

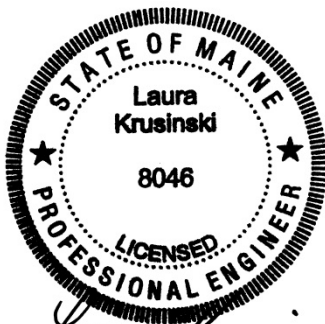
**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE**

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**GOODELL BRIDGE
ELLIOTSVILLE ROAD OVER DAVIS BROOK
WILLIMANTIC, MAINE**

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Goodell Bridge which carries Elliotsville Road over Davis Brook in Willimantic, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical recommendations, and geotechnical design parameters for the design of the new substructures.

The existing bridge structure was constructed in 1932 and has a span of 36 feet. The superstructure consists of painted steel girders and timber decking. The substructure consists of concrete capped stacked stone abutments constructed on soil. According to the 2017 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the abutments have settled and large voids are present between the stacked stones. The existing structure's Sufficiency Rating is 38.5. The bridge is classified as Structurally Deficient and scour critical.

The proposed replacement structure consists of a 76-foot single span precast concrete NEXT beam superstructure founded on H-pile supported integral abutments. The existing structure will be removed and 1.75H:1V (Horizontal:Vertical) riprap slopes will be placed in front of the new integral abutments. The new Goodell Bridge will be designed to match the existing horizontal alignment and the vertical profile will be increased approximately 1 foot from the existing. The curb-to-curb width of the new bridge will be increased to 26 feet. Elliotsville Road will be closed at the bridge project and traffic detoured during construction of the new bridge.

2.0 GEOLOGIC SETTING

The existing bridge carries Elliotsville Road over Davis Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Sebec Lake Quadrangle, Maine, Open-file No. 86-37 (1986), indicates the surficial soils in the vicinity of the bridge project consist of glacial outwash deposits with nearby glacial till. Glacial outwash deposits were deposited by meltwater streams and currents during melting of the Late Wisconsin glacier and generally consist of sand and gravel, but may include glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. Glacial till was deposited directly by glacial ice and is generally a poorly sorted mixture of sand, gravel, and silt.

The MGS Bedrock Geologic Map of Sebec Lake Quadrangle, Open File No. 71-6 (1971), cites the bedrock at the project site as a cyclically bedded dark gray metasiltstone and shale of the Solon Formation.

3.0 SUBSURFACE INVESTIGATION

Four test borings explored subsurface conditions at the site. Borings BB-WDB-101, BB-

WDB-101A, and BB-WDB-201, were drilled north of the existing bridge. Boring BB-WDB-202 was drilled south of the existing bridge. The test boring locations are shown on Sheet 2 – Boring Location Plan. An interpretive subsurface profile across the site is shown on Sheet 3 – Interpretive Subsurface Profile.

The 100-series test borings were drilled in February 2017 and the 200-series test borings in April 2017. 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.

Borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rig that performed the 100-series borings was equipped with a rope and cathead hammer system. The ratio of the available potential energy of the hammer to the energy transmitted by the hammer to the sampler is commonly accepted as 60 percent for a rope and cathead hammer system and is designated as N_{60} . The drill rig that performed the 200-series borings was equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” prior to the 200-series borings in March 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.6 for the 100-series borings and 0.75 for the 200-series borings to the raw field N-values. The hammer efficiency factors (0.6 and 0.75) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core calculated. The MaineDOT geotechnical engineer selected the boring locations, drilling methods, logged the subsurface conditions encountered, designated type and depth of sampling techniques, reviewed boring logs, and identified field testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of; three standard grain size analyses with natural water content and three grain size analyses with hydrometer and natural moisture content. The results of soil tests are included as Appendix C – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of granular fill and glacial till overlying metasedimentary bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Fill

Encountered in the borings was a layer of fill material. The thickness of the fill unit was approximately 9.7 to 16.7 feet at the boring locations. The fill encountered generally consisted of:

- Brown to grey, dry to wet, sand, some to little gravel, little to trace silt, trace clay, trace wood;
- Grey-brown, mottled, dry, sandy silt, little gravel, trace wood;
- Brown to grey, dry to moist, gravel, some to trace silt, trace sand;
- Voids;
- Cobbles; and
- Boulders.

The solid stem auger encountered refusal in BB-WDB-101 at 5.25 feet below the ground surface (bgs) on what is likely a cobble or boulder.

Corrected SPT N-values in the fill unit ranged from 4 to greater than 50 blows per foot (bpf) indicating the fill is very loose to very dense in consistency. Three grain size analyses conducted on samples of the fill material resulted in soil classifications of A-1-b, A-2-4, and A-4 under the AASHTO Soil Classification System and SC-SM and SM under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 10 to 52 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill materials in the borings BB-WDB-101A, BB-WDB-201, and BB-WDB-202. The thickness of the glacial till encountered ranged from approximately 9.0 to 18.1 feet. The glacial till generally consisted of:

- Grey and brown, dry, silt, some gravel, little sand;
- Grey, wet, sand, some gravel, some silt, trace clay;
- Grey, wet, gravelly sand, little silt,
- Grey, wet, sandy gravel, little silt;
- Grey, wet, gravel, little to trace silt;
- Cobbles; and
- Boulders.

Corrected SPT N-values of the glacial till deposit were all greater than 50 bpf indicating the glacial till is very dense in consistency. Three grain size analyses of the glacial till resulted in the soil being classified as A-1-a and A-1-b under the AASHTO Soil Classification System and SC-SM, SM and GW-GM under the USCS. The natural water contents of the samples tested ranged from approximately 9 to 12 percent.

5.3 Bedrock

Bedrock was encountered and cored in borings BB-WDB-101A, BB-WDB-201, and BB-WDB-202. Table 1 summarizes the approximate initial bedrock core depths, elevations, and RQD's. Observed drilling behavior indicates that approximately 2.8 feet of weathered bedrock was encountered in BB-WDB-101A and 5.0 feet of weathered bedrock was encountered in BB-WDB-201 above the bedrock core elevations listed in Table 1.

Boring	Station	Offset (feet)	Approximate Depth to Initial Bedrock Core (feet)	Approximate Initial Core Elevation (feet)	RQD (%)
BB-WDB-101A	8+99.6	11.0 Lt	22.3	360.9	0
BB-WDB-201	9+31.6	7.3 Rt	25.3	358.5	47
BB-WDB-202	9+93.4	1.6 Lt	35.1	348.2	0

Table 1 – Summary of Approximate Initial Bedrock Core Depth, Elevation, and RQD

The bedrock recovered from the borings is generally identified as grey, fine grained, slate, hard, fresh to slightly weathered, steep to vertical foliation, joints are very close to moderately close and open. The RQD of the bedrock cores ranged from 0 to 47 percent correlating to a rock mass quality of very poor to poor. Detailed bedrock descriptions and the RQD of each core run are provided on the boring logs in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. Photographs of rock cores are provided in Appendix B – Rock Core Photographs.

5.4 Groundwater

One groundwater measurement performed at the completion of the borings indicated a groundwater level of 8.6 bgs. Groundwater observations are provided on the boring logs in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. Note that because water was introduced into the borehole during drilling operations, the groundwater measurement may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The June 2018 Preliminary Design Report (PDR) identified pile-supported integral abutments as the most cost effective and preferred substructure type because jointless integral substructure/superstructure connections will allow for faster construction, lower maintenance costs, and a longer service life.

The presence of shallow sloping bedrock and numerous cobbles and boulders were considered during preliminary design. Achieving pile fixity at the pile tip is a concern when shallow bedrock is encountered. Sloping bedrock could cause piles to shift out of position during pile driving. Cobbles and boulders can obstruct pile driving. Predrilling through obstructions and into bedrock to create a sufficient pile length to achieve fixity then backfilling the excavation before pile driving to provide lateral support of the pile was identified as the solution to all the various challenges presented at the site. A special provision for predrilling holes for piles is included as Appendix E – Draft Special Provision.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral bridge abutments, which are the proposed substructures for the Goodell Bridge replacement project.

7.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of driven H-piles. The piles shall be end bearing on or within bedrock and driven to the required resistance. Piles may be HP 14x89 or 14x117 depending on the factored design axial loads and ability to resist lateral loads. H-piles shall be 50 ksi, Grade A572 steel. Abutment No. 1 and Abutment No. 2 piles require driving pile points conforming to MaineDOT Standard Specification 711.10 to protect pile tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on Table 2:

Location	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation After Predrilling (feet)	Estimated Pile Lengths (feet)
Abutment No. 1	374.0	349.0	25.0
Abutment No. 2	373.5	348.2	25.3

Table 2 – Estimated Pile Lengths for Integral Abutments

The estimated pile lengths in Table 2 do not take into account locations where bedrock predrilling is deeper or shallower than that specified on the plans, damaged pile, the additional five feet of pile required for dynamic testing instrumentation (per ASTM D4945), additional pile length needed to accommodate leads and driving equipment, or additional pile length needed for embedment in the abutment or pile cap.

7.1.1 Strength Limit State Design

The design of pile foundations bearing on bedrock at the strength limit state shall consider;

- compressive axial geotechnical resistance of individual piles bearing on bedrock,
- drivability resistance of individual piles driven to bedrock,
- structural resistance of individual piles in axial compression, and
- structural resistance of individual piles in combined axial loading and flexure.

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Per AASHTO LRFD Bridge Design Specifications 8th Edition with interim revisions through 2018 (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$ (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.70$ and the flexural resistance factor $\phi_f = 1.0$ shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2).

Abutment H-piles were analyzed for determination of unbraced lengths and fixity using LPILE 2016 (LPILE) software. The calculated unbraced lengths should be used to analyze the piles in combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. Supporting calculations are provided in Appendix D – Calculations.

Structural Resistance. The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the structural axial resistance of two H-pile sections were calculated for upper and lower unbraced pile segments, and for the lower braced pile segment. The controlling resistance shown in Table 3 is for the lower braced pile segment, using a resistance factor, $\phi_c = 0.50$ for severe driving conditions. The factored structural resistances for the approximated upper unbraced segments use an axial resistance factor $\phi_c = 0.70$ for combined axial and flexure are not provided in Table 3 because these did not govern. Supporting calculations are provided in Appendix D – Calculations. The unbraced pile lengths and effective length factors (K) assumed in these evaluations have been determined from LPILE analyses for a 14x117 pile section, which are also provided in Appendix D – Calculations. It is the responsibility of the

structural engineer to calculate the nominal axial structural compressive resistance (P_n) based on unbraced lengths (ℓ) and effective length factors (K) determined from LPile.

Geotechnical Resistance. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c , of 0.50, for severe driving conditions applied. The resulting limiting factored geotechnical compressive resistances for piles driven to rock are provided in Table 3.

Drivability Analyses. Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The drivability resistances were calculated using the resistance factor, ϕ_{dyn} , of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

A summary of the calculated factored axial compressive structural, geotechnical, and drivability resistances of two H-pile sections for the strength limit states are provided in Table 3. Supporting calculations are provided in Appendix D – Calculations.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance ¹ $\phi_c = 0.50$ (kips)	Controlling Geotechnical Resistance ² $\phi = 0.50$ (kips)	Drivability Resistance ³ $\phi_{dyn} = 0.65$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	652	582	436	436
HP 14 x 117	860	770	536	536

Table 3 – Factored Axial Compressive Resistances for H-Piles at Strength Limit States

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial geotechnical and structural resistance per LRFD Article 10.7.3.2.3. Therefore,

¹ Structural resistances were calculated for approximated upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for a braced segment in pure compression using a resistance factor, $\phi_c=0.50$, for severe driving conditions. The factored structural resistances for the upper segments use a resistance factor of $\phi_c=0.70$ for combined axial loading and bending are not shown here, but are provided in Appendix C – Calculations.

² Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*

³ Uses a resistance factor, $\phi_{dyn} = 0.65$, assuming the driving criteria is established by dynamic testing, and quality control by dynamic testing of at least two piles per site condition and no less than two percent of production piles.

drivability controls and the recommended governing resistances for pile design are the drivability resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 3. The maximum applied factored axial pile load for the strength limit states should not exceed the governing factored pile resistance shown in Table 3 above.

7.1.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles and pile group movements/stability considering changes in soil conditions due to scour due to the design flood (Q_{100}). For the service limit state, resistance factors of $\phi = 1.0$ should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, ϕ , of 0.65.

Extreme limit state design checks for the H-piles shall include pile axial compressive resistance, overall global stability of the pile group, pile failure by uplift in tension, and structural failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood (Q_{500}) can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as $\phi = 1.0$ with the exception of uplift of piles, for which the resistance factor, ϕ_{up} , shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The nominal axial geotechnical pile resistance at the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. The calculated factored axial structural, geotechnical, and drivability resistances of two H-pile sections for the extreme and service limit states are provided in Table 4. Supporting documentation is provided in Appendix D – Calculations.

Pile Section	Extreme and Service Limit State Factored Axial Pile Resistance			
	Structural Resistance ⁴ $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance ⁵ $\phi = 1.0$ (kips)	Drivability Resistance $\phi = 1.0$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	1165	1165	670	670
HP 14 x 117	1541	1541	825	825

Table 4 – Factored Axial Compressive Resistances for H-Piles at Service and Extreme Limit States

⁴ Normal conditions consider no soil loss due to scour. Nominal compressive resistances were calculated for upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for the lower unbraced pile segment, using a resistance factor, $\phi = 1.0$.

⁵ Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*.

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial geotechnical resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistances. Therefore, drivability controls, and the recommended governing resistances for pile design are the resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 4. The maximum applied factored axial pile load for the extreme and service limit states should not exceed the governing factored pile resistance shown in Table 4 above.

7.1.3 Lateral Pile Resistance/Behavior

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.12. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

A lateral pile resistance analysis was performed to evaluate pile behavior at both abutments using LPILE with pile head deflections, moments, and axial loads supplied by the structural engineer. The results of the LPILE analysis for a 14x117 section (the pile section selected for final design) was provided to the structural designer to evaluate the associated pile stresses, bending moments, and fixity due to factored pile head loads and displacements. The results of the LPILE analysis for the pile section selected for final design are provided in Appendix D – Calculations.

Geotechnical parameters used for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Table 5. In general, the model developed emulates the backfill material placed after predrilling holes for piles by using the soil layers (referenced in Table 5 by elevations) and using appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight (lbs/ft ³)	k _s (lb/in ³)	Internal Angle of Friction
Medium dense, SAND (Type C - Underdrain).	383.5 – 374.0	Above	125	90	32°
Medium dense, SAND (Type C - Underdrain).	374.0 – 349.0	Below	80	60	32°

Table 5 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves

7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching at each abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. There is a likelihood of pile ‘relaxation’ and minimum 24 hour restrrike pile tests will be required. Additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should pile behavior indicate a pile is refusing on a boulder or in a cobble layer above bedrock, should the pile tip be not firmly embedded in bedrock, or if the pile “walk” out of position.

With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, ϕ_{dyn} , of 0.65. The maximum factored axial 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. 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 pile resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving may be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral 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. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design. Strength limit state design shall also consider changes in foundation conditions and foundation resistance after scour due to the design (Q_{100}) flood.

A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65.

Extreme limit state design of integral abutments supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining

after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor of 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: angle of internal friction (ϕ) of 32 degrees, total unit weight (γ) of 125 pcf, and a soil-concrete interface friction angle (δ) of 20 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, K_p , of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. 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 coefficient of 3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to 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 may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 6:

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

Table 6 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

Backfill within 10 feet of the abutments and side slope fill shall conform to MaineDOT Specification 703.19 – Granular Borrow for Underwater Backfill. The gradation of this material specifies 7 percent or less of the material passing the No. 200 sieve. Limiting the amount of fines is intended to minimize frost action behind the structure.

Slopes in front of the pile supported integral abutments should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V in accordance with MaineDOT Standard Detail 610(03).

7.3 In-line Wingwalls

In-line, cantilevered “butterfly” wingwalls may be used in conjunction with the integral abutments. The wingwalls shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, and collision loads, as well as forces from, creep, temperature, and shrinkage deformations. The design of “butterfly” wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment. These additional bending stresses may require wingwalls longer than 10 feet to be independently supported.

The design of in-line “butterfly” wingwalls shall be designed for passive earth pressure. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p , of 3.25 assuming small wingwall movements. See Appendix D – Calculations for supporting documentation. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Article 3.11.6.4. An at-rest earth pressure coefficient, K_o , of 0.47 should be used for live load surcharge loads placed upon wingwalls cantilevered off of abutments with the top of the wall restrained from movement. See Appendix D – Calculations for supporting documentation.

7.4 Settlement

The approximately 10 to 17-foot thick fill unit encountered in the test borings is very loose to very dense in consistency. These coarse-grained materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The project calls for the vertical alignment of the new structure to increase approximately 1 foot. Any settlement resulting from this increase in overburden pressure is to be small and occur relatively quickly. Construction loads could introduce elastic settlements and these settlements are also anticipated to be small and occur relatively quickly. Post construction settlement should be negligible with proper compaction of any replaced fill materials used during construction.

Any settlement of the bridge abutments will be due to axial compression of the foundation piles and is anticipated to be minimal.

7.5 Frost Protection

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

Foundations placed on soil should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Willimantic has a design freezing index (DFI) of approximately 2200 F-degree days. The

anticipated coarse-grained fill material was assigned a water content of 10%. These components correlate to a frost depth of 8.3 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Houlton, Maine has an air DFI from the Modberg database of approximately 2189 F-degree days. Houlton was selected for the Modberg analysis because it lies along the same isoline as Willimantic and Willimantic is not available in the Modberg database. A water content of 10% was used. These components correlate to a frost depth of approximately 8.0 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 8.3 feet for frost protection. See Appendix D – Calculations for supporting calculations.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.6 Scour and Riprap

Grain size analyses were performed on samples of the glacial till deposit to generate grain size curves for determining parameters to be used in scour analyses. Sample BB-WDB-201;3D was 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, $D_{50} = 1.18$ mm (medium sand).
- Average diameter of particle at 95 percent passing, $D_{95} = 15.88$ mm (fine gravel).

The grain size curves are included in Appendix C – Laboratory Test Results.

The consequences of changes in foundation conditions resulting from the design (Q_{100}) and check (Q_{500}) 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 the check flood (Q_{500}) event is no less than the extreme limit state loads. At the service limit state, the design shall limit movements and ensure overall stability considering scour at the design load.

For scour protection of the pile supported abutments, the PDR indicates the bridge approach slopes and the abutment slopes will be armored with riprap. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding scour design. Typically, the top of the riprap is located at, or above, the Q_{50} elevation.

Plain riprap shall conform to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The toe of the riprap section shall be constructed at least 1 foot below the streambed elevation. The riprap section shall be underlain by a 1-foot thick layer of bedding material conforming MaineDOT Standard Specification 703.19 and Class 1 nonwoven erosion control geotextile per MaineDOT Standard Details 610(02) and 610(03).

7.7 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the LRFD Manual, and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provided the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. These results are summarized in Table 7:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.073g
Acceleration Coefficient (A_s)	0.087g
S_{DS} (Period = 0.2 sec)	0.19g
S_{D1} (Period = 1.0 sec)	0.08g
Site Class	C
Seismic Zone	1

Table 7 – Seismic Design Parameters

In conformance with LRFD Article 4.7.4 seismic analysis is not required for bridges in Seismic Zone 1 or single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

See Appendix D – Calculations for supporting documentation.

7.8 Construction Considerations

The new integral abutments will be constructed behind the existing abutments, which avoids the placement of fills in the river and minimizes the size of cofferdams placed in the river. Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments. The stacked stone abutments and obstructions encountered in the fill and glacial till will impede installation of temporary earth support systems if the obstructions are not removed.

Construction of the proposed structure will require predrilling holes for piles and pile driving. The contractor should assume that the existing abutments, if not removed entirely, will obstruct predrilling and pile driving operations. The contractor shall be responsible for excavating those portions of existing structure that conflict with predrilling and pile driving by conventional excavation methods.

Shallow bedrock, numerous cobbles and boulders were encountered at the project site. The contractor shall predrill through the bedrock, cobbles, boulders, or any other obstruction to the elevation shown on the plans. The Contractor should assume conventional excavation methods, rock chisels, or down-hole hammers are necessary to clear obstructions and allow predrilling activities. After predrilled holes are backfilled, care should be taken to drive H-piles within allowable tolerances without damaging the H-piles.

Excavations for the proposed abutments will expose soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

8.0 CLOSURE

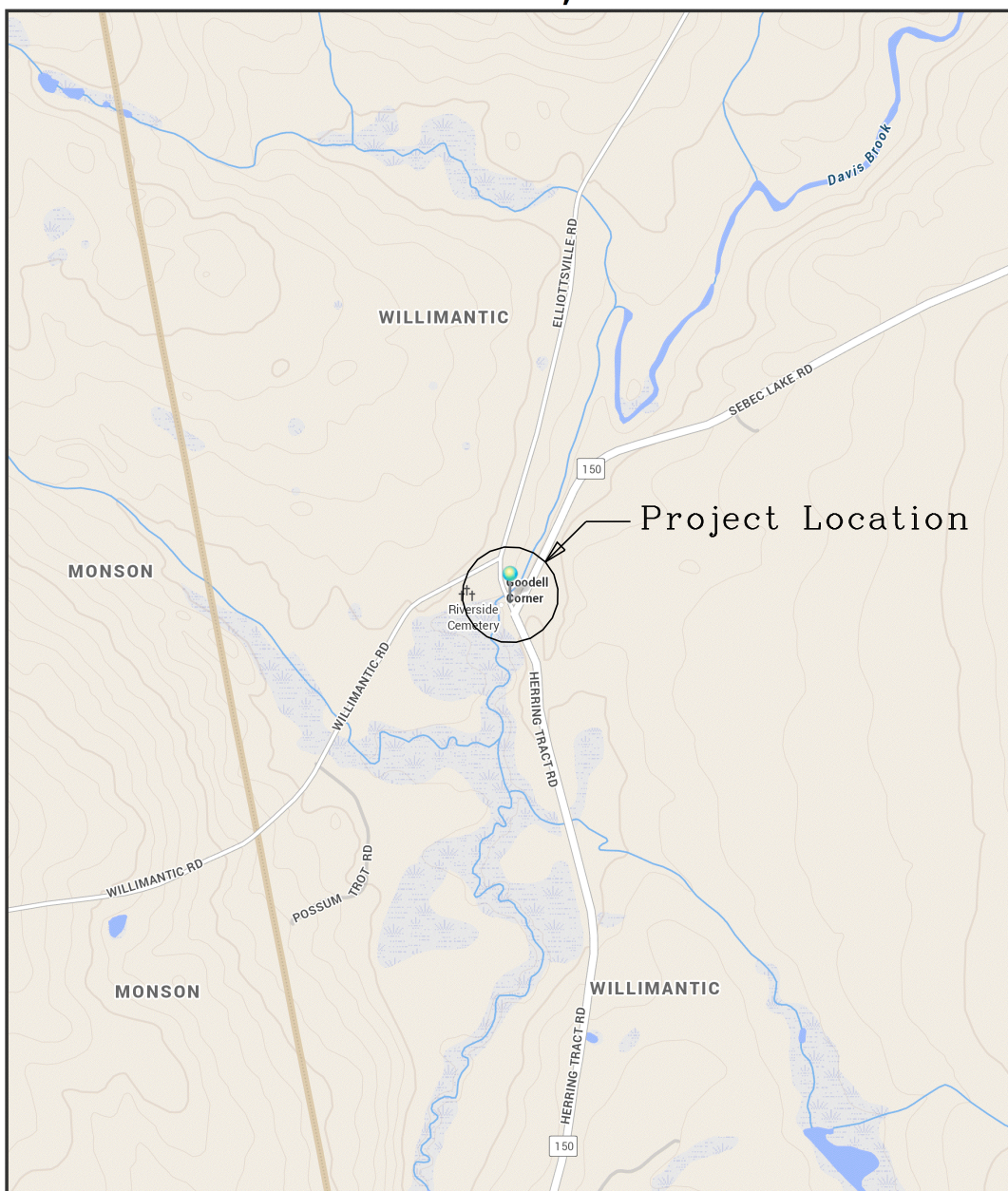
This report has been prepared for the use of the MaineDOT Bridge Program for specific application of the proposed replacement of Goodell Bridge in Willimantic, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed 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. These analyses and recommendations are based in part upon limited subsurface investigations at discrete exploratory 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.

It is recommended that a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report may be properly interpreted and implemented in the design and specifications.

Sheets

WILLIMANTIC, MAINE

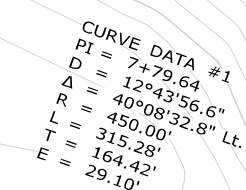


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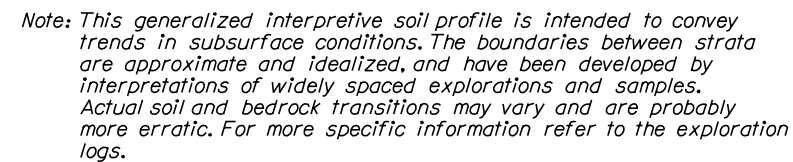
0.25 Miles
1 inch = 0.28 miles

Date: 1/24/2018
Time: 8:25:52 AM

SHEET NUMBER 1 OF 4	GOODELL BRIDGE DAVIS BROOK WILLIMANTIC PISCATAQUIS COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
		STP-2262(400)
	LOCATION MAP	WIN 22624.00 BRIDGE NO. 2317 BRIDGE PLANS



 WILLIMANTIC PISCATAQUIS COUNTY	GOODELL BRIDGE DAVIS BROOK				PROJ. MANAGER	M WIGHT	BY	DATE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
					DESIGN-DETAILED	T HEILM	W ROHMAN		
					CHECKED-REVIEWED	D SLAVEN	T WHITE	NOV 2008	SIGNATURE
					DESIGNS-DETAILED	D3			P.E. NUMBER
					REVISONS	1			
					REVISONS	2			
				REVISONS	3				WIN 22624.00
				REVISONS	4				
				FIELD CHANGES					BRIDGE NO. 2317
									BRIDGE PLANS



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-WDB-101A
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Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at 15' 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-WDB-101
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Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at 15 ft 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-WDB-201
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* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions either than those present at the time measurements were made.

GODELL BRIDGE DAVIS BROOK IMANTIC PISCATAQUIS COUNTY	BORING LOGS
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Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER 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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Descriptive Term		Portion of Total (%)		
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace	0 - 10			
					little	11 - 20			
					some	21 - 35			
					adjective (e.g. sandy, clayey)	36 - 50			
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	TERMS DESCRIBING DENSITY/CONSISTENCY				
		GC	Clayey gravels, gravel-sand-clay mixtures.						
		CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	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. Density is rated according to standard penetration resistance (N-value).				
		(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	Density of Cohesionless Soils				
SC	Clayey sands, sand-clay mixtures.		Standard Penetration Resistance N-Value (blows per foot)						
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.	Very loose	
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Loose					
		OL	Organic silts and organic silty clays of low plasticity.					Medium Dense	
		SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Dense				
			CH	Inorganic clays of high plasticity, fat clays.					Very Dense
	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, if applicable):					Desired Rock Observations (in this order, if applicable):			
	Color (Munsell color chart)					Color (Munsell color chart)			
	Moisture (dry, damp, moist, wet)					Texture (aphanitic, fine-grained, etc.)			
Density/Consistency (from above right hand side)					Rock Type (granite, schist, sandstone, etc.)				
Texture (fine, medium, coarse, etc.)					Hardness (very hard, hard, mod. hard, etc.)				
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)					Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)				
Gradation (well-graded, poorly-graded, uniform, etc.)					Geologic discontinuities/jointing:				
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)					-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)				
Structure (layering, fractures, cracks, etc.)					-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)				
Bonding (well, moderately, loosely, etc.,)					-tightness (tight, open, or healed)				
Cementation (weak, moderate, or strong)					-infilling (grain size, color, etc.)				
Geologic Origin (till, marine clay, alluvium, etc.)					Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)				
Groundwater level					RQD and correlation to rock mass quality (very poor, poor, etc.)				
					ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A				
					Recovery (inch/inch and percentage)				
					Rock Core Rate (X.X ft - Y.Y ft (min:sec))				
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information					Sample Container Labeling Requirements:				
					WIN		Blow Counts		
					Bridge Name / Town		Sample Recovery		
					Boring Number		Date		
					Sample Number		Personnel Initials		
					Sample Depth				

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goodell Bridge #2317 carries Ellitsville Road over Davis Brook Location: Willimantic, Maine				Boring No.: BB-WDB-101A WIN: 22624.00	
Driller: NEBC				Elevation (ft.): 383.2				Auger ID/OD: 4" OD SSA	
Operator: Enos				Datum: NAVD88				Sampler: Standard Split Spoon	
Logged By: B. Slaven				Rig Type: B-53 Track Mounted				Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 2/21/2017-2/22/2017				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"	
Boring Location: 8+99.6, 11.0 ft Lt.				Casing ID/OD: NW-3"				Water Level*: None Observed	
Hammer Efficiency Factor: 0.6				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 = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{u(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane 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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0							SSA					
5	1D	24/3	5.00 - 7.00	1/2/2/3	4	4					Grey, wet, very loose, SAND, some gravel, little silt, trace clay, (Fill).	G#271094 A-1-b, SC-SM WC=9.9%
10	2D	24/17	10.00 - 12.00	14/32/31/36	63	63	77		372.7		Grey and brown, dry, hard, SILT, some gravel, little sand, (Glacial Till).	
15	3D	9.6/5	15.00 - 15.80	85/50(3.6")	---		OPEN HOLE				Grey, wet, very dense, SAND, some gravel, some silt, trace clay.	G#271095 A-1-b, SC-SM WC=10.3%
20	MD	0/0	19.50 - 19.50	50(0")	---		RC		363.7		Failed sample attempt.	
											Top of Weathered Bedrock at Elev. 363.7 ft. Roller Coned ahead to 22.3 ft bgs.	
	R1	6/6	22.30 - 22.80	RQD = 0%			NQ-2		360.9		Top of Bedrock at Elev. 360.9 ft.	
	R2	60/58	22.80 - 27.80	RQD = 33%							R1: Bedrock: Grey, fine grained, SLATE, hard, fresh, breaks along steep to vertical foliation, joints are low angle, close, open. [SOLO FORMATION] Rock Mass Quality = Very Poor.	
25												

Remarks:

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

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

Page 1 of 2

Boring No.: BB-WDB-101A

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goodell Bridge #2317 carries Elliottsville Road over Davis Brook Location: Willimantic, Maine		Boring No.: BB-WDB-101A WIN: 22624.00					
Driller: NEBC		Elevation (ft.): 383.2		Auger ID/OD: 4" OD SSA							
Operator: Enos		Datum: NAVD88		Sampler: Standard Split Spoon							
Logged By: B. Slaven		Rig Type: B-53 Track Mounted		Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 2/21/2017-2/22/2017		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"							
Boring Location: 8+99.6, 11.0 ft Lt.		Casing ID/OD: NW-3"		Water Level*: None Observed							
Hammer Efficiency Factor: 0.6		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 = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane 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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25										R1: Core Times (min:sec) 22.3-22.8 ft (1:49) 100% Recovery Core Blocked R2: Bedrock: Similar to R1 except, joints are moderately close. Rock Mass Quality = Poor. R2: Core Times (min:sec) 22.8-23.8 ft (1:04) 23.8-24.8 ft (1:04) 24.8-25.8 ft (1:21) 25.8-26.8 ft (1:27) 26.8-27.8 ft (1:10) 97% Recovery Core Blocked Bottom of Exploration at 27.8 feet below ground surface.	
50											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2	
* 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-WDB-101A	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goodell Bridge #2317 carries Ellitsville Road over Davis Brook Location: Willimantic, Maine				Boring No.: BB-WDB-201 WIN: 22624.00			
Driller: NEBC				Elevation (ft.): 383.2				Auger ID/OD: 4" OD SSA			
Operator: Porter/Smith				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: B. Slaven				Rig Type: B-53 Track Mounted				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 4/20/2017-4/20/2017				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: 9+31.6, 7.3 ft Rt.				Casing ID/OD: NW-3" & HW-4"				Water Level*: None Observed			
Hammer Efficiency Factor: 0.75				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 = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane 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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							SSA	382.9		4" HMA.	G#270163 A-4, SM WC=15.4%
	1D	24/8	2.00 - 4.00	4/5/4/12	9	11				Grey-brown, mottled, dry, stiff, Sandy SILT, little gravel, trace wood.	
5	2D	24/5	5.00 - 7.00	6/9/14/17	23	29	34			Brown, moist, dense, GRAVEL, some silt, trace sand.	
							58				
							88			Voids	
							44				
							RC			Cobble 9.5-10 ft bgs. Wash water loss out of abutment face.	
10	MD	3/0	10.00 - 10.25	50(2.4")	---			373.2		Failed sample attempt. Advanced by Roller Cone to 15.0 ft. bgs. Cobbles from 10.0 - 11.0.	
										Cobbles from 11.5-13.0 ft bgs.	
15	3D	24/9	15.00 - 17.00	16/27/26/27	53	66	97		Grey, wet, very dense, Gravelly SAND, little silt. Drove NW Casing to 15 ft bgs. Pulled HW Casing.	G#270164 A-1-b, SM WC=12.1%	
							105				
							200+		Advance by Roller Cone to 20.0 ft bgs.		
							228				
							199				
20	4D	11/7	20.00 - 20.92	35/50(5")	---		RC	363.2	Grey, wet, very dense, platy GRAVEL, little silt, (Weathered Bedrock and Till seams). Roller Coned ahead to 25.0 ft bgs.		
									Drove NW Casing to Refusal at 22.4 ft bgs.		
25											
Remarks: Autohammer #NEBC1											
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 2 Boring No.: BB-WDB-201	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goodell Bridge #2317 carries Elliottsville Road over Davis Brook Location: Willimantic, Maine		Boring No.: BB-WDB-201 WIN: 22624.00					
Driller: NEBC		Elevation (ft.): 383.2		Auger ID/OD: 4" OD SSA							
Operator: Porter/Smith		Datum: NAVD88		Sampler: Standard Split Spoon							
Logged By: B. Slaven		Rig Type: B-53 Track Mounted		Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 4/20/2017-4/20/2017		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"							
Boring Location: 9+31.6, 7.3 ft Rt.		Casing ID/OD: NW-3" & HW-4"		Water Level*: None Observed							
Hammer Efficiency Factor: 0.75		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 = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt		R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person		S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected							
		T _v = Pocket Torvane 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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25	5D/R1	36/27	25.00 - 28.00	50(3") RQD = 47%	---		NQ-2	358.2		-25.0'	
	R2	36/28	28.00 - 31.00	RQD = 22%							
30											
	R3	18/17	31.00 - 32.50	RQD = 0%							
	R4	20.4/14	32.50 - 34.20	RQD = 0%							
35											
40											
45											
50											
Remarks: Autohammer #NEBC1											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2	
* 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-WDB-201	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goodell Bridge #2317 carries Elliottsville Road over Davis Brook Location: Willimantic, Maine				Boring No.: BB-WDB-202 WIN: 22624.00																																																																																																																																																																																																																																																																																						
Driller: NEBC		Elevation (ft.): 383.3		Auger ID/OD: 4" OD SSA																																																																																																																																																																																																																																																																																										
Operator: Porter/Smith		Datum: NAVD88		Sampler: Standard Split Spoon																																																																																																																																																																																																																																																																																										
Logged By: B. Slaven		Rig Type: B-53 Track Mounted		Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																																																																																																										
Date Start/Finish: 4/18/2017-4/19/2014		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"																																																																																																																																																																																																																																																																																										
Boring Location: 9+93.4, 1.6 ft Lt.		Casing ID/OD: NW-3", HW-4"		Water Level*: 8.6 ft bgs.																																																																																																																																																																																																																																																																																										
Hammer Efficiency Factor: 0.75		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																																																																												
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<table><tr><th colspan="8">Sample Information</th><th rowspan="2">Elevation (ft.)</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Depth (ft.)</th><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th></tr><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>SSA</td><td>383.0</td><td rowspan="19"></td><td>4" HMA.</td><td rowspan="19">G#270161 A-2-4, SC-SM WC=52.4%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>5</td><td>1D</td><td>24/13</td><td>5.00 - 7.00</td><td>9/17/13/23</td><td>30</td><td>38</td><td>16</td><td></td><td></td><td>Grey, dry, dense, platy GRAVEL, trace sand, trace silt, (Fill).</td><td rowspan="19">G#270162 A-1-a, GW-GM WC=8.8%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>46</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>50</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>67</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>44</td><td></td><td></td><td></td></tr><tr><td>10</td><td>2D</td><td>19.2/6</td><td>10.00 - 11.60</td><td>1/15/23/50(1.2")</td><td>38</td><td>48</td><td>19</td><td></td><td></td><td>Grey, wet, dense, GRAVEL, trace sand, trace silt.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>63</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>134</td><td></td><td></td><td>Advance by Roller Cone to 13.1 ft bgs Cobbles and Boulders from 11.6-13.0 ft bgs.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>67</td><td></td><td></td><td></td></tr><tr><td>15</td><td>3D</td><td>24/7</td><td>14.00 - 16.00</td><td>9/3/2/13</td><td>5</td><td>6</td><td></td><td></td><td></td><td>Grey, wet, loose, SAND, little silt, little gravel, trace clay, trace decomposed wood fiber.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>156</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>172</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>198 RC</td><td></td><td></td><td>Wood from 17.0-17.5 ft bgs. 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R1: Core Times (min:sec) 20.4-21.4 ft (2:46) 21.4-22.4 ft (1:54) 22.4-23.4 ft (3:19) 23.4-24.4 ft (2:57) 24.4-25.4 ft (2:52) 35% Recovery</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>25</td><td>5D</td><td>23/8</td><td>24.90 - 26.82</td><td>2/20/44/50(5")</td><td>64</td><td>80</td><td></td><td></td><td></td><td></td></tr></table>												Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	0							SSA	383.0		4" HMA.	G#270161 A-2-4, SC-SM WC=52.4%																																													5	1D	24/13	5.00 - 7.00	9/17/13/23	30	38	16			Grey, dry, dense, platy GRAVEL, trace sand, trace silt, (Fill).	G#270162 A-1-a, GW-GM WC=8.8%								46											50											67											44				10	2D	19.2/6	10.00 - 11.60	1/15/23/50(1.2")	38	48	19			Grey, wet, dense, GRAVEL, trace sand, trace silt.								63											134			Advance by Roller Cone to 13.1 ft bgs Cobbles and Boulders from 11.6-13.0 ft bgs.								67				15	3D	24/7	14.00 - 16.00	9/3/2/13	5	6				Grey, wet, loose, SAND, little silt, little gravel, trace clay, trace decomposed wood fiber.								156											172											198 RC			Wood from 17.0-17.5 ft bgs. Advance by Roller Cone to 19.0 ft bgs.												20	4D	17/8.5	19.00 - 20.42	20/36/50(5")	---					Grey, wet, very dense, Sandy platy GRAVEL, little silt, (Glacial Till).		R1	60/21	20.40 - 25.40				NQ-2			Changed to NW Casing at 20.0 ft bgs. R1: Cobbles and Boulders. R1: Core Times (min:sec) 20.4-21.4 ft (2:46) 21.4-22.4 ft (1:54) 22.4-23.4 ft (3:19) 23.4-24.4 ft (2:57) 24.4-25.4 ft (2:52) 35% Recovery												25	5D	23/8	24.90 - 26.82	2/20/44/50(5")	64	80				
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goodell Bridge #2317 carries Elliottsville Road over Davis Brook Location: Willimantic, Maine		Boring No.: BB-WDB-202 WIN: 22624.00					
Driller: NEBC			Elevation (ft.): 383.3		Auger ID/OD: 4" OD SSA						
Operator: Porter/Smith			Datum: NAVD88		Sampler: Standard Split Spoon						
Logged By: B. Slaven			Rig Type: B-53 Track Mounted		Hammer Wt./Fall: 140#/30"						
Date Start/Finish: 4/18/2017-4/19/2014			Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"						
Boring Location: 9+93.4, 1.6 ft Lt.			Casing ID/OD: NW-3", HW-4"		Water Level*: 8.6 ft bgs.						
Hammer Efficiency Factor: 0.75			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 = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_u(lab) = Lab Vane Undrained Shear Strength (psf) q_u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_v = Pocket Torvane 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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25							RC			Grey, wet, very dense, platy GRAVEL, little silt. Advance by Roller Cone to 30.5 ft bgs.	
30	6D	2.4/5	30.50 - 30.70	50(2.4")	---					Grey, wet, very dense, GRAVEL, trace silt. Advance by Roller Cone to 35.1 ft bgs.	
	MD	0/0	31.00 - 31.00	50(0")	---						
35											
	R2	60/27	35.10 - 40.10	RQD = 0%			NQ-2	348.2		Top of Bedrock at Elev. 348.2 ft. R2: Bedrock: Grey, fine grained, SLATE, hard, fresh, joints are low angle, very close, open. [SOLO FORMATION] Rock Mass Quality = Very Poor. R2: Core Times (min:sec) 35.1-36.1 ft (2:48) 36.1-37.1 ft (1:42) 37.1-38.1 ft (1:51) 38.1-39.1 ft (1:57) 39.1-40.1 ft (1:49) 45% Recovery	
40											
	R3	39.6/35	40.10 - 43.40	RQD = 0%						R3: Bedrock: Similar to R2 except vertical foliation. Rock Mass Quality = Very Poor. R3: Core Times (min:sec) 40.1-41.1 ft (1:13) 41.1-42.1 ft (1:16) 42.1-43.1 ft (1:48) 43.1-43.4 ft (0:46) 88% Recovery Core Blocked	
45											
	R4	60/42	43.40 - 48.40	RQD = 0%						R4: Bedrock: Similar to R2 except except grading to PHYLLITE. Rock Mass Quality = Very Poor. R4: Core Times (min:sec) 43.4-44.4 ft (1:54) 44.4-45.4 ft (1:58) 45.4-46.4 ft (1:15) 46.4-47.4 ft (2:19) 47.4-48.4 ft (3:02) 70% Recovery	
50								334.9		Bottom of Exploration at 48.4 feet below ground surface.	48.4
Remarks: Autohammer #NEBC1											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2 Boring No.: BB-WDB-202	

Appendix B

Rock Core Photographs



MaineDOT Goodell Bridge # 2317
Carries Elliottsville Road Over Davis Brook
Willimantic, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box/Box Row
BB-WDB-101A	R1	22.3 – 22.8	6	100	0	0	Slate	A/1
BB-WDB-101A	R2	22.8 – 27.8	58	97	19	33	Slate	A/1-2
BB-WDB-201	R1	25.3 – 28.0	27	75	17	47	Slate	B/1
BB-WDB-201	R2	28.0 – 31.0	28	72	8	22	Slate	B/1-2
BB-WDB-201	R3	31.0 – 32.5	17	94	0	0	Slate	B/2
BB-WDB-201	R4	32.5 – 34.2	14	69	0	0	Slate	B/2

Box A



Box B



- Notes:** 1. Box/Box Row corresponds to the core box section where the rock sample is contained. Row 1 = Top row. Row 4 = bottom.
2. Transition between core runs is marked by wood separator.



MaineDOT Goodell Bridge # 2317
Carries Elliottsville Road Over Davis Brook
Willimantic, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box/Box Row
BB-WDB-202	R2	35.1 – 40.1	27	45	0	0	Slate	C/1
BB-WDB-202	R3	40.1 – 43.4	35	88	0	0	Slate	C/1-2
BB-WDB-202	R4	43.4 – 48.4	42	70	0	0	Slate	C/2-3

Box C



- Notes:** 1. Box/Box Row corresponds to the core box section where the rock sample is contained. Row 1 = Top row. Row 4 = bottom.
2. Transition between core runs is marked by wood separator.

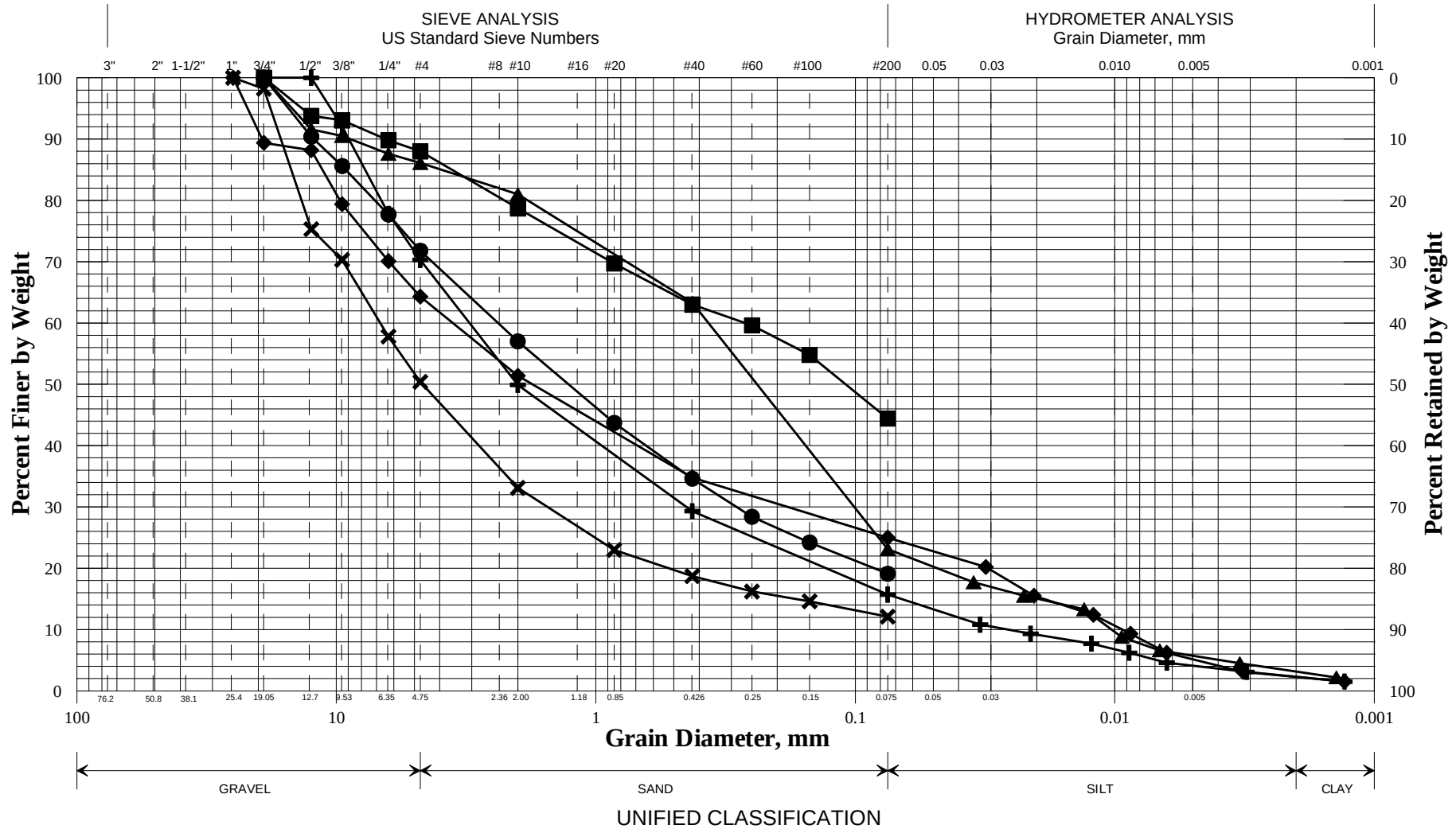
Appendix C

Laboratory Test Results

Work Number: 22624.00

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WDB-101A/1D	8+99.6	11.0 LT	5.0-7.0	SAND, some gravel, little silt, trace clay.	9.9			
◆	BB-WDB-101A/3D	8+99.6	11.0 LT	15.0-15.8	SAND, some gravel, some silt, trace clay.	10.3			
■	BB-WDB-201/1D	9+31.6	7.3 RT	2.0-4.0	Sandy SILT, little gravel.	15.4			
●	BB-WDB-201/3D	9+31.6	7.3 RT	15.0-17.0	Gravelly SAND, little silt.	12.1			
▲	BB-WDB-202/3D	9+93.4	1.6 LT	14.0-16.0	SAND, little silt, little gravel, trace clay.	52.4			
×	BB-WDB-202/4D	9+93.4	1.6 LT	19.0-20.4	Sandy GRAVEL, little silt.	8.8			

WIN
022624.00
Town
Willimantic
Reported by/Date
WHITE, TERRY A 5/16/2017

Appendix D

Calculations

H-Pile Resistance

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with 2018 Interim Revisions

Bedrock Properties

BB-WDB-201, R1 RQD = 47%

Rock Type: Slate, steep to vertical foliation.

$\phi = 20 - 27$ (AASHTO LRFD Table C.10.4.6.4-1);

$Co = 21,000 - 30,000$ psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

Pile Properties

Use the following piles: 14x89, 14x117

$$A_s := \begin{pmatrix} 26.1 \\ 34.4 \end{pmatrix} \cdot \text{in}^2 \quad d := \begin{pmatrix} 13.80 \\ 14.20 \end{pmatrix} \text{in} \quad b := \begin{pmatrix} 14.70 \\ 14.90 \end{pmatrix} \text{in}$$

Box Area

$$A_{\text{box}} := \overrightarrow{(d \cdot b)} \quad A_{\text{box}} = \begin{pmatrix} 202.9 \\ 211.6 \end{pmatrix} \cdot \text{in}^2 \quad \begin{matrix} \mathbf{14x89} \\ \mathbf{14x117} \end{matrix} \text{ Note: All matrices set up in this order}$$

Box Area Perimeter

$$A_{\text{boxPerimeter}} := d + d + b + b = \begin{pmatrix} 57 \\ 58.2 \end{pmatrix} \cdot \text{in}$$

$r_s =$ radius of gyration

$$r_s := \begin{pmatrix} 3.53 \\ 3.59 \end{pmatrix} \text{in}$$

radius of gyration about the Y-Y or weak axis per LRFD Article C6.9.4.1.2.

Pile yield strength

$$F_y := 50 \cdot \text{ksi}$$

1. Nominal and Factored Structural Compressive Resistance of H-piles

Use LRFD Equation 6.9.2.1-1 $Pr = \phi P_n$

Nominal Axial Structural Resistance

Determine equivalent yield resistance $P_o = QF_y A_s$ (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

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

$$P_o = \begin{pmatrix} 1305 \\ 1720 \end{pmatrix} \cdot \text{kip} \quad \begin{matrix} \mathbf{14x89} \\ \mathbf{14x117} \end{matrix}$$

A. Structural Resistance of upper "unbraced" segment of pile

Determine elastic critical buckling resistance P_e , LRFD eq. 6.9.4.1.2-1

E = Elastic Modulus

$$E := 29000 \cdot \text{ksi}$$

K = effective length factor

$$K_{\text{eff}} := 1.2$$

LRFD Table C4.6.2.5-1

(assume plastic hinge does not develop, K selected per Vtrans Design Procedure)

l = "unbraced" length

$$l_{\text{unbraced_Up}} := 3.3 \cdot \text{ft}$$

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \left(\frac{41223}{56194} \right) \cdot \text{kip}$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \left(\frac{31.588}{32.671} \right)$$

If $P_e/P_o > \text{or} = 0.44$, then:

$$P_{nUp} := \left(\frac{\frac{P_o}{P_e}}{0.658} \cdot P_o \right)$$

LRFD Eq.
6.9.4.1.1-1

then:

$$P_{nUp} = \left(\frac{1288}{1698} \right) \cdot \text{kip}$$

this applies to all pile sizes

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for upper unbraced segments in combined compression and flexure per LRFD 6.5.4.2:

$$\phi_{cu} := 0.7$$

The Factored Structural Resistance (P_r) per LRFD 6.9.2.1-1 is

$$P_r := \phi_{cu} \cdot P_{nUp}$$

Factored structural compressive resistance, P_r

this applies to all pile sizes

$$P_r = \left(\frac{901}{1189} \right) \cdot \text{kip}$$

B. Structural Resistance of lower "unbraced" segment of pile

Determine elastic critical buckling resistance P_e , LRFD eq. 6.9.4.1.2-1

E = Elastic Modulus

$$E := 29000 \cdot \text{ksi}$$

K = effective length factor

$$K_{\text{eff}} := 1.0$$

Per Vtrans Integral Abutment Design Guideline, use $K=1$ for 2nd unbraced segment that is pinned at the top and bottom. (see also LRFD Table C4.6.2.5-1)

l = "unbraced" length $l_{unbraced_Low} := 11.6 \cdot ft$

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \left(\frac{4804}{6549} \right) \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \left(\frac{3.681}{3.807} \right) \quad \text{If } P_e/P_o > \text{ or } = 0.44, \text{ then:}$$

$$P_{nLow} := \left(\frac{\frac{P_o}{P_e} \cdot P_o}{0.658} \right)$$

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

$$P_{nLow} = \left(\frac{1165}{1541} \right) \cdot kip$$

Extreme and
Service -
Structural

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for middle segment
of H-pile in combined compression and flexure:

$$\phi_{cu} := 0.7$$

The Factored Structural Resistance (P_r) per LRFD 6.9.2.1-1 is $P_r := \phi_{cu} \cdot P_{nLow}$

Factored structural compressive resistance, P_r

this applies to all pile sizes

$$P_r = \left(\frac{815}{1079} \right) \cdot kip$$

C. Structural Resistance of lower "braced" segment of pile

Determine elastic critical buckling resistance P_e , LRFD eq. 6.9.4.1.2-1

E = Elastic Modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 0.65$

LRFD Table C4.6.2.5-1. Use $K=0.65$ for segment in pure compression. Fixed top and bottom

l = "braced" length $l_{unbraced_bot} := .1 \cdot ft$

Assume only the very tip is in pure compression

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \left(\frac{2 \times 10^8}{2 \times 10^8} \right) \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 1.172 \times 10^5 \\ 1.213 \times 10^5 \end{pmatrix} \quad \text{If } P_e/P_o > \text{ or } = 0.44, \text{ then:} \quad P_{nBot} := \overrightarrow{\left(\frac{P_o}{0.658 \cdot P_e \cdot P_o} \right)} \quad \text{LRFD Eq. 6.9.4.1.1-1}$$

then:

this applies to all pile sizes

$$P_{nBot} = \begin{pmatrix} 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

Extreme and
Service -
Geotech**Factored Axial Structural Resistance for the Strength Limit State**

Resistance factor for H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary

$$\phi_c := 0.5$$

The Factored Structural Resistance (P_r) per LRFD 6.9.2.1-1 is

$$P_r := \phi_c \cdot P_{nBot}$$

Factored structural compressive resistance, P_r

$$P_r = \begin{pmatrix} 652 \\ 860 \end{pmatrix} \cdot \text{kip}$$

Strength -
Structural**LRFD 10.7.3.2.3 - Piles Driven to Hard Rock**

Article 10.7.3.2.3 states "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 nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile driving acceptance criteria shall be developed that will prevent pile damage."

Therefore limit the nominal axial geotechnical pile resistance to the nominal structural resistance with a resistance factor for severe driving conditions of 0.50 applied per 10.7.3.2.3.

Nominal Structural Resistance Previously Calculated:

$$P_{nLow} = \begin{pmatrix} 1165 \\ 1541 \end{pmatrix} \cdot \text{kip}$$

The factored geotechnical compressive resistance (P_r) for the **Strength Limit State**, per LRFD 6.9.2.1-1 is

$$\phi_c := 0.5$$

$$P_r := \phi_c \cdot P_{nLow}$$

$$P_r = \begin{pmatrix} 582 \\ 770 \end{pmatrix} \cdot \text{kip}$$

14x89
14x117Strength -
Geotech

The factored geotechnical compressive resistance (P_r) for the **Extreme and Service Limit States**, per LRFD 6.9.2.1-1 is

$$\phi_c := 1.0$$

$$P_{r_ee} := \phi_c \cdot P_{nLow}$$

$$P_{r_ee} = \begin{pmatrix} 1165 \\ 1541 \end{pmatrix} \cdot \text{kip}$$

14x89
14x117

Extreme and
Service -
Geotech

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechnical axial pile resistance for piles end bearing on rock determined by Intact Rock Method, proposed by Sanford, MaineDOT Transportation Research Division Technical Report 14-01, Phase 2 (January 2014), based on Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, (2006).

Nominal unit bearing resistance of pile point, Q_p

Design value of compressive strength of rock core

$$q_{u_1} := 21000 \cdot \text{psi}$$

Assumed

Geotechnical tip resistance.

$$q_{p_2} := 2.5 \cdot q_{u_1}$$

$$q_{p_2} = 7560 \cdot \text{ksf}$$

Nominal geotechnical tip resistance, R_p

$$R_p := \overrightarrow{(q_{p_2} \cdot A_s)}$$

$$R_p = \begin{pmatrix} 1370 \\ 1806 \end{pmatrix} \cdot \text{kip}$$

Factored Axial Geotechnical Compressive Resistance - Strength Limit States

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$$\phi_{stat} := 0.45 \quad \text{LRFD Table 10.5.5.2.3-1}$$

Factored Geotechnical Tip Resistance (R_r)

$$R_{r_p2} := \phi_{stat} \cdot R_p$$

$$R_{r_p2} = \begin{pmatrix} 617 \\ 813 \end{pmatrix} \cdot \text{kip}$$

Factored Axial Geotechnical Compressive Resistance - Service Limit States

Resistance factor,

$$\phi := 1.0$$

Factored Geotechnical Tip Resistance (R_r)

$$R_{r_p2} := \phi \cdot R_p$$

$$R_{r_p2} = \begin{pmatrix} 1370 \\ 1806 \end{pmatrix} \cdot \text{kip}$$

Factored Axial Geotechnical Compressive Resistance - Extreme Limit States

Resistance factor,

$$\phi_{ee} := 1.0$$

Factored Geotechnical Tip Resistance (R_r)

$$R_{r_p2} := \phi_{ee} \cdot R_p$$

$$R_{r_p2} = \begin{pmatrix} 1370 \\ 1806 \end{pmatrix} \cdot \text{kip}$$

Drivability Analyses - Piles End Bearing on Bedrock

Ref: LRFD Article 10.7.8

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

$$\phi_{da} := 1.0$$

Resistance factor from LRFD Table 10.5.5.2.3-1, Drivability Analysis, steel piles

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

$$\sigma_{dr} = 45 \cdot \text{ksi}$$

Driving stress cannot exceed 45 ksi

Limit driving stress to 45 ksi or limit blow count to 5-15 blows per inch (bpi) per Section 501 (Note: 6-10 bpi is considered optimal for diesel hammers).

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

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

$$\phi_{dyn} := 0.65$$

Reference LRFD Table 10.5.5.2.3-1 - for Strength Limit State

$$\phi := 1.0$$

For Extreme and Service Limit States

GRLWeap Soil and Pile Model Assumptions

Estimated pile embedment will be approx. 25 ft. Assume contractor drives pile lengths of 30 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment).

For piles driven to hard rock, use proportional shaft resistances so that GRLWeap will assign approx. 15% of the ultimate capacities as skin friction.

14 x 89 End Bearing on Hard Rock

The 14x89 pile can be driven to the resistance below to hard rock with a D 19-42 hammer at fuel setting Max-1 and 1.9 kip helmet at a reasonable blow count and level of driving stress.

See GRLWEAP results below:

Maine DOT
22624 Willimantic 14x89

28-Nov-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
600.0	41.37	3.08	12.0	9.34	17.39
610.0	41.68	3.19	12.4	9.38	17.47
620.0	41.98	3.29	12.8	9.42	17.59
630.0	42.29	3.36	13.3	9.46	17.65
640.0	42.58	3.42	13.7	9.51	17.77
650.0	42.77	3.48	14.1	9.54	17.85
660.0	43.11	3.55	14.6	9.58	17.92
670.0	43.37	3.61	15.0	9.62	18.04
680.0	43.53	3.76	15.6	9.65	18.07
690.0	43.83	3.88	16.1	9.68	18.14

Limiting blows to 15
per inch

$$R_{\text{ndr}} := 670 \cdot \text{kip}$$

Strength Limit State

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

$$R_{\text{fdr}} = 436 \cdot \text{kip}$$

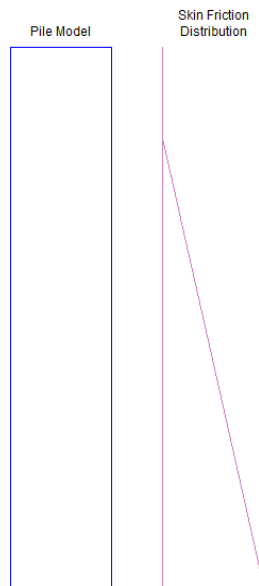
Extreme and Service
Limit States

$$R_{\text{dr}} := R_{\text{ndr}} \cdot \phi$$

$$R_{\text{dr}} = 670 \cdot \text{kip}$$

DELMAG D 19-42

Ram Weight	4.00 kips
Efficiency	0.800
Pressure	1440 (90%) psi
Helmet Weight	1.90 kips
Hammer Cushion	60155 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	30.00 ft
Pile Penetration	25.00 ft
Pile Top Area	26.10 in ²



Res. Shaft = 15 %
(Proportional)

14 x 117 End Bearing on Hard Rock - D19-42

The 14x117 pile can be driven to the resistances below to bear on hard rock with a D 19-42 hammer at max fuel setting and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT
22624 Willimantic 14x117 - D19x42

28-Nov-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
790.0	41.97	3.90	13.8	10.53	20.14
800.0	42.15	3.95	14.1	10.56	20.20
810.0	42.40	3.96	14.4	10.60	20.27
820.0	42.58	4.01	14.8	10.63	20.35
824.0	42.70	4.00	14.9	10.64	20.36
825.0	42.66	4.06	15.0	10.65	20.40
830.0	42.78	4.05	15.2	10.66	20.41
840.0	42.99	4.10	15.5	10.69	20.48
850.0	43.43	4.13	15.7	10.81	20.79
860.0	43.56	4.09	16.1	10.81	20.76

Limit blows to 15 bpi

$$R_{ndr} := 825 \cdot \text{kip}$$

DELMAG D 19-42
 Ram Weight 4.00 kips
 Efficiency 0.800
 Pressure 1600 (100%) psi
 Helmet Weight 1.90 kips
 Hammer Cushion 60155 kips/in
 COR of H.C. 0.800
 Skin Quake 0.100 in
 Toe Quake 0.040 in
 Skin Damping 0.050 sec/ft
 Toe Damping 0.150 sec/ft
 Pile Length 30.00 ft
 Pile Penetration 25.00 ft
 Pile Top Area 34.40 in²

Strength Limit State

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

$$R_{fdr} = 536 \cdot \text{kip}$$

Extreme and Service Limit States

$$R_{dr} := R_{ndr} \cdot \phi$$

$$R_{dr} = 825 \cdot \text{kip}$$

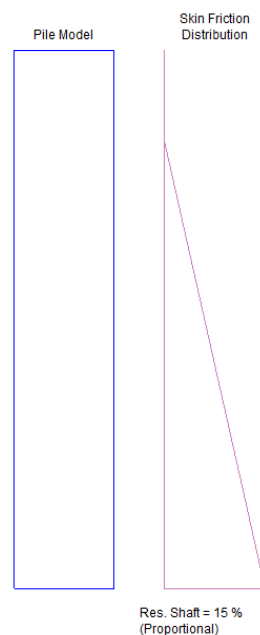


TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	$C_o^{(1)}$	
			(ksf)	(psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale ⁽²⁾	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

⁽¹⁾Range of Uniaxial Compressive Strength values reported by various investigations.⁽²⁾Not including oil shale.

$$\rho = q_o (1 - \nu^2) B I_p / E_m, \text{ with } I_p = (L/B)^{1/2} / \beta_z \quad (4.4.8.2.2-2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (ν) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \quad (4.4.8.2.2-3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \quad (4.4.8.2.2-4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m .

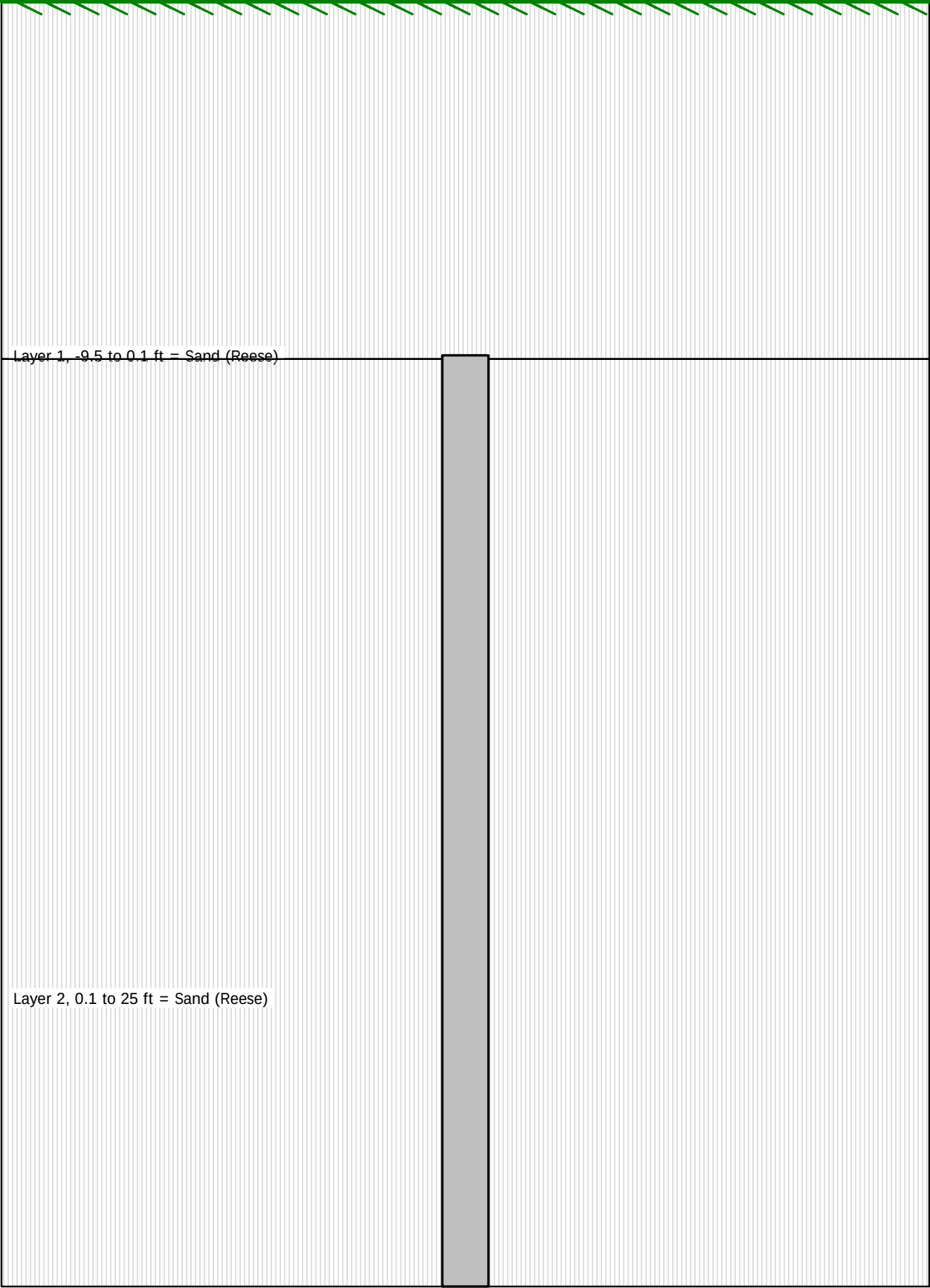
4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

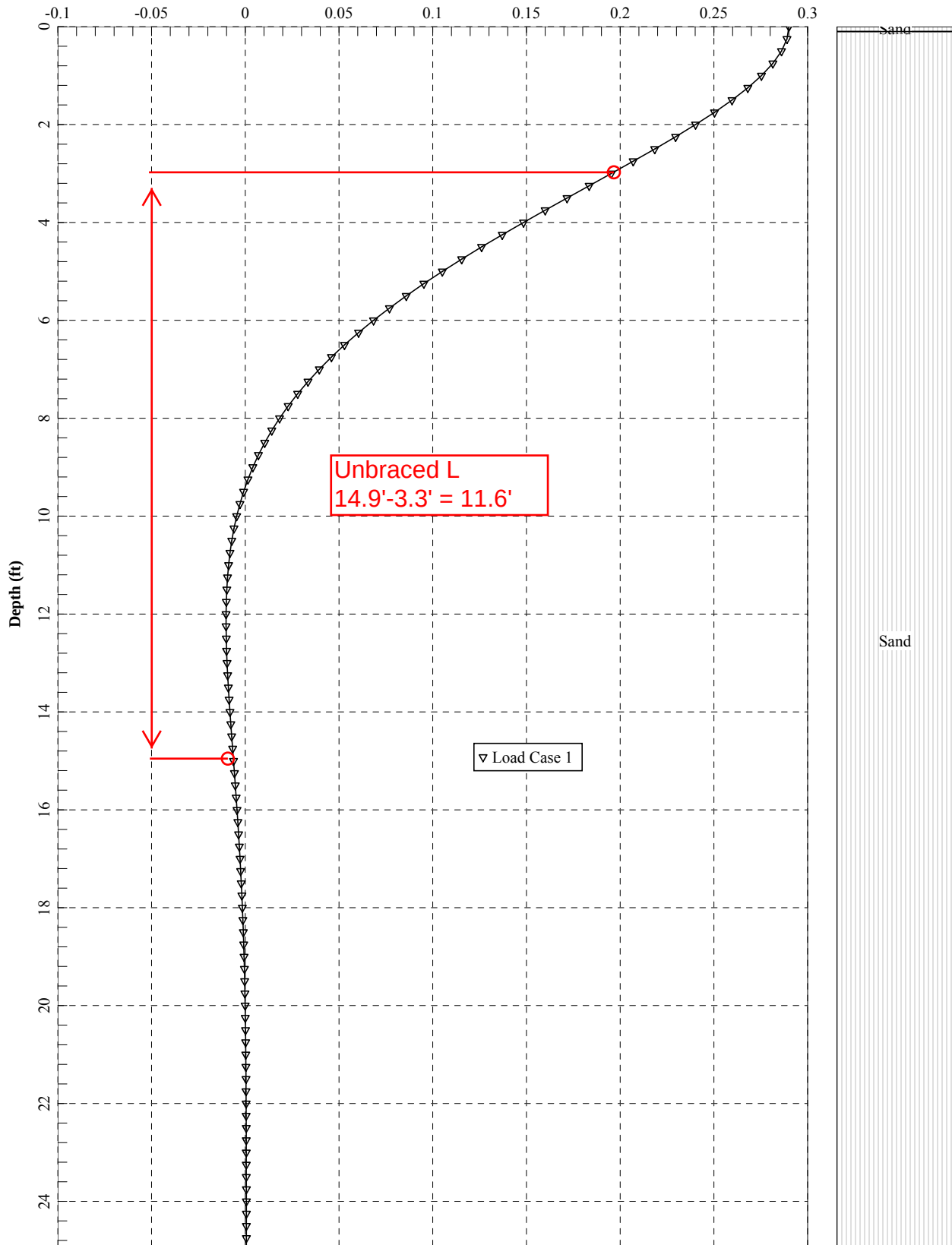
4.4.9 Overall Stability

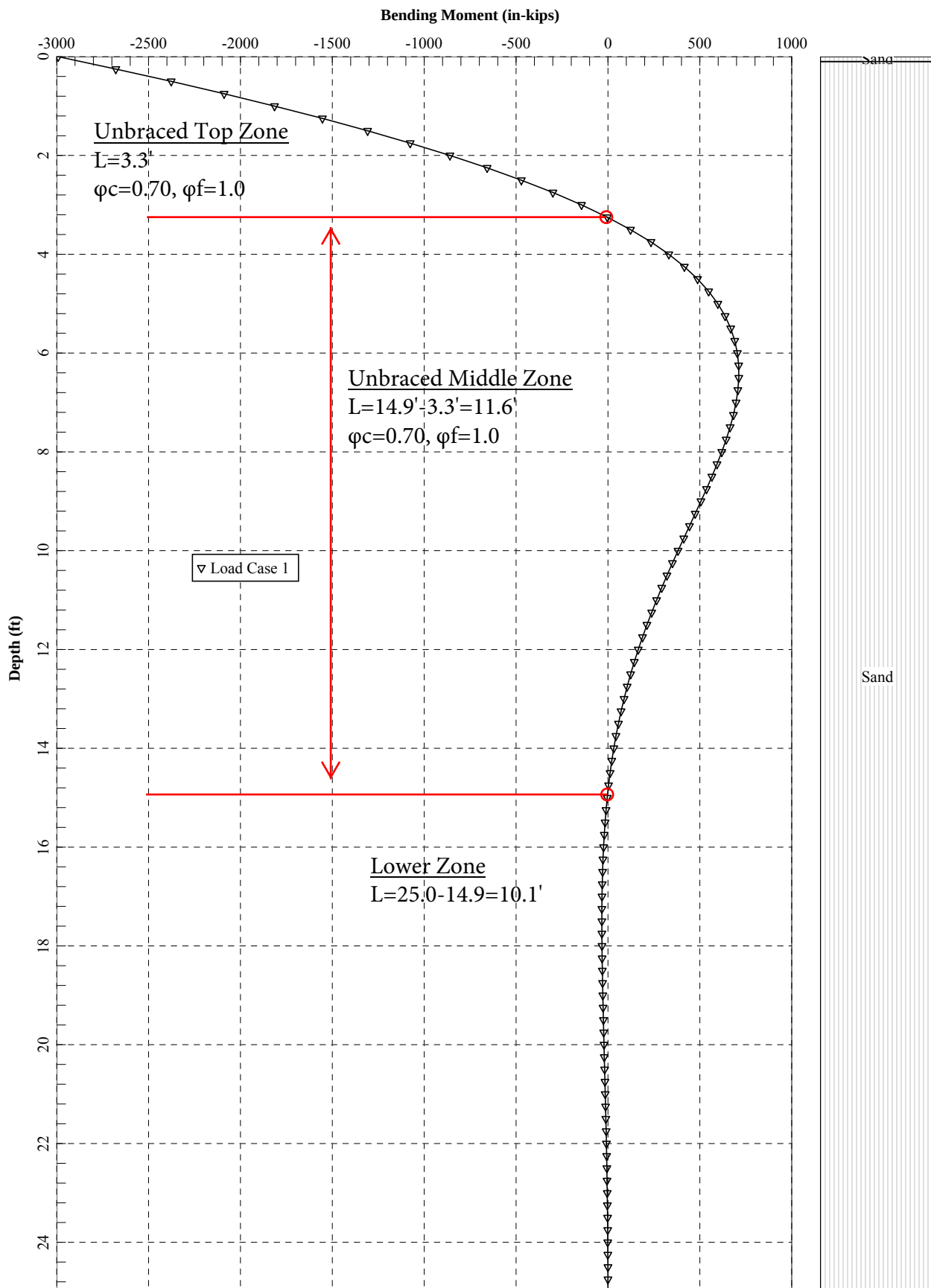
The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

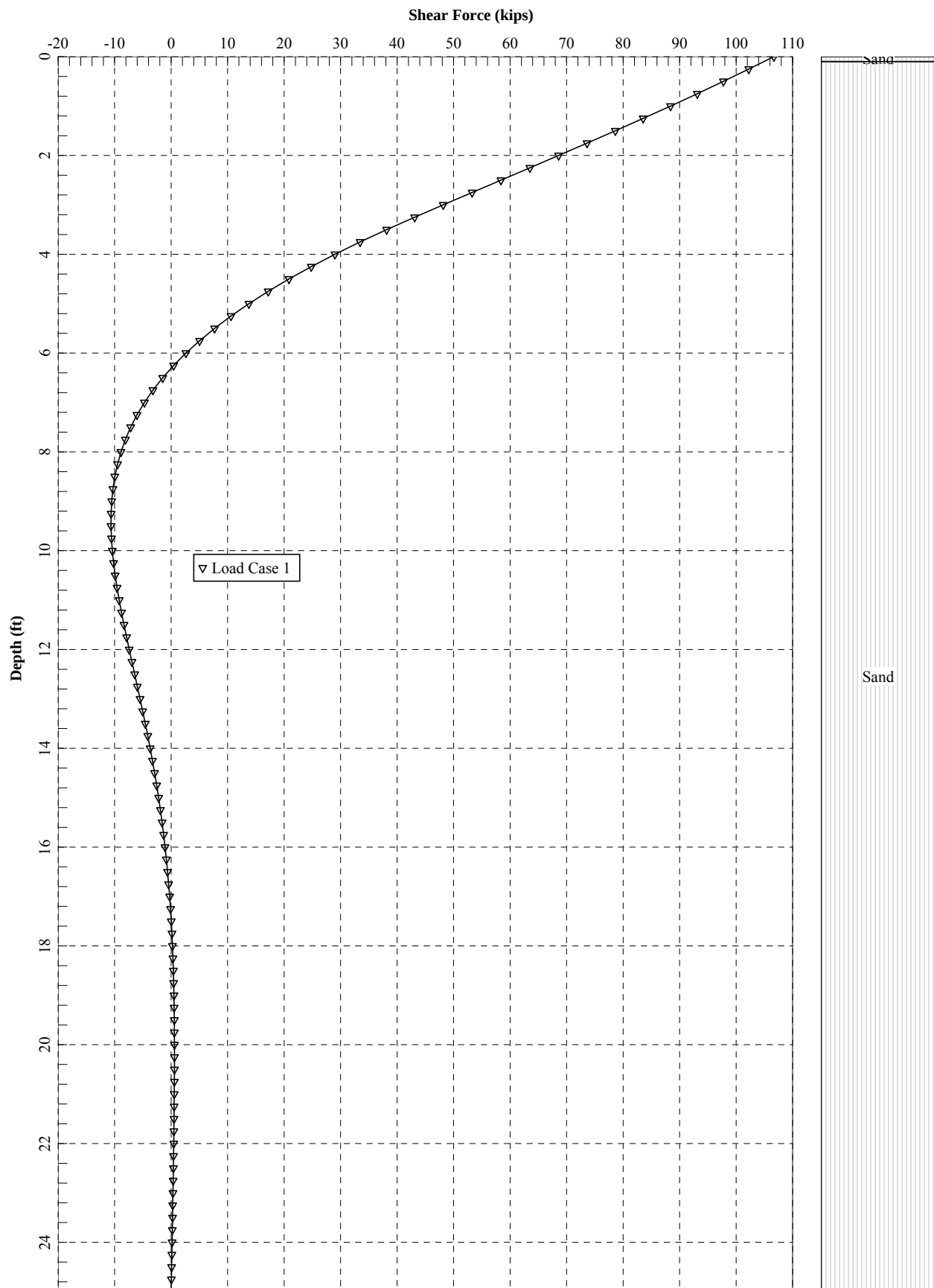
Lateral Pile Analysis



12x74 Weak 18L Medium Dense
Lateral Pile Deflection (inches)







14x117 Weak 25 med.lp9o

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LPile for Windows, Version 2016-09.008
Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Date and Time of Analysis

Date: August 24, 2018

Time: 11:03:52

Problem Title

Project Name: Williamantic Goodell Bridge
Job Number: 22624
Client: MaineDOT
Engineer: Slaven
Description: 14x89 Weak 25L Med

Program Options and Settings

Computational Options:

- Use unfactored loads in computations (conventional analysis)
- Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- | | | |
|--|---|---------------|
| - Maximum number of iterations allowed | = | 500 |
| - Deflection tolerance for convergence | = | 1.0000E-05 in |
| - Maximum allowable deflection | = | 100.0000 in |
| - Number of pile increments | = | 100 |

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using narrow report formats
- (Note: Some output information is omitted from the narrow report formats)

 Pile Structural Properties and Geometry

Number of pile sections defined = 1
 Total length of pile = 25.000 ft
 Depth of ground surface below top of pile = -9.5000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.2000
2	25.000	14.2000

 Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
 Cross-sectional Shape = Weak H-Pile
 Length of section = 25.000000 ft
 Flange Width = 14.900000 in
 Section Depth = 14.200000 in
 Flange Thickness = 0.805000 in
 Web Thickness = 0.805000 in
 Section Area = 34.400000 sq. in
 Moment of Inertia = 443.000000 in⁴
 Elastic Modulus = 29000000. psi

 Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
 = 0.000 radians
 Pile Batter Angle = 0.000 degrees
 = 0.000 radians

 Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = -9.500000 ft
 Distance from top of pile to bottom of layer = 0.100000 ft
 Effective unit weight at top of layer = 125.000000 pcf
 Effective unit weight at bottom of layer = 125.000000 pcf
 Friction angle at top of layer = 32.000000 deg.

14x117 Weak 25 med.lp90

Friction angle at bottom of layer	=	32.000000 deg.
Subgrade k at top of layer	=	90.000000 pci
Subgrade k at bottom of layer	=	90.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	0.100000 ft
Distance from top of pile to bottom of layer	=	25.000000 ft
Effective unit weight at top of layer	=	80.000000 pcf
Effective unit weight at bottom of layer	=	80.000000 pcf
Friction angle at top of layer	=	32.000000 deg.
Friction angle at bottom of layer	=	32.000000 deg.
Subgrade k at top of layer	=	60.000000 pci
Subgrade k at bottom of layer	=	60.000000 pci

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs
1	5	y = 0.290000 in	S = 0.0000 in/in	338500.

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

14x117 Weak 25 med.l.p90

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-9.5000	0.00	N.A.	No	0.00	2655.
2	0.1000	9.6000	Yes	No	2655.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 0.290000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 338500.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Soil Res. p lb/inch	Bending Stiffness in-lb^2
0.000	0.29000	-2991857.	106687.	-1415.	1.285E+10
0.25000	0.28895	-2678111.	102253.	-1474.	1.285E+10
0.50000	0.28603	-2376995.	97760.	-1521.	1.285E+10
0.75000	0.28144	-2089007.	93134.	-1563.	1.285E+10
1.00000	0.27539	-1814591.	88390.	-1599.	1.285E+10
1.25000	0.26806	-1554137.	83547.	-1630.	1.285E+10
1.50000	0.25965	-1307981.	78621.	-1655.	1.285E+10
1.75000	0.25032	-1076407.	73627.	-1675.	1.285E+10
2.00000	0.24024	-859650.	68580.	-1690.	1.285E+10
2.25000	0.22955	-657898.	63495.	-1700.	1.285E+10
2.50000	0.21841	-471289.	58388.	-1705.	1.285E+10
2.75000	0.20693	-299911.	53274.	-1704.	1.285E+10
3.00000	0.19524	-143802.	48170.	-1699.	1.285E+10
3.25000	0.18345	-2948.	43095.	-1684.	1.285E+10
3.50000	0.17167	122751.	38159.	-1607.	1.285E+10
3.75000	0.15996	233958.	33460.	-1526.	1.285E+10
4.00000	0.14842	331377.	29007.	-1443.	1.285E+10
4.25000	0.13712	415732.	24806.	-1357.	1.285E+10
4.50000	0.12610	487772.	20864.	-1271.	1.285E+10
4.75000	0.11542	548257.	17181.	-1184.	1.285E+10
5.00000	0.10513	597953.	13758.	-1098.	1.285E+10
5.25000	0.09526	637629.	10594.	-1012.	1.285E+10
5.50000	0.08584	668048.	7686.	-927.05475	1.285E+10
5.75000	0.07688	689966.	5029.	-844.16119	1.285E+10
6.00000	0.06841	704123.	2618.	-763.43749	1.285E+10
6.25000	0.06043	711241.	444.58453	-685.25714	1.285E+10
6.50000	0.05295	712024.	-1498.	-609.94393	1.285E+10
6.75000	0.04596	707148.	-3220.	-537.77336	1.285E+10
7.00000	0.03948	697265.	-4730.	-468.97421	1.285E+10
7.25000	0.03348	682995.	-6039.	-403.73048	1.285E+10

m=0
@ 3.3'

14x117 Weak 25 med.lp9o

7.50000	0.02796	664931.	-7158.	-342.18358	1.285E+10
7.75000	0.02290	643628.	-8098.	-284.43473	1.285E+10
8.00000	0.01830	619613.	-8870.	-230.54742	1.285E+10
8.25000	0.01413	593377.	-9487.	-180.55014	1.285E+10
8.50000	0.01037	565374.	-9959.	-134.43906	1.285E+10
8.75000	0.007015	536028.	-10299.	-92.18091	1.285E+10
9.00000	0.004033	505725.	-10518.	-53.71571	1.285E+10
9.25000	0.001404	474818.	-10627.	-18.95966	1.285E+10
9.50000	-0.000891	443629.	-10637.	12.19213	1.285E+10
9.75000	-0.002876	412443.	-10559.	39.86287	1.285E+10
10.00000	-0.004572	381519.	-10403.	64.19147	1.285E+10
10.25000	-0.006001	351082.	-10179.	85.32997	1.285E+10
10.50000	-0.007183	321330.	-9896.	103.44108	1.285E+10
10.75000	-0.008141	292432.	-9563.	118.69588	1.285E+10
11.00000	-0.008894	264534.	-9188.	131.27163	1.285E+10
11.25000	-0.009461	237754.	-8779.	141.34967	1.285E+10
11.50000	-0.009862	212190.	-8343.	149.11361	1.285E+10
11.75000	-0.01011	187918.	-7887.	154.74751	1.285E+10
12.00000	-0.01023	164993.	-7417.	158.43432	1.285E+10
12.25000	-0.01024	143456.	-6939.	160.35445	1.285E+10
12.50000	-0.01014	123328.	-6458.	160.68447	1.285E+10
12.75000	-0.009962	104616.	-5977.	159.59602	1.285E+10
13.00000	-0.009707	87316.	-5502.	157.25479	1.285E+10
13.25000	-0.009391	71411.	-5035.	153.81973	1.285E+10
13.50000	-0.009024	56873.	-4580.	149.44231	1.285E+10
13.75000	-0.008618	43667.	-4140.	144.26602	1.285E+10
14.00000	-0.008181	31749.	-3716.	138.42588	1.285E+10
14.25000	-0.007722	21069.	-3310.	132.04813	1.285E+10
14.50000	-0.007248	11572.	-2924.	125.25007	1.285E+10
14.75000	-0.006766	3200.	-2559.	118.13987	1.285E+10
15.00000	-0.006282	-4109.	-2216.	110.81662	1.285E+10
15.25000	-0.005801	-10420.	-1894.	103.37036	1.285E+10
15.50000	-0.005327	-15799.	-1595.	95.88222	1.285E+10
15.75000	-0.004864	-20310.	-1319.	88.42465	1.285E+10
16.00000	-0.004415	-24021.	-1065.	81.06165	1.285E+10
16.25000	-0.003983	-26997.	-832.42491	73.84910	1.285E+10
16.50000	-0.003570	-29302.	-621.39856	66.83513	1.285E+10
16.75000	-0.003178	-30998.	-431.05520	60.06045	1.285E+10
17.00000	-0.002807	-32147.	-260.62629	53.55883	1.285E+10
17.25000	-0.002459	-32805.	-109.25183	47.35748	1.285E+10
17.50000	-0.002134	-33030.	24.00070	41.47754	1.285E+10
17.75000	-0.001832	-32874.	140.11878	35.93451	1.285E+10
18.00000	-0.001552	-32386.	240.12863	30.73872	1.285E+10
18.25000	-0.001296	-31614.	325.08036	25.89577	1.285E+10
18.50000	-0.001062	-30602.	396.03453	21.40701	1.285E+10
18.75000	-0.000849	-29389.	454.05003	17.26999	1.285E+10
19.00000	-0.000657	-28014.	500.17326	13.47883	1.285E+10
19.25000	-0.000484	-26512.	535.42861	10.02474	1.285E+10
19.50000	-0.000330	-24912.	560.81018	6.89631	1.285E+10
19.75000	-0.000194	-23245.	577.27464	4.07999	1.285E+10
20.00000	-7.347E-05	-21536.	585.73523	1.56040	1.285E+10
20.25000	3.171E-05	-19807.	587.05683	-0.67933	1.285E+10
20.50000	0.000123	-18080.	582.05202	-2.65721	1.285E+10
20.75000	0.000202	-16372.	571.47807	-4.39209	1.285E+10
21.00000	0.000269	-14700.	556.03482	-5.90341	1.285E+10
21.25000	0.000326	-13078.	536.36333	-7.21091	1.285E+10
21.50000	0.000373	-11518.	513.04539	-8.33439	1.285E+10
21.75000	0.000413	-10029.	486.60357	-9.29350	1.285E+10
22.00000	0.000446	-8622.	457.50204	-10.10752	1.285E+10

m=0
@ 14.9'

14x117 Weak 25 med.lp9o					
22.25000	0.000472	-7304.	426.14794	-10.79521	1.285E+10
22.50000	0.000494	-6082.	392.89329	-11.37456	1.285E+10
22.75000	0.000511	-4960.	358.03738	-11.86271	1.285E+10
23.00000	0.000525	-3944.	321.82971	-12.27574	1.285E+10
23.25000	0.000536	-3037.	284.47327	-12.62856	1.285E+10
23.50000	0.000544	-2244.	246.12830	-12.93476	1.285E+10
23.75000	0.000552	-1566.	206.91639	-13.20651	1.285E+10
24.00000	0.000558	-1007.	166.92501	-13.45441	1.285E+10
24.25000	0.000563	-568.48661	126.21230	-13.68739	1.285E+10
24.50000	0.000568	-253.15489	84.81234	-13.91258	1.285E+10
24.75000	0.000573	-62.97640	42.74063	-14.13522	1.285E+10
25.00000	0.000578	0.000	0.000	-14.35853	1.285E+10

Output Summary for Load Case No. 1:

Pile-head deflection	=	0.29000000 inches
Computed slope at pile head	=	0.000000 radians
Maximum bending moment	=	-2991857. inch-lbs
Maximum shear force	=	106687. lbs
Depth of maximum bending moment	=	0.000000 feet below pile head
Depth of maximum shear force	=	0.000000 feet below pile head
Number of iterations	=	6
Number of zero deflection points	=	2

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	5	0.290000	0.0000	106687.	-2991857.

Maximum pile-head deflection = 0.2900000000 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.

Earth Pressure

Earth Pressure:**Backfill engineering strength parameters**

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot \text{pcf}$ Internal friction angle $\phi' := 32 \cdot \text{deg}$ Cohesion $c_1 := 0 \cdot \text{psf}$ **Integral Abutment - Passive Earth Pressure - Coulomb Theory** α = Angle of fill slope to the horizontal $\alpha := 0 \cdot \text{deg}$ ϕ_1 = Angle of internal friction $\phi' = 32 \cdot \text{deg}$ β = Angle of back face of wall to the horizontal $\beta := 90 \cdot \text{deg}$

Use Coulomb for cases where interface friction is considered; typically gravity shaped structures, and integral abutments where the ratio of wall height to wall movement is .005 or greater. Coulomb should also be used when the fill slope is greater than horizontal.

For precast IAB abutment against clean sand, silty sand-gravel mixture use $\delta = 17 - 22$, per LRFD Table 3.11.5.3-1

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

 $\delta' := 19.5 \cdot \text{deg}$

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

Das, Principles of
Foundation Engineering
7th Ed. p. 366 Eq. 7.71

$$K_{p_coulomb} = 6.73$$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Use Rankine only if the ratio of wall height to wall movement is significantly less than .005 and the fill slope is horizontal to the top of the wall. Bowles does not recommend use of Rankine method for K_p when $\alpha > 0$.

 α = Angle of fill slope to the horizontal $\alpha := 0 \cdot \text{deg}$

$$K_{p_rank} := \cos(\alpha) \cdot \frac{\cos(\alpha) + \sqrt{\cos(\alpha)^2 - \cos(\phi')^2}}{\cos(\alpha) - \sqrt{\cos(\alpha)^2 - \cos(\phi')^2}}$$

Das, Principles of
Foundation Engineering
7th Ed. p. 363 Eq. 7.67

$$K_{p_rank} = 3.25$$

P_p is oriented at an angle of α to the vertical plane

Cantilevered Wingwall Live Load Surcharge
At-Rest Earth Pressure - Rankine Theory

$$K_o := 1 - \sin(\phi')$$

$$K_o = 0.47$$

Das, Principles of
Geotechnical Engineering
7th Ed. p 427 Eq. 13.5

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, $\tan \delta$ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
o dressed soft rock on dressed soft rock	35	0.70
o dressed hard rock on dressed soft rock	33	0.65
o dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_r .

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

Table 7.10 Values of K_p [from Eq. (7.71)] for $\beta = 90^\circ$ and $\alpha = 0^\circ$

ϕ' (deg)	δ' (deg)				
	0	5	10	15	20
15	1.698	1.900	2.130	2.405	2.735
20	2.040	2.313	2.636	3.030	3.525
25	2.464	2.830	3.286	3.855	4.597
30	3.000	3.506	4.143	4.977	6.105
35	3.690	4.390	5.310	6.854	8.324
40	4.600	5.590	6.946	8.870	11.772

Figure 7.25b shows the force triangle at equilibrium for the trial wedge ABC_1 . From this force triangle, the value of P_p can be determined, because the direction of all three forces and the magnitude of one force are known.

Similar force triangles for several trial wedges, such as $ABC_1, ABC_2, ABC_3, \dots$, can be constructed, and the corresponding values of P_p can be determined. The top part of Figure 7.25a shows the nature of variation of the P_p values for different wedges. The *minimum* value of P_p in this diagram is *Coulomb's passive force*, mathematically expressed as

$$P_p = \frac{1}{2} \gamma H^2 K_p \quad (7.70)$$

where

$$K_p = \text{Coulomb's passive pressure coefficient} \\ = \frac{\sin^2(\beta - \phi')}{\sin^2 \beta \sin(\beta + \delta') \left[1 - \sqrt{\frac{\sin(\phi' + \delta') \sin(\phi' + \alpha)}{\sin(\beta + \delta') \sin(\beta + \alpha)}} \right]^2} \quad (7.71)$$

The values of the passive pressure coefficient, K_p , for various values of ϕ' and δ' are given in Table 7.10 ($\beta = 90^\circ, \alpha = 0^\circ$).

Note that the resultant passive force, P_p , will act at a distance $H/3$ from the bottom of the wall and will be inclined at an angle δ' to the normal drawn to the back face of the wall.

7.13

Comments on the Failure Surface Assumption for Coulomb's Pressure Calculations

Coulomb's pressure calculation methods for active and passive pressure have been discussed in Sections 7.5 and 7.12. The fundamental assumption in these analyses is the acceptance of *plane failure surface*. However, for walls with friction, this assumption does not hold in practice. The nature of *actual* failure surface in the soil mass for active and passive pressure is shown in Figure 7.26a and b, respectively (for a vertical wall with a horizontal backfill). Note that the failure surface BC is curved and that the failure surface CD is a plane.

Although the actual failure surface in soil for the case of active pressure is somewhat different from that assumed in the calculation of the Coulomb pressure, the results are not greatly different. However, in the case of passive pressure, as the value of δ' increases, Coulomb's

Table 7.9 (Continued)

ϕ' (deg)	α (deg)	$c'/\gamma z$			
		0.025	0.050	0.100	0.500
30	0	3.087	3.173	3.346	4.732
	5	3.042	3.129	3.303	4.674
	10	2.907	2.996	3.174	4.579
	15	2.684	2.777	2.961	4.394

7.12 Coulomb's Passive Earth Pressure

Coulomb (1776) also presented an analysis for determining the passive earth pressure (i.e., when the wall moves *into* the soil mass) for walls possessing friction ($\delta' =$ angle of wall friction) and retaining a granular backfill material similar to that discussed in Section 7.5.

To understand the determination of Coulomb's passive force, P_p , consider the wall shown in Figure 7.25a. As in the case of active pressure, Coulomb assumed that the potential failure surface in soil is a plane. For a trial failure wedge of soil, such as ABC_1 , the forces per unit length of the wall acting on the wedge are

1. The weight of the wedge, W
2. The resultant, R , of the normal and shear forces on the plane BC_1 , and
3. The passive force, P_p

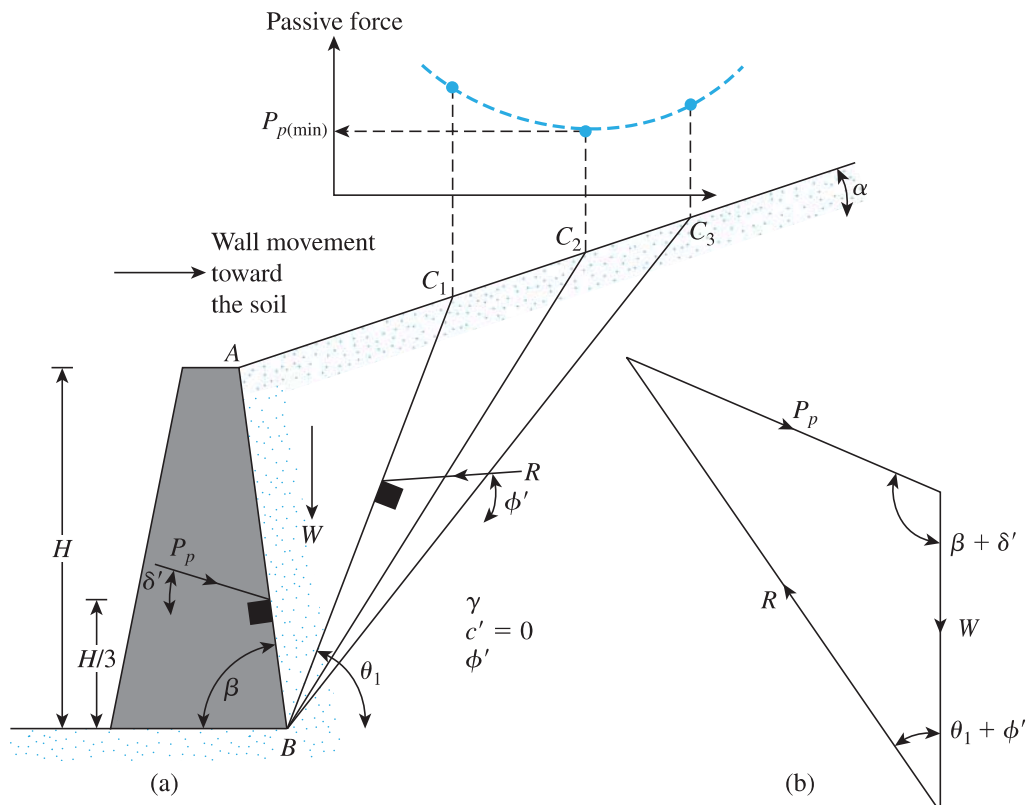


Figure 7.25 Coulomb's passive pressure

At this depth, that is $z = 2$ m, for the bottom soil layer

$$\begin{aligned}\sigma'_p &= \sigma'_o K_{p(2)} + 2c'_2 \sqrt{K_{p(2)}} = 31.44(2.56) + 2(10)\sqrt{2.56} \\ &= 80.49 + 32 = 112.49 \text{ kN/m}^2\end{aligned}$$

Again, at $z = 3$ m,

$$\begin{aligned}\sigma'_o &= (15.72)(2) + (\gamma_{\text{sat}} - \gamma_w)(1) \\ &= 31.44 + (18.86 - 9.81)(1) = 40.49 \text{ kN/m}^2\end{aligned}$$

Hence,

$$\begin{aligned}\sigma'_p &= \sigma'_o K_{p(2)} + 2c'_2 \sqrt{K_{p(2)}} = 40.49(2.56) + (2)(10)(1.6) \\ &= 135.65 \text{ kN/m}^2\end{aligned}$$

Note that, because a water table is present, the hydrostatic stress, u , also has to be taken into consideration. For $z = 0$ to 2 m, $u = 0$; $z = 3$ m, $u = (1)(\gamma_w) = 9.81 \text{ kN/m}^2$.

The passive pressure diagram is plotted in Figure 6.24b. The passive force per unit length of the wall can be determined from the area of the pressure diagram as follows:

Area no.	Area	
1	$(\frac{1}{2})(2)(94.32)$	$= 94.32$
2	$(112.49)(1)$	$= 112.49$
3	$(\frac{1}{2})(1)(135.65 - 112.49)$	$= 11.58$
4	$(\frac{1}{2})(9.81)(1)$	$= 4.905$
		$P_p \approx 223.3 \text{ kN/m}$

7.11

Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

For a frictionless vertical retaining wall (Figure 7.10) with a *granular backfill* ($c' = 0$), the Rankine passive pressure at any depth can be determined in a manner similar to that done in the case of active pressure in Section 7.4. The pressure is

$$\sigma'_p = \gamma z K_p \quad (7.65)$$

and the passive force is

$$P_p = \frac{1}{2} \gamma H^2 K_p \quad (7.66)$$

where

$$K_p = \cos \alpha \frac{\cos \alpha + \sqrt{\cos^2 \alpha - \cos^2 \phi'}}{\cos \alpha - \sqrt{\cos^2 \alpha - \cos^2 \phi'}} \quad (7.67)$$

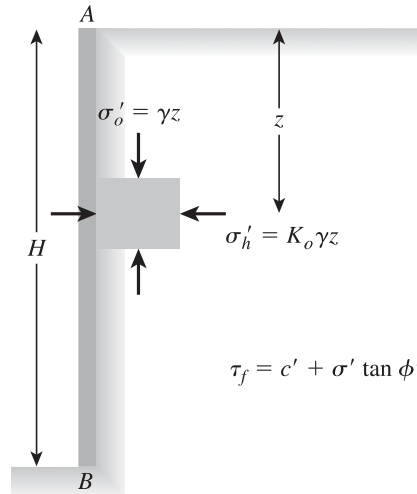


Figure 13.3
Earth pressure at rest

which shows a wall AB retaining a dry soil with a unit weight of γ . The wall is static. At a depth z ,

$$\text{Vertical effective stress} = \sigma'_o = \gamma z$$

$$\text{Horizontal effective stress} = \sigma'_h = K_o \gamma z$$

So,

$$K_o = \frac{\sigma'_h}{\sigma'_o} = \text{at-rest earth pressure coefficient}$$

For coarse-grained soils, the coefficient of earth pressure at rest can be estimated by using the empirical relationship (Jaky, 1944)

$$K_o = 1 - \sin \phi' \quad (13.5)$$

where $\phi' =$ drained friction angle.

While designing a wall that may be subjected to lateral earth pressure at rest, one must take care in evaluating the value of K_o . Sherif, Fang, and Sherif (1984), on the basis of their laboratory tests, showed that Jaky's equation for K_o [Eq. (13.5)] gives good results when the backfill is loose sand. However, for a dense, compacted sand backfill, Eq. (13.5) may grossly underestimate the lateral earth pressure at rest. This underestimation results because of the process of compaction of backfill. For this reason, they recommended the design relationship

$$K_o = (1 - \sin \phi) + \left[\frac{\gamma_d}{\gamma_{d(\min)}} - 1 \right] 5.5 \quad (13.6)$$

where $\gamma_d =$ actual compacted dry unit weight of the sand behind the wall
 $\gamma_{d(\min)} =$ dry unit weight of the sand in the loosest state (Chapter 3)

Frost Depth

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

From Design Freezing Index Map: **Willimantic, Maine**

DFI = 2200 degree-days.

Case 1 - coarse grained granular fill soils W=10% (assumed).

For DFI = 2200

$d := 100 \cdot \text{in}$

Depth of Frost Penetration

$d := 100 \text{ in}$

$d = 8.3 \cdot \text{ft}$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Gardiner lies along the same Maine Design Freezing Index contour - use Gardiner data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Houlton 5 N, Maine

Air Design Freezing Index = 2189 F-days

N-Factor = 0.80

Surface Design Freezing Index = 1751 F-days

Mean Annual Temperature = 40.8 deg F

Design Length of Freezing Season = 148 days

Layer

#:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	96.1	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).

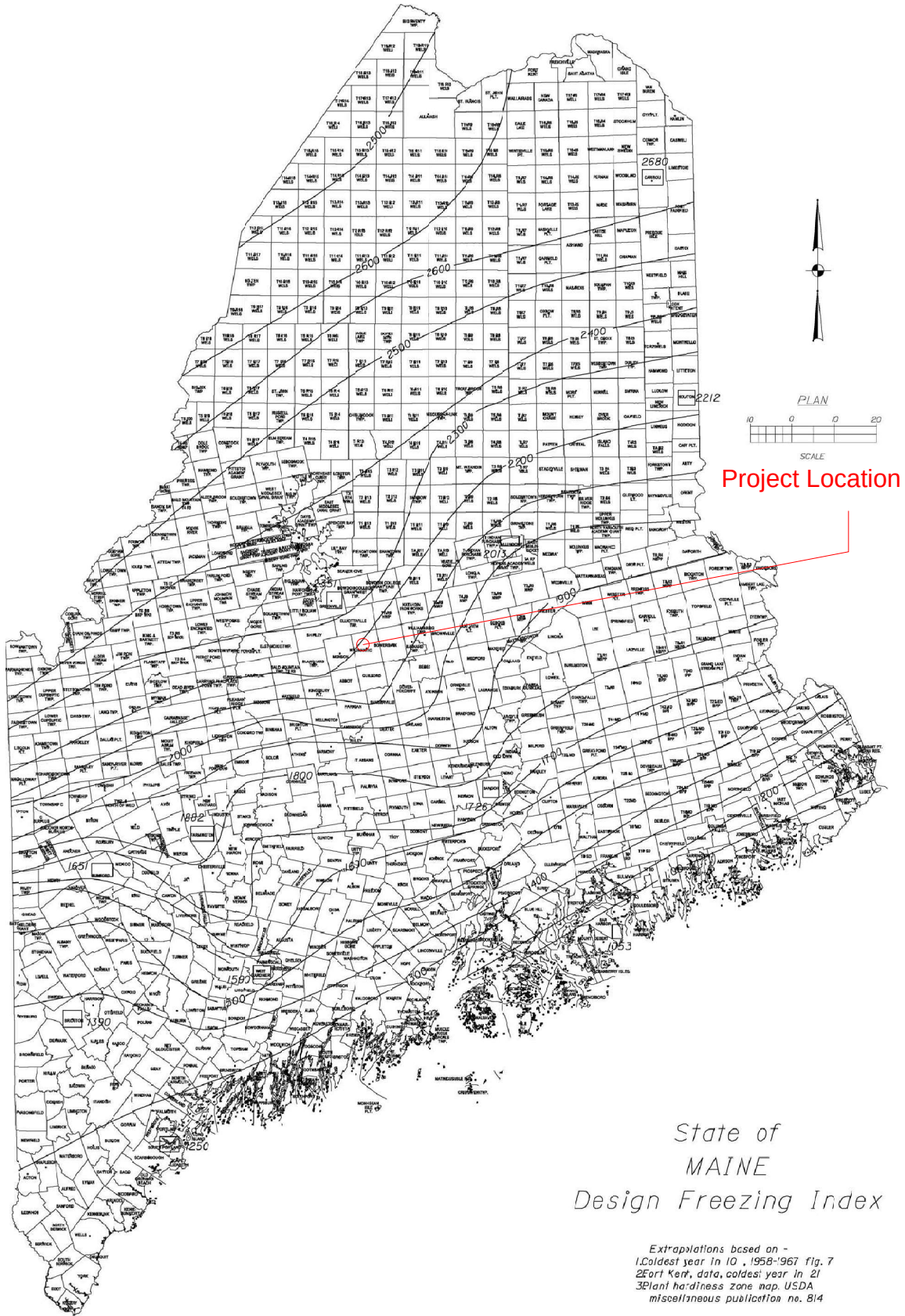
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

L = Latent heat of fusion, in BTU / cubic ft.

Total Depth of Frost Penetration = 8.01 ft = 96.1 in.

Recommendation: 8.3 feet for design of foundations constructed on coarse grained soils

Figure 5-1 Maine Design Freezing Index Map



5.2 General**MaineDOT Bridge Design Guide****5.2.1 Frost**

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Seismic Design

BB-WDB-101A				
Depth	N ₆₀		di	di/N
6	4		6	1.50
11	63		5	0.08
15.4	50		4.4	0.09
19.5	50	Bedrock	4.1	0.08
100	100		80.5	0.81
SUM			100	2.55

di/di/N 39.15

BB-WDB-201				
Depth	N ₆₀		di	di/N
3	11		3	0.27
6	29		3	0.10
10	50		4	0.08
16	66		6	0.09
20.5	50		4.5	0.09
25	100	Bedrock	4.5	0.05
100	100		75	0.75
SUM			100	1.43

di/di/N 69.83

BB-WDB-202				
Depth	N ₆₀		di	di/N
6	38		6	0.16
10.75	48		4.75	0.10
15	6		4.25	0.71
19.75	50		4.75	0.10
25.9	80		6.15	0.08
30.6	50		4.7	0.09
35.1	100	Bedrock	4.5	0.05
100	100		64.9	0.65
SUM			100	1.28

di/di/N 78.37

SUM	Nav.	62.45
-----	------	-------

N_{av.} > 50

Conclusion: Site Class C

Site Classification per LRFD Table C3.10.3.1-1 - Method B

22624 Willimantic
Goodell Br # 2317

Seismic Parameters

B. Slaven
Nov 2018
Check by: lk 11/2018

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years

Latitude = 45.283020

Longitude = -069.425690

Site Class B

Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)
0.0	0.073	PGA - Site Class B
0.2	0.160	Ss - Site Class B
1.0	0.049	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1

Latitude = 45.283020

Longitude = -069.425690

As = FpgaPGA, SDs = FaSs, and SD1 = FvS1

Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70

Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)
0.0	0.087	As - Site Class C
0.2	0.192	SDs - Site Class C
1.0	0.083	SD1 - Site Class

Appendix E

Draft Special Provision

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Predrilling Holes for Piles)

501.01 Description. The following is added to Subsection 501.01 of the Standard Specifications:

This work shall consist of providing all materials, equipment, and labor necessary for drilling, boring, or augering holes, casing holes, and backfilling holes for piles at the locations and to the elevation indicated on the Plans, or as authorized by the Resident. Predrilling holes for piles shall be as specified in Section 501 of the Standard Specifications, except as amended herein.

501.02 Materials. The following is added to Subsection 501.02 of the Standard Specifications:

Temporary casing shall be of a material sufficiently rigid to keep the pile hole from collapsing. It is the Contractor's responsibility to choose a material that will prevent collapse.

Backfill aggregate shall meet the requirements of Subsection 703.22, Underdrain Backfill Material, Type C, or as approved by the Resident.

501.03 Quality Control Plan. The following is added to Subsection 501.03 of the Standard Specifications:

The QCP shall meet the requirements of Subsection 501.02 and address the following additional elements:

- H. Proposed equipment and tooling for drilling, excavation, boring, or augering holes
- I. Methods for backfilling predrilled holes
- J. Construction sequence for predrilling, backfilling and driving piles
- K. Sample construction log

501.042 Equipment. The following is added to Subsection 501.042:

Predrilling. Drilling of holes for piles shall use conventional excavation and cased-hole drilling methods. The Contractor shall select equipment and tooling capable of excavating or drilling through in situ soils, rock, cobbles, boulders, buried obstacles, wood, debris, footings, and concrete slabs whether indicated on the Plans or not. It is the Contractor's responsibility to provide equipment and tooling capable of excavating or drilling to the required elevation.

Jetting or blasting is prohibited.

501.043 Location and Alignment Tolerances. The following is added to Subsection 501.043:

The bottom elevation of predrilled holes for piles shall vary no more than 6 inches above or 6 inches below the elevation indicated on the Plans.

501.049 Predrilling Holes for Piles. The following is added to Subsection 501.049 of the Standard Specifications:

The Contractor shall predrill holes for piles at locations and to the elevations and dimensions shown on the Plans. If no diameter is shown on the plans, the predrilled hole shall be at least 4 inches larger in diameter than the largest diagonal measurement of the pile. Predrilled holes for piles through soil overburden shall be cased.

The Contractor shall maintain a construction log during excavation and predrilling. For each pile, the log shall record the following: pile number, date, start time, stop time, personnel, weather, drilling methods, drilling resistance, obstructions, diameter of casing, diameter of predrilled hole, top and bottom elevation of temporary casing, depth/elevation of top of bedrock, elevation of bottom of predrilled hole, and theoretical volume of excavation.

The Contractor shall dispose of excavated materials removed from socket excavations in accordance with the applicable specifications for disposal of excavated materials.

The Contractor shall perform the necessary excavation for the Predrilling of Holes for Piles and backfilling under this item. No separate payment will be made for excavation of materials of different densities or employment of special tools and procedures necessary to accomplish the excavation.

Weighted tape or other approved methods shall determine final hole depths.

501.0491 Backfill Installation

The cased hole shall be filled with aggregate conforming to the materials portion of this provision. The aggregate shall be dropped. The casing shall be withdrawn as aggregate is placed. The Contractor shall maintain the bottom elevation of the casing a minimum of 24 inches below the aggregate as the aggregate is dropped and the casing is withdrawn. At a minimum, the Contractor shall place aggregate to the bottom elevation of the concrete pile jacket. Temporary casing removed shall remain the property of the Contractor.

The Contractor shall backfill predrilled holes prior to pile driving.

501.05 Method of Measurement. The following is added to Subsection 501.05 of the Standard Specifications:

- a) Drilling Equipment Mobilization Predrilling Holes for Piles. Mobilization of drilling equipment shall be as outlined in Subsection 501.05(a), Equipment Mobilization.
- b) Predrilling Holes for Piles. Method of measurement for predrilling holes for piles as described in this Section shall be measured by the linear foot of drilled and backfilled hole.

501.06 Basis of Payment. The following is added to subsection 501.06:

The accepted quantity of Predrilling Holes for Piles will be paid for at the Contract Unit Price per linear foot, complete and in place. Such payment will be full compensation for all materials, equipment, and labor to drill and excavate holes for piles, backfilling, and all incidentals necessary to complete the work specified in the Contract Documents.

Payment shall be under:

<u>Pay Items</u>		<u>Pay Unit</u>
501.81	Predrilling Holes for Piles	Linear foot
501.8041	Drilling Equipment Mobilization, Predrilling Holes for Piles	Lump Sum