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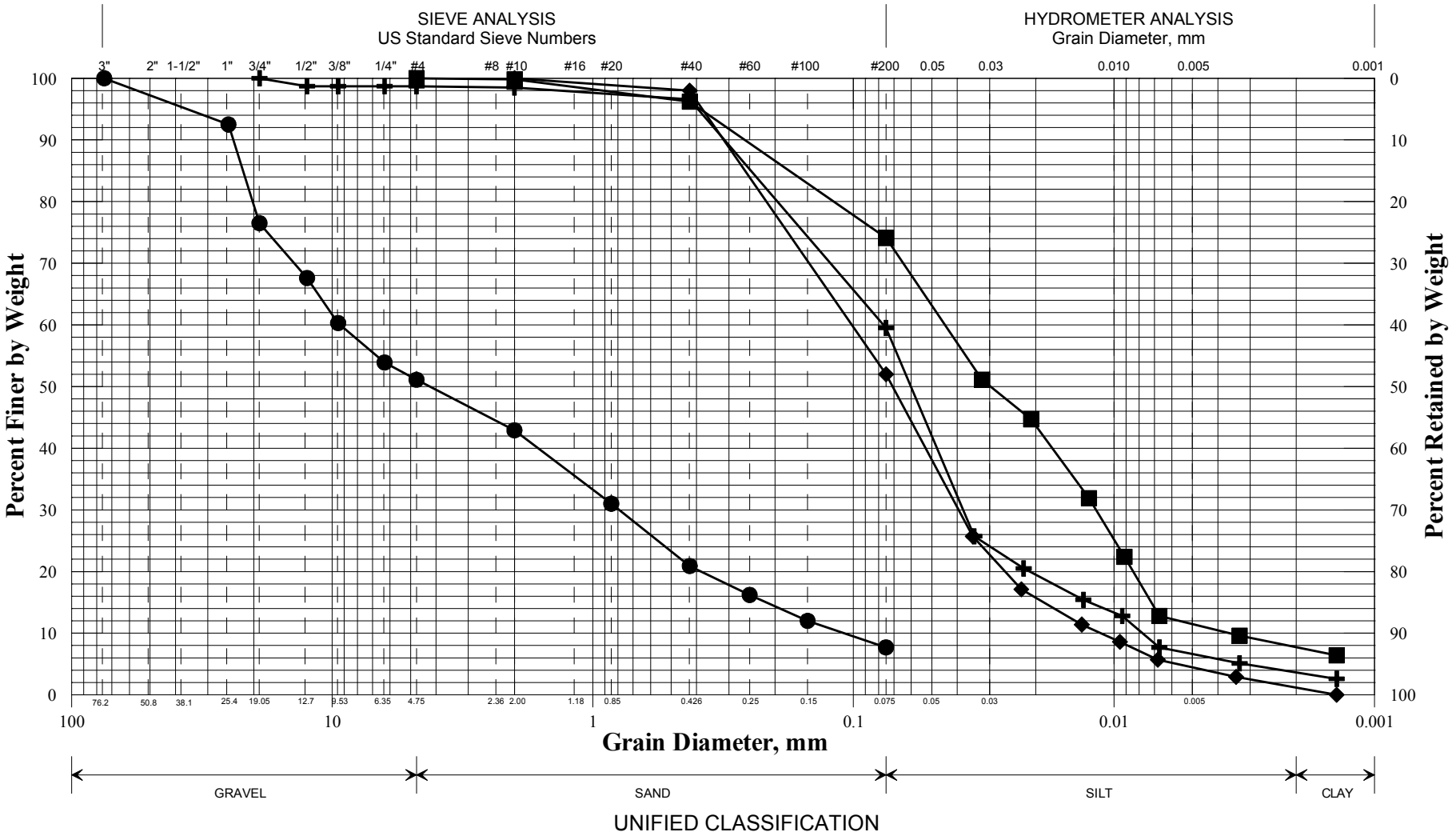

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-102/4D	15+44.8	13.1 LT	20.0-22.0	Sandy SILT, trace clay, trace gravel.	39.3			
◆	BB-KMR-102/5D	15+44.8	13.1 LT	25.0-27.0	Sandy SILT, trace clay.	40.3			
■	BB-KMR-102/6D	15+44.8	13.1 LT	30.5-32.5	SILT, some sand, trace clay.	46.3			
●	BB-KMR-102/7D	15+44.8	13.1 LT	35.0-37.0	Sandy GRAVEL, trace silt.	10.9			
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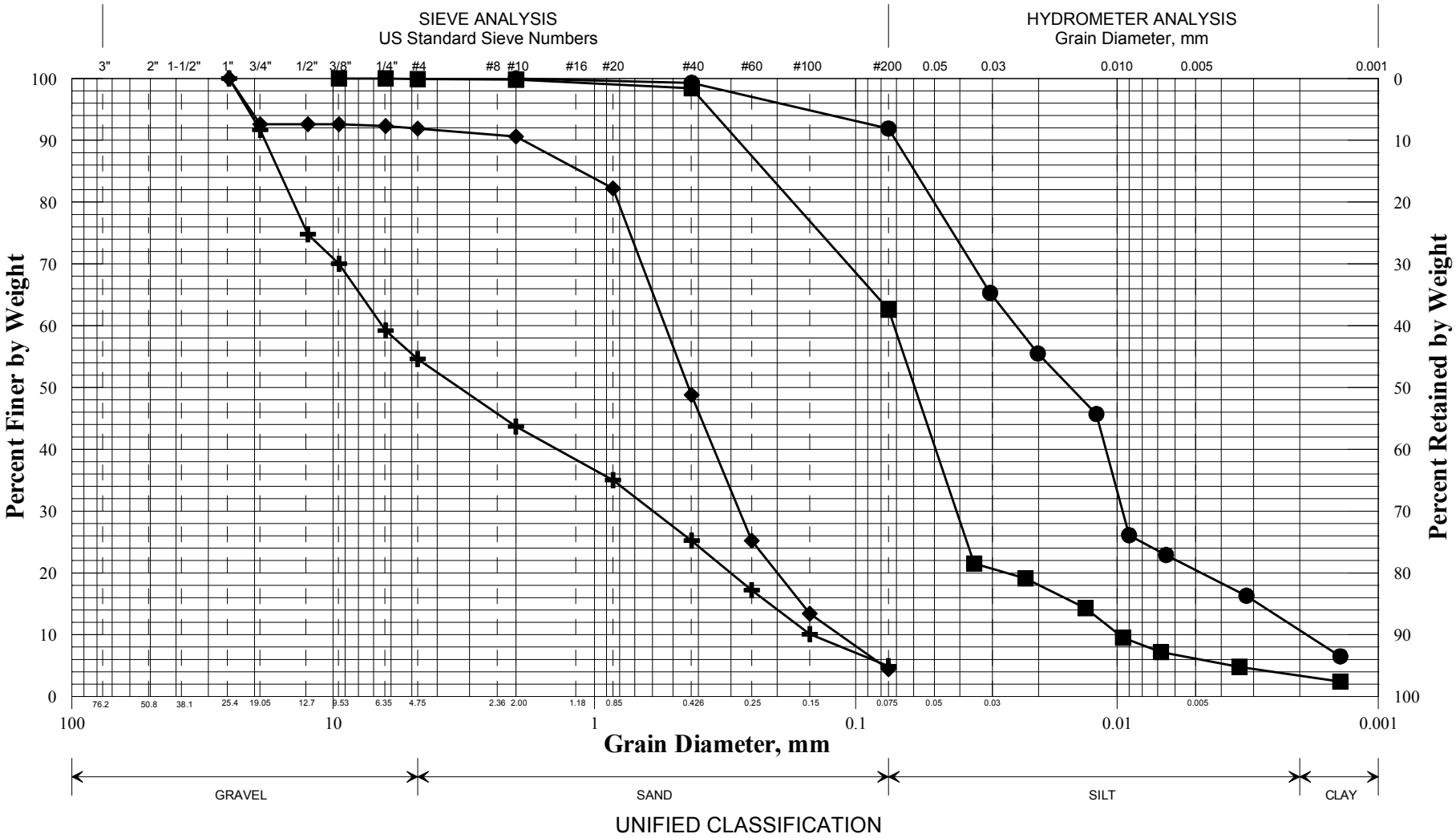

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-103/1D	16+03.8	0.6 RT	0.0-2.0	Gravelly SAND, trace silt.	15.0			
◆	BB-KMR-103/2D	16+03.8	0.6 RT	5.0-7.0	SAND, trace gravel, trace silt.	19.0			
■	BB-KMR-103/3D	16+03.8	0.6 RT	10.0-12.0	Sandy SILT, trace clay, trace gravel.	35.7			
●	BB-KMR-103/4D	16+03.8	0.6 RT	15.0-17.0	SILT, little clay, trace sand.	58.0			
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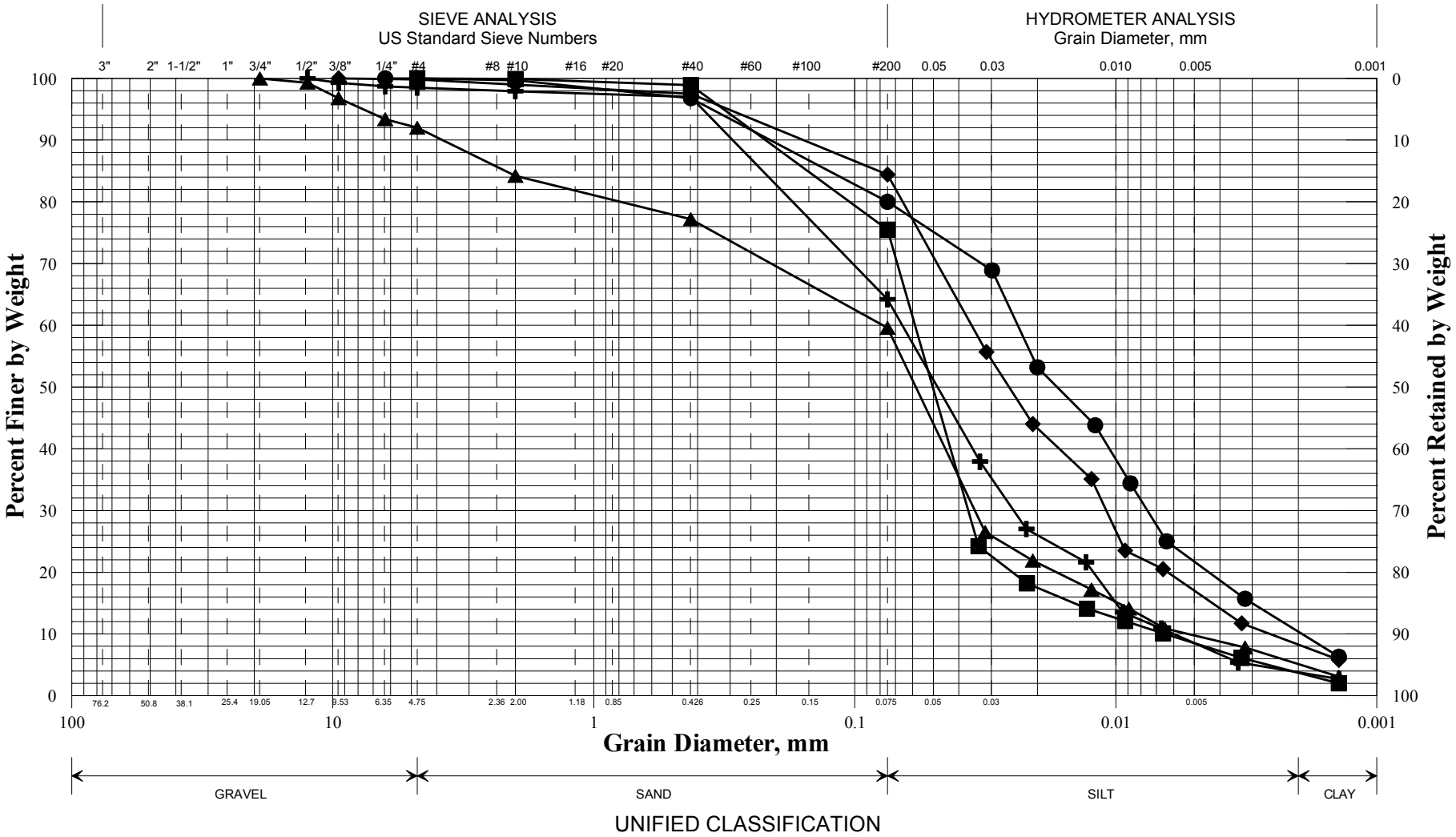

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-103/5D	16+03.8	0.6 RT	20.0-22.0	SILT, some sand, trace clay, trace gravel.	53.1			
◆	BB-KMR-103/6D	16+03.8	0.6 RT	25.0-27.0	SILT, little sand, trace clay, trace gravel.	67.9			
■	BB-KMR-103/7D	16+03.8	0.6 RT	30.0-32.0	SILT, some sand, trace clay.	47.1			
●	BB-KMR-103/8D	16+03.8	0.6 RT	35.0-37.0	SILT, little sand, trace clay.	54.4			
▲	BB-KMR-103/9D	16+03.8	0.6 RT	40.0-42.0	SILT, some sand, trace gravel, trace clay.	36.4			
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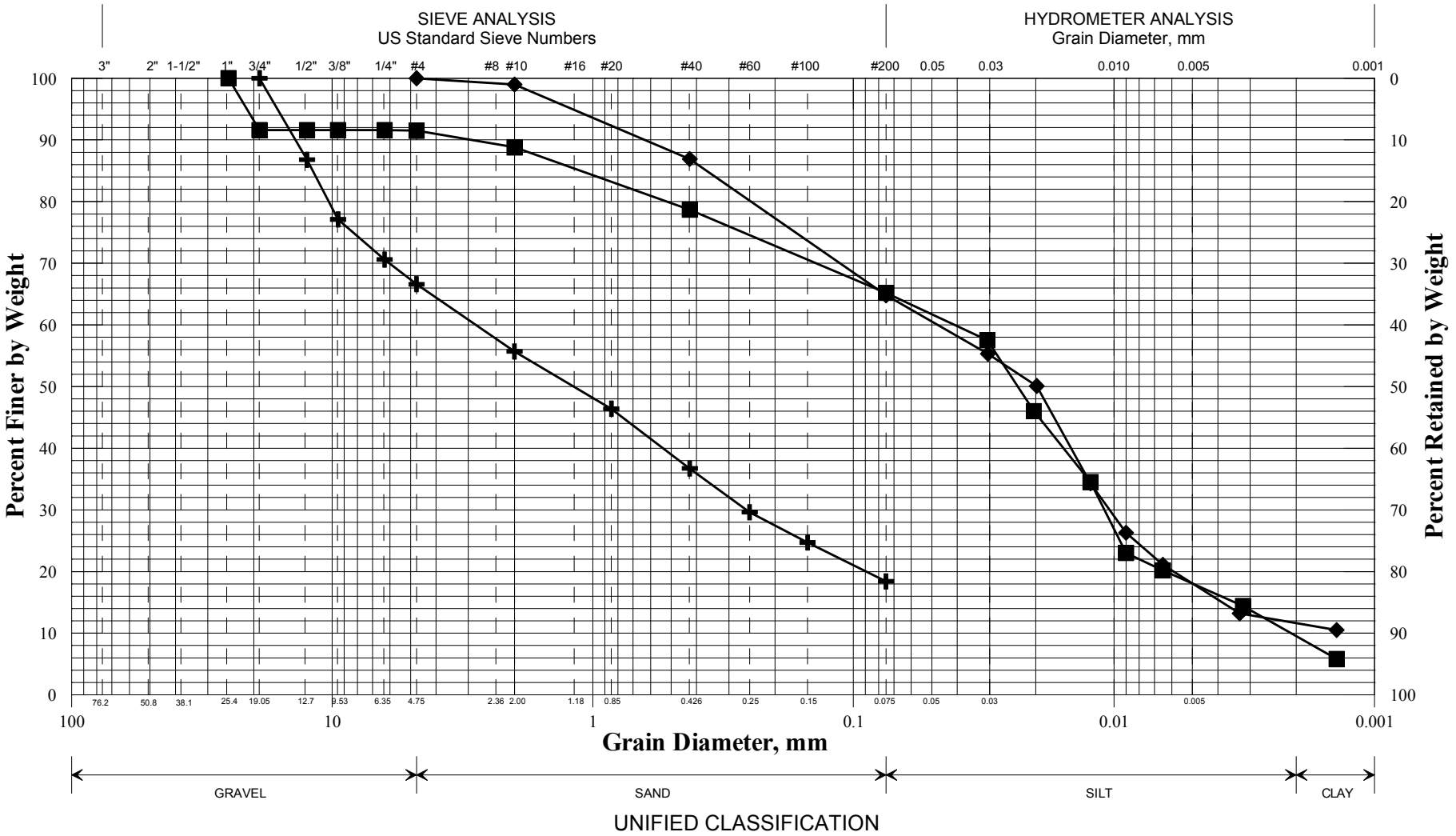

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-104/1D	16+55.8	13.7 LT	5.0-7.0	SAND, some gravel, little silt.	4.8			
◆	BB-KMR-104/3D	16+55.8	13.7 LT	15.0-17.0	SILT, some sand, little clay.	139.4			
■	BB-KMR-104/4D	16+55.8	13.7 LT	20.0-22.0	SILT, some sand, trace clay, trace gravel.	116.4			
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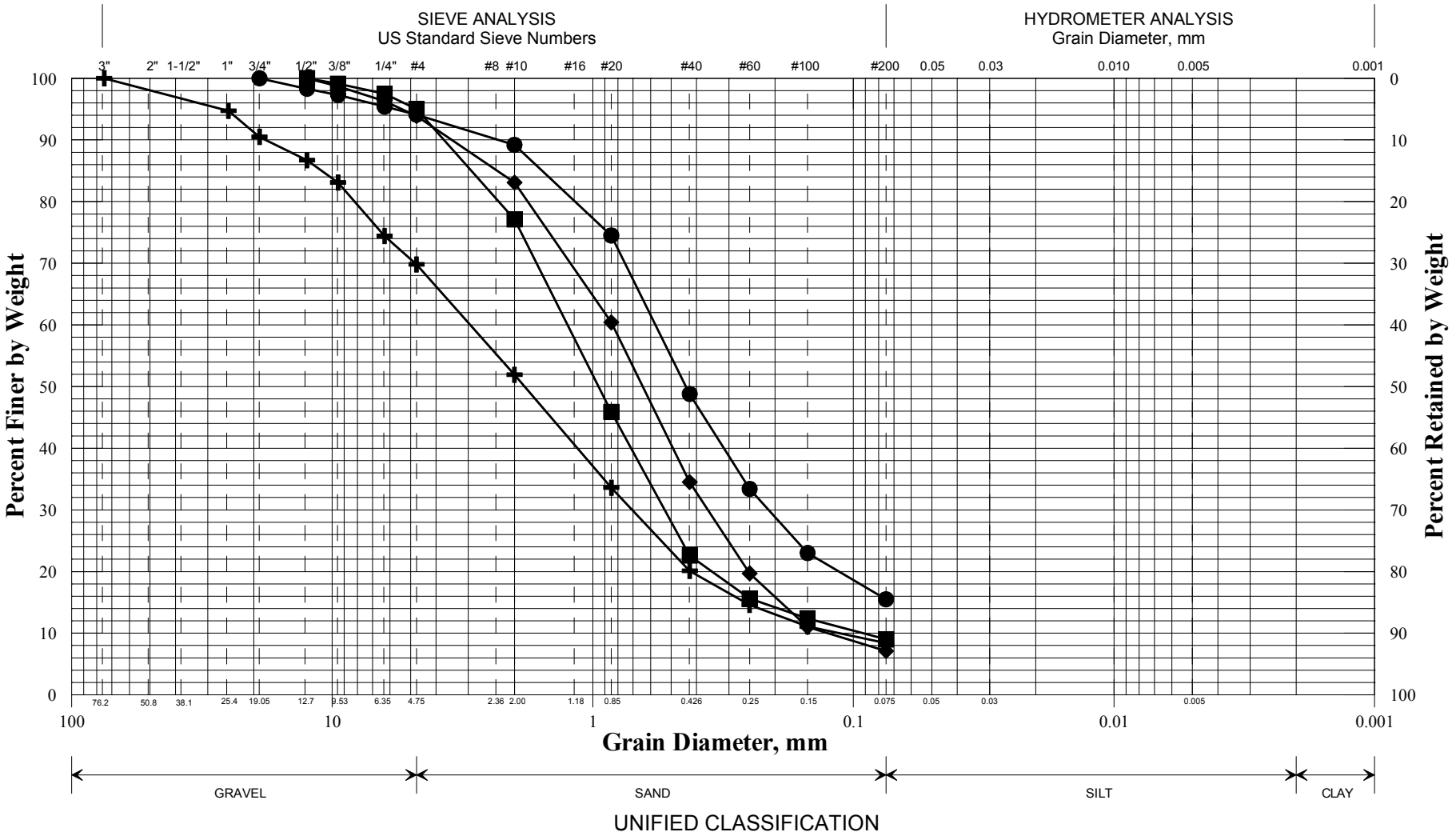

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-KMR-104/6D	16+55.8	13.7 LT	30.0-32.0	SAND, some gravel, trace silt.	13.8			
◆	BB-KMR-104/7D	16+55.8	13.7 LT	35.0-37.0	SAND, trace silt, trace gravel.	14.8			
■	BB-KMR-104/8D	16+55.8	13.7 LT	40.0-42.0	SAND, trace silt, trace gravel.	13.6			
●	BB-KMR-104/9D	16+55.8	13.7 LT	45.0-47.0	SAND, little silt, trace gravel.	14.9			
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Appendix C

Calculations

Abutment Foundations: Integral Driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

Look at the following piles:

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

Note: All matrices set up in this order

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

yield strength: $F_y := 50 \cdot ksi$

Determine equivalent yield resistance $P_o = QF_yA_s$ LRFD Article 6.9.4.1.1

$Q := 1.0$ LRFD Article 6.9.4.2 $F_y = 50 \cdot ksi$

$P_o := Q \cdot F_y \cdot A_s$

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

Determine elastic critical buckling resistance: $Pe = \pi^2 EA_s/(Kl/r_s)^2$ LRFD Eq. 6.9.4.1.2-1

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

K = effective length factor $K_{eff} := 1.2$

LRFD Table C4.6.2.5-1
Design Value of K when ideal conditions are approximated:
head - rotation fixed, translation free
tip - rotation fixed, translation fixed

l = unbraced length $l_{unbraced} := 48 \cdot in$ Assume 4 feet unbraced - scour

r_s = radius of gyration $r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot in$

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

LRFD Article C6.9.4.1.2 states that the critical flexural buckling resistances be calculated about the x- and y-axes with the smaller value taken as Pe.

Use y-axis as this results in the smaller value.

LRFD Eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s} \right)^2} \cdot A_s \right]}$$

$$P_e = \begin{pmatrix} 10937 \\ 16035 \\ 22486 \\ 28057 \\ 38247 \end{pmatrix} \cdot kip$$

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

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 14.1129 \\ 14.7112 \\ 21.0152 \\ 21.4997 \\ 22.2368 \end{pmatrix}$$

If Pe/Po> or = 0.44 then:

LRFD Equation 6.9.4.1.1-1

$$P_n := \left[\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o\right]$$

$$P_n = \begin{pmatrix} 752 \\ 1059 \\ 1049 \\ 1280 \\ 1688 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "good".

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance: Eq. 6.9.2.1-1

$$P_r := \phi_c \cdot P_n \qquad P_r = \begin{pmatrix} 451 \\ 636 \\ 629 \\ 768 \\ 1013 \end{pmatrix} \cdot kip$$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.5.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

Eq. 6.9.2.1-1

$$P_r := \phi \cdot P_n \qquad P_r = \begin{pmatrix} 752 \\ 1059 \\ 1049 \\ 1280 \\ 1688 \end{pmatrix} \cdot kip$$

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

Service/Extreme Limit States

Geotechnical Resistance

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

Bedrock Type:

Sandstone RQD ranges from 16 to 70%

Use RQD = 55% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

Look at these piles:

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

Note: All matrices set up in this order

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

Pile depth: $d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$

Pile width: $b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

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

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

q_u for sandstone compressive strength ranges from 9700 to 25000 psi

use $\sigma_c := 20000 \cdot psi$

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

Spacing of discontinuities: $c := 48 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b: $b = \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

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

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.6667 \\ 0.6614 \\ 0.6005 \\ 0.5981 \\ 0.5941 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 1 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$q_a := \sigma_c \cdot K_{sp} \cdot d_f$ $q_a = \begin{pmatrix} 1920 \\ 1905 \\ 1729 \\ 1723 \\ 1711 \end{pmatrix} \cdot ksf$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)}$ $R_p = \begin{pmatrix} 620 \\ 865 \\ 771 \\ 937 \\ 1226 \end{pmatrix} \cdot kip$

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

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

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

$R_f := \phi_{stat} \cdot R_p$ $R_f = \begin{pmatrix} 279 \\ 389 \\ 347 \\ 421 \\ 552 \end{pmatrix} \cdot kip$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.5.3

$\phi := 1.0$

$R_{fse} := \phi \cdot R_p$ $R_{fse} = \begin{pmatrix} 620 \\ 865 \\ 771 \\ 937 \\ 1226 \end{pmatrix} \cdot kip$

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

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

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

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

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1 Pile Drivability Analysis, Steel piles and 6.5.4.2 resistance during pile driving

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

Compute Resistance that can be achieved in a drivability analysis:

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

Table 10.5.5.2.3-1 pg 10-45 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

Pile Size = 12 x 53

Assume Contractor will use a Delmag D19-42 hammer

State of Maine Dept. Of Transportation — 28-Apr-2011
17079 Kennebunk Clayhill 12x53 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
445.0	44.76	6.42	6.7	9.27	22.23
446.0	44.88	6.45	6.7	9.28	22.29
447.0	44.96	6.47	6.8	9.29	22.34
448.0	44.99	6.47	6.8	9.30	22.32
449.0	45.09	6.49	6.8	9.31	22.38
450.0	45.13	6.50	6.9	9.32	22.37
451.0	45.20	6.51	6.9	9.34	22.40
452.0	45.31	6.54	7.0	9.35	22.45
453.0	45.36	6.56	7.0	9.36	22.50
454.0	45.41	6.56	7.0	9.37	22.49

Limited driving stress to 45 ksi

Strength Limit State:

$R_{dr\_12x53\_factored} := 448 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_12x53\_factored} = 291 \cdot kip$

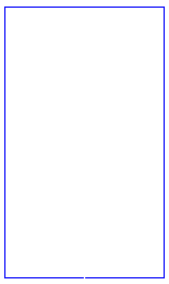
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_12x53\_servext} := 448 \cdot kip$

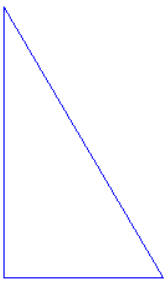
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	15.50	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 12 x 74

Assume Contractor will use a Delmag D19-42 hammer

State of Maine Dept. Of Transportation 28-Apr-2011
17079 Kennebunk Clayhill 12x74 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
620.0	43.26	7.34	14.5	9.56	21.67
621.0	43.26	7.37	14.6	9.56	21.69
622.0	43.31	7.40	14.6	9.57	21.72
623.0	43.34	7.43	14.7	9.57	21.74
624.0	43.36	7.41	14.8	9.58	21.72
625.0	43.41	7.44	14.9	9.58	21.74
626.0	43.41	7.48	14.9	9.58	21.77
627.0	43.46	7.45	15.1	9.59	21.74
628.0	43.47	7.48	15.2	9.59	21.77
629.0	43.51	7.52	15.2	9.60	21.80

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_12x74\_factored} := 626 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_12x74\_factored} = 407 \cdot kip$

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

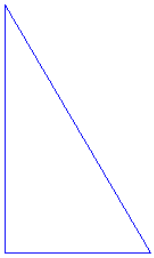
$R_{dr\_12x74\_servext} := 626 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	21.80	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D19-42 hammer

State of Maine Dept. Of Transportation 28-Apr-2011
17079 Kennebunk Clayhill 14x73 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
620.0	43.73	7.22	14.6	9.62	21.86
621.0	43.75	7.26	14.7	9.62	21.88
622.0	43.78	7.28	14.8	9.63	21.91
623.0	44.07	7.29	14.5	9.73	22.13
624.0	43.84	7.30	15.0	9.63	21.91
625.0	43.88	7.33	15.0	9.64	21.93
626.0	43.89	7.33	15.2	9.64	21.92
627.0	43.94	7.35	15.2	9.65	21.95
628.0	43.94	7.38	15.3	9.65	21.97
629.0	43.99	7.38	15.4	9.66	21.95

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x73\_factored} := 625 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x73\_factored} = 406 \cdot kip$

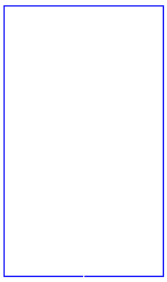
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x73\_servext} := 625 \cdot kip$

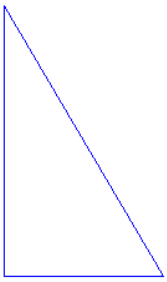
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89 **Assume Contractor will use a Delmag D36-32 hammer on lowest fuel setting**

State of Maine Dept. Of Transportation 18-Jul-2011
17079 Kennebunk Clayhill 14x89 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
725.0	44.91	4.93	9.5	7.18	29.94
726.0	44.95	4.93	9.5	7.19	30.03
727.0	44.99	4.93	9.5	7.19	30.05
728.0	45.01	4.94	9.6	7.20	30.05
729.0	45.04	4.94	9.6	7.20	30.06
730.0	45.08	4.95	9.7	7.21	30.07
731.0	45.11	4.95	9.7	7.21	30.08
732.0	45.14	4.96	9.8	7.22	30.10
733.0	45.19	4.96	9.8	7.22	30.10
734.0	45.22	4.96	9.8	7.23	30.11

Limited driving stress to 45 ksi

Strength Limit State:

$R_{dr\_14x89\_factored} := 728 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x89\_factored} = 473 \cdot kip$

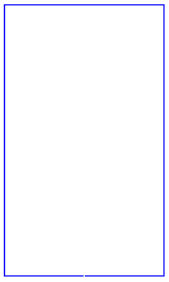
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x89\_servext} := 728 \cdot kip$

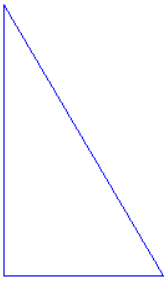
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	26.10	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 117 **Assume Contractor will use a Delmag D36-32 hammer on lowest fuel setting**

State of Maine Dept. Of Transportation — 18-Jul-2011
17079 Kennebunk Clayhill 14x117 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
890.0	42.78	4.54	14.3	7.37	29.73
892.0	42.87	4.57	14.4	7.37	29.80
894.0	42.94	4.59	14.4	7.39	29.85
896.0	42.92	4.60	14.5	7.38	29.86
898.0	43.00	4.61	14.7	7.39	29.85
900.0	43.04	4.62	14.8	7.39	29.85
902.0	43.08	4.65	14.9	7.40	29.92
904.0	43.08	4.66	15.0	7.40	29.91
906.0	43.10	4.66	15.2	7.41	29.90
908.0	43.20	4.69	15.2	7.41	29.97

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x117\_factored} := 904 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x117\_factored} = 588 \cdot kip$

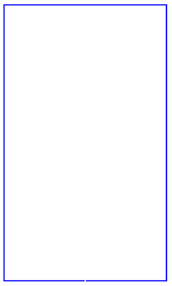
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x117\_servext} := 904 \cdot kip$

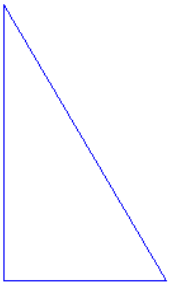
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	55.00	ft
Pile Top Area	34.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pipe Pile Supported Pier

Calculate Depth to Fixity for pipe piles:

Soil conditions at boring BB-KMR-102:
20 ft of sand, 13 ft of silt and 6.5 ft of gravel over bedrock

Consider Pile sizes:

24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Diameter of piles:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Pipe pile wall thickness:

$$wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

Corrosion loss per MaineDOT BDG:

$$cor := \frac{1}{8} in$$

$$dia_{steelcor} := dia_{steel} - 2 \cdot cor \qquad dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in \qquad wall_{cor} := wall_t - cor \qquad wall_{cor} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} \cdot in$$

$$dia_{conccore\_0.5} := dia_{steel} - 2 \cdot \frac{1}{2} \cdot in \qquad dia_{conccore\_0.5} = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} \cdot in$$

Diameter concrete core for 1/2" thick wall

$$dia_{conccore\_0.625} := dia_{steel} - 2 \cdot \frac{5}{8} \cdot in \qquad dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

$$A_{0.5} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.5}}{2}\right)^2 \qquad A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES with 1/8" corrosion loss

$$A_{0.625} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.625}}{2}\right)^2 \qquad A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES with 1/8" corrosion loss

Transformed pile properties of 1/2 inch wall pile:

unit weight of concrete: $wc := 0.15$ in kips per cubic foot

compressive strength of concrete: $f_c := 4.35$ in ksi Class A concrete

Modulus of elasticity of concrete: $E_c := 33000 \cdot wc^{1.5} \cdot \sqrt{f_c} \cdot 1000 \cdot psi$ $E_c = 3998 \cdot ksi$

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

$n := \frac{E_{steel}}{E_c}$ $n = 7.25$

MaineDOT Structural Engineers routinely use:

$n := 7.6$

Moment of inertia of concrete core:

$$I_{c\_0.5} := \frac{\pi \cdot dia_{conccore\_0.5}^{4}}{64} \qquad I_{c\_0.5} = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.5} := \frac{\pi \cdot \overrightarrow{\left(dia_{steelcor}^{4} - dia_{conccore\_0.5}^{4}\right)}}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.5} := \overrightarrow{\left(\frac{I_{c\_0.5}}{n} + I_{s\_0.5}\right)} \qquad I_{t\_0.5} = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.5} := \pi \cdot \frac{dia_{conccore\_0.5}^{2}}{4} \qquad A_{conc\_0.5} = \begin{pmatrix} 415.48 \\ 490.87 \\ 572.56 \\ 660.52 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.5} := A_{0.5} + \frac{A_{conc\_0.5}}{n} \qquad A_{t\_0.5} = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-2 for depth to fixity in feet: $1.8*(E_p I_w/n_h)^{0.2}$ (in sands)

E_p Young's modulus of pile in ksi

I_w moment of inertia of pile in ft^4

n_h= rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 in ksi/ft

$$E_{steel} = 2.9 \times 10^7 \text{ psi} \qquad I_{t\_0.5} = \begin{pmatrix} 0.1779 \\ 0.2377 \\ 0.3113 \\ 0.4003 \end{pmatrix} ft^4$$

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

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

Depth of Fixity:

$$D_{fix\_0.5} := 1.8 \cdot \left(\frac{E_{steel} \cdot I_{t\_0.5}}{n_h} \right)^{0.2}$$

$$D_{fix\_0.5} = \begin{pmatrix} 13.6 \\ 14.4 \\ 15.2 \\ 16 \end{pmatrix} ft$$

24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

Depth to fixity for 1/2" wall pipe piles

<u>Transformed pile properties of 5/8 inch wall pile:</u>

$$n = 7.6$$

Diameter of concrete core:

$$dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

Diameter of steel pipe

$$dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$$

Moment of inertia of concrete core:

$$I_{c\_0.625} := \frac{\pi \cdot dia_{conccore\_0.625}^{\ 4}}{64} \qquad I_{c\_0.625} = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.625} := \frac{\pi \cdot \left(dia_{steelcor}^{4} - dia_{conccore\_0.625}^{4}\right)}{64} \qquad I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.625} := \frac{I_{c\_0.625}}{n} + I_{s\_0.625} \qquad I_{t\_0.625} = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.625} := \pi \cdot \frac{dia_{conccore\_0.625}^{2}}{4} \qquad A_{conc\_0.625} = \begin{pmatrix} 406.49 \\ 481.11 \\ 562 \\ 649.18 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.625} := A_{0.625} + \frac{A_{conc\_0.625}}{n} \qquad A_{t\_0.625} = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-2 for depth to fixity in feet: $1.8*(E_p I_w/n_h)^{0.2}$ (in sands)

E_p Young's modulus of pile in ksi

I_w moment of inertia of pile in ft^4

n_h= rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 in ksi/ft

$$E_{steel} = 2.9 \times 10^7 \, psi \qquad I_{t\_0.625} = \begin{pmatrix} 0.2025 \\ 0.2694 \\ 0.3512 \\ 0.4498 \end{pmatrix} ft^4$$

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

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

Depth of Fixity:

$$D_{fix\_0.625} := 1.8 \cdot \left(\frac{E_{steel} \cdot I_{t\_0.625}}{n_h}\right)^{0.2}$$

$$D_{fix\_0.625} = \begin{pmatrix} 14 \\ 14.8 \\ 15.6 \\ 16.4 \end{pmatrix} ft$$

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Depth to fixity for 5/8" wall pipe piles

Nominal Axial Structural Resistance of pipe piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

Pier - Pipe Pile driven to bedrock

Axial pile resistance may be controlled by structural resistance if piles are driven to bedrock.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1 or 6.9.2.2-2.
Use LRFD Equation 6.9.5.1-1 or 6.9.5.1-2 to compute the nominal compressive structural resistance for pipe pile sections.

λ in Equation 6.9.5.1-2 has to be computed for the pipe piles since they have an unbraced length.

Yield strength of steel shell: $F_y := 45 \cdot ksi$

Compressive strength of concrete core: $f_c := 4350 \cdot psi$ Class A concrete

Yield strength of longitudinal reinforcement: $F_{yr} := 60 \cdot ksi$

Assume unsupported length is from bottom of pile cap including 13 feet of scour

Compute λ per 6.9.5.1-3 for composite members:

Effective length factor per LRFD Article 4.6.2.5:

Use case (e) in Table C4.6.2.5-1 $K := 2.0$

Exposed length of pile = Pile length through air + pile length trough water + calculated scour depth

Pile length through air + Pile length through water : $L_{air\_water} := 12 \cdot ft$

Scour depth calculated to be approximately 13 feet $L_{ex} := 13 \cdot ft$

Unbraced length of column:

$L_{UB} := L_{air\_water} + L_{ex}$ $L_{UB} = 25\,ft$ for all piles

Longitudinal reinforcement:

Assume longitudinal reinforcement of 12 - #8 bars (1-inch) bars equally spaced for all pile sections.

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.42 \cdot in^2$$

Composite Column Constant per Table 6.9.5.1-1

for tube filled sections: $C1 := 1.0$ $C2 := 0.85$ $C3 := 0.40$

Variable Fe:

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.5}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.5}}{A_{0.5}} \qquad F_{e\_0.5} = \begin{pmatrix} 121.32 \\ 124.63 \\ 128.18 \\ 131.91 \end{pmatrix} \cdot ksi$$ for 1/2" walls

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.625}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.625}}{A_{0.625}} \qquad F_{e\_0.625} = \begin{pmatrix} 101.64 \\ 104.11 \\ 106.76 \\ 109.55 \end{pmatrix} \cdot ksi$$ for 5/8" walls

Radius of gyration of both sets of steel sections:

$$r_{s\_0.5} := \overrightarrow{\sqrt{\frac{I_{s\_0.5}}{A_{0.5}}}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.6888 \\ 0.7477 \\ 0.8066 \\ 0.8655 \end{pmatrix} ft$$

for 1/2" walls

$$r_{s\_0.625} := \overrightarrow{\sqrt{\frac{I_{s\_0.625}}{A_{0.625}}}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.6852 \\ 0.7441 \\ 0.803 \\ 0.8619 \end{pmatrix} ft$$

for 5/8" walls

E_e term:

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc\_0.5}}{A_{0.5}}}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} \cdot ksi$$

for 1/2" walls

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc\_0.625}}{A_{0.625}}}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} \cdot ksi$$

for 5/8" walls

Lambda (λ) term for composite members LRFD Eq. 6.9.5.1-3

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 1.245 \\ 1.0445 \\ 0.8896 \\ 0.7673 \end{pmatrix}$$

for 1/2" walls

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 1.1925 \\ 1.0024 \\ 0.8552 \\ 0.7387 \end{pmatrix}$$

for 5/8" walls

Nominal Axial Structural Resistance of Composite member with 1/2-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_{0.5}\right)} \qquad P_{n\_0.5} = \begin{pmatrix} 1992 \\ 2414 \\ 2856 \\ 3319 \end{pmatrix} \cdot kip$$

for 1/2" walls

Nominal Axial Structural Resistance of Composite member with 5/8-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_{0.625}\right)} \qquad P_{n\_0.625} = \begin{pmatrix} 2262 \\ 2722 \\ 3203 \\ 3703 \end{pmatrix} \cdot kip$$

for 5/8" walls

Determine Axial Structural Resistance for Non-Composite Member (just steel shell)

Pipe pile Steel area:

$$A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls
with corrosion loss

$$A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls
with corrosion loss

yield strength: $F_y := 45 \cdot ksi$

Determine equivalent yield resistance $P_o = QF_yA_s$ LRFD Article 6.9.4.1.1

$Q := 1.0$ LRFD Article 6.9.4.2 $F_y = 45 \cdot ksi$

$P_{o0.5} := Q \cdot F_y \cdot A_{0.5}$

$$P_{o0.5} = \begin{pmatrix} 1239 \\ 1345 \\ 1451 \\ 1557 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$P_{o0.625} := Q \cdot F_y \cdot A_{0.625}$

$$P_{o0.625} = \begin{pmatrix} 1643 \\ 1785 \\ 1926 \\ 2068 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

Determine elastic critical buckling resistance: $Pe = \pi^2 EA_s/(Kl/r_s)^2$ LRFD Eq. 6.9.4.1.2-1

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

K = effective length factor $K_{eff} := 2.0$ LRFD Table C4.6.2.5-1 Design value of K when ideal conditions are approximated:
head: rotation fixed, translation free
tip: rotation free, translation fixed

l = unbraced length $L_{UB} = 25\,ft$ for all piles

r_s = radius of gyration

$$r_{s\_0.5} := \begin{pmatrix} 8.32 \\ 9.02 \\ 9.72 \\ 10.43 \end{pmatrix} \cdot in$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$r_{s\_0.625} := \begin{pmatrix} 8.27 \\ 8.97 \\ 9.68 \\ 10.39 \end{pmatrix} \cdot in$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

LRFD Eq. 6.9.4.1.2-1

$$P_{e\_0.5} := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot L_{UB}}{r_{s\_0.5}}\right)^2} \cdot A_{0.5}\right]} \qquad P_{e\_0.5} = \begin{pmatrix} 1516 \\ 1934 \\ 2422 \\ 2993 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{e\_0.625} := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot L_{UB}}{r_{s\_0.625}}\right)^2} \cdot A_{0.625}\right]} \qquad P_{e\_0.625} = \begin{pmatrix} 1986 \\ 2537 \\ 3189 \\ 3943 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

LRFD Article 6.9.4.1.1

$$\frac{P_{e\_0.5}}{P_{o0.5}} = \begin{pmatrix} 1.223 \\ 1.4375 \\ 1.6692 \\ 1.922 \end{pmatrix} \qquad \frac{P_{e\_0.625}}{P_{o0.625}} = \begin{pmatrix} 1.2084 \\ 1.4216 \\ 1.6555 \\ 1.9073 \end{pmatrix}$$

Nominal Axial Structural Resistance of Non-composite member with 1/2-inch wall

If Pe/Po> or = 0.44 then: LRFD Equation 6.9.4.1.1-1

$$P_{nc\_0.5} := \overrightarrow{\left[\left[0.658^{\left(\frac{P_{o0.5}}{P_{e\_0.5}}\right)}\right] \cdot P_{o0.5}\right]}$$

$$P_{nc\_0.5} = \begin{pmatrix} 880 \\ 1005 \\ 1129 \\ 1253 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

Nominal Axial Structural Resistance of Non-composite member with 5/8-inch wall

$$P_{nc\_0.625} := \overrightarrow{\left[\left[0.658^{\left(\frac{P_{o0.625}}{P_{e\_0.625}}\right)}\right] \cdot P_{o0.625}\right]}$$

$$P_{nc\_0.625} = \begin{pmatrix} 1162 \\ 1330 \\ 1496 \\ 1660 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

<u>Factored Axial Structural Resistance of a single Pipe Pile:</u>

Strength limit state resistance factor for pipe piles in compression, good driving conditions - LRFD 6.5.4.2

$$\phi_c := 0.7$$

Factored Structural Resistance (Pr):

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$$

$$P_{r\_0.5} = \begin{pmatrix} 1394 \\ 1690 \\ 1999 \\ 2323 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$$

$$P_{r\_0.625} = \begin{pmatrix} 1583 \\ 1906 \\ 2242 \\ 2592 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

Factored Structural Resistance (Pr) for the lower portion of open-ended piles or breached close-ended piles is a function of only the steel shell.

$$P_{r\_0.5tip} := \phi_c \cdot P_{nc\_0.5}$$

$$P_{r\_0.5tip} = \begin{pmatrix} 616 \\ 704 \\ 791 \\ 877 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{r\_0.625tip} := \phi_c \cdot P_{nc\_0.625}$$

$$P_{r\_0.625tip} = \begin{pmatrix} 814 \\ 931 \\ 1047 \\ 1162 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

Service and Extreme Limit States Axial Structural Resistance

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

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

$P_{\_0.5tipf} := \phi \cdot P_{nc\_0.5}$ $P_{\_0.5tipf} = \begin{pmatrix} 880 \\ 1005 \\ 1129 \\ 1253 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$P_{\_0.625tipf} := \phi \cdot P_{nc\_0.625}$ $P_{\_0.625tipf} = \begin{pmatrix} 1162 \\ 1330 \\ 1496 \\ 1660 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

COMPUTE GEOTECHNICAL RESISTANCE OF PIPE PILES

Pipe pile capacity based on steel shell end bearing on bedrock - driven through sand, and silt.

Bedrock Type:

Sandstone RQD ranges from 16 to 70%

Use RQD = 55% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Pipe piles evaluated:
24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Uniaxial Compressive Strength of Sandstone from AASHTO Standard Spec for Highway Bridges 17th Ed. Table 4.4.8.1.2B pg 64
Sandstone 9700 - 25000 psi Use 22000 psi

$Q_{uc} := 22000 \cdot psi$ $\phi_1 := 32 \cdot deg$

Diameter of piles: Pipe pile wall thickness: Corrosion loss per MaineDOT BDG:

$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$ $wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$ $cor := \frac{1}{8} in$

$$A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES with 1/8" corrosion loss

$$A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES with 1/8" corrosion loss

LRFD Code specifies Canadian Geotechnical Society Method 1985 for resistance determination of end bearing piles on bedrock. (LRFD Table 10.5.5.2.3-1)
Use Canadian Foundation Manual 4th Edition 2006 Section 18.6.3.3.

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

Spacing of discontinuities: $c := 48 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b:

$$b := dia_{steelcor} \qquad b = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$$

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 0 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_{aA} := Q_{uc} \cdot K_{sp} \cdot d_f \qquad q_{aA} = \begin{pmatrix} 1518 \\ 1471 \\ 1430 \\ 1395 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$R_{pA0.5} := \overrightarrow{(3q_{aA} \cdot A_{0.5})}$ $\quad R_{pA0.5} = \begin{pmatrix} 871 \\ 916 \\ 961 \\ 1006 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$R_{pA0.625} := \overrightarrow{(3q_{aA} \cdot A_{0.625})}$ $\quad R_{pA0.625} = \begin{pmatrix} 1155 \\ 1215 \\ 1275 \\ 1335 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:
Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

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

$R_{f0.5} := \phi_{stat} \cdot R_{pA0.5}$ $\quad R_{f0.5} = \begin{pmatrix} 392 \\ 412 \\ 432 \\ 453 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Strength Limit State for 1/2" walls

$R_{f0.625} := \phi_{stat} \cdot R_{pA0.625}$ $\quad R_{f0.625} = \begin{pmatrix} 520 \\ 547 \\ 574 \\ 601 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Strength Limit State for 5/8" walls

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

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

$\phi := 1.0$

$R_{fse0.5} := \phi \cdot R_{pA0.5}$ $\quad R_{fse0.5} = \begin{pmatrix} 871 \\ 916 \\ 961 \\ 1006 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Service/Extreme Limit States for 1/2" walls

$R_{fse0.625} := \phi \cdot R_{pA0.625}$ $\quad R_{fse0.625} = \begin{pmatrix} 1155 \\ 1215 \\ 1275 \\ 1335 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Service/Extreme Limit States for 5/8" walls

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
σ_{dr} = 0.9 x ϕ_{da} x f_y (Eq. 10.7.8-1)

$f_y := 45 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 40.5 \cdot ksi$ driving stresses in pile cannot exceed 40 ksi

Compute Resistance that can be achieved in a drivability analysis:

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

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

$\phi_{dyn} := 0.65$

Pile Size = 24"D x 1/2"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation 21-Jul-2011
17079 Kennebunk 24" x 1/2" pipe GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
735.0	40.26	6.21	7.6	7.33	33.17
736.0	40.33	6.22	7.6	7.34	33.17
737.0	40.38	6.23	7.6	7.34	33.24
738.0	40.38	6.24	7.6	7.35	33.26
739.0	40.42	6.25	7.6	7.36	33.32
740.0	40.50	6.25	7.7	7.36	33.33
741.0	40.50	6.26	7.7	7.37	33.33
742.0	40.52	6.28	7.7	7.38	33.41
743.0	40.62	6.29	7.8	7.38	33.42
744.0	40.63	6.30	7.8	7.39	33.48

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	27.54	in2

Limit driving stress to 40.5 ksi

Strength Limit State:

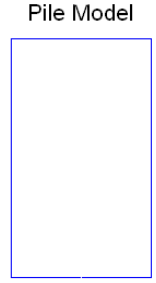
$R_{dr\_24x0.5\_factored} := 740 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_24x0.5\_factored} = 481 \cdot kip$

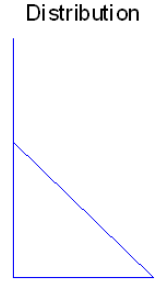
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_24x0.5\_servext} := 740 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use an APE D 36-26 hammer on lowest fuel setting

Pile Size = 26"D x 1/2"W

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation — 21-Jul-2011
17079 Kennebunk 26" x 1/2" pipe — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
800.0	40.27	5.94	9.3	7.45	32.90
801.0	40.33	5.96	9.3	7.45	32.97
802.0	40.37	5.96	9.4	7.46	32.96
803.0	40.38	5.96	9.4	7.46	32.97
804.0	40.38	5.97	9.5	7.47	32.97
805.0	40.40	5.98	9.5	7.47	33.03
806.0	40.49	5.98	9.5	7.48	33.03
807.0	40.51	5.99	9.6	7.48	33.03
808.0	40.54	6.00	9.6	7.49	33.09
809.0	40.54	6.00	9.6	7.49	33.10

APE D 36-26

Limit driving stress to 40.5 psi

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	29.89	in2

Strength Limit State:

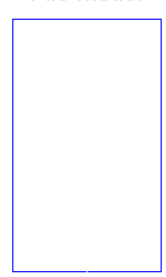
$R_{dr\_26x0.5\_factored} := 807 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_26x0.5\_factored} = 525 \cdot kip$

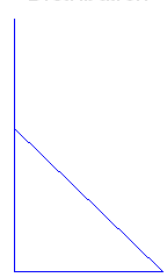
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_26x0.5\_servext} := 807 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use an APE D 36-26 hammer on lowest fuel setting

Pile Size = 28"D x 1/2"W

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation 21-Jul-2011
17079 Kennebunk 28"x1/2" pipe GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
870.0	40.37	5.66	11.4	7.58	32.71
871.0	40.42	5.65	11.5	7.59	32.71
872.0	40.46	5.65	11.5	7.59	32.70
873.0	40.49	5.67	11.5	7.60	32.75
874.0	40.49	5.67	11.6	7.60	32.76
875.0	40.50	5.67	11.6	7.60	32.75
876.0	40.52	5.67	11.7	7.61	32.74
877.0	40.58	5.68	11.7	7.61	32.81
878.0	40.61	5.68	11.7	7.61	32.82
879.0	40.62	5.68	11.8	7.61	32.82

APE D 36-26

Limit driving stress to 40.5 ksi

Strength Limit State:

$R_{dr\_28x0.5\_factored} := 875 \cdot kip \cdot \phi_{dyn}$

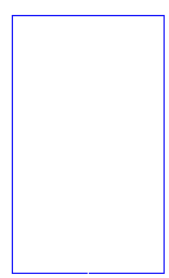
$R_{dr\_28x0.5\_factored} = 569 \cdot kip$

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

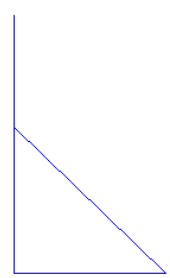
$R_{dr\_28x0.5\_servext} := 875 \cdot kip$

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	32.25	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use an APE D 36-26 hammer on lowest fuel setting

Pile Size = 30"D x 1/2"W

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation 21-Jul-2011
17079 Kennebunk 30"x1/2" pipe GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
930.0	40.08	5.46	13.4	7.62	32.71
935.0	40.17	5.48	13.6	7.63	32.75
940.0	40.27	5.51	13.8	7.65	32.85
945.0	40.37	5.52	14.1	7.66	32.89
950.0	40.48	5.54	14.3	7.68	33.00
955.0	40.55	5.55	14.6	7.69	33.04
960.0	40.64	5.58	14.8	7.71	33.14
965.0	40.80	5.59	15.0	7.73	33.25
970.0	40.90	5.61	15.3	7.74	33.30
975.0	40.99	5.63	15.6	7.76	33.39

APE D 36-26

Limit driving stress to 40.5 ksi

Strength Limit State:

$R_{dr\_30x0.5\_factored} := 955 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_30x0.5\_factored} = 621 \cdot kip$

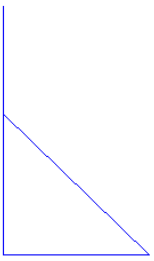
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_30x0.5\_servext} := 955 \cdot kip$

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	34.61	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use an APE D 36-26 hammer on lowest fuel setting

Pile Size = 24"D x 5/8"W

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation — 21-Jul-2011
17079 Kennebunk 24"x5/8" pipe — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
970.0	39.51	5.75	14.9	7.60	32.40
971.0	39.57	5.77	14.9	7.60	32.46
972.0	39.58	5.77	15.0	7.60	32.45
973.0	39.60	5.78	15.0	7.61	32.44
974.0	39.61	5.80	15.0	7.61	32.51
975.0	39.64	5.80	15.1	7.61	32.49
976.0	39.66	5.80	15.2	7.61	32.48
977.0	39.69	5.82	15.2	7.62	32.55
978.0	39.67	5.82	15.3	7.62	32.53
979.0	39.71	5.81	15.3	7.62	32.52

APE D 36-26

Limit blow count to 15 bpi

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	36.52	in2

Strength Limit State:

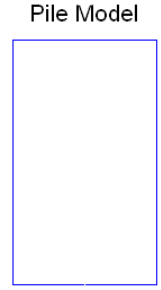
$R_{dr\_24x0.625\_factored} := 974 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_24x0.625\_factored} = 633 \cdot kip$

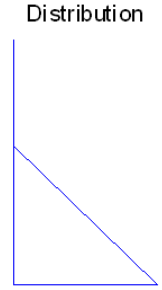
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_24x0.625\_servext} := 974 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use an APE D 36-26 hammer on lowest fuel setting

Pile Size = 26"D x 5/8"W

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation 21-Jul-2011
17079 Kennebunk 26"x5/8" pipe GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
980.0	37.71	5.79	14.8	7.42	31.19
981.0	37.73	5.82	14.8	7.42	31.25
982.0	37.72	5.82	14.8	7.42	31.23
983.0	37.77	5.83	14.8	7.42	31.30
984.0	37.78	5.83	14.9	7.42	31.27
985.0	37.78	5.84	15.0	7.43	31.26
986.0	37.80	5.86	15.0	7.43	31.31
987.0	37.83	5.87	15.1	7.43	31.31
988.0	37.84	5.86	15.2	7.43	31.29
989.0	37.85	5.88	15.1	7.43	31.36

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_26x0.625\_factored} := 986 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_26x0.625\_factored} = 641 \cdot kip$

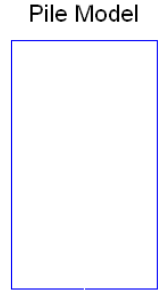
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_26x0.625\_servext} := 986 \cdot kip$

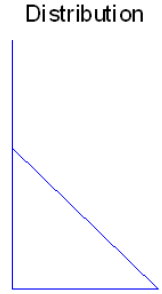
APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	39.66	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use an APE D 36-26 hammer on lowest fuel setting

Pile Size = 28"D x 5/8"W

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation — 21-Jul-2011
17079 Kennebunk 28"x5/8" pipe — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1008.0	36.55	5.74	14.9	7.35	30.60
1009.0	36.56	5.76	14.9	7.36	30.65
1010.0	36.56	5.76	15.0	7.35	30.64
1011.0	36.60	5.76	15.1	7.36	30.62
1012.0	36.61	5.78	15.1	7.36	30.67
1013.0	36.60	5.79	15.2	7.36	30.65
1014.0	36.64	5.79	15.2	7.36	30.63
1015.0	36.67	5.80	15.2	7.37	30.69
1016.0	36.68	5.81	15.3	7.37	30.67
1017.0	36.71	5.83	15.3	7.37	30.73

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_28x0.625\_factored} := 1010 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_28x0.625\_factored} = 657 \cdot kip$

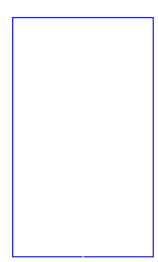
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_28x0.625\_servext} := 1010 \cdot kip$

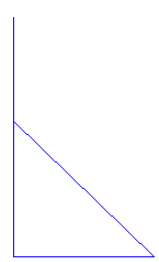
APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	42.80	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use an APE D 36-26 hammer on lowest fuel setting

Pile Size = 30"D x 5/8"W

Pile Bent Pier: Unbraced length = length in air + length in water + preliminary scour depth 13 ft = 25 ft.

State of Maine Dept. Of Transportation 21-Jul-2011
17079 Kennebunk 30"x5/8" pipe GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1035.0	35.41	5.56	15.0	7.28	29.89
1036.0	35.41	5.56	15.0	7.28	29.87
1037.0	35.45	5.57	15.0	7.28	29.94
1038.0	35.44	5.58	15.1	7.28	29.90
1039.0	35.46	5.59	15.1	7.28	29.96
1040.0	35.45	5.59	15.2	7.28	29.94
1041.0	35.52	5.60	15.2	7.29	29.99
1042.0	35.51	5.60	15.3	7.29	29.96
1043.0	35.53	5.60	15.3	7.29	29.94
1044.0	35.54	5.62	15.3	7.29	29.99

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_30x0.625\_factored} := 1037 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_30x0.625\_factored} = 674 \cdot kip$

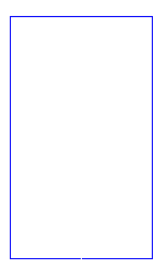
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_30x0.625\_servext} := 1037 \cdot kip$

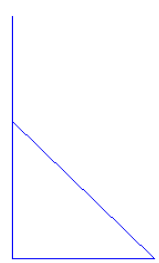
APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	58.00	ft
Pile Penetration	33.00	ft
Pile Top Area	45.95	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Abutment and Wingwall Passive and Active Earth Pressure:

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

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

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

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

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

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

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

$K_p = 6.89$

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

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

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

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

$K_{p\_rank} = 3.25$

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

Settlement Analysis:

Reference: FHWA Soils and Foundations Reference Manual - Volume 1 (FHWA NHI-06-088) Hough pg 7-16 and AASHTO LRFD Bridge Design Specifications 5th Edition 2010

The roadway grade at centerline may be raised by as much as 1.85 feet .
Look at a simplified soil profile based on BB-KMR-104

Finished Grade

Proposed Fill - Look at 1.85 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Existing Grade

Existing Fill and Native Sand - fine to coarse sand

Groundwater at 9.0 ft bgs

$\gamma_w := 62.4pcf$

$H_{1fill} := 9.0 \cdot ft$ $\gamma_{fill} := 125 \cdot pcf$ $N_{fill} := 46$

$H_{1sand} := 6.0 \cdot ft$ $\gamma_{1sand} := 125 \cdot pcf$ $N_{1sand} := 22$

Silt - soft
Total Layer height: H = 9.5 ft - divide into 3 layers

$H_{2silt1} := 3.2 \cdot ft$ $\gamma_{silt} := 115 \cdot pcf$ $N_{silt1} := 4$

$H_{2silt2} := 3.2 \cdot ft$ $N_{silt2} := 4$

$H_{2silt3} := 2.0 \cdot ft$ $N_{silt3} := 4$

Sand - fine to coarse sand, loose to medium dense

$H_3 := 26.0 \cdot ft$ $\gamma_{3sand} := 125 \cdot pcf$ $N_{3sand} := 11$

Bedrock - sandstone

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

Project Name:	Clayhill Bridge	Client:	Kennebunk
Project Number:	17079.00	Project Manager:	BCondon
Date:	05/11/11	Computed by:	km

Embank. slope a = 10.00(ft)
Embank. width b = 30.00(ft)
p load/unit area = 231.25(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 20.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	231.25
1.00	231.18
2.00	230.73
3.00	229.59
4.00	227.57
5.00	224.63
6.00	220.82
7.00	216.27
8.00	211.14
9.00	205.60
10.00	199.80
11.00	193.86
12.00	187.89
13.00	181.97
14.00	176.16
15.00	170.49
16.00	165.00
17.00	159.70
18.00	154.62
19.00	149.74
20.00	145.07
21.00	140.61
22.00	136.35
23.00	132.29
24.00	128.41
25.00	124.72
26.00	121.19
27.00	117.83
28.00	114.63
29.00	111.57
30.00	108.65
31.00	105.86
32.00	103.19
33.00	100.64
34.00	98.21
35.00	95.87
36.00	93.64
37.00	91.49
38.00	89.44
39.00	87.47
40.00	85.58
41.00	83.76
42.00	82.01
43.00	80.33
44.00	78.72
45.00	77.16
46.00	75.66
47.00	74.21
48.00	72.82
49.00	71.47

at 4.5 ft
$\Delta\sigma_{zfill} := 226.1 \cdot psf$

at 12.0 ft
$\Delta\sigma_{zsand1} := 187.89 \cdot psf$

at 16.6 ft
$\Delta\sigma_{zsilt1} := 162.88 \cdot psf$

at 19.8 ft
$\Delta\sigma_{zsilt2} := 148.81 \cdot psf$

at 22.4 ft
$\Delta\sigma_{zsilt3} := 133.91 \cdot psf$

at 37.5 ft
$\Delta\sigma_{zsand3} := 90.47 \cdot psf$

Existing Fill $tsf := psf \cdot 1000$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress: $\sigma_{1fill\_o} := \frac{H_{1fill}}{2} \cdot (\gamma_{fill})$ $\sigma_{1fill\_o} = 0.563 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_{fill} = 46$

At P_o = 0.563 tsf $C_{N\_1fill} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{1fill\_o}}\right)$ Article 10.4.6.2.4 LRFD

$C_{N\_1fill} = 1.426$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_1fill} \cdot N_{fill}$ $N1_{60} = 66$
From LRFD Eq 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C_{fill} := 190$

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

$\Delta\sigma_{zfill} = 226.1 \cdot psf$

Native Sand

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$\sigma_{1sand\_o} := \left[\frac{H_{1sand}}{2} \cdot (\gamma_{1sand} - \gamma_w)\right] + H_{1fill} \cdot (\gamma_{fill})$ $\sigma_{1sand\_o} = 1.3128 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_{1sand} = 22$

At P_o = 1.3 tsf $C_{N\_1sand} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{1sand\_o}}\right)$ Article 10.4.6.2.4 LRFD

$C_{N\_1sand} = 1.1426$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_1sand} \cdot N_{1sand}$ $N1_{60} = 25$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C_{1sand} := 80$

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

$\Delta\sigma_{zsand1} = 187.89 \cdot psf$

Silt - 3 layers

Silt Layer 1:

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{silt1\_o} := \left[\frac{H_{2silt1}}{2} \cdot (\gamma_{silt} - \gamma_w)\right] + H_{1fill} \cdot \gamma_{fill} + H_{1sand} \cdot (\gamma_{1sand} - \gamma_w) \qquad \sigma_{silt1\_o} = 1.5848 \cdot tsf$$

at mid-point

Corrected SPT N_{60}-value (bpf) $N_{silt1} = 4$

At P_o = 1.6 tsf

$$C_{N\_silt1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{silt1\_o}}\right)$$

Article 10.4.6.2.4 LRFD

$$C_{N\_silt1} = 1.0796$$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_silt1} \cdot N_{silt1}$ $\qquad N1_{60} = 4$
From LRFD Eq 10.4.6.2.4-1

From Figure 7-7 pg 7-17 using the "Inorganic silt" curve

Bearing Capacity Index: $C_{silt1} := 20$

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

$$\Delta\sigma_{zsilt1} = 162.88 \cdot psf$$

Silt Layer 2:

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{silt2\_o} := \left[\frac{H_{2silt2}}{2} \cdot (\gamma_{silt} - \gamma_w)\right] + H_{2silt1} \cdot (\gamma_{silt} - \gamma_w) + H_{1fill} \cdot \gamma_{fill} + H_{1sand} \cdot (\gamma_{1sand} - \gamma_w) \qquad \sigma_{silt2\_o} = 1.7531 \cdot tsf$$

at mid-point

Corrected SPT N_{60}-value (bpf) $N_{silt2} = 4$

At P_o = 1.75 tsf

$$C_{N\_silt2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{silt2\_o}}\right)$$

Article 10.4.6.2.4 LRFD

$$C_{N\_silt2} = 1.0459$$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_silt2} \cdot N_{silt2}$ $\qquad N1_{60} = 4$
From LRFD Eq 10.4.6.2.4-1

From Figure 7-7 pg 7-17 using the "Inorganic silt" curve

Bearing Capacity Index: $C_{silt2} := 20$

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

$$\Delta\sigma_{zsilt2} = 148.81 \cdot psf$$

Silt Layer 3:

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{silt3\_o} := \left[\frac{H_{2silt3}}{2} \cdot (\gamma_{silt} - \gamma_w)\right] + (H_{2silt1} + H_{2silt2}) \cdot (\gamma_{silt} - \gamma_w) + H_{1fill} \cdot \gamma_{fill} + H_{1sand} \cdot (\gamma_{1sand} - \gamma_w)$$

$\sigma_{silt3\_o} = 1.8898 \cdot tsf$

Corrected SPT N_{60}-value (bpf) $N_{silt3} = 4$

At P_o = 1.9 tsf

$$C_{N\_silt3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{silt3\_o}}\right)$$

Article 10.4.6.2.4 LRFD

at mid-point

$C_{N\_silt3} = 1.0207$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_silt3} \cdot N_{silt3}$ $N1_{60} = 4$
From LRFD Eq 10.4.6.2.4-1

From Figure 7-7 pg 7-17 using the "Inorganic silt" curve

Bearing Capacity Index: $C_{silt3} := 20$

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

$\Delta\sigma_{zsilt3} = 133.91 \cdot psf$

Sand

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{3sand\_o} := \left[\frac{H_3}{2} \cdot (\gamma_{3sand} - \gamma_w)\right] + (H_{2silt1} + H_{2silt2} + H_{2silt3}) \cdot (\gamma_{silt} - \gamma_w) + H_{1fill} \cdot \gamma_{fill} + H_{1sand} \cdot (\gamma_{1sand} - \gamma_w)$$

$\sigma_{3sand\_o} = 2.7562 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_{3sand} = 11$

At P_o = 2.8 tsf

$$C_{N\_3sand} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{3sand\_o}}\right)$$

Article 10.4.6.2.4 LRFD

$C_{N\_3sand} = 0.8945$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3sand} \cdot N_{3sand}$ $N1_{60} = 10$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C_{3sand} := 50$

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

$\Delta\sigma_{zsand3} = 90.47 \cdot psf$

Calculate Settlement:

Existing Fill: $\Delta H_{1fill} := H_{1fill} \cdot \frac{1}{C_{fill}} \cdot \log\left(\frac{\sigma_{1fill\_o} + \Delta\sigma_{zfill}}{\sigma_{1fill\_o}}\right)$ $\Delta H_{1fill} = 0.0834 \cdot in$

Native Sand: $\Delta H_{1sand} := H_{1sand} \cdot \frac{1}{C_{1sand}} \cdot \log\left(\frac{\sigma_{1sand\_o} + \Delta\sigma_{zsand1}}{\sigma_{1sand\_o}}\right)$ $\Delta H_{1sand} = 0.0523 \cdot in$

Silt Layer 1: $\Delta H_{2silt1} := H_{2silt1} \cdot \frac{1}{C_{silt1}} \cdot \log\left(\frac{\sigma_{silt1\_o} + \Delta\sigma_{zsilt1}}{\sigma_{silt1\_o}}\right)$ $\Delta H_{2silt1} = 0.0816 \cdot in$

Silt Layer 2: $\Delta H_{2silt2} := H_{2silt2} \cdot \frac{1}{C_{silt2}} \cdot \log\left(\frac{\sigma_{silt2\_o} + \Delta\sigma_{zsilt2}}{\sigma_{silt2\_o}}\right)$ $\Delta H_{2silt2} = 0.0679 \cdot in$

Silt Layer 3: $\Delta H_{2silt3} := H_{2silt3} \cdot \frac{1}{C_{silt3}} \cdot \log\left(\frac{\sigma_{silt3\_o} + \Delta\sigma_{zsilt3}}{\sigma_{silt3\_o}}\right)$ $\Delta H_{2silt3} = 0.0357 \cdot in$

Sand: $\Delta H_{3sand} := H_3 \cdot \frac{1}{C_{3sand}} \cdot \log\left(\frac{\sigma_{3sand\_o} + \Delta\sigma_{zsand1}}{\sigma_{3sand\_o}}\right)$ $\Delta H_{3sand} = 0.1787 \cdot in$

Total Settlement =

$$\Delta H_T := \Delta H_{1fill} + \Delta H_{1sand} + \Delta H_{2silt1} + \Delta H_{2silt2} + \Delta H_{2silt3} + \Delta H_{3sand}$$

$\Delta H_T = 0.4996 \cdot in$ Say less than 1" of settlement - occurring during construction.

Frost Protection:

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

From the Design Freezing Index Map:
Kennebunk, Maine
DFI = 1100 degree-days

From the lab testing: soils are coarse grained assume a water content = ~10%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1100 frost penetration = 69.8 inches

$Frost\_depth := 69.8in$ $\quad$ $Frost\_depth = 5.8167 \cdot ft$

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

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Sanford

--- ModBerg Results ---

Project Location: Sanford 2 NNW, Maine

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

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	60.2	10.0	125.0	28	34	2.0	1.6	1,800

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 = 5.02 ft = 60.2 in.
****

$Frost\_depth_{modberg} := 60.2 \cdot in$ $\quad$ $Frost\_depth_{modberg} = 5.0167\,ft$

Use Modberg Frost Depth = 5.0 feet for design

Seismic:

17079 Kennebunk Clayhill Bridge
Date and Time: 5/9/2011 3:22:45 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 43.350000
Longitude = -070.520000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.093	PGA - Site Class B
0.2	0.182	Ss - Site Class B
1.0	0.044	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 43.350000
Longitude = -070.520000
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.233	As - Site Class E
0.2	0.454	SDs - Site Class E
1.0	0.156	SD1 - Site Class E