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

## GEOTECHNICAL DESIGN REPORT

*For the Replacement of:*

**DUTCH GAP BRIDGE**
**DUTCH GAP ROAD OVER LITTLE NORRIDGEWOCK STREAM**
**CHESTERVILLE, 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 Dutch Gap Bridge which carries Dutch Gap Road over Little Norridgewock Stream in Chesterville, 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 structure was built in 1949 and consists of two 14-foot corrugated metal plate culverts. The plans for the existing culverts indicate the stone masonry abutments from the previous bridge, bearing on log grillage, are buried on site. According to the 2017 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the culverts are rated 4 for considerable damage. The report cites isolated areas of heavy deterioration and moderate to heavy rust nodules with moderate/heavy pitting, and unzipping at the north end of the culvert. The Sufficiency Rating of the existing culverts is 59.8.

The proposed replacement structure will consist of a 75 to 80-foot single span metalized steel plate girder or NEXT F-beam superstructure on pile-supported integral abutments. One lane of alternating traffic will be maintained on a temporary bridge using temporary traffic signals during construction. The temporary bridge will be constructed on the upstream side of the existing bridge. The centerline of the new Dutch Gap Bridge will shift slightly upstream of the existing centerline and the vertical profile will increase approximately 8 inches

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## 2.0 GEOLOGIC SETTING

The existing structure carries Dutch Gap Road over Little Norridgewock Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Farmington Quadrangle, Maine, Open-file No. 86-29 (1986), indicates the surficial soils in the vicinity of the bridge project consist of glacial stream deposits with minor inclusions of glacial till. Glacial stream deposits typically consist of sand and gravel. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. Glacial till includes two varieties: basal till and ablation till. Basal till is typically fine grained and very compact with low permeability and poor drainage. Ablation till is typically loose, sandy, and stony with moderate permeability and fair to good drainage. These soils generally overlie bedrock, but may overlie, or include, sand and gravel.

The Bedrock Geology Map of the Farmington Quadrangle (MGS), Open-File No. 78-16 (1978), cites the bedrock at the project site as a granite with nearby contacts to metasiltstone, metapelites and metalimestones of the Patch Mountain Member of the Sangerville Formation. Bedrock cores retrieved at the site are identified as limey pelite.

## 3.0 SUBSURFACE INVESTIGATION

Two test borings were drilled at the site 81 feet, center to center: BB-CNS-101 and BB-CNS-102. BB-CNS-101 was drilled at the proposed location of Abutment 1 and on the east side of the road; BB-CNS 102 was drilled at the proposed location of Abutment 2 and on the west side. The test boring locations are shown on Sheet 2 – Boring Location Plan.

The MaineDOT Drill Crew drilled the test borings in September 2017. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring and rock coring techniques. Soil samples were typically obtained 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 MaineDOT dill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" prior to the test borings in April 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.854 for both borings. The hammer efficiency factor (0.854) and both the raw field N-value and corrected N-value ($N_{60}$) are shown on the boring logs.

Bedrock was cored using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core calculated. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface 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 located in the field using taped measurements at the completion of the drilling program.

## 4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of: seven standard grain size analyses with natural water content, and one grain size analysis with hydrometer and natural moisture content, and one Atterberg limits test. 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 granular fill, underlain by stream alluvium and marine sand. The fill unit and subsurface soils are underlain by metamorphic bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

## 5.1 Fill

A layer of fill was encountered in the borings. The thickness of the fill unit was approximately 9 feet in both borings. The fill encountered generally consisted of:

- Light brown, damp, silty fine to medium sand, trace gravel;
- Brown, damp, gravelly sand, trace silt;
- Brown, moist, sand, little silt, little gravel.

SPT $N_{60}$-values in the fill unit ranged from 6 to 7 blows per foot (bpf), indicating the fill is loose in consistency. One grain size analysis of a sample from the fill unit resulted in a classification of A-4 under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 14 percent.

## 5.2 Stream Alluvium

Stream alluvium was encountered beneath the fill unit in the borings. The thickness of the alluvial deposit was approximately 5 to 14 feet thick and the deposit generally consisted of:

- Grey-brown, wet, silty fine to medium sand, some organics, trace wood;
- Grey-brown, wet, fine to medium sand, some silt, trace organics;
- Grey-dark brown, wet, silty fine sand, trace organic fibers;
- Mottled grey-brown, moist, silt, some sand, trace clay, trace gravel, trace organics.

SPT $N_{60}$-values in the alluvium deposit ranged from 1 to 9 bpf indicating that the alluvium is very soft or very loose to loose in consistency. Two grain size analyses conducted on samples from the alluvium deposit resulted in classifications of A-2-4 and A-4 under the AASHTO Soil Classification System and SM and ML under the USCS. The moisture contents of the samples tested were approximately 45 and 48 percent. An Atterberg limits test conducted on a silt sample of the alluvium returned a result of non-plastic.

### 5.3 Marine Sands

A deposit of marine sands was encountered beneath the stream alluvium material in the borings. The thickness of the deposit encountered ranged from approximately 27 to 35 feet. The deposit generally consisted of:

- Brown, grey-brown or grey, wet, fine sand, trace silt;
- Brown or grey, moist to wet, fine to medium sand, trace silt;
- Grey-brown, moist, sand;
- Brown-grey, moist, silty fine to medium sand, trace rounded gravel;
- Reddish-brown, wet, fine to medium sand, some silt;
- Brown, wet, silt, some sand;

SPT $N_{60}$-values ranged from 9 to 77 bpf indicating that the marine sand is loose to very dense in consistency. Five grain size analyses with water content were conducted resulting in the soils being classified as A-3, A-2-4, A-4 and A-3 under the AASHTO Soil Classification System and SP-SM, SM, CL and SP under the USCS. The moisture content of the samples tested ranged from approximately 22 to 24 percent.

### 5.4 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-CNS-101 | 14+31.8 | 7.0 R | 50.2 | 296.7 | 73, 83 |
| BB-CNS-102 | 15+12.7 | 6.5 L | 49.0 | 299.6 | 50, 97 |

**Table 1** – Summary of Approximate Bedrock Depths and Elevations

The bedrock recovered from the borings is generally identified as grey/greenish-grey, fine grained, limey pelite, hard, fresh, joint set horizontal to moderately dipping, close to widely spaced, tight to slightly open. The RQD of the bedrock ranged from 50 to 97 percent correlating to a rock mass quality of poor to very good. 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.

### 5.5 Groundwater

Groundwater was observed ranging from 12.0 to 14.0 feet bgs during the subsurface investigation. 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 draft February 2018 Preliminary Design Report (PDR) considered metalized steel girders, NEXT Type-F beams and butted concrete box beams for superstructure alternatives. Two foundation alternatives were considered: pile-supported integral abutments and conventional, full-height abutments.

The proposed bridge will span between 75 and 80 feet with a metallized steel plate girder or NEXT Type-F beam superstructure on pile-supported integral abutments.

## 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 Dutch Gap 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 | Approximate Bottom Elevation of Proposed Abutment (feet) | Approximate Top of Intact Bedrock Elevation (feet) | Estimated Pile Lengths (feet) |
|---|---|---|---|
| Abutment No. 1 | 337.0 | 296.7 | 40.3 |
| Abutment No. 2 | 339.0 | 299.6 | 39.4 |

**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 or pile cap.

### 7.1.1 Strength Limit State Design

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

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

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

Per AASHTO LRFD Bridge Design Specifications 7$^{th}$ Edition with interim revisions through 2016 (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$ (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for 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 $\phi_c = 0.70$ and the flexural resistance factor $\phi_f = 1.0$ shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). 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 $\phi_c$, 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 Report, Turner, Rock-Socketed Shafts for Highway Structure Foundations, 2006.

| Pile Section | Strength Limit State Factored Axial Pile Resistance | | | | |
|---|---|---|---|---|---|
| | Structural Resistance[2] $\phi_c$=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] | 147 | 387 | 299 | 299 |
| HP 14 x 73 | 535[6] | 202 | 535 | 377 | 377 |
| HP 14 x 89 | 652 | 247 | 652 | 403 | 403 |
| HP 14 x 117 | 860 | 326 | 860 | 527 | 527 |

**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.

### 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

2 Structural resistances were calculated for a braced pile segment in pure axial compression, using a resistance factor, $\phi_c$, 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, $\varphi_{dyn,}$ 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.

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.

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, $\phi$, 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, $\varphi_{up}$, 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[7] $\phi_c$=1.0 (kips) | Static Geotechnical Resistance [8] $\phi$ = 1.0 (kips) | Controlling Geotechnical Resistance[9] $\phi$=1.0 (kips) | Drivability Resistance $\phi$= 1.0 (kips) | Governing Axial Pile Resistance (kips) |
| HP 12 x 53 | 775 [10] | 326 | 775 | 460 | 460 |
| HP 14 x 73 | 1070[10] | 450 | 1070 | 580 | 580 |
| HP 14 x 89 | 1305 | 549 | 1305 | 620 | 620 |
| HP 14 x 117 | 1720 | 723 | 1720 | 810 | 810 |

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

[7] 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.

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

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

[10] 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).

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.

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 Table 5) 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 – 333 | Above | .072 (125) | 90 | 32° |
| Very loose, SAND. | 333-324 | Below | .042 (73) | 20 | 28° |
| Loose, SAND. | 324 – 317 | Below | .031 (53) | 20 | 30° |
| Medium dense, SAND. | 317 – 307 | Below | .040 (68) | 60 | 31° |
| Medium dense, SAND. | 307 – 296.7 | Below | .043 (75) | 60 | 33° |

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

### 7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching at each abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. 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, $\phi_{dyn}$, 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 ($\phi$) 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, $\phi$, 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 ($\phi$) of 32 degrees, total unit weight ($\gamma$) of 125 pcf, and a soil-concrete interface friction angle ($\delta$) 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 ($\gamma_{EH}$) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ($h_{eq}$) taken from Table 6:

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

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

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

Backfill within 10 feet of the abutments and side slope fill shall conform to MaineDOT Specification 703.19 – Granular Borrow for Underwater Backfill. The gradation of this material specifies 7 percent or less of the material passing the No. 200 sieve. Limiting the amount of fines is intended to 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 ($\gamma_{EH}$) 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

Approximately 5 to 14 feet of very loose to loose, silty sand and soft silt (stream alluvium) was encountered beneath the fill in the approaches to the existing bridge. The stream alluvium is underlain by loose to medium dense, marine sand. The project calls for an increase in the vertical profile of approximately 8 inches. Settlement calculations due to the proposed raise in grade have not been performed at this time.

To mitigate potential settlement, we recommend excavation of the soft silt layer encountered at proposed Abutment No. 2 to Elev. 334 and replacement with compacted granular borrow.

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, Chesterville has a design freezing index (DFI) of approximately 1775 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, Orono, Maine has a DFI from the Modberg database of approximately 1588 F-degree days. Orono was selected because it lies along the same isoline as Chesterville and Chesterville is not available in the Modberg database. A water content of 10% was used. These components correlate to a frost depth of approximately 6.5 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 draft PDR notes a scour hole immediately downstream of the existing bridge, and the stream channel exhibits bank erosion and scour damage. Grain size analyses were performed on samples from the stream alluvium and marine sand deposit to generate grain size curves for determining parameters to be used in scour analyses. Sample BB-CNS-101 3D was judged to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameter can be used in scour analyses:

- Average diameter of particle at 50 percent passing, $D_{50}$ = 0.1 mm (fine sand).

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 3-foot 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 7:

| Parameter | Design Value |
|---|---|
| Peak Ground Acceleration (PGA) | 0.082g |
| Acceleration Coefficient ($A_S$) | 0.206g |
| $S_{DS}$ (Period = 0.2 sec) | 0.426g |
| $S_{D1}$ (Period = 1.0 sec) | 0.17g |
| Site Class | E |
| Seismic Zone | 2 |

**Table 7 –** 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

The loose silty sands and saturated, loose marine sands pose constructability issues. Vibrating or driving temporary sheeting and piles may result in ground subsidence and ground loss. It is possible there may be subsidence associated with the withdrawal of temporary sheeting and shoring materials. Construction-phase dewatering to limit vibration-induced disturbance and settlement of the alluvium and marine sand deposits may be required.

Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments. The abutments of a pre-existing bridge, built at some time prior to the existing double metal corrugated culverts, are likely buried in the existing approaches. The 1946 plans indicate the presence of stone masonry abutments and walls resting on timber grillage. Implications of the remnants of previous bridge structures buried in the approaches include difficulty installing and damage to temporary lateral earth support systems. The previous structure may impede construction efforts and the contractor should assume excavation and removal is necessary.

Construction of the proposed structure will require pile driving. The contractor should assume that the existing culverts and remnants of the previous abutments, if not removed entirely, will obstruct pile driving operations. The contractor shall be responsible for excavating those portions of existing structures that conflict with piles by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers.

The contractor should assume the use of conventional excavation methods, pre-augering, predrilling, rock chisels, or down-hole hammers is necessary to clear obstructions and allow pile driving activities. The contractor should assume difficult pile driving even after the removal of obstructions. Care should be taken to drive H-piles within allowable tolerances without damaging the H-piles.

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

## 8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application of the proposed replacement of Dutch Gap Bridge in Chesterville, 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.

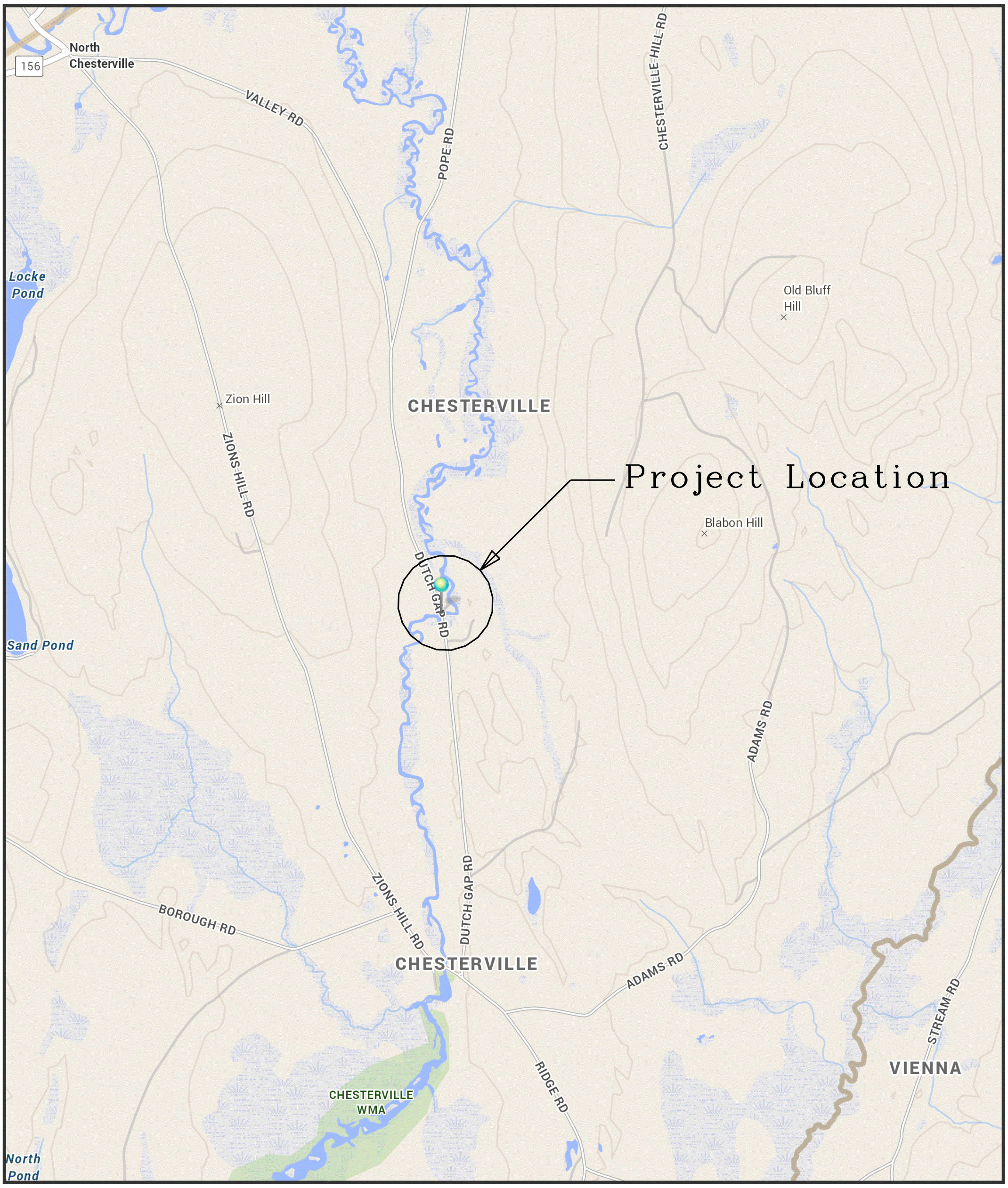
**Sheets**

Date:3/12/2018

Division: GEOTECH

Username: Terry.White

# CHESTERVILLE, 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.

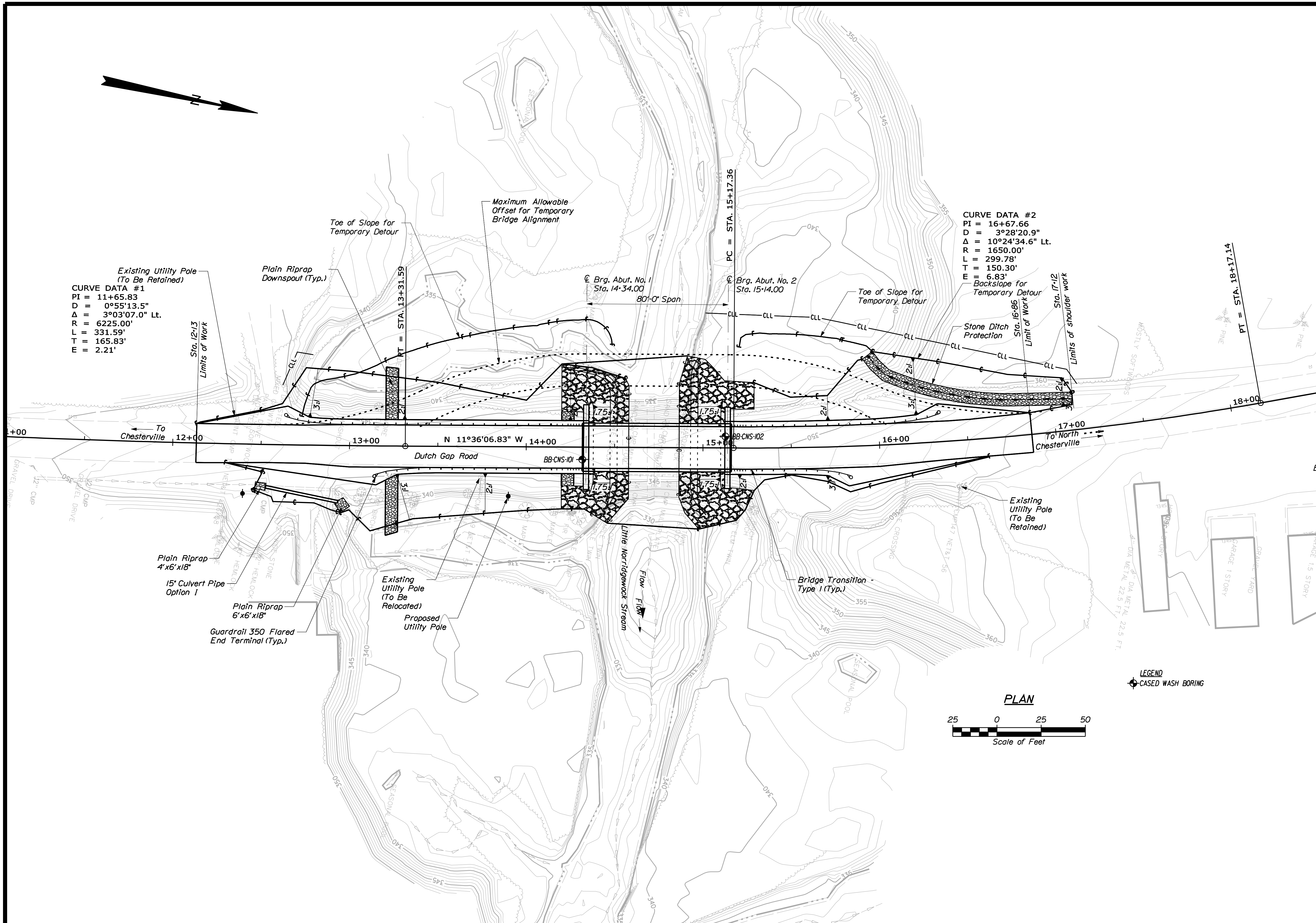
0.5 Miles

1 inch = 0.57 miles

Date: 3/12/2018
Time: 7:04:25 AM

| SHEET NUMBER | DUTCH GAP BRIDGE<br>LITTLE NORRIDGEWOCK STREAM<br>CHESTERVILLE FRANKLIN COUNTY | STATE OF MAINE<br>DEPARTMENT OF TRANSPORTATION |
|---|---|---|
| 1 | | STP-2166(800) |
| OF 4 | LOCATION MAP | WIN<br>BRIDGE NO. 3951 21688.00 BRIDGE PLANS |

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: Terry.White Date:4/6/2018




STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021688.00

WIN 21688.00

BRIDGE NO. 3951

BRIDGE PLANS

| | BY | DATE |
|---|---|---|
| PROJ. MANAGER | J.STILWELL | |
| DESIGN-DETAILED | | |
| CHECKED-REVIEWED | | |
| DESIGN2-DETAILED2 | ANDUSKRK | T.WHITE DEC 2017 |
| DESIGN3-DETAILED3 | | |
| REVISIONS 1 | | |
| REVISIONS 2 | | |
| REVISIONS 3 | | |
| REVISIONS 4 | | |
| FIELD CHANGES | | |

SIGNATURE

P.E. NUMBER

DATE

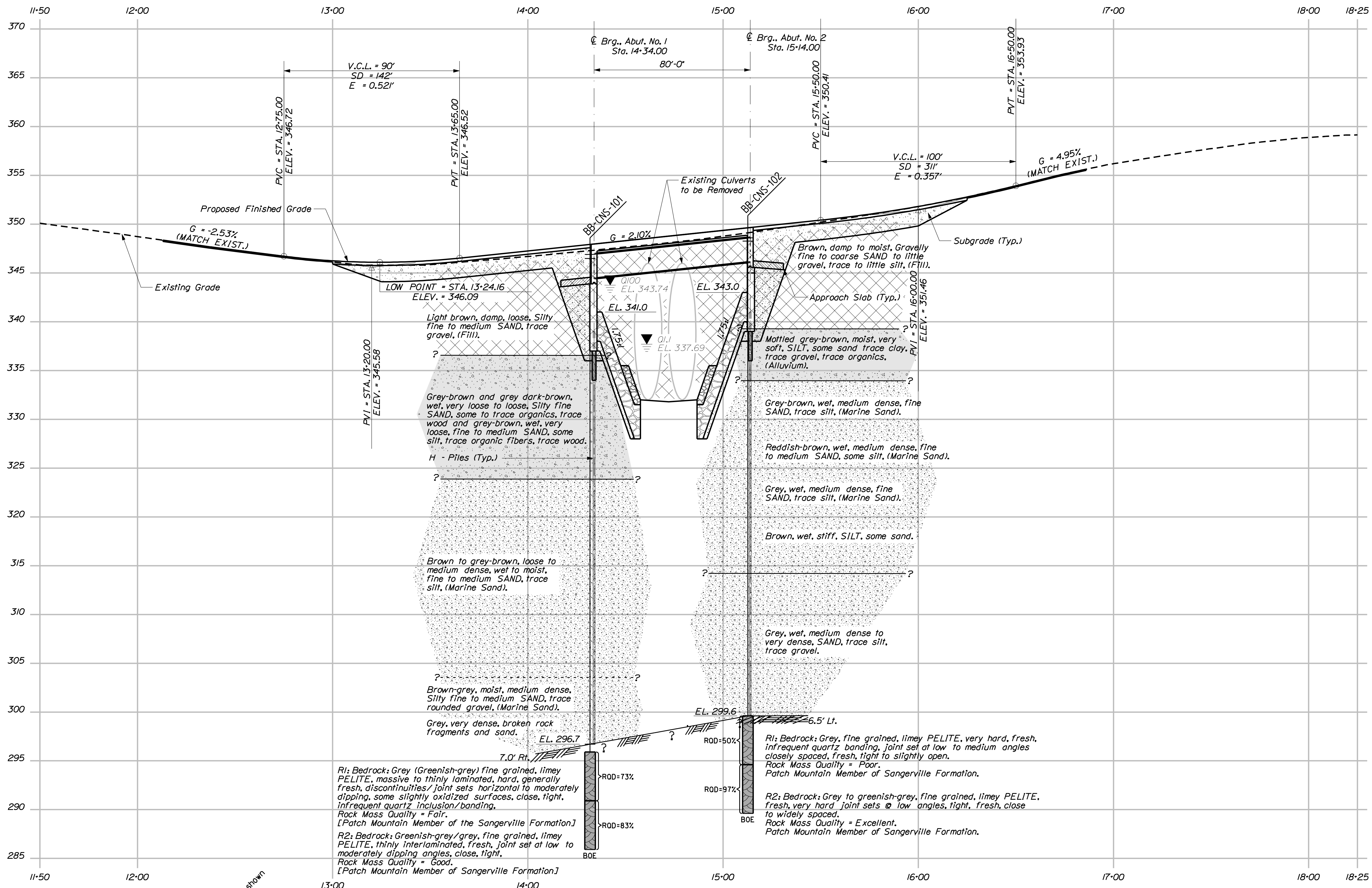
DUTCH GAP BRIDGE
LITTLE NORRIDGEWOCK STREAM
CHESTERVILLE FRANKLIN COUNTY

# BORING LOCATION PLAN

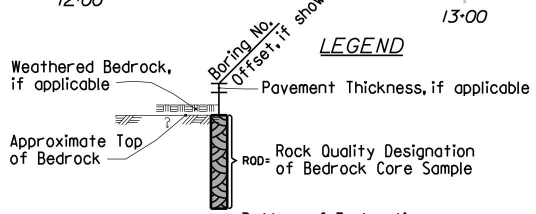
SHEET NUMBER

2

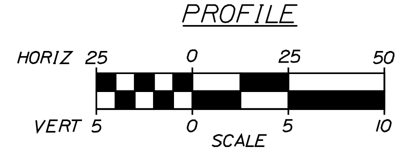
OF 4




## LEGEND




## PROFILE




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

021688.00

WIN 21688.00

BRIDGE PLANS

BRIDGE NO. 3951

| | | |
|---|---|---|
| PROJ. MANAGER | J.STILWELL | |
| DESIGN-DETAILED | | |
| CHECKED-REVIEWED | | |
| DESIGN2-DETAILED2 | VANBUSKIRK | DEC 2017 |
| DESIGN3-DETAILED3 | T.WHITE | |
| REVISIONS 1 | | |
| REVISIONS 2 | | |
| REVISIONS 3 | | |
| REVISIONS 4 | | |
| FIELD CHANGES | | |

BY / DATE

SIGNATURE

P.E. NUMBER

DATE

DUTCH GAP BRIDGE
LITTLE NORRIDGEWOCK STREAM
CHESTERVILLE FRANKLIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\MSTA\008_BORING LOGS1.dgn    Division: GEOTECH    Username: Terry.White    Date:4/10/2018

# Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little

Location: Chesterville, Maine

Boring No.: BB-CNS-101

WIN: 21688.00

| | | | | | |
|---|---|---|---|---|---|
| Driller: | MaineDOT | Elevation (ft.) | 346.9 | Auger ID/OD: | 5" Solid Stem |
| Operator: | Travis/James | Datum: | NAVD88 | Sampler: | Standard Split Spoon |
| Logged By: | B. Wilder | Rig Type: | CME 45C | Hammer Wt./Fall: | 140#/30" |
| Date Start/Finish: | 9/11/2017-9/12/2017 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ-2" |
| Boring Location: | 14+31.8, 7.0 ft Rt. | Casing ID/OD: | NW-3" | Water Level*: | 12.0 ft bgs. |
| Hammer Efficiency Factor: 0.854 | | 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

### Sample Information

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|---|---|---|---|
| 5 | 1D | 24/15 | 5.00 - 7.00 | 4/3/2/2 | 5 | 7 | G#300251 A-4, SM WC=14.0% |
| 10 | 2D | 24/14 | 10.50 - 12.50 | 1/2/4/3 | 6 | 9 | |
| 15 | 3D | 24/18 | 15.00 - 17.00 | 1/1/1/2 | 2 | 3 | G#300253 A-2-4, SM WC=47.5% |
| 20 | 4D | 24/18 | 20.00 - 22.00 | WOR/1/1/1 | 2 | 3 | |
| 25 | 5D | 24/15 | 25.00 - 27.00 | 2/2/4/5 | 6 | 9 | |
| 30 | 6D | 24/12 | 30.00 - 32.00 | 4/4/6/8 | 10 | 14 | G#300255 A-3, SP-SM WC=25.8% |
| 35 | 7D | 24/13 | 35.00 - 37.00 | 3/4/6/6 | 10 | 14 | |
| 40 | 8D | 24/13 | 40.00 - 42.00 | 4/4/10/15 | 14 | 20 | G#300256 A-3, SP-SM WC=22.5% |
| 45 | 9D | 24/14 | 45.00 - 47.00 | 4/7/7/7 | 14 | 20 | |
| 50 | 10D | 2.4/1 | 50.00 - 50.20 | 50(2.4") | --- | | |
| 51 | R1 | 60/60 | 51.00 - 56.00 | RQD = 73% | | | |
| 56 | R2 | 60/60 | 56.00 - 61.00 | RQD = 83% | | | |

Casing Blows (per ft, from 0 ft): SSA (0-5 ft); 10, 13, 13, 14, 11, 3, 8, 13, 18, 28, 18, 20, 28, 29, 31, 30, 31, 38, 54, 69, 33, 33, 42, 49, 64, 41, 47, 52, 76, 79, 40, 57, 88, 96, 158, 39, 93, 108, 100, 123, 46, 66, 77, 271; NQ-2 (51-61 ft)

### Visual Description and Remarks

| Elevation (ft.) | Depth (ft.) | Visual Description and Remarks |
|---|---|---|
| 346.5 | 0.4 | 5" HMA. |
| | | Light brown, damp, loose, Silty fine to medium SAND, trace gravel, (Fill). |
| 337.9 | 9.0 | Casing sunk to 10.5 ft bgs. Grey-brown, wet, loose, Silty fine to medium SAND, some organics, trace wood, (Alluvium). |
| | | Grey-brown, wet, very loose, fine to medium SAND, some silt, trace organic fibers, trace wood. |
| | | Grey-dark brown, wet, very loose, Silty fine SAND, trace organic fibers. |
| 323.9 | 23.0 | Change to grey at 23.0 ft bgs. |
| | | Brown, wet, loose, fine SAND, trace silt, (Marine Sand). |
| | | Brown, moist, medium dense, fine to medium SAND, trace silt, (Marine Sand). |
| | | Grey-brown, moist, medium dense, fine to medium SAND, (Marine Sand). |
| | | Sands running, filling casing. |
| | | Grey-brown, moist, medium dense, fine to medium SAND, trace silt. |
| 303.4 | 43.5 | Brown-grey, moist, medium dense, Silty fine to medium SAND, trace rounded gravel, (Marine Sand). |
| | | Roller Coned ahead to 50.0 ft bgs. |
| 296.7 | 50.2 | Grey, very dense, broken rock fragments and sand. |
| 295.9 | 51.0 | Top of Bedrock at Elev. 296.7 ft. Roller Coned ahead to 51.0 ft bgs. R1: Bedrock: Grey (Greenish-grey), fine grained, limey PELITE, massive to thinly laminated, hard, generally fresh, discontinuities/joint sets horizontal to moderately dipping, some slightly oxidized surfaces, close, tight, infrequent quartz inclusion/banding. Rock Mass Quality = Fair. Patch Mountain Member of the Sangerville Formation. R1: Core Times (min:sec) 51.0-52.0 ft (1:49) 52.0-53.0 ft (1:54) 53.0-54.0 ft (1:50) 54.0-55.0 ft (2:28) 55.0-56.0 ft (3:00) 100% Recovery. R2: Bedrock: Greenish-grey/grey, fine grained, limey PELITE, thinly interlaminated, fresh, joint set at low to moderately dipping angles, close, tight. Rock Mass Quality = Good. Patch Mountain Member of Sangerville Formation. R2: Core Times (min:sec) 56.0-57.0 ft (2:07) 57.0-58.0 ft (2:09) 58.0-59.0 ft (2:11) 59.0-60.0 ft (2:39) 60.0-61.0 ft (2:40) 100% Recovery |
| 285.9 | 61.0 | Bottom of Exploration at 61.0 feet below ground surface. |

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CNS-101

# Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little

Location: Chesterville, Maine

Boring No.: BB-CNS-102

WIN: 21688.00

| | | | | | |
|---|---|---|---|---|---|
| Driller: | MaineDOT | Elevation (ft.) | 348.6 | Auger ID/OD: | 5" Solid Stem |
| Operator: | Travis/James | Datum: | NAVD88 | Sampler: | Standard Split Spoon |
| Logged By: | B. Wilder | Rig Type: | CME 45C | Hammer Wt./Fall: | 140#/30" |
| Date Start/Finish: | 9/12/2017; 10:00-15:30 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ-2" |
| Boring Location: | 15+12.7, 6.5 ft Lt. | Casing ID/OD: | NW-3" | Water Level*: | 14.0 ft bgs. |
| Hammer Efficiency Factor: 0.854 | | 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

### Sample Information

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|---|---|---|---|
| 5 | 1D | 24/4 | 5.00 - 7.00 | 2/2/2/2 | 4 | 6 | |
| 10 | 2D | 24/16 | 10.00 - 12.00 | WOH/WOH/1/1 | 1 | 1 | G#300257 A-4, ML WC=44.7% Non-Plastic |
| 15 | 3D | 24/20 | 15.00 - 17.00 | 4/6/5/6 | 11 | 16 | |
| 20 | 4D | 24/17 | 20.00 - 22.00 | 3/4/5/8 | 9 | 13 | G#300258 A-2-4, SM WC=24.0% |
| 25 | 5D | 24/20 | 25.00 - 27.00 | 3/4/6/9 | 10 | 14 | |
| 30 | 6D | 24/22 | 30.00 - 32.00 | 3/3/5/6 | 8 | 11 | G#300259 A-4, CL WC=22.4% |
| 35 | 7D | 24/17 | 35.00 - 37.00 | 4/6/8/9 | 14 | 20 | |
| 40 | 8D | 24/19 | 40.00 - 42.00 | 3/4/5/7 | 9 | 13 | G#300260 A-3, SP WC=23.2% |
| 45 | 9D | 18/18 | 45.00 - 46.50 | 10/16/38 | 54 | 77 | |
| 49 | R1 | 60/55 | 49.00 - 54.00 | RQD = 50% | | | |
| 54 | R2 | 60/60 | 54.00 - 59.00 | RQD = 97% | | | |

Casing Blows (per ft, from 0 ft): SSA (0-5 ft); 7, 9, 8, 4, 3, 3, 5, 9, 12, 22, 18, 21, 35, 44, 44, 18, 20, 31, 47, 60, 20, 26, 49, 62, 71, 24, 28, 46, 43, 64, 35, 36, 51, 87, 93, 19, 41, 65, 72, 92, 22, 28, 37, 40; NQ-2 (49-59 ft)

### Visual Description and Remarks

| Elevation (ft.) | Depth (ft.) | Visual Description and Remarks |
|---|---|---|
| 348.3 | 0.3 | 4" HMA. |
| | | Brown, damp, Gravelly fine to coarse SAND, trace silt, (Fill). |
| 344.6 | 4.0 | Brown, moist, loose, fine to coarse SAND, little silt, little gravel, (Fill). |
| 339.6 | 9.0 | Mottled grey-brown, moist, very soft, SILT, some sand, trace clay, trace gravel, trace organics, (Alluvium). |
| 334.6 | 14.0 | Grey-brown, wet, medium dense, fine SAND, trace silt, (Marine Sands). |
| | | Reddish-brown, wet, medium dense, fine to medium SAND, some silt. |
| | | Grey, wet, medium dense, fine SAND, trace silt, (Marine Sands). |
| | | Brown, wet, stiff, SILT, some sand. |
| 314.6 | 34.0 | Grey, wet, medium dense, fine to medium SAND, trace silt. |
| | | Grey, wet, medium dense, SAND, trace silt, trace gravel. |
| | | Similar to above, except very dense. Roller Coned ahead to 49.0 ft bgs. |
| 299.6 | 49.0 | Top of Bedrock at Elev. 299.6 ft. R1: Bedrock: Grey, fine grained, limey PELITE, very hard, fresh, joint set at low to medium angles, closely spaced, tight to slightly open, infrequent quartz banding. Rock Mass Quality = Poor. Patch Mountain Member of Sangerville Formation. R1: Core Times (min:sec) 49.0.-50.0 ft (1:11) 50.0-51.0 ft (1:36) 51.0-52.0 ft (0:58) 52.0-53.0 ft (0:56) 53.0-54.0 ft (1:11) 83% Recovery. No water return. R2: Bedrock: Grey to greenish-grey, fine grained, limey PELITE, very hard joint sets at low angles, fresh, close to widely spaced, tight. Rock Mass Quality = Excellent. Patch Mountain Member of Sangerville Formation. R2: Core Times (min:sec) 54.0-55.0 ft (1:20) 55.0-56.0 ft (1:33) 56.0-57.0 ft (1:54) 57.0-58.0 ft (2:06) 58.0-59.0 ft (2:04) 100% Recovery. No water return. |
| 289.6 | 59.0 | Bottom of Exploration at 59.0 feet below ground surface. |

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CNS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021688.00

BRIDGE NO. 3951    WIN 21688.00    BRIDGE PLANS

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

SIGNATURE

P.E. NUMBER

DATE

DUTCH GAP BRIDGE
LITTLE NORRIDGEWOCK STREAM
CHESTERVILLE    FRANKLIN COUNTY

## BORING LOGS

SHEET NUMBER

4

OF 4

# Appendix A

Boring Logs

## UNIFIED SOIL CLASSIFICATION SYSTEM

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

**Desired Soil Observations (in this order, 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):**

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

*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:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream
**Location:** Chesterville, Maine

**Boring No.:** BB-CNS-101
**WIN:** 21688.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 346.9 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/11/2017-9/12/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 14+31.8, 7.0 ft Rt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 12.0 ft bgs. |
| **Hammer Efficiency Factor:** 0.854 | | **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 | 346.5 | | 5" HMA. 0.4 | |
| 5 | 1D | 24/15 | 5.00 - 7.00 | 4/3/2/2 | 5 | 7 | 10 | | | Light brown, damp, loose, Silty fine to medium SAND, trace gravel, (Fill). | G#300251 A-4, SM WC=14.0% |
| | | | | | | | 13 | | | | |
| | | | | | | | 13 | | | | |
| | | | | | | | 14 | 337.9 | | 9.0 | |
| | | | | | | | 11 | | | | |
| 10 | 2D | 24/14 | 10.50 - 12.50 | 1/2/4/3 | 6 | 9 | 3 | | | Casing sunk to 10.5 ft bgs. Grey-brown, wet, loose, Silty fine to medium SAND, some organics, trace wood, (Alluvium). | |
| | | | | | | | 8 | | | | |
| | | | | | | | 13 | | | | |
| | | | | | | | 18 | | | | |
| | | | | | | | 28 | | | | |
| 15 | 3D | 24/18 | 15.00 - 17.00 | 1/1/1/2 | 2 | 3 | 18 | | | Grey-brown, wet, very loose, fine to medium SAND, some silt, trace organic fibers, trace wood. | G#300253 A-2-4, SM WC=47.5% |
| | | | | | | | 20 | | | | |
| | | | | | | | 28 | | | | |
| | | | | | | | 29 | | | | |
| | | | | | | | 31 | | | | |
| 20 | 4D | 24/18 | 20.00 - 22.00 | WOR/1/1/1 | 2 | 3 | 30 | | | Grey-dark brown, wet, very loose, Silty fine SAND, trace organic fibers. | |
| | | | | | | | 31 | | | | |
| | | | | | | | 38 | 323.9 | | 23.0 | |
| | | | | | | | 54 | | | Change to grey at 23.0 ft bgs. | |
| 25 | | | | | | | 69 | | | | |

**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**<br>Soil/Rock Exploration Log<br>US CUSTOMARY UNITS | **Project:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream<br>**Location:** Chesterville, Maine | **Boring No.:** BB-CNS-101<br>**WIN:** 21688.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 346.9 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/11/2017-9/12/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 14+31.8, 7.0 ft Rt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 12.0 ft bgs. |
| **Hammer Efficiency Factor:** 0.854 | | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
$S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
$N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency
$N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected
$T_v$ = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample 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 | 24/15 | 25.00 - 27.00 | 2/2/4/5 | 6 | 9 | 33 | | | Brown, wet, loose, fine SAND, trace silt, (Marine Sand). | |
| | | | | | | | 33 | | | | |
| | | | | | | | 42 | | | | |
| | | | | | | | 49 | | | | |
| | | | | | | | 64 | | | | |
| 30 | 6D | 24/12 | 30.00 - 32.00 | 4/4/6/8 | 10 | 14 | 41 | | | Brown, moist, medium dense, fine to medium SAND, trace silt, (Marine Sand). | G#300255<br>A-3, SP-SM<br>WC=25.8% |
| | | | | | | | 47 | | | | |
| | | | | | | | 52 | | | | |
| | | | | | | | 76 | | | | |
| | | | | | | | 79 | | | | |
| 35 | 7D | 24/13 | 35.00 - 37.00 | 3/4/6/6 | 10 | 14 | 40 | | | Grey-brown, moist, medium dense, fine to medium SAND, (Marine Sand). | |
| | | | | | | | 57 | | | | |
| | | | | | | | 88 | | | | |
| | | | | | | | 96 | | | | |
| | | | | | | | 158 | | | Sands running, filling casing. | |
| 40 | 8D | 24/13 | 40.00 - 42.00 | 4/4/10/15 | 14 | 20 | 39 | | | Grey-brown, moist, medium dense, fine to medium SAND, trace silt. | G#300256<br>A-3, SP-SM<br>WC=22.5% |
| | | | | | | | 93 | | | | |
| | | | | | | | 108 | | | | |
| | | | | | | | 100 | 303.4 | | 43.5 | |
| | | | | | | | 123 | | | | |
| 45 | 9D | 24/14 | 45.00 - 47.00 | 4/7/7/7 | 14 | 20 | 46 | | | Brown-grey, moist, medium dense, Silty fine to medium SAND, trace rounded gravel, (Marine Sand). | |
| | | | | | | | 66 | | | | |
| | | | | | | | 77 | | | | |
| | | | | | | | 271 | | | | |
| 50 | | | | | | | | | | Roller Coned ahead to 50.0 ft bgs. | |

**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:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream
**Location:** Chesterville, Maine

**Boring No.:** BB-CNS-101
**WIN:** 21688.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 346.9 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/11/2017-9/12/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 14+31.8, 7.0 ft Rt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 12.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.854 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
$S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
$N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency
$N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected
$T_v$ = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 10D | 2.4/1 | 50.00 - 50.20 | 50(2.4") | --- | | | 296.7 | | Grey, very dense, broken rock fragments and sand. (50.2) | |
| | R1 | 60/60 | 51.00 - 56.00 | RQD = 73% | | | NQ-2 | 295.9 | | Top of Bedrock at Elev. 296.7 ft. Roller Coned ahead to 51.0 ft bgs. (51.0) R1: Bedrock: Grey (Greenish-grey), fine grained, limey PELITE, massive to thinly laminated, hard, generally fresh, discontinuities/joint sets horizontal to moderately dipping, some slightly oxidized surfaces, close, tight, infrequent quartz inclusion/banding, Rock Mass Quality = Fair. Patch Mountain Member of the Sangerville Formation R1: Core Times (min:sec) 51.0-52.0 ft (1:49) 52.0-53.0 ft (1:54) 53.0-54.0 ft (1:50) 54.0-55.0 ft (2:28) 55.0-56.0 ft (3:00) 100% Recovery | |
| 55 | | | | | | | | | | | |
| | R2 | 60/60 | 56.00 - 61.00 | RQD = 83% | | | | | | R2: Bedrock: Greenish-grey/grey, fine grained, limey PELITE, thinly interlaminated, fresh, joint set at low to moderately dipping angles, close, tight, Rock Mass Quality = Good. Patch Mountain Member of Sangerville Formation. R2: Core Times (min:sec) 56.0-57.0 ft (2:07) 57.0-58.0 ft (2:09) 58.0-59.0 ft (2:11) 59.0-60.0 ft (2:39) 60.0-61.0 ft (2:40) 100% Recovery | |
| 60 | | | | | | | | 285.9 | | (61.0) **Bottom of Exploration at 61.0 feet below ground surface.** | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |

**Remarks:**

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

| Maine Department of Transportation<br>Soil/Rock Exploration Log<br>US CUSTOMARY UNITS | Project: Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream<br>Location: Chesterville, Maine | Boring No.: BB-CNS-102<br>WIN: 21688.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 348.6 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/12/2017; 10:00-15:30 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 15+12.7, 6.5 ft Lt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.854 | **Hammer Type:** | Automatic ☒ | Hydraulic ☐ | Rope & Cathead ☐ |

Definitions:

| | | | |
|---|---|---|---|
| D = Split Spoon Sample | R = Rock Core Sample | $S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf) | $T_v$ = Pocket Torvane Shear Strength (psf) |
| MD = Unsuccessful Split Spoon Sample Attempt | SSA = Solid Stem Auger | $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf) | WC = Water Content, percent |
| U = Thin Wall Tube Sample | HSA = Hollow Stem Auger | $q_p$ = Unconfined Compressive Strength (ksf) | LL = Liquid Limit |
| MU = Unsuccessful Thin Wall Tube Sample Attempt | RC = Roller Cone | N-uncorrected = Raw Field SPT N-value | PL = Plastic Limit |
| V = Field Vane Shear Test, PP = Pocket Penetrometer | WOH = Weight of 140lb. Hammer | Hammer Efficiency Factor = Rig Specific Annual Calibration Value | PI = Plasticity Index |
| MV = Unsuccessful Field Vane Shear Test Attempt | WOR/C = Weight of Rods or Casing | $N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency | G = Grain Size Analysis |
| | WO1P = Weight of One Person | $N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected | C = Consolidation Test |

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | N60 | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 348.3 | | 4" HMA. 0.3<br>Brown, damp, Gravelly fine to coarse SAND, trace silt, (Fill). | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 5 | 1D | 24/4 | 5.00 - 7.00 | 2/2/2/2 | 4 | 6 | 7 | | | Brown, moist, loose, fine to coarse SAND, little silt, little gravel, (Fill). | |
| | | | | | | | 9 | | | | |
| | | | | | | | 8 | | | | |
| | | | | | | | 4 | | | | |
| | | | | | | | 3 | 339.6 | | 9.0 | |
| 10 | 2D | 24/16 | 10.00 - 12.00 | WOH/WOH/1/1 | 1 | 1 | 3 | | | Mottled grey-brown, moist, very soft, SILT, some sand, trace clay, trace gravel, trace organics, (Alluvium). | G#300257<br>A-4, ML<br>WC=44.7%<br>Non-Plastic |
| | | | | | | | 5 | | | | |
| | | | | | | | 9 | | | | |
| | | | | | | | 12 | | | | |
| | | | | | | | 22 | 334.6 | | 14.0 | |
| 15 | 3D | 24/20 | 15.00 - 17.00 | 4/6/5/6 | 11 | 16 | 18 | | | Grey-brown, wet, medium dense, fine SAND, trace silt, (Marine Sands). | |
| | | | | | | | 21 | | | | |
| | | | | | | | 35 | | | | |
| | | | | | | | 44 | | | | |
| | | | | | | | 44 | | | | |
| 20 | 4D | 24/17 | 20.00 - 22.00 | 3/4/5/8 | 9 | 13 | 18 | | | Reddish-brown, wet, medium dense, fine to medium SAND, some silt. | G#300258<br>A-2-4, SM<br>WC=24.0% |
| | | | | | | | 20 | | | | |
| | | | | | | | 31 | | | | |
| | | | | | | | 47 | | | | |
| | | | | | | | 60 | | | | |
| 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:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream
**Location:** Chesterville, Maine

**Boring No.:** BB-CNS-102
**WIN:** 21688.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 348.6 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/12/2017; 10:00-15:30 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 15+12.7, 6.5 ft Lt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.854 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
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$N_{60}$ = 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 | 24/20 | 25.00 - 27.00 | 3/4/6/9 | 10 | 14 | 20 | | | Grey, wet, medium dense, fine SAND, trace silt, (Marine Sands). | |
| | | | | | | | 26 | | | | |
| | | | | | | | 49 | | | | |
| | | | | | | | 62 | | | | |
| | | | | | | | 71 | | | | |
| 30 | 6D | 24/22 | 30.00 - 32.00 | 3/3/5/6 | 8 | 11 | 24 | | | Brown, wet, stiff, SILT, some sand. | G#300259 A-4, CL WC=22.4% |
| | | | | | | | 28 | | | | |
| | | | | | | | 46 | | | | |
| | | | | | | | 43 | | | | |
| | | | | | | | 64 | | | | |
| 35 | 7D | 24/17 | 35.00 - 37.00 | 4/6/8/9 | 14 | 20 | 35 | | | Grey, wet, medium dense, fine to medium SAND, trace silt. | |
| | | | | | | | 36 | | | | |
| | | | | | | | 51 | | | | |
| | | | | | | | 87 | | | | |
| | | | | | | | 93 | | | | |
| 40 | 8D | 24/19 | 40.00 - 42.00 | 3/4/5/7 | 9 | 13 | 19 | | | Grey, wet, medium dense, SAND, trace silt, trace gravel. | G#300260 A-3, SP WC=23.2% |
| | | | | | | | 41 | | | | |
| | | | | | | | 65 | | | | |
| | | | | | | | 72 | | | | |
| | | | | | | | 92 | | | | |
| 45 | 9D | 18/18 | 45.00 - 46.50 | 10/16/38 | 54 | 77 | 22 | | | Similar to above, except very dense. Roller Coned ahead to 49.0 ft bgs. | |
| | | | | | | | 28 | | | | |
| | | | | | | | 37 | | | | |
| | | | | | | | 40 | 299.6 | | 49.0 | |
| 50 | R1 | 60/55 | 49.00 - 54.00 | RQD = 50% | | | NQ-2 | | | Top of Bedrock at Elev. 299.6 ft. | |

**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:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream
**Location:** Chesterville, Maine

**Boring No.:** BB-CNS-102
**WIN:** 21688.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 348.6 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/12/2017; 10:00-15:30 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 15+12.7, 6.5 ft Lt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.854 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
$S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
$N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency
$N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected
$T_v$ = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | | | | | | | | | | R1: Bedrock: Grey, fine grained, limey PELITE, very hard, fresh, joint set at low to medium angles, closely spaced, tight to slightly open, infrequent quartz banding,<br>Rock Mass Quality = Poor.<br>Patch Mountain Member of Sangerville Formation.<br>R1: Core Times (min:sec)<br>49.0.-50.0 ft (1:11)<br>50.0-51.0 ft (1:36)<br>51.0-52.0 ft (0:58)<br>52.0-53.0 ft (0:56)<br>53.0-54.0 ft (1:11)<br>83% Recovery<br>No water return. | |
| | R2 | 60/60 | 54.00 - 59.00 | RQD = 97% | | | | | | R2: Bedrock: Grey to greenish-grey, fine grained, limey PELITE, very hard joint sets at low angles, fresh, close to widely spaced, tight,<br>Rock Mass Quality = Excellent.<br>Patch Mountain Member of Sangerville Formation.<br>R2: Core Times (min:sec)<br>54.0-55.0 ft (1:20)<br>55.0-56.0 ft (1:33)<br>56.0-57.0 ft (1:54)<br>57.0-58.0 ft (2:06)<br>58.0-59.0 ft (2:04)<br>100% Recovery<br>No water return. | |
| 55 | | | | | | | | | | | |
| | | | | | | | | 289.6 | | 59.0 | |
| 60 | | | | | | | | | | **Bottom of Exploration at 59.0 feet below ground surface.** | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |

**Remarks:**

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

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

# Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

## Laboratory Testing Summary Sheet

# Town(s): Chesterville Work Number: 21688.00

| 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-CNS-101, 1D | 14+31.8 | 7.0 Rt. | 5.0-7.0 | 300251 | 1 | 14.0 | | | SM | A-4 | III |
| BB-CNS-101, 3D | 14+31.8 | 7.0 Rt. | 15.0-17.0 | 300253 | 1 | 47.5 | | | SM | A-2-4 | II |
| BB-CNS-101, 6D | 14+31.8 | 7.0 Rt. | 30.0-32.0 | 300255 | 1 | 25.8 | | | SP-SM | A-3 | 0 |
| BB-CNS-101, 8D | 14+31.8 | 7.0 Rt. | 40.0-42.0 | 300256 | 1 | 22.5 | | | SP-SM | A-3 | 0 |
| BB-CNS-102, 2D | 15+12.7 | 6.5 Lt. | 10.0-12.0 | 300257 | 2 | 44.7 | -N | P- | ML | A-4 | IV |
| BB-CNS-102, 4D | 15+12.7 | 6.5 Lt. | 20.0-22.0 | 300258 | 2 | 24.0 | | | SM | A-2-4 | II |
| BB-CNS-102, 6D | 15+12.7 | 6.5 Lt. | 30.0-32.0 | 300259 | 2 | 22.4 | | | CL | A-4 | IV |
| BB-CNS-102, 8D | 15+12.7 | 6.5 Lt. | 40.0-42.0 | 300260 | 2 | 23.2 | | | SP | A-3 | 0 |

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

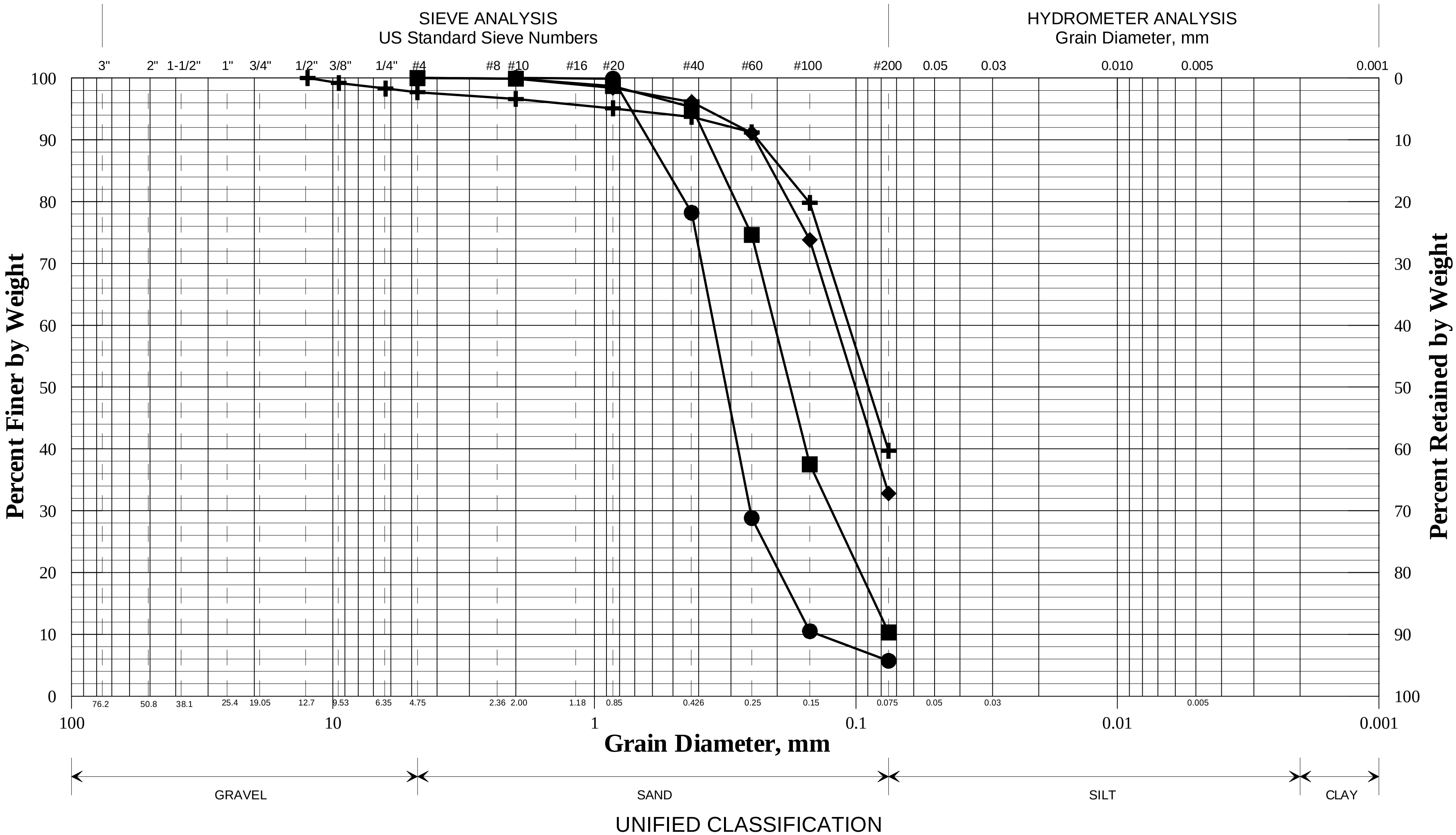
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

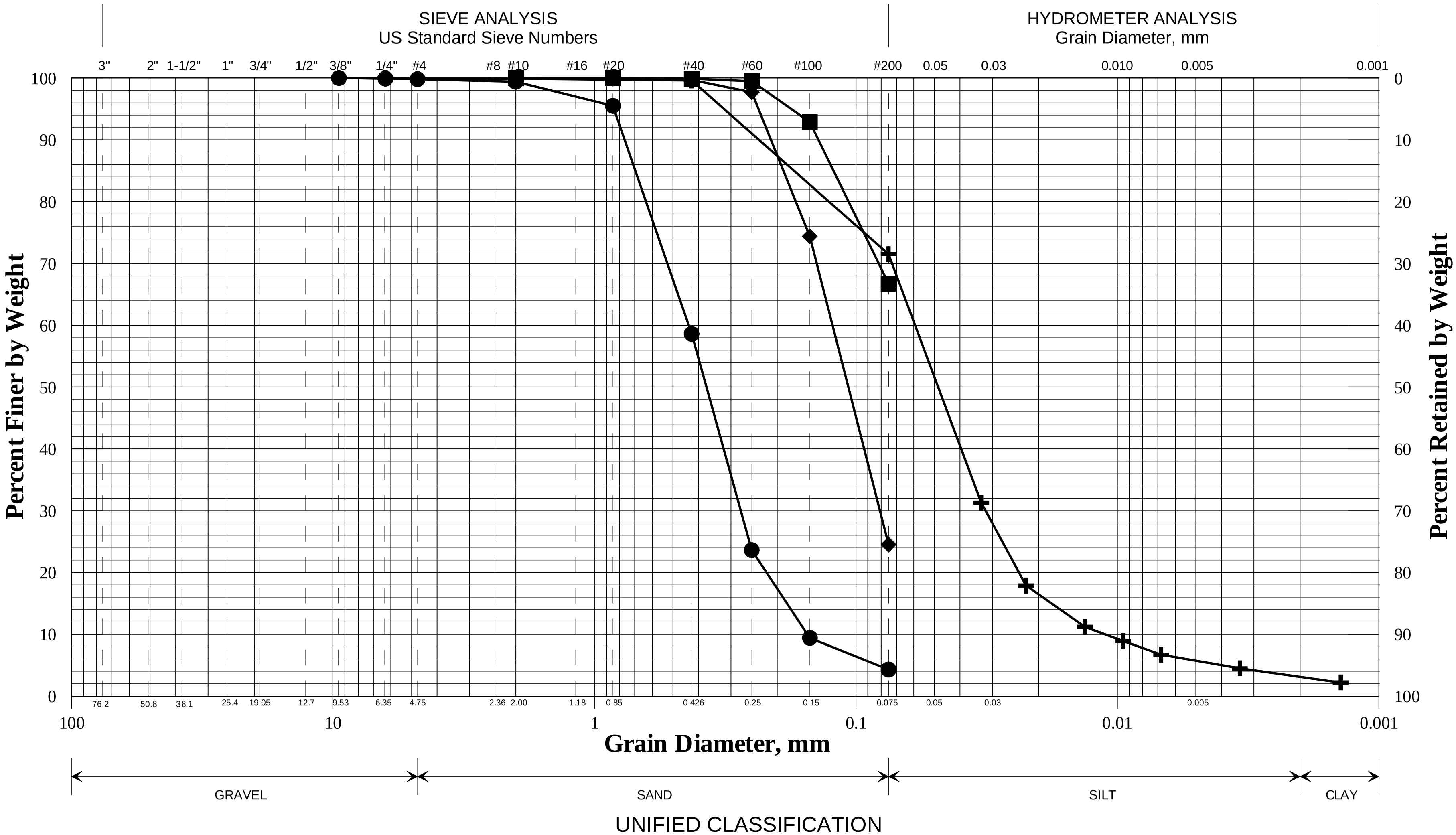



| | Boring/Sample No. | Station | Offset, ft | Depth, ft | Description | W, % | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|
| ✚ | BB-CNS-101/1D | 14+31.8 | 7.0 RT | 5.0-7.0 | Silty SAND, trace gravel. | 14.0 | | | |
| ◆ | BB-CNS-101/3D | 14+31.8 | 7.0 RT | 15.0-17.0 | SAND, some silt. | 47.5 | | | |
| ■ | BB-CNS-101/6D | 14+31.8 | 7.0 RT | 30.0-32.0 | SAND, trace silt. | 25.8 | | | |
| ● | BB-CNS-101/8D | 14+31.8 | 7.0 RT | 40.0-42.0 | SAND, trace silt. | 22.5 | | | |
| ▲ | | | | | | | | | |
| ✖ | | | | | | | | | |

| WIN |
|---|
| 021688.00 |
| **Town** |
| Chesterville |
| **Reported by/Date** |
| WHITE, TERRY A 10/4/2017 |

**SHEET 1**

# State of Maine Department of Transportation
## GRAIN SIZE DISTRIBUTION CURVE




| | Boring/Sample No. | Station | Offset, ft | Depth, ft | Description | W, % | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|
| ✚ | BB-CNS-102/2D | 15+12.7 | 6.5 LT | 10.0-12.0 | SILT, some sand, trace clay, trace gravel. | 44.7 | | | NP |
| ◆ | BB-CNS-102/4D | 15+12.7 | 6.5 LT | 20.0-22.0 | SAND, some silt. | 24.0 | | | |
| ■ | BB-CNS-102/6D | 15+12.7 | 6.5 LT | 30.0-32.0 | SILT, some sand. | 22.4 | | | |
| ● | BB-CNS-102/8D | 15+12.7 | 6.5 LT | 40.0-42.0 | SAND, trace silt, trace gravel. | 23.2 | | | |
| ▲ | | | | | | | | | |
| ✖ | | | | | | | | | |

| WIN |
|---|
| 021688.00 |
| Town |
| Chesterville |
| Reported by/Date |
| WHITE, TERRY A 10/4/2017 |

**SHEET 2**

MaineDOT

# GEOTECHNICAL TEST REPORT
## Central Laboratory

## SAMPLE INFORMATION

| Reference No. | Boring No./Sample No. | Sample Description | Sampled | Received |
|---|---|---|---|---|
| 300257 | BB-CNS-102/2D | GEOTECHNICAL (DISTURBED) | 9/12/2017 | 9/19/2017 |

Sample Type: **GEOTECHNICAL** Location: Station: **15+12.7** Offset, ft: **6.5** **LT** Dbfg, ft: **10.0-12.0**

WIN/Town **021688.00 - CHESTERVILLE** Sampler: **BRUCE WILDER**

## TEST RESULTS

**Sieve Analysis (T 88)**

Wash Method

| SIEVE SIZE U.S. [SI] | % Passing |
|---|---|
| 3 in. [75.0 mm] | |
| 1 in. [25.0 mm] | |
| ¾ in. [19.0 mm] | |
| ½ in. [12.5 mm] | |
| ⅜ in. [9.5 mm] | |
| ¼ in. [6.3 mm] | 100.0 |
| No. 4 [4.75 mm] | 99.9 |
| No. 10 [2.00 mm] | 99.9 |
| No. 20 [0.850 mm] | |
| No. 40 [0.425 mm] | 99.6 |
| No. 60 [0.250 mm] | |
| No. 100 [0.150 mm] | |
| No. 200 [0.075 mm] | 71.5 |
| [0.0332 mm] | 31.3 |
| [0.0224 mm] | 17.9 |
| [0.0133 mm] | 11.2 |
| [0.0095 mm] | 8.9 |
| [0.0068 mm] | 6.7 |
| [0.0034 mm] | 4.5 |
| [0.0014 mm] | 2.2 |

**Miscellaneous Tests**

| Test | Result |
|---|---|
| Liquid Limit @ 25 blows (T 89), % | |
| Plastic Limit (T 90), % | |
| Plasticity Index (T 90), % | NP |
| Specific Gravity, Corrected to 20°C (T 100) | 2.59 |
| Loss on Ignition, % (T 267) | |
| Water Content (T 265), % | 44.7 |

**Consolidation (T 216)**

Trimmings, Water Content, %

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

**Vane Shear Test on Shelby Tubes (Maine DOT)**

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

Comments:

## AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **9/29/2017**

*Paper Copy: Lab File; Project File; Geotech File*

# **Appendix C**

Calculations

H Pile Resistance

**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 |

| | |
|---|---|
| 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 |
|---|---|---|
| $\Phi_c$ | 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 |

**Compressive Strength of Bedrock Samples**

| Material | Boring | Sample | RQD | $q_u$ (ksi) | Source of $q_u$ |
|---|---|---|---|---|---|
| metasandstone | BB-CNS-101 | R1 | 73% | 8.41 | Wadsworth Street Bridge, Thomaston, Maine WIN 16755.00, lowest value taken from unconfined compression test of similar bedrock. |
| metasandstone/ metapelite | BB-CNS-101 | R2 | 83% | 8.41 | |
| | BB-CNS-102 | R1 | 50% | 8.41 | |
| | BB-CNS-102 | R2 | 100% | 8.41 | |

**Intact Rock Method**

| | depth (d) | flange width (b) | $A_{tip}$ | $q_{tip}$ = 2.5 * min($q_u$) | $R_{n\_tip} = A_{tip} * q_{tip}$ | $R_{r\_tip} = \varphi_{static} * R_n$<br>$\varphi_{static}$ = .45 |
|---|---|---|---|---|---|---|
| | in. | in. | in.$^2$ | ksi | kips | kips |
| 12x53 | 11.80 | 12.00 | 15.5 | 21.0 | 325.9 | 146.6 |
| 14x73 | 13.60 | 14.60 | 21.4 | 21.0 | 449.9 | 202.5 |
| 14x89 | 13.80 | 14.70 | 26.1 | 21.0 | 548.8 | 246.9 |
| 14x117 | 14.20 | 14.90 | 34.4 | 21.0 | 723.3 | 325.5 |

**Notes:**

For $\varphi_{static}$ see LRFD Table 10.5.5.2.3-1 CGS Method on Rock.

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.

## Controlling Geotechnical Resistance

| | Structural Resistance (kips) | $\phi_c$ | Controlling Geotechnical Resistance (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 $\phi_c$ 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. 40 ft. Assume contractor drives pile lengths of 50 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use constant shaft resistances so that GRLWeap will assign approx. 38 to 46 kips of the ultimate capacities as skin friction consistent with preliminary side resistance calculations per Meyerhof SPT-Method; (38 kips for 12x53 and 46 kips for 14x73, 14x89, 14x117).

Pile size = 12 x 53. Assume Contractor will use a Delmag D19-42 hammer with 2.70 kip helmet on fuel setting 4 (73%).

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke ft | Energy kips-ft |
|---|---|---|---|---|---|
| 200.0 | 24.18 | 1.11 | 2.9 | 5.56 | 11.49 |
| 300.0 | 31.62 | 3.25 | 5.3 | 6.26 | 12.77 |
| 400.0 | 38.56 | 5.52 | 10.0 | 7.05 | 14.38 |
| 410.0 | 39.16 | 5.67 | 10.6 | 7.13 | 14.53 |
| 420.0 | 39.77 | 5.78 | 11.2 | 7.20 | 14.71 |
| 430.0 | 40.40 | 5.84 | 11.9 | 7.28 | 14.88 |
| 450.0 | 41.53 | 5.90 | 13.6 | 7.42 | 15.14 |
| 460.0 | 42.11 | 5.94 | 14.5 | 7.49 | 15.31 |
| 470.0 | 42.65 | 5.97 | 15.6 | 7.56 | 15.44 |

Limit bpi to less than 15.

$R_{dr\_12x53} := 460 kip$

Strength Limit State

$\varphi_{dyn} := .65$

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

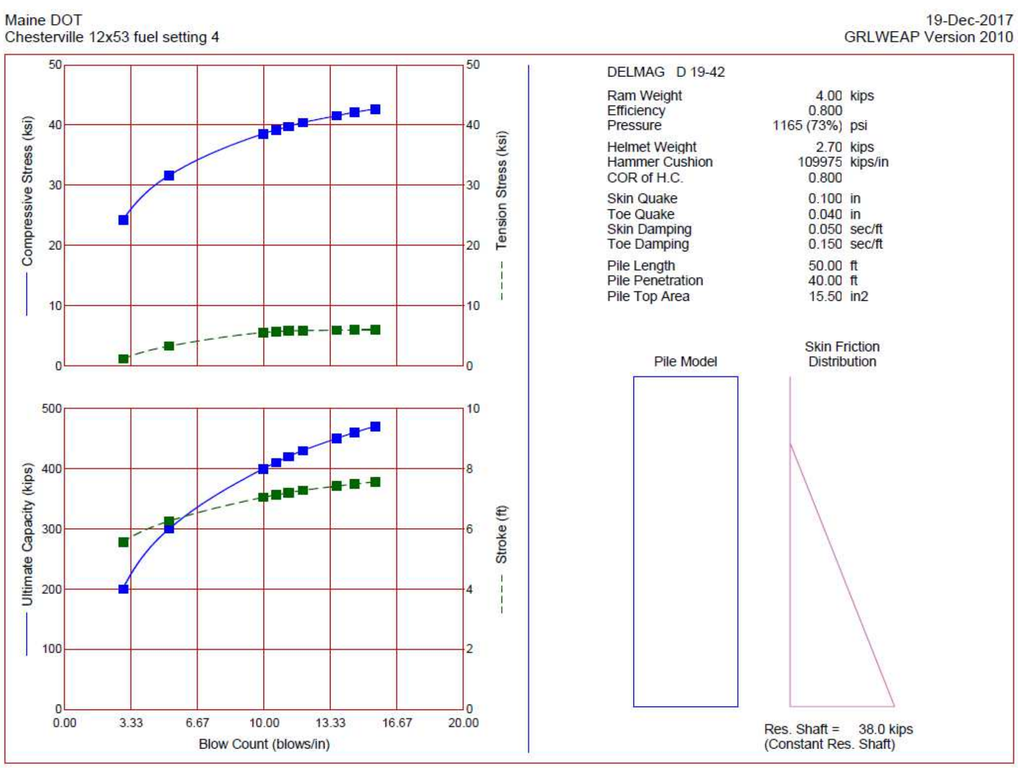
$R_{dr\_12x53\_strength} = 299 \cdot kip$

Service and Extreme Limit State:

$\varphi := 1.0$

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

$R_{dr\_12x53\_servext} = 460 \cdot kip$



Pile Size = 14 x 73. Assume Contractor will use a Delmag 19-42 with 2.70 kip helmet at fuel setting 2 (90%).

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke ft | Energy kips-ft |
|---|---|---|---|---|---|
| 200.0 | 23.37 | 0.69 | 2.3 | 6.77 | 15.48 |
| 400.0 | 35.68 | 4.22 | 5.5 | 8.15 | 17.47 |
| 500.0 | 41.16 | 4.31 | 8.5 | 8.82 | 19.10 |
| 510.0 | 41.70 | 4.50 | 8.9 | 8.88 | 19.28 |
| 530.0 | 42.62 | 4.86 | 9.7 | 9.00 | 19.57 |
| 540.0 | 43.09 | 5.07 | 10.1 | 9.06 | 19.76 |
| 560.0 | 44.00 | 5.48 | 11.0 | 9.17 | 20.01 |
| 570.0 | 44.43 | 5.70 | 11.5 | 9.23 | 20.17 |
| 580.0 | 44.88 | 5.93 | 12.0 | 9.29 | 20.35 |

Limit driving stress to less than 45 ksi.

$R_{dr\_14x73} := 580kip$

Strength Limit State

$\varphi_{dyn} := .65$

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

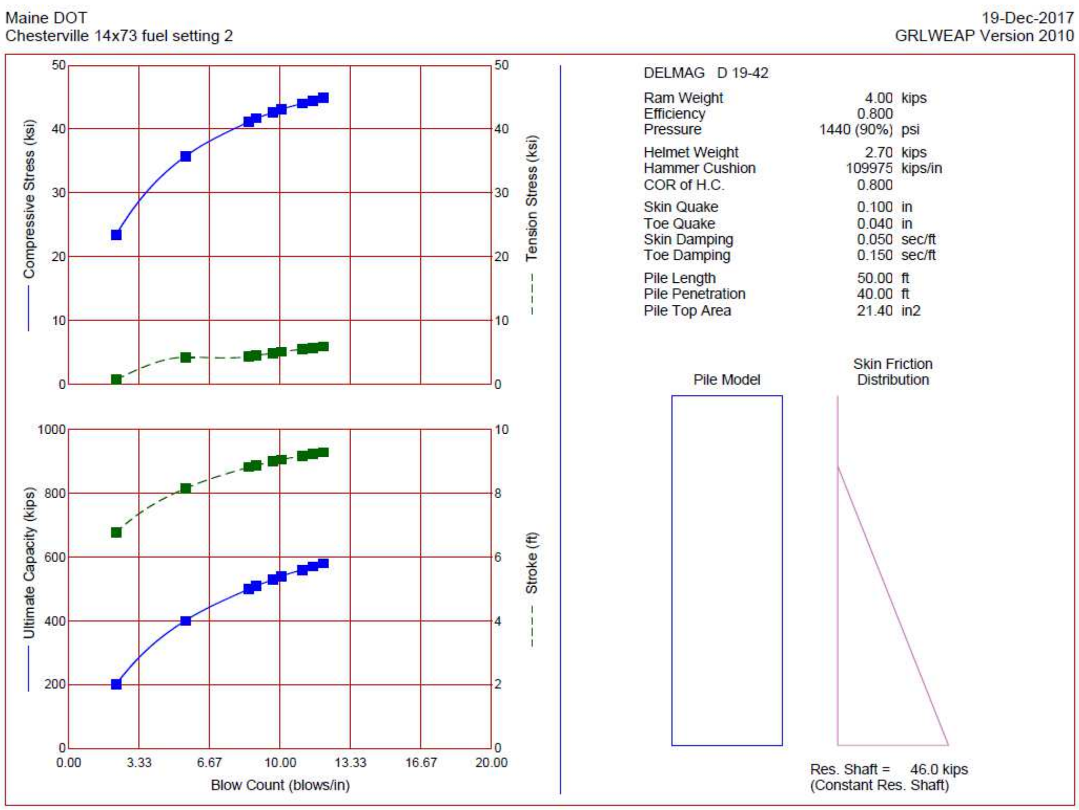
$R_{dr\_14x73\_strength} = 377 \cdot kip$

Service and Extreme Limit State:

$\varphi := 1.0$

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

$R_{dr\_14x73\_servext} = 580 \cdot kip$



Pile Size = 14 x 89. Assume Contractor will use a Delmag D 19-42 hammer with 2.70 kip helmet on fuel setting 1 (100%).

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke ft | Energy kips-ft |
|---|---|---|---|---|---|
| 200.0 | 23.93 | 0.80 | 2.0 | 7.47 | 17.88 |
| 400.0 | 33.52 | 2.26 | 4.7 | 8.71 | 18.97 |
| 600.0 | 43.48 | 5.01 | 9.3 | 10.05 | 22.30 |
| 610.0 | 43.89 | 5.12 | 9.6 | 10.11 | 22.46 |
| 620.0 | 44.21 | 5.23 | 10.0 | 10.15 | 22.57 |
| 640.0 | 45.04 | 5.40 | 10.8 | 10.25 | 22.78 |
| 660.0 | 45.78 | 5.59 | 11.6 | 10.34 | 23.00 |
| 670.0 | 46.14 | 5.68 | 12.1 | 10.38 | 23.11 |

Limit driving stresses to 45 ksi.

$R_{dr\_14x89} := 620kip$

Strength Limit State

$\varphi_{dyn} := .65$

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

$R_{dr\_14x89\_strength} = 403 \cdot kip$

Service and Extreme Limit State

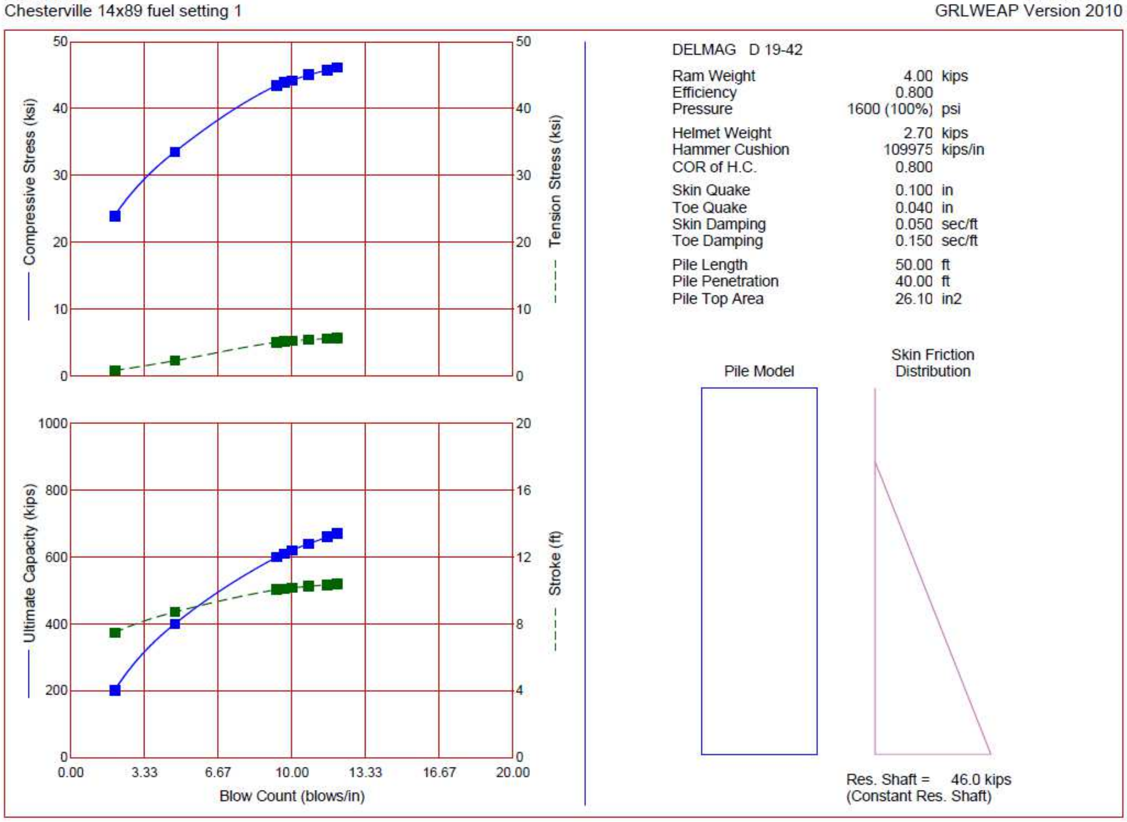
$\varphi := 1.0$

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

$R_{dr\_14x89\_servext} = 620 \cdot kip$

Maine DOT
Chesterville 14x89 fuel setting 1

19-Dec-2017
GRLWEAP Version 2010



Pile Size = 14x117. Assume Contractor will use Delmag 19-42 hammer with 2.70 kip helmet on fuel setting 1 (100%).

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke ft | Energy kips-ft |
|---|---|---|---|---|---|
| 300.0 | 23.84 | 0.76 | 3.5 | 8.13 | 17.39 |
| 400.0 | 28.33 | 1.61 | 4.9 | 8.59 | 17.95 |
| 600.0 | 36.33 | 2.65 | 8.5 | 9.47 | 20.07 |
| 800.0 | 42.74 | 4.89 | 14.3 | 10.30 | 22.18 |
| 810.0 | 42.98 | 4.93 | 14.7 | 10.33 | 22.23 |
| 820.0 | 43.23 | 4.97 | 15.1 | 10.36 | 22.32 |
| 830.0 | 43.49 | 5.00 | 15.5 | 10.39 | 22.38 |
| 840.0 | 43.76 | 5.04 | 15.8 | 10.41 | 22.49 |

Limit to less than 15 bpi.

$R_{dr\_14x117} := 810kip$

Strength Limit State

$\varphi_{dyn} := .65$

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

$R_{dr\_14x117\_strength} = 527 \cdot kip$

Service and Extreme Limit State:

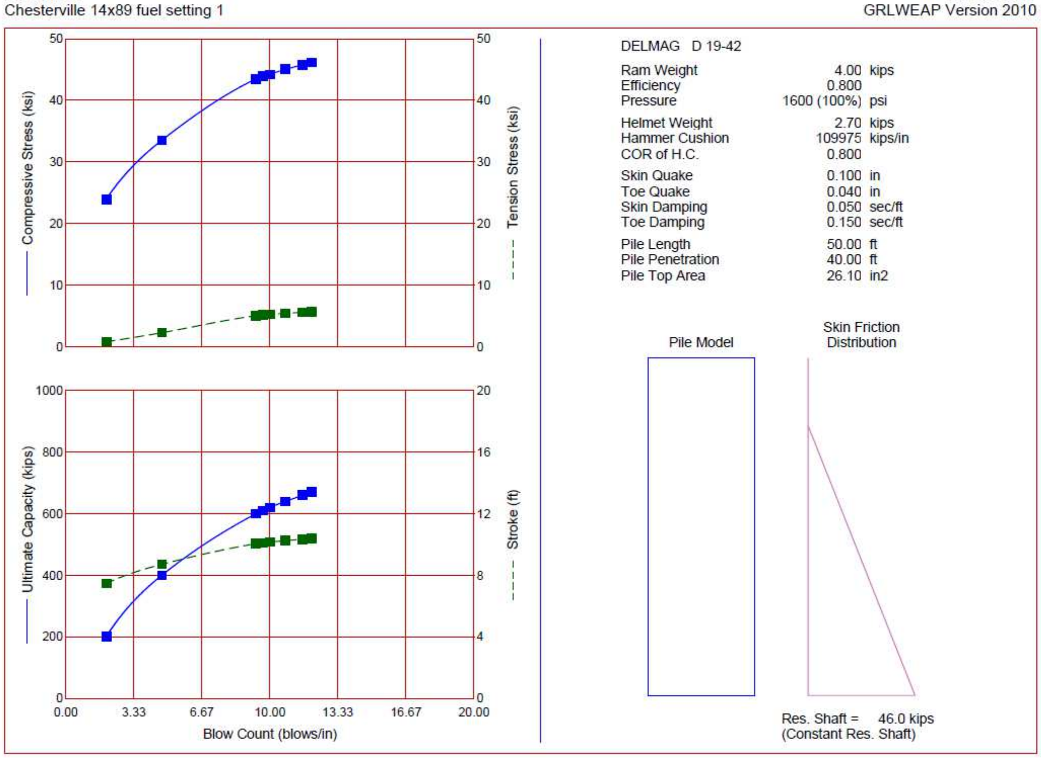
$\varphi := 1.0$

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

$R_{dr\_14x117\_servext} = 810 \cdot kip$

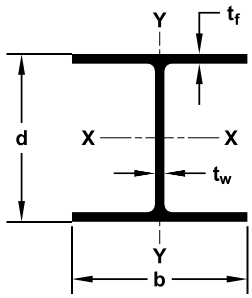
Maine DOT
Chesterville 14x89 fuel setting 1

19-Dec-2017
GRLWEAP Version 2010



# HP

## Steel H-Pile




| SECTION | Weight | Area | Depth d | Flange Width b | THICKNESS: Flange ($t_f$) | THICKNESS: Web ($t_w$) | Coating Area | PROPERTIES AXIS X-X: I | AXIS X-X: S | AXIS X-X: Z | AXIS X-X: r | AXIS Y-Y: I | AXIS Y-Y: S | AXIS Y-Y: Z | AXIS Y-Y: r |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | lb/ft (kg/m) | $in^2$ ($cm^2$) | in (mm) | in (mm) | in (mm) | in (mm) | $ft^2/ft$ ($m^2/m$) | $in^4$ ($cm^4$) | $in^3$ ($cm^3$) | $in^3$ ($cm^3$) | in (cm) | $in^4$ ($cm^4$) | $in^3$ ($cm^3$) | $in^3$ ($cm^3$) | in (cm) |
| HP 8 | 36 | 10.6 | 8.02 | 8.16 | 0.445 | 0.445 | 3.92 | 119 | 29.8 | 33.6 | 3.36 | 40.3 | 9.88 | 15.2 | 1.95 |
| HP 200 | 54 | 68.4 | 204 | 207 | 11.3 | 11.3 | 1.19 | 4953 | 488 | 550.6 | 8.53 | 1677 | 162 | 249.1 | 4.95 |
| HP 10 | 42 | 12.4 | 9.70 | 10.10 | 0.420 | 0.415 | 4.83 | 210 | 43.4 | 48.3 | 4.13 | 71.7 | 14.2 | 21.8 | 2.41 |
| HP 250 | 63 | 80.0 | 246 | 257 | 10.7 | 10.5 | 1.47 | 8741 | 711 | 791.5 | 10.5 | 2984 | 233 | 357.2 | 6.12 |
| | 57 | 16.7 | 9.99 | 10.20 | 0.565 | 0.565 | 4.91 | 294 | 58.8 | 66.5 | 4.18 | 101 | 19.7 | 30.3 | 2.45 |
| | 85 | 108 | 254 | 259 | 14.4 | 14.4 | 1.50 | 12237 | 964 | 1089.7 | 10.6 | 4204 | 323 | 496.5 | 6.22 |
| HP 12 | 53 | 15.5 | 11.80 | 12.00 | 0.435 | 0.435 | 5.82 | 393 | 66.7 | 74.0 | 5.03 | 127 | 21.1 | 32.2 | 2.86 |
| HP 310 | 79 | 100 | 300 | 305 | 11.0 | 11.0 | 1.77 | 16358 | 1093 | 1212.6 | 12.8 | 5286 | 346 | 527.7 | 7.26 |
| | 63 | 18.4 | 11.90 | 12.10 | 0.515 | 0.515 | 5.86 | 472 | 79.1 | 88.3 | 5.06 | 153 | 25.3 | 38.7 | 2.88 |
| | 94 | 119 | 302 | 307 | 13.1 | 13.1 | 1.79 | 19646 | 1296 | 1447.0 | 12.9 | 6368 | 415 | 634.2 | 7.32 |
| | 74 | 21.8 | 12.10 | 12.20 | 0.610 | 0.605 | 5.91 | 569 | 93.8 | 105 | 5.11 | 186 | 30.4 | 46.6 | 2.92 |
| | 110 | 141 | 307 | 310 | 15.5 | 15.4 | 1.80 | 23683 | 1537 | 1720.6 | 13.0 | 7742 | 498 | 763.6 | 7.42 |
| | 84 | 24.6 | 12.30 | 12.30 | 0.685 | 0.685 | 5.97 | 650 | 106 | 120 | 5.14 | 213 | 34.6 | 53.2 | 2.94 |
| | 125 | 159 | 312 | 312 | 17.4 | 17.4 | 1.82 | 27055 | 1737 | 1966.4 | 13.1 | 8866 | 567 | 871.8 | 7.47 |
| | 89 | 25.9 | 12.36 | 12.32 | 0.720 | 0.720 | 6.04 | 689 | 111.6 | 126.3 | 5.16 | 225 | 36.5 | 56.2 | 2.94 |
| | 132 | 167 | 314 | 313 | 18.3 | 18.3 | 1.84 | 28700 | 1830 | 2070 | 13.1 | 9370 | 599 | 922 | 7.48 |
| | 102 | 29.9 | 12.56 | 12.64 | 0.819 | 0.819 | 6.17 | 811 | 129.3 | 147.6 | 5.20 | 276 | 43.7 | 67.1 | 3.04 |
| | 152 | 193 | 319 | 321 | 20.8 | 20.8 | 1.88 | 33800 | 2120 | 2420 | 13.2 | 11500 | 716 | 1100 | 7.71 |
| | 117 | 34.4 | 12.76 | 12.87 | 0.929 | 0.929 | 6.26 | 946 | 148.2 | 170.8 | 5.24 | 331 | 51.4 | 79.3 | 3.11 |
| | 174 | 222 | 324 | 327 | 23.6 | 23.6 | 1.91 | 39400 | 2430 | 2800 | 13.3 | 13800 | 843 | 1300 | 7.89 |
| HP 14 | 73 | 21.4 | 13.60 | 14.60 | 0.505 | 0.505 | 6.96 | 729 | 107 | 118 | 5.84 | 261 | 35.8 | 54.6 | 3.49 |
| HP 360 | 109 | 138 | 345 | 371 | 12.8 | 12.8 | 2.12 | 30343 | 1753 | 1933.7 | 14.8 | 10864 | 587 | 894.7 | 8.86 |
| | 89 | 26.1 | 13.80 | 14.70 | 0.615 | 0.615 | 7.02 | 904 | 131 | 146 | 5.88 | 326 | 44.3 | 67.7 | 3.53 |
| | 132 | 168 | 351 | 373 | 15.6 | 15.6 | 2.14 | 37627 | 2147 | 2392.5 | 14.9 | 13569 | 726 | 1109.4 | 8.97 |
| | 102 | 30.1 | 14.00 | 14.80 | 0.705 | 0.705 | 7.06 | 1050 | 150 | 169 | 5.92 | 380 | 51.4 | 78.8 | 3.56 |
| | 152 | 194 | 356 | 376 | 17.9 | 17.9 | 2.15 | 43704 | 2458 | 2769.4 | 15.0 | 15817 | 842 | 1291.3 | 9.04 |
| | 117 | 34.4 | 14.20 | 14.90 | 0.805 | 0.805 | 7.12 | 1220 | 172 | 194 | 5.96 | 443 | 59.5 | 91.4 | 3.59 |
| | 174 | 222 | 361 | 378 | 20.4 | 20.4 | 2.34 | 50780 | 2819 | 3179.1 | 15.1 | 18439 | 975 | 1497.8 | 9.12 |
| HP 16 | 88 | 25.8 | 15.30 | 15.70 | 0.540 | 0.540 | 7.52 | 1110 | 145 | 161 | 6.56 | 349 | 44.5 | 68.2 | 3.68 |
| HP 410 | 131 | 167 | 389 | 399 | 13.7 | 13.7 | 2.29 | 46201 | 2376 | 2638.3 | 16.7 | 14526 | 729 | 1117.6 | 9.35 |
| | 101 | 29.9 | 15.50 | 15.80 | 0.625 | 0.625 | 7.56 | 1300 | 168 | 187 | 6.59 | 412 | 52.2 | 80.1 | 3.71 |
| | 150 | 193 | 394 | 401 | 15.9 | 15.9 | 2.30 | 54110 | 2753 | 3064.4 | 16.7 | 17149 | 855 | 1312.6 | 9.42 |
| | 121 | 35.8 | 15.80 | 15.90 | 0.750 | 0.750 | 7.62 | 1590 | 201 | 226 | 6.66 | 504 | 63.4 | 97.6 | 3.75 |
| | 180 | 231 | 401 | 404 | 19.1 | 19.1 | 2.32 | 66180 | 3294 | 3703.5 | 16.9 | 20978 | 1039 | 1599.4 | 9.53 |
| | 141 | 41.7 | 16.00 | 16.00 | 0.875 | 0.875 | 7.69 | 1870 | 234 | 264 | 6.70 | 599 | 74.9 | 116 | 3.79 |
| | 210 | 269 | 406 | 406 | 22.2 | 22.2 | 2.34 | 77835 | 3835 | 4326.2 | 17.0 | 24932 | 1227 | 1900.9 | 9.63 |
| | 162 | 47.7 | 16.30 | 16.10 | 1.000 | 1.000 | 7.75 | 2190 | 269 | 306 | 6.78 | 697 | 86.6 | 134 | 3.82 |
| | 241 | 308 | 414 | 409 | 25.4 | 25.4 | 2.36 | 91154 | 4408 | 5014.4 | 17.2 | 29011 | 1419 | 2195.9 | 9.70 |
| | 183 | 54.1 | 16.50 | 16.30 | 1.130 | 1.130 | 7.81 | 2510 | 304 | 349 | 6.81 | 818 | 100.0 | 156 | 3.89 |
| | 272 | 349 | 419 | 414 | 28.7 | 28.7 | 2.38 | 104473 | 4982 | 5719.1 | 17.3 | 34047 | 1639 | 2556.4 | 9.88 |
| HP 18 | 135 | 39.9 | 17.50 | 17.80 | 0.750 | 0.750 | 8.54 | 2200 | 251 | 281 | 7.43 | 706 | 79.3 | 122 | 4.21 |
| HP 460 | 201 | 257 | 445 | 452 | 19.1 | 19.1 | 2.60 | 91570 | 4113 | 4604.7 | 18.9 | 29386 | 1299 | 1999.2 | 10.7 |
| | 157 | 46.2 | 17.70 | 17.90 | 0.870 | 0.870 | 8.60 | 2570 | 290 | 327 | 7.46 | 833 | 93.1 | 143 | 4.25 |
| | 234 | 298 | 450 | 455 | 22.1 | 22.1 | 2.62 | 106971 | 4752 | 5358.5 | 18.9 | 34672 | 1526 | 2343.3 | 10.8 |
| | 181 | 53.2 | 18.00 | 18.00 | 1.000 | 1.000 | 8.66 | 3020 | 336 | 379 | 7.53 | 974 | 108.0 | 167 | 4.28 |
| | 269 | 343 | 457 | 457 | 25.4 | 25.4 | 2.64 | 125701 | 5506 | 6210.7 | 19.1 | 40541 | 1770 | 2736.6 | 10.9 |
| | 204 | 60.2 | 18.30 | 18.10 | 1.130 | 1.130 | 8.73 | 3480 | 380 | 433 | 7.60 | 1120 | 124.0 | 191 | 4.31 |
| | 304 | 388 | 465 | 460 | 28.7 | 28.7 | 2.66 | 144847 | 6227 | 7095.6 | 19.3 | 46618 | 2032 | 3129.9 | 11.0 |

# LABORATORY TESTING DATA SHEET

Project Name Wadsworth Street Bridge Replacement
Location Thomaston, ME
Reviewed By Matthew Palyka

Project No. 09.0025781.00
Assigned By Andrew Blaisdell

Project Manager Andrew Blaisdell
Report Date 5/26/2013
Date Reviewed 5/26/2013

| Boring No. | Sample No. | Depth Ft. | Lab No. | Sample Data: Water Content % | Do in. | L in. | (1) Unit Wt. PCF | (2) Wet Density PCF | Bulk Gs. | Compression Tests: (3) Other Tests | (4) Strength PSI | (5) Strain % | (6) Conf. Stress | (7) E sec PSI EE+06 | (8) Poisson's Ratio | $\sigma\tau$ PSI | $Is_{50}$ PSI | Rock Formation or Description or Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| BB-TSGR-202 | R-1 | 5.8-6.2 | 1 | | 1.934 | 4.547 | 170.6 | | | U | 15,761 | 0.21 | | 6.46 | 0.21 | | | Metapelite |
| BB-TSGR-203 | R-1 | 25.2-25.6 | 2 | | 1.987 | 4.535 | 171.8 | | | U | 8,411 | 0.16 | | 4.99 | 0.15 | | | Metapelite |
| BB-TSGR-301 | R-1 | 11.5-11.9 | 3 | | 1.982 | 4.552 | 168.8 | | | U | 21,463 | 0.28 | | 6.25 | 0.07 | | | Metapelite |
| BB-TSGR-303 | R-2 | 29.0-29.4 | 4 | | 1.983 | 4.628 | 178.7 | | | U | 13,897 | 0.15 | | 7.44 | 0.14 | | | Metasandstone |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |

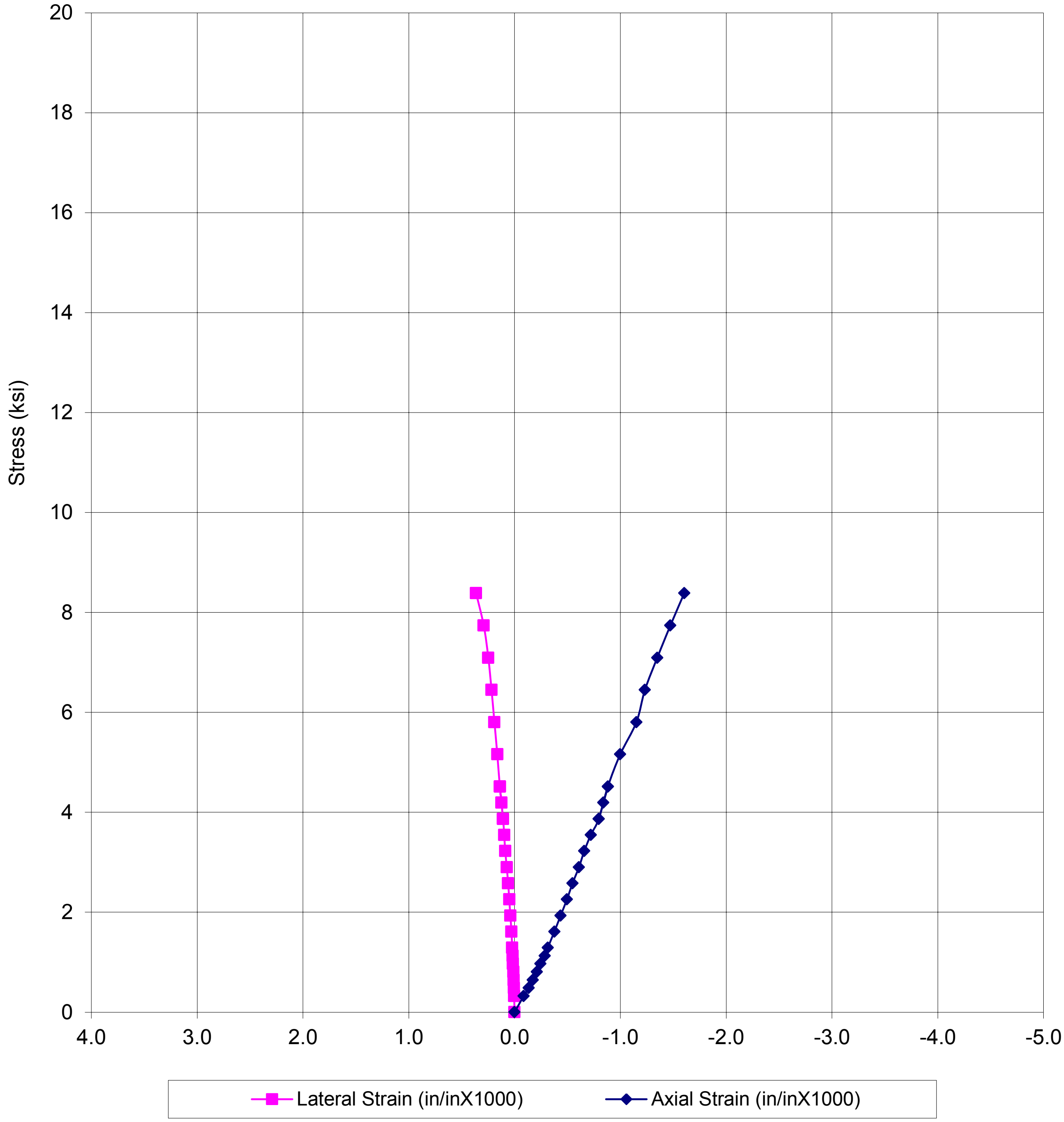
(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) P=Petrographic PLD=Point Load (diametrical
PLA= Point Load (Axial) RST= Splitting Tensile
U= Unconfined Compressive Strength
(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress
(6) Represents Confining Stress on Triaxial Tests
(7) Represents Secant Modulus at 50% of Total Failure Stress
(8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

THIELSCH ENGINEERING
195 Frances Ave.
Cranston, RI 02910
401-467-6454

**Wadsworth Street Bridge Replacement**
**Thomaston, ME**




Rock Unconfined Compression Testing

| | | | |
|---|---|---|---|
| Boring No. | BB-TSGR-203 | File No. | 09.0025781.00 |
| Sample No. | R-1 | Date: | 05/21/13 |
| Depth: | 25.2-25.6' | Test No. | U 2 |

LPile Parameters

# Development of soil model for LPile

**OBJECTIVE**
Estimate soil parameters for lateral pile analyses at both abutments.

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

**Assumptions:**
1) Soil model based on boring BB-CNS-101.
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 indicates the top and bottom elevations of the soil layers based on differing engineering properties.

Soil Layer No. 1 (Granular Borrow for Underwater Backfill) El. 346.9 - 333.0.

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

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

$\boxed{\gamma_{1dry} = 0.072 \cdot pci}$

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 (Very Loose Sands, Trace Organics) El. 333-324

Design $N_{60}$ = 3 bpf

$\boxed{\phi_2 := 28deg}$

$\gamma_d := 92pcf$

$w := 47.5\%$

$\gamma_{sat} := \gamma_d \cdot (1 + w)$

$\gamma_{sat} = 135.7 \cdot pcf$

$\gamma_{water} := 62.4pcf$

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

$\boxed{\gamma' = 73.3 \cdot pcf}$

$\boxed{\gamma' = 0.042 \cdot pci}$

Representative constant giving the variation of soil modulus with depth, k:
Loose sand below water table for static loading = 20 pci

## Soil Layer No. 3 (Submerged, Loose, Sand) El. 324-317.

$N_{60.design} := 9 \ \text{bpf}$

Internal Angle of Friction $\boxed{\phi_2 := 30}$

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

Dry Unit Weight

Submerged, Loose, Sand: 92 pcf

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

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

Saturated Unit Weight

Natural water content at a saturated state: 25.8% (BB-CNS-101 3D).

$w_2 := 25.8\%$

$\gamma_{2saturated} := \gamma_{2dry} \cdot (1 + w_2)$

$\gamma_{2saturated} = 115.7 \cdot \text{pcf}$

Effective Unit Weight

$\gamma'_2 := \gamma_{2saturated} - \gamma_{water}$

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

$\boxed{\gamma'_2 = 53.3 \cdot \text{pcf}}$

$\boxed{\gamma'_2 = 0.031 \cdot \text{pci}}$

Representative constant giving the variation of soil modulus with depth, k:
Loose sand below water table for static loading = 20 pci

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## Soil Layer No. 4 (Submerged, Medium Dense Sand) El. 317 - 307

Design $N_{60}$ = 14 bpf

Internal Angle of Friction

$\boxed{\phi_3 := 31}$

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

Dry Unit Weight

Submerged, 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: 25.8% (BB-CNS-101, 6D)

$w_{3sat} := 25.8\%$

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

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

$\gamma_{3saturated} = 130.8 \cdot \text{pcf}$

Soil Effective Unit Weight

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

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

$\boxed{\gamma'_3 = 68.4 \cdot pcf}$

$\boxed{\gamma'_3 = 0.040 \cdot pci}$

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. 5 (Submerged, medium dense sand) El. 307 - 296.7

Design $N_{60}$ = 20

Internal Angle of Friction $\boxed{\phi_1 := 33\,deg}$

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

Soil Dry Unit Weight $\boxed{\gamma_{4dry} := 112pcf}$

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

Saturated Unit Weight

Natural Water Content in a Saturated State: 22.5% (BB-CNS-101 8D).

$w_{4sat} := 22.5\%$

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

$\gamma_{4sat} = 137.2 \cdot pcf$

Effective Unit Weight

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

$\boxed{\gamma'_4 = 74.8 \cdot pcf}$

$\boxed{\gamma'_4 = 0.043 \cdot pci}$

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

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# Maine Department of Transportation

Soil/Rock Exploration Log

Abut. No. 1 LPile Soil Model
Layer Delineation

**Project:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream
**Location:** Chesterville, Maine

**Boring No.:** BB-CNS-101
**WIN:** 21688.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 346.9 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/11/2017-9/12/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 14+31.8, 7.0 ft Rt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 12.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.854 | **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 | 346.5 | | 5" HMA. 0.4 | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 5 | 1D | 24/15 | 5.00 - 7.00 | 4/3/2/2 | 5 | 7 | 10 | | | Light brown, damp, loose, Silty fine to medium SAND, trace gravel, (Fill). | G#300251 A-4, SM WC=14.0% |
| | | | | | | | 13 | | | | |
| | | | | | | | 13 | | | | |
| | | | | | | | | | | 9.0 | |
| | | | | | | | 11 | | | | |
| 10 | 2D | 24/14 | 10.50 - 12.50 | 1/2/4/3 | 6 | 9 | 3 | | | Casing sunk to 10.5 ft bgs. Grey-brown, wet, loose, Silty fine to medium SAND, some organics, wood, (Alluvium). | |
| | | | | | | | 8 | | | | |
| | | | | | | | 13 | | | | |
| | | | | | | | 18 | | | | |
| | | | | | | | 28 | | | | |
| 15 | 3D | 24/18 | 15.00 - 17.00 | 1/1/1/2 | 2 | 3 | 18 | | | Grey-brown, wet, very loose, fine to medium SAND, some silt, trace organics (small fibers, wood pieces) similar to 2D. | G#300253 A-2-4, SM WC=47.5% |
| | | | | | | | 20 | | | | |
| | | | | | | | 28 | | | | |
| | | | | | | | 29 | | | | |
| | | | | | | | 31 | | | | |
| 20 | 4D | 24/18 | 20.00 - 22.00 | WOR/1/1/1 | 2 | 3 | 30 | | | Grey-dark brown, wet, very loose, Silty fine SAND, trace fibers. | |
| | | | | | | | 31 | | | | |
| | | | | | | | 38 | | | | |
| | | | | | | | 54 | 323.9 | | 23.0 Change to grey at 23.0 ft bgs. | |
| | | | | | | | 69 | | | | |
| 25 | | | | | | | | | | | |

Layer 1 - Replaced with Granular Borrow

Layer 1 (El. 346.9 - 333.0)

337

Bottom of Abutment

333

Ground water

Layer 2 - Loose sand, submerged

Layer 2 (El. 333.0 - 316.9)

**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:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream

**Location:** Chesterville, Maine

**Boring No.:** BB-CNS-101

**WIN:** 21688.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 346.9 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/11/2017-9/12/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 14+31.8, 7.0 ft Rt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 12.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.854 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
$S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
$N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency
$N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected
$T_v$ = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample Information: 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 | 24/15 | 25.00 - 27.00 | 2/2/4/5 | 6 | 9 | 33 | | | Brown, wet, loose, fine SAND, trace silt, (Marine Sand). | |
| | | | | | | | 33 | | | Layer 3, loose sand, submerged | |
| | | | | | | | 42 | | | | |
| | | | | | | | [illegible] | | | Layer 3 (El. 323.9-316.9) | |
| | | | | | | | 64 | | | | |
| 30 | 6D | 24/12 | 30.00 - 32.00 | 4/4/6/8 | 10 | 14 | 41 | | | Brown, moist, medium dense, fine to medium SAND, trace silt, poorly graded, (Marine Sand). | G#300255 A-3, SP-SM WC=25.8% |
| | | | | | | | 47 | | | | |
| | | | | | | | 52 | | | Layer 4, medium dense sand, submerged | |
| | | | | | | | 76 | | | | |
| | | | | | | | 79 | | | | |
| 35 | 7D | 24/13 | 35.00 - 37.00 | 3/4/6/6 | 10 | 14 | 40 | | | Grey-brown, moist, medium dense, SAND, (Marine Sand). | |
| | | | | | | | 57 | | | Layer 4 (El. 316.9 - 306.9) | |
| | | | | | | | 88 | | | | |
| | | | | | | | 96 | | | Sands running, filling casing. | |
| | | | | | | | 158 | | | | |
| 40 | 8D | 24/13 | 40.00 - 42.00 | 4/4/10/15 | 14 | 20 | 39 | | | Similar to 7D, trace silt. | G#300256 A-3, SP-SM WC=22.5% |
| | | | | | | | 93 | | | | |
| | | | | | | | 108 | | | Layer 4, medium dense sand, submerged | |
| | | | | | | | 100 | 303.4 | | 43.5 | |
| | | | | | | | 123 | | | | |
| 45 | 9D | 24/14 | 45.00 - 47.00 | 4/7/7/7 | 14 | 20 | 46 | | | Brown-grey, moist, medium dense, Silty fine to medium SAND, trace rounded gravel, (Marine Sand). | |
| | | | | | | | [illegible] | | | Layer 5, (El. 306.9 - 296.7) | |
| | | | | | | | 77 | | | | |
| | | | | | | | 271 | | | | |
| | | | | | | | | | | Roller Coned ahead to 50.0 ft bgs. | |
| 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:** Dutch Gap Bridge #3951 carries Dutch Gap Rd over Little Norridgewock Stream
**Location:** Chesterville, Maine

**Boring No.:** BB-CNS-101

**WIN:** 21688.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 346.9 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Travis/James | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/11/2017-9/12/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 14+31.8, 7.0 ft Rt. | **Casing ID/OD:** | NW-3" | **Water Level*:** | 12.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.854 | **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
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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. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 10D | 2.4/1 | 50.00 - 50.20 | 50(2.4") | --- | | | 296.7 | | Grey, very dense, broken rock fragments and sand. 50.2 | |
| | | | | | | | | 295.9 | | Top of Bedrock at Elev. 296.7 ft.<br>Roller Coned ahead to 51.0 ft bgs. | |
| | R1 | 60/60 | 51.00 - 56.00 | RQD = 73% | | | NQ-2 | | | 51.0<br>R1: Bedrock: Grey (Greenish-grey), fine grained, limey PELITE, massive to thinly laminated, hard, generally fresh, discontinuities/joint sets horizontal to moderately dipping, some slightly oxidized surfaces, close, tight, infrequent quartz inclusion/banding,<br>Rock Mass Quality - Fair.<br>Patch Mountain Member of the Sangerville Formation<br>R1: Core Times (min:sec)<br>51.0-52.0 ft (1:49)<br>52.0-53.0 ft (1:54)<br>53.0-54.0 ft (1:50)<br>54.0-55.0 ft (2:28)<br>55.0-56.0 ft (3:00)<br>100% Recovery | |
| 55 | | | | | | | | | | | |
| | R2 | 60/60 | 56.00 - 61.00 | RQD = 83% | | | | | | R2: Bedrock: Greenish-grey/grey, fine grained, limey PELITE, thinly interlaminated, fresh, joint set at low to moderately dipping angles, close, tight,<br>Rock Mass Quality - Good.<br>Patch Mountain Member of Sangerville Formation.<br>R2: Core Times (min:sec)<br>56.0-57.0 ft (2:07)<br>57.0-58.0 ft (2:09)<br>58.0-59.0 ft (2:11)<br>59.0-60.0 ft (2:39)<br>60.0-61.0 ft (2:40)<br>100% Recovery | |
| 60 | | | | | | | | 285.9 | | 61.0 | |
| | | | | | | | | | | **Bottom of Exploration at 61.0 feet below ground surface.** | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |

**Remarks:**

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

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



**Boring No.:** BB-CNS-101

## 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, $\phi$ | Soil Total Unit Weight (pcf) | Coeff. of Friction, tan $\delta$, Concrete to Soil | Interface Friction, Angle, Concrete to Soil $\delta$ |
|---|---|---|---|---|---|
| 1 | Very loose to loose silty sand and gravel<br>Very loose to loose sand<br>Very loose to medium density sandy silt<br>Stiff to very stiff clay or clayey silt | $29^{\circ}$ * | 100 | 0.35 | $19^{\circ}$ |
| 2 | Medium density silty sand and gravel<br>Medium density to dense sand<br>Dense to very dense sandy silt | $33^{\circ}$ | 120 | 0.40 | $22^{\circ}$ |
| 3 | Dense to very dense silty sand and gravel<br>Very dense sand | $36^{\circ}$ | 130 | 0.45 | $24^{\circ}$ |
| 4 | Granular underwater backfill<br>Granular borrow | $32^{\circ}$ | 125 | 0.45 | $24^{\circ}$ |
| 5 | Gravel Borrow | $36^{\circ}$ | 135 | 0.50 | $27^{\circ}$ |

* The value given for the internal angle of friction ($\phi$) 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

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

b. Compute the power of the parabolic section using

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

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

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

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

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

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 $\gamma$, $\gamma_d$, and $\gamma_{sat}$ 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 $\gamma$, $\gamma_d$, and $\gamma_{sat}$

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

**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, $\gamma_d$ lb/ft$^3$ | Dry unit weight, $\gamma_d$ 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

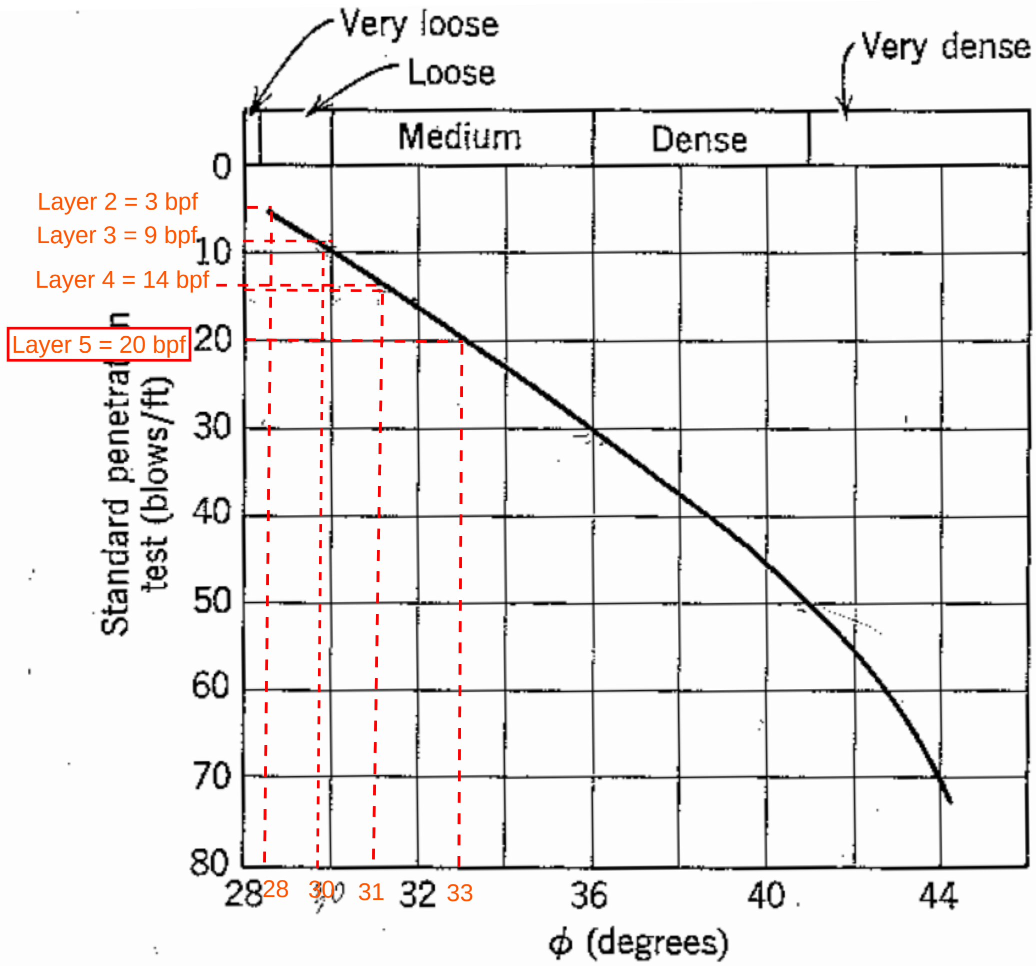


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Holtz and Kovacs , Intro to Geotechnical Eng

common mineral in sands is quartz; its $\rho_s$ = 2.65 Mg/m$^3$. Most clay soils have a value of $\rho_s$ between 2.65 and 2.80 Mg/m$^3$, depending on the predominant mineral in the soil, whereas organic soils may have a $\rho_s$ as low as 2.5 Mg/m$^3$. Consequently, it is usually close enough for geotechnical work to *assume* a $\rho_s$ of 2.65 or 2.70 Mg/m$^3$ for most phase problems, unless a specific value of $\rho_s$ is given.

The density of water varies slightly, depending on the temperature. At 4°C, when water is at its densest, $\rho_w$ exactly equals 1000 kg/m$^3$ (1 g/cm$^3$), and this density is sometimes designated by the symbol $\rho_o$. For ordinary engineering work, it is sufficiently accurate to take $\rho_w \approx \rho_o = 1000$ kg/m$^3$ = 1 Mg/m$^3$.

There are three other useful densities in soils engineering. They are the dry density $\rho_d$, the saturated density $\rho_{sat}$, and the submerged or buoyant density $\rho'$.

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

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

$$\rho' = \rho_{sat} - \rho_w \tag{2-11}$$

Strictly speaking, total $\rho$ should be used instead of $\rho_{sat}$ 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 $\rho_d$ is a common basis for judging the degree of compaction of earth embankments (Chapter 5). A typical range of values of $\rho_d$, $\rho_{sat}$, and $\rho'$ 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*

| | Density (Mg/m$^3$) | | |
|---|---|---|---|
| Soil Type | $\rho_{sat}$ | $\rho_d$ | $\rho'$ |
| 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

$\alpha$ = 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 $\delta$ = 17 - 22, per LRFD Table 3.11.5.3-1

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

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

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

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

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

### 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 $\alpha > 0$.

$\alpha$ = 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 $\alpha$ 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 | $\boxed{\text{17 to 22}}$ | 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: | | |
| ○ dressed soft rock on dressed soft rock | 35 | 0.70 |
| ○ dressed hard rock on dressed soft rock | 33 | 0.65 |
| ○ 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, $\phi_f$.

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.

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,

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

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

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

**Table 7.9** (*Continued*)

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

## 7.12 Coulomb's Passive Earth Pressure

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

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

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

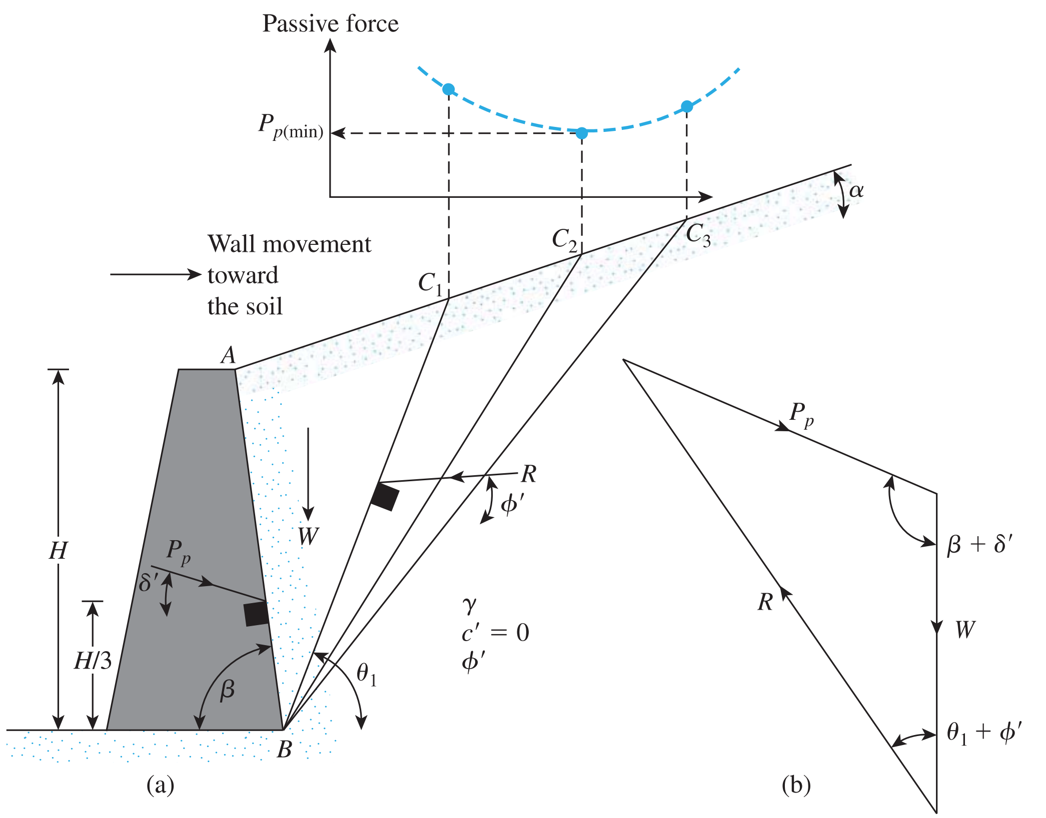


**Figure 7.25** Coulomb's passive pressure

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

| | $\delta'$ (deg) | | | | |
|---|---|---|---|---|---|
| $\phi'$ (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

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

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 $\phi'$ and $\delta'$ 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 $\delta'$ 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 $\delta'$ increases, Coulomb's

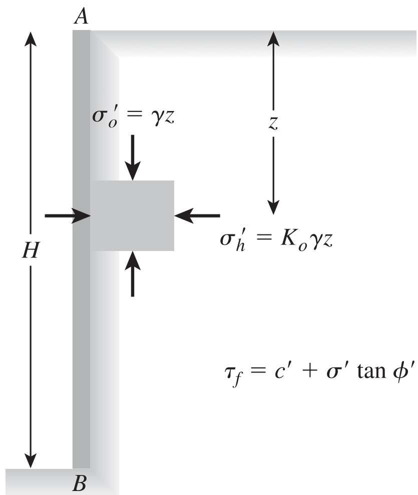


**Figure 13.3**
Earth pressure at rest

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

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

So,

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

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

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

where $\phi'$ = drained friction angle.

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

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

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

Frost Depth

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

From Design Freezing Index Map: Chesterville**, Maine**
Case 1 - coarse grained granular fill soils W=10%

$DFI_1 := 1700 \qquad d_1 := 87.5 \cdot in$

$DFI_2 := 1800 \qquad d_2 := 90.1 in$

Approximate DFI at project = 1775 find frost depth by interpolation:

$DFI_3 := 1775$

$$d_3 := d_1 + \frac{(DFI_3 - DFI_1)\cdot(d_2 - d_1)}{(DFI_2 - DFI_1)} \qquad d_3 = 89.5 \cdot in$$

Depth of Frost Penetration $\qquad d_3 = 7.5 \cdot ft$

**Method 2 - ModBerg Software**

Examine foundations placed on coarse grained fill soils

------------------------
ModBerg Results
------------------------

Project Location: Chesterville, Maine (data used from Orono, Maine)

Air Design Freezing Index = 1588 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1270 F-days
Mean Annual Temperature = 43.5 deg F
Design Length of Freezing Season = 132 days

| Layer #:Type | t | w% | d | Cf | Cu | Kf | Ku | L |
|---|---|---|---|---|---|---|---|---|
| 1-Coarse | 77.5 | 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 = 6.46 ft = 77.5 in.
**********************************************************

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

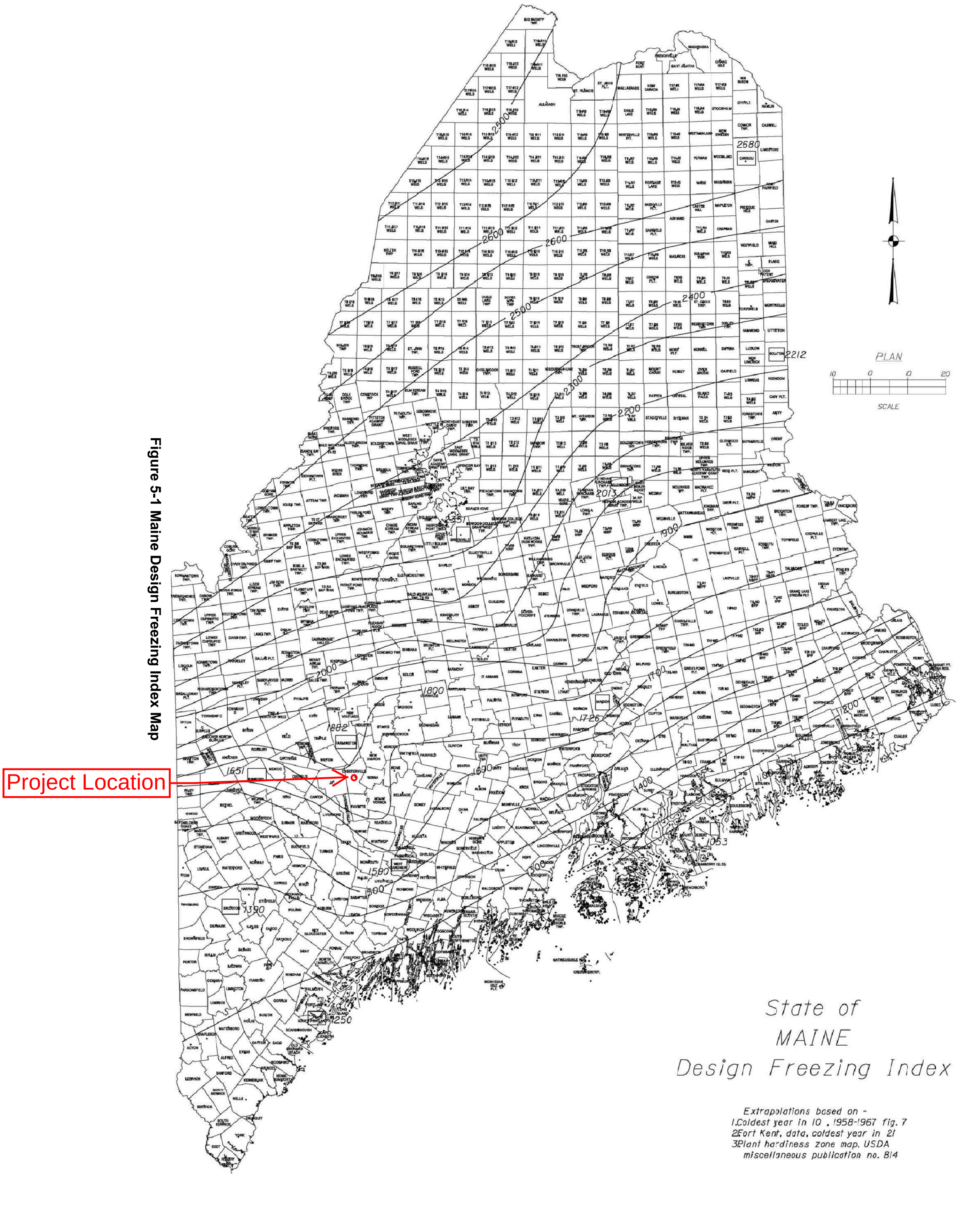


Figure 5-1 Maine Design Freezing Index Map

## 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 |

Interpolate-

Seismic Parameters

| BB-CNS-101 | | | |
|---|---|---|---|
| | | | |
| **Depth** | **101 $N_{60}$** | **di** | **di/N** |
| 6 | 7 | 6 | 0.86 |
| 11 | 9 | 5 | 0.56 |
| 16 | 3 | 5 | 1.67 |
| 21 | 3 | 5 | 1.67 |
| 26 | **9** | 5 | 0.56 |
| 31 | 14 | 5 | 0.36 |
| 36 | 14 | 5 | 0.36 |
| 41 | 20 | 5 | 0.25 |
| 46 | 20 | 5 | 0.25 |
| 51 | 20 | 5 | 0.25 |
| 100 | 100 | 49 | 0.49 |
| | | | |
| | | | |
| **SUM** | | **100** | **6.40** |
| | | | |
| | | **di/di/N** | **15.63** |

| BB-CNS-102 | | | |
|---|---|---|---|
| | | | |
| **Depth** | **102 $N_{60}$** | **di** | **di/N** |
| 6 | 6 | 6 | 1.00 |
| 11 | 1 | 5 | 5.00 |
| 16 | 16 | 5 | 0.31 |
| 21 | 13 | 5 | 0.38 |
| 26 | 14 | 5 | 0.36 |
| 31 | 11 | 5 | 0.45 |
| 36 | 20 | 5 | 0.25 |
| 41 | 13 | 5 | 0.38 |
| 46 | 77 | 5 | 0.06 |
| 49 | 77 | 3 | 0.04 |
| 100 | 100 | 51 | 0.51 |
| | | | |
| | | | |
| **SUM** | | **100** | **7.76** |
| | | | |
| | | **di/di/N** | **12.89** |

| **SUM** | **Nav.** | **14.26** |
|---|---|---|

$N_{av.} < 15$

## Conclusion: Site Class E

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

Chesterville Dutch Gap Bridge #3951 Calculations by A. Van Buskirk
WIN 21688.00 Checked by LK 3/26/18
Seismic
2/22/2018

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.568074
Longitude = -070.087112
Site Class B
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.082 | PGA - Site Class B |
| 0.2 | 0.170 | Ss - Site Class B |
| 1.0 | 0.048 | S1 - Site Class B |

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.568074
Longitude = -070.087112
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.206 | As - Site Class E |
| 0.2 | 0.426 | SDs - Site Class E |
| 1.0 | 0.167 | SD1 - Site Class E |

Chesterville Dutch Gap Bridge #3951
WIN 21688.00
Seismic
2/22/2018

Calculations by A. Van Buskirk
Checked by __3/26/18__