MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MERRILL BRIDGE
ROUTE 120 OVER WEST BRANCH ELLIS RIVER
ANDOVER, 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 Merrill Bridge which carries Route 120 over West Branch Ellis River in Andover, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical recommendations, and geotechnical design parameters for the design of the new substructures.

The existing bridge is a simple span riveted steel half through truss. The substructures consist of concrete capped stacked stone abutments bearing on soil. According to the 2015 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the superstructure and substructure are rated 4 for poor condition. A few stacked stones are cracked and there are multiple areas with large voids between them. The bridge is non-redundant, structurally deficient and in need of replacement. The Sufficiency Rating of the bridge is 9.7.

The recommended bridge replacement alternative is a 130-foot single span structure, comprised of four metallized welded steel plate girders on pile-supported integral abutments. The proposed bridge will be built on an off-alignment, 40 feet downstream of the existing bridge. During construction of the new bridge, one lane of alternating traffic will be maintained on the existing bridge using temporary traffic signals.

The new bridge alignment will require 970 feet of approach work. Matching into the grade at the western end of the project will require a raise in profile grade up to 8 feet. Matching into the grade at the east end of the project will require a raise in grade of 8 to 11 feet.

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2.0 GEOLOGIC SETTING

The existing structure carries Route 120 over West Branch Ellis River as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of Maine (1985), indicates the surficial soils in the vicinity of the bridge project consist of glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay and stones, and may include many boulders.

Based upon extrapolation of surficial geologic mapping conveyed in the Surficial Geology Map of the East Andover Quadrangle, MGS Open File 17-7, 2017, the site is located within a stream alluvium deposit associated with the West Branch Ellis River. This unit consists of sand, gravel, silt and organic sediment deposited along streams.

The MGS Bedrock Map of Maine (1985), indicates that the bedrock at the project site is an intrusion of Devonian muscovite-biotite granite and tonalite. Bedrock cores retrieved at the site are identified as tonalite.

3.0 SUBSURFACE INVESTIGATION

Two test borings were drilled at the site: BB-AWBER-101 and BB-AWBER-102. BB-

AWBER-101 was drilled at proposed Abutment No. 1 and BB-AWBER-102 was drilled at proposed Abutment No. 2. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The test borings were drilled in December 2017 by New England Boring Contractors (NEBC) of Hermon, Maine. 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 3 – 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 in 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 NEBC drill rig is equipped with a 140-pound, rope and cathead hammer falling 30 inches. No correction of N-values is required for the N-values obtained with the standard rope and cathead system where common practice assumes rope and cathead systems have a theoretical 60 percent hammer efficiency. The theoretical hammer efficiency factor, the N-values, and the N_{60} values 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. A consultant inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring location and drilling methods, designated type and depth of sampling techniques, reviewed boring log and identified field testing requirements. The borings were surveyed in the field 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: six standard grain size analyses with natural water contents. The results of soil tests are included as Appendix B – 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 stream alluvium underlain by bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Stream Alluvium

A layer of stream alluvium was encountered in the borings. The deposit was approximately 21 to 29 feet thick and generally consisted of:

- Brown or brown-grey, damp to moist, fine to medium sand, trace coarse sand, trace to little gravel, little silt;
- Brown, sandy gravel or gravelly sand, trace silt;
- Brown, damp, fine to medium sand, trace coarse sand, trace to little silt, trace gravel;
- Brown, sand, some gravel, trace silt;
- Brown, fine sandy silt, trace coarse to medium sand, trace gravel;
- Boulders.

A 4.2-foot tonalite boulder was cored in BB-AWBER-101 at Elev. 634.1 (20.4 feet below ground surface [bgs]).

SPT N_{60}-values in the alluvium deposit ranged from 2 to 29 blows per foot (bpf) indicating that the alluvium is very loose to medium dense in consistency. Six grain size analyses with water content tests were conducted on samples from the alluvium deposit resulting in the soils being classified as A-1-a, A-1-b, A-2-4, and A-4 under the AASHTO Classification System and as GW, SW, SW-SM, SM, and CL under the Unified Soil Classification System (USCS). The moisture contents of the samples tested ranged from approximately 9 to 21 percent.

5.2 Bedrock

Bedrock was encountered and cored in the borings. Table 1 summarizes approximate depth to bedrock, corresponding approximate top of the bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD R1, R2 (%)
BB-AWBER-101	15+15.2	11.0 R	29.0	625.5	63, 97
BB-AWBER-102	16+49.0	7.1 R	20.6	634.0	83, 77

Table 1 – Summary of Approximate Bedrock Depths and Elevations

The bedrock recovered from the boring is generally identified as black and white, fine to medium grained, very hard, typically fresh, tonalite, moderately dipping and low angle breaks, close to moderately close, open, occasional mud and sand infilling and open fracture zones. The RQD of the bedrock ranged from 63 to 97 percent correlating to a rock mass quality of fair to excellent. Detailed bedrock descriptions and the RQD of each core run are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.3 Groundwater

Groundwater was not observed in the boreholes. Note that water was introduced into the borehole during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The February 2018 Preliminary Design Report considered precast, prestressed concrete beams, and steel plate girders for superstructure options. Both superstructure options considered integral abutments on H-piles driven to bedrock.

Steel plate girders were determined to be cost effective and flexible in terms of structure depth, and were selected as the preferred alternative. The 130-foot single span bridge will have a girder depth that allows for a clearance of approximately 1-foot above the Q10 elevation, therefore the girders will be metalized and the structure designed to withstand stream and ice floe forces.

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 Merrill 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 resistances. Piles may be HP 12x53, 14x73, 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 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	Appromixate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation (feet)	Estimated Pile Lengths (feet)
Abutment No. 1	651.8	625.5	26.3
Abutment No. 2	654.6	634.0	20.6

Table 2 – Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths in Table 2 do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, 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.

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 7th Edition with interim revisions through 2016 (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_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 resistance against 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 ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). H-piles shall also be analyzed for fixity using LPile® v2016 (LPile) software, or similar.

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 four H-pile sections were calculated for the lower braced pile segment. The resistances shown in Table 3 are for the lower braced pile segment, using a resistance factor, $\phi_c = 0.50$ for severe driving conditions and an assumed effective length factor (K). Supporting calculations are provided in Appendix C – Calculations.

Factored structural resistances should be calculated for upper and lower unbraced segments based on LPile results using a resistance factor $\phi_c = 0.70$, for combined axial loading and bending. This is the responsibility of the structural engineer.

Geotechnical Resistance. The static geotechnical resistance of piles driven to hard rock was estimated using the Intact Rock Method (IRM).[1] The nominal axial geotechnical resistance in the strength limit state was also calculated using the guidance in LRFD Article 10.7.3.2.1 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, $\varphi_{dyn,}$ 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 four H-piles for the strength limit states are provided in Table 3. Supporting calculations are provided in Appendix C – Calculations.

[1] MaineDOT Transportation Research Division Technical Report 14-01, Sandford, January 2014, based on Rowe and Armitage (1987b) per NCHRP Synthesis 360, Turner, Rock-Socketed Shafts for Highway Structure Foundations, 2006.

Pile Section	Strength Limit State Factored Axial Pile Resistance				
	Structural Resistance[2] ϕ_c=0.50 (kips)	Static Geotechnical Resistance $\varphi_{static} = 0.45$[3] (kips)	Controlling Geotechnical Resistance[4] (kips)	Drivability Resistance[5] $\varphi_{dyn} = 0.65$ (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	387[6]	134	387	312	312
HP 14 x 73	535[6]	185	535	364	364
HP 14 x 89	652	226	652	468	468
HP 14 x 117	860	298	860	540(572)[7]	540(572)[7]

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, for the site conditions, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Local experience also supports the estimated factored resistances from the drivability analyses. 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 3. The maximum applied factored axial pile load should not exceed the governing factored axial pile resistance shown in Table 3 above.

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[2] Structural resistances were calculated for a braced pile segment in pure axial compression, using a resistance factor, ϕ_c, for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced pile segments based upon L-Pile results using a resistance factor of $\phi_c = 0.70$ for combined axial loading and bending. These resistances may be the controlling values.

[3] Static geotechnical resistance was estimated using the Intact Rock Method (IRM) proposed by Sandford, MaineDOT Transportation Research Division Technical Report 14-01, January 2014.

[4] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.*

[5] 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.

[6] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions for slenderness. HP12x53 and 14x73 sections do not comply with LRFD slenderness requirements and generally should be avoided for simplified pile design methods, (ref: Integral Abutment Bridge Design Guidelines, VTrans Structures Section, 2008).

[7] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

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 four H-pile sections for the extreme and service limit states are provided in Table 4. Supporting documentation is provided in Appendix C – Calculations.

Pile Section		Service and Extreme Limit State Factored Axial Pile Resistance			
	Structural Resistance[8] ϕ_c=1.0 (kips)	Static Geotechnical Resistance [9] ϕ = 1.0 (kips)	Controlling Geotechnical Resistance[10] ϕ=1.0 (kips)	Drivability Resistance ϕ= 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	775 [11]	298	775	480	480
HP 14 x 73	1070[11]	412	1070	560	560
HP 14 x 89	1305	502	1305	720	720
HP 14 x 117	1720	662	1720	830(880)[12]	830(880)[12]

Table 4 – Factored Axial Compressive Resistances for H-Piles at Service and Extreme 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 structural 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 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

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[8] Structural resistances were calculated for a braced pile segment in pure axial compression. Factored structural resistances should be calculated for upper and lower unbraced pile segments upon L-Pile results. These resistances may be the controlling values.

[9] Static geotechnical resistance was estimated using the Intact Rock Method (IRM) per Sandford (2014).

[10] Nominal pile axial compressive resistance calculated per guidance in LRFD Article 10.7.3.2.3. *Piles Driven to Hard Rock.*

[11] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions for slenderness. HP12x53 and 14x73 sections do not comply with LRFD slenderness requirements and generally should be avoided for simplified pile design methods, (ref: Integral Abutment Bridge Design Guidelines, 2nd Ed., VTrans, 2008).

[12] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

Geotechnical parameters used for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Table 5. In general, the models developed should emulate the soil at the site by using the soil layers (referenced in Tables 5 and 6) 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/in^3 (lbs/ft^3)	Horizontal Soil Modulus,k_s (lb/in^3)	Internal Angle of Friction
Medium dense, SAND (Granular Borrow).	Finished Grade – 650.5	Above	.072 (125)	90	32°
Loose, SAND.	650.5 – 644.5	Above	.053 (92)	25	28°
Medium dense, sandy GRAVEL.	644.5 – 639.5	Below	.033 (57)	60	32°
Medium dense, SAND.	639.5 – 634.1	Below	.033 (57)	60	31°
Medium dense, gravelly SAND.	634.1 – 625.5	Below	.03 (51)	60	35°

Table 5 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 1

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	Horizontal Soil Modulus k_s (lb/in^3)	Internal Angle of Friction
Medium dense, SAND (Granular Borrow).	Finished Grade – 653.6	Above	.072 (125)	90	32°
Very loose, SAND.	653.6 – 644.6	Above	.053 (92)	25	28°
Medium dense, GRAVEL.	644.6 – 639.6	Below	.033 (57)	60	31°
Medium dense, SAND.	639.6 – 634.0	Below	.032 (55)	60	34°

Table 6 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2

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. Restrikes or 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.

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 abutment 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 7:

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

Table 7 – 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 facilitate drainage and 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, 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 the "butterfly" wingwalls shall at a minimum consider a load case where the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25 assuming small wingwall movements. See Appendix C – 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 C – Calculations for supporting documentation.

There are no bearing resistance considerations or special foundation supports needed for wingwalls that are cantilevered off the abutment.

7.4 Settlement

The approximately 21 to 29-foot stream alluvium deposit is loose to medium dense in consistency. These coarse-grained materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. An increase in vertical overburden pressure will result from embankment construction on the western and eastern approaches to the new bridge. The proposed grade is 2.5 to 11 feet higher than the existing grade. Elastic settlements resulting from this grade increase are anticipated to be on the order of 1.1 inch at the western approach and 1.3 inch at the eastern approach.

LRFD Article 3.11.8 states that piles can be subject to downdrag when settlement is 0.4 inches or greater. The elastic settlement is anticipated to occur 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 the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Andover has a design freezing index (DFI) of approximately 1800 F-degree days. The anticipated coarse-grained fill material was assigned a water content of 10%. These components correlate to a frost depth of 7.5 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Farmington, Maine has a DFI from the Modberg database of approximately 2023 F-degree days. Farmington was selected because it lies along the same isoline as Andover and Andover is not available in the Modberg database. A water content of 10% was used. These components correlate to a frost depth of approximately 7.7 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 7.5 feet for frost protection. See Appendix C – 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

The fine, sandy alluvial deposit encountered at the Merrill Bridge site is very erodible. The condition of the existing bridge channel protection is rated a ‘5’ corresponding to “Bank eroded; major damage.” Scour countermeasures have been installed to correct scour problems, according to the 2015 MaineDOT Bridge Inspection Report.

Grain size analyses were performed on samples from the stream alluvium deposit to generate grain size curves for determining parameters to be used in scour analyses. Three soil samples were judged to be similar in nature to the soils likely to be exposed to scour conditions. The recommended streambed grain size parameter was derived from the average D_{50} of these samples and is presented in Table 7, below.

Sample	Sample Depth (ft)	D_{50} (mm)
BB-AWBER-101 3D	10-12	5.1
BB-AWBER-102 4D	15-17	1.8
BB-AWBER-102 5D	20-20.6	.07
	Average	2.3

Table 7 – Average D_{50} of Representative Samples for Scour

The grain size curves are included in Appendix B – 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 a layer of plain 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 to 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 8.

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.082g
Acceleration Coefficient (A_S)	0.13g
S_{DS} (Period = 0.2 sec)	0.28g
S_{D1} (Period = 1.0 sec)	0.12g
Site Class	D
Seismic Zone	1

Table 8 – Seismic Design Parameters

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

See Appendix C – Calculations for supporting documentation.

7.8 Construction Considerations

Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments.

One 4.3-foot diameter boulder was encountered in the borings conducted for the proposed Merrill Bridge. The actual number of boulders present in the overburden is difficult to characterize with limited, widely-spaced borings. During the construction of Brickett Bridge in Andover (0.7 miles upstream) numerous large boulders were encountered in the overburden which had to be excavated to permit pile driving. Similar conditions may present at the Merrill Bridge project site. The contractor should assume for the purposes of bidding and construction that it is likely that cobbles and boulders are present at the proposed abutment locations. The cobbles and boulders will impact construction operations and may require excavation. These impacts include, but are not limited to, driving H-piles for abutment foundations, installation of sheet piles for cofferdams and installing pile driving templates. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, use of rock chisels, or downhole hammers. Alternative methods to clear obstructions may be used as approved by the Resident. Excavation by these methods, or proposed alternative methods, shall be incidental to related pay items. The potential for obstructions to slow construction activities should be considered by the contractor. Care should be taken to drive piles within allowable tolerances without damaging the H-piles.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application of the proposed replacement of Merrill Bridge in Andover, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

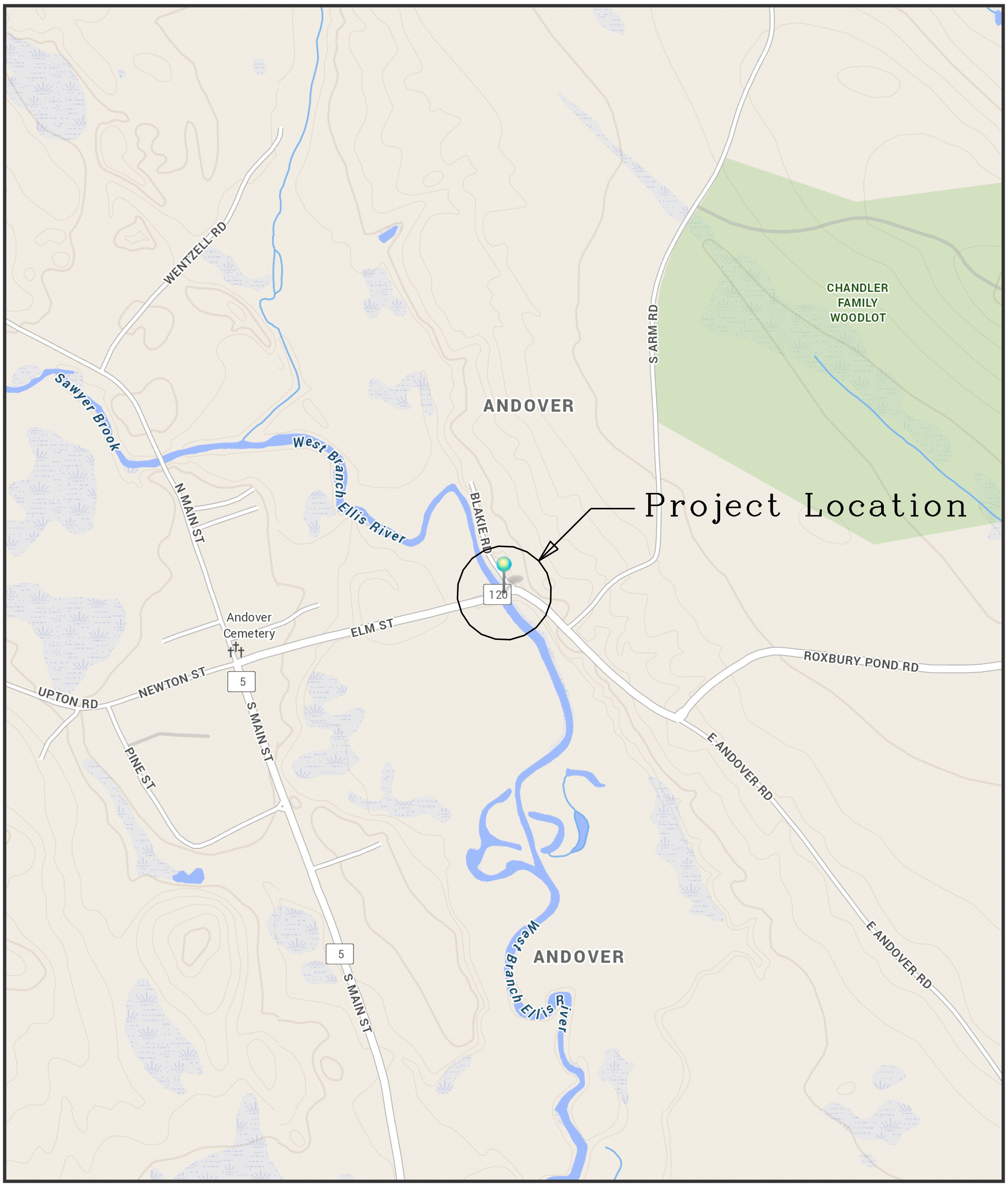
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon limited subsurface explorations 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 are properly interpreted and implemented in the design and specifications.

Sheets

Username: Terry.White Division: GEOTECH Date:5/21/2018

ANDOVER, MAINE

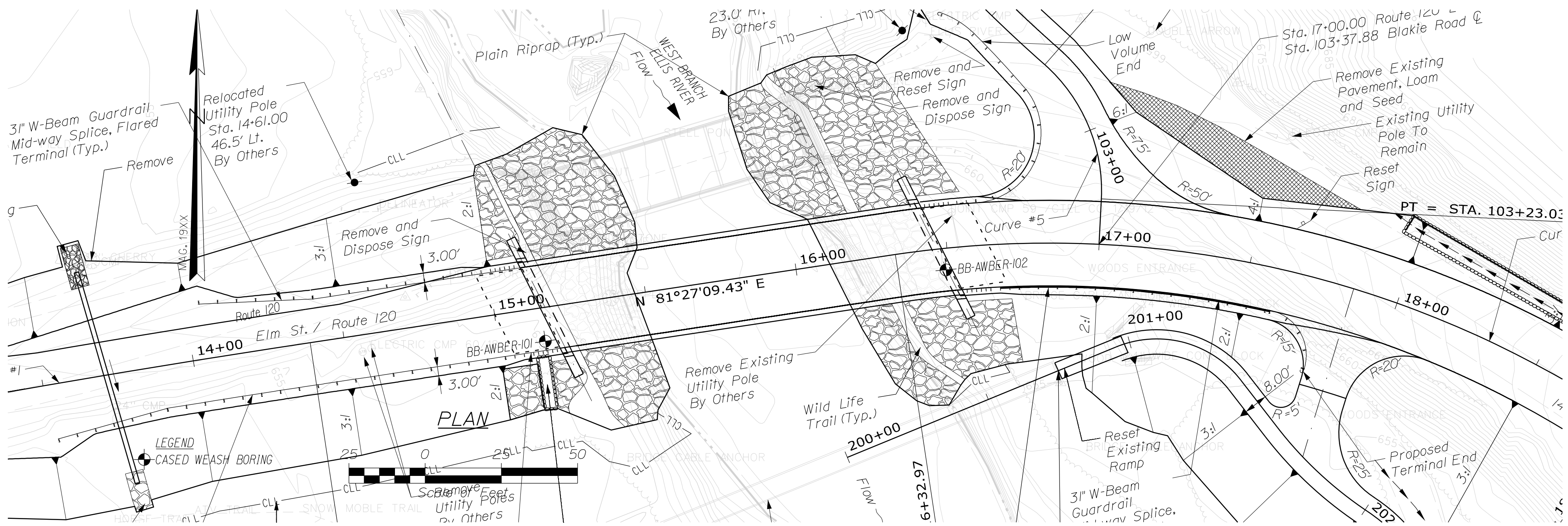


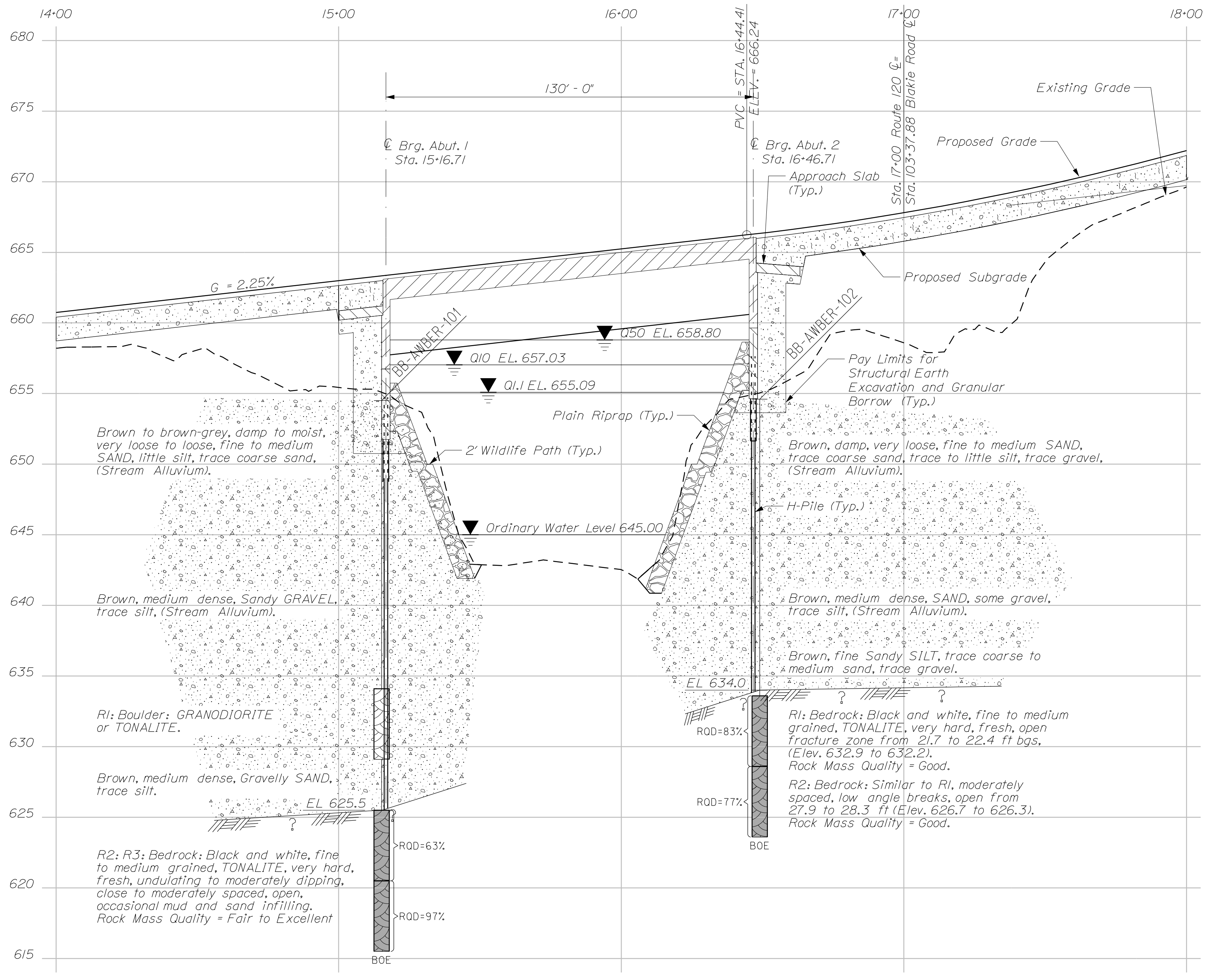
The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information.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

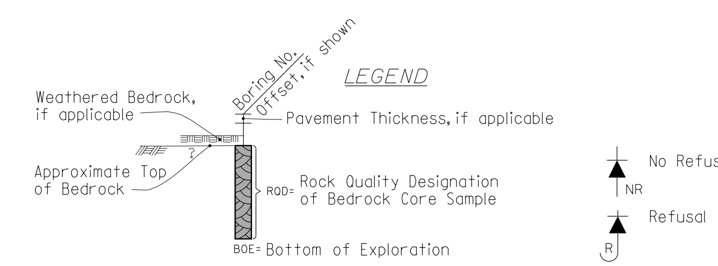
0.25 Miles
1 inch = 0.28 miles

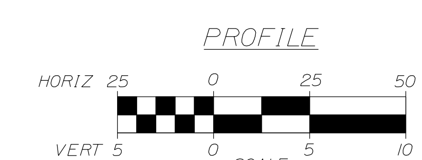
Date: 5/21/2018
Time: 10:54:45 AM

SHEET NUMBER	MERRILL BRIDGE WEST BRANCH ELLIS RIVER	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	ANDOVER OXFORD COUNTY	STP-2165(800)
OF 3	LOCATION MAP	WIN 21658.00 BRIDGE NO. 3215 BRIDGE PLANS









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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2165(800)

WIN 21658.00

BRIDGE PLANS

BRIDGE NO. 3215

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	A.VANBUSKIRK	
DESIGN3-DETAILED3	T.WHITE	MAY 2018
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MERRILL BRIDGE
WEST BRANCH ELLIS RIVER
ANDOVER OXFORD COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

Filename: ...\msta\007_BORING LOGS1.dgn   Division: GEOTECH   Username: Terry.White   Date:5/30/2018

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

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West	Location: Andover, Maine	Boring No.:	BB-AWBER-101
		WIN:	21658.00
Driller: New England Boring Contractors	Elevation (ft.) 654.5	Auger ID/OD:	5" Solid Stem
Operator: Schaefer/Royal	Datum: NAVD88	Sampler:	Standard Split Spoon
Logged By: Be Schonewald	Rig Type: Mobile Drill B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish: 12/14,19/2017	Drilling Method: Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location: 15+15.2, 11.0 ft Rt.	Casing ID/OD: HW & NW	Water Level*:	Dry, caved 10.6 ft bgs
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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			
	1D	24/3	2.00 - 4.00	1/1/1/2	2	2			Brown, damp, very loose, fine to medium SAND, little silt, trace coarse sand, (Alluvium).	
5	2D	24/14	5.00 - 7.00	2/3/3/7	6	6	HYD PUSH		Brown-grey, moist, loose, fine to medium SAND, little gravel, little silt, trace coarse sand, (Alluvium).	G#303057 A-2-4, SM WC=10.5%
							31			
							28			
							35			
10	3D	24/8	10.00 - 12.00	11/9/9/8	18	18	21		Brown, medium dense, Sandy GRAVEL, trace silt.	G#303058 A-1-a, GW WC=14.3%
							21			
							15			
							11			
							10			
15	4D	24/6	15.00 - 17.00	9/8/5/5	13	13	17		Brown, medium dense, Sandy GRAVEL, trace silt.	
							17			
							17			
							19			
							25			
20	MD R1	1.2/0 60/51	20.00 - 20.10 20.40 - 25.40	50(1.2") RQD = ---%	---		NQ-2	634.1	Failed sample attempt, granite (pink feldspar-rich) chips in spoon. HW Casing Refusal, roller cone Refusal, set in NW Casing at 20.2 ft bgs. 20.4 Pop through Boulder at 24.7 ft bgs. Boulder consists of Granodiorite or Tonalite.	
25	5D	24/17	25.40 - 27.40	24/17/12/13	29	29		629.1	25.4 Brown, medium dense, Gravelly SAND, trace silt. Pulled NW Casing, roller coned through boulder, spin shoe on NW Casing and reinstall at 27.4 ft bgs.	G#303059 A-1-b, SW-SM WC=9.3%
30	R2	60/56	29.00 - 34.00	RQD = 63%			NQ-2	625.5	29.0 Top of Bedrock at Elev. 625.5 ft. R2: Bedrock: Very hard, typically fresh, fine to medium grained, black and white TONALITE, with two 1/2-inch to 1-inch fine grained layers. Close to moderately spaced, moderately dipping and low angle breaks; undulating, rough, fresh to discolored, and open, with occasional mud and sand infilling. Open fractures: 29.8 to 29.9, 30.7 to 30.8 and 31.0 to 31.5 ft bgs. Larger open fracture contains rounded gravel pieces of rock. Rock Mass Quality = Fair R2: Core Times (min:sec) 29.0-30.0 ft (1:20) 30.0-31.0 ft (1:10) 31.0-32.0 ft (1:00) 32.0-33.0 ft (1:30) 33.0-34.0 ft (1:30) 93% Recovery	
35	R3	60/58	34.00 - 39.00	RQD = 97%				620.5	34.0 R3: Bedrock: Similar to R2, except one 2.5-inch thick, moderately dipping, fine grained seam. Rock Mass Quality = Excellent R3: Core Times (min:sec) 34.0-35.0 ft (1:25) 35.0-36.0 ft (1:25) 36.0-37.0 ft (1:25) 37.0-38.0 ft (1:25) 38.0-39.0 ft (1:30) 97% Recovery	
40								615.5	39.0 Bottom of Exploration at 39.0 feet below ground surface.	
45										
50										

Remarks:

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

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

Page 1 of 1
Boring No.: BB-AWBER-101

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

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West	Location: Andover, Maine	Boring No.:	BB-AWBER-102
		WIN:	21658.00
Driller: New England Boring Contractors	Elevation (ft.) 654.6	Auger ID/OD:	5" Solid Stem
Operator: Schaefer/Royal	Datum: NAVD88	Sampler:	Standard Split Spoon
Logged By: Be Schonewald	Rig Type: Mobile Drill B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish: 12/19-20/2017	Drilling Method: Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location: 16+49, 7.1 ft Rt.	Casing ID/OD: HW & NW	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
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			
	1D	24/16	2.00 - 4.00	1/2/1/1	3	3			Brown, damp, very loose, fine to medium SAND, trace to little silt.	
5	2D	24/8	5.00 - 7.00	1/1/2/2	3	3	HYD PUSH		Brown, damp, very loose, fine to medium SAND, little silt, trace gravel, trace coarse sand.	G#303060 A-2-4, SM WC=14.7%
							17			
							21			
10	3D	24/4	10.00 - 12.00	10/6/6/5	12	12	21		Brown, medium dense, GRAVEL, little sand, trace silt, (large piece of gravel wedged in spoon, not representative sample).	
							20			
							42			
							47			
							42			
15	4D	24/9	15.00 - 17.00	8/11/14/12	25	25	32		Brown, medium dense, SAND, some gravel, trace silt.	G#303061 A-1-b, SW WC=12.6%
							42			
							98			
							65			
							42			
20	5D	7.2/7.2	20.00 - 20.60	6/50(1.2")	---			634.0	Brown, fine Sandy SILT, trace coarse to medium sand, trace gravel. 20.6	G#303062 A-4, CL WC=20.5%
	R1	60/60	21.00 - 26.00	RQD = 83%			NQ-2	633.6	Top of Bedrock at Elev. 634.0 ft. Roller coned ahead to 21.0 ft bgs. 21.0 R1: Bedrock: Very hard, fresh, fine to medium grained, black and white TONALITE with quartz-rich veins and flow structure. Open fracture zone from 21.7 to 22.4 ft, otherwise core is single piece. Rock Mass Quality = Good R1: Core Times (min:sec) 21.0-22.0 ft (1:30) 22.0-23.0 ft (1:25) 23.0-24.0 ft (1:40) 24.0-25.0 ft (1:55) 25.0-26.0 ft (1:40) 100% Recovery	
25	R2	60/56	26.00 - 31.00	RQD = 77%				628.6	26.0 R2: Bedrock: Similar to R1, with feldspar-rich calesilicate layer from 27.1 to 27.9 ft. Typically moderately spaced, low angle breaks; undulating, rough, discolored and open. Open fracture from 27.9 to 28.3 ft. Rock Mass Quality = Good. R2: Core Times (min:sec) 26.0-27.0 ft (1:20) 27.0-28.0 ft (1:15) 28.0-29.0 ft (1:05) 29.0-30.0 ft (1:20) 30.0-31.0 ft (1:25) 93% Recovery	
30								623.6	31.0 Bottom of Exploration at 31.0 feet below ground surface.	
35										
40										
45										
50										

Remarks:

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

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

Page 1 of 1
Boring No.: BB-AWBER-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2165(800)
BRIDGE PLANS
WIN 21658.00
BRIDGE NO. 3215

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	VANBUSKIRK	
DESIGN3-DETAILED3	T.WHITE	MAY 2018
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MERRILL BRIDGE
WEST BRANCH ELLIS RIVER
ANDOVER   OXFORD COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations (in this order, if applicable):

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

$$RQD\ (\%) = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: 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))

Sample Container Labeling Requirements:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River. **Location:** Andover, Maine	**Boring No.:** BB-AWBER-101 **WIN:** 21658.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	654.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/14,19/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+15.2, 11.0 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Dry, caved 10.6 ft bgs
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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA				
	1D	24/3	2.00 - 4.00	1/1/1/2	2	2				Brown, damp, very loose, fine to medium SAND, little silt, trace coarse sand, (Alluvium).	
5	2D	24/14	5.00 - 7.00	2/3/3/7	6	6	HYD PUSH			Brown-grey, moist, loose, fine to medium SAND, little gravel, little silt, trace coarse sand, (Alluvium).	G#303057 A-2-4, SM WC=10.5%
							31				
							28				
							35				
10	3D	24/8	10.00 - 12.00	11/9/9/8	18	18	21			Brown, medium dense, Sandy GRAVEL, trace silt.	G#303058 A-1-a, GW WC=14.3%
							21				
							15				
							11				
							10				
15	4D	24/6	15.00 - 17.00	9/8/5/5	13	13	17			Brown, medium dense, Sandy GRAVEL, trace silt.	
							17				
							17				
							19				
							25				
20	MD R1	1.2/0 60/51	20.00 - 20.10 20.40 - 25.40	50(1.2") RQD = ---%	---		NQ-2	634.1		Failed sample attempt, granite (pink feldspar-rich) chips in spoon. HW Casing Refusal, roller cone Refusal, set in NW Casing at 20.2 ft bgs. 20.4	
25											

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River. Location: Andover, Maine	Boring No.: BB-AWBER-101 WIN: 21658.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	654.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/14,19/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+15.2, 11.0 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Dry, caved 10.6 ft bgs
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/17	25.40 - 27.40	24/17/12/13	29	29		629.1		Pop through Boulder at 24.7 ft bgs. Boulder consists of Granodiorite or Tonalite. 25.4 Brown, medium dense, Gravelly SAND, trace silt. Pulled NW Casing, roller coned through boulder, spin shoe on NW Casing and reinstall at 27.4 ft bgs.	G#303059 A-1-b, SW-SM WC=9.3%
								625.5		29.0	
30	R2	60/56	29.00 - 34.00	RQD = 63%			NQ-2			Top of Bedrock at Elev. 625.5 ft. R2: Bedrock: Very hard, typically fresh, fine to medium grained, black and white TONALITE, with two 1/2-inch to 1-inch fine grained layers. Close to moderately spaced, moderately dipping and low angle breaks; undulating, rough, fresh to discolored, and open, with occasional mud and sand infilling. Open fractures: 29.8 to 29.9, 30.7 to 30.8 and 31.0 to 31.5 ft bgs. Larger open fracture contains rounded gravel pieces of rock. Rock Mass Quality = Fair R2: Core Times (min:sec) 29.0-30.0 ft (1:20) 30.0-31.0 ft (1:10) 31.0-32.0 ft (1:00) 32.0-33.0 ft (1:30) 33.0-34.0 ft (1:30) 93% Recovery	
35	R3	60/58	34.00 - 39.00	RQD = 97%						R3: Bedrock: Similar to R2, except one 2½-inch thick, moderately dipping, fine grained seam. Rock Mass Quality = Excellent R3: Core Times (min:sec) 34.0-35.0 ft (1:25) 35.0-36.0 ft (1:25) 36.0-37.0 ft (1:25) 37.0-38.0 ft (1:25) 38.0-39.0 ft (1:30) 97% Recovery	
								615.5		39.0 **Bottom of Exploration at 39.0 feet below ground surface.**	
40											
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River.
Location: Andover, Maine

Boring No.: BB-AWBER-102

WIN: 21658.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	654.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/19-20/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+49, 7.1 ft Rt.	**Casing ID/OD:**	HW & NW	**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
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA				
	1D	24/16	2.00 - 4.00	1/2/1/1	3	3				Brown, damp, very loose, fine to medium SAND, trace to little silt.	
5	2D	24/8	5.00 - 7.00	1/1/2/2	3	3	HYD PUSH			Brown, damp, very loose, fine to medium SAND, little silt, trace gravel, trace coarse sand.	G#303060 A-2-4, SM WC=14.7%
							17				
							21				
10	3D	24/4	10.00 - 12.00	10/6/6/5	12	12	21			Brown, medium dense, GRAVEL, little sand, trace silt, (large piece of gravel wedged in spoon, not representative sample).	
							20				
							42				
							47				
							42				
15	4D	24/9	15.00 - 17.00	8/11/14/12	25	25	32			Brown, medium dense, SAND, some gravel, trace silt.	G#303061 A-1-b, SW WC=12.6%
							42				
							98				
							65				
							42				
20	5D	7.2/7.2	20.00 - 20.60	6/50(1.2")	---			634.0		Brown, fine Sandy SILT, trace coarse to medium sand, trace gravel. 20.6	G#303062 A-4, CL WC=20.5%
	R1	60/60	21.00 - 26.00	RQD = 83%			NQ-2	633.6		Top of Bedrock at Elev. 634.0 ft. Roller coned ahead to 21.0 ft bgs. 21.0	
										R1: Bedrock: Very hard, fresh, fine to medium grained, black and white TONALITE with quartz-rich veins and flow structure. Open fracture zone from 21.7 to 22.4 ft, otherwise core is single piece. Rock Mass Quality = Good	
										R1: Core Times (min:sec) 21.0-22.0 ft (1:30)	
25										22.0-23.0 ft (1:25)	

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.

Boring No.: BB-AWBER-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River.
Location: Andover, Maine

Boring No.: BB-AWBER-102

WIN: 21658.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	654.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/19-20/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+49, 7.1 ft Rt.	**Casing ID/OD:**	HW & NW	**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
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SSA = Solid Stem Auger
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										23.0-24.0 ft (1:40) 24.0-25.0 ft (1:55) 25.0-26.0 ft (1:40) 100% Recovery	
	R2	60/56	26.00 - 31.00	RQD = 77%						R2: Bedrock: Similar to R1, with feldspar-rich calesilicate layer from 27.1 to 27.9 ft. Typically moderately spaced, low angle breaks; undulating, rough, discolored and open. Open fracture from 27.9 to 28.3 ft. Rock Mass Quality = Good. R2: Core Times (min:sec) 26.0-27.0 ft (1:20) 27.0-28.0 ft (1:15) 28.0-29.0 ft (1:05)	
30								623.6		29.0-30.0 ft (1:20) 30.0-31.0 ft (1:25) 93% Recovery 31.0 **Bottom of Exploration at 31.0 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

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

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Andover   Work Number: 21658.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-AWBER-101, 2D	15+15.2	11.0 Rt.	5.0-7.0	303057	1	10.5			SM	A-2-4	0
BB-AWBER-101, 3D	15+15.2	11.0 Rt.	10.0-12.0	303058	1	14.3			GW	A-1-a	0
BB-AWBER-101, 5D	15+15.2	11.0 Rt.	25.4-27.4	303059	1	9.3			SW-SM	A-1-b	0
BB-AWBER-102, 2D	16+49	7.1 Rt.	5.0-7.0	303060	1	14.7			SM	A-2-4	0
BB-AWBER-102, 4D	16+49	7.1 Rt.	15.0-17.0	303061	1	12.6			SW	A-1-b	0
BB-AWBER-102, 5D	16+49	7.1 Rt.	20.0-20.6	303062	1	20.5			CL	A-4	IV

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

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

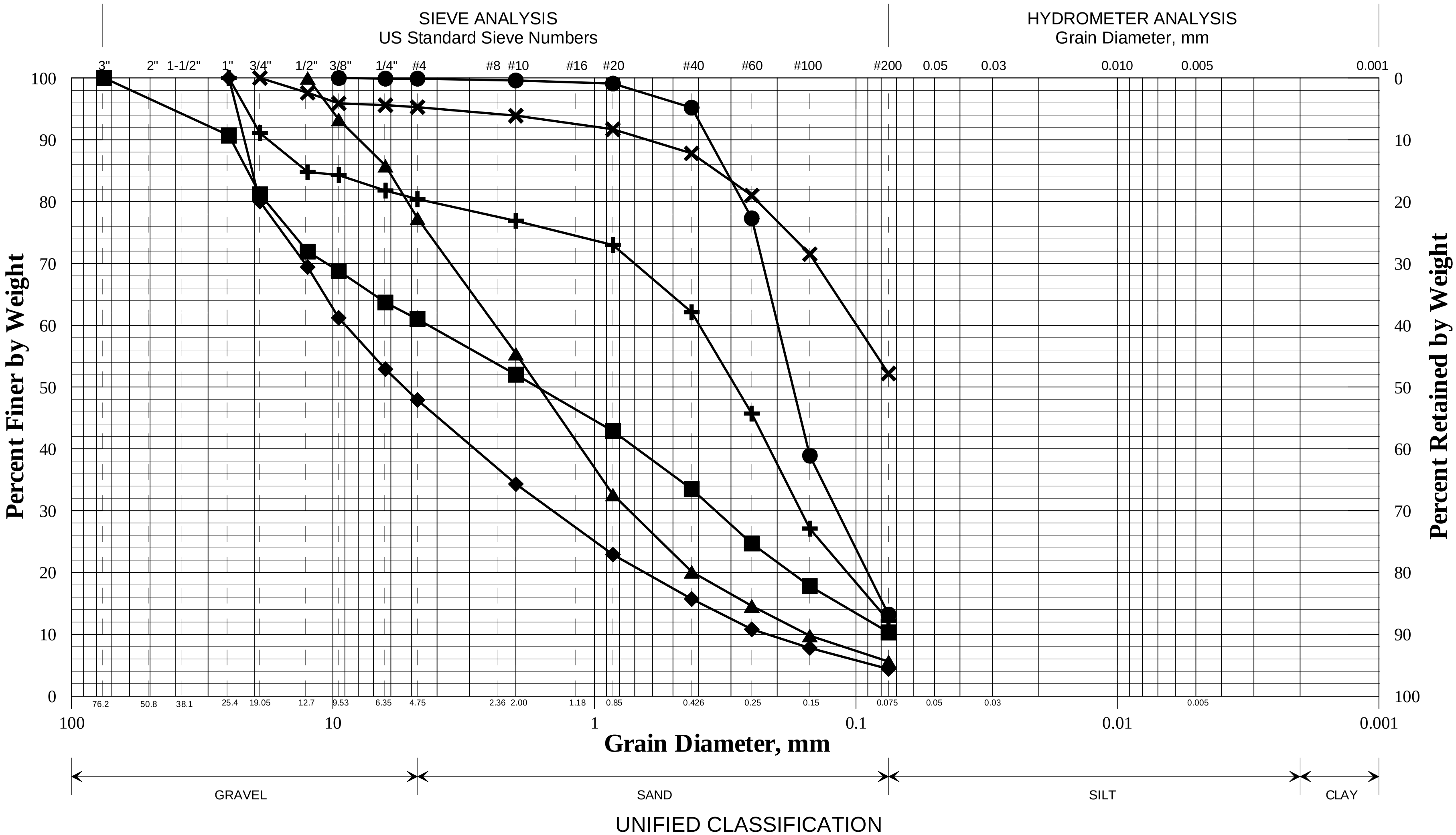
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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-AWBER-101/2D	15+15.2	11.0 RT	5.0-7.0	SAND, little gravel, little silt.	10.5			
◆	BB-AWBER-101/3D	15+15.2	11.0 RT	10.0-12.0	Sandy GRAVEL, trace silt.	14.3			
■	BB-AWBER-101/5D	15+15.2	11.0 RT	25.4-27.4	Gravelly SAND, trace silt.	9.3			
●	BB-AWBER-102/2D	16+49	7.1 RT	5.0-7.0	SAND, little silt, trace gravel.	14.7			
▲	BB-AWBER-102/4D	16+49	7.1 RT	15.0-17.0	SAND, some gravel, trace silt.	12.6			
✖	BB-AWBER-102/5D	16+49	7.1 RT	20.0-20.6	Sandy SILT, trace gravel.	20.5			

WIN	
021658.00	
Town	
Andover	
Reported by/Date	
WHITE, TERRY A	1/17/2018

SHEET 1

<u>**Appendix C**</u>

Calculations

H-Pile Resistance

Intact Rock Method (Sandford [2014])

Input
Calculation
Output
Linked Cell

Constants

φ_static (tip)	0.45	LRFD Table 10.5.5.2.3-1 for CGS Method on Rock

Pile Dimensions

	Depth (d)	Flange Width (b)	A_{tip}
	(in.)	(in.)	in.^{2}
12x53	11.80	12.00	15.5
14x73	13.60	14.60	21.4
14x89	13.80	14.70	26.1
14x117	14.20	14.90	34.4

Notes:

Dimensions are manufacturer suppled from skylinesteel (see attached).

Compressive Strength of Bedrock Samples

Material	**Boring**	**Core Run**	**RQD**	**q_u (ksi)**	**Source of q_u**
tonalite	BB-AWBER-101	R2	63%	7.7	AASHTO 17th edition, 2002, Table 4.4.8.1.2B, see under quartzdiorite (comparable to tonalite).
	BB-AWBER-101	R3	97%	7.7	
	BB-AWBER-102	R1	83%	7.7	
	BB-AWBER-102	R2	77%	7.7	

Intact Rock Method

	depth (d)	flange width (b)	A_{tip}	q_{tip} = 2.5 * min(q_u)	R_{n_tip} = A_{tip}*q_{tip}	R_{r_tip} = ϕ_{static}*R_n
	in.	in.	in.^{2}	ksi	kips	kips
12x53	11.80	12.00	15.5	**19.3**	**298.4**	**134.3**
14x73	13.60	14.60	21.4	**19.3**	**412.0**	**185.4**
14x89	13.80	14.70	26.1	**19.3**	**502.4**	**226.1**
14x117	14.20	14.90	34.4	**19.3**	**662.2**	**298.0**

Note: For Intact Rock Method, see Sandford (2014) citing NCHRP Sythesis 360, which cites Rowe and Armitage (1987b).

Reference:

Sandford, Thomas, PhD, P.E. and Stuart, Cameron, E.I.T. MaineDOT Transportation Research Division Technical Report 14-01. January 2014.

Turner, John, NCHRP Synthesis 360, Rock-Socketed Shafts for Highway Structure Foundations, 2006.

Rowe, R.K. and H.H. Armitage. "A Design Method for Drilled Piers in Soft Rock." *Canadian Geotechnical Journal*, Vol. 24, 1987b, pp. 126-142.

For H-pile dimensions, see skylinesteel data sheet (attached).

AASHTO LRFD Bridge Design Specifications. 7th Edition. 2014, with interims through 2016.

Compressive Structural Strength of H-Pile (pure axial compression)

Constants			Comments
E	ksi	29,000	LRFD C4.6.2.5-1 (a)
K	-	0.65	
F_y	ksi	50	
l_unbraced	in.	1	nominal unbraced length

Legend
Input
Calculation
Output

	r (y-y)	A_s	
	in.	in.^{2}	Comments
12x53	2.86	15.5	ref: Skyline steel Data Sheet
14x73	3.49	21.4	
14x89	3.53	26.1	
14x117	3.59	34.4	

		Comments
Φ_c	0.50	LRFD 6.5.4.2: "For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary."
Φ_service	1.0	LRFD 1.3.2.1 "For service and extreme limit states resistance factors shall be taken as 1.0, except for . . . bolts . . . and concrete columns."
Φ_extreme	1.0	

	P_{e_braced}
	(kips)
12x53	8.59E+07
14x73	1.77E+08
14x89	2.20E+08
14x117	3.00E+08

$$Pe = \frac{\pi^2 E}{(\frac{Kl}{r})^2}\, x\, Ag$$ (LRFD Eq. 6.9.4.1.2-1)

	P_o
	(kips)
12x53	775
14x73	1070
14x89	1305
14x117	1720

$$P_o = QFyAg$$ (LRFD Article 6.9.4.1.1)

	P_{e_braced} / P_o	>= .44
12x53	110824	TRUE
14x73	165026	TRUE
14x89	168830	TRUE
14x117	174618	TRUE

$$If\ \frac{Pe}{Po} < .44, then\ Pn = .877Pe$$
(LRFD Eq. 6.9.4.1.1-2)

$$If\ \frac{Pe}{Po} \geq .44, then\ Pn = [.658^{(\frac{Po}{Pe})}]\ P_o$$
(LRFD Eq. 6.9.4.1.1-1)

	P_n_braced
	(kips)
12x53	775
14x73	1070
14x89	1305
14x117	1720

	$P_r = \Phi_{\_strength} P_n$	$P_r = \Phi_{\_service} P_n$	$P_r = \Phi_{\_extreme} P_n$
	(kips)	(kips)	(kips)
12x53	387	775	775
14x73	535	1070	1070
14x89	652	1305	1305
14x117	860	1720	1720

Input
Linked Cell
Output

Controlling Geotechnical Resistance

	Structural Resistance	ϕ_c	Controlling Geotechnical Resistance
	(kips)		(kips)
12x53	775	0.5	**387**
14x73	1070	0.5	**535**
14x89	1305	0.5	**652**
14x117	1720	0.5	**860**

Note: Based on guidance in LRFD Article 10.7.3.2.3., Piles Driven to Hard Rock. 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 resistance values obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c of 0.50 for severe driving conditions applied.

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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 drivablity analysis:

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

$\phi_{dyn} := 0.65$ 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

Based on Table 2 of this Report, estimated pile lengths will be approx. from 20 to 26 ft. Assume contractor drives pile lengths of 30 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use proportional shaft resistances so that GRLWeap will assign approx. 10% of the ultimate capacities as skin friction, based on local experience.

Maine DOT
Andover 21658, HP 12x53, D 19-42 set 4

02-Mar-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
200.0	26.53	0.34	3.0	5.63	10.90
300.0	33.05	2.23	4.9	6.25	11.76
400.0	39.42	3.13	7.7	6.93	12.99
470.0	43.77	4.19	11.2	7.35	13.70
480.0	44.36	4.33	11.8	7.42	13.85
490.0	44.91	4.46	12.4	7.48	13.97
500.0	45.48	4.58	13.2	7.55	14.10
600.0	50.70	5.35	23.3	8.33	15.65
650.0	52.69	5.57	33.7	8.66	16.23
700.0	54.20	5.59	56.1	8.90	16.66

Pile size = 12x53. Assume Contractor will use a Delmag D 19-42 hammer with 2.70 kip helmet on fuel setting 4. Limit driving stress to less than 45 ksi.

$R_{dr\_12x53} := 480\text{kip}$

Strength Limit State

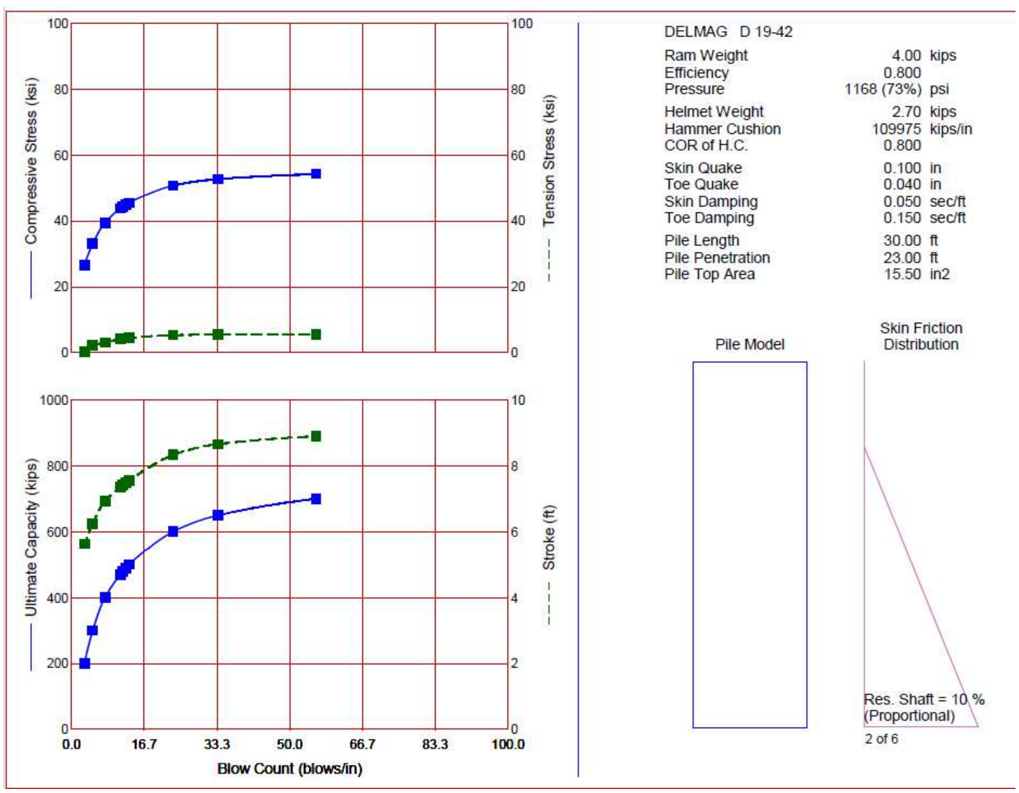
$\varphi_{dyn} := 0.65$

$R_{dr\_12x53\_strength} := R_{dr\_12x53} \cdot \varphi_{dyn} = 312 \cdot \text{kip}$

Service and Extreme Limit

$\varphi := 1.0$

$R_{dr\_12x53\_servext} := R_{dr\_12x53} \cdot \varphi = 480 \cdot \text{kip}$



Maine DOT
Andover 21658, HP 14x73, D 19-42 set 4

01-Mar-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
350.0	29.63	2.15	6.1	6.31	11.16
450.0	34.45	3.36	9.0	6.91	12.09
500.0	36.53	3.59	11.2	7.20	12.49
530.0	37.69	3.89	12.9	7.36	12.78
540.0	38.09	3.99	13.6	7.42	12.87
550.0	38.52	4.07	14.2	7.50	12.98
560.0	39.03	4.15	14.7	7.59	13.18
570.0	39.50	4.28	15.2	7.68	13.35
670.0	42.99	4.99	24.3	8.26	14.66
770.0	45.77	4.55	43.6	8.76	15.81

Pile size = 14x73. Assume Contractor will use a Delmag D 19-42 hammer with 2.70 kip helmet on fuel setting 4. Limit bpi to less than 15.

$R_{dr\_14x73} := 560\text{kip}$

Strength Limit State

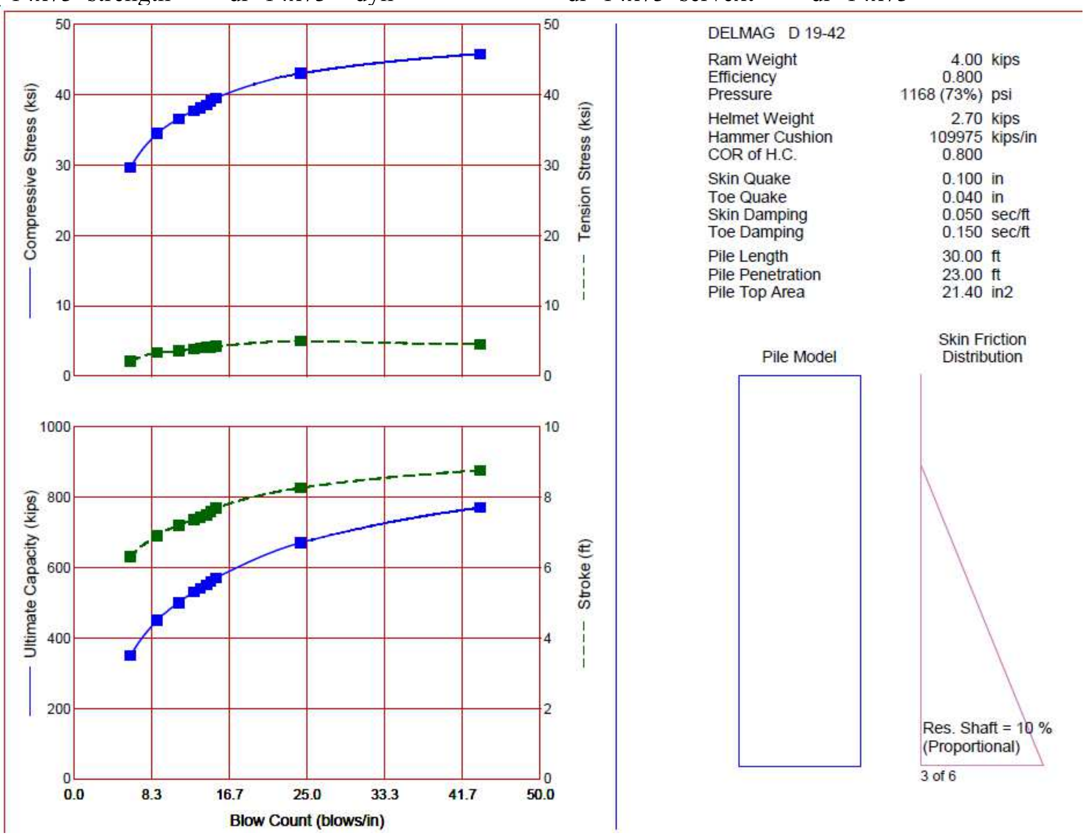
$\varphi_{dyn} := 0.65$

$R_{dr\_14x73\_strength} := R_{dr\_14x73} \cdot \varphi_{dyn} = 364 \cdot \text{kip}$

Service and Extreme Limit

$\varphi := 1.0$

$R_{dr\_14x73\_servext} := R_{dr\_14x73} \cdot \varphi = 560 \cdot \text{kip}$



Maine DOT
Andover 21658, HP 14x89, D 19-42 set 2

05-Mar-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	32.03	1.96	5.3	7.78	14.99
500.0	36.56	2.47	7.0	8.44	16.01
600.0	40.71	3.87	9.6	9.15	17.33
710.0	44.34	4.95	14.1	9.75	18.79
720.0	44.65	4.92	14.6	9.81	18.88
730.0	44.99	4.92	15.1	9.88	19.05
740.0	45.34	4.90	15.5	9.95	19.22
800.0	47.39	4.29	18.3	10.39	20.29
900.0	49.70	4.87	27.2	10.81	21.25
1000.0	51.16	5.15	46.2	11.02	21.74

Pile size = 14x89. Assume Contractor will use a Delmag D 19-42 hammer with 2.70 kip helmet on fuel setting 2. Limit bpi to less than 15.

$R_{dr\_14x89} := 720\text{kip}$

Strength Limit State

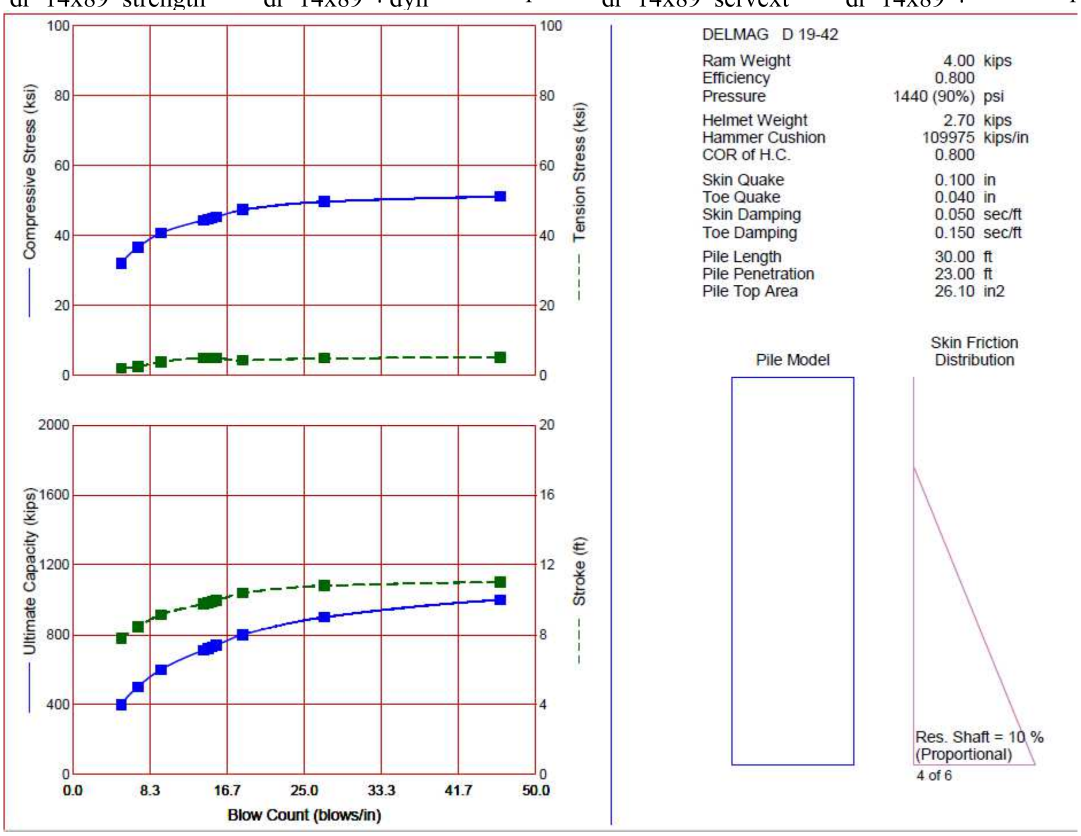
$\varphi_{dyn} := 0.65$

$R_{dr\_14x89\_strength} := R_{dr\_14x89} \cdot \varphi_{dyn} = 468 \cdot \text{kip}$

Service and Extreme Limit

$\varphi := 1.0$

$R_{dr\_14x89\_servext} := R_{dr\_14x89} \cdot \varphi = 720 \cdot \text{kip}$



Maine DOT
Andover 21658, HP 14x117, D 19-42 set 1

02-Mar-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
500.0	32.79	1.11	6.2	9.03	17.52
600.0	36.29	2.30	8.0	9.58	18.76
700.0	39.49	3.29	10.5	10.06	19.91
820.0	42.42	3.77	14.5	10.50	20.95
830.0	42.69	3.79	14.9	10.54	21.03
840.0	42.84	3.79	15.3	10.56	21.09
850.0	43.22	3.90	15.6	10.63	21.27
950.0	45.23	5.31	20.0	10.94	21.98
1050.0	46.87	5.57	26.2	11.19	22.58
1150.0	48.51	5.31	34.4	11.51	23.39

Pile size = 14x117. Assume Contractor will use a Delmag D 19-42 hammer with 2.70 kip helmet on fuel setting 1. Limit bpi to less than 15.

$R_{dr\_14x117} := 830kip$

Strength Limit State

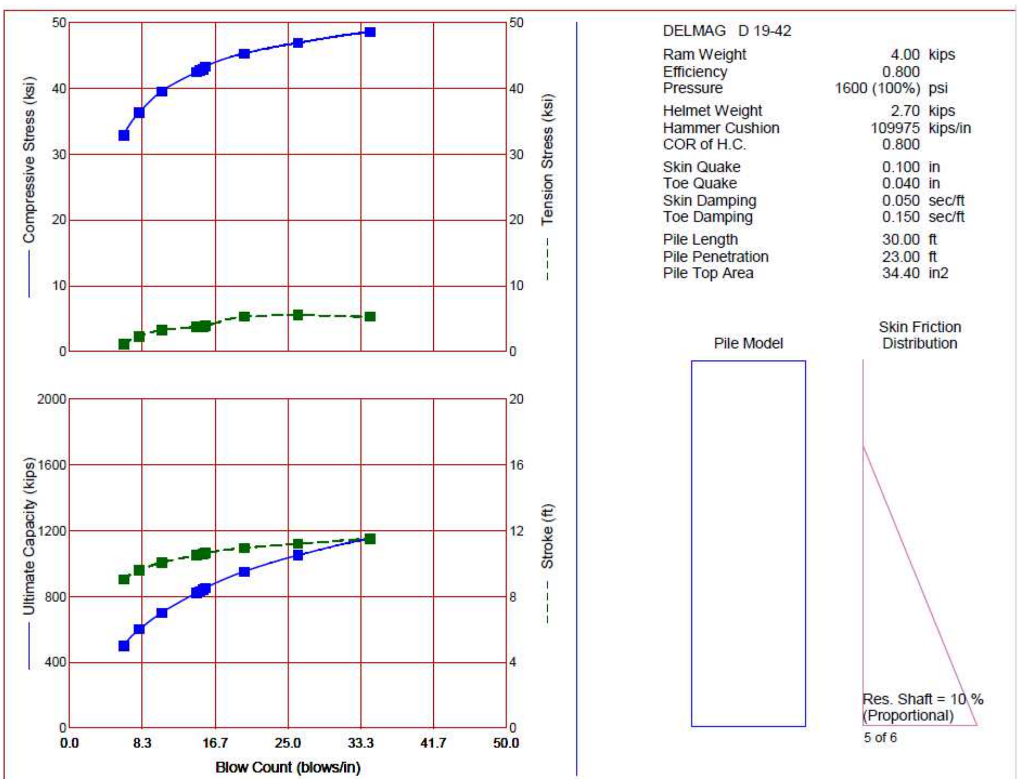
$\varphi_{dyn} := 0.65$

$R_{dr\_14x117\_strength} := R_{dr\_14x117} \cdot \varphi_{dyn} = 540 \cdot kip$

Service and Extreme Limit

$\varphi := 1.0$

$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \varphi = 830 \cdot kip$



Maine DOT
Andover 21658, HP 14x117, D 36-32 set 4

02-Mar-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	35.93	1.21	4.9	6.55	23.66
700.0	39.58	2.28	6.0	6.96	24.91
800.0	42.49	3.13	7.5	7.28	25.63
870.0	44.53	3.45	8.8	7.55	26.52
880.0	44.88	3.53	9.0	7.58	26.62
890.0	45.09	3.57	9.3	7.61	26.64
900.0	45.38	3.63	9.4	7.65	26.80
950.0	46.84	3.12	10.6	7.86	27.47
1000.0	48.24	2.78	11.8	8.08	28.30
1100.0	50.47	1.50	15.5	8.39	29.25

Pile size = 14x117. Assume Contractor will use a Delmag D 36-32 hammer with 3.20 kip helmet on fuel setting 4. Limit driving stress to less than 45 ksi.

$R_{dr\_14x117} := 880 kip$

Strength Limit State

$\varphi_{dyn} := 0.65$

$R_{dr\_14x117\_strength} := R_{dr\_14x117} \cdot \varphi_{dyn} = 572 \cdot kip$

Service and Extreme Limit

$\varphi := 1.0$

$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \varphi = 880 \cdot kip$

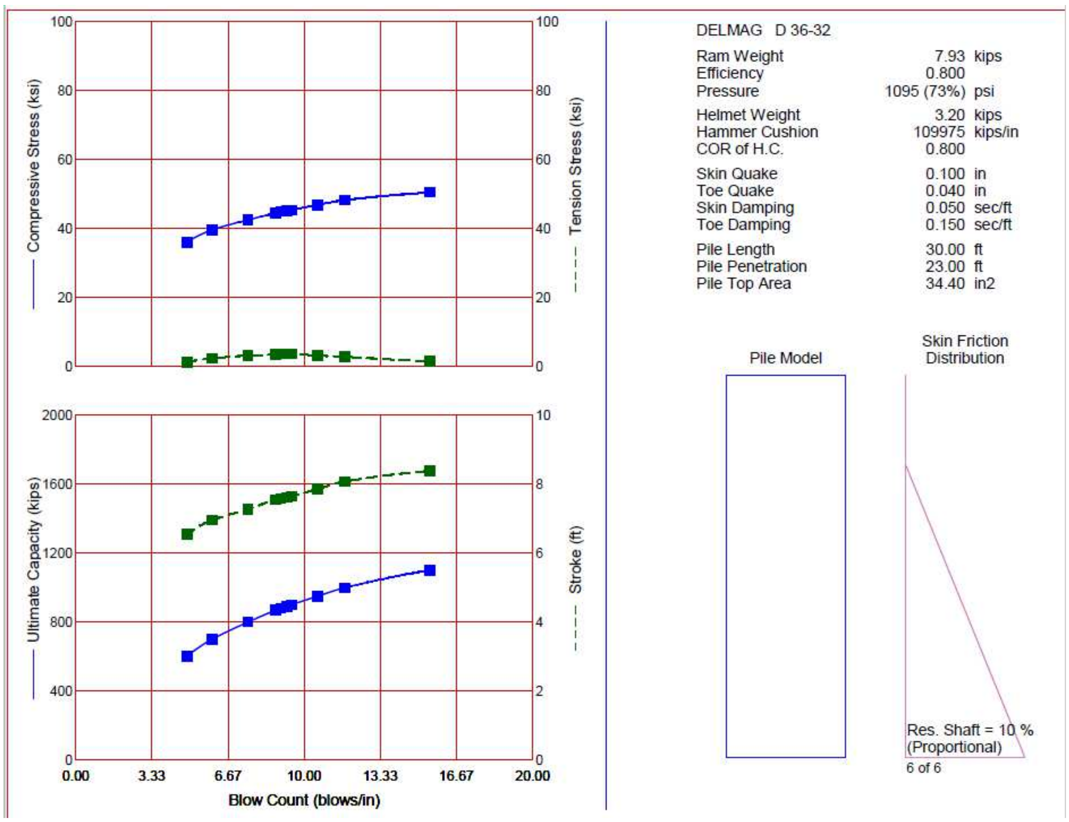


TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[1] (ksf)	C_o[1] (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.

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

Values of I_ρ 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 (υ) 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):

$$E_m = \alpha_E E_o \qquad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \qquad (4.4.8.2.2\text{-}4)$$

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

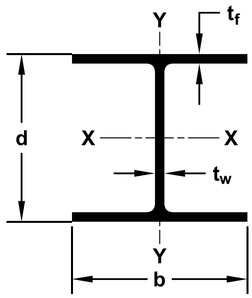
Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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

HP

Steel H-Pile



SECTION	Weight	Area	Depth d	Flange Width b	THICKNESS		Coating Area	PROPERTIES							
					Flange (t_f)	Web (t_w)		AXIS X-X				AXIS Y-Y			
								I	S	Z	r	I	S	Z	r
	lb/ft (kg/m)	in² (cm²)	in (mm)	in (mm)	in (mm)	in (mm)	ft²/ft (m²/m)	in⁴ (cm⁴)	in³ (cm³)	in³ (cm³)	in (cm)	in⁴ (cm⁴)	in³ (cm³)	in³ (cm³)	in (cm)
HP 8 **HP 200**	36 54	10.6 68.4	8.02 204	8.16 207	0.445 11.3	0.445 11.3	3.92 1.19	119 4953	29.8 488	33.6 550.6	3.36 8.53	40.3 1677	9.88 162	15.2 249.1	1.95 4.95
HP 10 **HP 250**	42 63	12.4 80.0	9.70 246	10.10 257	0.420 10.7	0.415 10.5	4.83 1.47	210 8741	43.4 711	48.3 791.5	4.13 10.5	71.7 2984	14.2 233	21.8 357.2	2.41 6.12
	57 85	16.7 108	9.99 254	10.20 259	0.565 14.4	0.565 14.4	4.91 1.50	294 12237	58.8 964	66.5 1089.7	4.18 10.6	101 4204	19.7 323	30.3 496.5	2.45 6.22
HP 12 **HP 310**	53 79	15.5 100	11.80 300	12.00 305	0.435 11.0	0.435 11.0	5.82 1.77	393 16358	66.7 1093	74.0 1212.6	5.03 12.8	127 5286	21.1 346	32.2 527.7	2.86 7.26
	63 94	18.4 119	11.90 302	12.10 307	0.515 13.1	0.515 13.1	5.86 1.79	472 19646	79.1 1296	88.3 1447.0	5.06 12.9	153 6368	25.3 415	38.7 634.2	2.88 7.32
	74 110	21.8 141	12.10 307	12.20 310	0.610 15.5	0.605 15.4	5.91 1.80	569 23683	93.8 1537	105 1720.6	5.11 13.0	186 7742	30.4 498	46.6 763.6	2.92 7.42
	84 125	24.6 159	12.30 312	12.30 312	0.685 17.4	0.685 17.4	5.97 1.82	650 27055	106 1737	120 1966.4	5.14 13.1	213 8866	34.6 567	53.2 871.8	2.94 7.47
	89 132	25.9 167	12.36 314	12.32 313	0.720 18.3	0.720 18.3	6.04 1.84	689 28700	111.6 1830	126.3 2070	5.16 13.1	225 9370	36.5 599	56.2 922	2.94 7.48
	102 152	29.9 193	12.56 319	12.64 321	0.819 20.8	0.819 20.8	6.17 1.88	811 33800	129.3 2120	147.6 2420	5.20 13.2	276 11500	43.7 716	67.1 1100	3.04 7.71
	117 174	34.4 222	12.76 324	12.87 327	0.929 23.6	0.929 23.6	6.26 1.91	946 39400	148.2 2430	170.8 2800	5.24 13.3	331 13800	51.4 843	79.3 1300	3.11 7.89
HP 14 **HP 360**	73 109	21.4 138	13.60 345	14.60 371	0.505 12.8	0.505 12.8	6.96 2.12	729 30343	107 1753	118 1933.7	5.84 14.8	261 10864	35.8 587	54.6 894.7	3.49 8.86
	89 132	26.1 168	13.80 351	14.70 373	0.615 15.6	0.615 15.6	7.02 2.14	904 37627	131 2147	146 2392.5	5.88 14.9	326 13569	44.3 726	67.7 1109.4	3.53 8.97
	102 152	30.1 194	14.00 356	14.80 376	0.705 17.9	0.705 17.9	7.06 2.15	1050 43704	150 2458	169 2769.4	5.92 15.0	380 15817	51.4 842	78.8 1291.3	3.56 9.04
	117 174	34.4 222	14.20 361	14.90 378	0.805 20.4	0.805 20.4	7.12 2.34	1220 50780	172 2819	194 3179.1	5.96 15.1	443 18439	59.5 975	91.4 1497.8	3.59 9.12
HP 16 **HP 410**	88 131	25.8 167	15.30 389	15.70 399	0.540 13.7	0.540 13.7	7.52 2.29	1110 46201	145 2376	161 2638.3	6.56 16.7	349 14526	44.5 729	68.2 1117.6	3.68 9.35
	101 150	29.9 193	15.50 394	15.80 401	0.625 15.9	0.625 15.9	7.56 2.30	1300 54110	168 2753	187 3064.4	6.59 16.7	412 17149	52.2 855	80.1 1312.6	3.71 9.42
	121 180	35.8 231	15.80 401	15.90 404	0.750 19.1	0.750 19.1	7.62 2.32	1590 66180	201 3294	226 3703.5	6.66 16.9	504 20978	63.4 1039	97.6 1599.4	3.75 9.53
	141 210	41.7 269	16.00 406	16.00 406	0.875 22.2	0.875 22.2	7.69 2.34	1870 77835	234 3835	264 4326.2	6.70 17.0	599 24932	74.9 1227	116 1900.9	3.79 9.63
	162 241	47.7 308	16.30 414	16.10 409	1.000 25.4	1.000 25.4	7.75 2.36	2190 91154	269 4408	306 5014.4	6.78 17.2	697 29011	86.6 1419	134 2195.9	3.82 9.70
	183 272	54.1 349	16.50 419	16.30 414	1.130 28.7	1.130 28.7	7.81 2.38	2510 104473	304 4982	349 5719.1	6.81 17.3	818 34047	100.0 1639	156 2556.4	3.89 9.88
HP 18 **HP 460**	135 201	39.9 257	17.50 445	17.80 452	0.750 19.1	0.750 19.1	8.54 2.60	2200 91570	251 4113	281 4604.7	7.43 18.9	706 29386	79.3 1299	122 1999.2	4.21 10.7
	157 234	46.2 298	17.70 450	17.90 455	0.870 22.1	0.870 22.1	8.60 2.62	2570 106971	290 4752	327 5358.5	7.46 18.9	833 34672	93.1 1526	143 2343.3	4.25 10.8
	181 269	53.2 343	18.00 457	18.00 457	1.000 25.4	1.000 25.4	8.66 2.64	3020 125701	336 5506	379 6210.7	7.53 19.1	974 40541	108.0 1770	167 2736.6	4.28 10.9
	204 304	60.2 388	18.30 465	18.10 460	1.130 28.7	1.130 28.7	8.73 2.66	3480 144847	380 6227	433 7095.6	7.60 19.3	1120 46618	124.0 2032	191 3129.9	4.31 11.0

LPile Parameters

Development of soil model for LPile

OBJECTIVE
Estimate soil parameters for lateral pile analyses at Abutment No. 1 and 2.

Given:
1) 100-series boring logs and lab data.

Assumptions:
1) Groundwater was not observed during the investigation; therefore, assume the groundwater table is at Elevation 645, at the stream elevation.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 will be used for integral abutment backfill.
3) Piles shall be driven to, or within, bedrock.

Abutment No. 1 Soil Model

1) The design soil layers are delineated as depicted on the attached annotated boring logs, which indicate the top and bottom elevations of the soil layers based on differing engineering properties.

Soil Layer No. 1 (Granular Borrow for Underwater Backfill) El. Finished Grade - 650.5

Internal Angle of Friction $\boxed{\phi_1 := 32\,deg}$

Soil Dry Unit Weight $\boxed{\gamma_{1dry} := 125pcf}$

MaineDOT BDG Table 3-3

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand above water table for static loading = 90 pci

Technical Manual
LPile 2016
p. 96

Soil Layer No. 2 (Unsubmerged, Loose, Sand) El. 650.5 - 644.5.

$N_{60} := 6\ bpf$

Internal Angle of Friction $\boxed{\phi_2 := 28}$

Lambe and Whitman, Soil Mechanics, 1969, Fig. 11.14.

Dry Unit Weight

Loose, uniform sand: 92 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2, Loose uniform sand.

$\gamma_{2dry} := 92pcf$

Representative constant giving the variation of soil modulus with depth, k:
Loose sand above water table for static loading = 25 pci

Technical Manual
LPile 2016
p. 96

Soil Layer No. 3 (Submerged, Medium Dense, Sandy Gravel) El. 644.5 - 639.5

N_{60} = 18 bpf

Internal Angle of Friction

$\boxed{\phi_3 := 32}$

Lambe and Whitman, Soil Mechanics, 1969, Fig. 11.14.

Dry Unit Weight

Medium dense, sandy gravel = 104 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - dry unit weight, average of loose and dense uniform sand

$\gamma_{3dry} := 104pcf$

Saturated Unit Weight

Natural water content at saturated state: 14.3% (BB-AWBER-101 3D)

$w_{3sat} := 14.3\%$

$\gamma_{3saturated} := \gamma_{3dry} \cdot (1 + w_{3sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{3saturated} = 118.9 \cdot pcf$

Soil Effective Unit Weight

$\gamma_{water} := 62.4pcf$

$\gamma'_3 := \gamma_{3saturated} - \gamma_{water}$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 15 Eq (2-11).

$\boxed{\gamma'_3 = 56.5 \cdot pcf}$

Representative constant giving the variation of soil modulus with depth, k: medium dense sand below water table for static loading = 60 pci

Technical Manual LPile 2016 p. 96

Soil Layer No. 4 (Submerged, Medium Dense, Sand) El. 639.5 - 634.10

Design N_{60} = 13

Internal Angle of Friction $\boxed{\phi_1 := 31\,deg}$

Lambe and Whitman, Soil Mechanics, 1969, Fig. 11.14.

Soil Dry Unit Weight $\boxed{\gamma_{4dry} := 104pcf}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships, average of loose and dense sand

Saturated Unit Weight

Natural Water Content in a Saturated State: 14.3% (BB-AWBER-101 3D).

$w_{4sat} := 14.3\%$

$\gamma_{4sat} := \gamma_{4dry} \cdot (1 + w_{4sat})$

$\gamma_{4sat} = 118.9 \cdot pcf$

Effective Unit Weight

$\gamma'_4 := \gamma_{4sat} - \gamma_{water}$

$\boxed{\gamma'_4 = 56.5 \cdot pcf}$

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand below water table for static loading = 60 pci

Technical Manual
LPile 2016
p. 96

Soil Layer No. 5 (Submerged, Medium Dense, Gravelly Sand potentially with boulders) El. 634.1 - 625.5

N_{60} = 29

Internal Angle of Friction $\boxed{\phi_1 := 35\,deg}$ Lambe and Whitman, Soil Mechanics, 1969, Fig. 11.14.

Soil Dry Unit Weight $\boxed{\gamma_{5dry} := 104pcf}$ Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships, average of loose and dense uniform sand

Saturated Unit Weight

Natural Water Content in a Saturated State: 9.3% (BB-AWBER-101 5D).

$w_{5sat} := 9.3\%$

$\gamma_{5sat} := \gamma_{5dry} \cdot (1 + w_{5sat})$

$\gamma_{5sat} = 113.7 \cdot pcf$

Effective Unit Weight

$\gamma'_5 := \gamma_{5sat} - \gamma_{water}$

$\boxed{\gamma'_5 = 51.3 \cdot pcf}$

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand below water table for static loading = 60 pci

Technical Manual
LPile 2016
p. 96

Abutment No. 2 Soil Model

1) The design soil layers are delineated as depicted on the attached annotated boring logs, which indicate the top and bottom elevations of the soil layers based on differing engineering properties.

Soil Layer No. 1 (Granular Borrow for Underwater Backfill) Finished Grade - 653.6.

Internal Angle of Friction $\boxed{\phi_1 := 32\,deg}$

Soil Dry Unit Weight $\boxed{\gamma_{1dry} := 125pcf}$

MaineDOT BDG Table 3-3

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand above water table for static loading = 90 pci

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Soil Layer No. 2 (Unsubmerged, Very loose, Sand) El. 653.6 - 644.6.

$N_{60} := 3\ \text{bpf}$

Internal Angle of Friction $\boxed{\phi_2 := 28}$

Lambe and Whitman, Soil Mechanics, 1969, Fig. 11.14.

Dry Unit Weight

Loose, uniform sand: 92 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2, Loose uniform sand.

$\gamma_{2dry} := 92\text{pcf}$

Representative constant giving the variation of soil modulus with depth, k:
Loose sand above water table for static loading = 25 pci

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Soil Layer No. 3 (Submerged, Medium Dense, Gravel) El. 644.6 - 639.6

Gravel wedged in SPT basket - assume predominantly sand deposit.

$N_{60} = 12\ \text{bpf}$

Internal Angle of Friction

$\boxed{\phi_3 := 31}$

Lambe and Whitman, Soil Mechanics, 1969, Fig. 11.14.

Dry Unit Weight

Medium dense, sand = 104 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - dry unit weight, average of loose and dense uniform sand

$\gamma_{3dry} := 104\text{pcf}$

Saturated Unit Weight

Natural water content at saturated state: 14.7%
(BB-AWBER-102, 2D)

$w_{3sat} := 14.7\%$

$\gamma_{3saturated} := \gamma_{3dry} \cdot (1 + w_{3sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{3saturated} = 119.3 \cdot \text{pcf}$

Soil Effective Unit Weight

$\gamma'_3 := \gamma_{3saturated} - \gamma_{water}$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 15 Eq (2-11).

$\boxed{\gamma'_3 = 56.9 \cdot \text{pcf}}$

Representative constant giving the variation of soil modulus with depth, k: medium dense sand below water table for static loading = 60 pci

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Soil Layer No. 4 (Submerged, Medium Dense Sand) El. 639.6 - 634.0

Design N_{60} = 25

Internal Angle of Friction $\phi_1 := 34\,deg$

Lambe and Whitman, Soil Mechanics, 1969, Fig. 11.14.

Soil Dry Unit Weight $\gamma_{4dry} := 104pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships, average of loose and dense uniform sand

Saturated Unit Weight

Natural Water Content in a Saturated State: 12.6% (BB-AWBER-102 4D).

$w_{4sat} := 12.6\%$

$\gamma_{4sat} := \gamma_{4dry} \cdot (1 + w_{4sat})$

$\gamma_{4sat} = 117.1 \cdot pcf$

Effective Unit Weight

$\gamma'_4 := \gamma_{4sat} - \gamma_{water}$

$\boxed{\gamma'_4 = 54.7 \cdot pcf}$

Representative constant giving the variation of soil modulus with depth, k: Medium dense sand below water table for static loading = 60 pci

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River.

Location: Andover, Maine

Boring No.: BB-AWBER-101

WIN: 21658.00

Driller:	New England Boring	**Elevation (ft.)**	654.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/14,19/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+15.2, 11.0 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Dry, caved 10.6 ft bgs
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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Layer 1 (Finished Grade - 650.5)	
										Granular Fill for Underwater Backfill	
	1D	24/3	2.00 - 4.00	1/1/1/2	2	2				Brown, damp to moist, very loose, fine to medium SAND, trace coarse sand, little silt, (Alluvium).	
											650.5
5	2D	24/14	5.00 - 7.00	2/3/3/7	6	6	HYD PUSH			Brown-grey, damp to moist, loose, fine to medium SAND, trace coarse sand, little gravel, little silt, (Alluvium). Layer 2 (650.5 - 644.5)	G#303057 A-2-4, SM WC=10.5%
										Unsubmerged, Loose, Sand	
							31				
							28				
							35				644.5
10	3D	24/8	10.00 - 12.00	11/9/9/8	18	18	21			Brown, medium dense, Sandy GRAVEL, trace silt. Layer 3 (644.5 - 639.5)	G#303058 A-1-a, GW WC=14.3%
							21			Submerged, Medium Dense, Sandy Gravel	
							15				
							11				
							10				639.5
15	4D	24/6	15.00 - 17.00	9/8/5/5	13	13	17			Brown, medium dense, Sandy GRAVEL, trace silt. Layer 4 (639.5 - 634.10)	
							17			Submerged, Medium Dense, Sand	
							17				
							19				
							25				
20	MD R1	1.2/0 60/51	20.00 - 20.10 20.40 - 25.40	50(1.2") RQD = ---%	---		NQ-2	634.1		Failed sample attempt, granite (pink feldspar-rich) chips in spoon. HW Casing Refusal, roller cone Refusal, set in NW Casing at 20.2 ft bgs.	634.1
										20.4 Pop through Boulder at 24.7 ft bgs. Boulder consists of Granodiorite or Tonalite.	
										Layer 5 (634.1 - 625.5)	
										Submerged, Medium Dense, Gravelly Sand	
25										potentially with Boulders	

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River.
Location: Andover, Maine

Boring No.: BB-AWBER-101
WIN: 21658.00

Driller:	New England Boring	**Elevation (ft.)**	654.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/14,19/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+15.2, 11.0 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Dry, caved 10.6 ft bgs
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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MV = Unsuccessful Field Vane Shear Test Attempt
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/17	25.40 - 27.40	24/17/12/13	29	29		629.1		25.4 Brown, medium dense, Gravelly SAND, trace silt. Pulled NW Casing, roller coned through boulder, spin shoe on NW Casing and reinstall at 27.4 ft bgs.	G#303059 A-1-b, SW-SM WC=9.3%
	R2	60/56	29.00 - 34.00	RQD = 63%			NQ-2	625.5		29.0 Top of Bedrock at Elev. 625.5 ft. R2: Bedrock: Very hard, typically fresh, fine to medium grained, black and white TONALITE, with two 1/2-inch to 1-inch fine grained layers. Close to moderately spaced, moderately dipping and low angle breaks; undulating, rough, fresh to discolored, and open, with occasional mud and sand infilling. Open fractures: 29.8 to 29.9, 30.7 to 30.8 and 31.0 to 31.5 ft bgs. Larger open fracture contains rounded gravel pieces of rock. Rock Mass Quality = Fair R2: Core Times (min:sec) 29.0-30.0 ft (1:20) 30.0-31.0 ft (1:10) 31.0-32.0 ft (1:00) 32.0-33.0 ft (1:30) 33.0-34.0 ft (1:30) 93% Recovery	
30											
	R3	60/58	34.00 - 39.00	RQD = 97%						R3: Bedrock: Same as R2, except one 2½-inch thick, moderately dipping, fine grained seam. One drillers break. Rock Mass Quality = Excellent R3: Core Times (min:sec) 34.0-35.0 ft (1:25) 35.0-36.0 ft (1:25) 36.0-37.0 ft (1:25) 37.0-38.0 ft (1:25) 38.0-39.0 ft (1:30) 97% Recovery	
35											
								615.5		39.0	
40										**Bottom of Exploration at 39.0 feet below ground surface.**	
45											
50											

Remarks:

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

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

Boring No.: BB-AWBER-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River.
Location: Andover, Maine

Boring No.: BB-AWBER-102
WIN: 21658.00

Driller:	New England Boring	**Elevation (ft.)**	654.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/19-20/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+49, 7.1 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = 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
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Layer 1 (Finished Grade - 653.6) Granular Borrow	653.6
	1D	24/16	2.00 - 4.00	1/2/1/1	3	3				Brown, damp, very loose, fine to medium SAND, trace to little silt. Layer 2 (653.6 - 644.6) Unsubmerged, Very Loose, Sand	
5	2D	24/8	5.00 - 7.00	1/1/2/2	3	3	HYD PUSH			Brown, damp, very loose, fine to medium SAND, trace coarse sand, little silt, trace gravel.	G#303060 A-2-4, SM WC=14.7%
							17				
							21				
10	3D	24/4	10.00 - 12.00	10/6/6/5	12	12	21			Brown, medium dense, GRAVEL, little sand, trace silt, large piece gravel wedged in spoon, not representative sample. Layer 3 (644.6 - 639.6) Submerged, Medium Dense, Gravel	644.6
							20				
							42				
							47				
							42				639.6
15	4D	24/9	15.00 - 17.00	8/11/14/12	25	25	32			Brown, medium dense, SAND, some gravel, trace silt. Layer 4 (639.6 - 634.0) Submerged, Medium Dense, Sand	G#303061 A-1-b, SW WC=12.6%
							42				
							98				
							65				
							42				
20	5D	7.2/7.2	20.00 - 20.60	6/50(1.2")	---			634.0 633.6		Brown, fine Sandy SILT, trace coarse to medium sand, trace gravel. 20.6	G#303062 A-4, CL WC=20.5%
	R1	60/60	21.00 - 26.00	RQD = 83%			NQ-2			Top of Bedrock at Elev. 634.0 ft. Roller coned ahead to 21.0 ft bgs; difficult to advance, set in NW Casing and prepare to core. 21.0 R1: Bedrock: Very hard, fresh, fine to medium grained, black and white TONALITE with quartz-rich veins and flow structure. Open fracture zone from 21.7 to 22.4 ft, otherwise core is single piece. Rock Mass Quality = Good R1: Core Times (min:sec) 21.0-22.0 ft (1:30)	
25											

Remarks:

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Boring No.: BB-AWBER-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Merrill Bridge #3215 carries Route 120 (Elm St.) over West Branch Ellis River.
Location: Andover, Maine

Boring No.: BB-AWBER-102
WIN: 21658.00

Driller:	New England Boring	**Elevation (ft.)**	654.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Schaefer/Royal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/19-20/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+49, 7.1 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										22.0-23.0 ft (1:25) 23.0-24.0 ft (1:40) 24.0-25.0 ft (1:55) 25.0-26.0 ft (1:40) 100% Recovery	
	R2	60/56	26.00 - 31.00	RQD = 77%						R2: Bedrock: Same Rock as R1, with feldspar-rich calesilicate layer from 27.1 to 27.9 ft. Typically moderately spaced, low angle breaks; undulating, rough, discolored and open. Open fracture from 27.9 to 28.3 ft. Rock Mass quality = Fair to Good R2: Core Times (min:sec) 26.0-27.0 ft (1:20) 27.0-28.0 ft (1:15) 28.0-29.0 ft (1:05) 29.0-30.0 ft (1:20) 30.0-31.0 ft (1:25) 93% Recovery	
30								623.6		31.0	
										Bottom of Exploration at 31.0 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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

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

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Table 3-6 Representative Values of *k* for Fine Sand Below the Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	5.4 (20.0)	16.3 (60.0)	34 (125.0)

Table 3-7 Representative Values of *k* for Fine Sand Above Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	6.8 (25.0)	24.4 (90.0)	61.0 (225.0)

If the sand profile is coarse or well-graded sand, the user may consider using a higher value of *k* that those suggested in the tables above. While experimental data for *k* in well-graded sands is poorly documented, use of values 10 to 50 percent higher may be appropriate in dense and very dense well-graded sands that do not contain any compressible minerals such as mica.

7. Fit the parabola between point *k* and point *m* as follows:

a. Compute the slope of the *p-y* curve between point *m* and point *u* using

$$m = \frac{p_u - p_m}{y_u - y_m} \qquad (3\text{-}62)$$

b. Compute the power of the parabolic section using

$$n = \frac{p_m}{m y_m} \qquad (3\text{-}63)$$

c. Compute the coefficient $\bar{C}$ using

$$\bar{C} = \frac{p_m}{y_m^{1/n}} \qquad (3\text{-}64)$$

8. Compute the *y* value defining point *k* using

$$y_k = \left(\frac{\bar{C}}{kx} \right)^{\frac{n}{n-1}} \qquad (3\text{-}65)$$

Compute the *p* value defining point *k* using

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft^3	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Abutment 1

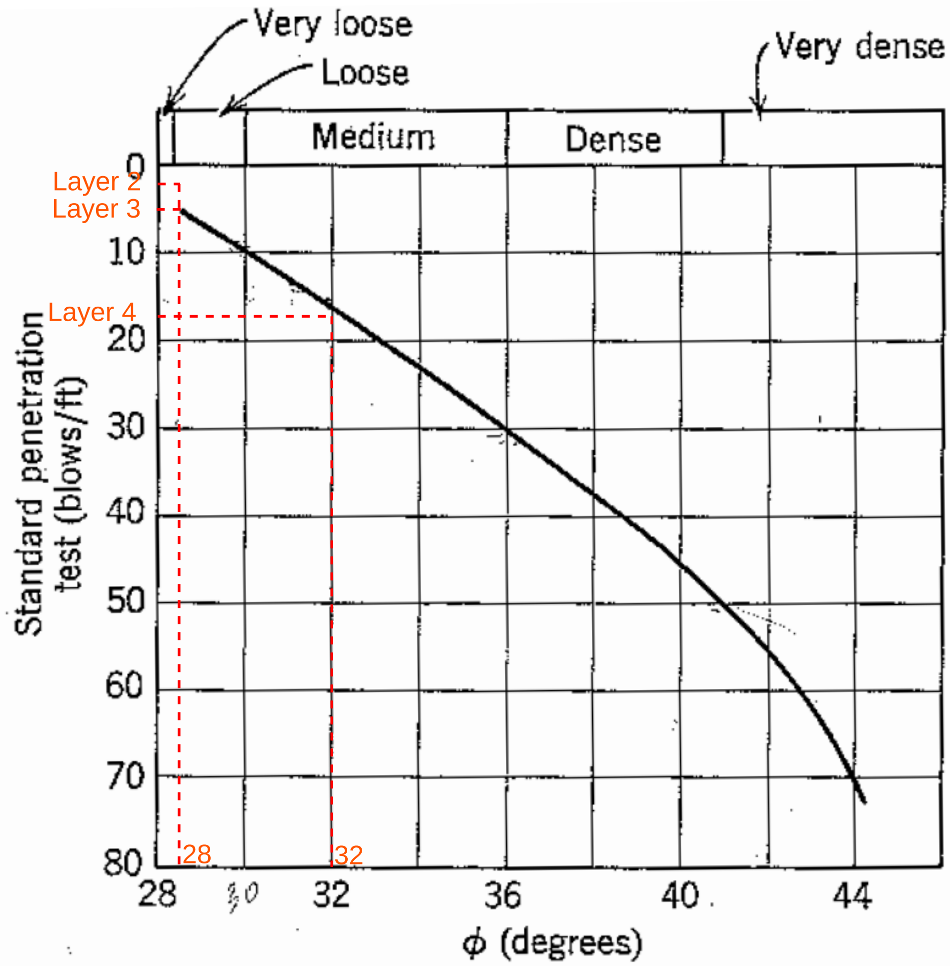


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Abutment 2

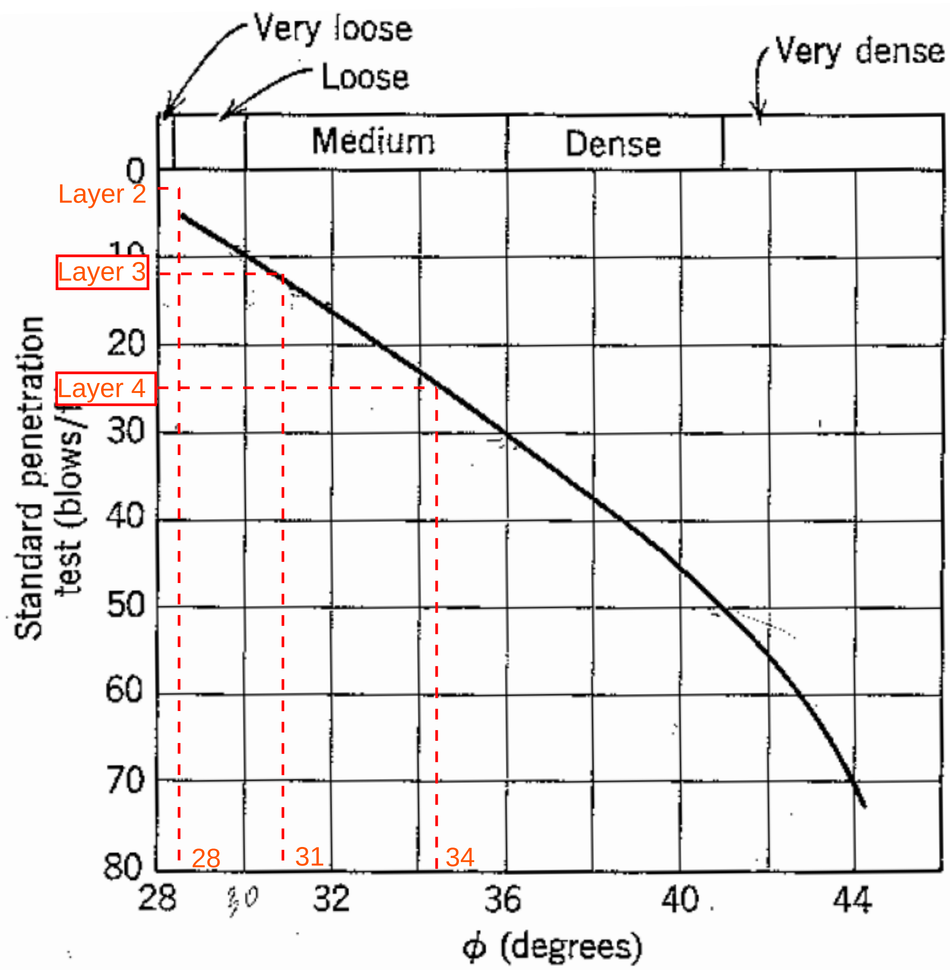


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Holtz and Kovacs , Intro to Geotechnical Eng

common mineral in sands is quartz, its $\rho_s = 2.65$ Mg/m³. Most clay soils have a value of ρ_s between 2.65 and 2.80 Mg/m³, depending on the predominant mineral in the soil, whereas organic soils may have a ρ_s as low as 2.5 Mg/m³. Consequently, it is usually close enough for geotechnical work to *assume* a ρ_s of 2.65 or 2.70 Mg/m³ for most phase problems, unless a specific value of ρ_s is given.

The density of water varies slightly, depending on the temperature. At 4°C, when water is at its densest, ρ_w exactly equals 1000 kg/m³ (1 g/cm³), and this density is sometimes designated by the symbol ρ_o. For ordinary engineering work, it is sufficiently accurate to take $\rho_w \approx \rho_o = 1000$ kg/m³ = 1 Mg/m³.

There are three other useful densities in soils engineering. They are the dry density ρ_d, the saturated density ρ_{sat}, and the submerged or buoyant density ρ'.

$$\rho_d = \frac{M_s}{V_t} \tag{2-9}$$

$$\rho_{sat} = \frac{M_s + M_w}{V_t} (V_a = 0,\ \ S = 100\%) \tag{2-10}$$

$$\boxed{\rho' = \rho_{sat} - \rho_w \qquad (2\text{-}11)}$$

Strictly speaking, total ρ should be used instead of ρ_{sat} in Eq. 2-11, but in most cases completely submerged soils are also completely saturated, or at least it is reasonable to assume they are saturated. The dry density ρ_d is a common basis for judging the degree of compaction of earth embankments (Chapter 5). A typical range of values of ρ_d, ρ_{sat}, and ρ' for several soil types is shown in Table 2-1.

From the basic definitions provided in this section, other useful relationships can be derived, as we show in the examples in the next section.

TABLE 2-1 Some Typical Values for Different Densities of Some Common Soil Materials*

Soil Type	Density (Mg/m³)		
	ρ_{sat}	ρ_d	ρ'
Sands and gravels	1.9–2.4	1.5–2.3	1.0–1.3
Silts and clays	1.4–2.1	0.6–1.8	0.4–1.1
Glacial tills	2.1–2.4	1.7–2.3	1.1–1.4
Crushed rock	1.9–2.2	1.5–2.0	0.9–1.2
Peats	1.0–1.1	0.1–0.3	0.0–0.1
Organic silts and clays	1.3–1.8	0.5–1.5	0.3–0.8

*Modified after Hansbo (1975).

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

$\phi_1 =$ Angle of internal friction $\phi' = 32 \cdot \text{deg}$

$\beta =$ 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 δ = 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)

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

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

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

$\boxed{K_{p\_coulomb} = 6.73}$

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 α > 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

$\boxed{K_{p\_rank} = 3.25}$

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

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

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

$\boxed{K_o = 0.47}$

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 δ (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_f.

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 $\beta = 90°$ and $\alpha = 0°$

	δ' (deg)				
ϕ' (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, . . . , 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

$$P_p = \tfrac{1}{2}\gamma H^2 K_p \tag{7.70}$$

where

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

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

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 (δ' = 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

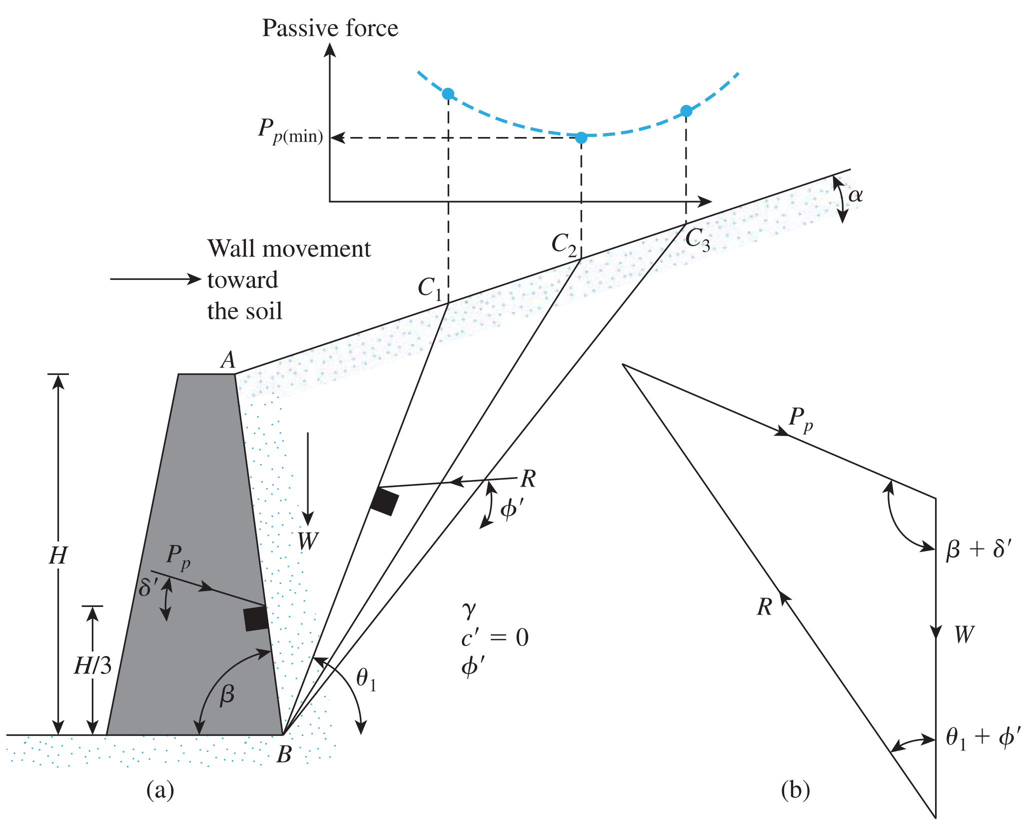


Figure 7.25 Coulomb's passive pressure

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

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

Again, at $z = 3$ m,

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

Hence,

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

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, $u = (1)(\gamma_w) = 9.81$ 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 **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

$$\sigma'_p = \gamma z K_p \tag{7.65}$$

and the passive force is

$$P_p = \tfrac{1}{2}\gamma H^2 K_p \tag{7.66}$$

where

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

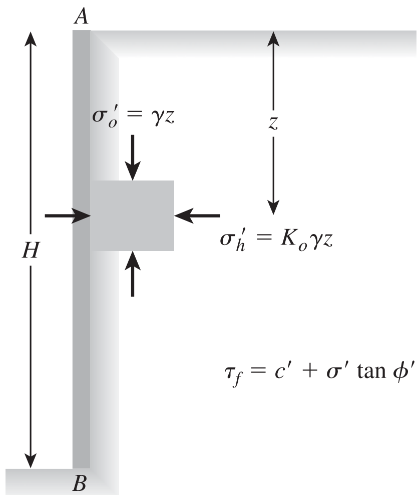


Figure 13.3
Earth pressure at rest

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)

$$\boxed{K_o = 1 - \sin \phi'} \tag{13.5}$$

where ϕ' = 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

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

where γ_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)

Settlement

Development of embankment soil model for Settle 3D

OBJECTIVE
To estimate soil parameters for Settle 3D analysis.

Given:
Boring logs and lab data.
Assumptions:
1) Groundwater table is at Elev. 645, equal to the ordinary water level of West Branch Ellis River.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 is used to construct the proposed raise in grade.

Calculations for approach embankment behind Abutment 1

Overburden

Maximum depth of new fill = 8 ft.

$H_{fill} := 8ft$

$\gamma\_fill := 125pcf$ BDG Table 3-3, Soil Type 4, Granular Borrow

$\sigma_{z\_induced} := \gamma\_fill \cdot H_{fill}$

$\sigma_{z\_induced} = 1.0 \cdot ksf$

Soil Layer 1 (Elev. 654.5 - 649.5): Very loose, SAND

$N_{60\_1} := 2$

$E_{s1} := 500 \frac{(N_{60\_1} + 15)}{50}$ Bowles Table 5-6, Equations for stress-strain modulus E_s

$\boxed{E_{s1} = 170}$ ksf

$E_{ur\_1} := 4 \cdot E_{s1}$ Equation 5, Mayne and Cox, Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur\_1} = 680}$ ksf

$\boxed{\gamma_{dry\_1} := 92pcf}$ Das, Principles of Geotechnical Engineering, Table 3.2: Dry Unit Weights, Loose uniform sand

$w_{sat\_1} := 30\%$ Das, Principles of Geotechnical Engineering, Table 3.2: Natural Moisture Content in a Saturated State, Loose uniform sand

$\gamma_{sat\_1} := \gamma_{dry\_1} \cdot (1 + w_{sat\_1})$

$\boxed{\gamma_{sat\_1} = 119.6 \cdot pcf}$

Soil Layer 2 (Elev. 649.5 - 644.5): Very loose to loose, SAND

$N_{60\_2} := 6$

$E_{s2} := 500 \frac{(N_{60\_2} + 15)}{50}$ Bowles Table 5-6, Equations for stress-strain modulus E_s

$\boxed{E_{s2} = 210}$ ksf

$E_{ur\_2} := 4 \cdot E_{s2}$ Equation 5, Mayne and Cox, Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur\_2} = 840}$ ksf

$\boxed{\gamma_{dry\_2} := 92\text{pcf}}$ Das, Principles of Geotechnical Engineering, Table 3.2: Dry Unit Weights, Loose uniform sand

$w_{sat\_2} := 30\%$ Das, Principles of Geotechnical Engineering, Table 3.2: Natural Moisture Content in a Saturated State, Loose uniform sand

$\gamma_{sat\_2} := \gamma_{dry\_2} \cdot (1 + w_{sat\_2})$

$\boxed{\gamma_{sat\_2} = 119.6 \cdot \text{pcf}}$

Soil Layer 3 (644.5 - 639.5) Medium dense, Sandy GRAVEL

$N_{60\_3} := 18$

$E_{s3} := \dfrac{1200(N_{60\_3} + 6)}{50}$ Bowles Table 5-6, Equations for stress-strain modulus E_s for gravelly sand

$\boxed{E_{s3} = 576}$

$E_{ur\_3} := 4 \cdot E_{s3}$ Equation 5, Mayne and Cox, Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur\_3} = 2304}$ ksf

$\gamma_{dry\_3} := \dfrac{(115\text{pcf} - 92\text{pcf})}{2} + 92\text{pcf}$ Das, Principles of Geotechnical Engineering, Table 3.2: Dry Unit Weights, Average of Loose and dense uniform sand unit weight

$\boxed{\gamma_{dry\_3} = 103.5 \cdot \text{pcf}}$

$w_{sat\_3} := 14.3\%$ BB-AWBER-101 3D, Natural water content

$\gamma_{sat\_3} := \gamma_{dry\_3} \cdot (1 + w_{sat\_3})$

$\boxed{\gamma_{sat\_3} = 118.3 \cdot \text{pcf}}$

Soil Layer 4 (639.5 - 634.1) Medium dense, Sandy GRAVEL

$N_{60\_4} := 13$

$E_{s4} := \dfrac{600 \cdot (N_{60\_4} + 6)}{50}$ Bowles Table 5-6, Equations for stress-strain modulus E_s for Gravelly Sand N ≤ 15

$\boxed{E_{s4} = 228}$ ksf

$E_{ur\_4} := 4 \cdot E_{s4}$ Equation 5, Mayne and Cox, Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur\_4} = 912}$ ksf

$$\gamma_{dry\_4} := \frac{(115pcf - 92pcf)}{2} + 92pcf$$

Das, Principles of Geotechnical Engineering, Table 3.2: Dry Unit Weights, Average of Loose and dense uniform sand unit weight

$\boxed{\gamma_{dry\_4} = 103.5 \cdot pcf}$

$w_{sat\_4} := 14.3\%$ BB-AWBER-101 3D, Natural water content 14.3%

$$\gamma_{sat\_4} := \gamma_{dry\_4} \cdot (1 + w_{sat\_4})$$

$\boxed{\gamma_{sat\_4} = 118.3 \cdot pcf}$

Soil Layer 5 (634.1 - 625.5) Medium dense, Sandy GRAVEL

$N_{60\_5} := 29$

$$E_{s5} := \left[600 \cdot (N_{60\_5} + 6) + 2000\right] \cdot \frac{1}{50}$$

$\boxed{E_{s5} = 460}$ ksf

Bowles Table 5-6, Equations for stress-strain modulus E_s for Gravelly Sand, N>15

$E_{ur\_5} := 4 \cdot E_{s5}$ Equation 5, Mayne and Cox, Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur\_5} = 1840}$ ksf

$$\gamma_{dry\_5} := \frac{(115pcf - 92pcf)}{2} + 92pcf$$

Das, Principles of Geotechnical Engineering, Table 3.2: Dry Unit Weights, Average of Loose and dense uniform sand unit weight

$\boxed{\gamma_{dry\_5} = 103.5 \cdot pcf}$

$w_{sat\_5} := 9.3\%$ BB-AWBER-101 5D, Natural water content 9.3%

$$\gamma_{sat\_5} := \gamma_{dry\_5} \cdot (1 + w_{sat\_5})$$

$\boxed{\gamma_{sat\_5} = 113.1 \cdot pcf}$

Approach embankment behind Abutment 2

Overburden

Maximum depth of new fill = 11 ft.

$H_{fill} := 11ft$

$\gamma\_fill := 125pcf$ BDG Table 3-3, Soil Type 4, Granular Borrow

$\sigma_{z\_induced} := \gamma\_fill \cdot H_{fill}$

$\boxed{\sigma_{z\_induced} = 1.38 \cdot ksf}$

Soil Layer 1 (Elev. 654.6 - 644.6): Very Loose, Fine to Medium Sand

$N_{60\_1} := 3$

$E_{s1} := 500 \dfrac{(N_{60\_1} + 15)}{50}$ Bowles Table 5-6, Equations for stress-strain modulus E_s

$\boxed{E_{s1} = 180}$ ksf

$E_{ur} := 4 \cdot E_{s1}$ Equation 5, Mayne and Cox, Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur} = 720}$ ksf

$\boxed{\gamma_{dry\_1} := 92pcf}$ Das, Principles of Geotechnical Engineering, Table 3.2: Dry Unit Weights, Loose uniform sand

$w_{sat\_1} := 30\%$ Das, Principles of Geotechnical Engineering, Table 3.2: Natural Moisture Content in a Saturated State, Loose uniform sand

$\gamma_{sat\_1} := \gamma_{dry\_1} \cdot (1 + w_{sat\_1})$

$\boxed{\gamma_{sat\_1} = 119.6 \cdot pcf}$

Soil Layer 2 (Elev. 644.6 - 634.0): Medium dense, Gravel, little Sand

$N_{60\_2} := 25$

$E_{s2} := \dfrac{[600 \cdot (N_{60\_2} + 6) + 2000]}{50}$ Bowles Table 5-6, Equations for stress-strain modulus $E_{s,}$ Gravelly Sand, N≥15

$\boxed{E_{s2} = 412}$ ksf

$E_{ur2} := 4 \cdot E_{s2}$ Equation 5, Mayne and Cox, Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur2} = 1648}$ ksf

$\gamma_{dry\_2} := \dfrac{(115pcf - 92pcf)}{2} + 92pcf$ Das, Principles of Geotechnical Engineering, Table 3.2: Dry Unit Weights, Average of Loose and dense uniform sand unit weight

$\boxed{\gamma_{dry\_2} = 103.5 \cdot pcf}$

$w_{sat\_2} := 12.6\%$ BB-AWBER-102, 4D Natural moisture content

$\gamma_{sat\_2} := \gamma_{dry\_2} \cdot (1 + w_{sat\_2})$

$\boxed{\gamma_{sat\_2} = 116.5 \cdot pcf}$

Settle3D Analysis Information
Andover WIN 21658.00

Project Settings

Document Name	Abutment 1.s3z
Project Title	Andover WIN 21658.00
Analysis	Settlement behind Abutment 1
Author	A. Van Buskirk
Company	MaineDOT
Date Created	4/19/2018, 3:04:43 PM
Stress Computation Method	Boussinesq
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 1.06084 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.08416
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	1.08416
Loading Stress [ksf]	0	1
Total Stress [ksf]	0	3.43588
Total Strain	0	0.00588235
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.023	3.43514
Over-consolidation Ratio	1	1
Void Ratio	0	0
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0.592447

Embankments

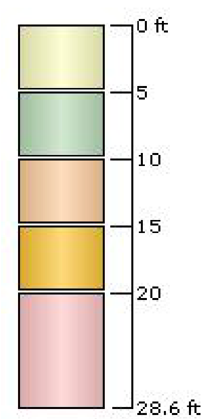
1. Embankment

Center Line	(0, 0) to (50, 0)
Number of Layers	1
Near End Angle	90 degrees
Far End Angle	90 degrees
Base Width	60

Layer	Stage	Left Bench Width (ft)	Left Angle (deg)	Height (ft)	Unit Weight (kips/ft^3)	Right Angle (deg)	Right Bench Width (ft)
1	Stage 1	0	27	8	0.125	27	0

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Soil Property 1	5	0
2	Soil Property 2	5	5
3	Soil Property 3	5	10
4	Soil Property 4	5	15
5	Soil Property 5	8.6	20



Soil Properties

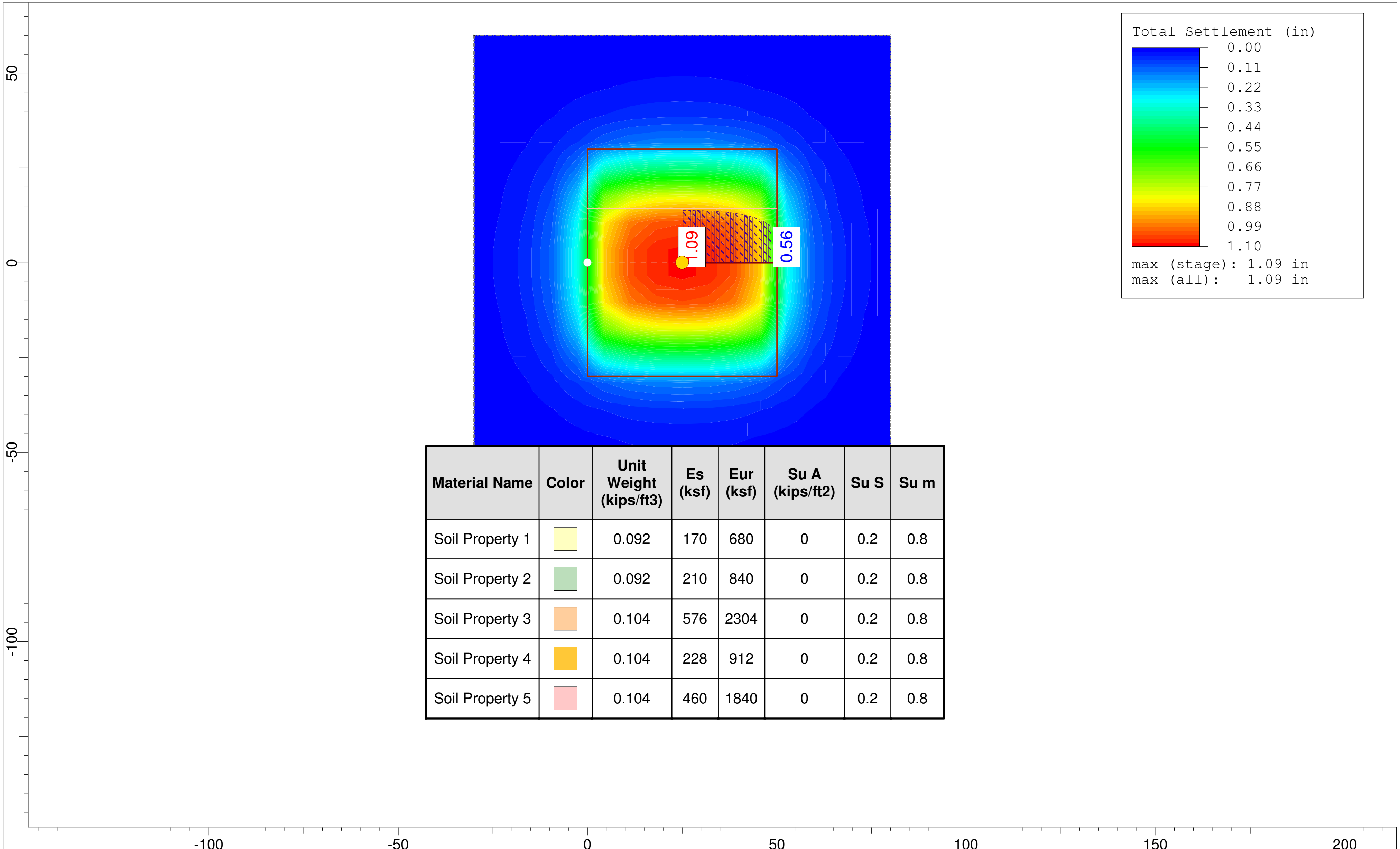
Property	Soil Property 1	Soil Property 2	Soil Property 3	Soil Property 4	Soil Property 5
Color					
Unit Weight [kips/ft^3]	0.092	0.092	0.104	0.104	0.104
Immediate Settlement	Enabled	Enabled	Enabled	Enabled	Enabled
Es [ksf]	170	210	576	228	460
Esur [ksf]	680	840	2304	912	1840
Undrained Su A [kips/ft2]	0	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8	0.8

Field Point Grid

Number of points 306
Expansion Factor 2

Grid Coordinates

X [ft]	Y [ft]
80	60
80	-60
-30	-60
-30	60



Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Su A (kips/ft2)	Su S	Su m
Soil Property 1		0.092	170	680	0	0.2	0.8
Soil Property 2		0.092	210	840	0	0.2	0.8
Soil Property 3		0.104	576	2304	0	0.2	0.8
Soil Property 4		0.104	228	912	0	0.2	0.8
Soil Property 5		0.104	460	1840	0	0.2	0.8

SETTLE3D 3.020

Project	Andover WIN 21658.00		
Analysis Description	Settlement behind Abutment 1		
Drawn By	A. Van Buskirk	*Company*	MaineDOT
Date	4/19/2018, 3:04:43 PM	*File Name*	Abutment 1.s3z

Settle3D Analysis Information
Andover WIN 21658.00

Project Settings

Document Name	Abutment 2.s3z
Project Title	Andover WIN 21658.00
Analysis	Settlement behind abutment 2
Author	A. Van Buskirk
Company	MaineDOT
Date Created	4/25/2018, 8:39:14 AM
Stress Computation Method	Boussinesq
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0.884038 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.29128
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	1.29128
Loading Stress [ksf]	0	1.375
Effective Stress [ksf]	0	2.62346
Total Stress [ksf]	0	3.30986
Total Strain	0	0.00763889
Pore Water Pressure [ksf]	0	0.6864
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.0008832	2.62311
Over-consolidation Ratio	1	1
Void Ratio	0	0
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0.332802

Embankments

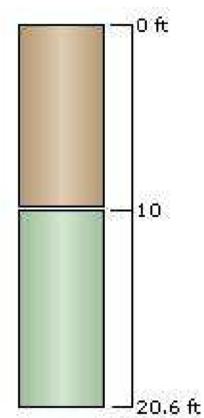
1. Embankment

Center Line	(0, 0) to (75, 0)
Number of Layers	1
Near End Angle	90 degrees
Far End Angle	90 degrees
Base Width	72

Layer	Stage	Left Bench Width (ft)	Left Angle (deg)	Height (ft)	Unit Weight (kips/ft^3)	Right Angle (deg)	Right Bench Width (ft)
1	Stage 1	0	27	11	0.125	27	0

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Soil Property 1	10	0
2	Soil Property 2	10.6	10



Soil Properties

Property	Soil Property 1	Soil Property 2
Color		
Unit Weight [kips/ft^3]	0.092	0.104
Saturated Unit Weight [kips/ft^3]	0.12	0.117
Immediate Settlement	Enabled	Enabled
Es [ksf]	180	412
Esur [ksf]	720	1648
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8
Piezo Line ID	1	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

Piezometric Line Entities

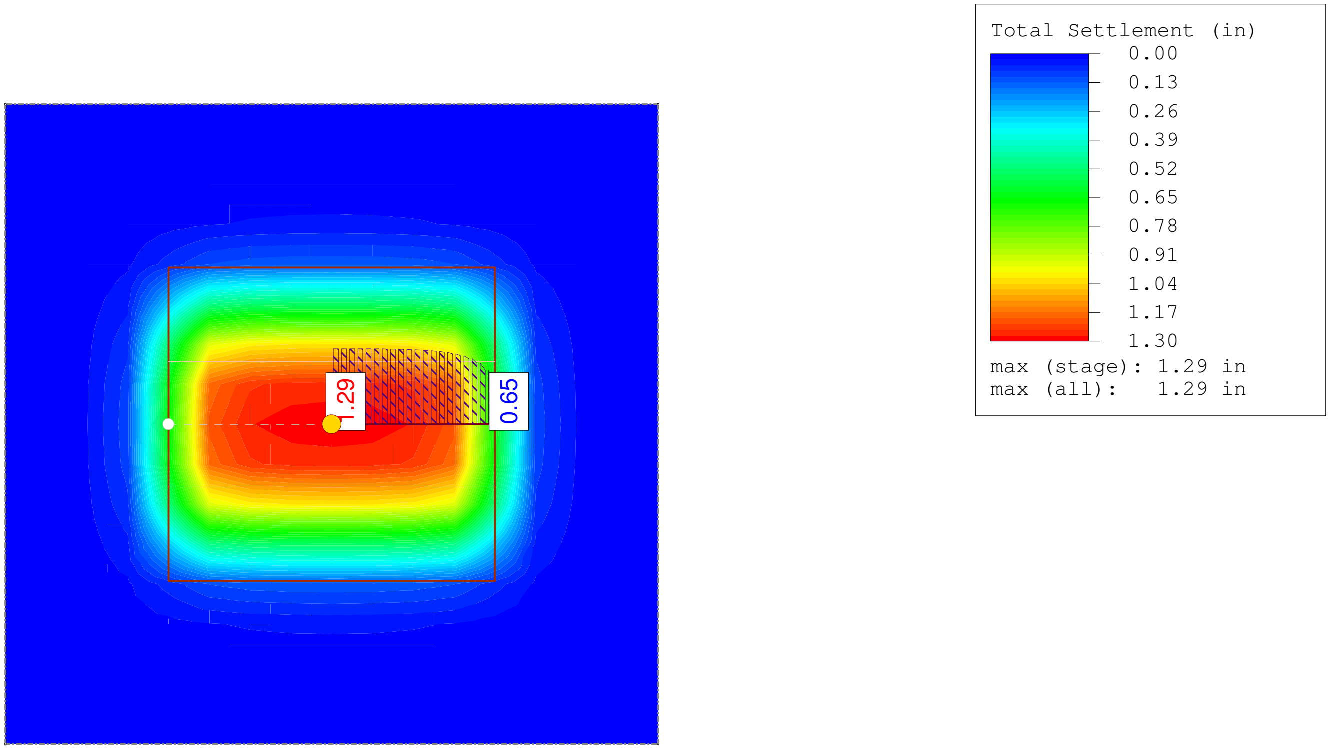
ID	Depth (ft)
1	9.6 ft

Field Point Grid

Number of points 289
Expansion Factor 2

Grid Coordinates

X [ft]	Y [ft]
112.5	73.5
112.5	-73.5
-37.5	-73.5
-37.5	73.5



Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Su A (kips/ft2)	Su S	Su m
Soil Property 1		0.092	180	720	0	0.2	0.8
Soil Property 2		0.104	412	1648	0	0.2	0.8

SETTLE3D 3.020	*Project* Andover WIN 21658.00		
	Analysis Description Settlement behind abutment 2		
	Drawn By A. Van Buskirk	*Company* MaineDOT	
	Date 4/25/2018, 8:39:14 AM	*File Name* Abutment 2.s3z	

TABLE 5-6
Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$\boxed{E_s = 500(N + 15)}$ $= 7000\sqrt{N}$ $= 6000N$ — — — $^{\ddagger}E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ $^{*}E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	$^{\P}E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	$^{\dagger}E_s = 40\,000 + 1050N$ $E_{s(\mathrm{OCR})} \approx E_{s,\mathrm{nc}}\sqrt{\mathrm{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$\boxed{\begin{aligned} E_s &= 1200(N + 6) \\ &= 600(N + 6) \quad N \le 15 \\ &= 600(N + 6) + 2000 \quad N > 15 \end{aligned}}$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$ If $q_c < 2500$ kPa use $\quad ^{\$}E'_s = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E'_s = 4q_c + 5000$ where $E'_s = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	$E_s = (1 \text{ to } 2)q_c$
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

4. It is not easy to determine if a cohesionless deposit is overconsolidated or what the OCR might be. Cementation may be less difficult to discover, particularly if during drilling or excavation sand "lumps" are present. Carefully done consolidation tests will aid in obtaining the OCR of cohesive deposits as noted in Chap. 2.

In general, with an OCR > 1 you should carefully ascertain the site conditions that will prevail at the time settlement becomes the design concern. This evaluation is, of course, true for any site, but particularly so if OCR > 1.

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

5-9.1 Effects on Settlements

A major problem in foundation design is to proportion the footings and/or contact pressure so that settlements between adjacent footings are nearly equal. Figure 5-9 illustrates the problem

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: Chesterville**, Maine**
Case 1 - coarse grained granular fill soils W=10%

Andover lies on the 1800 F-days isoline.

Frost penetration = 90.1 in. = 7.5 feet.

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Farmington lies near the 1800 F-days isoline.

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

Project Location: Farmington, Maine

Air Design Freezing Index         =  2023 F-days
N-Factor                          =  0.80
Surface Design Freezing Index     =  1618 F-days
Mean Annual Temperature           =  41.2 deg F
Design Length of Freezing Season =  145 days

---------------------------------------------------------
Layer
#:Type              t    w%     d     Cf   Cu    Kf    Ku      L
---------------------------------------------------------
1-Coarse          92.8 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 = 7.73 ft = 92.8 in.
*********************************************************"
```

Recommendation: 7.5 feet for design of foundations constructed on coarse grained soils

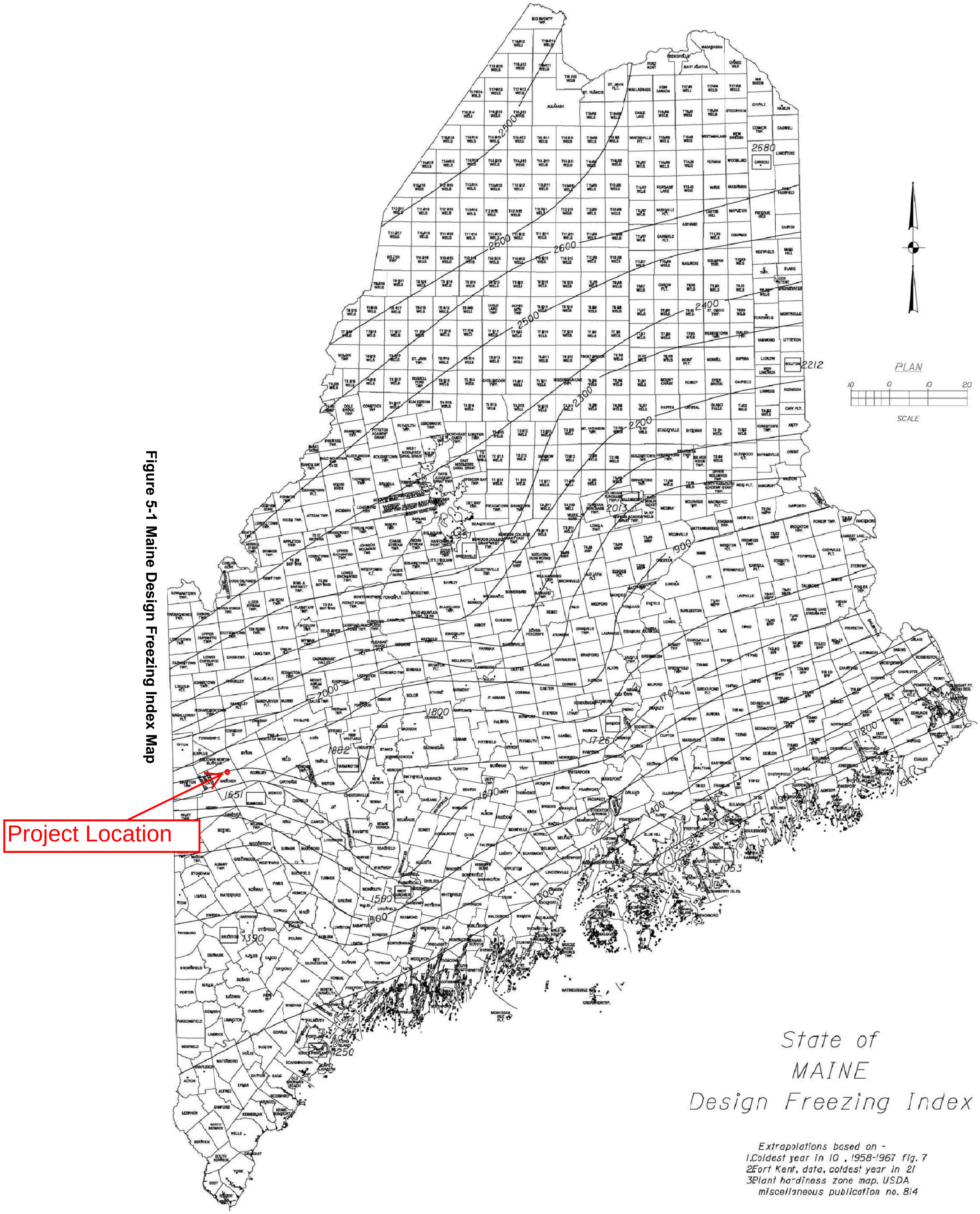


Figure 5-1 Maine Design Freezing Index Map

5.2 General

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

90.1 in.

Seismic Parameters

BB-AWBER-101			
Depth	**101 N_{60}**	**di**	**di/N**
3	2	3	1.50
6	6	3	0.50
11	18	5	0.28
16	13	5	0.38
26.4	29	10.4	0.36
100	100	73.6	0.74
SUM		**100**	**2.26**
		di/di/N	**44.31**

BB-AWBER-102			
Depth	**102 N_{60}**	**di**	**di/N**
3	3	3	1.00
6	3	3	1.00
11	12	5	0.42
16	25	5	0.20
100	100	84	0.84
SUM		**100**	**2.46**
		di/di/N	**40.71**

SUM	**$N_{av.}$**	**42.51**

$15 < N_{av.} < 50$

Conclusion: Site Class D

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

Andover 21658.00
Merrill Bridge #3215
3/21/2018

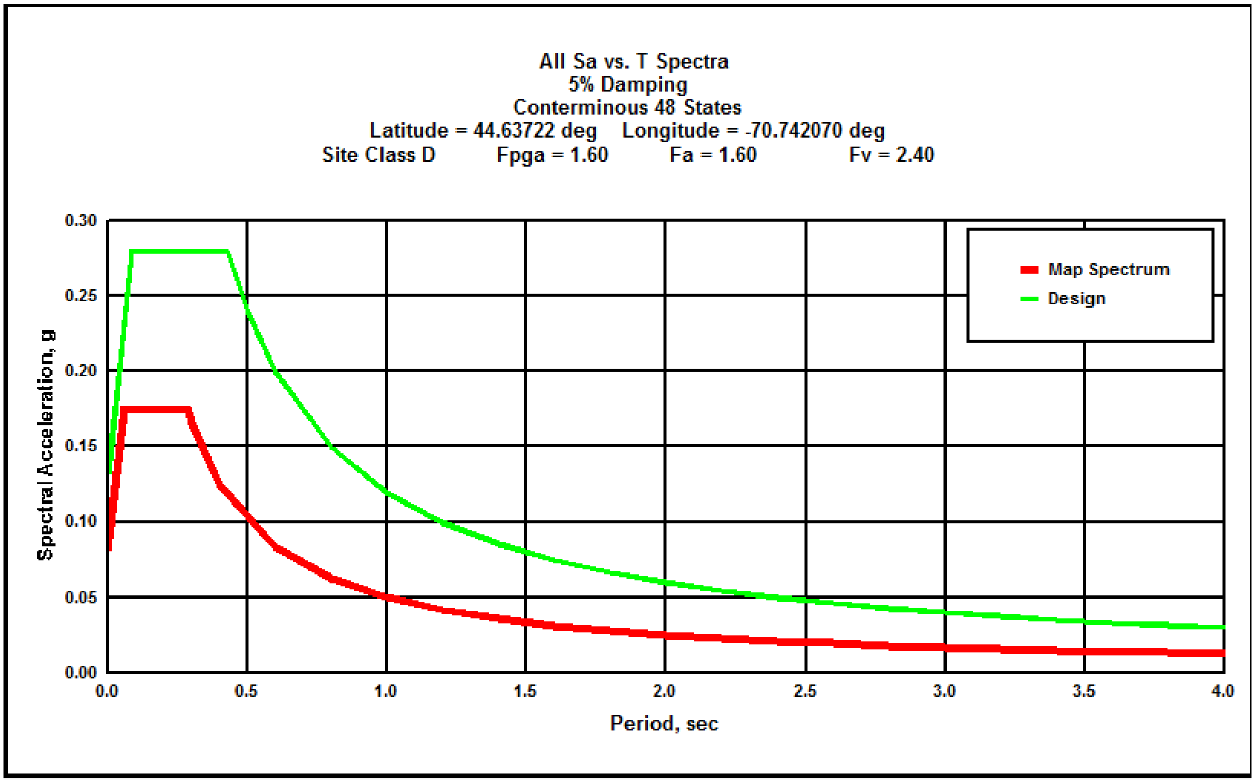
Calculations by A. Van Buskirk
Checked by LK
Date 5/2018

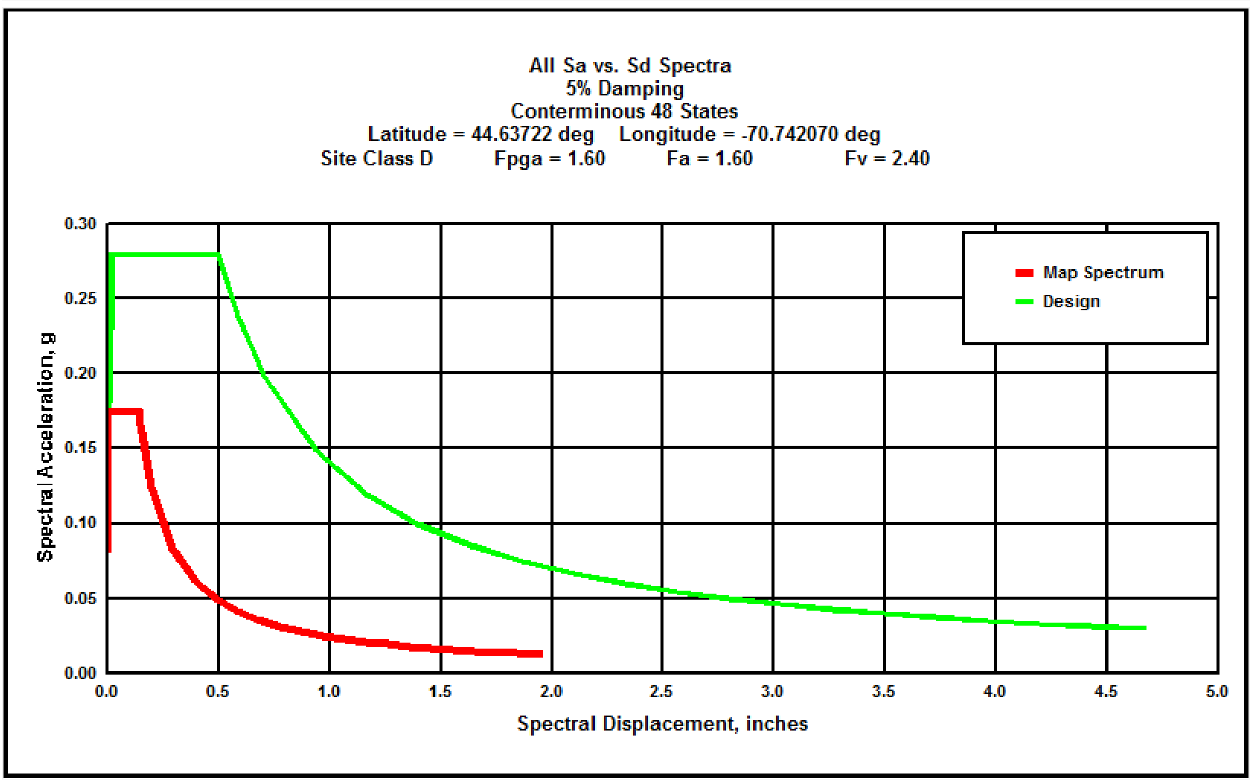
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.637223
Longitude = -070.742076
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.083	PGA - Site Class B
0.2	0.174	Ss - Site Class B
1.0	0.050	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.637223
Longitude = -070.742076
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.133	As - Site Class D
0.2	0.279	SDs - Site Class D
1.0	0.119	SD1 - Site Class D


All Sa vs. T Spectra
5% Damping
Conterminous 48 States
Latitude = 44.63722 deg Longitude = -70.742070 deg
Site Class D Fpga = 1.60 Fa = 1.60 Fv = 2.40
Map Spectrum
Design
Spectral Acceleration, g
Period, sec
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


All Sa vs. Sd Spectra
5% Damping
Conterminous 48 States
Latitude = 44.63722 deg Longitude = -70.742070 deg
Site Class D Fpga = 1.60 Fa = 1.60 Fv = 2.40
Map Spectrum
Design
Spectral Acceleration, g
Spectral Displacement, inches
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0