Proactive by Design

GEOTECHNICAL DESIGN REPORT
C.P.R. CROSSING BRIDGE NO. 3256

ROUTE 6/15 OVER CENTRAL MAINE & QUEBEC RAILWAY
MAINE DOT WIN 021701.00
SAPLING TOWNSHIP, MAINE

December 2017
File No. 09.0025956.00

PREPARED FOR:
Maine Department of Transportation

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Via Email

December 18, 2017
File No. 09.0025956.00

Ms. Laura Krusinski
Maine Department of Transportation
16 State House Station
Augusta, Maine 04333-0016

Re: Geotechnical Design Report
Canadian Pacific Railway Crossing Bridge No. 3256
Route 6/15 over Central Maine & Quebec Railway
MaineDOT WIN 21701.00
Sapling Township, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report for the proposed bridge replacement for the Canadian Pacific Railway Crossing bridge in Sapling Township, Maine. Our services were provided in accordance with Assignment Letter No. 13 (dated September 21, 2017) associated with the Bridge Program Multi-PIN Project Contract Number 20150608000000000793 between MaineDOT and GZA dated July 22, 2015, which incorporates GZA's Proposal No. 09.P000053.18, dated September 12, 2017. This report is subject to the *Limitations* included in **Appendix A**.

It has been a pleasure serving you on this project. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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Engineer I

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Attachment

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation completed by GZA GeoEnvironmental, Inc. (GZA) for the proposed replacement of Maine Department of Transportation (MaineDOT) Canadian Pacific Railway Crossing Bridge No. 3256. Our services were provided in accordance with Assignment Letter No. 13 (dated September 21, 2017) associated with the Bridge Program Multi-PIN Project Contract Number 20150608000000000793 between MaineDOT and GZA dated July 22, 2015, which incorporates GZA's Proposal No. 09.P000053.18, dated September 12, 2017. This report is subject to the *Limitations* included in **Appendix A**.

1.1 BACKGROUND

The existing bridge (No. 3256) spans north-to-south and carries State Route 6/15 over the Central Maine & Quebec Railway (CMQR) in Sapling Township, Maine as shown on the **Figure 1, Locus Plan.** The existing bridge was constructed in 1936, with subsequent modifications, and consists of three spans supporting a total bridge length of 168 feet. The existing bridge foundations consist of spread footings bearing on or near bedrock. The existing approach embankments carry the roadway approximately 28 feet above the railbed level, resulting in embankment heights approximately 25 to 30 feet above adjacent ground surface near the bridge.

The proposed replacement bridge will be constructed on the existing alignment and will consist of an 88-foot-long single-span structure supported on semi-integral abutments, as shown on the **Figure 2, Boring Location Plan**. Mechanically stabilized earth (MSE) walls are planned to support the approach embankments. The bridge foundations are planned to consist of spread footings supported above the MSE reinforcing. The bridge will be constructed on a 40-degree skew from the railway alignment, with the new abutments and MSE walls parallel to the railway alignment. Due to the reduced bridge length, the face of the new MSE walls will be constructed between the existing abutments and piers, primarily near existing railbed grades, resulting in placement of up to approximately 27 feet of fill up to roadway level. Toward the back of the MSE walls, most or all of the existing embankment will be excavated for placement of the straps. The roadway profile will be roughly 2 feet higher than existing, resulting in placement of approximately 2 feet of fill above the existing roadway grade in the vicinity of the existing abutments.

The existing bridge superstructure and portions of the substructure will be demolished prior to bridge construction. The portions of substructures that do not interfere with the proposed construction are planned to be left in place, including the below-grade portions of existing abutments and the lower approximately 6 feet of Pier 2. Pier 1 superstructure and substructure will be completely removed due to interference with Abutment 1 and the MSE wall.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and to provide geotechnical engineering recommendations and construction considerations for the project. To meet these objectives, GZA completed the following Scope of Services:

- Reviewed test boring logs and laboratory test data previously completed by MaineDOT;
- Conducted geotechnical engineering analyses to evaluate: Foundation types and design parameters for the proposed abutments and retaining walls; American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) load and resistance factors associated with geotechnical design elements; lateral earth pressures for retaining wall design; seismic design considerations; and global stability and settlement of approach embankments and MSE walls;
- Developed geotechnical design parameters and recommendations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

Between March 9 and 21, 2017, New England Boring Contractors of Hermon, Maine drilled four test borings, including one boring drilled through each approach (BB-SCPR-101 and -104) and one boring drilled adjacent to each existing bridge pier (BB-SCPR-102 and -103). The borings were drilled using a track-mounted drill rig and cased rotary wash drilling methods to depths ranging from approximately 23 to 50 feet below ground surface (bgs) and were terminated approximately 9 to 11 feet into bedrock. The as-drilled boring locations and elevations were included on the draft boring logs prepared by MaineDOT and are shown on **Figure 2.** Elevations referenced are in feet and refer to the North American Vertical Datum of 1988 (NAVD88).

The borings were drilled using 3- and 4-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with a safety hammer operated with a rope and cathead. Bedrock cores were obtained using NQ2 wire-line coring equipment in each test boring.

Drafts of the logs were prepared in Logdraft by MaineDOT. GZA subsequently reviewed the logs and provided edits to reflect laboratory soil test results and interpreted stratification. The final logs are provided in **Appendix B**.

3.0 LABORATORY TESTING

MaineDOT completed a laboratory soil testing program consisting of 13 water content tests and 13 gradation analysis/AASHTO Classification/MaineDOT Frost Classification assessments (including 10 hydrometer analyses). Results of the testing are included in **Appendix C**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available literature, surficial geologic units mapped in the area include glacial till consisting of sandy silt, sand and gravel, and bedrock outcrops[1].

Bedrock at the site is mapped as the lower member of the Dead River Formation. The Dead River Formation in the site vicinity is described as thin bedded greenish to purplish gray slate and phyllite[2].

4.2 SUBSURFACE PROFILE

Two soil units were encountered in the test borings overlying bedrock: Fill and Glacial Till. Approximately 4 to 6 inches of asphalt pavement was encountered in the borings drilled through the approaches. The thicknesses and generalized descriptions of the soil units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. The subsurface conditions are also shown in relation to the bridge alignment on **Figure 3**.

The encountered thicknesses, generalized descriptions of the units encountered, in descending order from ground surface, are summarized in the following table.

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	8 to 34	Soft to very stiff, brown and gray, SILT, some fine Sand, trace to some Gravel, (USCS: ML) AND medium dense to very dense, brown, medium SAND, little to some Silt, little to some Gravel (USCS: SM) MaineDOT Frost Classification: II, III, IV *Encountered in all borings*
Glacial Till	4 to 8	Very stiff to hard, olive-gray to gray, SILT, some fine Sand, trace to little gravel (USCS: ML) AND dense, gray, Silty fine SAND, trace gravel (USCS: SM) MaineDOT Frost Classification: IV *Encountered in all borings*
Top of Bedrock Elevation	Encountered Top of Rock: Approximate El. 1039.5 to 1041.5	

GZA did not review the soil samples collected during drilling. We relied on classifications made by MaineDOT and shown on the draft boring logs combined with laboratory test results to develop descriptions of the soils. The interface between the Fill and Glacial Till was interpreted based on the known height of the embankment and variations in blow counts.

[1] Genes, Andrew N., Caldwell, Dabney W. and Hanson, Lindley S., 1986. Reconnaissance surficial geology of the Moosehead Lake [15-minute] quadrangle, Maine: Maine Geological Survey, Open-File Map 86-34, map, scale 1:62,500. *Maine Geological Survey Maps*. 620. http://digitalmaine.com/mgs_maps/620

[2] Osberg, Philip H., Hussey II, Arthur M., Boone, Gary, M., 1985. Bedrock Geologic Map of Maine, Maine Geological Survey, Department of Conservation, map, scale 1:500,000.

4.2.1 Bedrock

Bedrock was cored in all of the test borings and was classified by MaineDOT as Phyllite. The bedrock was generally described as hard, fresh, fine grained, and olive-green and purple. The joints were typically very close to closely spaced, and dipped at low angles to near vertical. The Rock Quality Designation (RQD) ranged from 17 to 76 percent, with an average RQD of 44 percent.

GZA did not review the rock core specimens and relied entirely upon the MaineDOT rock descriptions.

4.2.2 Groundwater

The boring logs indicate that groundwater was not observed while drilling the test borings. In the embankment borings, soil samples were described as wet at a depth of approximately 20 feet. Therefore, it is possible that groundwater is perched as high as El. 1060 within portions of the embankments. At the rail level, samples were described as wet at a depth of 2 feet (El. 1053) in boring BB-SCPR-102 and were all described as moist in BB-SCPR-103. Fluctuations in groundwater level occur due to variations in season, precipitation, and construction activities in the area. Consequently, water levels during construction are likely to vary from those encountered at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with 2014 AASHTO LRFD Bridge Design Specifications, 7th Edition, with Interims (herein known as AASHTO) and the MaineDOT Bridge Design Guide, 2014 Edition (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations developed by GZA for the project are attached in **Appendix D** of this report.

5.2 APPROACH EMBANKMENTS

Existing approach embankments will be lengthened toward the railway to reduce the length of the new bridge, resulting in placement of up to 27 feet above existing grade at the face of the MSE wall and excavation into the existing north approach embankment to as deep as about 34 feet below existing grade.

MSE fill will be placed over a thin layer of proof-compacted existing fill and very stiff to hard Glacial Till or directly on Glacial Till. We conducted settlement evaluations based on the maximum anticipated thickness of new MSE fill underlain by proof-compacted fill and Glacial Till. We estimate total settlement of the MSE wall, including the combined self-weight and the bridge footing above, will be 1 inch or less, including approximately half of the settlement resulting from MSE wall construction up to bottom of footing level and half resulting from bridge foundation loading and MSE wall construction up to finish grade. We anticipate that settlement will primarily occur elastically as the load is applied. Therefore, post-construction settlement of the MSE foundation soil is anticipated to be negligible.

Global stability was evaluated for a longitudinal section through the abutment face. The basis for acceptable performance for embankments is specified in AASHTO Article 11.6.2.3 and is based on a resistance factor of 0.65 (corresponding to a safety factor of 1.5) for slopes that support structures. The reinforcement length was assumed to be the minimum specified in the MSE Wall Review Set dated November 28, 2017, prepared by Becker Structural Engineering, Inc., equal to 0.6 times the MSE wall height plus 6.5 feet.

Global stability analyses were performed using the analytical software Slope-W. Potential circular and block slip surfaces were considered. The calculated minimum factor of safety against slope instability was 2.4 for a circular surface and 2.0 for a block surface. Therefore, we conclude global stability meets the required criteria.

5.3 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the proposed approach fill (including MSE walls), proof-compacted existing fill, glacial till and bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1 using the average SPT N-value from the soil materials encountered in the borings and an assumed value of at least 20 blows per foot for proposed fill above (i.e., below the proposed roadway surface). On this basis, the SPT N-value fell between 15 and 50 blows per foot, and the bridge is assigned to Site Class D.

The available subsurface data indicates that the natural materials encountered at the site are sufficiently dense and/or silty that the potential for liquefaction is very low.

Seismic design recommendations are provided in **Section 6.3** of this report.

5.4 EVALUATIONS OF PROPOSED FOUNDATIONS

5.4.1 Foundation Type Assessment

The bridge is proposed to be supported on spread footings that bear above reinforced fill of the MSE retaining walls.

Based on the results of the test borings, we conclude the proposed bridge concept is feasible from a geotechnical standpoint. The abutment footing design is controlled by the ability of the reinforced soil to carry the design loads. The existing fill (following proof compaction as appropriate) and/or glacial till anticipated at subgrade level are considered suitable for support of MSE walls.

5.4.2 Load and Resistance Factors

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV) and earth surcharge (ES) loads using the load factors for permanent loads (γ_p) provided in AASHTO Table 3.4.1-2 for strength and extreme limit state design.

The planned foundation types include footings bearing above MSE walls and the MSE walls themselves. Recommended AASHTO LRFD resistance factors for strength limit state design from AASHTO Tables 10.5.5.2.2-1 and 11.5.7-1 are presented below.

RESISTANCE FACTORS – STRENGTH LIMIT STATE		
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ)**
MECHANICALLY STABILIZED EARTH WALLS		
Bearing	MSE Walls	0.65
Sliding	MSE Walls	1.0
SPREAD FOOTINGS		
Sliding	Cast-in-Place Concrete on Sand	0.8

Resistance factors for service limit and extreme limit state design should be taken as 1.0 and 0.9, respectively.

5.4.3 Spread Footings over MSE Walls

Design parameters for spread footings bearing on MSE walls are presented in Section 5.6.5.4 of the BDG and include using a bearing resistance of 4 kips per square foot (ksf) for service limit state and a factored bearing resistance of 7 ksf for strength limit state to limit settlement to less than approximately 1/2 inch.

For evaluation of resistance of lateral loads imposed on the abutment, a representative sliding friction angle (ϕ_s') of 36 degrees should be assumed for crushed stone to be placed directly beneath abutment footings. Passive resistance should be neglected.

The MSE walls are anticipated to be designed by the wall vendor or their consultant. Recommended design parameters, materials and performance requirements are presented in **Section 6.6**.

5.4.4 MSE Wall Support

The subsurface profile below the MSE walls consists primarily of glacial till (very stiff to hard SILT, some Sand), potentially overlain by a thin layer of fill (recompacted glacial till). The MSE wingwalls will also typically bear on these materials, except near the ends of each wingwall where they may be supported over an increased thickness of existing fill.

Foundation design evaluations were based on the engineering properties of the glacial till and reinforced structural fill, which are summarized below:

- Total Unit Weight (Reinforced Fill) = 135 pounds per cubic foot (pcf)
- Total Unit Weight (Glacial Till) = 130 pcf
- Representative Internal Friction Angle (ϕ') = 35 degrees (for Glacial Till; MSE foundation soil)
- Representative Sliding Friction Angle (ϕ_s') = 32 degrees (for soil anticipated at Reinforced Fill-Glacial Till interface, reduced from internal friction angle due to silty nature of interface material)

Bearing resistance recommendations have been developed for MSE walls bearing on glacial till or proof-compacted existing fill based on these parameters and are presented in **Section 6.6** of this report.

5.4.5 Frost Penetration

Fine-grained soils are anticipated to be present beneath the MSE walls, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 2350. Therefore, in accordance with Table 5.1 of the BDG, the estimated depth of frost penetration in fine-grained soil with a moisture content of 10 percent is approximately 73 inches.

Standard MaineDOT practice has been to embed the bottom of the concrete leveling pad a minimum of 5 feet below grade. Considering the inclusion of a 1-foot-thick crushed stone below the leveling pad, the MSE wall will be underlain by non-frost susceptible material to 6 feet below finish grade; approximately the estimated frost penetration depth. Therefore, we conclude the bottom of the leveling pad should be a minimum of 5 feet below finish grade for this project.

Spread footings constructed above MSE walls are not typically embedded at the estimated frost penetration depth to avoid conflict with reinforcement in the soil mass, instead being underlain by coarse aggregate in accordance with Section 5.6.5.4 of the BDG. Spread footings should be underlain by crushed stone extending to the frost depth as recommended in **Section 6.4**.

5.4.6 Lateral Earth Pressure

We understand that the proposed abutments will be semi-integral abutments supported on spread footings bearing on the reinforced fill. In general, walls that are free to move at the top may be designed to resist active earth pressures. However, it is possible that the rotation of the abutments will be insufficient to mobilize active earth pressure. The minimum movement of the abutment to achieve active earth pressure, through translation of the abutment and/or rotation or bending of the stem, is lateral deflection equal to approximately 0.002 times the wall height (AASHTO Table C3.11.3-1). Therefore, at-rest earth pressure should be used to assess the deflection for structural modeling purposes, and if deflection is sufficient, active earth pressure can be used. Design live load surcharge should not be included in the deflection assessment.

Lateral earth pressure evaluations for abutments are based on LRFD and the BDG and are summarized below:

- The proposed abutment footings are anticipated to consist of "short-heeled" walls in accordance with LRFD Figure C3.11.5.3-1. Therefore, Coulomb Theory was used to calculate the active earth pressure coefficients.
- Passive resistance in front of the abutment footing is neglected for sliding and overturning evaluations.
- Soil Type 4 (Granular Borrow for Underwater Backfill Material or Granular Borrow; MaineDOT Item 703.19) will be placed as backfill behind abutments and was used to develop earth pressure coefficients in accordance with Table 3-3 of the BDG.
- Live load horizontal surcharge pressures will be evaluated in accordance with Table 3-4 of the BDG and AASHTO Article 3.11.6.4.
- AASHTO Commentary C3.10.9.1 specifies that single-span bridges are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

Design lateral earth pressure recommendations are provided in **Section 6.5** of this report.

6.0 RECOMMENDATIONS

6.1 EARTH SLOPES

- Earth fill embankment slopes designed with slope angles of 2H:1V or flatter should be provided with loam and seed for permanent erosion protection. If soil slopes steeper than approximately 2H:1V are anticipated, bio-degradable, erosion control mat should be provided in addition to the loam and seed to protect the slopes from surface erosion until a root mass has developed. Alternatively, riprap slope protection may be used in accordance with MaineDOT standard details.

6.2 FROST AND SUBGRADE CONDITIONS FOR PAVEMENT DESIGN

- Based on the MaineDOT BDG, the Design Freezing Index for the site is approximately 2350. In accordance with Table 5.1 of the BDG, the estimated depth of frost penetration in fine-grained soil with a moisture content of 10 percent is about 73 inches for this project. The pavement design should consider the presence of frost-susceptible soil below the pavement section within the depth of freezing.
- The new approach paved areas will be constructed within a few feet of existing embankment grades, and the pavement section will either be underlain by existing subbase material or existing embankment fill. We recommend assuming a Frost Classification of IV for the existing fill pavement subgrade materials.

6.3 SEISMIC DESIGN PARAMETERS

The United States Geological Survey online Design Maps Tool was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years. These results are summarized for the site as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.111 g
SDs (Period = 0.2 sec)	0.256 g
SD1 (Period = 1.0 sec)	0.123 g

Per LRFD Section 4.7.4.2, single span bridges need not be analyzed for seismic loads, but the minimum requirements for superstructure connections and support lengths apply as specified in AASHTO Articles 4.7.4.4 and 3.10.9.

6.4 SPREAD FOOTING DESIGN CONSIDERATIONS (OVER REINFORCED SOIL)

- Abutments should be supported on spread footing foundations bearing directly on reinforced soil behind MSE walls. Footings should be sized using the design parameters for spread footings bearing over MSE walls presented Section 5.6.5.4 of the BDG to limit settlement to approximately ½ inch or less. We recommend the following for bearing resistance of a spread footing on top of the MSE reinforced soil zone:
  - Strength: Factored bearing resistance = 7 ksf; and
  - Service: Bearing resistance = 4 ksf
- Nominal sliding resistance for footings underlain by crushed stone is equal to the vertical force multiplied by the sliding resistance coefficient (tan ϕ_s, or tan 36°), which is equal to 0.73. Based on a resistance factor of 0.8 for the Strength condition, the recommended factored sliding resistance coefficient is 0.58.
- Passive resistance of soil backfill on the front of abutment footings and wingwalls should be neglected.
- Footings should be embedded at least 2 feet below the backfill level at the top of the wall, per section 5.6.5.4 per the BDG, and should be underlain by at least 6.5 feet of crushed stone for frost protection. Crushed stone should consist of Crushed Stone ¾-Inch (MaineDOT Item 703.13). The lower portion of the crushed stone layer will extend within the reinforced zone, where it will replace structural fill that is otherwise recommended for use as reinforced soil (see **Section 6.6**).
- The MSE wall designer should be responsible for developing a wall design that is capable of supporting the recommended foundation bearing resistances, and should confirm that these parameters can be achieved in their reinforced earth design.

6.5 ABUTMENT DESIGN

- Backfill behind new abutments, above the top of MSE wall elevation, should consist of Granular Borrow for Underwater Backfill (MaineDOT Item 703.19), BDG Type 4 soil. Recommended soil properties for Type 4 soils to be used as backfill are as follows:
  - Internal Friction Angle of Soil = 32°
  - Soil Total Unit Weight = 125 pounds per cubic foot
  - Coefficient of Active Earth Pressure, K_a = 0.44
  - Coefficient of At-Rest Earth Pressure, K_o = 0.47
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with AASHTO Article 3.11.6.4. A minimum H_{eq} of 2 feet is recommended.

- Considering the presence of the approach slab and the below-grade geomembrane that slopes away from the abutment to a perforated drain pipe, foundation drainage is not required on the uphill side of abutments.

6.6 MSE WALL DESIGN

- MSE wall design should be in accordance with the requirements of Standard Specification 677 (Mechanically Stabilized Earth Retaining Wall), Section 11.10 of LRFD and Section 5.6.5.4 of the BDG.
- The bearing surface for the concrete leveling pad and reinforcing straps should consist of proof-compacted existing fill or undisturbed, naturally-deposited glacial till. If weak/unstable areas are exposed during proof compaction, these areas should be dewatered as necessary and overexcavated to expose firm, undisturbed material, and the resulting excavation should be backfilled with structural fill meeting the requirements of Granular Borrow for Underwater Backfill (MaineDOT Item 703.19).
- Bearing resistance values were developed assuming the bearing material will consist of proof-compacted existing fill or glacial till. The bearing pressure imposed by MSE retaining walls should be less than or equal to the bearing resistance values presented in the table below.

BEARING RESISTANCE FOR MSE WALLS			
Nominal Bearing Resistance, φ=1.0	**Extreme Limit State, φ=0.9**	**Strength Limit State, φ=0.65**	**Service Limit State**
30 ksf	27 ksf	19.5 ksf	11 ksf

- The base resistance against sliding should be evaluated using $\phi_s' = \delta = 32$ degrees. Nominal sliding resistance for MSE walls is equal to the vertical force multiplied by the sliding resistance coefficient (tan δ), which is equal to 0.62. Based on a resistance factor of 0.8 for the Strength condition, the recommended factored sliding resistance coefficient is 0.50.
- MSE wall systems should be capable of sustaining post-construction differential settlements up to 1V:100H. However, differential settlement will be significantly less than 1V:100H for this project.
- The bottom of the concrete leveling pad placed beneath the MSE wall facing should be embedded at least 5 feet below finish grades and should be underlain by 12 inches of crushed stone for frost protection.
- Fill in the reinforcement zone should be specified as Gravel Borrow (MaineDOT Item 703.20), except where crushed stone is required beneath the abutment footing. The fill in the reinforcement zone should meet the electrochemical and plasticity requirements in Standard Specification Section 677.048.
- A minimum uniform vertical surcharge load of 250 pounds per square foot should be applied on the roadway surface above the wall. Lateral surcharge resulting from this load should be calculated and provided by the wall designer.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 EXCAVATION AND DEWATERING

Excavation to construct the MSE walls will extend between 9 feet (adjacent to existing track level) and 34 feet (the maximum depth into the existing embankment) below existing grade to reach the bottom elevation of the MSE wall. The excavations are primarily anticipated to be in existing fill, extending a few feet into glacial till in some locations. Bedrock is not anticipated to be encountered during excavation.

Sloped, open-cut excavations generally appear to be feasible to construct the MSE walls, assuming Route 6/15 is closed or re-routed during bridge construction. However, temporary excavation support (e.g., sheet piling) may be preferred in some areas by the contractor to limit excavation and replacement requirements outside of the MSE reinforced soil zone. Excavations adjacent to the active rail line should be completed in accordance with the rail operator's requirements. Typically, any excavation that extends into the theoretical railroad embankment zone will require an approved temporary lateral support system designed by registered professional engineer. The theoretical railroad embankment zone is typically defined as the area within a line that extends horizontally, 10 feet from the centerline of track, then downward and outward on a 1.5H:1V slope, but this may vary based on specific requirements of the rail operator.

Temporary dewatering will likely be required to control groundwater inflow in excavations or surface water runoff into excavations. We anticipate that the inflow of groundwater and infiltration to excavations can be handled by open pumping from sumps installed at the bottoms of excavations. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping.

The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater should comply with all local, State, and federal regulations.

7.2 MSE BEARING PAD AND FOUNDATION SUBGRADE PREPARATION

Excavation to MSE bearing pad level should be performed by means that minimize disturbance of soil subgrades. In general, the exposed soil bearing surface beneath the leveling pad and MSE reinforcement should be proof-compacted with a minimum 10,000-pound vibratory roller prior to MSE wall construction. Weak areas identified during proof compaction should be excavated and replaced with Granular Borrow for Underwater Backfill. Where soil bearing surfaces are silty and/or near groundwater level, proof compaction may create subgrade instability and can be conducted with non-vibratory methods or waived by the Resident if appropriate. Excavations in saturated, fine-grained soils should be made with a smooth-edged bucket to limit disturbance. If the subgrade becomes disturbed by construction activities, the disturbed material should be excavated and replaced with Granular Borrow for Underwater Backfill.

7.3 FILL MATERIAL AND PLACEMENT RECOMMENDATIONS

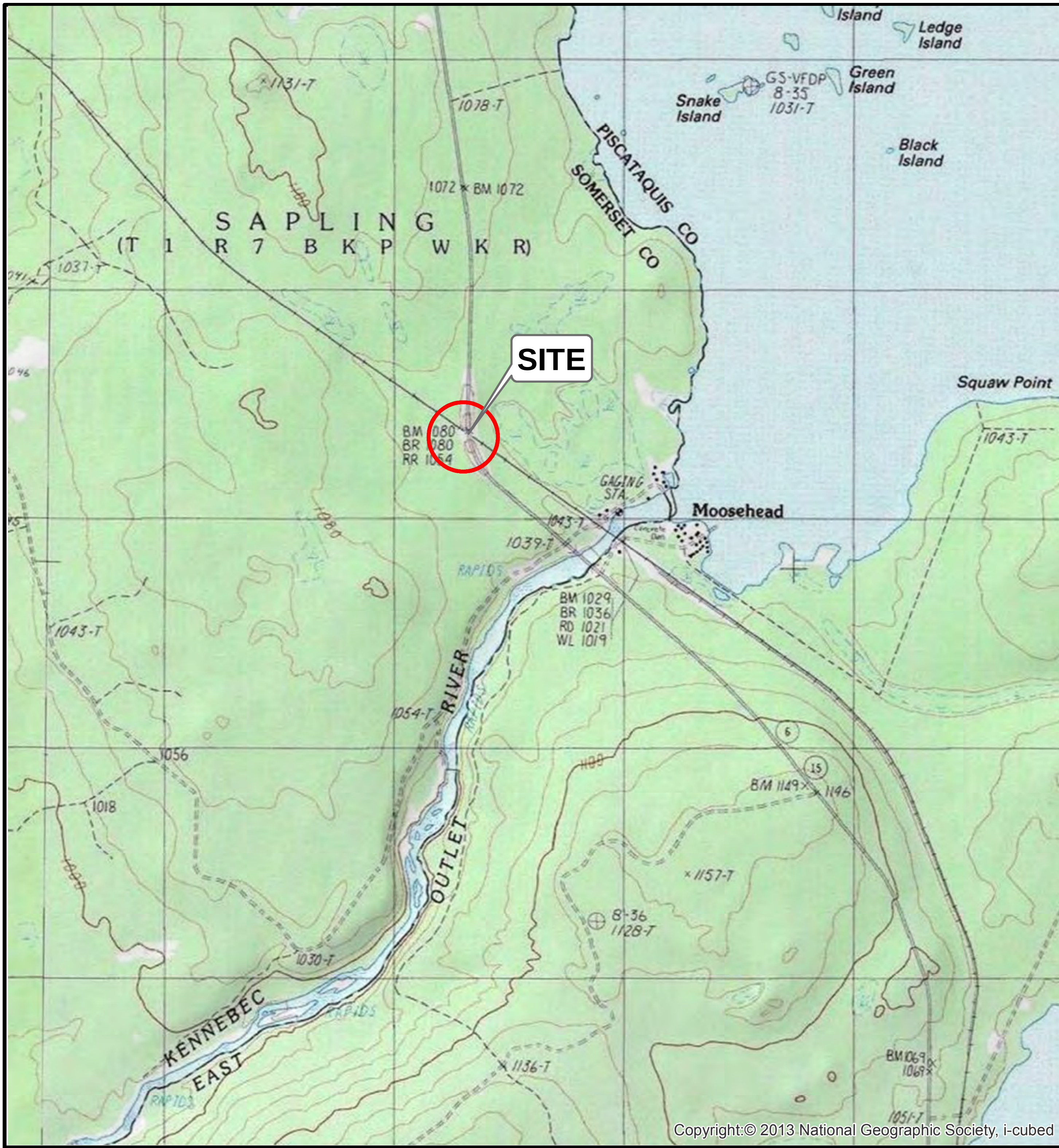
Backfill for the proposed abutment wall should consist of Granular Borrow for Underwater Backfill and should be placed and compacted in accordance with the MaineDOT Standard Specifications, Section 203. Fill placed within or behind the reinforced zone should be in accordance with MaineDOT Standard Specification Section 677.

7.4 REUSE OF EXCAVATED MATERIALS

- In general, we anticipate that the excavated fill and Glacial Till will be suitable for reuse as Common Borrow (MaineDOT Item 703.18), assuming it can be moisture conditioned to within 2 percent of optimum moisture content to allow for compaction of the material.
- We anticipate that the excavated fill and Glacial Till will not be suitable for reuse as Granular Borrow (MaineDOT Item 703.19).

If the contractor wishes to reuse portions of the excavated material as embankment fill, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.

Figures



USGS
QUADRANGLE
LOCATION

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SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS MAP ALSO CONTAINS THE ESRI ARCGIS ONLINE USA COUNTIES WHICH PROVIDES DETAILED BOUNDARIES THAT ARE CONSISTENT WITH THE TRACT, BLOCK GROUP, AND STATE DATA SETS AND ARE EFFECTIVE AT REGIONAL AND STATE LEVELS.

0 1,000 2,000 4,000 6,000

SCALE IN FEET

Data Supplied by :

esri

USGS science for a changing world

PROJ. MGR.: BMC
DESIGNED BY: EDF
REVIEWED BY: ARB
OPERATOR: ADM

DATE: 11-08-2017

LOCUS PLAN

BRIDGE NO. 3256
SAPLING TOWNSHIP, MAINE

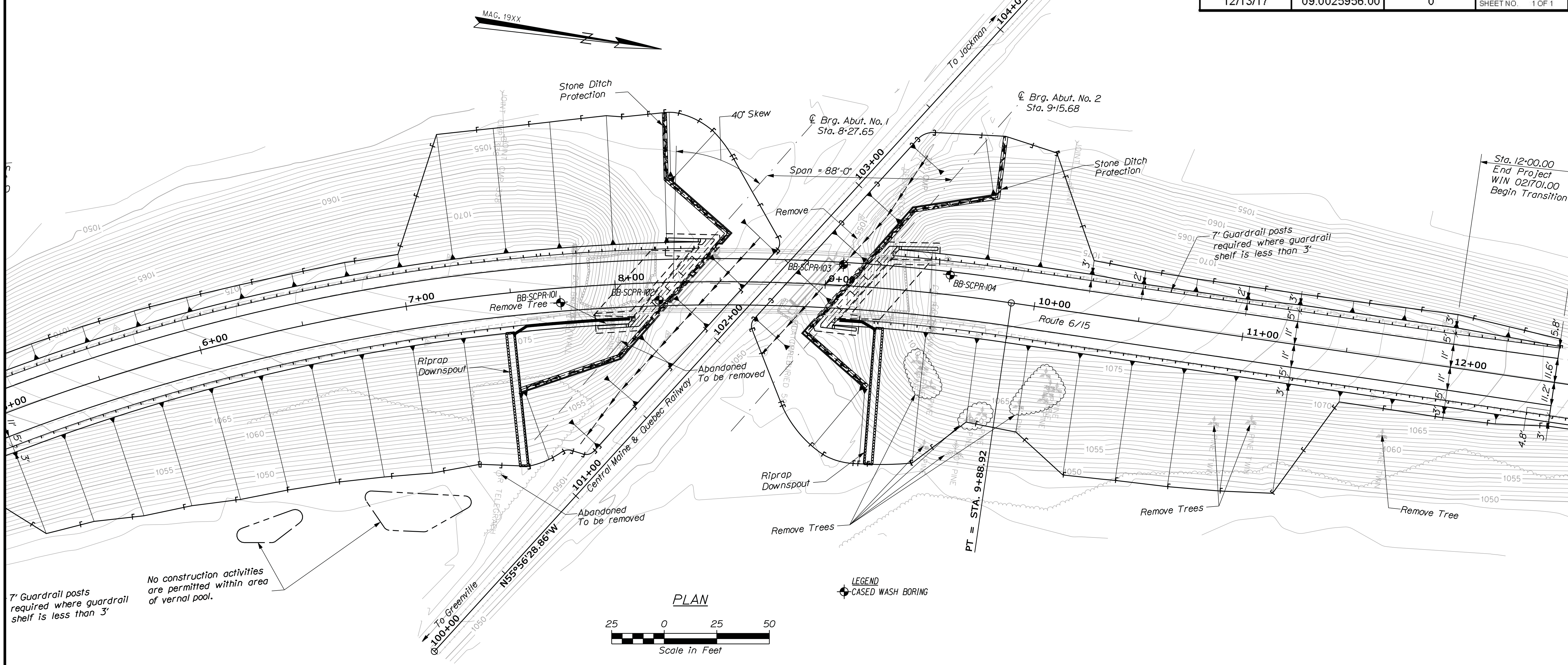
JOB NO.
09.0025956.00

FIGURE NO.
1

CPR CROSSING BRIDGE, CENTRAL MAINE & QUEBEC RAILWAY
MAINEDOT WIN 021701.00
SAPLING TOWNSHIP, ME

BORING LOCATION PLAN

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINE DEPARTMENT OF TRANSPORTATION	
PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE 2
DESIGNED BY: EF	DRAWN BY: TW	SCALE: AS SHOWN	
DATE: 12/13/17	PROJECT NO. 09.0025956.00	REVISION NO. 0	SHEET NO. 1 OF 1



STATE OF MAINE DEPARTMENT OF TRANSPORTATION

021701.00

WIN 021701.00

BRIDGE NO. 3256

BRIDGE PLANS

	MAP	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	REM	BJN	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	A.BLAISDELL	T.WHITE	NOV 2017
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

CPR CROSSING BRIDGE
CENTRAL MAINE & QUEBEC RAILWAY
SAPLING TWP SOMERSET COUNTY

BORING LOCATION PLAN

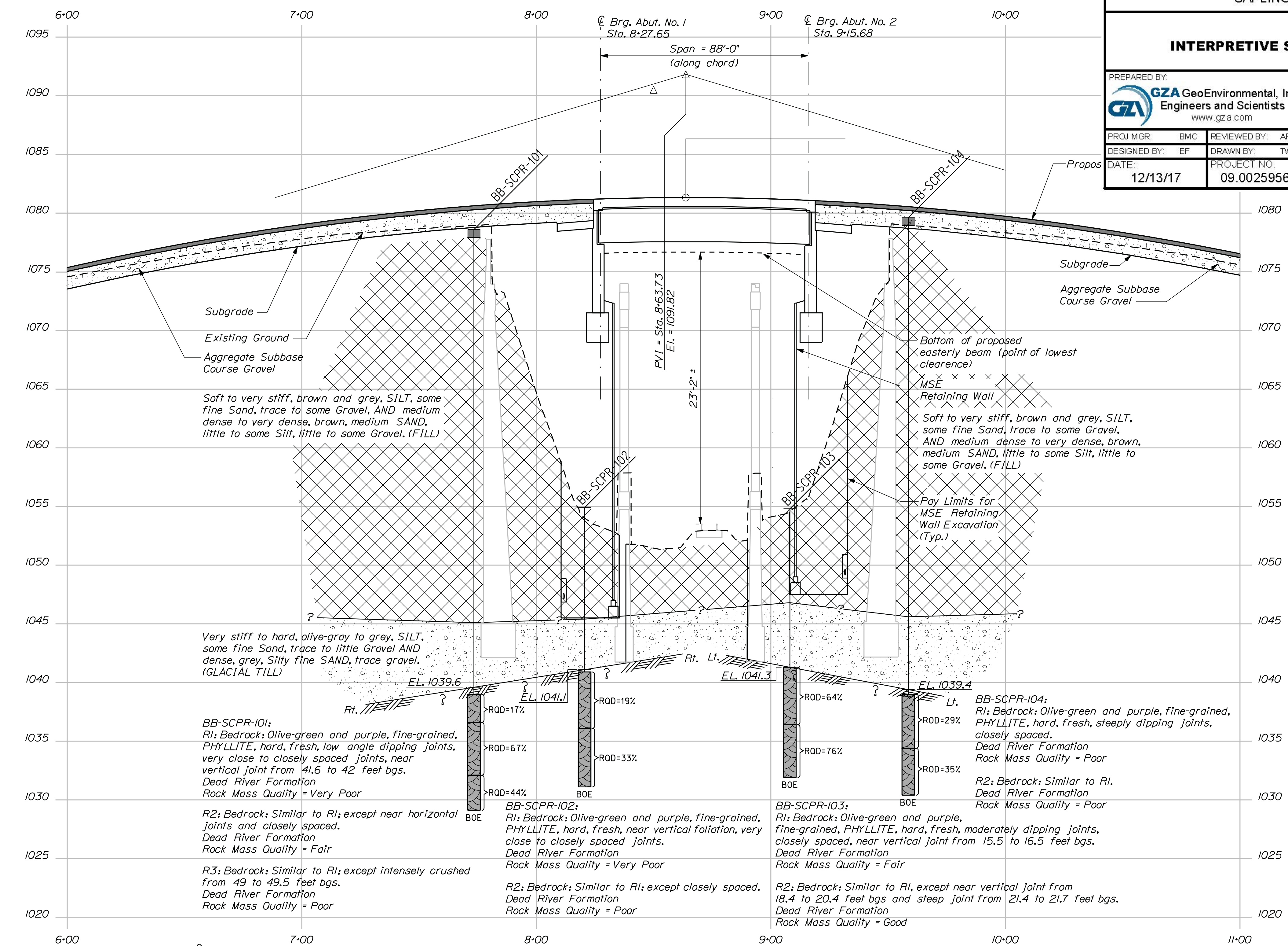
SHEET NUMBER
2
OF 4

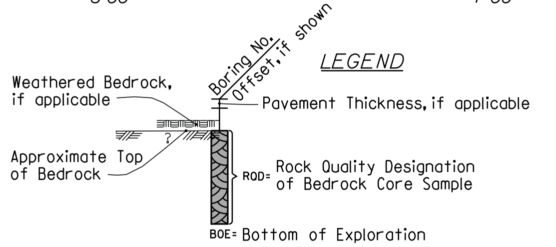
Filename: ...\00\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: Terry.White Date:11/14/2017

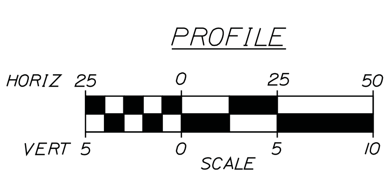
CPR CROSSING BRIDGE, CENTRAL MAINE & QUEBEC RAILWAY
MAINEDOT WIN 021701.00
SAPLING TOWNSHIP, ME

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINE DEPARTMENT OF TRANSPORTATION	
PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE 3
DESIGNED BY: EF	DRAWN BY: TW	SCALE: AS SHOWN	
DATE: 12/13/17	PROJECT NO. 09.002595600	REVISION NO. 0	SHEET NO. 1 OF 1







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.

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: Terry.White Date: 11/14/2017

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
021701.00
WIN 021701.00
BRIDGE PLANS
BRIDGE NO. 3256

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	REM	BJN	
CHECKED-REVIEWED	A.BLAISDELL	-----	NOV 2017
DESIGN2-DETAILED2	E.FREDE	T.WHITE	NOV 2017
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE
P.E. NUMBER
DATE

CPR CROSSING BRIDGE
CENTRAL MAINE & QUEBEC RAILWAY
SAPLING TWP SOMERSET COUNTY
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Appendix A – Limitations

GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the contract documents, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions .

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

11. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

Appendix B – Test Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-101
WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1078.6	**Auger ID/OD:**	2.75-inch ID/6.25-inch OD
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/13,20/2017	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 7+73.4, 6.9 feet Rt.	**Casing ID/OD:**	NW (3/3.5-inch)	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	1077.9		8-inch-thick layer of pavement. 0.67	
										Frost depth to 2 feet bgs.	
5	1D	24/17	5.00 - 7.00	9/10/9/8	19	19				Brown, moist, very stiff, SILT, some fine sand, little fine gravel. (FILL)	G#270154 A-4, ML WC=9.3%
10	2D	24/16	10.00 - 12.00	5/5/5/5	10	10				Similar to Sample 1D; except stiff.	
15	3D	24/3	15.00 - 17.00	2/4/5/6	9	9				Similar to Sample 1D; except stiff.	
20	4D	24/14	20.00 - 22.00	2/2/3/4	5	5				Olive-grey, wet, medium stiff, SILT, some fine sand, little fine gravel, trace clay. (FILL)	G#270155 A-4, ML WC=13.4%
25											

Remarks:

-bgs = below existing ground surface (roadway)

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: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-101
WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1078.6	**Auger ID/OD:**	2.75-inch ID/6.25-inch OD
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/13,20/2017	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 7+73.4, 6.9 feet Rt.	**Casing ID/OD:**	NW (3/3.5-inch)	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/18	25.50 - 27.50	3/5/3/8	8	8				Similar to Sample 4D.	
30	6D	24/16	30.00 - 32.00	2/2/11/5	13	13				Olive-grey, wet, stiff, SILT, some fine sand, little fine gravel, trace clay. (FILL)	
								1045.1		33.50	
35	7D	24/15	35.00 - 37.00	5/11/21/37	32	32				Olive-grey, wet, hard, SILT, some fine sand, little fine gravel, trace clay.	G#270156 A-4, ML WC=9.2%
										Cobble from 38.2 to 38.5 feet bgs.	
								1039.6		39.00	
40	R1	28.8/25	39.60 - 42.00	RQD = 17%			NQ2	1039.0		Top of Bedrock at Elevation 1039.6 feet. Set in NW Casing to 39 feet bgs. RC to 39.6 feet bgs.	
										39.60	
								1036.6		R1: Bedrock: Olive-green and purple, fine-grained, PHYLLITE, hard, fresh, low angle dipping joints, very close to closely spaced joints, near vertical joint from 41.6 to 42 feet bgs. R1: Core Times (min:sec) 39.6-40.6 feet (6:12) 40.6-41.6 feet (4:50) 41.6-42.0 feet (5:00) Core Blocked Dead River Formation Rock Mass Quality = Very Poor	
	R2	54/43	42.00 - 46.50	RQD = 67%							
45										42.00	
	R3	36/36	46.50 - 49.50	RQD = 44%				1032.1		R2: Bedrock: Similar to R1; except near horizontal joints and closely spaced. R2: Core Times (min:sec) 42-43 feet (3:12) 43-44 feet (3:26) 44-45 feet (3:07) 45-46 feet (3:30) 46-46.5 feet (4:00)	
50								1029.1			

Remarks:

-bgs = below existing ground surface (roadway)

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:** CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad **Location:** Sapling Township, Maine	**Boring No.:** BB-SCPR-101 **WIN:** 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1078.6	**Auger ID/OD:**	2.75-inch ID/6.25-inch OD
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/13,20/2017	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 7+73.4, 6.9 feet Rt.	**Casing ID/OD:**	NW (3/3.5-inch)	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										Dead River Formation Rock Mass Quality = Fair 46.50 R3: Bedrock: Similar to R1; except intensely crushed from 49 to 49.5 feet bgs. R3: Core Times (min:sec) 46.5-47.5 feet (3:31) 47.5-48.5 feet (2:48) 48.5-49.5 feet (3:00) Dead River Formation Rock Mass Quality = Poor 49.50	
55										**Bottom of Exploration at 49.50 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

-bgs = below existing ground surface (roadway)

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: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-102
WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1054.9	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/20,21/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 8+20.6, 8.3 feet Rt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not Observed
Hammer Efficiency Factor: 0.60		**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	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0											
							41				
	1D	24/15	2.00 - 4.00	8/11/7/9	18	18	26			Brown, wet, very stiff, SILT, some fine sand, some fine gravel, trace clay. (FILL)	G#271124 A-4, ML WC=12.8%
							26				
							19				
5							24				
	2D	24/13	6.00 - 8.00	8/4/3/4	7	7	13			Grey, wet, medium stiff, SILT, some fine sand, little fine gravel, trace clay. (FILL)	G#271125 A-4, ML WC=14.5%
							9				
							12				
							14	1045.4		9.50	
10							95				
	3D	24/15	11.00 - 13.00	12/13/25/30	38	38	49			Grey, wet, dense, Silty fine SAND, trace fine gravel, trace clay.	G#270151 A-4, SM WC=11.9%
							63				
							a153	1041.1		a153 blows for 9.6 inches. 13.80	
	R1	57.6/55	14.00 - 18.80	RQD = 19%			NQ2	1040.9		Top of Bedrock at Elevation 1041.1 feet. RC from 13.8 to 14 feet bgs. 14.00	
15										R1: Bedrock: Olive-green and purple, fine-grained, PHYLLITE, hard, fresh, near vertical foliation, very close to closely spaced joints.	
										R1: Core Times (min:sec) 14-15 feet (2:55) 15-16 feet (3:16)	
	R2	60/55	18.80 - 23.80	RQD = 33%				1036.1		16-17 feet (3:21) 17-18 feet (3:20) 18-18.8 feet (3:45) Core Blocked	
20										Dead River Formation Rock Mass Quality = Very Poor 18.80	
										R2: Bedrock: Similar to R1; except closely spaced. R2: Core Times (min:sec) 18.8-19.8 feet (2:40) 19.8-20.8 feet (2:42)	
								1031.1		20.8-21.8 feet (2:32) 21.8-22.8 feet (2:43) 22.8-23.8 feet (2:30) Dead River Formation	
25											

Remarks:

-bgs = below existing ground surface (near the railroad elevation).
-24.2 feet from bridge deck to existing ground surface.
-Augered 8.5 inches through the concrete bridge deck.

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: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-102
WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1054.9	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/20,21/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 8+20.6, 8.3 feet Rt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										Rock Mass Quality = Poor	
										23.80	
										Bottom of Exploration at 23.80 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

-bgs = below existing ground surface (near the railroad elevation).
-24.2 feet from bridge deck to existing ground surface.
-Augered 8.5 inches through the concrete bridge deck.

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: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-103

WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1054.8	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/16,20/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	
Boring Location:	Sta 9+08.1, 10.0 feet Lt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not Observed
Hammer Efficiency Factor: 0.60		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	6/3	0.00 - 0.50	40	---		84	1054.3		Brown, frozen, concrete slag. 0.50	
							293				
							87				
							48				
							24				
5	2D	24/14	5.00 - 7.00	17/46/17/17	63	63	32			Grey-brown, moist, very dense, medium SAND, some silt, some fine gravel, trace clay. (FILL)	G#270152 A-2-4, SM WC=10.4%
							58				
							52				
							81	1046.8		8.00	
							31				
10	3D	24/15	10.00 - 12.00	5/18/28/28	46	46	53			Grey, moist, hard, SILT, some fine sand, trace clay, trace fine gravel.	G#270153 A-4, ML WC=9.9%
							202				
							163			a106 blows for 6 inches.	
	R1	58.8/52	13.50 - 18.40	RQD = 64%			a106 NQ-2	1041.3		13.50 Top of Bedrock at Elevation 1041.3 feet. R1: Bedrock: Olive-green and purple, fine-grained, PHYLLITE, hard, fresh, moderately dipping joints, closely spaced, near vertical joint from 15.5 to 16.5 feet bgs. R1: Core Times (min:sec) 13.5-14.5 feet (4:00) 14.5-15.5 feet (2:26) 15.5-16.5 feet (2:20) 16.5-17.5 feet (2:19) 17.5-18.4 feet (3:00) Core Blocked Dead River Formation Rock Mass Quality = Fair	
15											
	R2	54/53	18.40 - 22.90	RQD = 76%				1036.4		18.40 R2: Bedrock: Similar to R1, except near vertical joint from 18.4 to 20.4 feet bgs and steep joint from 21.4 to 21.7 feet bgs. R2: Core Times (min:sec) 18.4-19.4 feet (2:15) 19.4-20.4 feet (2:30) 20.4-21.4 feet (3:06) 21.4-22.4 feet (3:50) 22.4-22.9 feet (4:00) Core Blocked Dead River Formation	
20											
								1031.9			
25											

Remarks:

-bgs = below existing ground surface (near the railroad elevation).
-25.4 feet from bridge deck to existing ground surface.
-Augered 8 inches through the concrete bridge deck.

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

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-103

WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1054.8	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/16,20/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	
Boring Location:	Sta 9+08.1, 10.0 feet Lt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										Rock Mass Quality = Good	
										22.90	
										Bottom of Exploration at 22.90 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

-bgs = below existing ground surface (near the railroad elevation).
-25.4 feet from bridge deck to existing ground surface.
-Augered 8 inches through the concrete bridge deck.

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: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-104
WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1079.6	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/9,13/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 9+58.6, 9.4 feet Lt.	**Casing ID/OD:**	HW (4/4.5") & NW (3/3.5")	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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.
0							SSA	1078.9		8-inch-thick layer of pavement. 0.67	
	1D	6/3	1.00 - 1.50	50	---					Brown, frozen, medium SAND, little fine gravel, little silt. (FILL) Frost depth to 2 feet bgs.	G#271119 A-1-b, SM WC=3.3%
5	2D	24/15	5.00 - 7.00	12/14/11/12	25	25				Similar to Sample 1D; except medium dense. (FILL)	
								1070.6		9.00	
10	3D	24/16	10.00 - 12.00	4/5/7/9	12	12				Olive-brown, moist, stiff, SILT, some fine sand, trace fine gravel. (FILL)	G#271120 A-4, ML WC=11.1%
15	4D	24/16	15.00 - 17.00	3/4/5/8	9	9	36			Similar to Sample 3D.	
							39				
							85				
							44				
							39				
20	5D	24/10	20.00 - 22.00	9/6/6/6	12	12	35			Brown, wet, medium dense, Silty fine SAND , some fine gravel, trace clay. (FILL)	G#271121 A-4, SM WC=10.3%
							31				
							39				
							43				
							33				
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-104
WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1079.6	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/9,13/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 9+58.6, 9.4 feet Lt.	**Casing ID/OD:**	HW (4/4.5") & NW (3/3.5")	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/11	25.00 - 27.00	6/6/6/9	12	12	OPEN HOLE			Similar to Sample 5D.	
30	7D	24/18	30.50 - 32.50	7/7/9/14	16	16				Grey-brown, wet, very stiff, SILT, some fine sand, trace fine gravel, trace clay.	G#271122 A-4, ML WC=12.1%
								1045.6		34.00	
35	8D	24/19	35.50 - 37.50	35/41/33/48	74	74				Grey, wet, hard, SILT, some fine sand, little fine gravel, little clay.	G#271123 A-4, ML WC=8.5%
40	9D	2.4/2.4	40.00 - 40.20	50(2.4-in)	---		a80	1039.4		Similar to Sample 8D. a80 blows for 2.4 inches.	
	R1	55.2/51	40.60 - 45.20	RQD = 29%			NQ2	1039.0		40.20 Top of Bedrock at Elevation 1039.4 feet. RC to 40.6 feet bgs. 40.60 R1: Bedrock: Olive-green and purple, fine-grained, PHYLLITE, hard, fresh, steeply dipping joints, closely spaced. R1: Core Times (min:sec) 40.6-41.6 feet (6:15) 41.6-42.6 feet (6:23) 42.6-43.6 feet (3:10) 43.6-44.6 feet (3:19) 44.6-45.2 feet (2:41) Core Blocked Dead River Formation Rock Mass Quality = Poor	
45	R2	48/48	45.20 - 49.20	RQD = 35%				1034.4		45.20 R2: Bedrock: Similar to R1. R2: Core Times (min:sec) 45.2-46.2 feet (3:20) 46.2-47.2 feet (3:05)	
								1030.4			
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CPR Crossing Bridge (No. 3256) carries Routes 6 & 15 over CP Railroad
Location: Sapling Township, Maine

Boring No.: BB-SCPR-104
WIN: 21701.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	1079.6	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lbs/30 inches
Date Start/Finish:	3/9,13/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-diameter)
Boring Location:	Sta 9+58.6, 9.4 feet Lt.	**Casing ID/OD:**	HW (4/4.5") & NW (3/3.5")	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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N_{60} = (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										47.2-48.2 feet (3:00) 48.2-49.2 feet (5:00) Dead River Formation Rock Mass Quality = Poor	
										49.20 **Bottom of Exploration at 49.20 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

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Appendix C – Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Sapling Township Work Number: 21701.00

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-SCPR-101, 1D	7+73.4	6.9 Rt.	5.0-7.0	270154	1	9.3			ML	A-4	IV
BB-SCPR-101, 4D	7+73.4	6.9 Rt.	20.0-22.0	270155	1	13.4			ML	A-4	IV
BB-SCPR-101, 7D	7+73.4	6.9 Rt.	35.5-37.0	270156	1	9.2			ML	A-4	IV
BB-SCPR-102, 1D	8+20.6	8.3 Rt.	2.0-4.0	271124	2	12.8			ML	A-4	IV
BB-SCPR-102, 2D	8+20.6	8.3 Rt.	6.0-8.0	271125	2	14.5			ML	A-4	IV
BB-SCPR-102, 3D	8+20.6	8.3 Rt.	11.0-13.0	270151	2	11.9			SM	A-4	IV
BB-SCPR-103, 2D	9+08.1	10.0 Lt.	5.0-7.0	270152	3	10.4			SM	A-2-4	III
BB-SCPR-103, 3D	9+08.1	10.0 Lt.	10.0-12.0	270153	3	9.9			ML	A-4	IV
BB-SCPR-104, 1D	9+58.6	9.4 Lt.	1.0-1.5	271119	4	3.3			SM	A-1-b	II
BB-SCPR-104, 3D	9+58.6	9.4 Lt.	10.0-12.0	271120	4	11.1			ML	A-4	IV
BB-SCPR-104, 5D	9+58.6	9.4 Lt.	20.0-22.0	271121	4	10.3			SM	A-4	IV
BB-SCPR-104, 7D	9+58.6	9.4 Lt.	30.5-32.5	271122	4	12.1			ML	A-4	IV
BB-SCPR-104, 8D	9+58.6	9.4 Lt.	35.5-37.5	271123	4	8.5			ML	A-4	IV

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

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

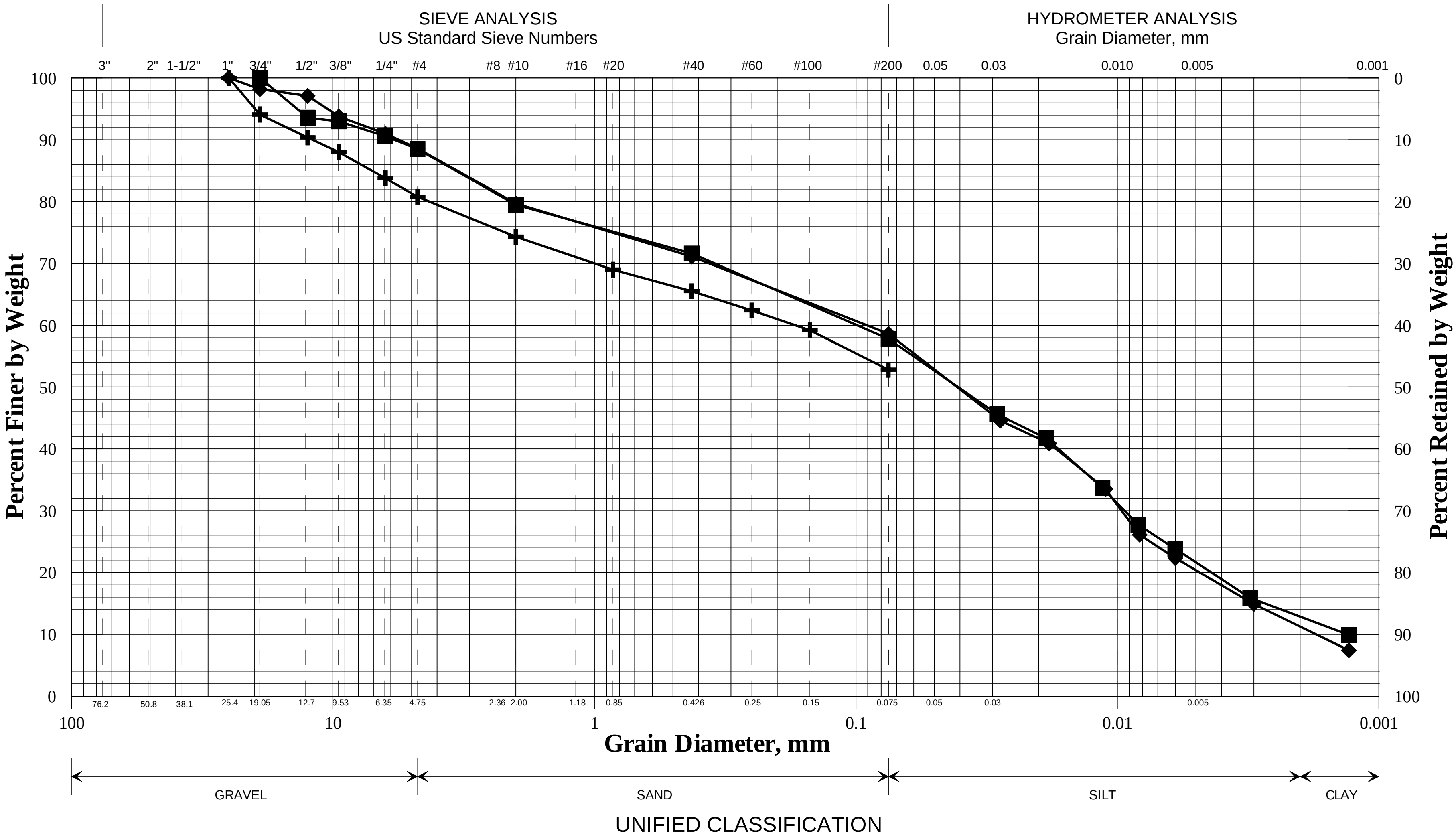
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE

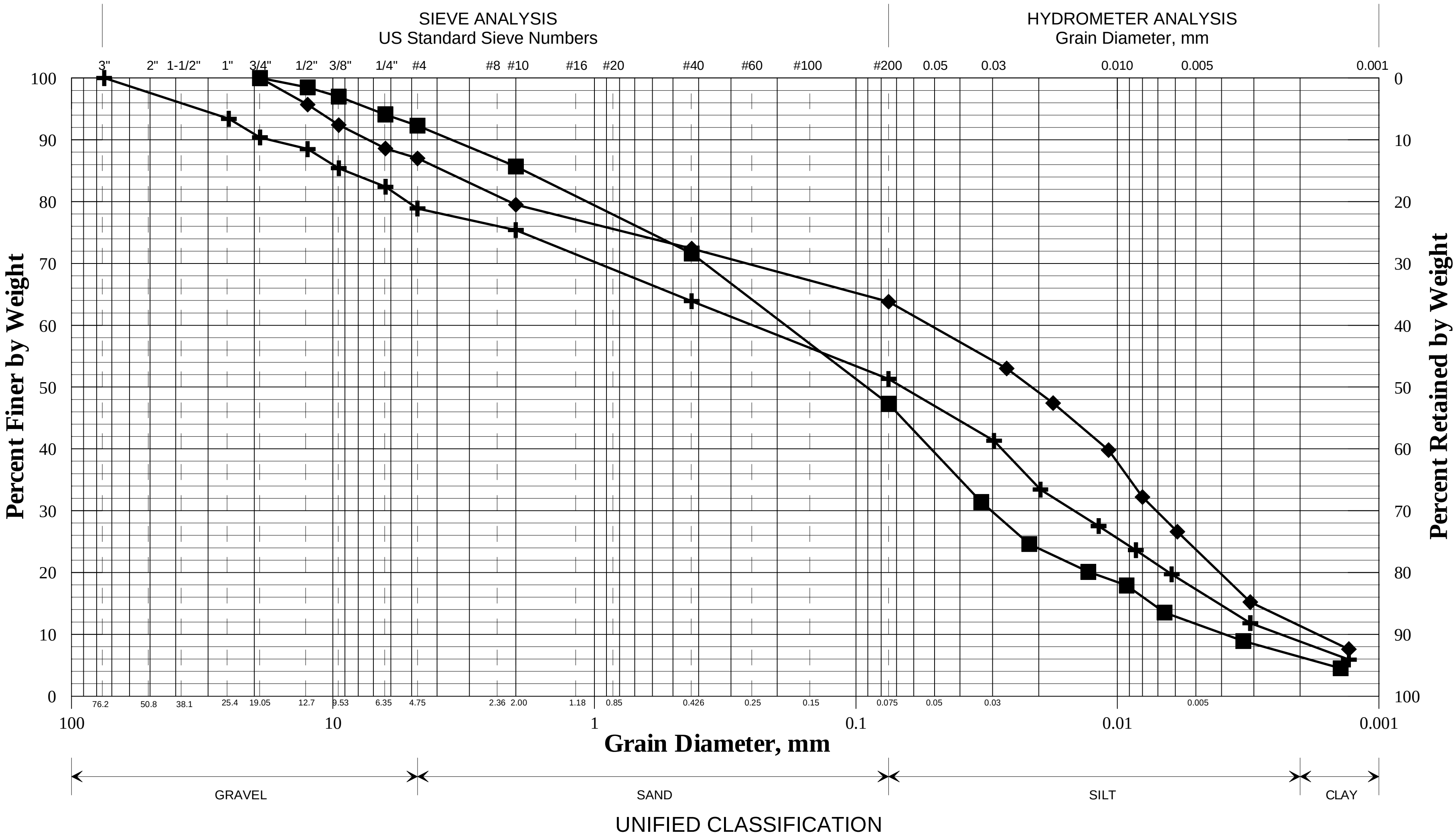


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SCPR-101/1D	7+73.4	6.9 RT	5.0-7.0	SILT, some sand, little gravel.	9.3			
◆	BB-SCPR-101/4D	7+73.4	6.9 RT	20.0-22.0	SILT, some sand, trace gravel, trace clay.	13.4			
■	BB-SCPR-101/7D	7+73.4	6.9 RT	35.5-37.0	SILT, some sand, trace clay, trace gravel.	9.2			
●									
▲									
✖									

WIN
021701.00
Town
Sapling Twp
Reported by/Date
WHITE, TERRY A 4/28/2017

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

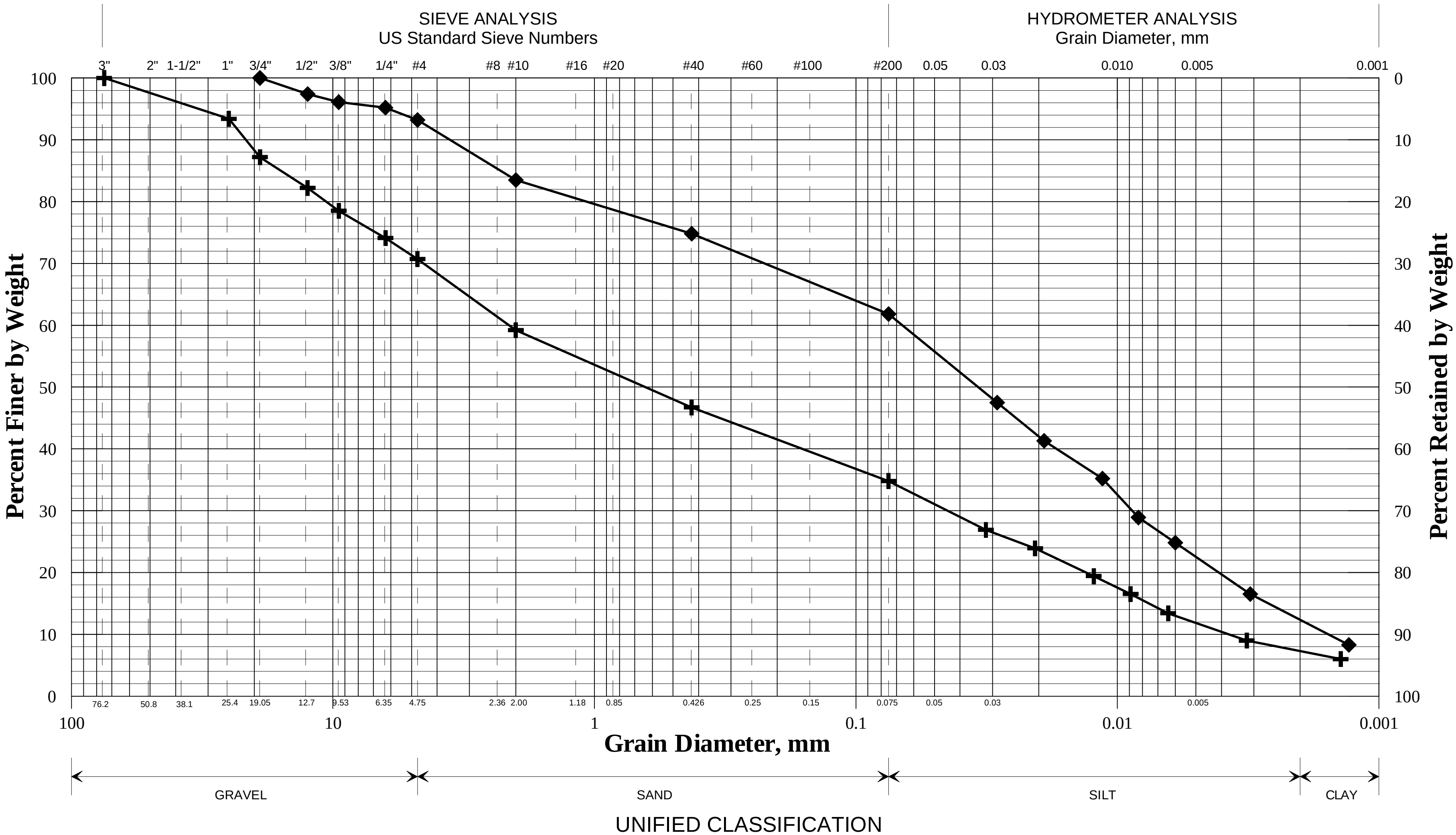


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SCPR-102/1D	8+20.6	8.3 RT	2.0-4.0	SILT, some sand, some gravel, trace clay.	12.8			
◆	BB-SCPR-102/2D	8+20.6	8.3 RT	6.0-8.0	SILT, some sand, little gravel, little clay.	14.5			
■	BB-SCPR-102/3D	8+20.6	8.3 RT	11.0-13.0	Silty SAND, trace gravel, trace clay.	11.9			
●									
▲									
✖									

WIN
021701.00
Town
Sapling Twp
Reported by/Date
WHITE, TERRY A 4/28/2017

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SCPR-103/2D	9+08.1	10.0 LT	5.0-7.0	SAND, some gravel, some silt, trace clay.	10.4			
◆	BB-SCPR-103/3D	9+08.1	10.0 LT	10.0-12.0	SILT, some sand, little clay, trace gravel.	9.9			
■									
●									
▲									
✖									

WIN
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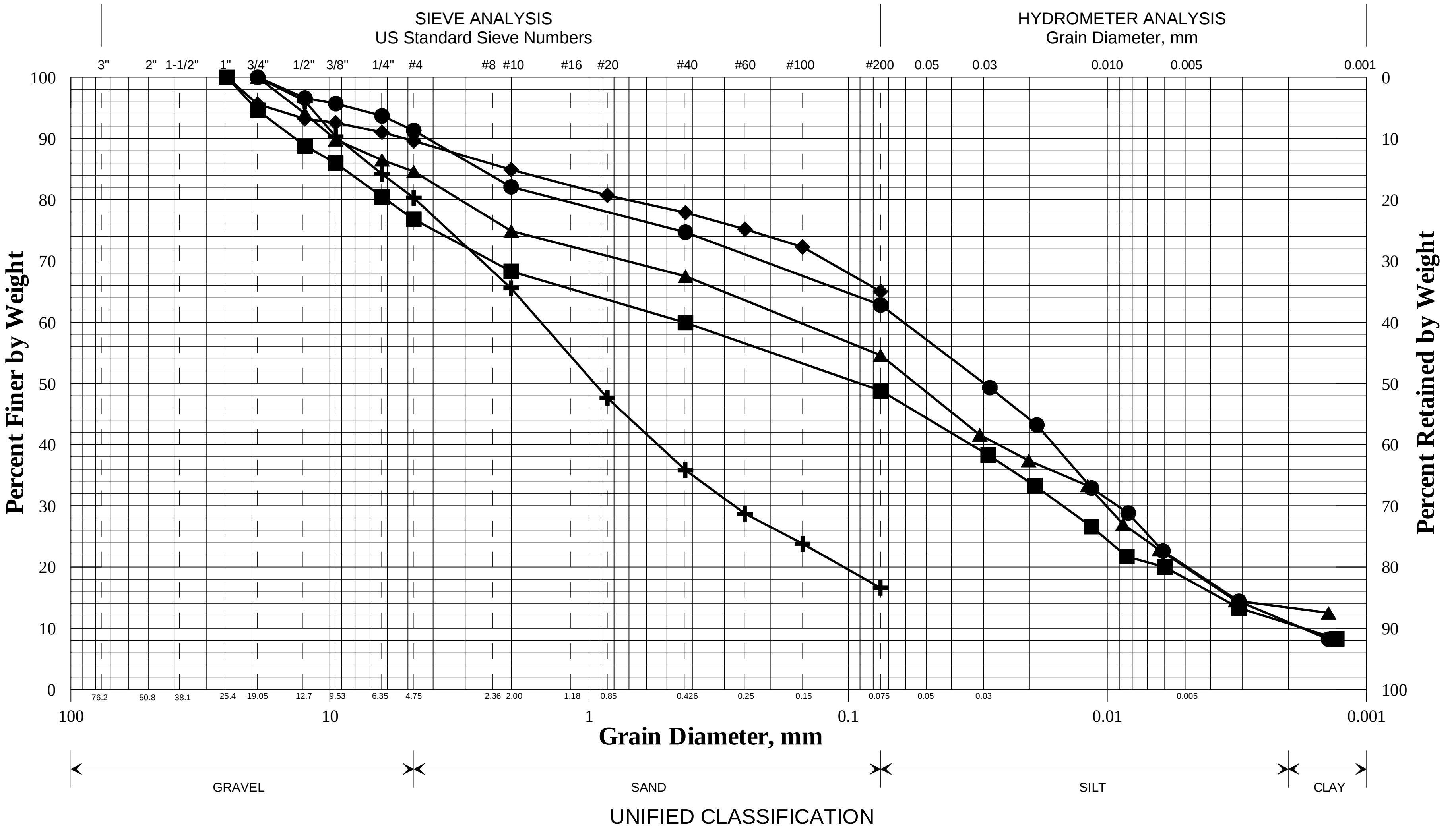
Town
Sapling Twp
Reported by/Date
WHITE, TERRY A 4/28/2017

SHEET 3

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SCPR-104/1D	9+58.6	9.4 LT	1.0-1.5	SAND, little gravel, little silt.	3.3			
◆	BB-SCPR-104/3D	9+58.6	9.4 LT	10.0-12.0	SILT, some sand, trace gravel.	11.1			
■	BB-SCPR-104/5D	9+58.6	9.4 LT	20.0-22.0	SILT, some sand, some gravel, little clay.	10.3			
●	BB-SCPR-104/7D	9+58.6	9.4 LT	30.5-32.5	SILT, some sand, little clay, trace gravel.	12.1			
▲	BB-SCPR-104/8D	9+58.6	9.4 LT	35.5-37.5	SILT, some sand, little gravel, little clay.	8.5			
✖									

WIN
021701.00
Town
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Reported by/Date
WHITE, TERRY A 4/28/2017

SHEET 4

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270154	BB-SCPR-101/1D	GEOTECHNICAL (DISTURBED)	3/10/2017	4/14/2017

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **7+73.4** Offset, ft: **6.9 RT** Dbfg, ft: **5.0-7.0**

WIN/Town **021701.00 - SAPLING TWP** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method: Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	94.1
½ in. [12.5 mm]	90.4
⅜ in. [9.5 mm]	88.0
¼ in. [6.3 mm]	83.8
No. 4 [4.75 mm]	80.8
No. 10 [2.00 mm]	74.3
No. 20 [0.850 mm]	69.0
No. 40 [0.425 mm]	65.5
No. 60 [0.250 mm]	62.4
No. 100 [0.150 mm]	59.2
No. 200 [0.075 mm]	52.8

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	9.3

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: **4/19/2017**

Paper Copy: Lab File; Project File; Geotech File

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270155	BB-SCPR-101/4D	GEOTECHNICAL (DISTURBED)	3/10/2017	4/14/2017

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **7+73.4** Offset, ft: **6.9 RT** Dbfg, ft: **20.0-22.0**

WIN/Town **021701.00 - SAPLING TWP** 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]	100.0
¾ in. [19.0 mm]	98.2
½ in. [12.5 mm]	97.1
⅜ in. [9.5 mm]	93.8
¼ in. [6.3 mm]	91.0
No. 4 [4.75 mm]	88.6
No. 10 [2.00 mm]	79.7
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	71.1
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	58.6
[0.0280 mm]	44.6
[0.0182 mm]	40.9
[0.0111 mm]	33.5
[0.0082 mm]	26.1
[0.0060 mm]	22.3
[0.0030 mm]	14.9
[0.0013 mm]	7.4

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	2.74
Loss on Ignition, % (T 267)	
Water Content (T 265), %	13.4

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: **4/24/2017**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270156	BB-SCPR-101/7D	GEOTECHNICAL (DISTURBED)	3/10/2017	4/14/2017

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **7+73.4** Offset, ft: **6.9 RT** Dbfg, ft: **35.5-37.0**

WIN/Town **021701.00 - SAPLING TWP** 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]	100.0
½ in. [12.5 mm]	93.6
⅜ in. [9.5 mm]	93.0
¼ in. [6.3 mm]	90.6
No. 4 [4.75 mm]	88.5
No. 10 [2.00 mm]	79.5
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	71.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	57.8
[0.0288 mm]	45.6
[0.0187 mm]	41.7
[0.0114 mm]	33.7
[0.0083 mm]	27.7
[0.0060 mm]	23.8
[0.0031 mm]	15.9
[0.0013 mm]	9.9

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	2.68
Loss on Ignition, % (T 267)	
Water Content (T 265), %	9.2

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.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

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Reported by: **GREGORY LIDSTONE** Date Reported: **4/24/2017**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
271124	**BB-SCPR-102/1D**	**GEOTECHNICAL (DISTURBED)**	**3/21/2017**	**4/14/2017**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **8+20.6** Offset, ft: **8.3 RT** Dbfg, ft: **2.0-4.0**

WIN/Town **021701.00 - SAPLING TWP** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**93.4**
¾ in. [19.0 mm]	**90.4**
½ in. [12.5 mm]	**88.5**
⅜ in. [9.5 mm]	**85.4**
¼ in. [6.3 mm]	**82.4**
No. 4 [4.75 mm]	**78.9**
No. 10 [2.00 mm]	**75.4**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**63.9**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**51.3**
[0.0296 mm]	**41.3**
[0.0197 mm]	**33.4**
[0.0118 mm]	**27.5**
[0.0085 mm]	**23.6**
[0.0062 mm]	**19.7**
[0.0031 mm]	**11.8**
[0.0013 mm]	**5.9**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	**2.68**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**12.8**

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:

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Reported by: **GREGORY LIDSTONE** Date Reported: **4/24/2017**

Paper Copy: Lab File; Project File; Geotech File

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
271125	BB-SCPR-102/2D	GEOTECHNICAL (DISTURBED)	3/21/2017	4/14/2017

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **8+20.6** Offset, ft: **8.3 RT** Dbfg, ft: **6.0-8.0**

WIN/Town **021701.00 - SAPLING TWP** 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]	100.0
½ in. [12.5 mm]	95.7
⅜ in. [9.5 mm]	92.4
¼ in. [6.3 mm]	88.6
No. 4 [4.75 mm]	87.0
No. 10 [2.00 mm]	79.5
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	72.4
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	63.8
[0.0265 mm]	53.0
[0.0176 mm]	47.4
[0.0108 mm]	39.8
[0.0080 mm]	32.2
[0.0059 mm]	26.6
[0.0031 mm]	15.2
[0.0013 mm]	7.6

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	2.71
Loss on Ignition, % (T 267)	
Water Content (T 265), %	14.5

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:

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Reported by: **GREGORY LIDSTONE** Date Reported: **4/26/2017**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270151	BB-SCPR-102/3D	GEOTECHNICAL (DISTURBED)	3/21/2017	4/14/2017

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **8+20.6** Offset, ft: **8.3 RT** Dbfg, ft: **11.0-13.0**

WIN/Town **021701.00 - SAPLING TWP** 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]	100.0
½ in. [12.5 mm]	98.5
⅜ in. [9.5 mm]	97.0
¼ in. [6.3 mm]	94.1
No. 4 [4.75 mm]	92.3
No. 10 [2.00 mm]	85.7
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	71.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	47.3
[0.0332 mm]	31.4
[0.0217 mm]	24.6
[0.0129 mm]	20.1
[0.0092 mm]	17.9
[0.0066 mm]	13.5
[0.0033 mm]	8.9
[0.0014 mm]	4.5

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	2.61
Loss on Ignition, % (T 267)	
Water Content (T 265), %	11.9

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.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

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Reported by: **GREGORY LIDSTONE** Date Reported: **4/24/2017**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270152	**BB-SCPR-103/2D**	**GEOTECHNICAL (DISTURBED)**	**3/20/2017**	**4/14/2017**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **9+08.1** Offset, ft: **10.0 LT** Dbfg, ft: **5.0-7.0**

WIN/Town **021701.00 - SAPLING TWP** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**93.4**
¾ in. [19.0 mm]	**87.2**
½ in. [12.5 mm]	**82.2**
⅜ in. [9.5 mm]	**78.5**
¼ in. [6.3 mm]	**74.1**
No. 4 [4.75 mm]	**70.7**
No. 10 [2.00 mm]	**59.2**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**46.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**34.8**
[0.0318 mm]	**26.9**
[0.0206 mm]	**23.9**
[0.0123 mm]	**19.4**
[0.0089 mm]	**16.5**
[0.0064 mm]	**13.4**
[0.0032 mm]	**9.0**
[0.0014 mm]	**6.0**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	**2.61**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**10.4**

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: **4/24/2017**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270153	BB-SCPR-103/3D	GEOTECHNICAL (DISTURBED)	3/20/2017	4/14/2017

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **9+08.1** Offset, ft: **10.0 LT** Dbfg, ft: **10.0-12.0**

WIN/Town **021701.00 - SAPLING TWP** 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]	100.0
½ in. [12.5 mm]	97.4
⅜ in. [9.5 mm]	96.1
¼ in. [6.3 mm]	95.2
No. 4 [4.75 mm]	93.2
No. 10 [2.00 mm]	83.5
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	74.8
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	61.8
[0.0288 mm]	47.5
[0.0190 mm]	41.3
[0.0114 mm]	35.2
[0.0083 mm]	28.9
[0.0060 mm]	24.8
[0.0031 mm]	16.5
[0.0013 mm]	8.3

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	2.71
Loss on Ignition, % (T 267)	
Water Content (T 265), %	9.9

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: **4/24/2017**

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MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
271119	**BB-SCPR-104/1D**	**GEOTECHNICAL (DISTURBED)**	**3/9/2017**	**4/14/2017**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **9+58.6** Offset, ft: **9.4 LT** Dbfg, ft: **1.0-1.5**

WIN/Town **021701.00 - SAPLING TWP** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**96.1**
⅜ in. [9.5 mm]	**90.3**
¼ in. [6.3 mm]	**84.2**
No. 4 [4.75 mm]	**80.3**
No. 10 [2.00 mm]	**65.5**
No. 20 [0.850 mm]	**47.6**
No. 40 [0.425 mm]	**35.8**
No. 60 [0.250 mm]	**28.7**
No. 100 [0.150 mm]	**23.8**
No. 200 [0.075 mm]	**16.6**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**3.3**

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: **4/19/2017**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
271120	**BB-SCPR-104/3D**	**GEOTECHNICAL (DISTURBED)**	**3/9/2017**	**4/14/2017**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **9+58.6** Offset, ft: **9.4 LT** Dbfg, ft: **10.0-12.0**

WIN/Town **021701.00 - SAPLING TWP** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**95.6**
½ in. [12.5 mm]	**93.2**
⅜ in. [9.5 mm]	**92.6**
¼ in. [6.3 mm]	**91.0**
No. 4 [4.75 mm]	**89.6**
No. 10 [2.00 mm]	**84.9**
No. 20 [0.850 mm]	**80.7**
No. 40 [0.425 mm]	**77.9**
No. 60 [0.250 mm]	**75.2**
No. 100 [0.150 mm]	**72.3**
No. 200 [0.075 mm]	**65.0**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**11.1**

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.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **4/19/2017**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
271121	**BB-SCPR-104/5D**	**GEOTECHNICAL (DISTURBED)**	**3/9/2017**	**4/14/2017**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **9+58.6** Offset, ft: **9.4 LT** Dbfg, ft: **20.0-22.0**

WIN/Town **021701.00 - SAPLING TWP** 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]	**100.0**
¾ in. [19.0 mm]	**94.6**
½ in. [12.5 mm]	**88.8**
⅜ in. [9.5 mm]	**86.0**
¼ in. [6.3 mm]	**80.5**
No. 4 [4.75 mm]	**76.8**
No. 10 [2.00 mm]	**68.3**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**59.9**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**48.8**
[0.0288 mm]	**38.3**
[0.0190 mm]	**33.3**
[0.0115 mm]	**26.6**
[0.0084 mm]	**21.7**
[0.0060 mm]	**20.0**
[0.0031 mm]	**13.3**
[0.0013 mm]	**8.3**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	**2.69**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**10.3**

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: **4/26/2017**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
271122	**BB-SCPR-104/7D**	**GEOTECHNICAL (DISTURBED)**	**3/9/217**	**4/14/2017**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **9+58.6** Offset, ft: **9.4 LT** Dbfg, ft: **30.5-32.5**

WIN/Town **021701.00 - SAPLING TWP** 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]	**100.0**
½ in. [12.5 mm]	**96.6**
⅜ in. [9.5 mm]	**95.7**
¼ in. [6.3 mm]	**93.7**
No. 4 [4.75 mm]	**91.3**
No. 10 [2.00 mm]	**82.1**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**74.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**62.8**
[0.0284 mm]	**49.3**
[0.0187 mm]	**43.2**
[0.0115 mm]	**32.9**
[0.0083 mm]	**28.8**
[0.0061 mm]	**22.6**
[0.0031 mm]	**14.4**
[0.0014 mm]	**8.2**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	**2.72**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**12.1**

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: **4/26/2017**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
271123	**BB-SCPR-104/8D**	**GEOTECHNICAL (DISTURBED)**	**3/9/2017**	**4/14/2017**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **9+58.6** Offset, ft: **9.4 LT** Dbfg, ft: **35.5-37.5**

WIN/Town **021701.00 - SAPLING TWP** 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]	**100.0**
½ in. [12.5 mm]	**94.1**
⅜ in. [9.5 mm]	**89.8**
¼ in. [6.3 mm]	**86.5**
No. 4 [4.75 mm]	**84.6**
No. 10 [2.00 mm]	**74.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**67.5**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**54.6**
[0.0310 mm]	**41.6**
[0.0201 mm]	**37.4**
[0.0119 mm]	**33.3**
[0.0087 mm]	**27.0**
[0.0063 mm]	**22.8**
[0.0032 mm]	**14.5**
[0.0014 mm]	**12.5**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	**2.62**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**8.5**

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: **4/24/2017**

Paper Copy: Lab File; Project File; Geotech File

Appendix D – Geotechnical Engineering Calculations

Soil Properties and Global Stability Evaluation

SPT Blow Count Conversions and Design Parameter Development
Maine DOT - Sapling Township Bridge
GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0025956.00
CALCULATED BY E. Friede 10/18/17
CHECKED BY A. Blaisdell, 10/19/17

Test Boring Elevations:

Boring Designation	Approx. Ground Surface El.	Groundwater Depth (ft bgs)
BB-SCPR-101	1078.6	37.0
BB-SCPR-102	1054.9	13.0
BB-SCPR-103	1054.8	11.0
BB-SCPR-104	1079.6	41.0

Stratum	γt (pcf)	Typical Soil Type/Description
Silty Fill	125	F-c Sandy SILT
Sandy Fill	120	Silty med SAND
Sandy SILT	130	Fine Sandy SILT

Conversion of Blow Counts:

Boring (BB-SCPR-XXX)	Depth to center (ft)	Sample ID	Elev.	Blows				Field N (bpf)	Total Unit Wt (pcf)	Effective overburden pressure (psf)	Cn	CE	CR	CB	Cs	Corrected N60	Corrected N160	Stratum
101	6	1D	1072.6	9	10	9	8	19	125	750	1.63	1.00	0.75	1	1	14	23	Silty FILL
101	11	2D	1067.6	5	5	5	5	10	125	1375	1.21	1.00	0.75	1	1	8	9	Silty Fill
101	16	3D	1062.6	2	4	5	6	9	125	2000	1.00	1.00	0.85	1	1	8	8	Silty FILL
101	21	4D	1057.6	2	2	3	4	5	125	2625	0.87	1.00	0.95	1	1	5	4	Silty FILL
101	26.5	5D	1052.1	3	5	3	8	8	125	3313	0.78	1.00	0.95	1	1	8	6	Silty FILL
101	31	6D	1047.6	2	2	11	5	13	125	3875	0.72	1.00	0.95	1	1	12	9	Silty FILL
101	36	7D	1042.6	5	11	21	37	32	130	4525	0.66	1.00	1.00	1	1	32	21	Sandy SILT
102	3	1D	1051.9	8	11	7	9	18	125	375	1.70	1.00	0.75	1	1	14	23	Silty FILL
102	7	2D	1047.9	8	4	3	4	7	125	875	1.51	1.00	0.75	1	1	5	8	Silty FILL
102	12	3D	1042.9	12	13	25	30	38	130	1525	1.15	1.00	0.75	1	1	29	33	Sandy SILT
103	0.25	1D	1054.6	40	-	-	-	R	120	30	1.70	1.00	0.75	1	1	---	---	Sandy FILL
103	6	2D	1048.8	17	46	17	17	63	120	720	1.67	1.00	0.75	1	1	47	79	Sandy FILL
103	11	3D	1043.8	5	18	28	28	46	130	1370	1.21	1.00	0.75	1	1	35	42	Sandy SILT
104	1.25	1D	1078.4	50	-	-	-	R	120	150	1.70	1.00	0.75	1	1	---	---	Sandy FILL
104	6	2D	1073.6	12	14	11	12	25	120	720	1.67	1.00	0.75	1	1	19	31	Sandy FILL
104	11	3D	1068.6	4	5	7	9	12	125	1345	1.22	1.00	0.75	1	1	9	11	Silty FILL
104	16	4D	1063.6	3	4	5	8	9	125	1970	1.01	1.00	0.85	1	1	8	8	Silty FILL
104	21	5D	1058.6	9	6	6	6	12	125	2595	0.88	1.00	0.95	1	1	11	10	Silty FILL
104	26	6D	1053.6	6	6	6	9	12	125	3220	0.79	1.00	0.95	1	1	11	9	Silty FILL
104	31.5	7D	1048.1	7	7	9	14	16	125	3908	0.72	1.00	0.95	1	1	15	11	Silty FILL
104	36.5	8D	1043.1	35	41	33	48	74	130	4558	0.66	1.00	1.00	1	1	74	49	Sandy SILT
104	40.1	9D	1039.5	50/2				R	130	5026	0.63	1.00	1.00	1	1	---	---	Sandy SILT

Statistics and Friction Angles:

	Avg N60	N60 St. Dev	Min N60 (Design Basis)
Silty Fill	10	3	---
Sandy SILT	42	2	29

	Avg N160	N160 St. Dev	Design Basis N160 (Avg-0.5 St. Dev)	Friction Angle (Deg)
Silty Fill	11	2	10	30
Sandy SILT	36	8	32	35

Notes:
1) Phi estimated based on the correlation of N160 and friction angle presented in AASHTO LRFD Table 10.4.6.2.4-1.
2) Friction angle was calculated using N160 Avg-0.5*Standard Deviation.
3) Design basis N60 used for settlement evaluation in D'Appolonia 1970 method (correlation to compressibility modulus, M). Consists of minimum value of recorded N60 values for conservatism.

SPT Blow Count Conversions and Design Parameter Development	**GZA FILE NO.**	09.0025956.00
Maine DOT - Sapling Township Bridge	**CALCULATED BY**	E. Friede 10/18/17
GZA GeoEnvironmental, Inc.	**CHECKED BY**	A. Blaisdell, 10/19/17

Drained Friction Angle of Granular Soils
Reference: AASHTO LRFD 6th Ed, Table 10.4.6.2.4-1 (see insert)

Note: Fine, Medium and Coarse values selected as lower, middle, and upper portions of ranges consistent with intent of Bowles 77.

N160	ϕ_f (Fine)	ϕ_f (Med)	ϕ_f (Coarse)
2	25	27.5	30
4	27	29.5	32
10	30	32.5	35
30	35	37.5	40
50	40	41.5	43

$$N1_{60} = C_N N_{60} \qquad (10.4.6.2.4\text{-}3)$$

The drained friction angle of granular deposits should be determined based on the following correlation.

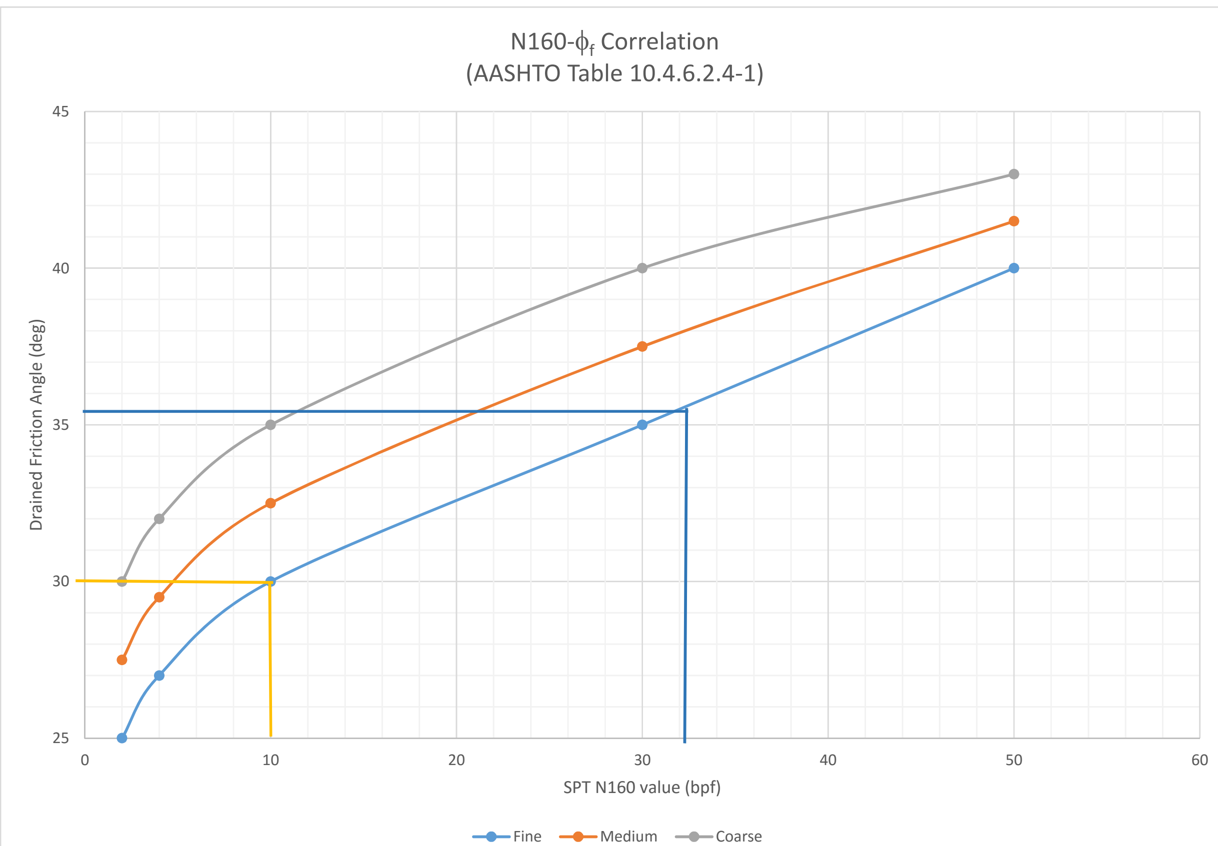
Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

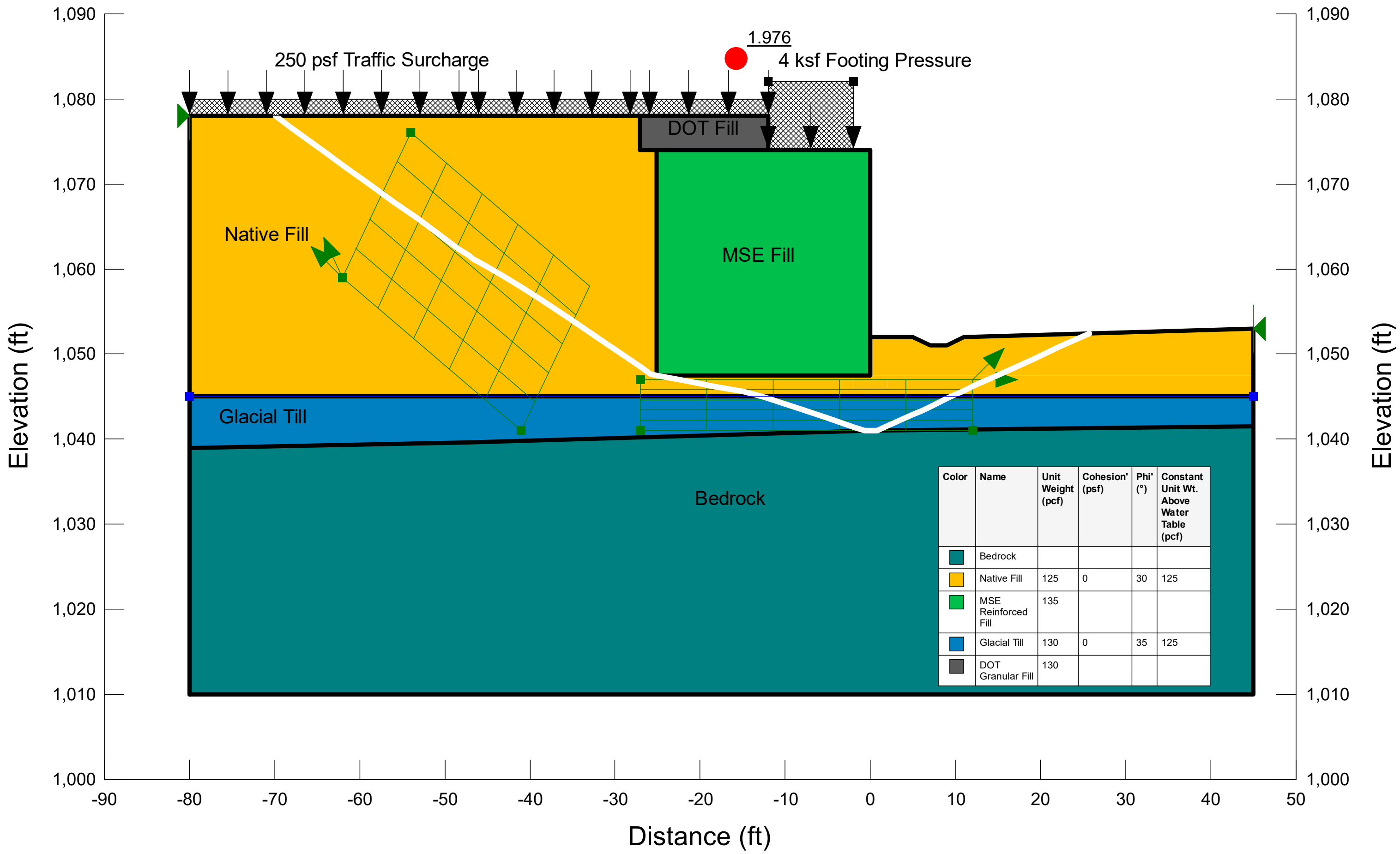
$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in Youd and Idriss (1997).

The $N1_{60}$-ϕ_f correlation used is modified after Bowles (1977). The correlation of Peck, Hanson, and Thornburn (1974) falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant silt-sized material will fall in the lower portion of the range. Coarser materials with less than five percent fines will fall in the upper portion of the ranges. The geologic history and angularity of the particles may also need to be considered when selecting a value for ϕ_f.

Care should be exercised when using other correlations of *SPT* results to soil parameters. Some published correlations are based on corrected values ($N1_{60}$) and some are based on uncorrected values (N).



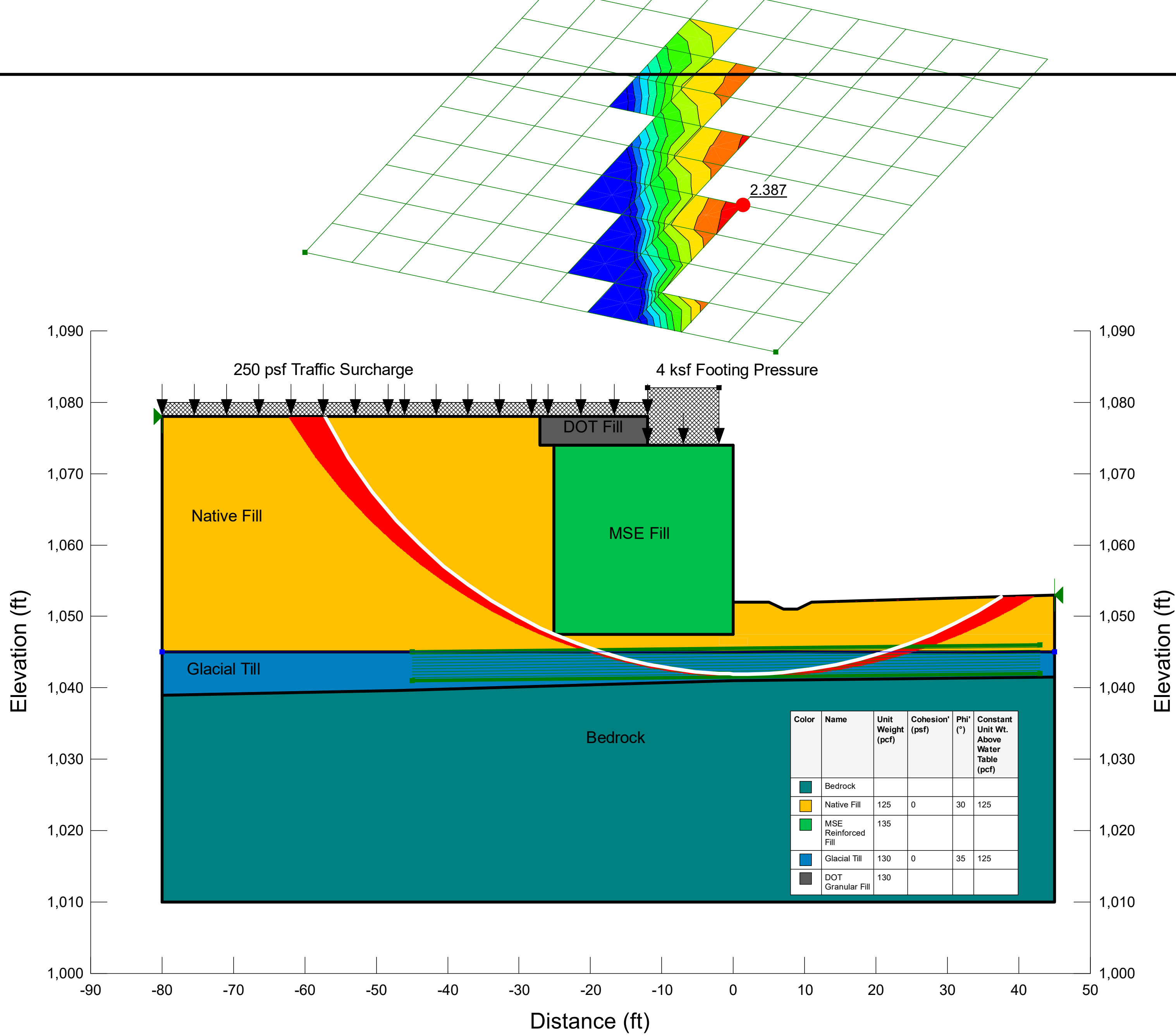


Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Constant Unit Wt. Above Water Table (pcf)
	Bedrock				
	Native Fill	125	0	30	125
	MSE Reinforced Fill	135			
	Glacial Till	130	0	35	125
	DOT Granular Fill	130			

MSE Stability - Block

GS_Sapling Twnsp.gsz

11/7/2017 1:250



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Constant Unit Wt. Above Water Table (pcf)
	Bedrock				
	Native Fill	125	0	30	125
	MSE Reinforced Fill	135			
	Glacial Till	130	0	35	125
	DOT Granular Fill	130			

MSE Stability - Grid

GS_Sapling Twnsp.gsz

11/7/2017 1:250

Mechanically Stabilized Earth Wall Bearing Resistance and Settlement

Objective

Calculate soil bearing resistance for foundation (MSE wall) bearing on soils with a friction angle greater than 0 using the Theoretical method (Munfakh et al., 2001) in sand using SPT data. Evaluate strength and service limit bearing resistance for a range of effective footing widths based on anticipated MSE geometry.

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 7th edition, 2014, with 2015 and 2016 interims (AASHTO LRFD), Articles 10.5.5.2.2, 10.6.3.1. 11.5.7, 11.10.2.1, and 11.10.4.
2. MaineDOT Bridge Design Guide (BDG)
3. D'Appolonia et. al, 1970 Method, taken from Engineering Manual for Shallow Foundations, NCHRP Project 24-4, pg 97-103

Soil Properties and Geotechnical Inputs

$\phi_f := 35\text{deg}$ Friction angle of supporting soil, Based on $N_{1,60}$ Bowles 4th Edition

$\phi_{b1} := 0.65$ MSE bearing resistance factor as specified in Table 11.5.7-1 (Theoretical Method, SPT Data, Strength Limit)

$c := 0\text{ksf}$ Cohesion, taken as undrained shear strength (Assume foundation soil is cohesionless)

$\gamma := 125\text{pcf}$ Unit weight of soil above or below the bearing depth of the footing

$N_c := 46.1$ Cohesion term bearing capacity factor as specified in Table 10.6.3.1.2a-1

$N_q := 33.3$ Surcharge term bearing capacity factor as specified in Table 10.6.3.1.2a-1

$N_\gamma := 48.0$ Total unit weight term bearing capacity factor as specified in Table 10.6.3.1.2a-1

$C_{wq}, C_{w\gamma}:=$ Correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2

Depth to water table at depth of footing (D_f) = 0 $C_{wq} := 0.5$ $C_{w\gamma} := 0.5$

$d_q:=$ Correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4

$s_c, s_\gamma, s_q:=$ Footing shape correction factors as specified in Table 10.6.3.1.2a-3

Service limit bearing resistance for allowable settlement

$N_{60} := 29$ Minimum N_{60} value from SPT below bottom of MSE.

Load inclination factors are omitted considering modest embedment of MSE per C10.6.3.1.2a.

MSE Height and Width

For foundation evaluation, take the height as distance from proposed grade above wall to bottom of MSE Fill.

$D_f := 4.5ft$ Footing embedment depth below grade on the exterior of the MSE wall.

$EL_{rd1} := 1081ft$ Proposed top of roadway elevation at Abutment 1

$EL_{top} := 1072.5ft$ Proposed top elevation of MSE Wall based on top of wall or footing bearing elevation, whichever is greater.

$EL_{ex1} := 1052ft$ Existing grade at the toe of Abutment 1 MSE wall.

$EL_{rock1} := 1041ft$ Top of Rock Elevation below Abutment 1 MSE wall.

$H_{t1} := EL_{ex1} - D_f - EL_{rock1} = 7 \cdot ft$ Thickness of soil between bottom of MSE and top of rock at Abutment 1.

Considering minimal variation between H_{t1} and H_{t2}, say $H_t := H_{t1} = 7 \cdot ft$

$H1_{MSE1} := EL_{top} - EL_{ex1} + D_f = 25 \cdot ft$ Total height of Abutment 1 MSE wall.

$D_{MSE1} := EL_{rd1} - EL_{top} = 9 \cdot ft$ Depth from road elevation to top of the MSE Wall for Abutment 1

$H_{MSE} := H1_{MSE1} + D_{MSE1} = 34 \cdot ft$ Total effective height of the MSE wall including height of footing and fill above the top of the MSE

Foundation Dimensions

Assume MSE Width, B, is between 70% and 100% of H_{MSE}.

$$B := \begin{pmatrix} 0.7 \cdot H_{MSE} \\ 0.8 \cdot H_{MSE} \\ 0.9 \cdot H_{MSE} \\ 1.0 \cdot H_{MSE} \end{pmatrix} = \begin{pmatrix} 23 \\ 27 \\ 30 \\ 34 \end{pmatrix} \cdot ft$$

Range of footing widths considered corresponding to anticipated MSE strap length plus facing thickness. Due to flexibility of MSE wall systems, the load is considered uniformly distributed at the bearing depth per C11.10.5.4.

$$L := 50ft$$

Length of full height MSE wall. Scaled from sheet 3 of 7 of the Preliminary Plan Set, WIN 021701.00 Bridge Plans

Strength Limit Design

$q_r = q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{\gamma m} C_{w\gamma}$ Nominal Bearing Resistance Formula

$\phi_b \ q_n$ Factored Bearing Resistance Formula

Correction Factors

$$d_{qtable} := \frac{D_f}{B} \qquad d_{qtable} = \begin{pmatrix} 0.19 \\ 0.17 \\ 0.15 \\ 0.13 \end{pmatrix}$$

Using Table 10.6.3.1.2a-4

$$d_q := 1$$

dq assumed soil above footing less competent than soil below footing.

$$s_c := 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) \qquad s_c = \begin{pmatrix} 1.34 \\ 1.39 \\ 1.44 \\ 1.48 \end{pmatrix}$$

$$s_q := 1 + \left(\frac{B}{L} \tan(\phi_f)\right) \qquad s_q = \begin{pmatrix} 1.33 \\ 1.38 \\ 1.42 \\ 1.47 \end{pmatrix}$$

$$s_\gamma := 1 - 0.4\left(\frac{B}{L}\right) \qquad s_\gamma = \begin{pmatrix} 0.81 \\ 0.79 \\ 0.76 \\ 0.73 \end{pmatrix}$$

Bearing Capacity Factors

$$N_{cm} := N_c \cdot s_c \qquad N_{cm} = \begin{pmatrix} 61.7 \\ 63.9 \\ 66.2 \\ 68.4 \end{pmatrix} \qquad N_{qm} := N_q \cdot s_q \cdot d_q \qquad N_{qm} = \begin{pmatrix} 44.2 \\ 45.8 \\ 47.4 \\ 48.9 \end{pmatrix} \qquad N_{\gamma m} := N_\gamma \cdot s_\gamma \qquad N_{\gamma m} = \begin{pmatrix} 39 \\ 37.7 \\ 36.4 \\ 35.1 \end{pmatrix}$$

Nominal Bearing Resistance

$$q_n := \overrightarrow{\left(c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}\right)} \qquad q_n = \begin{pmatrix} 41 \\ 44 \\ 48 \\ 51 \end{pmatrix} \cdot ksf$$

MSE Factored Bearing Resistance - Strength Limit State

$$q_{R1} := \phi_{b1} \cdot q_n \qquad q_{R1} = \begin{pmatrix} 26.7 \\ 28.9 \\ 31 \\ 32.9 \end{pmatrix} \cdot ksf \qquad \text{for} \quad B = \begin{pmatrix} 23 \\ 27 \\ 30 \\ 34 \end{pmatrix} \cdot ft$$

Service Limit Design

Evaluate settlement at estimated sustained service bearing pressure using the semi-empirical SPT method by D'Appolonia et al. (1970). Method selected to model a limited thickness of soil below the loaded area. Equations and figures referenced are from the Engineering Manual for Shallow Foundations, NCHRP Project 24-2 published by the Virginia Tech Department of Civil Engineering, May 1991.

To calculate the settlement of a footing on sand (ρ): $\rho := \mu_o \cdot \mu_1 \cdot \frac{(p \cdot B)}{M}$ equation 5.2.2

Settlement Influence Factors

μ_o, μ_1 Settlement influence factors taken from Figure 5.3

μ_1 a function of H_t/B and H_t/L:

$$\frac{H_t}{B} = \begin{pmatrix} 0.28 \\ 0.24 \\ 0.22 \\ 0.19 \end{pmatrix}$$

From Figure 5.3: $\mu_{1min} := 0.21$ and $\mu_{1max} := 0.30$ Considering small range say $\mu_1 := \mu_{1max} = 0.3$

μ_o a function of D_f/B and L/B:

$$\frac{D_f}{B} = \begin{pmatrix} 0.19 \\ 0.17 \\ 0.15 \\ 0.13 \end{pmatrix} \quad \text{and} \quad \frac{L}{B} = \begin{pmatrix} 2.1 \\ 1.9 \\ 1.7 \\ 1.5 \end{pmatrix}$$

From Figure 5.3: $\mu_{omin} := 0.96$ and $\mu_{omax} := 0.98$ Considering small range say $\mu_o := \mu_{omax} = 0.98$

Modulus of Compressibility

M - Modulus of Compressibility (tsf on Figure 5.4) from Figure 5.4

Minimum N_{60} Value: $N_{60} = 29$ bpf

$M := 1440\text{ksf}$ Sand is pre loaded (Glacial Till).

Applied Pressure

p - Applied pressure from MSE Wall Backfill and Footing Surcharge, $p_{fill} + p_{footing}$

$\gamma_{fill} := 135pcf$ Based on dense silty sand and gravel, BDG Table 3-3

$H_{MSE} = 34 \cdot ft$

$p_{fill} := 1.5\gamma_{fill} \cdot H_{MSE} = 6.8 \cdot ksf$ Conservative, allows for higher bearing pressure near the toe.

$p_{footing} := 4ksf$ Typical service bearing resistance for bridge footing on MSE backfill, assumes no load distribution through straps. The bearing pressure includes the weight of soil above the footing.

$p := p_{fill} + p_{footing} = 10.8 \cdot ksf$

Estimated Settlement

Total settlement, MSE and Footing:

$$\rho := \mu_o \cdot \mu_1 \cdot \frac{(p \cdot B)}{M} = \begin{pmatrix} 0.62 \\ 0.71 \\ 0.8 \\ 0.89 \end{pmatrix} \cdot in \quad \text{for} \quad B = \begin{pmatrix} 23 \\ 27 \\ 30 \\ 34 \end{pmatrix} \cdot ft$$

Settlement from MSE only, prior to footing construction:

$$\rho_{fill} := \mu_o \cdot \mu_1 \cdot \frac{\left(p_{fill} \cdot B\right)}{M} = \begin{pmatrix} 0.39 \\ 0.45 \\ 0.5 \\ 0.56 \end{pmatrix} \cdot in \quad \text{for} \quad B = \begin{pmatrix} 23 \\ 27 \\ 30 \\ 34 \end{pmatrix} \cdot ft$$

Maximum estimated settlement is approximately 1 inch. Roughly half of the total settlement will occur as a result of MSE construction, and is anticipated to occur primarily as the MSE wall is built.

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

Friction Angle, ϕ_f (degrees)	D_f/B	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

Linear interpolations may be made for friction angles in between those values shown in Table 10.6.3.1.2a-4.

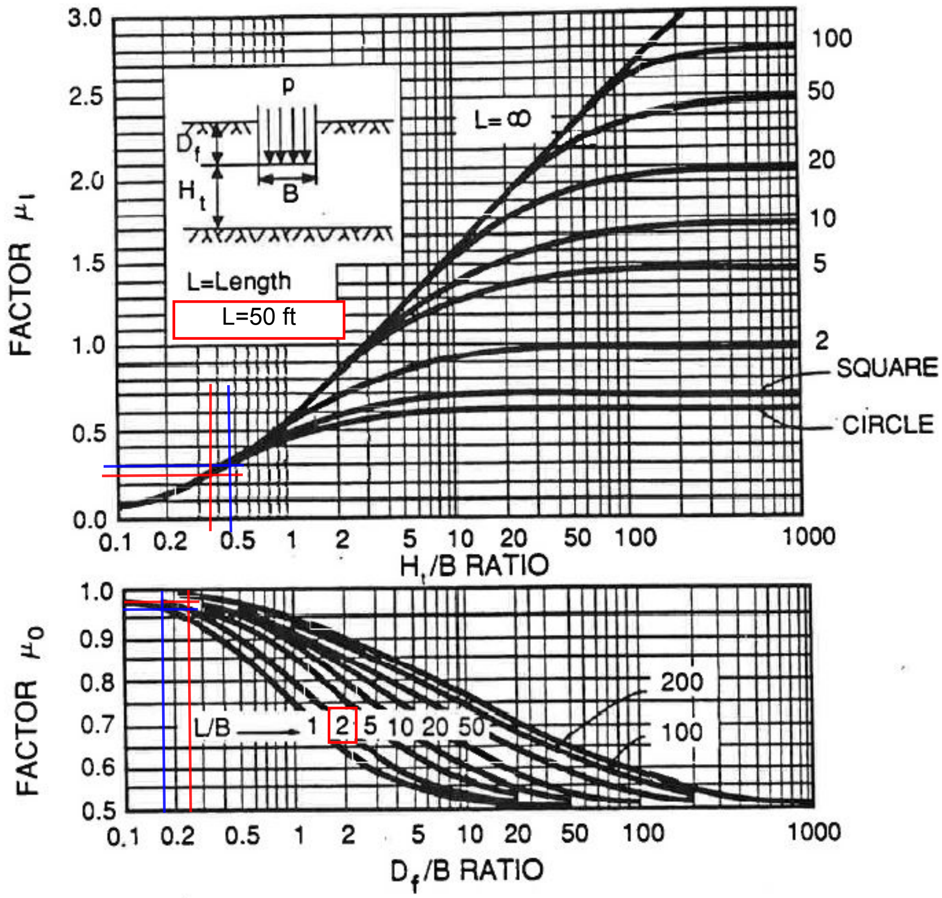


Figure 5.3 Settlement Influence Factors μ_0 and μ_1 for D'Appolonia et al. Procedure (After D'Appolonia, et al., 1970)

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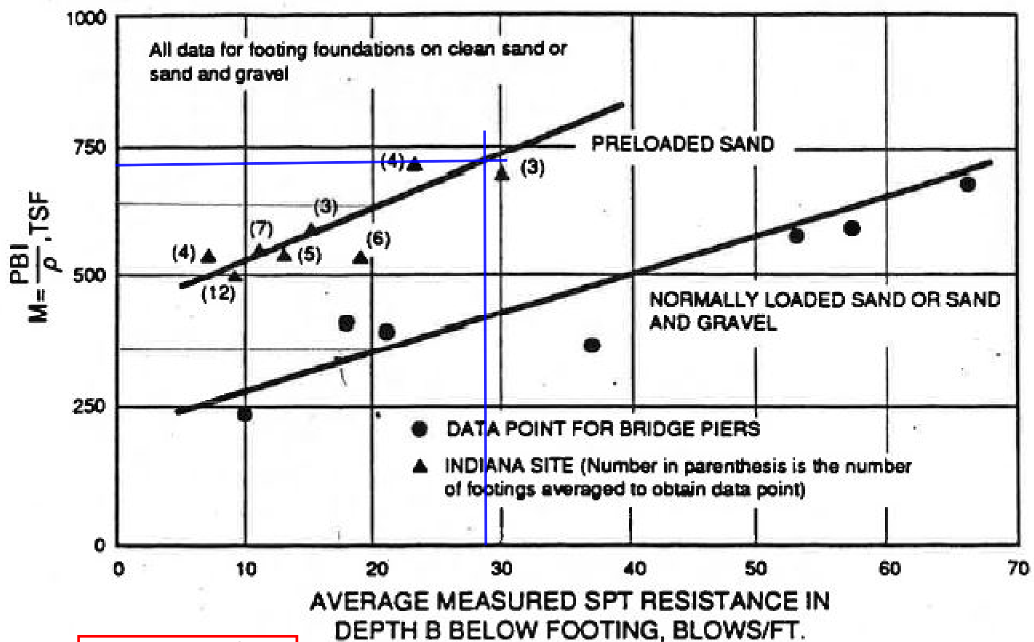
Engineers and Scientists



M= 720 tsf;
1440 ksf

Figure 5.4 Correlation Between Modulus of Compressibility and Average Value of SPT Blow Count (After D'Appolonia, et al., 1970)

101

Frost Penetration

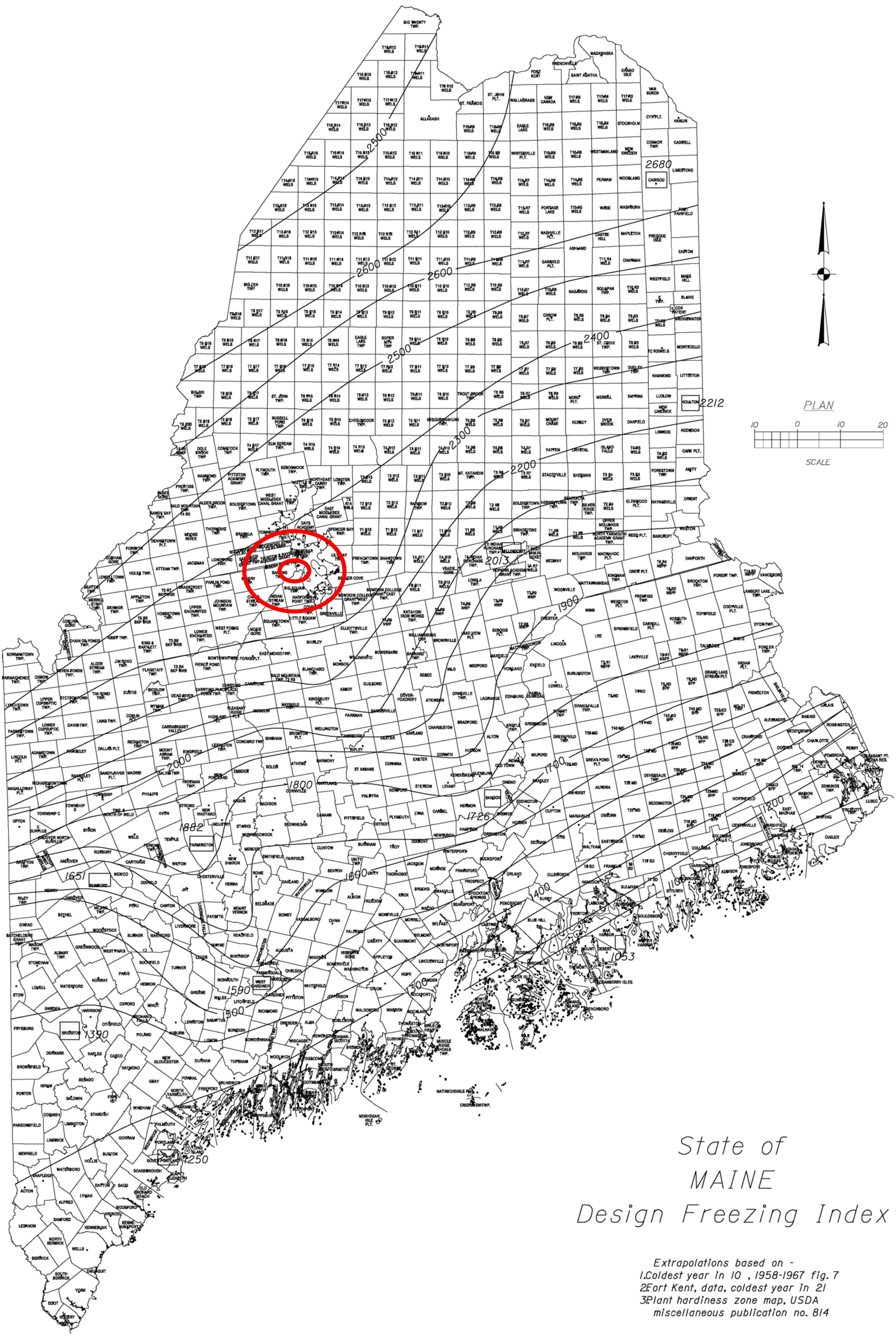


Figure 5-1 Maine Design Freezing Index Map

Sapling Township Bridge - Maine
Lab Summary for Freezing Index
Project No: 09.0025956.00
Date 11/3/17

Boring	Sample	Depth	Water %	Soil Type
SCPR-102		2-4	12.8%	Silt
SCPR-102		6-8	14.5%	Silt
SCPR-103		5-7	10.4%	Silty Sand
SCPR-103		10-12	9.9%	Silt
		MSE Avg	**11.9%**	

Note: Per BDG Table 5.1, Frost Pentration=74.3" for fine grained soils with a water content of 10%

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

Site lies on approximate 2350 line

Seismic Evaluation

Design Maps Detailed Report USGS

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (45.58329°N, 69.7213°W)

Site Class D – "Stiff Soil"

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B. Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 [1]	PGA = 0.070 g
From Figure 3.4.1-3 [2]	S_S = 0.160 g
From Figure 3.4.1-4 [3]	S_1 = 0.051 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1–1 Site Class Definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_S > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_S \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_S \leq 2{,}500$	$\bar{N} > 50$	>2,000 psf
D	Stiff soil profile	$600 \leq \bar{v}_S < 1{,}200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_S < 600$	$\bar{N} < 15$	<1,000 psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index *PI* > 20, 2. Moisture content *w* ≥ 40%, and 3. Undrained shear strength $\bar{s}_u$ < 500 psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (*H* > 10 feet of peat and/or highly organic clay where *H* = thickness of soil) 3. Very high plasticity clays (*H* > 25 feet with plasticity index *PI* > 75) 4. Very thick soft/medium stiff clays (*H* > 120 feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.070 g, F_{PGA} = 1.600

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	S_S ≤ 0.25	S_S = 0.50	S_S = 0.75	S_S = 1.00	S_S ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.160 g, F_a = 1.600

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_1

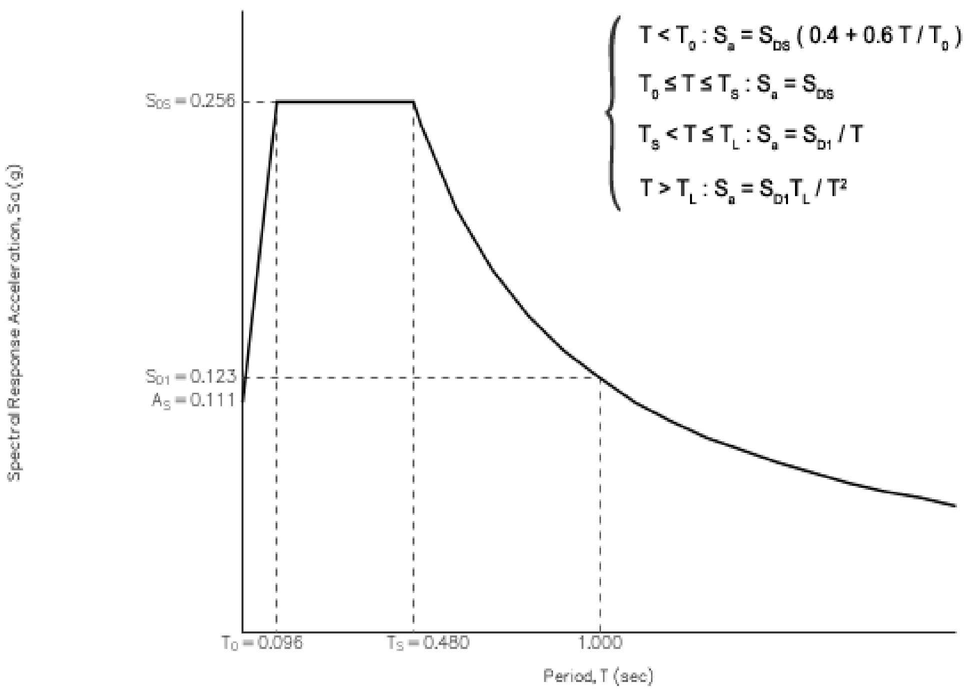
For Site Class = D and S_1 = 0.051 g, F_v = 2.400

Equation (3.4.1-1): $A_S = F_{PGA}$ PGA = 1.600 x 0.070 = 0.111 g

Equation (3.4.1-2): $S_{DS} = F_a S_S$ = 1.600 x 0.160 = 0.256 g

Equation (3.4.1-3): $S_{D1} = F_v S_1$ = 2.400 x 0.051 = 0.123 g

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \le S_{D1} < 0.30g$	B
$0.30g \le S_{D1} < 0.50g$	C
$0.50g \le S_{D1}$	D

For S_{D1} = 0.123 g, Seismic Design Category = A

Seismic Design Category ≡ "the design category in accordance with Table 3.5-1" = A

References

1. *Figure 3.4.1-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf
2. *Figure 3.4.1-3*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf
3. *Figure 3.4.1-4*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf

Lateral Earth Pressure

Subject:

Evaluate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 7th Edition (2014, with 2015 and 2016 Interims)

Input Parameters:

$\beta := 0\text{deg}$ Angle of backfill to the horizontal

$\theta := 72\text{deg}$ Angle of backface of wall to the horizontal

$\phi := 32\text{deg}$ Effective angle of internal friction *(Granular borrow, Soil Type 4, BDG Table 3-3)*

$\delta_f := 24\text{deg}$ Interface friction angle, concrete to soil *(Granular borrow, Soil Type 4, BDG Table 3-3)*

Earth Pressure Coefficients:

Coloumb Active Earth Pressure Coefficient (Short-Heeled Wall)

$$\Gamma := \left[1 + \sqrt{\left[\frac{\sin(\phi + \delta_f)\cdot(\sin(\phi - \beta))}{\sin(\theta - \delta_f)\cdot\sin(\theta + \beta)}\right]}\right]^2 = 3.2$$

$$K_{ac} := \frac{(\sin(\theta + \phi))^2}{\Gamma\cdot\left[(\sin(\theta))^2\cdot\sin(\theta - \delta_f)\right]}$$

$K_{ac} = 0.44$

At-rest Earth Pressure Coefficient

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

$K_o = 0.47$