GEOTECHNICAL DESIGN REPORT

Maxwell Bridge Culvert Replacement
BRIDGE NO. 2524
MAINE DOT WIN 028246.00
LITCHFIELD, MAINE

February 2025
09.0026259.00

Prepared for:
Maine Department of Transportation
Augusta, Maine

Prepared by:
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VIA EMAIL

February 4, 2025
File No. 09.0026259.00

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

Re: Geotechnical Design Report
Maxwell Bridge No. 2524 Culvert Replacement
Richmond Road over Maxwell Brook
Maine Department of Transportation WIN 028246.00
Litchfield, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the replacement of Maxwell Bridge Culvert, which carries Richmond Road over Maxwell Brook in Litchfield, Maine. Our work was completed in accordance with GZA GeoEnvironmental, Inc.'s (GZA's) August 19, 2020 Multi-PIN contract number 20200603000000000709 with the Maine Department of Transportation (MaineDOT) Bridge Program and Assignment Letter No. 20 dated September 27, 2024 for WIN 028246.00, and the *Limitations* contained in **Appendix A** of this report.

It has been a pleasure serving MaineDOT on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Erin Tome, P.E.
Assistant Project Manager

STATE OF MAINE
Andrew
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Blaisdell
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LICENSED
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02/04/2025

Christopher L. Snow, P.E.
Consultant Reviewer

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Associate Principal

ET/ARB/CLS:cc
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Attachment: Geotechnical Design Report

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1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the replacement of Maxwell Bridge No. 2524 Culvert in Litchfield, Maine. Our work was completed in accordance with GZA GeoEnvironmental, Inc.'s (GZA's) August 19, 2020, Multi-PIN contract number 20200603000000000709 with the Maine Department of Transportation (MaineDOT) Bridge Program and Assignment Letter No. 20 dated September 27, 2024 for WIN 028246.00, and the *Limitations* contained in **Appendix A** of this report.

1.1 BACKGROUND

The project includes the replacement of Maxwell Bridge No. 2524 carrying Richmond Road over Maxwell Brook in Litchfield, Maine, the location of which is shown in **Figure 1**. The existing bridge was constructed in 1991 and consists of a single 7-foot by 11-foot corrugated steel arch culvert. Recent inspections have shown that the condition of the substructure is poor and is considered structurally deficient. There is a 12-foot-long area of section loss in the west wall, which is causing fill to spill through. A recently encountered, 5-foot-diameter sinkhole in the westbound lane was a result of the failed culvert. A temporary repair was enacted that included filling the hole and patching the asphalt.

We understand plans are to construct a new 75-foot-long box culvert with a span of 18 feet, a rise of 6 feet, and a 7-degree skew. We anticipate the culvert will have precast headwalls and 2-foot-deep toe walls at the inlet and outlet. The prepared subgrade is anticipated to consist of a 1-foot-thick layer of Underdrain backfill material placed on stabilization/reinforcement geotextile overlying the natural subgrade. The project is planned to maintain the current road alignment, as shown on **Figure 2.**

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Reviewed the results of two test borings and two probe borings and results of laboratory testing completed by MaineDOT;
- Conducted final design phase geotechnical engineering analyses for:
  - soil and bedrock properties;
  - stability and settlement of approach embankments;
  - frost susceptibility and drainage of approach subgrade materials;
  - AASHTO LRFD load and resistance factors associated with geotechnical design elements;
  - spread footing design considerations, including bearing resistance, sliding resistance, and settlement; and
  - seismic design considerations;

- Developed geotechnical engineering recommendations including bearing on soil, culvert backfill type and properties, and earth pressures; geotechnical construction considerations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

Two test borings and two probe borings were drilled and logged by MaineDOT on September 4, 2025. Boring BB-LMB-101 and probe BP-LMB-104 were drilled on the east side of the existing culvert and BB-LMB-102 and BP-LMB-103 were drilled on the west side. The borings were drilled using a CME-45C drill rig to depths ranging from approximately 11 to 27 feet below ground surface (bgs). Borings were terminated in the bedrock after coring, and probes were terminated at practical auger refusal.

The borings were drilled using 3-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at typical 5-foot intervals. Sampling was completed using a 24-inch-long, 1-3/8-inch inside-diameter sampler. The sampler was driven with a 140-lb calibrated automatic hammer with a 30-inch drop from a truck-mounted drill rig. The boring logs indicate a hammer efficiency factor at the time of drilling of 0.962. Approximately 10 feet of bedrock core was obtained in both borings using NQ2 coring equipment. The auger probes were drilled using a 5-inch solid stem auger to refusal; samples were not collected. At the completion of drilling, the borings and probes were backfilled with cuttings and sand and capped with asphalt cold patch. The as-drilled locations and elevations were surveyed by MaineDOT.

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

3.0 LABORATORY TESTING

Soil testing was performed by MaineDOT Testing Laboratories in Bangor, Maine. The testing program included:

- Four (4) gradation analysis / MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments; and
- Four (4) moisture content tests.

Results of the testing are included in **Appendix C**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping[1], the surficial unit in the vicinity of the culvert consists of ablation till, which consists of sand, silt, and gravel deposited by glacial ice and described as not very compact.

Bedrock mapping[2] in the vicinity of the site shows the bedrock at the culvert site mapped as the Hutchins Corner Formation, and is described as medium grey, fine- to medium-grained, quartz-plagioclase-biotite granofels and schist.

4.2 SUBSURFACE PROFILE

Two soil units were encountered in the test borings below surficial asphalt and above bedrock: Fill and Glacial Till. Approximately five to seven inches of asphalt pavement was encountered in the test borings. The thicknesses and generalized descriptions of the soil units are presented in the following table in descending order from the ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. An interpretive subsurface profile based on the test boring results is presented as **Figure 2, Boring Location Plan & Interpretive Subsurface Profile**.

GENERALIZED SUBSURFACE CONDITIONS		
Subsurface Unit	**Approximate Encountered Thickness (ft)**	**Generalized Description**
Fill	7.9 to 14.1	Brown, medium dense to very dense, fine to coarse SAND, little to some gravel, trace silt (USCS: SW-SM). MaineDOT Frost Classification = 0 *Encountered in both bridge borings.*
Glacial Till	1.7 to 2.2	Grey, medium dense to very dense, fine to coarse SAND, some gravel, some silt to Silty. (USCS: SM). MaineDOT Frost Classification = II-III *Encountered in both bridge borings.*
Top of Bedrock Elevation	Approximately El. 157.0 to 162.9 (11.6 to 17.4 feet depth)**	
**Note: Reported top of rock elevations above were taken from auger probes. Top of rock was judged based on a refusal surface in the probes and should be considered approximate.		

4.2.1 Bedrock

Bedrock was cored in both test borings. Bedrock was described as hard, fresh to slightly weathered, fine to medium grained, grey, SCHIST with granofels inclusions. Joints were described as very close to

[1]Locke, Daniel B. and Hildreth, Carol T., 2004, Surficial materials of the Purgatory quadrangle, Maine: Maine Geological Survey, Open-File Map 04-43, map, scale 1:24,000.

[2] West, David P., and Ellenberger, Evan D., 2010, Bedrock geology of the Purgatory quadrangle, Maine: Maine Geological Survey, Open-File Map 10-21, color map, scale 1:24,000.

moderately spaced, low angle to moderately dipping, planar, rough, fresh to discolored, tight to open, with sand or silt infilling. The core sample taken from BB-LMB-101 had fractured zones from 14.0 feet to 14.5 feet and 15.8 feet to 16.3 feet. The Rock Quality Designation in the core runs ranged from 40 to 80 percent, corresponding to Rock Quality of poor to good rock.

4.2.2 Groundwater

Groundwater was measured in BB-LMB-101 at a depth of 10.5 feet below ground surface, corresponding to approximately El. 164. Groundwater was not measured in the other explorations. Water levels may have been affected by drilling procedures, which included introduction of water for drilling purposes.

Fluctuations in groundwater levels will occur due to variations in season, precipitation, stream levels and construction activity in the area. Consequently, water levels during and after construction are likely to vary from those encountered at the time of the borings.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in accordance with *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix D**.

5.2 APPROACH EMBANKMENTS

The roadway will remain on the current horizontal alignment and vertical profile. Minor grading of the side slopes is anticipated to achieve the final slope angles of 2 horizontal to 1 vertical (2H:1V) or flatter.

Due to the limited extent of modification to the embankments and the subsurface conditions, embankment global stability and settlement are not considered to be concerns for the project.

5.3 FOUNDATION TYPE

The culvert is proposed to consist of a 4-sided precast concrete box culvert with a span of 18 feet and a rise of 6 feet, bearing on a 1-foot minimum thickness of Underdrain Backfill Material, Type C (MaineDOT Pay Item 203.55 Culvert Bedding Stone), placed on a glacial till and or bedrock subgrade.

5.4 LOAD AND RESISTANCE FACTORS

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

The recommended LRFD resistance factors for strength limit state design of foundations were derived from LRFD Tables 10.5.5.2.2-1, 10.5.5.2.3-1 and 10.5.5.2.4-1 and are presented in the following table.

GEOTECHNICAL RESISTANCE FACTORS – STRENGTH LIMIT STATE			
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ)**	**AASHTO Reference**
Bearing	Theoretical Method in Sand using SPT	0.45	10.5.5.2.2-1
Sliding	Precast Concrete Placed on Sand	0.90	10.5.5.2.2-1

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

5.5 CULVERT BASE DESIGN CONSIDERATIONS

The bottom of the culvert and inlet and outlet walls will be underlain by 12 inches of Type C (MaineDOT Pay Item 203.55 Culvert Bedding Stone). At these depths, the exposed materials are anticipated to include medium dense to very dense glacial till and bedrock. The following sections discuss settlement and bearing related to the proposed culvert foundations.

5.5.1 Strength Bearing Resistance

Bearing resistance values for the strength limit state were developed for equivalent footings bearing on Underdrain Backfill Material using the theoretical method (Munfakh et al., 2001) using an internal friction angle typical of compacted granular fill and glacial till. Bearing resistances were evaluated in accordance with Articles 10.6.3.1.1 and 10.6.3.1.2a of AASHTO LRFD.

5.5.2 Service Bearing Resistance

Bearing resistance values for the service limit state were evaluated for the specified allowable settlement of approximately ½ to ¾ inch using the semi-empirical SPT Method of Burland and Burbidge (1985) provided in Terzaghi, Peck & Mesri, 1996.

The calculated bearing resistance values for the culvert in the strength and service limit states are presented in **Appendix E** and summarized in the table below.

BEARING RESISTANCE VALUES FOR CULVERT BASE ON SOIL				
Footing	**Effective Footing Width (feet)**	**Nominal Bearing Resistance (ksf)**	**Factored Bearing Resistance, Strength Limit State (ksf)**	**Service Bearing Resistance (ksf)**
Precast Culvert	16 to 18	24.5	11	3.1

5.5.3 Settlement

The box culvert will bear on less than 5 feet of medium dense to very dense glacial till or gravel fill over bedrock. Since these are drained granular soils, settlement is anticipated to occur elastically as the structure and backfill are placed. We estimate the post-construction foundation settlement will be ½ inch or less.

5.6 SEISMIC DESIGN CONSIDERATIONS

Per AASHTO LRFD Article 3.10.1, seismic analysis is not required for buried structures except where they cross active faults. Therefore, seismic design parameters are not required.

5.7 LATERAL EARTH PRESSURE

The precast culvert sides will be restrained at the top and bottom from lateral movement. Therefore, the box culvert walls should be designed for at-rest earth pressure conditions. Culvert inlet and outlet headwalls are a few feet high or less. These short walls should be designed for at-rest earth pressure conditions since they are not free to rotate. Inlet and Outlet Walls that extend beyond the box culvert and are independent from the top of the box culvert are considered free to rotate and should be designed for Rankine active earth pressure with a 2H:1V backslope (currently proposed). The material properties will be controlled by the backfill material, which is anticipated to consist of BDG Type 4 soil. Soil properties for Type 4 soil are provided in **Section 6.2** of this report.

5.8 FROST PROTECTION

Fill soils are anticipated to be present above and adjacent to the culvert and embankments, either as existing fill, or imported backfill. The bearing material below the culvert is anticipated to be Underdrain Backfill Material, Type C, glacial till or bedrock. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,490, and with coarse-grained materials and low moisture content (Approx. 15 percent), the estimated depth of frost penetration is approximately 6.3 feet below surfaces exposed to freezing temperatures. The BDG does not specify frost embedment requirements for culverts.

Since the fill between the roadway and the culvert will be exposed to freezing from above and below, we recommend non-frost-susceptible fill, such as granular borrow for underwater backfill be used to backfill above the culvert.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes should be designed with MaineDOT typical slope angles of 2H:1V or flatter with a loam and seed surface finish. Where a riprap surface treatment is used, a 1.75H:1V slope angle is acceptable. Riprap may also be provided as scour protection where the embankment side slopes will be near or below typical water levels in Maxwell Brook. The extent and nature of scour countermeasures will be evaluated by others.

6.2 BOX CULVERT AND INLET AND OUTLET WALL DESIGN

Backfill between the culvert and inlet and outlet should consist of MaineDOT 703.19 Granular Borrow, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:

- Internal Friction Angle of Soil = 32°
- Soil Total Unit Weight = 125 pcf

- At-rest Earth Pressure, K_o = 0.47 (use for design of box culvert walls and inlet and outlet headwalls)
- Rankine Active Earth Pressure, K_a = 0.46 (use for design of culvert inlet and outlet walls unsupported from box and free to rotate, assumes slope of 2H:1V rising behind the wall)

Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the culvert/ inlet and outlet wall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.

6.3 RECOMMENDATIONS FOR FOUNDATIONS

The proposed box culvert should be supported on 12 inches of MaineDOT 703.22 Underdrain Backfill Material, Type C separated on bottom and sides by Stabilization/Reinforcement Geotextile installed over undisturbed glacial till or bedrock, except for the precast concrete toe walls, which should bear directly on naturally deposited glacial till. Prior to placement of the 1-foot-thick layer of Underdrain Backfill Material, any bedrock above the bearing elevation should be excavated using conventional excavation methods for placement of the full thickness of Underdrain Backfill Material, and fully encapsulated by Stabilization/Reinforcement Geotextile (MaineDOT Standard Specification 722.01). If necessary, bedrock should be excavated to a minimum of 1 foot below bearing elevation and replaced with encapsulated Underdrain Backfill Material beneath wingwall footings to avoid creation of a hard point beneath the footing. Culvert and footing bearing pressures should be checked to confirm that they are less than the resistance values presented in **Section 5.5** of this report.

In order to limit seepage beneath the culvert, the Underdrain backfill should not extend upstream or downstream beyond the limits of the key/cutoff walls on the base. The cutoff walls should bear directly on naturally deposited Glacial Till or compacted Underdrain Backfill Material.

The culvert subgrade surfaces should be cleaned of soil and rock that is loosened by the excavation process prior to placement of the Underdrain Backfill Material, and if the subgrade is dry, the surface can be proof-compacted. Bearing surface preparation should be in accordance with **Section 7.2**.

The Underdrain Backfill Material, Type C bedding for the culvert should be placed in maximum 6-inch lifts and densified with several passes of a walk-behind roller or large plate compactor.

The base resistance against sliding was evaluated in accordance with AASHTO Article 10.6.3.4 using ϕ_f' = 32 degrees and C = 0.8 for the culvert base (precast concrete). Nominal sliding resistance coefficient for culvert was calculated as C*tan ϕ_f' and is equal to 0.50. The factored sliding resistance coefficient for the strength condition is 0.45 for the culvert and inlet/outlet walls, based on a resistance factor ($\phi\tau$) of 0.9 for the strength limit state.

Passive resistance on the toe of footings should be neglected when evaluating sliding and overturning.

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during excavation, dewatering, and foundation subgrade preparation and protection. These items are discussed in the paragraphs that follow.

7.1 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavations for culvert foundations are anticipated to extend approximately 12 to 13 feet below existing pavement grades and up to 5 feet below the Q 1.1 (El. 167.0). An estimated 1 to 2 feet of bedrock excavation may be necessary to reach bearing elevation. Blasting is not recommended; conventional excavation methods such as a hydraulic excavator equipped with a hoe ram should be used. A bid item should be included in the plans for structural excavation of rock.

Sloped open cut excavation should be suitable for this project depending on the effectiveness of dewatering. Damming and diversion and/or temporary dewatering are anticipated to be necessary to control groundwater and/or stream inflow in excavations. Depending on permitting and water levels at the time of construction, we anticipate that it would be possible to dam the stream with sand bags and/or an impermeable membrane and temporarily divert the flow through a pipe so the contractor can construct foundations in the dry. It may also be necessary to employ localized pumping from sumps to maintain dewatering. Cantilever sheetpiles may be difficult due to the shallow bedrock and inability to achieve toe embedment. It is anticipated that inflow of surface water or runoff 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. Dewatering discharge should be managed in accordance with the contractor’s Stormwater Prevention Plan and MaineDOT Best Management Practices.

7.2 SUBGRADE PREPARATION

Even with damming and diversion, excavation bases may be wet. If the exposed surface of the glacial till is saturated, the stabilization/reinforcement geotextile should be placed directly on the subgrade and then the first lift of Underdrain Backfill, Type C placed. The surface of the Type C material may then be densified as previously described. In the event that the subgrade exhibits weaving or rutting, compaction should be continued without vibration.

TABLE

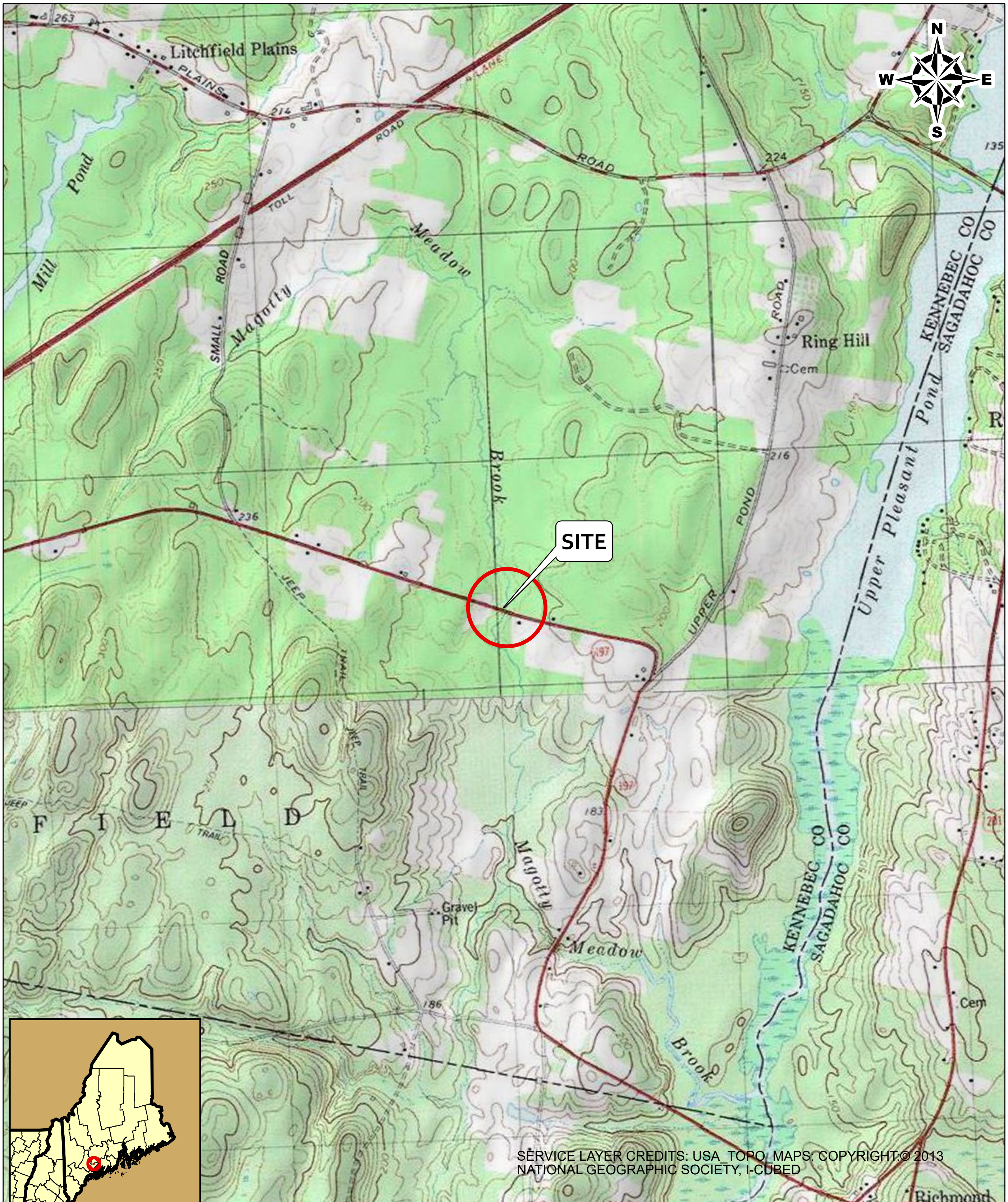
TABLE 1
Summary of Subsurface Explorations
Maxwell Bridge #2524 carries Richmond Road over Maxwell Brook
Litchfield, Maine
GZA job#: 09.0026259.00

Boring ID	Northing	Easting	Station	Offset (ft)		Ground Surface El. (ft)	Top of Stratum Elevation			Stratum Thickness			Depth to Bedrock (ft)	Top of Rock Elevation (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
							Asphalt	Fill	Glacial Till	Asphalt	Fill	Glacial Till					El. (ft)	Depth (ft)
BB-LMB-101	472442.5	1105949.5	12+70.5	8.6	R	174.5	174.5	173.9	163.0	0.6	10.9	1.2	12.7	161.8	22.7	151.8	164.0	10.5
BB-LMB-102	472464.0	1105943.4	12+57.7	9.8	L	174.5	174.5	174.1	160.0	0.4	14.1	1.7	16.2	158.3	26.2	148.3	NM	NM
BP-LMB-103	472447.3	1105934.2	12+54.4	9.0	R	174.4	--	--	--	--	--	--	17.4*	157.0*	17.4	157.0	NM	NM
BP-LMB-104	472456.5	1105960.7	12+76.4	8.3	L	174.5	--	--	--	--	--	--	11.6*	162.9*	11.6	162.9	NM	NM

El. = Elevation, NE = Not Encountered, NM = Not Measured, NP = Not Penetrated, > = Boring Terminated in Stratum

Notes:
1. Refer to the boring logs in Appendix B for additional information.
2. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
3. As-drilled locations were surveyed by MaineDOT.
4. Stratum depths, thickness and elevations are rounded to the nearest 0.1 foot as interpreted on the boring logs, but this does not represent the precision of the data.
5. "*" indicates top of rock was estimated using an auger probe and should be considered approximate to the degree implied by the method used.

FIGURES



0 1,000 2,000 4,000

SCALE IN FEET

MAXWELL BRIDGE #2524 OVER MAXWELL BROOK

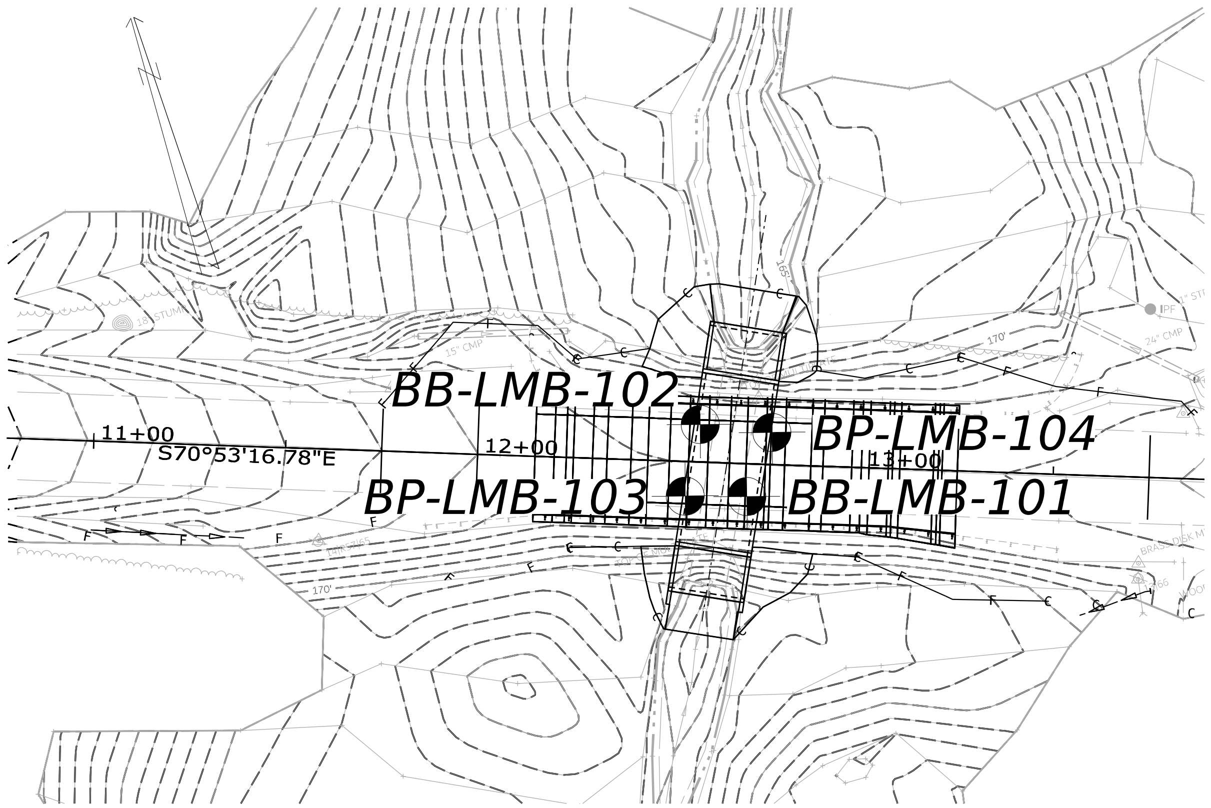
LITCHFIELD, ME

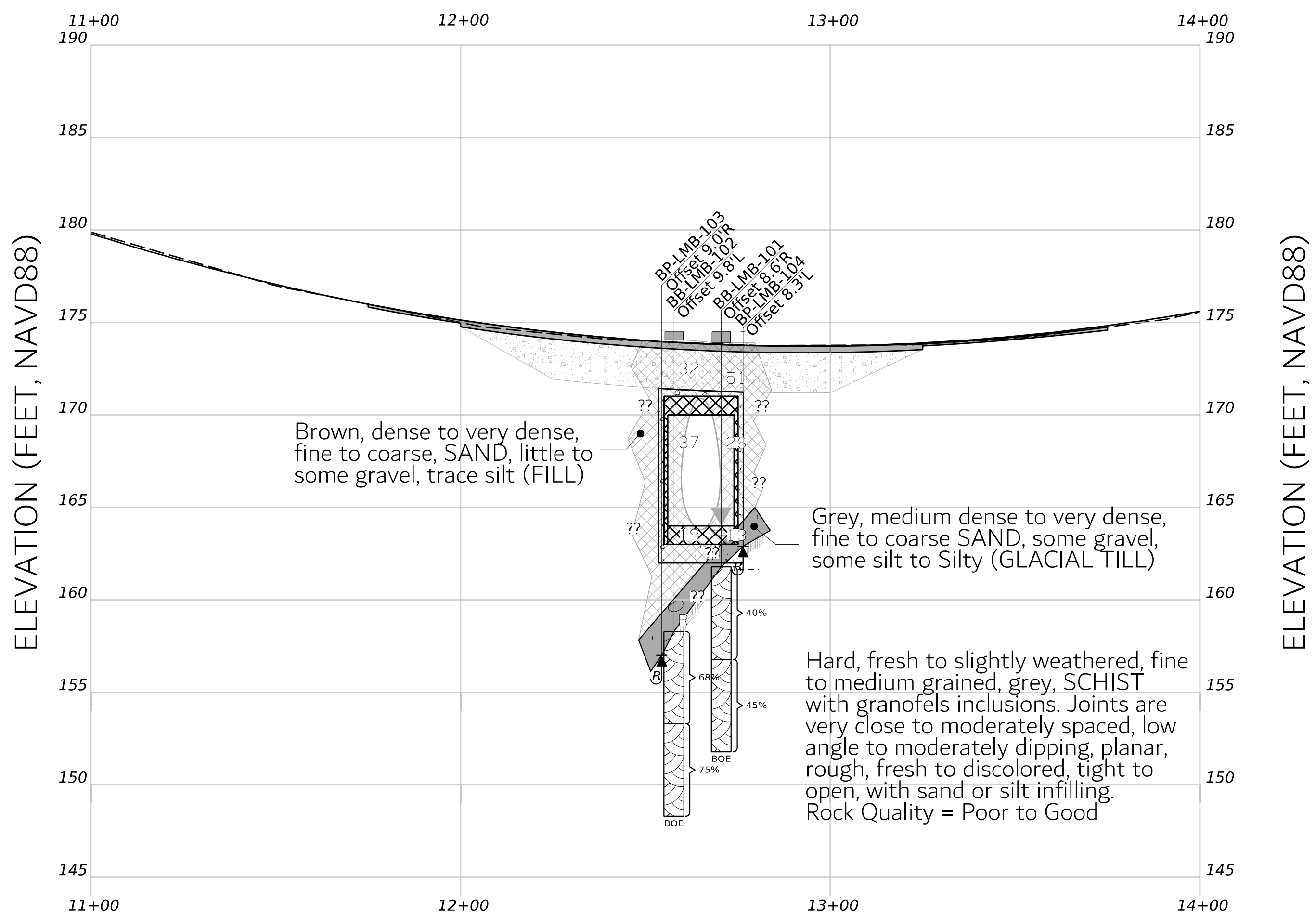
PREPARED BY: GZA GeoEnvironmental, Inc. www.gza.com

PREPARED FOR: MEDOT

LOCUS PLAN

PROJ MGR: ENT	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE
DESIGNED BY: EAF	DRAWN BY: EAF	SCALE: 1 in = 2,000 ft	1
DATE: JANUARY 2025	PROJECT NO: 09.0026259.00	REVISION NO:	





NOTES

1) Base map developed from electronic files (WIN 28246.00 Workset) provided by Stantec on January 15, 2025.

2) Profile developed from electronic files (WIN 28246.00 Workset) provided by Stantec on January 15, 2025.

3) The as-drilled locations of the test borings were surveyed and provided by MaineDOT in an electronic file (2D BORINGS 6 SEPT 24 028246.00) on January 17, 2025.

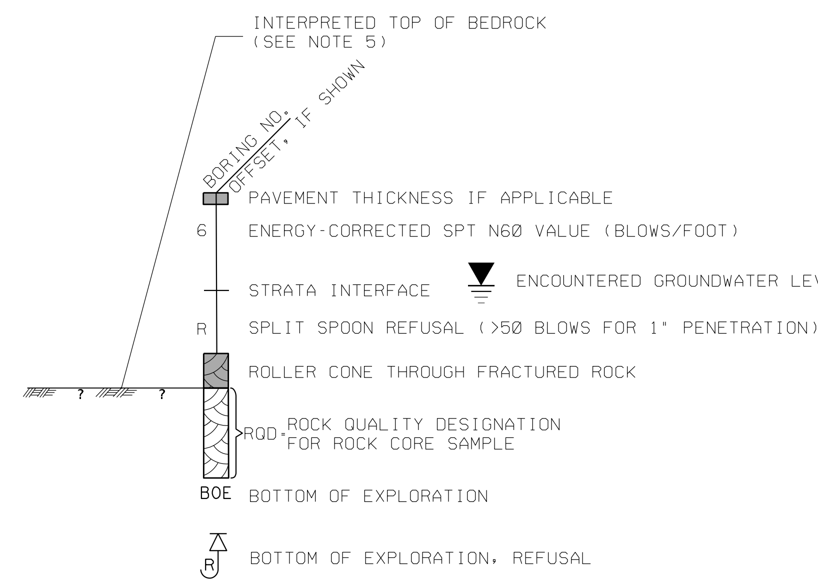
4) BB-LMB-100 series borings were performed by MaineDOT on September 4, 2024.

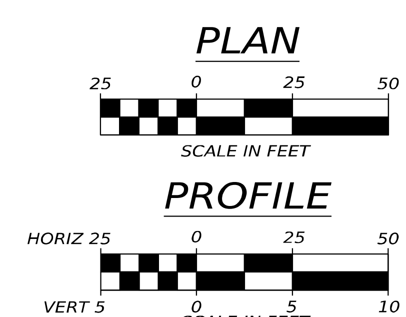
5) This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more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

BORING LOCATION PLAN LEGEND

BP-LMB-104 — Locations and designations of BB-LMB-100 series borings performed by MaineDOT on September 4, 2024.

INTERPRETIVE SUBSURFACE PROFILE LEGEND





STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2824600

WIN 28246.00

BRIDGE PLANS

BRIDGE NO. 2524

SIGNATURE

P.E. NUMBER

DATE

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	L. HAILEY	01/2025
CHECKED-REVIEWED	E. TOME	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

LITCHFIELD MAXWELL BRIDGE

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
0
OF

Username: Logan.Hailey Date: 2/5/2025

Filename: P:\09 Jobs\MEDOT_ORD\WIN\28246_00\GEOTECH\Boring_Location_Plan.dgn

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.

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APPENDIX B – TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Density of Cohesionless Soils	Standard Penetration Resistance N_{60}-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

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

Consistency of Cohesive soils	SPT N_{60}-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

Desired Rock Observations (in this order, if applicable):
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
-tightness (tight, open, or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Maxwell Bridge #2524 carries Richmond Road over Maxwell Brook
Location: Litchfield, Maine

Boring No.: BB-LMB-101
WIN: 28246.00

Driller:	MaineDOT	**Elevation (ft.)**	174.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/4/2024; 08:00-10:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+70.5, 8.6 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	10.5 ft bgs.
Hammer Efficiency Factor:	0.962	**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							SSA	173.9		7" HMA. 0.6	
	1D	24/17	1.50 - 3.50	10/16/16/25	32	51				Brown, damp, very dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
5	2D	24/18	5.00 - 7.00	10/7/9/17	16	26				Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	G#241497 A-1-b, SW-SM WC=1.7%
10	3D/A	24/15	10.00 - 12.00	4/4/4/12	8	13	20			3D (10.0-11.5 ft bgs) Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
							19	163.0		11.5 3D/A (11.5-12.0 ft bgs) Grey, wet, medium dense to dense, Silty SAND, some gravel, (Glacial Till).	G#241498 A-4, SM WC=13.0%
	R1	60/60	12.70 - 17.70	RQD = 40%			a60 NQ-2	161.8		a60 blows for 0.7 ft. 12.7	
										Top of Bedrock at Elev. 161.8 ft. R1: Bedrock: Hard, slightly weathered, fine to medium grained, gray, SCHIST with granofels inclusions. Joints are very close to moderately spaced, low angle to moderately dipping, planar, rough, discolored, open, with sand infilling. Completely fractured zones at 14 to 14.5 ft and 15.75 to 16.25 ft. Recovery: 100% Rock Quality: Poor R1: Core Times (min:sec) 12.7-13.7 ft (1:42) 13.7-14.7 ft (1:50) 14.7-15.7 ft (1:55) 15.7-16.7 ft (1:55) 16.7-17.7 ft (1:51)	
15											
	R2	60/60	17.70 - 22.70	RQD = 45%				156.8		17.7 R2: Bedrock: Hard, slightly weathered, fine to medium grained, gray, SCHIST with granofels inclusions. Joints are very close to moderately spaced, low angle with one vertical joint from 21ft to 22.1 ft, planar, rough, fresh to discolored, tight to open, with silt infilling. Recovery: 100% Rock Quality: Poor R2: Core Times (min:sec) 17.7-18.7 ft (2:02) 18.7-19.7 ft (2:28) 19.7-20.7 ft (1:37)	
20											
								151.8			
25											

Remarks:

1. As-drilled boring locations were surveyed by MaineDOT in the field (N472442.5, E1105949.5).

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: Maxwell Bridge #2524 carries Richmond Road over Maxwell Brook
Location: Litchfield, Maine

Boring No.: BB-LMB-101
WIN: 28246.00

Driller:	MaineDOT	**Elevation (ft.)**	174.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/4/2024; 08:00-10:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+70.5, 8.6 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	10.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										20.7-21.7 ft (2:01) 21.7-22.7 ft (2:04)	
										22.7	
										Bottom of Exploration at 22.7 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. As-drilled boring locations were surveyed by MaineDOT in the field (N472442.5, E1105949.5).

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: Maxwell Bridge #2524 carries Richmond Road over Maxwell Brook
Location: Litchfield, Maine

Boring No.: BB-LMB-102
WIN: 28246.00

Driller:	MaineDOT	**Elevation (ft.)**	174.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/4/2024; 13:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+57.7, 9.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.962	**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							SSA	174.1		5" HMA. 0.4	
	1D	24/20	1.00 - 3.00	8/10/10/8	20	32				Brown, damp, dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#241499 A-1-b, SW-SM WC=2.6%
5	2D	24/15	5.00 - 7.00	5/6/17/9	23	37				Similar to above, (Fill).	
10	3D	24/13	10.00 - 12.00	7/8/4/7	12	19	15			Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
							17				
							36				
							39				
							73	160.0		14.5	
15	4D	14.4/13	15.00 - 16.20	11/48/40(2.4")	R			158.3		Olive, wet, very dense, fine to medium SAND, some gravel, some silt, (Glacial Till). 16.2	G#241500 A-2-4, SM WC=10.9%
	R1	60/58	16.20 - 21.20	RQD = 75%			NQ-2			Top of Bedrock at Elev. 158.3 ft. R1: Bedrock: Hard, fresh, fine to medium grained, gray, SCHIST with granofels inclusions. Joints are close to moderately spaced, low angle, planar, rough, fresh, tight to open, with silt infilling. Recovery: 97% Rock Quality: Fair R1: Core Times (min:sec) 16.2-17.2 ft (1:29) 17.2-18.2 ft (2:29) 18.2-19.2 ft (1:32) 19.2-20.2 ft (1:24) 20.2-21.2 ft (1:50)	
20	R2	60/60	21.20 - 26.20	RQD = 80%				153.3		21.2 R2: Bedrock: Hard, fresh, fine to medium grained, gray, SCHIST with granofels inclusions. Joints are moderately spaced, low angle, planar, rough, fresh, tight to open, with silt infilling. Recovery: 100% Rock Quality: Good	
25											

Remarks:

1. As-drilled boring locations were surveyed by MaineDOT in the field (N472464.0, E1105943.4).
2. R = Refusal for N-uncorrected.

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: Maxwell Bridge #2524 carries Richmond Road over Maxwell Brook Location: Litchfield, Maine	Boring No.: BB-LMB-102 WIN: 28246.00

Driller:	MaineDOT	**Elevation (ft.)**	174.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/4/2024; 13:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+57.7, 9.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								148.3		R2: Core Times (min:sec) 21.2-22.2 ft (1:34) 22.2-23.2 ft (2:11) 23.2-24.2 ft (4:20) 24.2-25.2 ft (4:01) 25.2-26.2 ft (3:3.6)	
										26.2	
										Bottom of Exploration at 26.2 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. As-drilled boring locations were surveyed by MaineDOT in the field (N472464.0, E1105943.4).
2. R = Refusal for N-uncorrected.

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: Maxwell Bridge #2524 carries Richmond Road over Maxwell Brook
Location: Litchfield, Maine

Boring No.: BP-LMB-103

WIN: 28246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	174.4	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	9/4/2024; 10:30:-11:00	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	12+54.4, 9.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	Not Measured

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			Probe, no material samples taken.	
5										
10										
15										
							157.0		17.4	
									Bottom of Exploration at 17.4 feet below ground surface. Very solid REFUSAL.	
20										
25										

Remarks:

1. As-drilled boring locations were surveyed by MaineDOT in the field (N472447.3, E1105934.2).

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: Maxwell Bridge #2524 carries Richmond Road over Maxwell Brook

Location: Litchfield, Maine

Boring No.: BP-LMB-104

WIN: 28246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	174.5	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	9/4/2024; 11:15-11:30	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	12+76.4, 8.3 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	Not Measured

Definitions:
D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			Probe, no material samples taken.	
5										
10										
							162.9		11.6	
									Bottom of Exploration at 11.6 feet below ground surface. Very solid REFUSAL.	
15										
20										
25										

Remarks:

1. As-drilled boring locations were surveyed by MaineDOT in the field (N472456.5, E1105960.7).

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

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

APPENDIX C – LABORATORY TEST RESULTS

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Litchfield   Work Number: 28246.00

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-LMB-101, 2D	12+70.5	8.6 Rt.	5.0-7.0	241497	1	1.7			SW-SM	A-1-b	0
BB-LMB-101, 3D/A	12+70.5	8.6 Rt.	11.5-12.0	241498	1	13.0			SM	A-4	III
BB-LMB-102, 1D	12+57.7	9.8 Lt.	1.0-3.0	241499	1	2.6			SW-SM	A-1-b	0
BB-LMB-102, 4D	12.57.7	9.8 Lt.	15.0-16.2	241500	1	10.9			SM	A-2-4	II

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

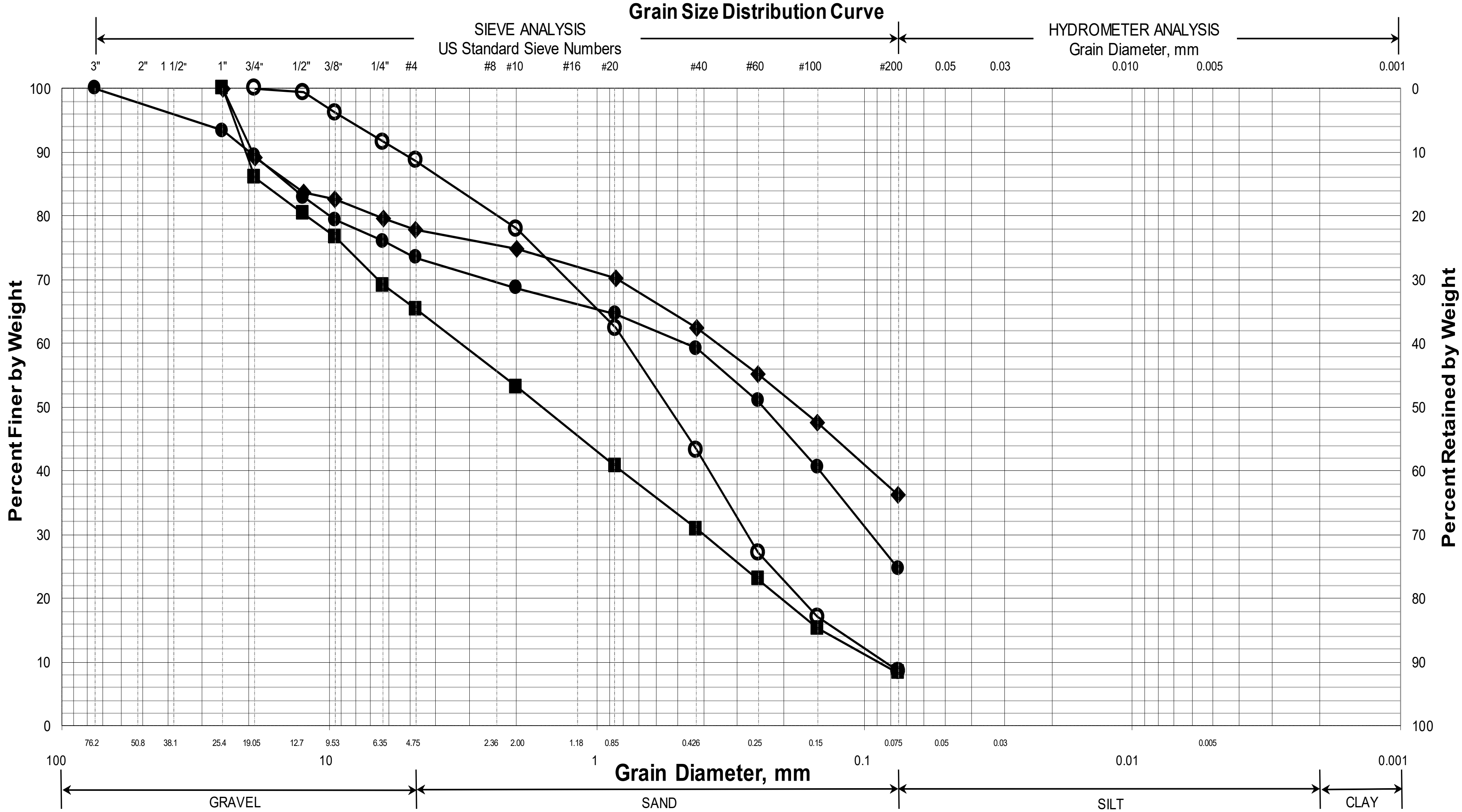
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LMB-101/2D	12+70.5	8.6 RT	5.0-7.0	SAND, little gravel, trace silt.	1.7			
◆	BB-LMB-101/3D(A)	12+70.5	8.6 RT	11.5-12.0	Silty SAND, some gravel.	13			
■	BB-LMB-102/1D	12+57.7	9.8 LT	1.0-3.0	SAND, some gravel, trace silt.	2.6			
●	BB-LMB-102/4D	12+57.7	9.8 LT	15.0-16.2	SAND, some gravel, some silt.	10.9			
▲									
×									

WIN
028246.00
Town
Litchfield
Reported by/Date
WHITE, TERRY A 1/29/2025

SHEET 1

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
241497	**BB-LMB-101/2D**	**GEOTECHNICAL (DISTURBED)**	**9/4/2024**	**1/17/2025**

Sample Type: **GEOTECHNICAL** Location: Station: **12+70.5** Offset, ft: **8.6** **RT** Dbfg, ft: **5.0-7.0**

WIN/Town **028246.00 - LITCHFIELD** 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]	**99.4**
⅜ in. [9.5 mm]	**96.2**
¼ in. [6.3 mm]	**91.6**
No. 4 [4.75 mm]	**88.7**
No. 10 [2.00 mm]	**78.0**
No. 20 [0.850 mm]	**62.4**
No. 40 [0.425 mm]	**43.3**
No. 60 [0.250 mm]	**27.2**
No. 100 [0.150 mm]	**17.1**
No. 200 [0.075 mm]	**8.6**

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), %	**1.7**

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **1/27/2025**

Paper Copy: Lab File; Project File; Geotech File

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
241498	**BB-LMB-101/3D(A)**	**GEOTECHNICAL (DISTURBED)**	**9/4/2024**	**1/17/2025**

Sample Type: **GEOTECHNICAL** Location: Station: **12+70.5** Offset, ft: **8.6 RT** Dbfg, ft: **11.5-12.0**

WIN/Town **028246.00 - LITCHFIELD** 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]	**89.2**
½ in. [12.5 mm]	**83.7**
⅜ in. [9.5 mm]	**82.7**
¼ in. [6.3 mm]	**79.7**
No. 4 [4.75 mm]	**77.9**
No. 10 [2.00 mm]	**74.9**
No. 20 [0.850 mm]	**70.3**
No. 40 [0.425 mm]	**62.4**
No. 60 [0.250 mm]	**55.1**
No. 100 [0.150 mm]	**47.5**
No. 200 [0.075 mm]	**36.2**

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)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**13.0**

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:

Insuficient amount of material to run T88. T27 run instead.

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **1/27/2025**

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MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
241499	**BB-LMB-102/1D**	**GEOTECHNICAL (DISTURBED)**	**9/4/2024**	**1/17/2025**

Sample Type: **GEOTECHNICAL** Location: Station: **12+57.7** Offset, ft: **9.8** **LT** Dbfg, ft: **1.0-3.0**

WIN/Town **028246.00 - LITCHFIELD** 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]	**86.0**
½ in. [12.5 mm]	**80.4**
⅜ in. [9.5 mm]	**76.7**
¼ in. [6.3 mm]	**69.1**
No. 4 [4.75 mm]	**65.4**
No. 10 [2.00 mm]	**53.2**
No. 20 [0.850 mm]	**40.7**
No. 40 [0.425 mm]	**30.9**
No. 60 [0.250 mm]	**23.0**
No. 100 [0.150 mm]	**15.3**
No. 200 [0.075 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)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**2.6**

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: **1/27/2025**

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
241500	**BB-LMB-102/4D**	**GEOTECHNICAL (DISTURBED)**	**9/4/2024**	**1/17/2025**

Sample Type: **GEOTECHNICAL** Location: Station: **12+57.7** Offset, ft: **9.8** **LT** Dbfg, ft: **15.0-16.2**

WIN/Town **028246.00 - LITCHFIELD** 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]	**100.0**
1 in. [25.0 mm]	**93.4**
¾ in. [19.0 mm]	**89.4**
½ in. [12.5 mm]	**82.9**
⅜ in. [9.5 mm]	**79.4**
¼ in. [6.3 mm]	**76.0**
No. 4 [4.75 mm]	**73.5**
No. 10 [2.00 mm]	**68.7**
No. 20 [0.850 mm]	**64.6**
No. 40 [0.425 mm]	**59.2**
No. 60 [0.250 mm]	**51.0**
No. 100 [0.150 mm]	**40.5**
No. 200 [0.075 mm]	**24.7**

Miscellaneous Tests

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), %	**10.9**

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: **1/27/2025**

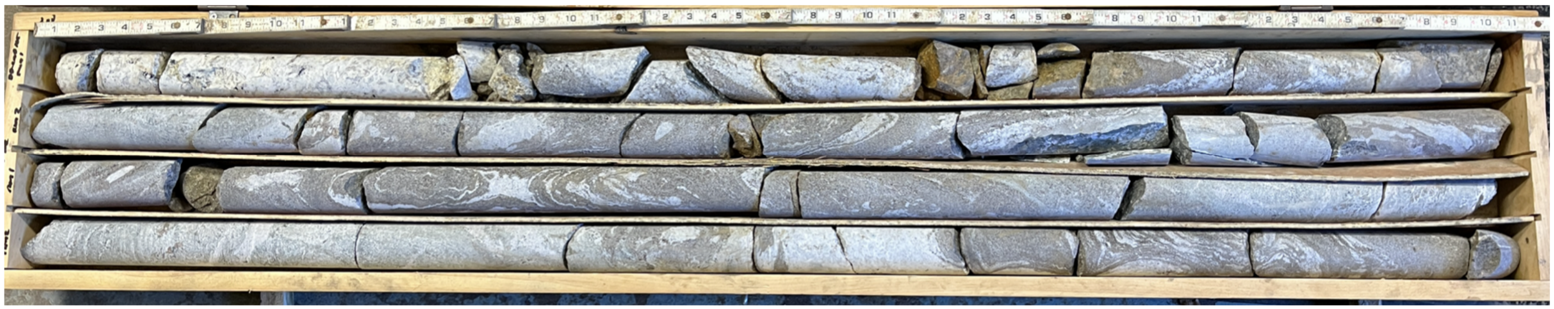
Paper Copy: Lab File; Project File; Geotech File

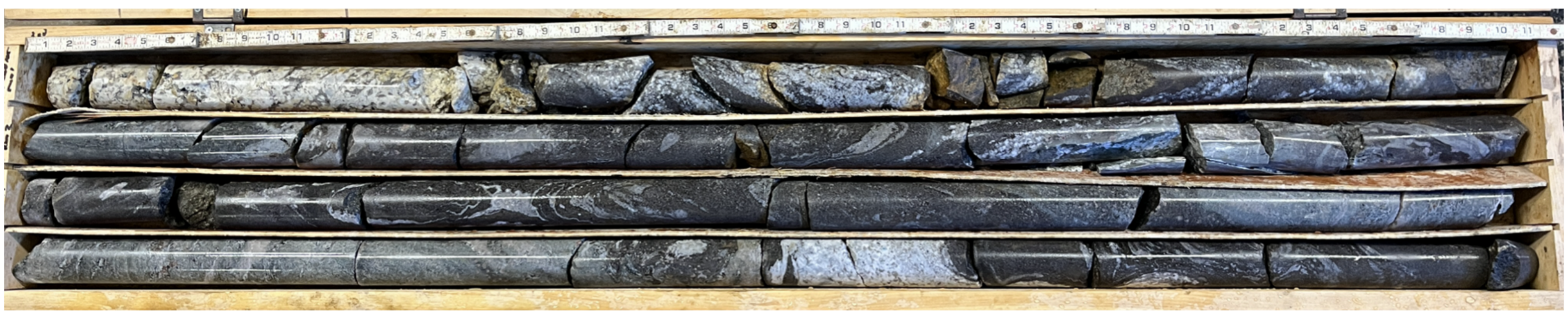
APPENDIX D – ROCK CORE PHOTOGRAPHS

MaineDOT Bridge No. 2524
Maxwell Bridge over Maxwell Brook
Litchfield, ME
WIN 28246.00

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB – LMB– 101	R1	12.7 - 17.7	60	100%	24	40%	SCHIST	1
BB – LMB – 101	R2	17.7 - 22.7	60	100%	27	45%	SCHIST	2
BB – LMB – 102	R1	16.2 - 21.2	58	97%	45	75%	SCHIST	3
BB – LMB – 102	R2	21.2 - 26.2	60	100%	48	80%	SCHIST	4





Notes:
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.

APPENDIX E – CALCULATIONS

Objective

Calculate soil bearing resistance for a culvert bearing on granular borrow and/or glacial till. Evaluate strength and service bearing resistance .

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 9th edition, 2020, (AASHTO LRFD), Articles 10.5.5.2.2 and 10.6.3.1.
2. Terzaghi, Peck & Mesri, Soil Mechanics in Engineering Practice, Third Edition, 1996.

Soil Properties and Geotechnical Inputs

$\phi_f := 32\text{deg}$ Internal friction angle of cohesionless soil of the granular borrow (considered suitable/conservative for glacial till)

$\phi_b := 0.45$ Bearing resistance factor as specified in Table 10.5.5.2.2-1 (Theoretical Method, SPT Data, Strength Limit, Spread Footing)

$c := 0\text{ksf}$ Cohesion, taken as undrained shear strength

$\gamma := 120\text{pcf}$ Unit weight of soil above or below the bearing depth of the footing

$N_c := 30.1$ Cohesion term bearing capacity factor as specified in Table 10.6.3.1.2a-1

$N_q := 18.4$ Surcharge term bearing capacity factor as specified in Table 10.6.3.1.2a-1

$N_\gamma := 22.4$ 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 or below depth of footing (D_f) $C_{wq} := 0.5$ $C_{w\gamma} := 0.5$

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_γ, s_q:= Footing shape correction factors as specified in Table 10.6.3.1.2a-2

$S_c := 0.75\text{in}$ Allowable settlement. Because the supporting soil is about 5' thick or less, using 0.75" criteria for infinite half space assumption will result in 0.5" or less of settlement.

q_s= Service limit bearing resistance for allowable settlement

$N_{60} := 20$ N_{60} value of granular borrow interpolated from Table 10.4.6.2.4-1

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

GZA
GeoEnvironmental, Inc
707 Sable Oaks Drive
Suite 150
South Portland, Maine 04106
207-879-9190
Fax 207-879-0099

Engineers and Scientists

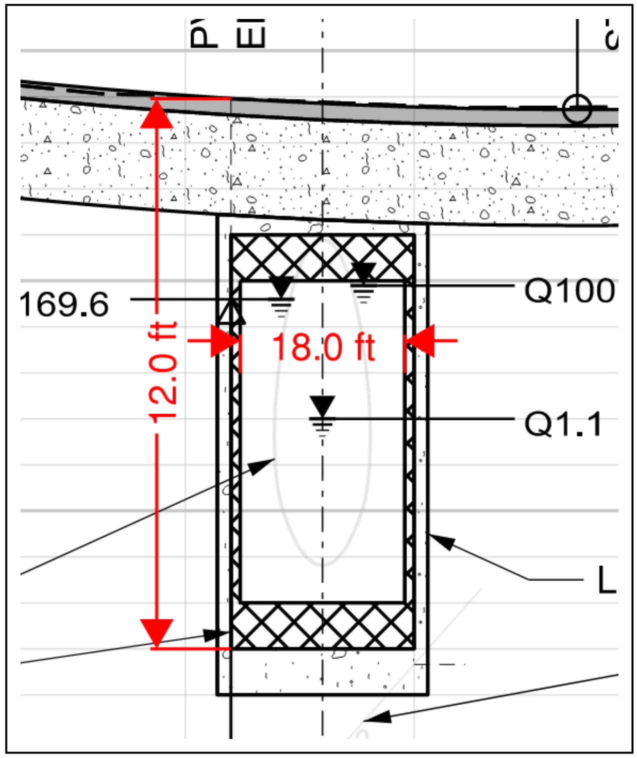
Footing Dimensions

$B_{1.max} := 20 \cdot ft$ Maximum Footing Width

$B_1 := 16 \cdot ft, 18 \cdot ft .. B_{1.max}$ Range of effective footing widths considered (includes eccentricity)

$L_1 := 75ft$ Length of culvert Base

$D_f := 12ft$ Footing embedment depth



Strength Limit Design

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

$q.D = \phi_b \times q_n$ Factored Bearing Resistance Formula

<u>Correction Factors</u>

$d_{qtable}(B_1) := \frac{D_f}{B_1}$

$d_{qtable}(B_1)$ Using Table 10.6.3.1.2a-4

$d_q := 1$ dq assumed soil above footing less competent than soil below footing.

$s_c(B_1) := 1 + \left(\frac{B_1}{L_1}\right)\left(\frac{N_q}{N_c}\right)$

$s_q(B_1) := 1 + \left(\frac{B_1}{L_1} \tan(\phi_f)\right)$

$s_\gamma(B_1) := 1 - 0.4\left(\frac{B_1}{L_1}\right)$

$s_c(B_1) =$
1.13
1.15
1.16

$s_q(B_1) =$
1.13
1.15
1.17

$s_\gamma(B_1) =$
0.91
0.9
0.89

Bearing Capacity Factors

$N_{cm}(B_1) := N_c \cdot s_c(B_1)$

$N_{cm}(B_1) =$

34.03
34.52
35.01

$N_{qm}(B_1) := N_q \cdot s_q(B_1) \cdot d_q$

$N_{qm}(B_1) =$

20.9
21.2
21.5

$N_{\gamma m}(B_1) := N_\gamma \cdot s_\gamma(B_1)$

$N_{\gamma m}(B_1) =$

20.5
20.2
20

Nominal Bearing Resistance

$$q_n(B_1) := \overrightarrow{\left(c \cdot N_{cm}(B_1) + \gamma \cdot D_f \cdot N_{qm}(B_1) \cdot C_{wq} + 0.5 \cdot \gamma \cdot B_1 \cdot N_{\gamma m}(B_1) \cdot C_{w\gamma}\right)}$$

$q_n(B_1) =$ · ksf

24.8
26.2
27.5

Factored Bearing Resistance - Strength Limit State

$q_D(B_1) := \phi_b \cdot q_n(B_1)$

$q_D(B_1) =$ · ksf

11.2
11.8
12.4

for $B_1 =$ · ft

16
18
20

Service Limit Design

Evaluate service limit bearing resistance for the specified allowable settlement using the semi-empirical SPT Method by Burland and Burbidge (1985) provided in Terzaghi, Peck & Mesri, 96.

$S_{cm} := S_c \cdot \frac{1}{1\text{mm}}$ $S_{cm} = 19$ Allowable settlement in millimeters and unitless

$B_{1m}(B_1) := B_1 \cdot \frac{1}{1\text{m}}$ Effective footing width in meters and unitless

$B_{1m}(B_1) =$

4.9
5.5
6.1

Correction formula for rectangular footings (Terzaghi EQ 50.14)

$$S_{cmr}(B_1) := S_{cm} \cdot \left[\frac{\left[1.25 \cdot \left(\frac{L_1}{B_1} \right) \right]}{\left(\frac{L_1}{B_1} \right) + 0.25} \right]^2$$

$S_{cmr}(B_1) =$

27
26
26

$$EQ_1(B_1) := S_{cm} \cdot \left(\frac{S_{cm}}{S_{cmr}(B_1)} \right) \qquad EQ_2(B_1) := \frac{N_{60}^{\ 1.4}}{1.7 \cdot B_{1m}(B_1)^{0.75}}$$

$EQ_1(B_1) =$

13.53
13.7
13.87

$EQ_2(B_1) =$

11.88
10.88
10.05

$$q_{snc}(B_1) := \overrightarrow{\left(EQ_1(B_1) \cdot EQ_2(B_1)\right)}$$

$q_{snc}(B_1) =$

160.7
149.0
139.4

Formula results are in kPa (Terzaghi EQ 50.28). Results represent normally consolidated soil.

$q_s(B_1) := q_{snc}(B_1)$

$q_s(B_1) =$

161
149
139

Assumes supporting sand is normally consolidated at current effective stress

$q_{sm}(B_1) := q_s(B_1) \cdot 1kPa$

$q_{sm}(B_1) =$

161
149
139

$\cdot kPa$

Service limit bearing resistance for allowable settlement (metric units)

$q_{se}(B_1) := q_{sm}(B_1)$

$q_{se}(B_1) =$

3.4
3.1
2.9

$\cdot ksf$

Service limit bearing resistance for allowable settlement (English units)

$q_{se.c}(B_1) := q_{se}(B_1) \cdot m^{.75}$

$q_{se.c}(B_1) =$

3.4
3.1
2.9

$m^{0.8} \cdot ksf$

$B_1 =$

16
18
20

$\cdot ft$

English Units

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
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

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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.

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

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JOB: 09.0026259.00 Maxwell Bridge, Litchfield, ME
SUBJECT: Lateral Earth Pressures
SHEET: 1 OF 1
CALCULATED BY E. Tome 1/21/25
CHECKED BY A. Blaisdell 2/3/25

Subject:

Evaluate lateral earth pressure coefficients for a precast box culvert walls, inlet and outlet head walls and in-line wingwalls.

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 9th Edition (2020)
3. U.S. Army Corps of Engineers Engineer Manual 1110-2-2502, Retaining and Flood Walls

Input Parameters:

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

$\delta_f := 19.5\text{deg}$ — Average value, precast concrete against clean sand/silty sand-gravel mixture *(AASHTO LRFD Table 3.11.5.3-1)*

$\beta := 26.6\text{deg}$ — Angle of backfill to the horizontal (2H:1V backslope)

$\theta := 90\cdot\text{deg}$ — Angle of back face of wall to the horizontal

Earth Pressure Coefficients:

Outlet Walls Fixed to Box Culvert:

Assume translation and rotation of culvert with inlet and outlet walls is inadequate to achieve active earth pressure. Therefore, design for at-rest earth pressure.

$K_o := 1 - \sin(\phi) = 0.47$ — At-rest Earth Pressure Coefficient, Level Ground

Outlet Walls free to rotate:

The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considereed as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall. Use Rankine theory for active earth pressure.

For unsupported culvert walls extending beyond the box, with horizontal backslope:

$$K_{ar} := \tan\left(45\text{deg} - \frac{\phi}{2}\right)^2 \qquad K_{ar} = 0.31$$

For a sloped 2H:1V backfill:

$$K_{ar} := \cos(\beta)\cdot\frac{\left[\cos(\beta) - \sqrt{(\cos(\beta))^2 - (\cos(\phi))^2}\right]}{\left[\cos(\beta) + \sqrt{(\cos(\beta))^2 - (\cos(\phi))^2}\right]}$$

$K_{ar} = 0.46$

March 2014

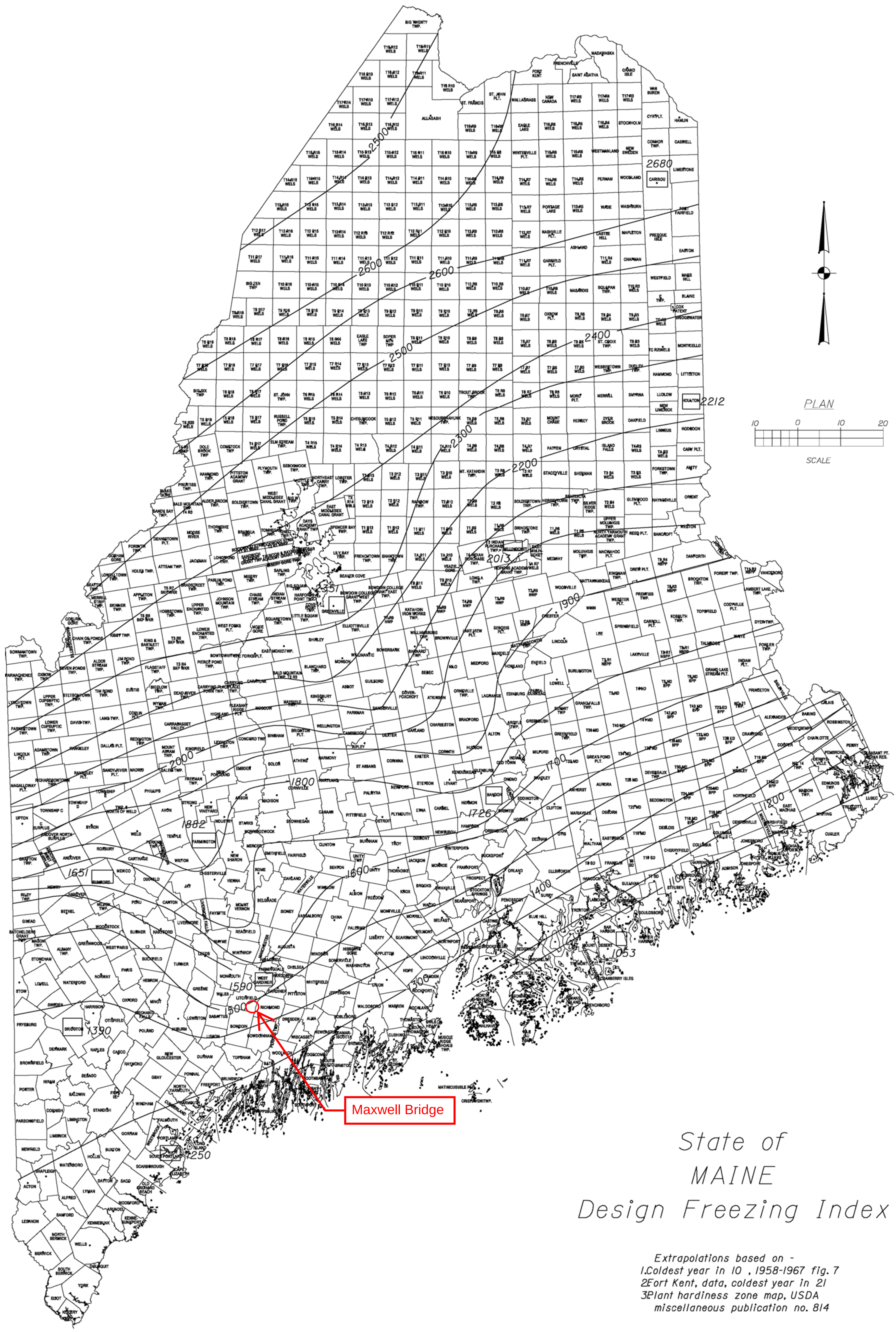

Figure 5-1 Maine Design Freezing Index Map

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

1490

75" = 6.3'

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

Granular materials anticipated near the culvert bearing elevations have an average water content of 15 percent. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing index of 1,490 the estimated depth of frost penetration is 75 inches.