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GEOTECHNICAL DESIGN REPORT

Meadow Brook Bridge Culvert Replacement

BRIDGE NO. 2535

MAINE DOT WIN 025449.00

NEWCASTLE, MAINE

January 2025
09.0026234.00

Prepared for:
Maine Department of Transportation
Augusta, Maine

Prepared by:
GZA GeoEnvironmental, Inc.
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VIA EMAIL

January 8, 2025
File No. 09.0026234.00

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

Re: Geotechnical Design Report
Meadow Brook Bridge No. 2535 Culvert Replacement
Jones Wood Road over Deer Meadow Brook
Maine Department of Transportation WIN 025449.00
Newcastle, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the replacement of Meadow Brook Bridge Culvert, which carries Jones Wood Road over Deer Meadow Brook in Newcastle, 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. 17 dated May 2, 2024 for WIN 025449.00, and the attached *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.

Blaine M. Cardali, P.E.
Senior Project Manager

STATE OF MAINE
CHRISTOPHER L. SNOW
No. 7275
LICENSED
PROFESSIONAL ENGINEER
1/08/2025

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Consultant Reviewer

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Principal

BMC/ARB/CLS:pa
p:\09 jobs\0026200s\09.0026234.00 - medot - newcastle meadow brook bridge 2535\report\26234.00 newcastle meadow brk bridge culvert gdr 1.8.2025.docx

Attachment: Geotechnical Design Report

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APPENDICES

1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the replacement of Meadow Brook Bridge No. 2535 Culvert in Newcastle, 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. 17 dated May 2, 2024 for WIN 025449.00, and the attached *Limitations* contained in **Appendix A** of this report.

1.1 BACKGROUND

The project includes the replacement of Meadow Brook Bridge No. 2535 carrying Jones Wood Road over Deer Meadow Brook in Newcastle, Maine, the location of which is shown in **Figure 1**. The bridge was originally constructed in 1931 or earlier and consisted of timber stringers resting on stone abutments. In 1981, it was widened to its current width and the bridge deck was replaced. The northeast wingwall was replaced in 2019. The current bridge is a single-span structure with a cast-in-place concrete slab deck and a maximum span of 10 feet. Recent inspections have shown that the condition of the substructure is poor and is considered structurally deficient. The corners of the breast and wingwalls are heavily spalled and cracked. Additionally, there are signs of reinforcement corrosion and concrete staining as well.

We understand plans are to construct a new 66-foot-long box culvert with a span of 22 feet, a rise of 8 feet, and a 10-degree skew. The culvert will have 1-foot-tall precast headwalls and 2-foot-deep toe walls at the inlet and outlet. The box culvert invert will be filled with 2 feet of special fill to create a natural streambed. 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 bridge replacement. To meet these objectives, GZA completed the following Scope of Services:

- Reviewed the results of two test 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, earth pressures, and seismic design parameters; geotechnical construction considerations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

Two test borings were drilled and logged by MaineDOT between January 4 and 11, 2023. Boring BB-NDMB-101 was drilled on the northwest side of the existing bridge and BB-NDMB-102 was drilled on the southeast side. The test borings were drilled using a CME-45C drill rig to depths ranging from approximately 29 to 32 feet below ground surface (bgs) and were terminated in the bedrock.

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 0.906. In situ field vane shear tests were conducted in expected clay soils. Two vane shear tests were attempted at each test interval. One vane shear test and one thin-walled tube sample was attempted in BB-NDMB-102, but the vane advancement and sample recovery were unsuccessful. Approximately 10 feet of bedrock core was obtained in both borings using NQ2 coring equipment. At the completion of drilling, the borings were backfilled with cuttings and sand, and were 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:

- Five (5) gradation analysis (Two [2] with hydrometer) / MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments;
- Two (2) sets of Atterberg Limits; and
- Five (5) 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 bridge consists of Wetland deposits, described as peat, muck, silt, and clay in poorly drained areas. Presumpscot Formation, which consists of a stratified mixture of sand, silt and clay deposited by glacial melt, is also present. Thin drift areas of Till are mapped to the east and west of the site and described as loose to very compact, massive to weakly stratified mixture of sand, silt, and gravel-size rock debris deposited by glacial ice.

Mapping[2] in the vicinity of the site shows the bedrock at the bridge site mapped as the Bucksport Formation. The Bucksport Formation is described as medium-bedded, medium to light grey, quartz-feldspar-biotite granofels interbedded with greenish grey granofels.

4.2 SUBSURFACE PROFILE

Three soil units were encountered in the test borings below surficial asphalt and above bedrock: Fill, Wetland Deposit, and Marine Sand and Gravel. Approximately five to six 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	7.5	Brown, medium to very dense, Silty Sand, little gravel to Sandy GRAVEL little silt (USCS: SM, GM, GP-GM). MaineDOT Frost Classification = I *Encountered in both borings.*
Wetland Deposit	6 to 6.5	Varies from: Dark grey, very soft to soft, Silty CLAY, trace fine sand to: very loose, fine to coarse SAND, little to some silt (USCS: CL, SM). MaineDOT Frost Classification = II-III *Encountered in both borings.*
Marine Sand and Gravel	4.5 to 8	Brown/Grey, loose to medium dense, Sandy GRAVEL to Gravelly SAND, little to trace silt (USCS: SM, GP-GM). MaineDOT Frost Classification = 0-III *Encountered in both borings.*
Top of Bedrock Elevation	Approximately El. 77.1 to 74.8 (19.0 to 21.7 feet bgs)	

[1]Thompson, Woodrow B., 2009, Surficial geology of the Damariscotta quadrangle, Maine: Maine Geological Survey, Open-File Map 09-6, map, scale 1:24,000. Maine Geological Survey Maps. 1838. http://digitalmaine.com/mgs_maps/1838

[2] Grover, Timothy W., and Newberg, Donald W., 2016, Reconnaissance bedrock geology of the Damariscotta quadrangle, Maine: Maine Geological Survey, Open-File Map 16-22, color map, scale 1:24,000. Maine Geological Survey Maps. 59. http://digitalmaine.com/mgs_maps/59

4.2.1 Bedrock

Bedrock was cored in both test borings. Bedrock was described as moderately hard to hard, light to medium greenish grey, very fine to medium grained SYENITE and GRANOFELS. Joints were described as fresh, steeply dipping. The core sample taken from BB-NDMB-101 had a large, brecciated zone from 23.9 feet to 26.7 feet. The Rock Quality Designation in the core runs ranged from 45 to 88 percent indicating poor to good quality rock.

4.2.2 Groundwater

Groundwater was not measured in either of the borings. However sample descriptions on the boring logs indicate wet samples starting between El. 91.1 and 86.5 which is roughly coincident with the Q1.1 (El. 91.2). 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 feet horizontal to 1 foot vertical (2H:1V) or flatter.

We anticipate that the embankments will be reconstructed over fill and soft / very loose Wetland Deposits overlying loose to medium dense Marine Sand and Gravel. Due to the limited thickness of Wetland Deposits to remain in place and the density of the Marine Sand and Gravel layers and since no raise in grade is proposed, 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 22-foot wide 8-foot-high box culvert bearing on a 1-foot-thick layer of Underdrain Backfill Material, Type C (MaineDOT Pay Item 203.55 Culvert Bedding Stone), separated from the natural Marine Sand and Gravel on the bottom and sides by Stabilization/Reinforcement Geotextile (MaineDOT Standard Specification 722.01). However, the Wetland deposit was found to extend approximately 1.4 to 2.5 feet below the anticipated Culvert bearing elevation. Therefore, prior to placement of the 1-foot-thick layer of Underdrain Backfill Material the bearing area should have any unsuitable clay or silt from the Wetland deposit removed and replaced with Underdrain Backfill Material, Type C, anticipated to be 0.4 to 1.5 feet below the bottom of the bedding material, and fully encapsulated by Stabilization/Reinforcement Geotextile (MaineDOT Standard Specification 722.01).

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 SPREAD FOOTING 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) and could require up to an additional 1.5 feet for removal of the Wetland Deposit, resulting in excavation depths of approximately 15 to 16 feet below existing grades. At these depths, the exposed soils are anticipated to range from very soft to soft wetland deposit to loose to medium dense Marine Sand and Gravel. However, as previously noted, it is anticipated that if the Wetland deposit is encountered, it will be removed and replaced with Type C (MaineDOT Pay Item 203.55 Culvert Bedding Stone) fully encapsulated by Stabilization/Reinforcement Geotextile (MaineDOT Standard Specification 722.01). The following sections discuss settlement and bearing related to the proposed culvert foundations.

5.5.1 Settlement

GZA evaluated the effective stress at the bearing elevation of the proposed box culvert under the existing conditions and the proposed conditions. The results indicate that the proposed construction will result in a slight increase in effective stress. Since unsuitable clay or silt from the wetland deposit will be removed, the box culvert will bear directly on loose to medium dense Marine Sand and Gravel. Therefore, settlement is anticipated to occur elastically as the structure and backfill are placed. We estimate the post-construction foundation settlement will be 1 inch or less. Calculations are presented in **Appendix D**.

5.5.2 Strength Bearing Resistance

Bearing resistance values have been developed for equivalent footings bearing on marine sand and gravel using the theoretical method (Munfakh et al., 2001) based on SPT data. Bearing resistances were evaluated in accordance with Articles 10.6.3.1.1 and 10.6.3.1.2a of AASHTO LRFD.

The calculated bearing resistance values are presented in **Appendix D** and are presented in the table below for the culvert.

Service Bearing Resistance

Since the Marine Sand and Gravel is a granular material, we anticipate that any compression of that layer would occur elastically as the culvert and backfill are placed. Based on the elastic settlement anticipated, we estimate the post-construction foundation settlement will be 1 inch or less. Calculations are presented in **Appendix E**. The calculated service bearing resistance value is presented in **Appendix D** and in the table below for the culvert.

BEARING RESISTANCE VALUES FOR FOOTINGS ON SOIL				
Footing	**Footing Width (feet)**	**Nominal Bearing Resistance (ksf)**	**Factored Bearing Resistance, Strength Limit State (ksf)**	**Service Bearing Resistance (ksf)**
Precast Culvert	24	28	12.6	3.5

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 at the culvert and embankments, either as existing fill, or imported backfill. The bearing material below the culvert is anticipated to be imported fill or Marine Sand and Gravel. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,350, and with coarse-grained materials and moderate moisture content (Approx. 30 percent), the estimated depth of frost penetration is approximately 5.9 feet. However, 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 should also be provided for scour protection where the embankment side slopes will be near or below typical water levels in Meadow 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 behind 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 Marine Sand and Gravel, except for the precast concrete toe walls, which should bear directly on naturally deposited Marine Sand and Gravel. Prior to placement of the 1-foot-thick layer of Underdrain Backfill Material, the bearing area should have any unsuitable clay or silt from the Wetland deposit removed and replaced with Type C (MaineDOT Pay Item 203.55 Culvert Bedding Stone), anticipated to be 0.4 to 1.5 feet below the bottom of the bedding material, and fully encapsulated by Stabilization/Reinforcement Geotextile (MaineDOT Standard Specification 722.01). Culvert 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 Marine Sand and Gravel or compacted Granular Borrow for Underwater Backfill MaineDOT 703.19.

- The culvert subgrade surfaces should be cleaned of soil 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 (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 15 to 16 feet below existing pavement grades and up to 4 feet below the Q 1.1 (El. 91.2). Sheet-pile-supported and/or sloped open cut excavation techniques may both 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. 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 marine sand and gravel is saturated, the separation 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.

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TABLES

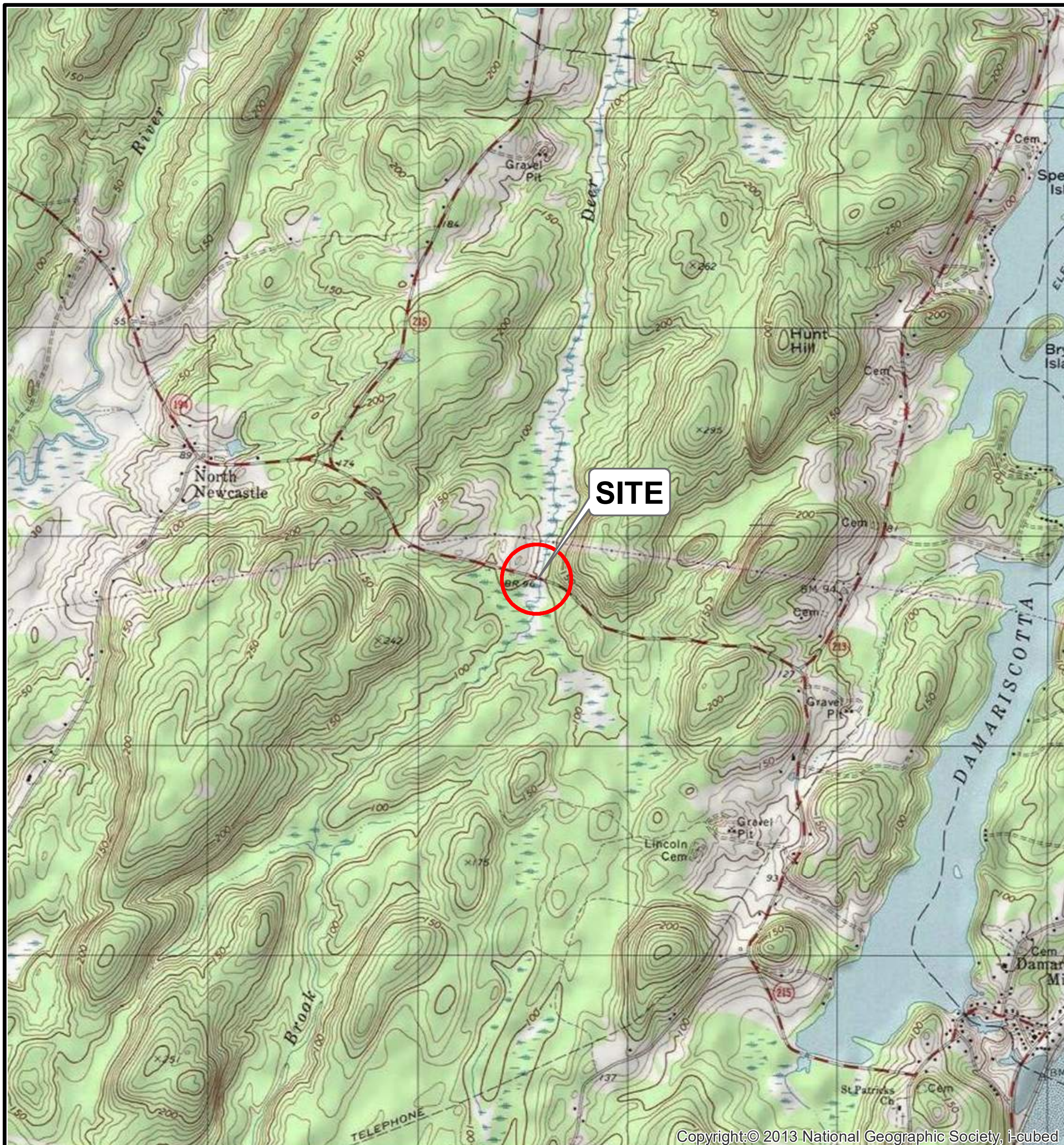
TABLE 1
Summary of Subsurface Explorations
Meadow Brook Bridge #2535 carries Jones Wood Road over Deer Meadow Brook
Newcastle, Maine
GZA job#: 09.0026234.00

Boring ID	Station	Offset	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	Wetland Deposit	Marine Sand & Gravel	Bedrock	Asphalt	Fill	Wetland Deposit	Marine Sand & Gravel					El. (ft)	Depth (ft)
BB-NDMB-101	13+02.6	7.1' Lt.	96.5	96.5	96.0	88.5	82.7	74.8	0.5	7.5	5.8	7.9	21.7	74.8	31.7	64.8	NM	NM
BB-NDMB-102	13+23.0	9.6' Rt.	96.1	96.1	95.7	88.1	81.6	77.1	0.4	7.6	6.5	4.5	19.0	77.1	29.0	67.1	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.

FIGURES



USGS QUADRANGLE LOCATION

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.

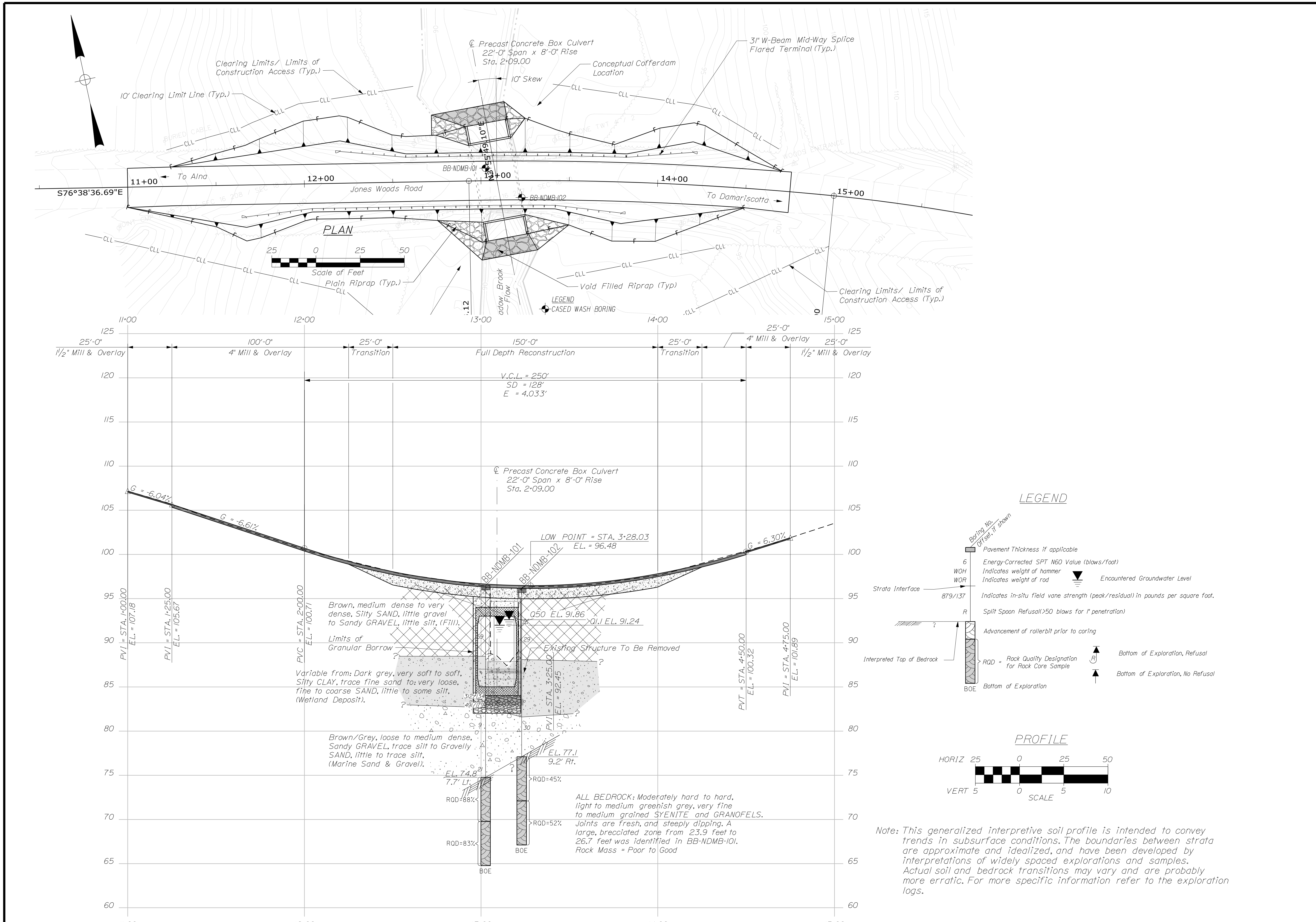
0 1,000 2,000 4,000 6,000

SCALE IN FEET

Data Supplied by :

PROJ. MGR.: BMC DESIGNED BY: EAF REVIEWED BY: BMC OPERATOR: EAF DATE: 09-27-2024	**LOCUS PLAN**	JOB NO. 09.0026234.00	
	MEADOW BROOK BRIDGE NO. 2535 NEWCASTLE, MAINE	FIGURE NO. 1	

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: Terry.White Date:12/13/2024



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN 025449.00

BRIDGE PLANS

BRIDGE NO. 2535

		BY	DATE
PROJ. MANAGER	DEVAN EATON		
DESIGN-DETAILED	E. BREWER	MAC	
CHECKED-REVIEWED	J. MANAHAN	T. WHITE	MAR 2023
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MEADOW BROOK BRIDGE
DEER MEADOW BROOK
NEWCASTLE LINCOLN COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 2

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

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):

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

*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: Meadow Brook Bridge #2535 carries Jones Wood Road over Deer Meadow Brook
Location: Newcastle, Maine

Boring No.: BB-NDMB-101

WIN: 25449.00

Driller:	MaineDOT	**Elevation (ft.)**	96.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/4/2023; 08:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+02.6, 7.1 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.906		**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	96.0		6½" HMA. — 0.5	
5	1D	24/16	5.00 - 7.00	3/25/14/55	39	59				Brown with white veins, damp, very dense, Silty SAND, little gravel, trace rock fragments, (Fill).	
								88.5		Wood at 7.5-8.0 ft bgs. — 8.0	
10	2D	24/24	10.00 - 12.00	WOR/WOR/WOH/ WOH	---		OH			Dark grey, wet, very soft, Silty CLAY, trace fine sand, (Wetland Deposit). Roller Coned ahead from 10.0-15.0 ft bgs.	G#380776 A-6, CL WC=34.8% LL=32 PL=20 PI=12
	V1		12.63 - 13.00	Su=312/134 psf						55x110 mm vane raw torque readings: V1: 7.0-3.0 ft-lbs Only pushed 10", felt gritty, not accurate reading.	
	V2		13.46 - 13.83	Su=491/179 psf				82.7		V2: 11.0-4.0 ft-lbs — 13.8	
15	3D	24/18	15.00 - 17.00	3/3/3/1	6	9				Grey, wet, loose, SAND, little silt, trace gravel, (Marine Sand and Gravel).	G#380777 A-2-4, SM WC=20.9% Non-Plastic
							6				
							13	79.5		— 17.0	
							15				
							19				
20	4D	20.4/6	20.00 - 21.70	6/8/6/7(2.4")	14	21	26			Brown, wet, medium dense, Gravelly fine to coarse SAND, little silt, (Marine Sand and Gravel). a60 blows for 0.7 ft.	
	R1	60/60	21.70 - 26.70	RQD = 88%			a60 NQ-2	74.8		— 21.7 Top of Bedrock at Elev. 74.8 ft. R1: Bedrock: Light grey to medium greenish grey, very fine to medium-grained SYENITE and GRANOFELS, hard, fresh, massive one fracture with rock gouge at 23.1' bgs. Brecciated zone from 23.9' to 26.7'. [Bucksport Formation]	
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meadow Brook Bridge #2535 carries Jones Wood Road over Deer Meadow Brook
Location: Newcastle, Maine

Boring No.: BB-NDMB-101
WIN: 25449.00

Driller:	MaineDOT	**Elevation (ft.)**	96.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/4/2023; 08:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+02.6, 7.1 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.906	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										Rock Quality = Good. R1: Core Times (min:sec) 21.7-22.7 ft (1:54) 22.7-23.7 ft (2:58) 23.7-24.7 ft (2:52) 24.7-25.7 ft (3:01) 25.7-26.7 ft (3:11) 100% Recovery	
	R2	60/60	26.70 - 31.70	RQD = 83%				69.8		26.7	
30										R2: Bedrock: Similar to R1, except moderately hard GRANOFELS with clasts of SYENITE and two joints at steep angles, one annealed, one fresh. Rock Quality = Good. R2: Core Times (min:sec) 26.7-27.7 ft (2:50) 27.7-28.7 ft (3:06) 28.7-29.7 ft (3:31) 29.7-30.7 ft (3:12) 30.7-31.7 ft (3:31) 100% Recovery	
								64.8		31.7	
										Bottom of Exploration at 31.7 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meadow Brook Bridge #2535 carries Jones Wood Road over Deer Meadow Brook
Location: Newcastle, Maine

Boring No.: BB-NDMB-102
WIN: 25449.00

Driller:	MaineDOT	**Elevation (ft.)**	96.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2023; 08:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+23.0, 9.6 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.906	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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	95.7		5" HMA. 0.4	
5	1D	24/7	5.00 - 7.00	6/5/14/60	19	29				Brown, wet, medium dense, Sandy GRAVEL, little silt, (Fill).	G#380778 A-1-a, GM WC=8.9%
								88.1		8.0 Peat on auger from 8.0-9.0 ft bgs. Sand and clay in wash water.	
							11				
10	MV/2D	24/.5	10.00 - 12.00	WOH/WOH/WOH/2	---		HYD PUSH			Failed 55x110 mm vane attempt. Dark grey, wet, very loose, fine to coarse SAND, some silt, moderate cementation, (Wetland Deposit).	
	MU1	24/24	12.00 - 14.00	Hydraulic Push						Grey, wet fine SAND, little silt, (Wetland Deposit). See Remarks	G#380779 A-2-4, SM WC=21.4%
							2				
							3	81.6		14.5	
15	3D	24/6	15.00 - 17.00	5/13/7/12	20	30	31			Brown, wet, medium dense, Sandy GRAVEL, trace silt, (Marine Sand and Gravel).	G#380780 A-1-b, GP-GM WC=11.5%
							30				
							15				
							30				
	R1	60/60	19.00 - 24.00	RQD = 45%			NQ-2	77.1		19.0 Top of Bedrock at Elev. 77.1 ft. R1: Bedrock: Light grey to medium greenish-grey, very fine to medium- grained, GRANOFELS, moderately hard, fresh, joint at steep angles, few Syenite clasts. [Bucksport Formation] Rock Quality = Poor. R1: Core Times (min:sec) 19.0-20.0 ft (2:25) 20.0-21.0 ft (3:31) 21.0-22.0 ft (2:37) 22.0-23.0 ft (4:55) 23.0-24.0 ft (6:41)	
20											
	R2	60/60	24.00 - 29.00	RQD = 52%				72.1			
25											

Remarks:

Sample fell out of tube as soon as it was out of water.

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: Meadow Brook Bridge #2535 carries Jones Wood Road over Deer Meadow Brook
Location: Newcastle, Maine

Boring No.: BB-NDMB-102
WIN: 25449.00

Driller:	MaineDOT	**Elevation (ft.)**	96.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2023; 08:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+23.0, 9.6 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.906	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
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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										100% Recovery	
										24.0 R2: Bedrock: Similar to R1, except more fractures and with rock gouge; zones of steep joints annealed with quartz/calcite present. Rock Quality = Fair. R2: Core Times (min:sec) 24.0-25.0 ft (3:18) 25.0-26.0 ft (3:05) 26.0-27.0 ft (3:35) 27.0-28.0 ft (3:43) 28.0-29.0 ft (4:32) 100% Recovery	
								67.1		29.0	
30										**Bottom of Exploration at 29.0 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

Sample fell out of tube as soon as it was out of water.

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): Newcastle   Work Number: 25449.00

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-NDMB-101, 2D	13+02.6	7.1 Lt.	10.0-12.0	380776	1	34.8	32	12	CL	A-6	III
BB-NDMB-101, 3D	13+02.6	7.1 Lt.	15.0-17.0	380777	1	20.9	-N	P-	SM	A-2-4	III
BB-NDMB-102, 1D	13+23	9.6 Rt.	5.0-7.0	380778	1	8.9			GM	A-1-a	I
BB-NDMB-102, MU1	13+23	9.6 Rt.	12.0-14.0	380779	1	21.4			SM	A-2-4	II
BB-NDMB-102, 3D	13+23	9.6 Rt.	15.0-17.0	380780	1	11.5			GP-GM	A-1-b	0

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

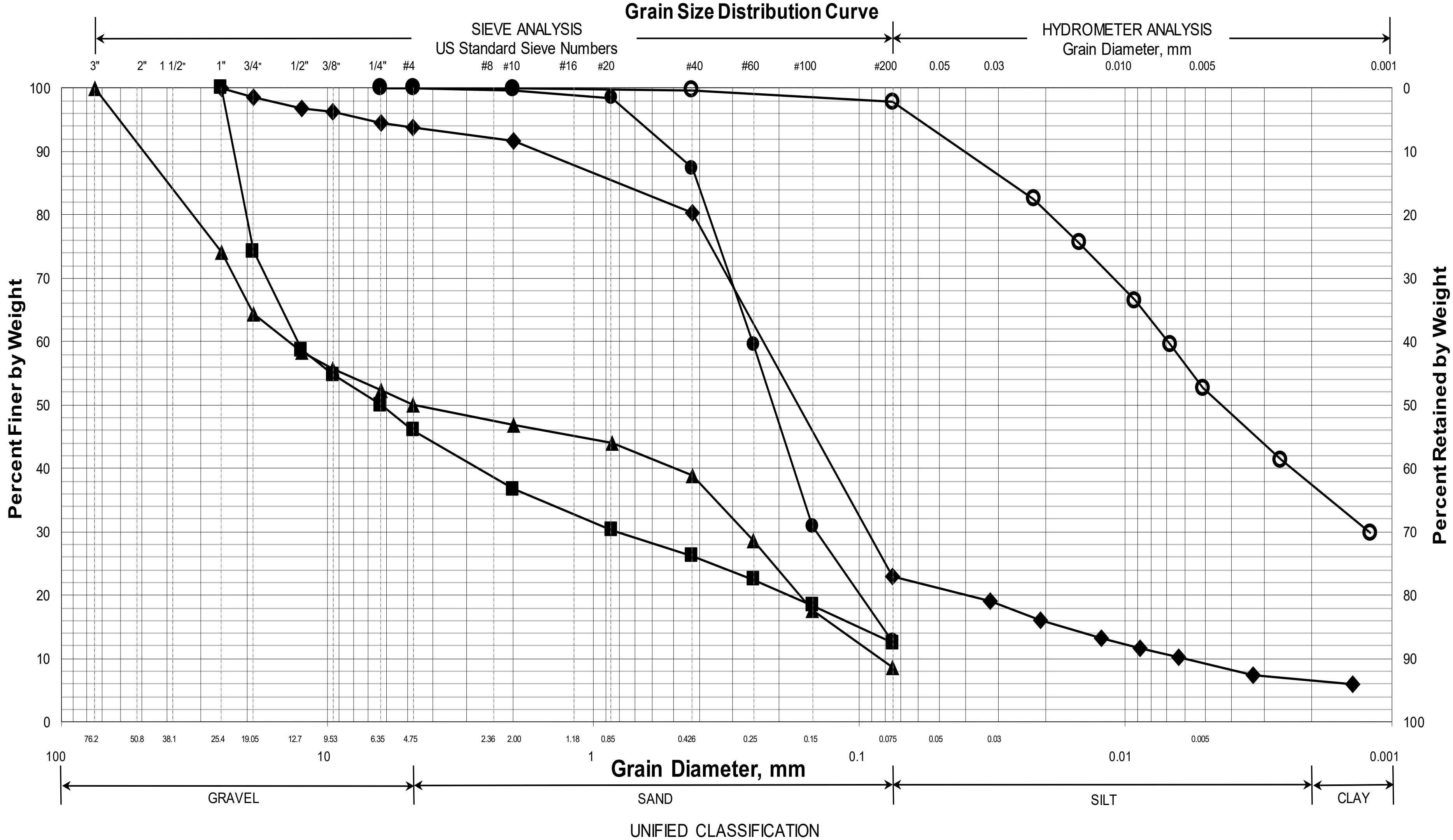
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-NDMB-101/2D	13+02.6	7.1 LT	10.0-12.0	Clayey SILT, trace sand.	34.8	32	20	12
◆	BB-NDMB-101/3D	13+02.6	7.1 LT	15.0-17.0	SAND, little silt, trace clay, trace gravel.	20.9			NP
■	BB-NDMB-102/1D	13+23	9.6 RT	5.0-7.0	Sandy GRAVEL, little silt.	8.9			
●	BB-NDMB-102/MU1	13+23	9.6 RT	12.0-14.0	SAND, little silt.	21.4			
▲	BB-NDMB-102/3D	13+23	9.6 RT	15.0-17.0	Sandy GRAVEL, trace silt.	11.7			
×									

WIN
025449.00
Town
Newcastle
Reported by/Date
WHITE, TERRY A 9/12/2024

SHEET 1

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380776	**BB-NDMB-101/2D**	**GEOTECHNICAL (DISTURBED)**	**1/4/2023**	**2/1/2023**

Sample Type: **GEOTECHNICAL** Location: Station: **13+02.6** Offset, ft: **7.1** **LT** Dbfg, ft: **10.0-12.0**

WIN/Town **025449.00 - NEWCASTLE** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	**100.0**
No. 4 [4.75 mm]	**100.0**
No. 10 [2.00 mm]	**99.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**97.8**
[0.0221 mm]	**82.5**
[0.0149 mm]	**75.6**
[0.0092 mm]	**66.5**
[0.0068 mm]	**59.6**
[0.0051 mm]	**52.7**
[0.0026 mm]	**41.3**
[0.0012 mm]	**29.8**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**32**
Plastic Limit (T 90), %	**20**
Plasticity Index (T 90), %	**12**
Specific Gravity, Corrected to 20°C (T 100)	**2.78**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**34.8**

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.		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: **2/14/2023**

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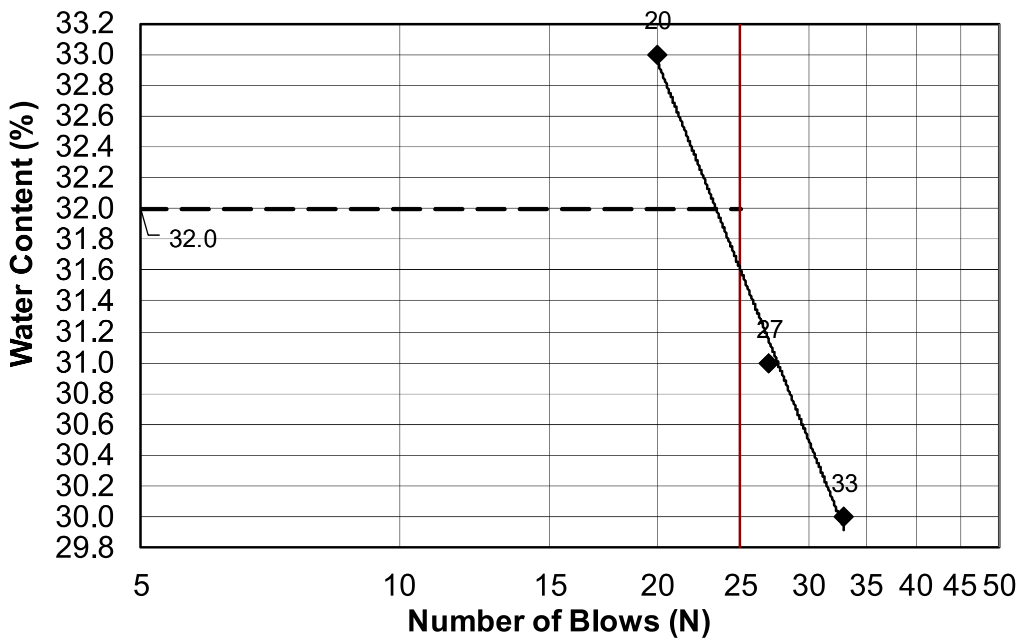
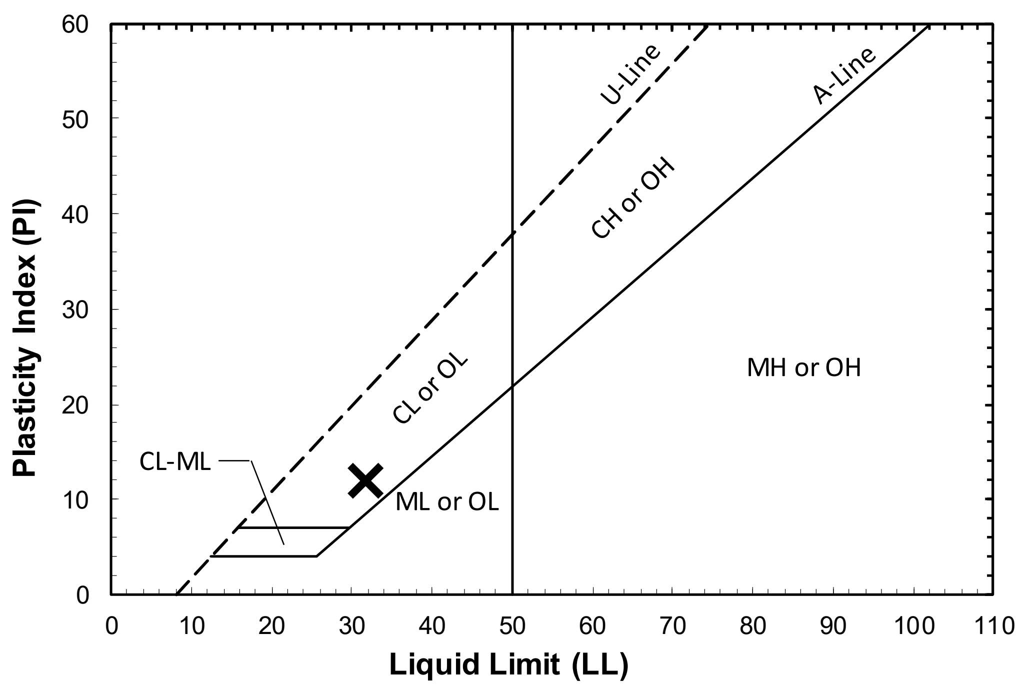
TOWN	Newcastle	Reference No.	380776
WIN	025449.00	Water Content, %	34.8
Sampled	1/4/2023	Liquid Limit @ 25 blows (T 89), %	32
Boring No./Sample No.	BB-NDMB-101/2D	Plastic Limit (T 90), %	20
Station	13+02.6	Plasticity Index (T 90), %	12
Depth	10.0-12.0	Tested By	TXLUO





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380777	BB-NDMB-101/3D	GEOTECHNICAL (DISTURBED)	1/4/2023	2/1/2023

Sample Type: **GEOTECHNICAL** Location: Station: **13+02.6** Offset, ft: **7.1 LT** Dbfg, ft: **15.0-17.0**

WIN/Town **025449.00 - NEWCASTLE** Sampler: **JAMES MANAHAN**

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]	**98.6**
½ in. [12.5 mm]	**96.8**
⅜ in. [9.5 mm]	**96.3**
¼ in. [6.3 mm]	**94.5**
No. 4 [4.75 mm]	**93.8**
No. 10 [2.00 mm]	**91.7**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**80.3**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**23.0**
[0.0323 mm]	**19.1**
[0.0208 mm]	**16.1**
[0.0123 mm]	**13.2**
[0.0088 mm]	**11.7**
[0.0063 mm]	**10.3**
[0.0033 mm]	**7.3**
[0.0014 mm]	**5.9**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	**NP**
Specific Gravity, Corrected to 20°C (T 100)	**2.68**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**20.9**

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: **2/14/2023**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380778	**BB-NDMB-102/1D**	**GEOTECHNICAL (DISTURBED)**	**1/1/2023**	**2/1/2023**

Sample Type: **GEOTECHNICAL** Location: Station: **13+23** Offset, ft: **9.6** **RT** Dbfg, ft: **5.0-7.0**

WIN/Town **025449.00 - NEWCASTLE** Sampler: **JAMES MANAHAN**

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]	**74.3**
½ in. [12.5 mm]	**58.7**
⅜ in. [9.5 mm]	**54.8**
¼ in. [6.3 mm]	**50.0**
No. 4 [4.75 mm]	**46.0**
No. 10 [2.00 mm]	**36.7**
No. 20 [0.850 mm]	**30.3**
No. 40 [0.425 mm]	**26.2**
No. 60 [0.250 mm]	**22.5**
No. 100 [0.150 mm]	**18.4**
No. 200 [0.075 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)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**8.9**

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:

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Reported by: **GREGORY LIDSTONE** Date Reported: **2/3/2023**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380780	**BB-NDMB-102/3D**	**GEOTECHNICAL (DISTURBED)**	**1/11/2023**	**2/1/2023**

Sample Type: **GEOTECHNICAL** Location: Station: **13+23** Offset, ft: **9.6** **RT** Dbfg, ft: **15.0-17.0**

WIN/Town **025449.00 - NEWCASTLE** Sampler: **JAMES MANAHAN**

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]	**74.2**
¾ in. [19.0 mm]	**64.4**
½ in. [12.5 mm]	**58.3**
⅜ in. [9.5 mm]	**55.7**
¼ in. [6.3 mm]	**52.3**
No. 4 [4.75 mm]	**50.1**
No. 10 [2.00 mm]	**46.9**
No. 20 [0.850 mm]	**44.1**
No. 40 [0.425 mm]	**38.9**
No. 60 [0.250 mm]	**28.7**
No. 100 [0.150 mm]	**17.6**
No. 200 [0.075 mm]	**8.7**

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), %	**11.7**

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:

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Reported by: **GREGORY LIDSTONE** Date Reported: **2/3/2023**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380779	BB-NDMB-102/MU1	GEOTECHNICAL (DISTURBED)	1/11/2023	2/1/2023

Sample Type: **GEOTECHNICAL** Location: Station: **13+23** Offset, ft: **9.6 RT** Dbfg, ft: **12.0-14.0**

WIN/Town **025449.00 - NEWCASTLE** Sampler: **JAMES MANAHAN**

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]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	100.0
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	99.7
No. 20 [0.850 mm]	98.4
No. 40 [0.425 mm]	87.4
No. 60 [0.250 mm]	59.6
No. 100 [0.150 mm]	30.9
No. 200 [0.075 mm]	12.7

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), %	21.4

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: **2/3/2023**

Paper Copy: Lab File; Project File; Geotech File

APPENDIX D – CALCULATIONS

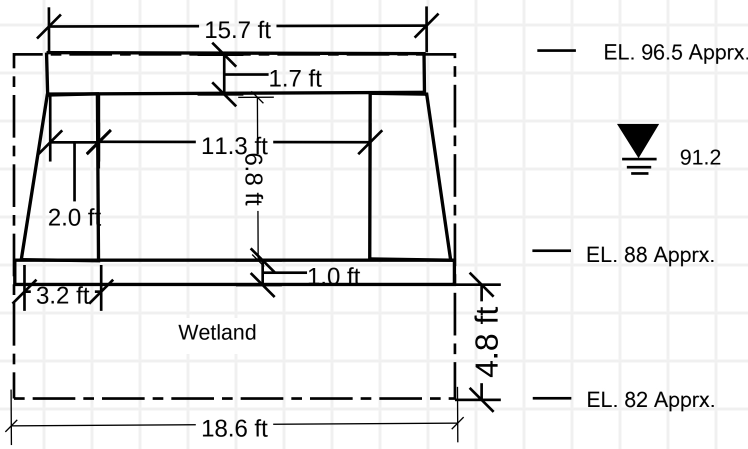
GZA GeoEnvironmental, Inc.
707 Sable Oaks Drive, Suite 150
South Portland, ME 04106
(207) 879-9190
http://www.gza.com

Engineers and Scientists

JOB Meadow Brook Bridge No. 2535
SHEET NO
Calculated By B.Cardali Date October 4, 2024
Checked By C.Snow Date October 4, 2024
Scale

Objective: Calculate the net pressure from the existing culvert and the proposed culvert at the elevation bearing stratum or the proposed culvert (top of the Marine Sand and Gravel [EL. 82']).

Existing Bridge Sketch and Pressure Calculation:



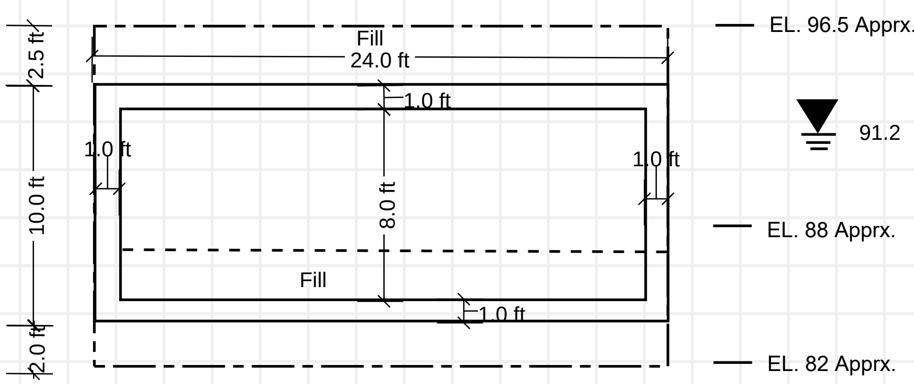
Area of open box culvert = 11.3' x 6.8' = 76.8 sf

Weight of Concrete = [[15.7*1.7] + [2 (6.8*(3.2+2.0/2)]+[18.6*1]*150 pcf]/ 18.6' = 650 psf

Weight of Soil below concrete pad = (4.8' x (120 pcf - 62.4pcf)) = 276.5 psf

Weight of Concrete and Soil = 650 psf + 276.5 psf = 926.5 psf

Proposed Box Culvert Sketch and Pressure Calculation:



Area of open box culvert = 8' x 22' = 176 sf

Weight of Concrete = [2 (23+9 x 1 x150 pcf]/24 feet = 400 psf

Weight of Fill above, Fill in, and Crushed Stone below box = (2.5' x 125 pcf) + (2' x (125 pcf - 62.4pcf)) + (2' x (130 pcf - 62.4pcf)) = 582 psf

Weight of Concrete and Soil = 400 psf + 582 psf = 982 psf

Conclusion:

The net pressure at the bearing stratum below the proposed culvert is slightly greater than the existing, however, it is anticipated that the compressible Wetland Deposit will be removed and the culvert will bear on 12 inches of bedding stone, underlain by approximately 12 inches of a fully encapsulated bedding stone, and over the Marine Sand and Gravel. Therefore, settlement is anticipated to be limited to elastic settlement of the cohesionless materials.

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

Engineers and Scientists

Objective

Calculate soil bearing resistance for a culvert bearing on naturally deposited marine sand and gravel. Evaluate strength and service bearing resistance in the marine sand and gravel.

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

$_f := 30\text{deg}$	Internal friction angle of cohesionless soil of the Marine Sand and Gravel
$_b := 0.45$	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
$:= 120\text{pcf}$	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
$N := 22.4$	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) $C_{wq} := 0.5$ $C_w := 0.5$
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
$S_c := 1\text{in}$	Allowable settlement
q_s=	Service limit bearing resistance for allowable settlement
$N_{60} := 20$	Average N_{60} values from SPT below footing
	Load inclination factors are omitted considering modest embedment of footing per C10.6.3.1.2a.

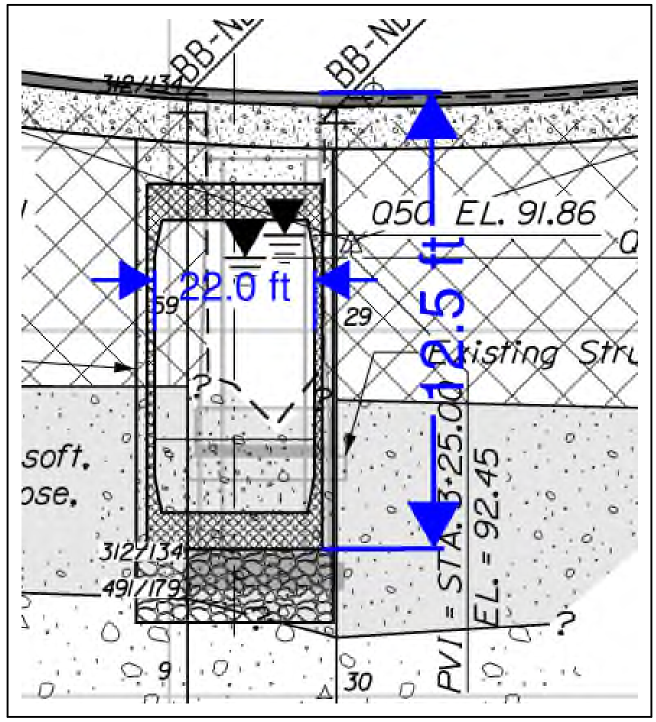
Footing Dimensions

$B_{1.max} := 24 \cdot ft$ Maximum Footing Width

$B_1 := 20 \cdot ft, 22 \cdot ft .. B_{1.max}$ Range of effective footing widths considered (includes eccentricity)

$L_1 := 66ft$ Length of culvert Base

$D_f := 12.5ft$ 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

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

Correction Factors

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

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

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

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

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

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

$s_c(B_1) =$
1.19
1.2
1.22

$s_q(B_1) =$
1.17
1.19
1.21

$s\ (B_1) =$
0.88
0.87
0.85

Bearing Capacity Factors

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

$N_{cm}(B_1) =$

35.68
36.23
36.79

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

$N_{qm}(B_1) =$

21.6
21.9
22.3

$N\ _m(B_1) := N\ \cdot s\ (B_1)$

$N\ _m(B_1) =$

19.7
19.4
19.1

Nominal Bearing Resistance

$q_n(B_1) := \overrightarrow{\left(c \cdot N_{cm}(B_1) + \ \cdot D_f \cdot N_{qm}(B_1) \cdot C_{wq} + 0.5 \cdot \ \cdot B_1 \cdot N\ _m(B_1) \cdot C_w\right)}$

$q_n(B_1) =$ ·ksf

28
29.3
30.5

Factored Bearing Resistance - Strength Limit State

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

$q_D(B_1) =$ ·ksf

12.6
13.2
13.7

for $B_1 =$ ·ft

20
22
24

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.

$S_{cm} := S_c \cdot \frac{1}{1mm}$ $S_{cm} = 25.4$ Allowable settlement in millimeters and unitless

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

$B_{1m}(B_1) =$

6.1
6.7
7.3

Correction formula for rectangular footings (Terzaghi EQ 50.14)

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

$S_{cmr}(B_1) =$

34
34
33

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

$EQ_1(B_1) =$

18.81
19.08
19.35

$EQ_2(B_1) =$

10.05
9.36
8.77

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

$q_{snc}(B_1) =$

189.1
178.5
169.6

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

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

$q_s(B_1) =$

189
179
170

Assumes Marine Sand and Gravel is normally consolidated at current effective stress

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

$q_{sm}(B_1) =$

189
179
170

$\cdot kPa$

Service limit bearing resistance for allowable settlement (metric units)

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

$q_{se}(B_1) =$

3.9
3.7
3.5

$\cdot ksf$

Service limit bearing resistance for allowable settlement (English units)

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

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

3.9
3.7
3.5

$m^{0.8} \cdot ksf$

$B_1 =$

20
22
24

$\cdot ft$

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

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.

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:

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

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

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

$:= 90\cdot\text{deg}$ — 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.

$K_o := 1 - \sin(\ ) = 0.47$ — 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:

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

For a sloped 2H:1V backfill:

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

$$K_{ar} = 0.46$$

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Figure 5-1 Maine Design Freezing Index Map

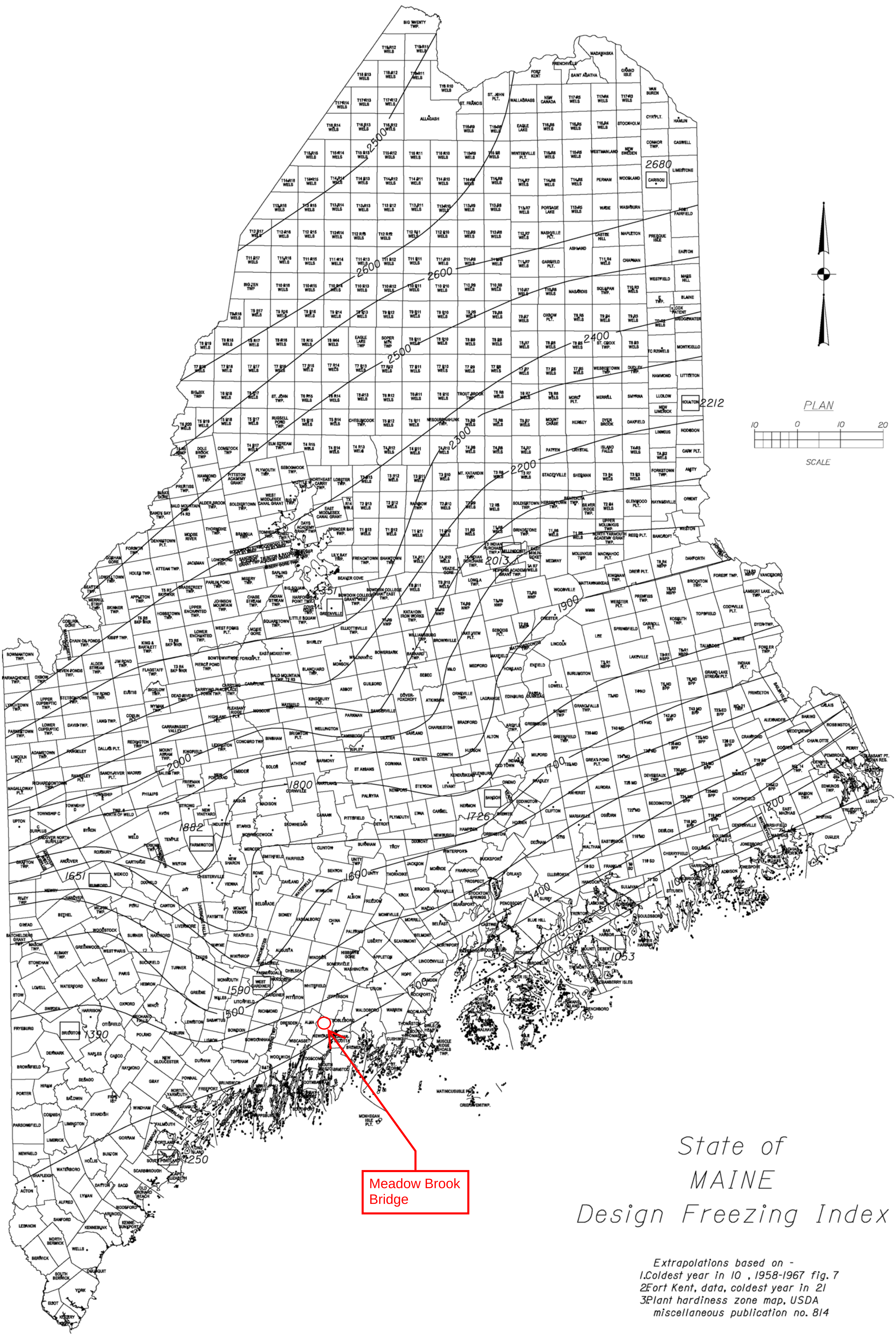

Table 5-1 Depth of Frost Penetration

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	[illegible]	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

1350

71" = 5.9'

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 10 to 20 percent. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing index of 1,350 the estimated depth of frost penetration is 71 inches or 5.9 feet.