www.haleyaldrich.com

REPORT ON

ROUTE 202 OVER CROCKER BROOK, BRIDGE NO. 5424
MAINEDOT WIN 26097.00
DIXMONT, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

for
Fuss & O'Neill
Manchester, New Hampshire

File No. 0207786-000
September 2024

HALEY & ALDRICH, INC.
75 Washington Avenue
Suite 1A
Portland, ME 04101
207.482.4600

20 September 2024
File No. 0207786-000

Fuss & O'Neill, Inc.
50 Commercial Street Unit 2S
Manchester, New Hampshire 03101

Attention: Shannon Beaumont, P.E.
Senior Project Manager

Subject: Geotechnical Design Report
Route 202 over Crocker Brook, Bridge No. 5424
MaineDOT WIN 26097.00
Dixmont, Maine

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit herewith our report entitled, "Geotechnical Design Report, Route 202 over Crocker Brook, Bridge No. 5424, MaineDOT WIN 26097.00, Dixmont, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our proposed work scope and fee estimate document, dated 30 January 2023, and executed by you on 11 May 2024.

Introduction

This GDR presents the results of our geotechnical field investigation and laboratory testing program, technical evaluations, and geotechnical design recommendations completed by Haley & Aldrich on behalf of Fuss & O'Neill for the proposed replacement bridge that will carry vehicular traffic on Route 202 over Crocker Brook in Dixmont, Maine (see Figure 1).

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

Plan locations of test borings (borings) are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 Central Zone. The project elevation datum and elevations referenced herein are in feet (ft) and reference the North American Vertical Datum of 1988 (NAVD 88).

The proposed limit of work extends from approximately Sta. 28+10 (west) to Sta. 29+70 (east).

PROJECT LOCATION, EXISTING SITE CONDITIONS, AND EXISTING BRIDGE STRUCTURE

The existing bridge carries eastbound (EB) and westbound (WB) vehicular traffic on Route 202 over Crocker Brook in Dixmont, Maine, approximately 1.1 miles west of Moosehead Trail Highway. The project site consists of a semi-wooded area. Existing roadway grades within the limit of work generally slope downwards from west to east and range from approximately El. 400 (west) to El. 394 (east). The bottom of Crocker Brook, which flows from south to north, within the limits of Route 202 is approximately El. 385.

Based on our review of the draft preliminary design report (PDR) plans for the subject project dated 11 July 2023, it is our understanding that the existing bridge consists of a single corrugated metal pipe arch culvert. The invert of the culvert is approximately El. 385. Historic bridge plans are included in Appendix C.

PROPOSED BRIDGE STRUCTURE

Based on our review of the draft PDR plans, it is our understanding that the bridge replacement alternative recommended by MaineDOT generally consists of the following:

- A 77-ft long x 24-ft wide x 8-ft tall precast concrete box culvert with a 2-ft thick layer of special fill along the culvert bottom. The culvert invert elevation (top of special fill) will vary between El. 384.5 and El. 385.3. The bottom of the culvert structure ranges from El. 381.2 to El. 382.0.
- A 24-in. thick roadway section that is 32-ft wide, which consists of two 11-ft wide travel lanes and two 5-ft wide shoulders.
- Horizontal and vertical alignments of Route 202 will roughly match existing site conditions and therefore, minimal raises in grade will be required (i.e., less than 6 in.).

Existing and proposed site conditions are shown on Figure 2.

Geologic Setting

According to the Geologic Map of Maine (1985), bedrock at the site is mapped as interbedded granofels and biotite schist, Silurian-Ordovician in age. Surficial deposits mapped within the site and nearby vicinity consist of sand and gravel stream deposits overlying glacial till. Surficial materials maps indicate overburden thickness to vary; generally 20 to 30 ft thick.

Geotechnical Field Investigation

Haley & Aldrich completed a final design phase (Phase II) geotechnical field investigation at the site between 30 May and 1 June 2023. Three borings, designated BB-DCB-101 through BB-DCB-103, were drilled at/near the existing bridge. Boring BB-DCB-101 encountered concrete from approximately 6 to 16 ft below ground surface (BGS), therefore an additional boring, BB-DCB-103, was conducted approximately 10 ft west of BB-DCB-101 to obtain additional soil information.

The boring locations were laid out in the field by Haley & Aldrich prior to the start of drilling. "As-drilled" boring locations and ground surface elevations were determined in the field by MaineDOT using GPS survey equipment upon the completion of drilling and were provided to Haley & Aldrich. The "as-drilled" boring locations and ground surface elevations are summarized in Table I and are shown on Figure 2.

The borings were drilled by New England Boring Contractors of Hermon, Maine, using a Mobile Drill B-53 track-mounted drill rig. The borings were advanced to depths ranging from approximately 17 to 31 ft BGS using cased-washed drilling methods and a combination of solid-stem augers and 4-in. (HW-size) inside diameter (ID) steel casing.

Soil samples were generally collected continuously by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the boring logs. The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the boring logs. The uncorrected SPT N-value (N-uncorrected) is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The drill rig was equipped with a calibrated automatic hammer per MaineDOT requirements. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.742; 72.4 percent theoretical hammer efficiency) divided by 0.6. The energy-corrected SPT N-values are also provided on the boring logs.

Soil samples recovered from boring BB-DCB-101 were screened in the field with a photoionization detector (PID) for the presence of volatile organic compounds (VOCs). The measured PID readings did not exceed 0.0 ppm and are shown on the boring logs.

Approximately 5 to 9 ft of bedrock was sampled (cored) in borings BB-DCB-101 and B-DCB-102. The bedrock was cored using a 2-in. (NQ-size) ID diamond-tipped core barrel.

Additionally, and per the request of Fuss & O'Neill, bulk samples were collected by hand from the streambed at ground surface both upstream and downstream of the culvert to aid in scour evaluations.

A Haley & Aldrich geologist was present on-site full time to monitor the drilling and to document the soil, bedrock, and groundwater conditions encountered and to prepare boring logs. Soil and bedrock samples were collected and preserved in plastic containers and wooden boxes, respectively, and are currently being stored in our storage facility in Portland. The soil and bedrock samples that were not submitted for laboratory testing are available for review upon request.

All drilling and sampling activities were performed in accordance with MaineDOT requirements.

Generalized Subsurface Conditions

The subsurface conditions encountered in the borings generally consist of man-placed fill soils (fill) overlying naturally-deposited glaciomarine soils, glacial till, and bedrock. Refer to Table II for a detailed summary of the soil units and thicknesses encountered in each boring. A general description of each soil/bedrock unit is provided separately, below. Detailed soil and bedrock descriptions are provided on the boring logs included Appendix A. Refer to the interpretive subsurface profile on Figure 2 for a graphical representation of the subsurface conditions present along the Route 202 centerline.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	9.5 to 15.8	Loose to dense, fine to coarse SAND with various amounts of silt and gravel. Approximately 9.7 ft of concrete was encountered and cored in boring BB-DCB-101. This may have been a historic box culvert that was present prior to construction of the current corrugated metal culvert, as shown on the historic drawings in Appendix C. *(encountered in all borings)*
Glaciomarine Deposit	4 to 5.5	Medium dense, fine to coarse SAND with various amounts of silt and gravel; medium dense, fine GRAVEL, little coarse sand, trace fine sand, trace silt; very stiff SILT, little fine gravel, little fine to medium sand, trace coarse sand. *(encountered in borings BB-DCB-102 and BB-DCB-103)*
Glacial Till	>1.8 to 4.8	Medium dense to very dense, fine to coarse GRAVEL with varying amounts of silt and sand; hard SILT, some fine gravel, little fine sand, trace medium to coarse sand, trace coarse gravel. *(encountered in all borings)*
Bedrock		Top of bedrock surface encountered at depths ranging from approximately 18 to 21 ft BGS (El. 377 to El. 378).

Please note that soil and bedrock descriptions provided on the boring logs and summarized above do not represent actual field conditions other than at the specific boring locations. The actual conditions will likely vary from those described herein.

BEDROCK CONDITIONS

As stated previously, approximately 5 to 9 ft of bedrock was sampled in the borings and the top of bedrock surface varied between approximately 18 to 21 ft BGS (El. 376.5 to El. 377.8).

The sampled and recovered bedrock generally consisted of moderately hard to hard, slightly weathered, fine-grained to aphanitic PHYLLITE and SCHIST. Primary joints dip at horizontal to high angles, very close to moderately spaced, and tight to open.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for bedrock encountered at the site ranged from 50 to 72 percent (average = 59 percent) indicating poor to fair bedrock quality.

Detailed bedrock core data and descriptions are provided on Table III and on the boring logs in Appendix A. In addition, photographs of the recovered bedrock core samples are provided for reference in Appendix A.

GROUNDWATER CONDITIONS

Groundwater observation wells were not installed in the borings. As a result, long-term static water levels at the site could not be determined. Because of this, the following general observations were made relative to groundwater conditions during or immediately after drilling:

- recovered soil samples from each boring were visually observed to be "wet" at approximately 10 to 11 ft BGS (El. 386 to El. 387).
- water levels measured in each boring during or after drilling was approximately 10 to 11 ft BGS (El. 386 to El. 387).
- In addition, current flood levels in Croker Brook, as reported in the draft PDR plans, are summarized below.

Discharge	Headwater Elevation (ft, NAVD 88)
$Q_{1.1}$	El. 387.3
Q_{25}	El. 390.1
Q_{50}	El. 390.6
Q_{100}	El. 391.1

Please note that the visual observations made during drilling and groundwater levels measured during or after drilling was completed may have been affected by drilling means/methods and may not be representative of actual static water levels at the site. In general, groundwater levels can be expected to fluctuate, subject to boring drilling means/methods, changing water levels in Crocker Brook, seasonal variation, local soil conditions, topography, and precipitation. Groundwater levels encountered during construction may differ from those observed in the borings.

Geotechnical Laboratory Testing Program

A geotechnical laboratory testing program was undertaken on disturbed soil samples collected during the geotechnical field investigation to assist in soil classification and determination of engineering soil properties. Geotechnical laboratory testing was performed by R.W. Gillespie in Biddeford, Maine. Geotechnical laboratory testing was performed in accordance with applicable ASTM International testing procedures. A summary of geotechnical laboratory test results is provided below.

Laboratory Test	ASTM Test Designation	Geologic Unit	No. of Tests	Range in Test Results[1]
Grain Size	ASTM D422	Streambed	2	AASHTO Classification: A-1-a USCS Classification: GW, GP Water Content: 5.5%<WC<6.0%
		Fill	2	AASHTO Classification: A-1-b, A-2-4 USCS Classification: SM Water Content: 4.9%<WC<13.1%
		Glaciomarine Deposit	1	AASHTO Classification: A-4 USCS Classification: SM Water Content: 14.3%
		Glacial Till	1	AASHTO Classification: A-4 USCS Classification: GM Water Content: 8.4%

Notes:
[1] WC = Water Content; NP = non plastic.

Laboratory test results are shown on the boring logs included in Appendix A and complete results are provided in Appendix B.

Geotechnical Design Recommendations

Geotechnical design recommendations, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design Bridge Design Specifications, 9th Edition, 2020, referred to herein as AASHTO LRFD.
- MaineDOT Bridge Design Guide, August 2003, with Interim Revisions through June 2018, referred to herein as the BDG.

Engineering calculations that support the design recommendations provided in this GDR are provided for reference in Appendix D.

ANTICIPATED SUBGRADE CONDITIONS AND BEDDING DETAIL

Based on the proposed bottom of culvert elevation in the draft PDR plans (El. 381.2 to El. 382.0), the thickness of "typical" bedding details (i.e.,1 ft; see below), and the subsurface conditions encountered in the borings, medium dense or hard to stiff glaciomarine and glacial till soils (i.e., sand and gravel with varying amounts of silt, silt with varying amounts of sand and gravel) are anticipated to be present at the proposed subgrade level (i.e., bottom of bedding/excavation). Construction of the box culvert and prepared subgrade may require removal of bedrock if the top of rock surface varies beyond the elevations recorded in the borings.

We recommend the following bedding detail:

- The culvert structure should be placed on a 1-ft thick layer of Granular Borrow, Material for Underwater Backfill (MaineDOT Item 203.25, Granular Borrow). Stabilization/Reinforcement Geotextile (MaineDOT Standard Specification 722.01) should be placed below and wrapped around the sides of the Granular Borrow. The bedding material should be placed in lifts of 6 to 8 in. loose measure and compacted to at least 95 percent of the AASHTO T-180 maximum dry density.

We recommend that all unsuitable material (if present) be over-excavated (removed) from within the zone of influence (ZOI) of the culvert prior to subgrade preparation and placement of the bedding material described above. The ZOI is defined as the area below the culvert and below imaginary lines that extend 1 ft laterally beyond the edge of the culvert and down on a one horizontal to one vertical (1H:1V) slope to the top of acceptable bearing material (glaciomarine or glacial till). If over-excavation is required, we recommend that the area be backfilled with Granular Borrow. Based on the subsurface conditions encountered in the borings, we do not anticipate that over-excavation will be required.

Subgrade preparation and protection and backfill placement and compaction recommendations are provided in the Construction Considerations section of this report.

BEARING RESISTANCE

Bearing resistance calculations were completed in accordance with AASHTO LRFD Section 10.6.3.1.2 for the culvert based on the preliminary culvert dimensions and the subgrade conditions summarized above (i.e., underdrain backfill material placed and compacted after the over-excavation of unsuitable material [if any]), and the subsurface conditions present at depth. Recommended bearing resistances for the culvert are as follows:

- The culvert should be evaluated at the Service Limit State not to exceed a presumptive bearing resistance equal to 6 kips per square foot (ksf) in accordance with AASHTO LRFD Table C10.6.2.6.1-1.
- Strength Limit State bearing resistance should not exceed 28.3 ksf, which is based on a resistance factor (ϕ_f) equal to 0.45 (AASHTO LRFD Table 10.5.5.2.2-1).

MODULUS OF SUBGRADE REACTION

The modulus of subgrade reaction relates foundation contact pressure to immediate (elastic) settlement and is typically used in soil-structure interaction models and to determine reinforcing requirements for mat foundations. In general, and as reported by DeSimone and Gould (1972), the design of mat foundations requires estimates not only of differential settlement resulting from long-term consolidation, but also the relatively rapid displacement due to load concentrations on the subgrade surface. This rapid displacement can produce large moments within the mat as compared to consolidation settlements and often controls mat design. The modulus of subgrade reaction is dependent on many factors, including the material properties and thickness of subgrade and foundation materials, geometry of the loaded area, and the stiffness and configuration of the structure.

We recommend that a modulus of subgrade reaction equal to 120 pounds per cubic inch (pci) be used to design the culvert.

SETTLEMENT

Based on our review of plan and profile drawings included in the draft PDR plans and as stated above, the horizontal and vertical alignments of Route 202 will roughly match existing site conditions and therefore, minimal raises in grade will be required (i.e., less than 6 in.). Because of this and because the soils present below the proposed subgrade level are primarily granular (i.e., not organic or cohesive soils that are subject to long-term consolidation settlement), we anticipate that any elastic settlement that does occur will be negligible and will likely occur rapidly during and immediately after culvert placement and backfilling operations are completed, prior to final paving. Because of this we anticipate that post-construction settlement of the culvert will be negligible.

LATERAL EARTH PRESSURE

Per the MaineDOT BDG, concrete box culverts are to be backfilled with a free-draining material (i.e., Soil Type 4, BDG Table 3-3). Box culvert walls should be designed using an at-rest earth pressure coefficient, K_0, of 0.47. Cantilevered cast in place concrete culvert inlet and outlet walls should be designed using an active earth pressure coefficient, K_a, which is dependent on the soil backslope angle at the inlet and outlet walls. For level backfill $K_a = 0.27$, and for a 2H:1V backslope $K_a = 0.46$. Refer to Appendix D for K_a values for other backslope angles. Passive earth pressure due to embedment of the box culvert should be neglected.

The culvert walls should be designed for a live load surcharge equivalent to the earthfill height summarized in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2. A uniform lateral load equal to the surcharge times the lateral earth pressure coefficient should be applied to the culvert walls to account for the live load surcharge.

SEISMIC DESIGN CONSIDERATIONS

In accordance with AASHTO LRFD Article 3.10.1 and MaineDOT BDG Section 5.2.5, seismic effects for culverts and buried structures do not need to be considered, unless they cross active faults. Based on our review of the Geologic Map of Maine (1985), active faults are not present at the site and therefore, seismic effects do not need to be considered.

FROST PROTECTION

The minimum depth of embedment/cover for foundation or other below-grade structures needed for frost protection was evaluated in accordance with the requirements of MaineDOT BDG Section 5.2.1 based on a design freezing index equal to 1,700 freezing degree days and the subgrade material type (granular), and the subgrade material water content data presented above. We recommend that the culvert bear a minimum of 7.3 ft below the lowest adjacent ground surface exposed to freezing. Riprap should not be considered as contributing to the overall thickness of soils required for frost protection.

Construction Considerations

The primary purpose of this section is to comment on geotechnical aspects of proposed construction. This section is written primarily for the individuals having responsibility for preparation of geotechnical-related plans and special provisions as well as personnel appointed to monitor construction activities (i.e., Resident Engineer and Inspectors). Prospective Contractors should evaluate the potential for construction problems on the basis of their own knowledge and experience in the area, and on the basis of similar projects in other localities, taking into account their proposed construction methods, procedures, equipment and personnel. Please note that the construction considerations provided below relate to this project only.

EXCAVATION

Excavation will be required to construct the culvert. Based on existing ground surface elevations, the proposed culvert invert elevation, and the anticipated subgrade conditions, excavation depths ranging between approximately 15 and 16 ft BGS will likely be required to construct the culvert. We anticipate that excavation and over-excavation (where and if required) of the in-situ soils (primarily fill and glaciomarine soils) can be accomplished using normal earth-excavating equipment. If bedrock is encountered within the culvert footprint, we anticipate that bedrock removal can be accomplished using mechanical means (excavation and/or hoe-ram) and conventional earth moving equipment. Bedrock was encountered approximately 2.5 to 4.5 ft below the anticipated bottom of excavation.

We anticipate that excavations may be made using sloped, open cut techniques. We recommended that the Contractor be responsible for the design, stability, and safety of all excavations in accordance with local, state, and federal regulations.

DEWATERING

Based on the current flood levels in Crocker Brook (El. 387.3) and the proposed subgrade level (El. 380.2 to El. 381.0), we anticipate that excavations will typically extend up to approximately 7 ft below the water level in Crocker Brook. Because of this, we anticipate that a continuous sheetpile cofferdam and temporary dewatering will be needed to ensure that the work is completed in the dry. We anticipate dewatering can be completed by open pumping using sumps from within the limits of the cofferdam. We recommend that the Contractor be made responsible for controlling all infiltration from groundwater and surface runoff to allow subgrade preparation, bedding material placement and compaction, and culvert construction and backfilling to be completed in-the-dry.

Excavation and control of water should be conducted using methods that prevent disturbance to subgrade soils. Sumps and pumps should be designed with proper filters to control the loss of fine-grained soils.

Dewatering and discharge of dewatering effluent should be performed in accordance with all applicable local, state, and federal regulations. Dewatering discharge should be recharged on-site, if possible. Sedimentation tanks and other treatment methods may be required for legal disposal of the effluent into Crocker Brook.

SUBGRADE PREPARATION

As discussed in previous sections of this GDR, we anticipate that soils present at the excavation subgrade will likely consist of medium dense or hard to stiff glaciomarine and glacial till soils (i.e., sand and gravel with varying amounts of silt, silt with varying amounts of sand and gravel). These soils are fined grained and can be disturbed by construction activities if care is not taken in excavating within a few feet of proposed subgrade levels and protecting the subgrade surfaces after preparation and prior to bedding material placement and compaction. The following guidelines are recommended to protect subgrade soils beneath the culvert:

- Make final excavations into natural bearing soils using smooth-bladed equipment to limit disturbance. We recommend the use of lightweight tracked grading equipment, such as low ground-pressure bulldozers, within 2 ft of subgrade elevation to the extent possible.
- Prevent water from accumulating on soil surfaces to reduce the possibility of soil disturbance. All filling should be performed in-the-dry. Subgrades that become disturbed due to water infiltration should be re-excavated and stabilized. Subgrade stabilization methods could include placement of a concrete mudmat or additional bedding material that is fully encapsulated in separation geotextile with approval of the Resident and/or Geotechnical Engineer.
- Exposed subgrades should be examined in the field by the Resident and/or Geotechnical Engineer to verify strength and bearing resistance. Excavation may be necessary to remove weak, disturbed, or otherwise unacceptable soils.
- All unsuitable material, if present as determined by the Resident and/or Geotechnical Engineer, shall be over-excavated (removed) prior to subgrade preparation and placement of fill/backfill.

- Granular subgrade surfaces could be proofrolled with self-propelled, static compaction equipment until firm and prior to placement of bedding material if the soil appears dry and no “free” water is observed as determined by the Resident and/or Geotechnical Engineer. To minimize disturbance, we recommend that wet/saturated granular or cohesive soils exposed at subgrade level not be proofrolled.
- Disturbance due to water and adverse weather could be reduced by maintaining excavations at least 12 in. above the final bearing level until immediately before placing fill material. Alternatively, it may be desirable to protect the exposed soil subgrade areas, as soon as possible after acceptance by the Resident and/or Geotechnical Engineer, by placing the culvert and backfill materials.
- Limit equipment traffic across the exposed soil bearing surfaces.
- If disturbance and rutting occur, the disturbed materials should be removed and replaced to the satisfaction of the Resident and/or Geotechnical Engineer.
- We recommend that the Contractor be made responsible for protecting subgrade surfaces. Any damage to the subgrade surface resulting from Contractor means and methods should be repaired to the satisfaction of the Resident and/or Geotechnical Engineer at no additional expense to MaineDOT.
- Due to the fined grained nature of the subgrade soils, we recommend that vibratory compaction equipment not be used to compact lifts of backfill within 5 ft (vertically) of the bottom of excavation.

Limitations

This GDR is prepared for the exclusive use of Fuss & O’Neill or MaineDOT relative to the subject project. There are no intended beneficiaries other than Fuss & O’Neill or MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the GDR. Use of this GDR by any person or entity other than Fuss & O’Neill or MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from Fuss & O’Neill and Haley & Aldrich. Use of this GDR by such other person or entity without the written authorization of Fuss & O’Neill and Haley & Aldrich shall be at such other person’s or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

Use of this GDR by any person or entity, including by Fuss & O’Neill or MaineDOT, for a purpose other than relative to the subject project is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the GDR is adequate for such other use. Use of this GDR by any other person or entity for such other purpose without written authorization by Haley & Aldrich shall be at such person’s or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

The information provided herein is based, in part, upon the data obtained from the referenced borings. The nature and extent of variations between the referenced borings may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this GDR.

It is our understanding that this GDR may be included as a reference document in the documents that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the documents and that the information contained in the documents takes precedence over the information provided in this GDR.

Closure

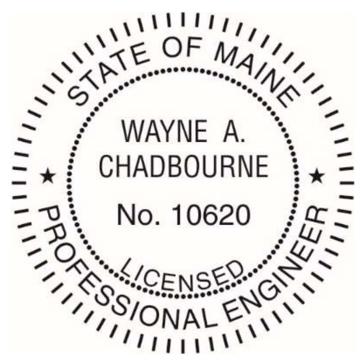
We appreciate the opportunity to provide geotechnical engineering services on this project. Please do not hesitate to contact the undersigned if have any questions or comments.

Sincerely yours,
HALEY & ALDRICH, INC.

Eric M. Hunstein, E.I.
Staff Geotechnical Engineer

Erin A. Force, P.E.
Project Manager

Wayne A. Chadbourne, P.E.
Principal/Lead Quality Control Engineer



Enclosures:

Table I – Subsurface Exploration Location Data
Table II – Subsurface Exploration Subsurface Data
Table III – Subsurface Exploration Bedrock Core Data
Figure 1 – Project Locus
Figure 2 – Exploration Location Plan and Interpretive Subsurface Profile
Appendix A – Test Boring Logs and Bedrock Core Photographs
Appendix B – Laboratory Test Results
Appendix C – Historic Bridge Drawings
Appendix D – Calculations

\\haleyaldrich.com\share\CF\Projects\0207786\000\Deliverables\2024-0920-HAI-Dixmont Culvert GDR-f1.docx

TABLES

TABLE I

Subsurface Exploration Location Data
Route 202 over Crocker Brook, Bridge No. 5424
MaineDOT WIN 26097.00
Dixmont, Maine

Haley & Aldrich, Inc. File No.: 0207786-000

Test Boring No.[1]	Ground Surface Elevation (ft)[2]	Station[3]	Offset Distance (ft) & Direction[3,4]
BB-DCB-101	397.1	128+81	9 RT
BB-DCB-102	395.9	129+05	8 LT
BB-DCB-103	397.7	128+70	9 RT

Notes:

[1] Test boring locations are shown on Figure 2, Exploration Location Plan and Interpretive Subsurface Profile.

[2] Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

[3] Station and offset information shown are approximate and are relative to the US Route 202 baseline and were determined by Haley & Aldrich based on survey information and plans provided by Fuss & O'Neill and rounded to the nearest ft.

[4] LT = offset distance toward left direction; RT = offset distance toward right direction.

	Individual	Date
Prepared By:	EMH	9/4/2024
Checked By:	TPJ	9/4/2024
Reviewed By:	EAF	9/4/2024

TABLE II
Subsurface Exploration Subsurface Data
Route 202 over Crocker Brook, Bridge No. 5424
MaineDOT WIN 26097.00
Dixmont, Maine

Haley & Aldrich, Inc. File No.: 0207786-000

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Stratigraphy Data[2,3]											Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration[2] (ft)
		Fill[4,5]			Glaciomarine Deposits			Glacial Till			Bedrock			
		Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)		
BB-DCB-101	397.1	0.0	397.1	15.8	NE	NE	NE	15.8	381.3	4.8	20.6	376.5	30.8	366.3
BB-DCB-102	395.9	0.0	395.9	9.5	9.5	386.4	5.5	15.0	380.9	3.1	18.1	377.8	23.5	372.4
BB-DCB-103	397.7	0.0	397.7	11.0	11.0	386.7	4.0	15.0	382.7	>1.8	NE	NE	16.8	380.9

Notes:
1 Test boring locations are shown on Figure 2, Exploration Location Plan and Interpretive Subsurface Profile.
2 Ground surface elevations atboring locations were determined in the field using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
3 "NE" indicates stratum was not encountered in test boring. ">1.8" indicates total thickness of stratum is greater than value shown
4 Bituminous concrete was encountered in the upper 1.1 ft of test borings BB-DCB-101 and BB-DCB-103 as well as the upper 1.5 ft of test boring BB-DCB-102.
5 Concrete was encountered in test boring BB-DCB-101 from 6.1 ft to 15.8 ft (El. 391.0 to 381.3)

	Individual	Date
Prepared By:	EMH	7/26/2023
Checked By:	MMB	8/11/2023
Reviewed By:	EAF	9/4/2024

TABLE III
Subsurface Exploration Bedrock Core Data
Route 202 over Crocker Brook, Bridge No. 5424
MaineDOT WIN 26097.00
Dixmont, Maine

Haley & Aldrich, Inc. File No.: 0207786-000

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Bedrock Core Diameter (in.)	Core Run[3]					Total Core Recovery[4]		Rock Quality Designation[5]	Physical Rock Parameters		Lithologic, Rock Mass, and Discontinuity Description
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (ft)	%	%	Weathering	Estimated Field Strength	
				Top	Bottom	Midpoint							
BB-DCB-101	397.1	NQ (2")	R3	21.2	25.8	23.5	4.6	4.6	100	56	Slightly	Moderately hard	Grey, fine-grained to aphanitic PHYLLITE. Joints horizontal to low angle, planar to undulating, close to moderate spacing, tight to open. Frequent 0.5 to 1-in. thick quartz veins. One 5-in. thick silt infilled joint. Single secondary moderately dipping joint. Oxidation on some joint surfaces.
			R4	25.8	30.4	28.1	4.6	4.5	98	50	Slightly	Moderately hard	Grey, fine-grained to aphanitic PHYLLITE. Joints at low to moderate angles, very close to close spacing, tight to open. Secondary high angle joints. Highly fractured zone from approximately 28.8 to 30.8 ft.
BB-DCB-102	395.9	NQ (2")	R1	18.5	23.5	21.0	5.0	5.0	100	72	Slightly	Hard	Grey, white, fine-grained SCHIST. Joints dipping at moderate to vertical angles, planar to undulating, close to moderate spacing, tight to open. Frequent quartz veins 0.25 to 0.5-in. thick.

Notes:
1 Test boring locations are shown on Figure 2, Exploration Location Plan and Interpretive Subsurface Profile.
2 Ground surface elevations at test boring locations were determined in the field using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
3 Cores R1 and R2 from boring BB-DCM-101 were concrete cores and not included in this table.
4 Total core recovery percentage is the length of core recovered divided by the length of the run.
5 RQD = rock quality designation. RQD is the total length of intact, full-diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of at least 4 in.) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.

	Individual	Date
Prepared By:	EMH	7/26/2023
Checked By:	MMB	8/11/2023
Reviewed By:	EAF	9/4/2024

FIGURES

SITE COORDINATES: 44°40'25"N, 69°11'05"W

N

MAP SOURCE: USGS

HALEY ALDRICH

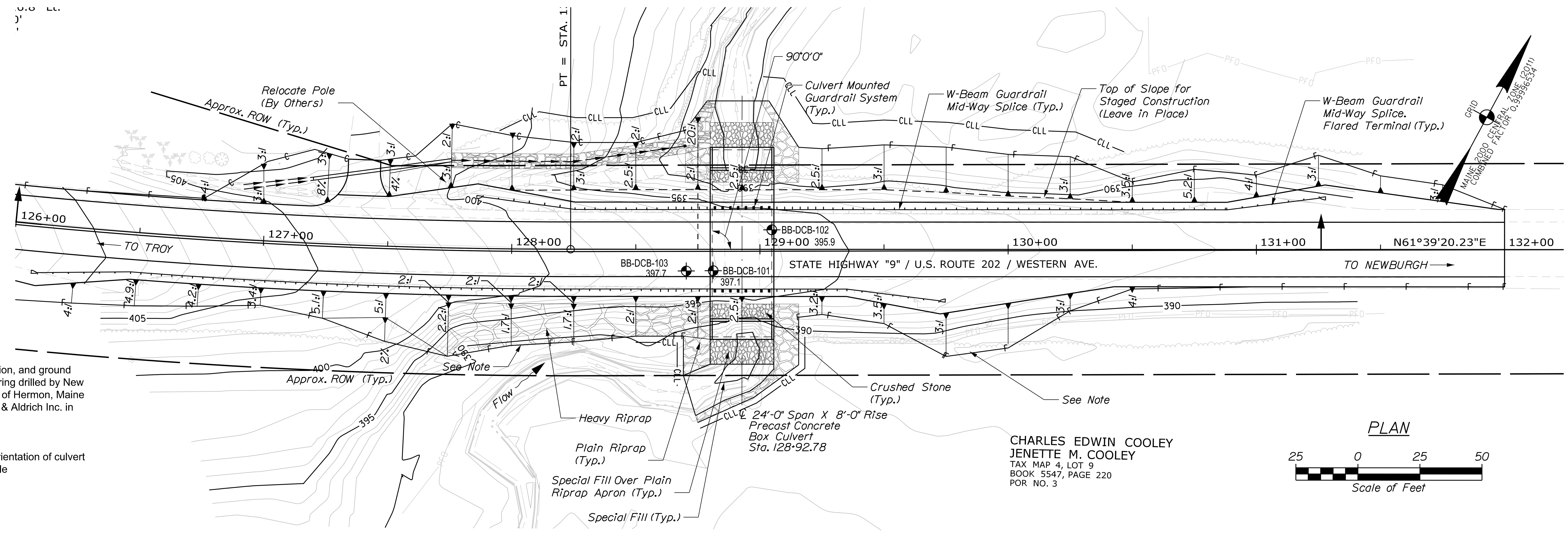
ROUTE 202 OVER CROCKER BROOK, BRIDGE NO. 5424
MAINEDOT WIN 26097.00
DIXMONT, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 3,000 FEET
SEPTEMBER 2024

FIGURE 1

0207786_000_LOCUS HALEYALDRICH\EHUNSTEIN



PLAN LEGEND:

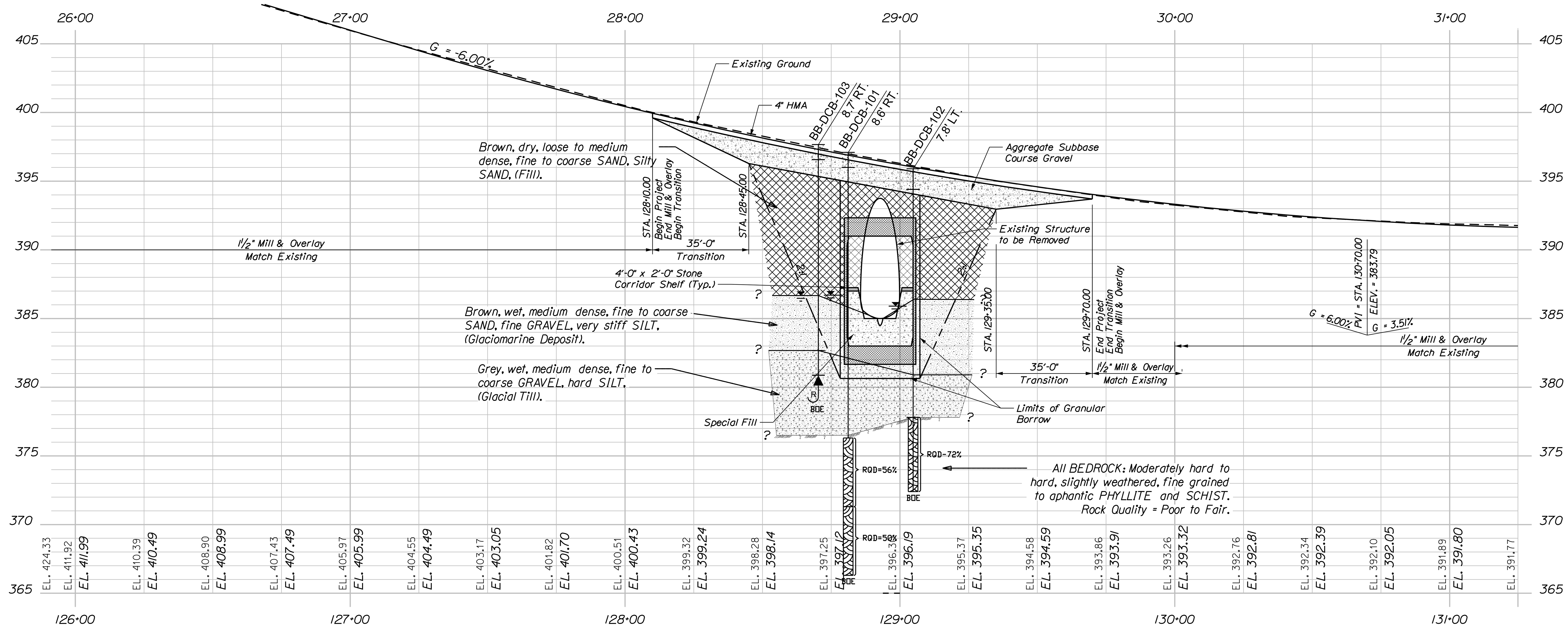
BB-DCB-101
397.1

Designation, as-drilled location, and ground surface elevation of test boring drilled by New England Boring Contractors of Hermon, Maine under the direction of Haley & Aldrich Inc. in May and June 2023

Approximate location and orientation of culvert interpretive subsurface profile

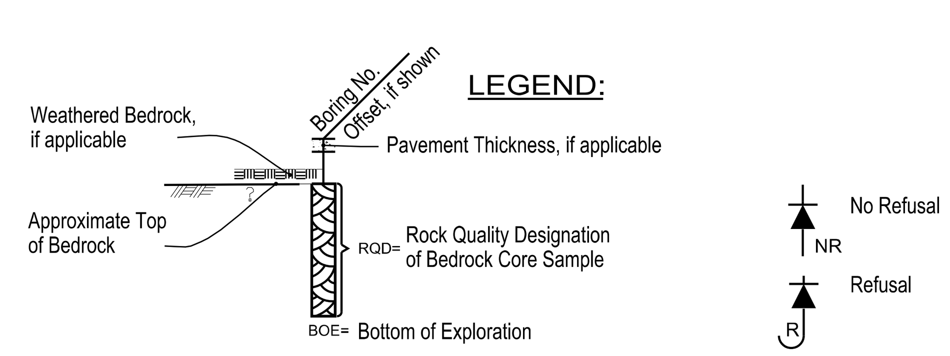
Key

BB = Bridge boring



INTERPRETIVE SUBSURFACE PROFILE

PROFILE
HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE



NOTES:

1. Exisitng and proposed site conditions, the location and orientation of existing site features, and the proposed strucutres are taken from electronic Microstation files provided by Fuss & O'Neill.
2. The plan locations of and ground surface elevations at test borings shown were determined upon the completion of drilling by the Maine Department of Transportation using GPS survey equipment.
3. Boring offset shown on the profile is based on the proposed Route 202 baseline.
4. The interpretive subsurface proifle is intended to convey generalized 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 rock transitions may vary and are likely to be more erratic than shown. For more specific information refer to the test boring logs in Appendix A.
5. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
6. Test borings were monitored in the field by a Haley & Aldrich, Inc. geologist.
7. Refer to the geotechnical design report for test boring logs and rock core photographs.

HALEY ALDRICH

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

26097.00

WIN 26097.00

BRIDGE PLANS

BRIDGE NO. 5424

PROJ. MANAGER	B. Nichols	BY	DATE
DESIGN-DETAILED	E. Hunstein	K. Postolowski	08/26/24
CHECKED-REVIEWED	E. Force	W. Chodbourne	09/18/24
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

CROCKER BROOK BRIDGE
CROCKER BROOK
DIXMONT PENOBSCOT COUNTY

EXPLORATION LOCATION PLAN AND INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 2

Filename: ...\0207786_002_PLAN AND PROFILE.dgnDivision:
Username:
Date:9/19/2024

APPENDIX A

Test Boring Logs and Bedrock Core Photographs

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

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))

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

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: Crocker Brook Bridge No. 5424 over Crocker Brook
Location: Route 202, Dixmont, Maine

Boring No.: BB-DCB-101
WIN: 026097.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	397.1	**Auger ID/OD:**	--
Operator:	G. McDougl	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile D53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	5-30-2023/5-30-2023	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	128+81, 9 ft RT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	10.4 ft
Hammer Efficiency Factor:	0.742	**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							NW			Bituminous Concrete	
	1D	23/13	1.1 - 3.0	15/12/19/10	31	38		396.0		1.1 Brown, dry, dense, fine to coarse SAND, trace gravel, trace silt (Fill).	
	2D	24/12	3.0 - 5.0	8/1/7/7	8	10				Similar to 1D, except loose (Fill).	
5	3D	7/2	5.0 - 5.6	6/50(1")				391.0		Brown, moist, medium dense, fine to coarse SAND, some fine gravel, trace silt (Fill). Note: Encountered refusal at 5.6 ft. Advanced rollerbit from 5.6 to 6.1 ft.	
										6.1 Concrete	
10	R1	60/58	10.2 - 15.2							Note: Cored from 10.2 to 15.2 ft with 58 in. recovery.	
15	R2	41/6	15.2 - 18.6					381.3		Note: Cored from 15.2 to 18.6 ft with 6 in. recovery.	
										15.8 Note: Driller noted bottom of concrete at 15.8 ft.	
	4D	24/5	18.6 - 20.6	4/5/12/25	17	21				Grey, wet, medium dense, fine to coarse GRAVEL, some fine to coarse sand, little silt (Glacial Till).	
20								376.5		20.6 Note: Encountered top of bedrock at 20.6 ft. Advance rollerbit from 20.6 to 21. 2 ft. Begin NQ core at 21.2 ft. Top of Bedrock at El. 376.5.	
	R3	55/55	21.2 - 25.8	RQD = 56%			NQ CORE			R3:Grey, fine-grained to aphanitic PHYLLITE. Moderately hard slightly weathered. Joints horizontal to low angle, planar to undulating, close to moderate, tight to open, frequent 0.5 to 1-in. thick quartz veins. One 5-in. thick silt infilled joint. Single secondary moderately dipping joint. Oxidation on some joint surfaces.	
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: Crocker Brook Bridge No. 5424 over Crocker Brook
Location: Route 202, Dixmont, Maine

Boring No.: BB-DCB-101
WIN: 026097.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	397.1	**Auger ID/OD:**	--
Operator:	G. McDougl	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile D53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	5-30-2023/5-30-2023	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	128+81, 9 ft RT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	10.4 ft
Hammer Efficiency Factor:	0.742	**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	R4	55/54	25.8 - 30.4	RQD = 50%			NQ CORE			Rock Quality=Fair Recovery=100% R3 Core Times (min:sec): 21.2-22.2' (2:34); 22.2-23.2' (2:50); 23.2-24.2' (3:15); 24.2-25.2' (2:14); 25.2-25.8' (3:10) Note: Begin R4 at 25.8 ft. Core bit broke off in borehole, unable to retrieve bit. R4: Similar to R3, except joints dipping at low to moderate angles, very close to close. Secondary high angle joints. Highly fractured zone from approximately 28.8 to 30.8 ft. Rock Quality=Poor Recovery=90% R2 Core Times (min:sec): 25.8-26.8' (6:31); 26.8-27.8' (6:37); 27.8-28.8' (6:31); 28.8-29.8' (6:52); 29.8-30.8' (7:00)	
30								366.3		30.8	
										Bottom of Exploration at 30.8 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: Crocker Brook Bridge No. 5424 over Crocker Brook
Location: Route 202, Dixmont, Maine

Boring No.: BB-DCB-102
WIN: 026097.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	395.9	**Auger ID/OD:**	--
Operator:	G. McDougl	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile D53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	5-31-2023/5-31-2023	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	129+05, 8 ft LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	10.0 ft
Hammer Efficiency Factor:	0.742	**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							NW			Bituminous Concrete	
	1D	24/14	1.5 - 3.5	7/8/9/7	17	21		394.4		1.5 Brown, dry, medium dense, fine SAND, little medium to coarse sand, little gravel, trace silt (Fill).	
	2D	24/15	3.5 - 5.5	7/7/6/9	13	16				Brown, dry, medium dense, fine SAND, little gravel, trace medium to coarse sand, trace silt (Fill).	
5	3D	24/15	5.5 - 7.5	5/4/3/4	7	9				Brown, dry, loose, fine to coarse SAND, little gravel, trace silt (Fill).	
	4D	24/12	7.5 - 9.5	4/5/4/2	9	11				Brown, dry, medium dense, fine to coarse SAND, some gravel, little silt (Fill).	G#17569-03 A-1-b, SM
10	5D	24/3	9.5 - 11.5	5/8/8/9	16	20		386.4		9.5 Brown, wet, medium dense, medium to coarse SAND, some gravel, trace fine sand, trace silt, poorly-graded (Glaciomarine Deposit).	
	6D	24/13	11.5 - 13.5	8/6/7/9	13	16		384.4		11.5 Brown, grey, wet, medium dense, fine SAND, some gravel, some silt, trace medium to coarse sand (Glaciomarine Deposit).	
15	7D	24/11	15.0 - 17.0	7/10/24/30	34	42		380.9		15.0 Grey, wet, hard, SILT, some fine gravel, little fine sand, trace medium to coarse sand, trace coarse gravel, moderately bonded (Glacial Till).	G#17569-04 A-4(0), GM
20	R1	60/60	18.5 - 23.5	RQD = 72%			NQ CORE	377.8		18.1 Note: Encountered top of bedrock at 18.1 ft. Advance rollerbit to 18.5 ft. Begin NQ core at 18.5 ft. Top of Bedrock El. 377.8. R1: Grey, white, fine-grained SCHIST. Hard, slightly weathered. Joints dipping at moderate to vertical angles, planar to undulating, close to moderate, tight to open. Frequent quartz veins 0.25 to 0.5-in. thick. Rock Quality=Fair Recovery=100% R1 Core Times (min:sec): 18.5-19.5' (4:10); 19.5-20.5' (6:30); 20.5-21.5' (8:15); 21.5-22.5' (7:40); 22.5-23.5' (5:04)	
								372.4		23.5 **Bottom of Exploration at 23.5 feet below ground surface.**	
25											

Remarks:

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

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

Boring No.: BB-DCB-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crocker Brook Bridge No. 5424 over Crocker Brook
Location: Route 202, Dixmont, Maine

Boring No.: BB-DCB-103
WIN: 026097.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	397.7	**Auger ID/OD:**	--
Operator:	G. McDougl	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile D53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	5-31-2023/6-01-2023	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	--
Boring Location:	128+70, 9 ft RT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	11.0 ft
Hammer Efficiency Factor:	0.742	**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							NW			Bituminous	
								396.6		1.1	
	1D	23/15	1.1 - 3.0	7/10/10/9	20	25				Brown, dry, medium dense, fine SAND, some medium to coarse sand, little silt, trace gravel (Fill).	
	2D	24/15	3.0 - 5.0	9/6/7/7	13	16				Similar to 1D (Fill).	
5	3D	24/14	5.0 - 7.0	4/4/2/2	6	7				Brown, grey, moist, loose, fine to medium SAND, some silt, little fine gravel (Fill).	G#17569-05 A-2-4(0), SM
	4D	24/8	7.0 - 9.0	9/7/5/5	12	15				Brown, moist, medium dense, Silty fine SAND, trace fine gravel (Fill).	
								388.7		9.0	
	5D	24/13	9.0 - 11.0	11/23/25/12	48	59				Brown, dry, very dense, fine to coarse SAND, some gravel, trace silt, well graded (Fill).	
10											
								386.7		11.0	
	6D	24/1	11.0 - 13.0	11/6/13/22	19	23				Dark grey, wet, medium dense, fine GRAVEL, little coarse sand, trace fine sand, trace silt, poorly-graded (piece of broken rock) (Glaciomarine Deposit).	
								384.7		13.0	
	7D	24/10	13.0 - 15.0	18/12/6/9	18	22				Brown, wet, very stiff, SILT, little fine gravel, little fine to medium sand, trace coarse sand (Glaciomarine Deposit).	G#17569-06 A-4(0), SM
15								382.7		15.0	
	8D	22/6	15.0 - 16.8	37/24/23/50(3")	47	58				Dark grey-brown, wet, very dense, fine to coarse GRAVEL, little fine to coarse sand, trace silt, poorly-graded (Glacal Till).	
								380.9		16.8	
										Bottom of Exploration at 16.8 feet below ground surface.	
20											
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.

BEDROCK CORE PHOTOGRAPHS
ROUTE 202 OVER CROCKER BROOK BRIDGE NO. 5424
MAINEDOT WIN 26097.00
DIXMONT, MAINE
FILE NO. 0207786-000

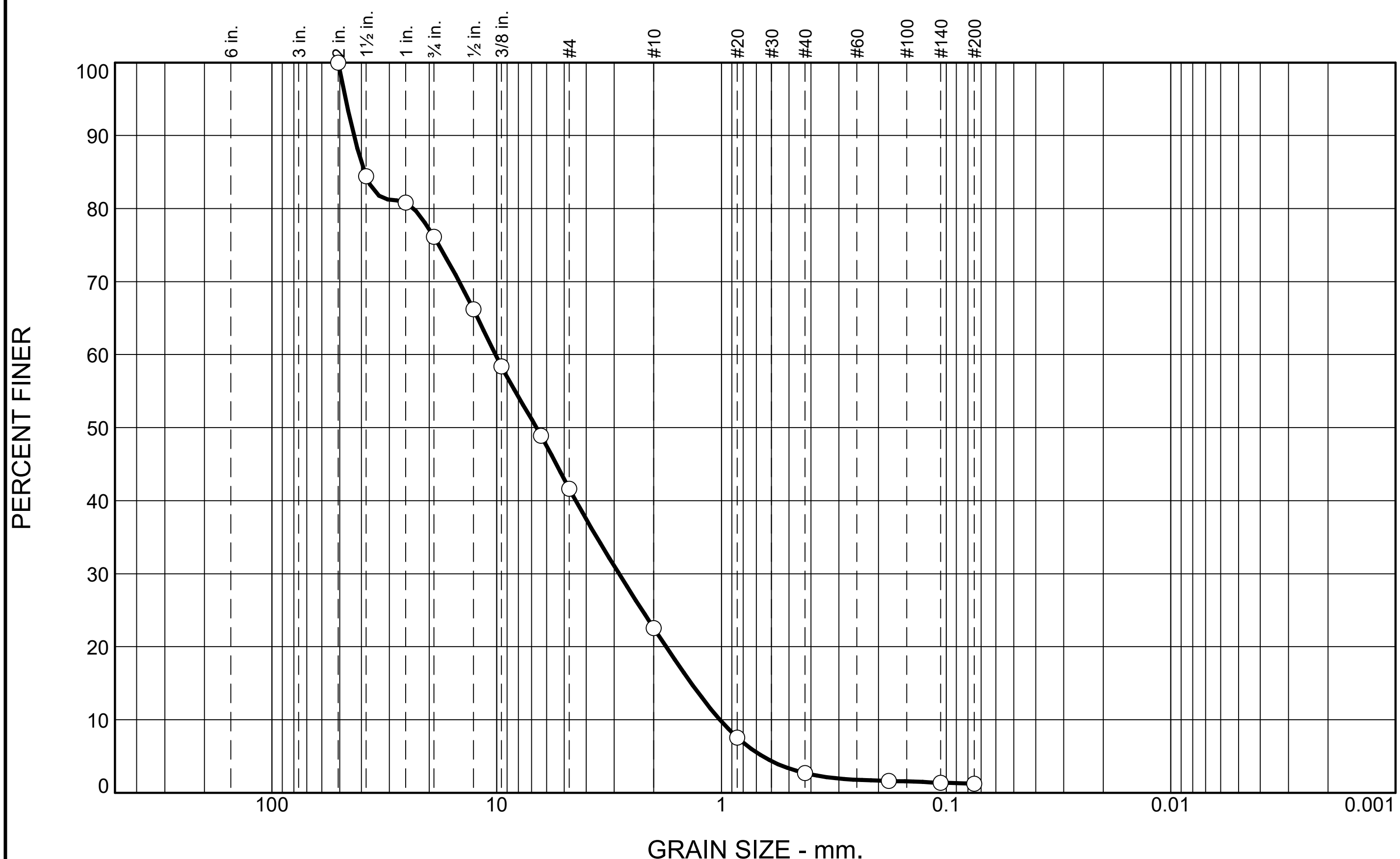
Top Row: BB-DCB-101, Run No. R1 10.2 (left) to 15.2 (right)
Top Middle Row: BB-DCB-101, Run No. R2 15.2 (left) to 18.6 (center left); Run No. R3 21.2 (center left) to 25.8 (right)
Bottom Middle Row: BB-DCB-101, Run No. R3 (continued) 21.2 (left) to 25.8 (center left); Run No. R4 25.8 (center left) to 30.8 (right)
Bottom Row: BB-DCB-102, Run No. R1 18.5 (left) to 23.5 (right)

APPENDIX B

Laboratory Test Results

HALEY ALDRICH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	23.9	34.5	19.0	19.9	1.5	1.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/2"	84.4		
1"	80.8		
3/4"	76.1		
1/2"	66.2		
3/8"	58.4		
1/4"	48.9		
#4	41.6		
#10	22.6		
#20	7.6		
#40	2.7		
#80	1.6		
#140	1.4		
#200	1.2		

* (no specification provided)

Soil Description

poorly graded gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 43.1636 D_{85}= 38.7310 D_{60}= 10.1275
D_{50}= 6.6588 D_{30}= 2.8584 D_{15}= 1.3635
D_{10}= 1.0157 C_u= 9.97 C_c= 0.79

Classification

USCS= GP AASHTO= A-1-a

Remarks

Moisture Content: 6.0%

Location: 20' Downstream
Sample Number: Bulk

Date: 07/11/2023

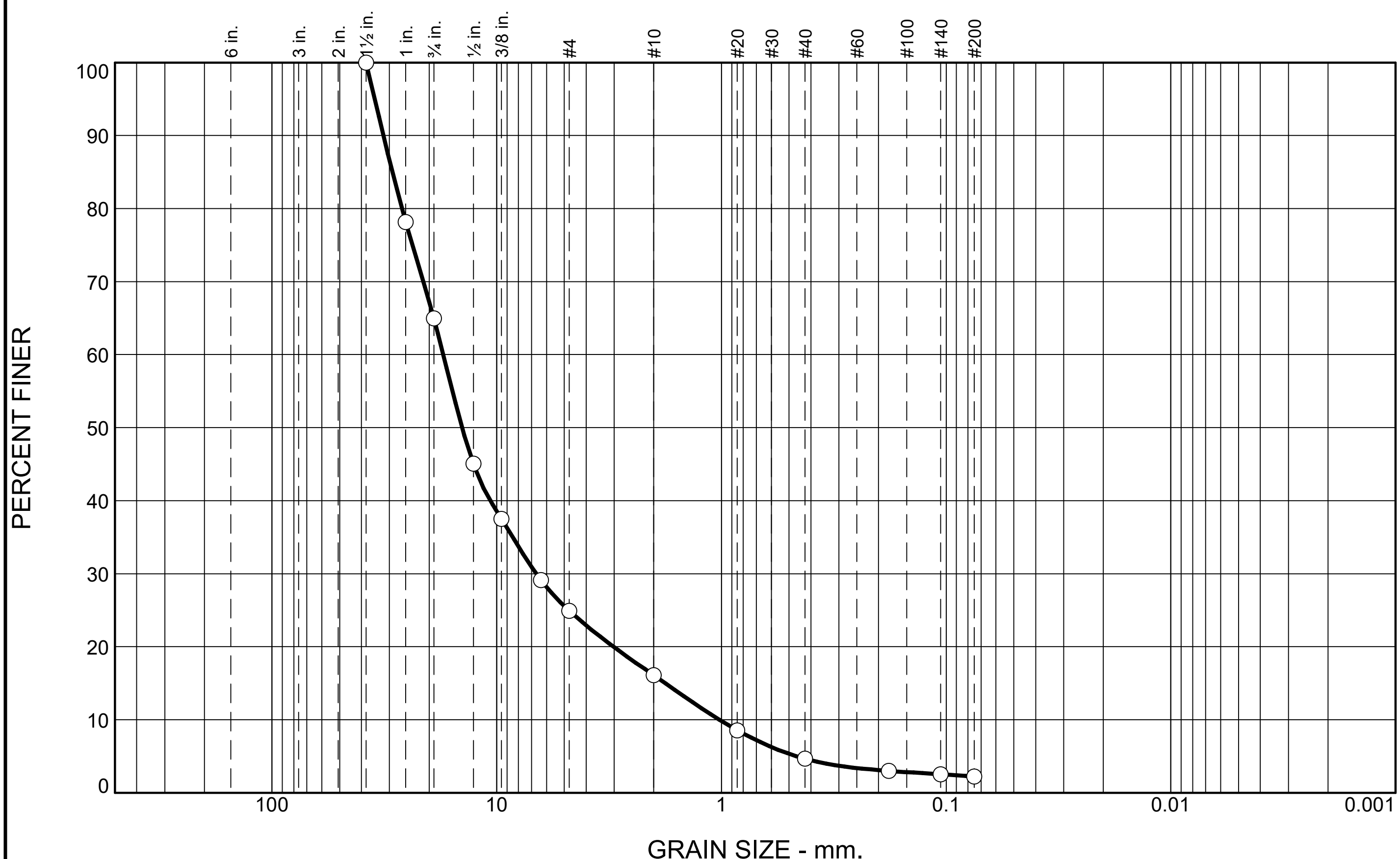
R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Crooker Brooke (No 207786)
Dixmont, ME
Project No: 0956-013BG12 **Lab No.** 17569-01

Tested By: CAG/SJV **Checked By:** MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	35.0	40.1	8.8	11.4	2.5	2.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	78.2		
3/4"	65.0		
1/2"	45.0		
3/8"	37.5		
1/4"	29.1		
#4	24.9		
#10	16.1		
#20	8.6		
#40	4.7		
#80	3.0		
#140	2.5		
#200	2.2		

* (no specification provided)

Soil Description

well-graded gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 31.9076 D_{85}= 29.0891 D_{60}= 17.2916
D_{50}= 14.2523 D_{30}= 6.6706 D_{15}= 1.7802
D_{10}= 1.0217 C_u= 16.92 C_c= 2.52

Classification

USCS= GW AASHTO= A-1-a

Remarks

Moisture Content: 5.5%

Location: 30' Upstream
Sample Number: Bulk

Date: 07/11/2023

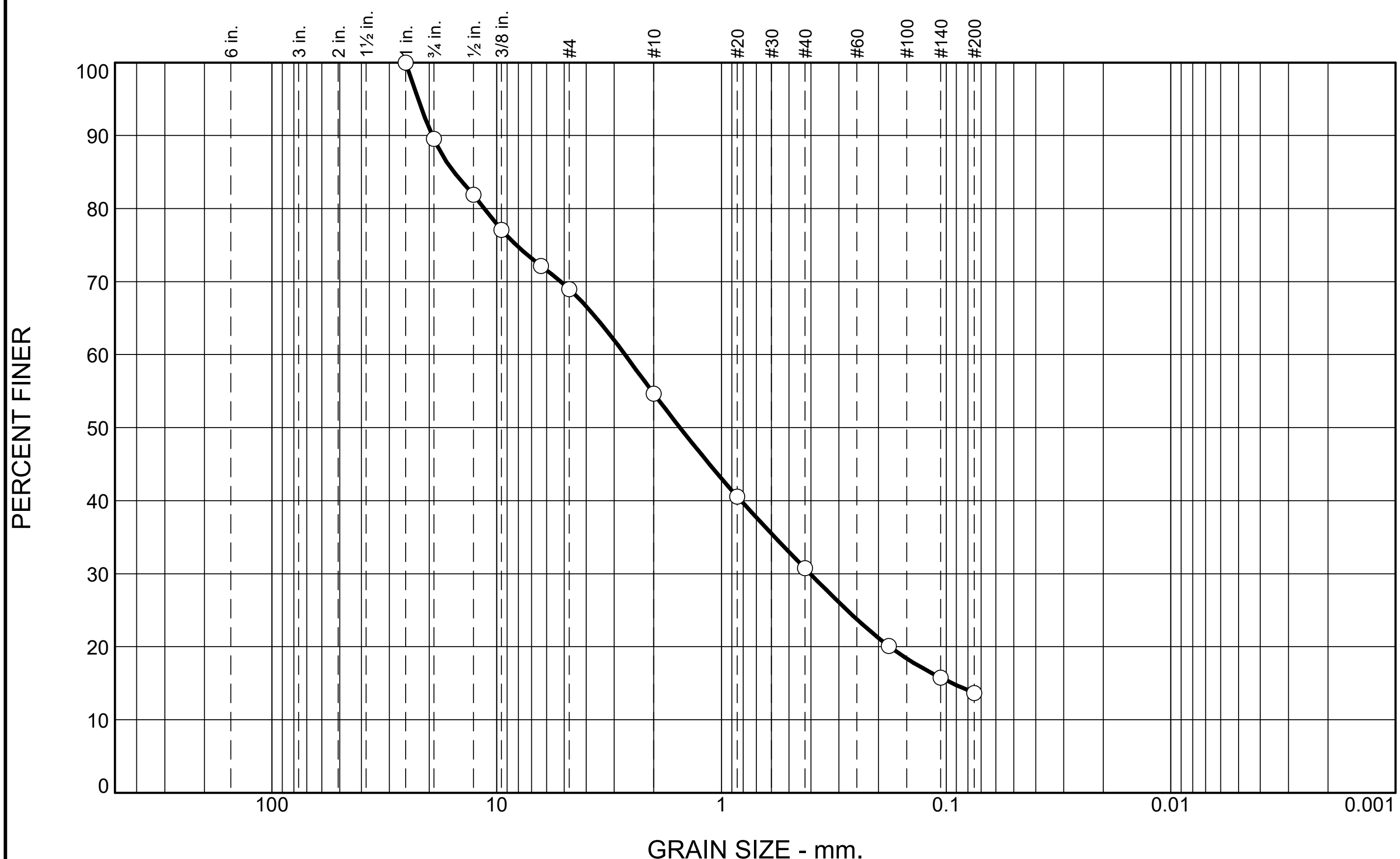
R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Crooker Brooke (No 207786)
Dixmont, ME
Project No: 0956-013BG12 **Lab No.** 17569-02

Tested By: CAG/SJV **Checked By:** MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.4	20.6	14.3	24.0	17.0	13.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	89.6		
1/2"	81.9		
3/8"	77.1		
1/4"	72.1		
#4	69.0		
#10	54.7		
#20	40.6		
#40	30.7		
#80	20.1		
#140	15.8		
#200	13.7		

* (no specification provided)

Soil Description

silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 19.3418 D_{85}= 15.4836 D_{60}= 2.6877
D_{50}= 1.5314 D_{30}= 0.4026 D_{15}= 0.0939
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Remarks

Moisture Content: 4.9%

Location: BB-DCB-102
Sample Number: 4D **Depth:** 7.5-9.5' **Date:** 07/11/2023

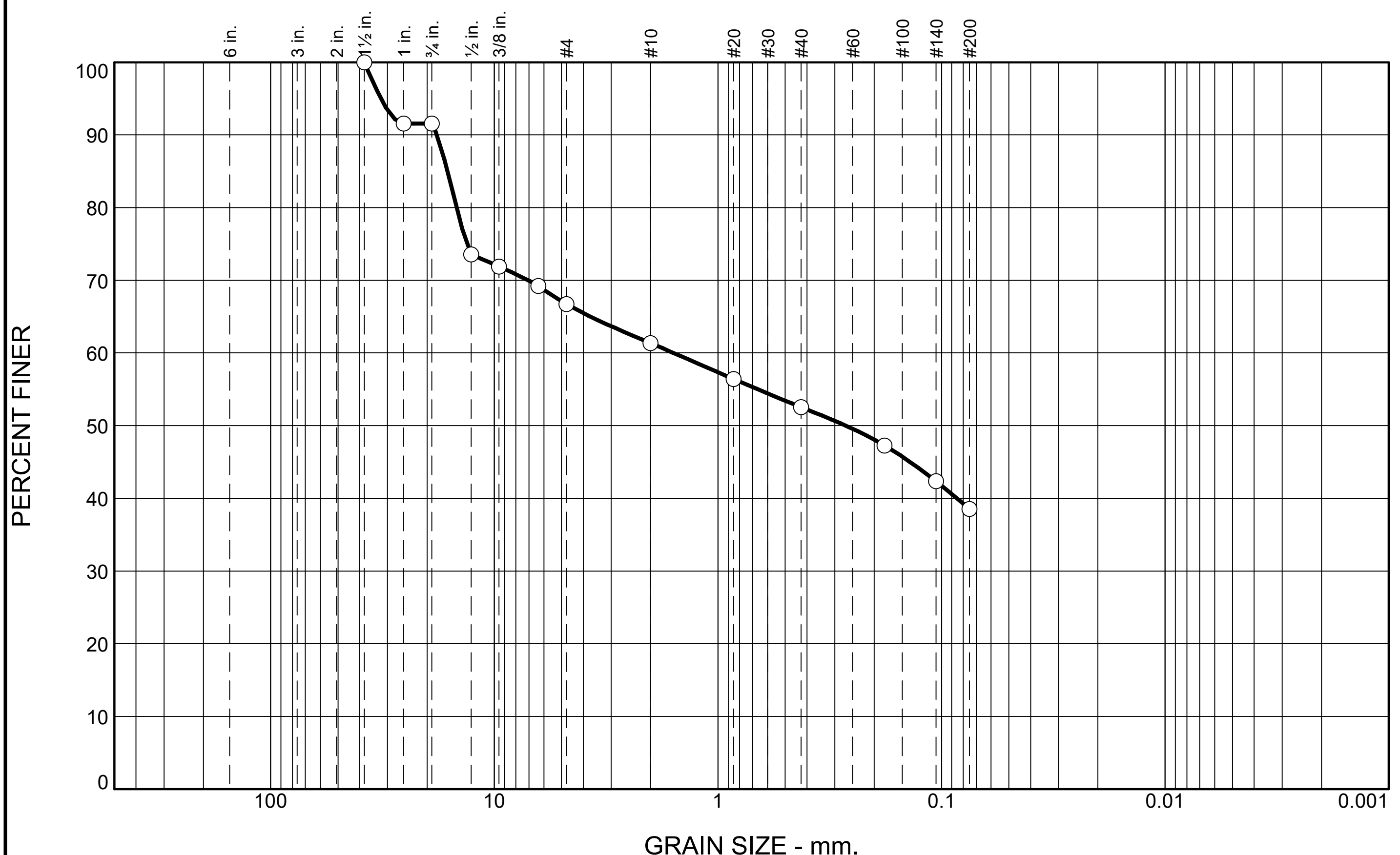
R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Crooker Brooke (No 207786)
Dixmont, ME
Project No: 0956-013BG12 **Lab No.** 17569-03

Tested By: CAG/SJV **Checked By:** MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	8.5	24.8	5.3	8.8	14.1	38.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	91.5		
3/4"	91.5		
1/2"	73.6		
3/8"	71.9		
1/4"	69.2		
#4	66.7		
#10	61.4		
#20	56.4		
#40	52.6		
#80	47.3		
#140	42.3		
#200	38.5		

* (no specification provided)

Soil Description

silty gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 18.0613 D_{85}= 16.1756 D_{60}= 1.5791
D_{50}= 0.2683 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= GM AASHTO= A-4(0)

Remarks

Moisture Content: 8.4%

Location: BB-DCB-102
Sample Number: 7D **Depth:** 15.0-17.0' **Date:** 07/11/2023

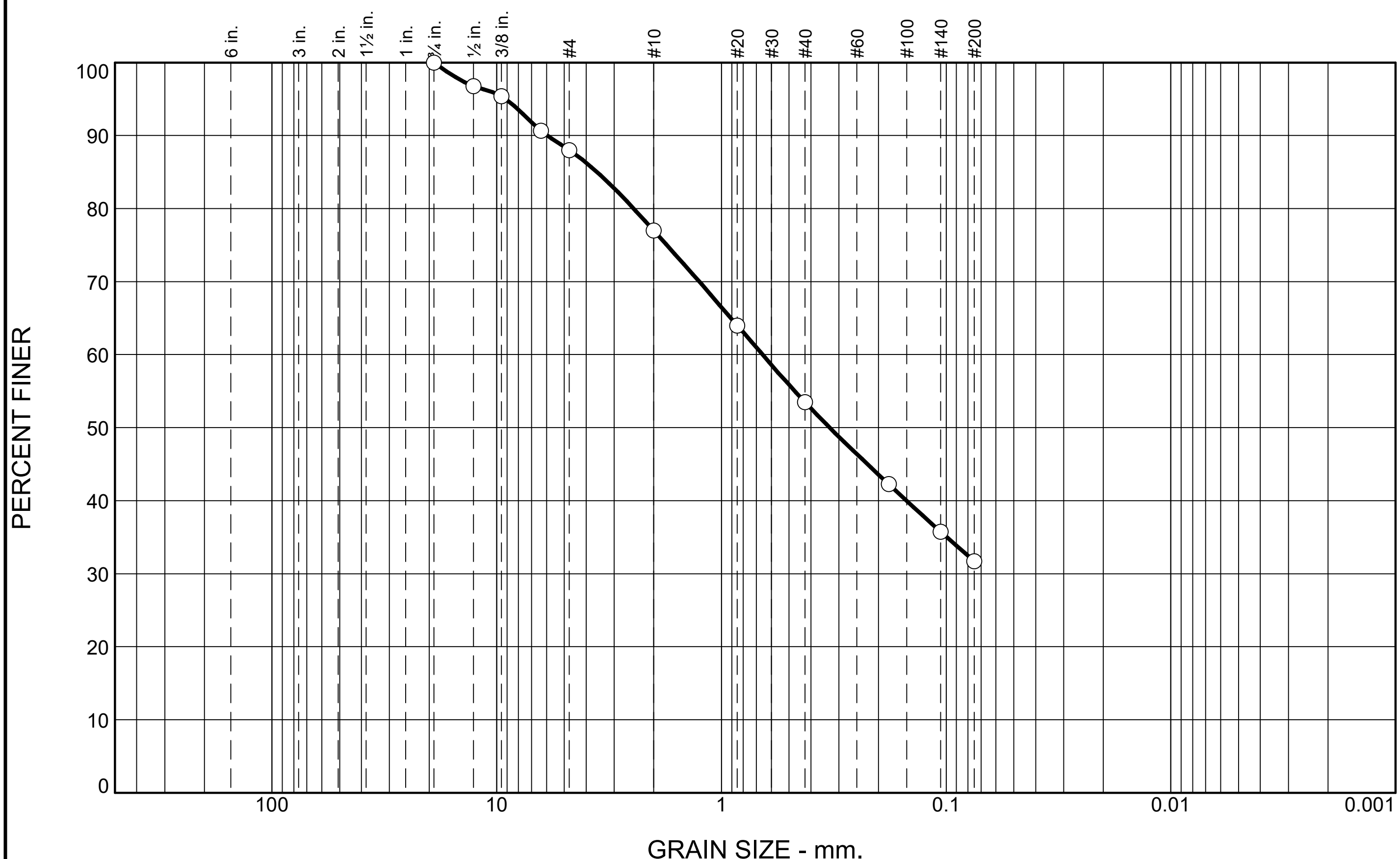
R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Crooker Brooke (No 207786)
Dixmont, ME
Project No: 0956-013BG12 **Lab No.** 17569-04

Tested By: CAG/FMG **Checked By:** MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	12.0	11.0	23.5	21.8	31.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	96.8		
3/8"	95.4		
1/4"	90.7		
#4	88.0		
#10	77.0		
#20	64.0		
#40	53.5		
#80	42.3		
#140	35.8		
#200	31.7		

* (no specification provided)

Soil Description

silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 5.9553 D_{85}= 3.5793 D_{60}= 0.6568
D_{50}= 0.3298 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Moisture Content: 13.1%

Location: BB-DCB-103
Sample Number: 3D **Depth:** 5.0-7.0' **Date:** 07/11/2023

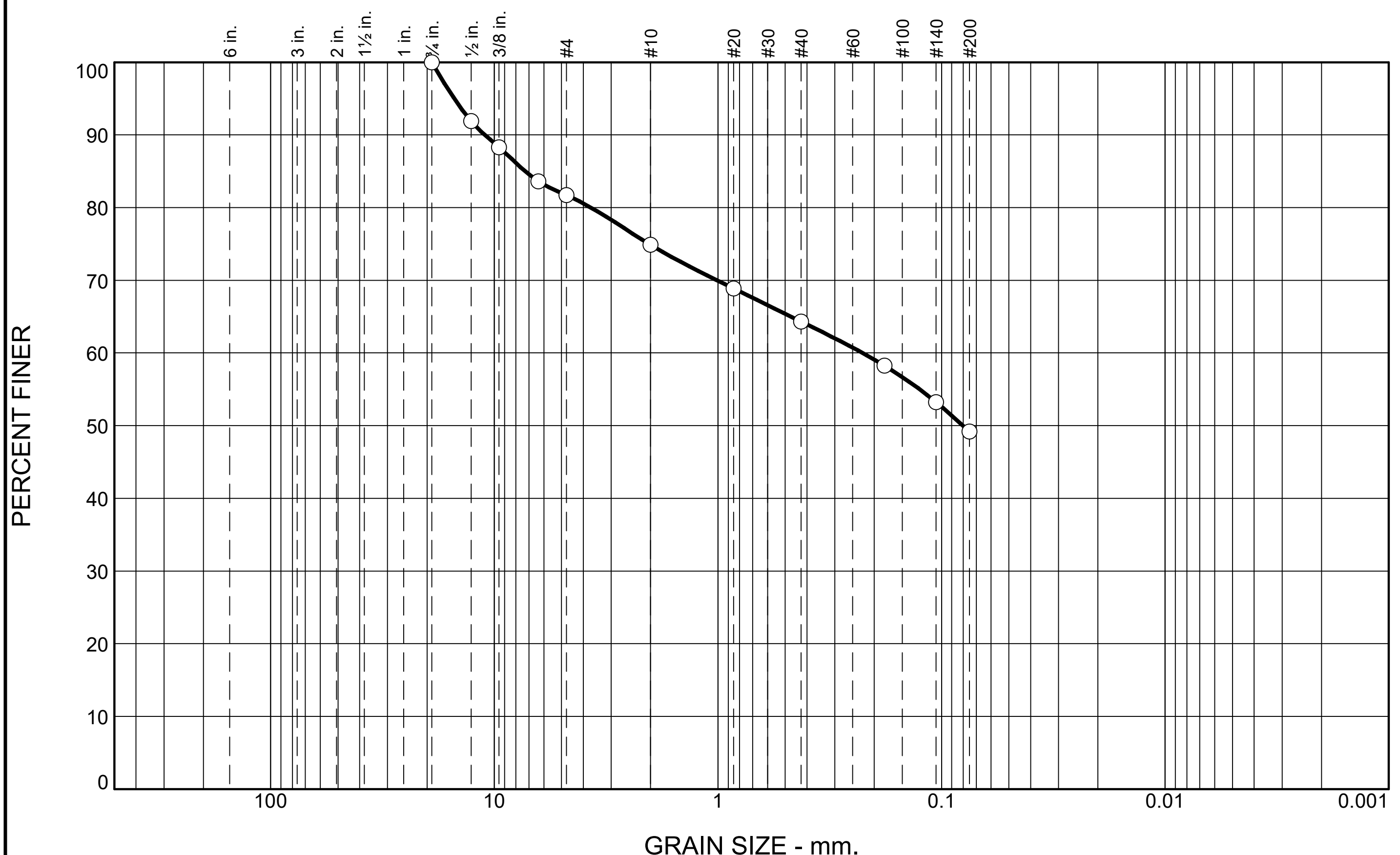
R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Crooker Brooke (No 207786)
Dixmont, ME
Project No: 0956-013BG12 **Lab No.** 17569-05

Tested By: CAG/FMG **Checked By:** MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	18.3	6.8	10.6	15.1	49.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	91.9		
3/8"	88.3		
1/4"	83.6		
#4	81.7		
#10	74.9		
#20	68.9		
#40	64.3		
#80	58.2		
#140	53.2		
#200	49.2		

* (no specification provided)

Soil Description

silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 11.0199 D_{85}= 7.2768 D_{60}= 0.2256
D_{50}= 0.0803 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 14.3%

Location: BB-DCB-103
Sample Number: 7D **Depth:** 13.0-15.0' **Date:** 07/11/2023

R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Crooker Brooke (No 207786)
Dixmont, ME
Project No: 0956-013BG12 **Lab No.** 17569-06

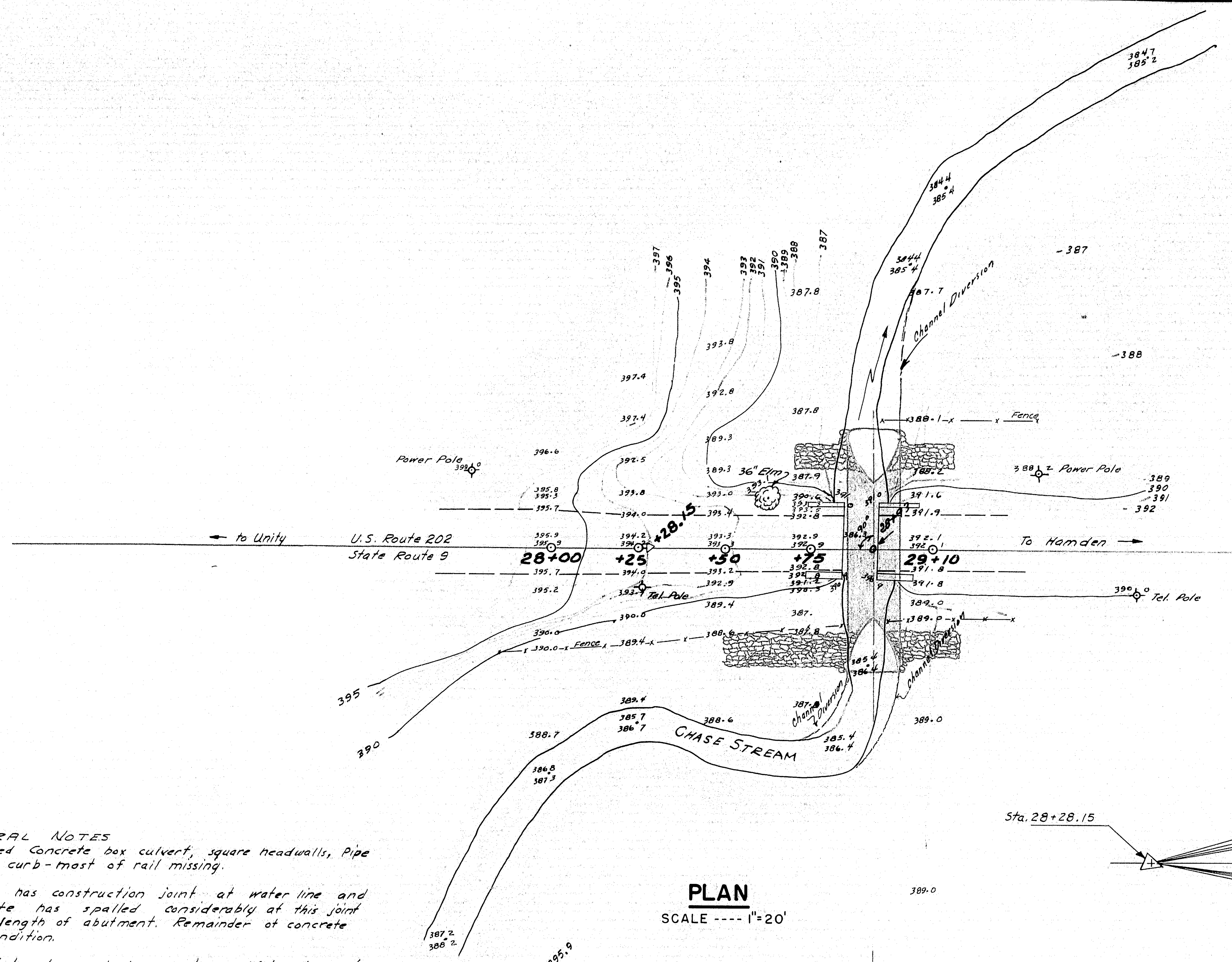
Tested By: CAG/SJV **Checked By:** MTG

MTG

APPENDIX C

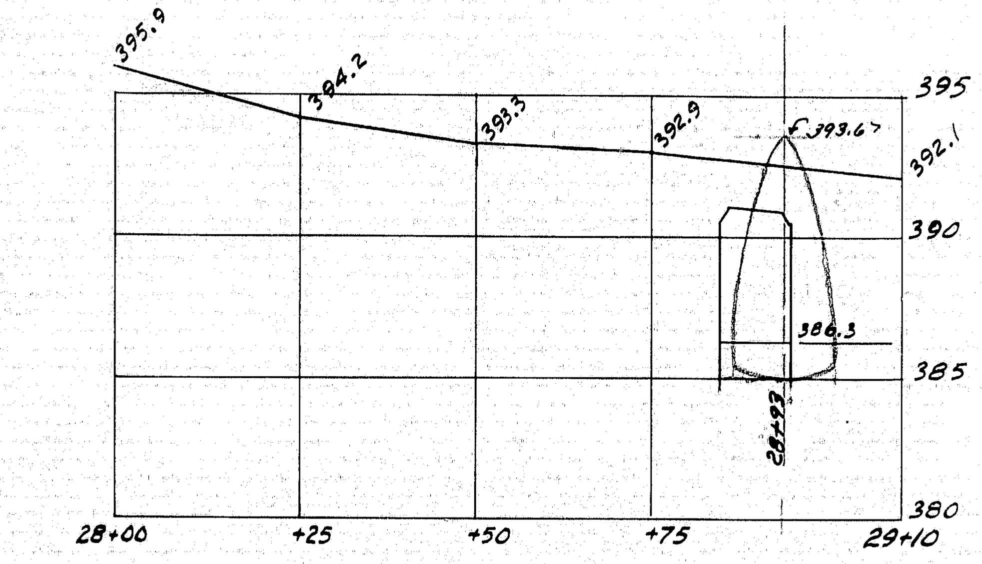
Historic Bridge Drawings

B.P.R. DIV. NO.	STATE	F.A.S. PROJ. NO.	SHEET NO.	TOTAL SHEETS
1	MAINE	5-0210(4)	5	40



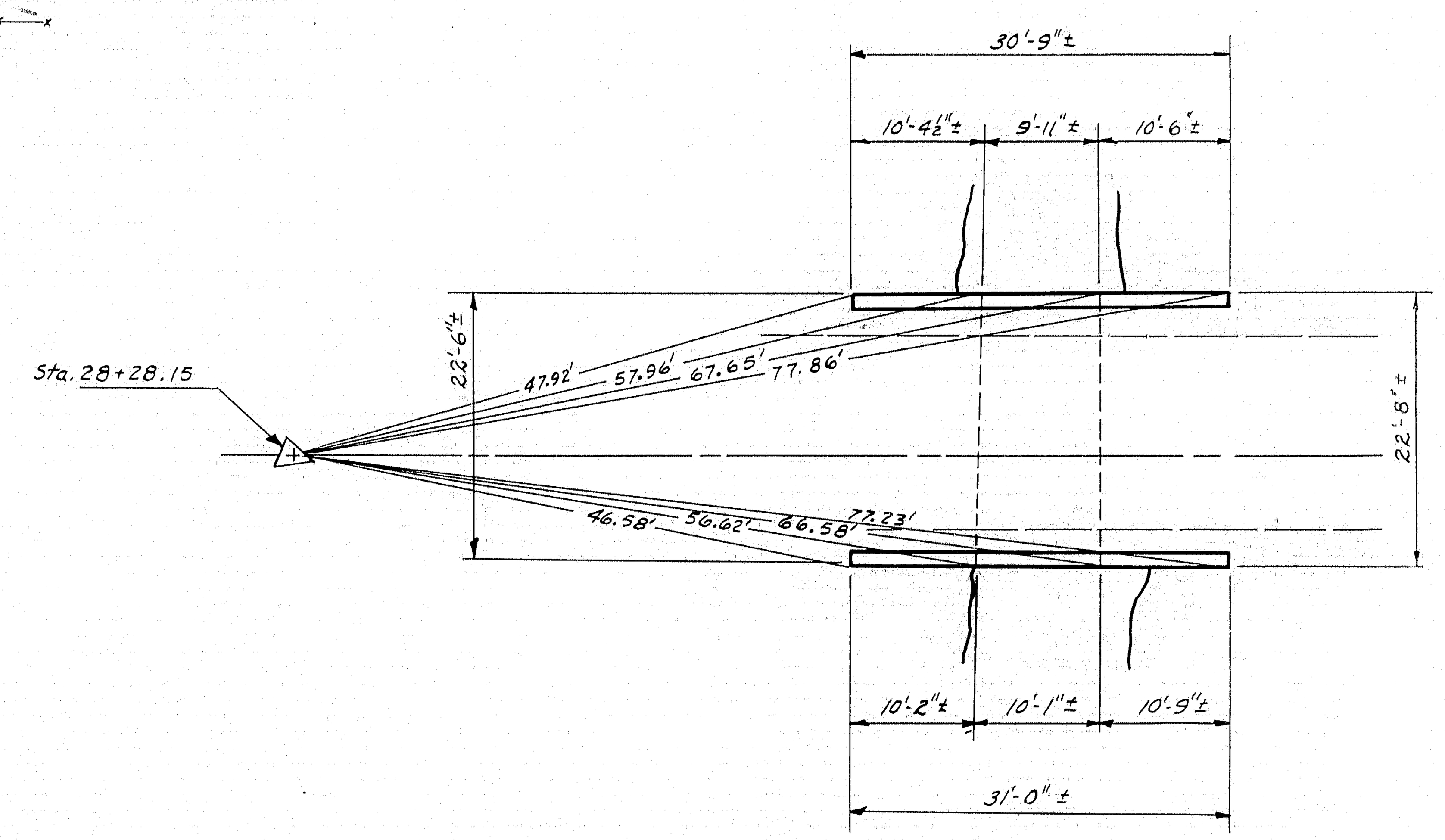
PLAN

SCALE ---- 1" = 20'



PROFILE

SCALES
HOR. ---- 1" = 20'
VERT. --- 1" = 5'



LOCATION - EXISTING STRUCTURE

Scale --- 1" = 10'

GENERAL NOTES

EXISTING SUPERSTRUCTURE : Reinforced Concrete box culvert, square headwalls, Pipe rail on curb - most of rail missing.

EXISTING SUBSTRUCTURE : Abut. #2 has construction joint at water line and concrete has spalled considerably at this joint entire length of abutment. Remainder of concrete fair Condition.

STREAM : Flat intervale and bog upstream. Water does not flow over road.

FOUNDATION : Rod penetrated easily 3.0' below water and then gradually slower 7.0' below water to very firm material. Located 10' D.S. from D.S. corner Abut. #2.

SURVEY - BLAKE & DENNIS
PLOT - ELLIS
BRIDGE — 5424

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CROCKER BRIDGE
OVER
CHASE STREAM
IN THE TOWN OF
DIXMONT
PENOBSCOT COUNTY

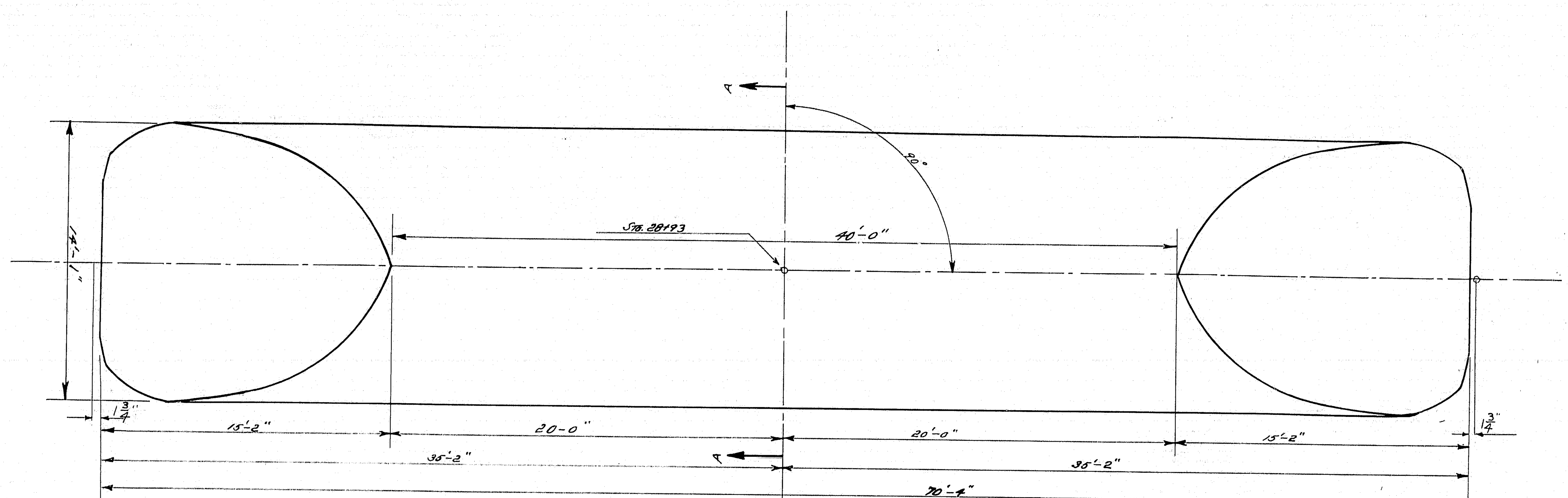
SURVEY PLAN

SHEET 1 OF 2 AUGUSTA, MAINE DEC. 1951

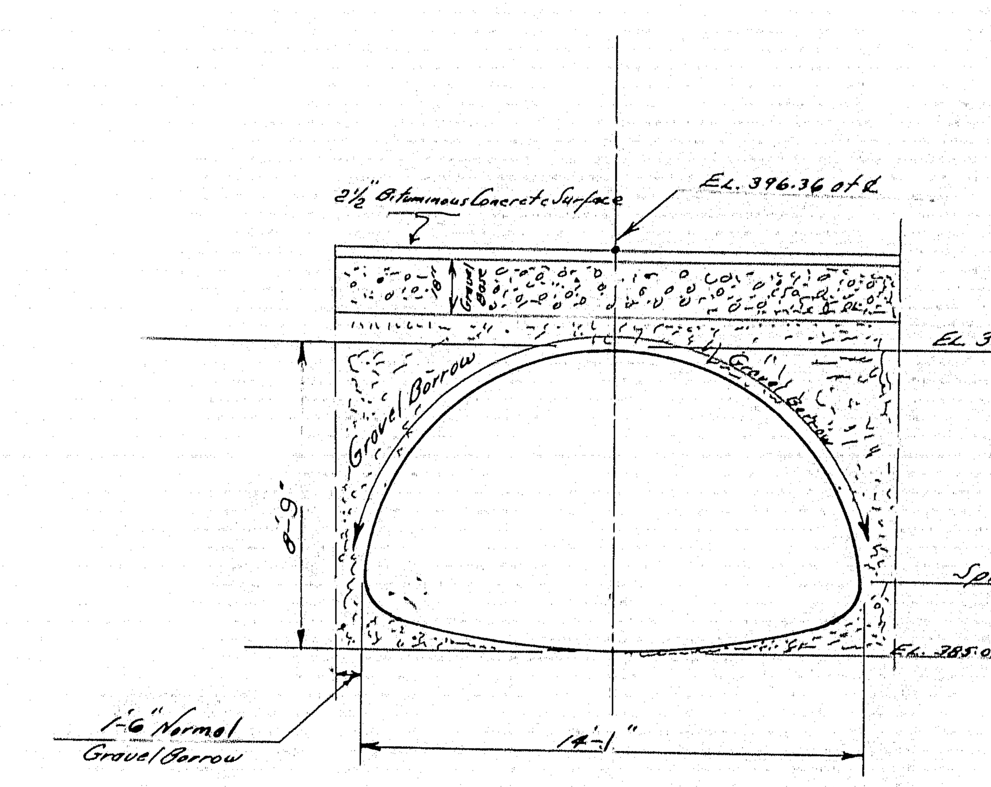
M 588

0 1 2 3 4 5 INCHES

B.P.R. DIV. NO.	STATE	F.A.S. PROJ. NO.	SHEET NO.	TOTAL SHEETS
I	MAINE	S-0210(4)	6	40



PLAN



SECTION A-A

NOTES

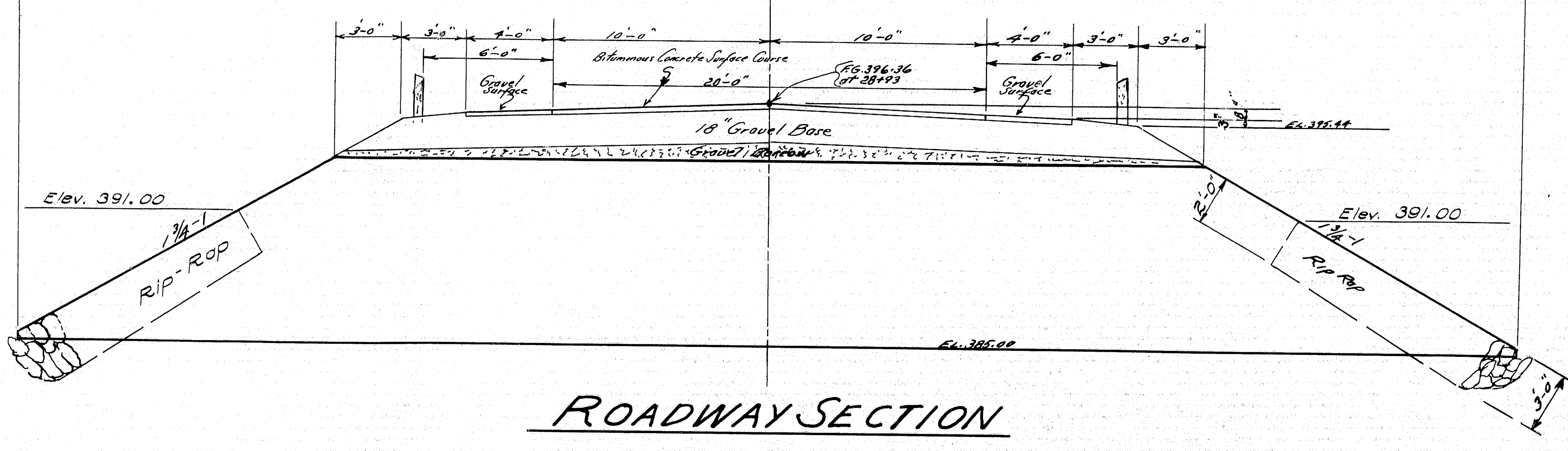
All plates shall have a 6"x2" corrugations.
All plates entirely above the springline to be of 5 gage.
All plates entirely or partially below spring line to be of next heavier gage. Any rock or ledge formation in stream bed below pipe shall be removed to a minimum depth of 8" and replaced with gravel borrow. There shall be a minimum of 4-3/4" bolts per foot of longitudinal seam.

REQUIRED

1-Sectional Plate Pipe Arch. Span 14'-1"
Rise 8'-9". Loading H-20-44

Specifications: A.A.S.H.O. Standard Specifications for Bridges

The span and rise of the arch may be varied slightly at the approval of the Engineer.



ROADWAY SECTION

Design Bunker
Check [illegible]

Bridge 5424

STATE HIGHWAY COMMISSION
BRIDGE DIVISION
CROCKER BRIDGE
OVER
CHASE STREAM
IN THE TOWN OF
DIXMONT
PENOBSCOT COUNTY
PIPE ARCH
SHEET 2 OF 2 AUGUSTA, Me. MARCH '51

M. 589

0 1 2 3 4 5 INCHES

APPENDIX D

Calculations

Bearing Resistance and
Modulus of Subgrade Reaction

PROBLEM STATEMENT & OBJECTIVE

Calculate the Strength and Service Limit bearing resistances of the proposed culvert and calculate the modulus of subgrade reaction.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. Bowles (1997) Foundation Analysis and Design, Fifth Edition.
3. Maine Department of Transportation Bridge Design Guide (BDG) 2003, with updates to June 2018.

AVAILABLE INFORMATION

1. Boring logs dated 5-30-2023 to 6-1-2023 by New England Boring Contractors (monitored by Haley & Aldrich, Inc.).
2. Preliminary plan set prepared by MaineDOT dated July 2023.

ASSUMPTIONS

1. There is no vertical load eccentricity.
2. Groundwater is located at a depth of 10 ft below the finished roadway based on the highest encountered water levels in the boring logs.
3. Bottom of culvert is 14.8 ft below ground surface based on scaling from the preliminary plans
4. Footing size considered: 24 ft length x 77 ft long.
5. Footing will bear on granular borrow.
6. Soil properties for the granular borrow will be 125 pcf (unit weight) and 32 degrees (phi angle), per MaineDOT BDG.

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The factored bearing resistance for the Strength Limit State is 28.3 ksf

Service Limit State

The factored bearing resistance for the Service Limit State is 6 ksf for 1 in. settlement.

Modulus of Subgrade Reaction

The recommended modulus of subgrade reaction is 120 pci

BACKGROUND INFORMATION FROM AASHTO LRFD

$$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a\text{-}1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \quad (10.6.3.1.2a\text{-}4)$$

where:

- c = cohesion, taken as undrained shear strength (ksf)
- N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
- N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
- N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
- γ_q = total (moist) unit weight of soil above the bearing depth of the footing (kcf)
- γ_f = total (moist) unit weight of soil below the bearing depth of the footing (kcf)
- D_f = footing embedment depth (ft)
- B = footing width (ft)
- $C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
- s_c, s_γ, s_q = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)
- d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation determined from Eq. 10.6.3.1.2a-10 (dim)
- i_c, i_γ, i_q = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

For ϕ_f = 0:

$$i_c = 1 - (nH/cBLN_c) \quad (10.6.3.1.2a\text{-}5)$$

For ϕ_f > 0:

$$i_c = i_q - [(1 - i_q)/(N_q - 1)] \quad (10.6.3.1.2a\text{-}6)$$

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL\cot\phi_f)}\right]^n \quad (10.6.3.1.2a\text{-}7)$$

$$i_\gamma = \left[1 - \frac{H}{V + cBL\cot\phi_f)}\right]^{(n+1)} \quad (10.6.3.1.2a\text{-}8)$$

$$n = [(2 + L/B)/(1 + L/B)]\cos^2\theta + [(2 + B/L)/(1 + B/L)]\sin^2\theta \quad (10.6.3.1.2a\text{-}9)$$

where:

- B = footing width (ft)
- L = footing length (ft)
- H = unfactored horizontal load (kips)
- V = unfactored vertical load (kips)
- θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees)

10.6.3.1.2b—Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2b\text{-}1)$$

$$\phi^* = \tan^{-1}(0.67\tan\phi_f) \quad (10.6.3.1.2b\text{-}2)$$

where:

- c^* = reduced effective stress soil cohesion for punching shear (ksf)
- ϕ^* = reduced effective stress soil friction angle for punching shear (degrees)

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

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

$$d_q = 1 + 2\tan\phi_f(1 - \sin\phi_f)^2 \arctan\left(\frac{D_f}{B}\right) \quad (10.6.3.1.2a\text{-}10)$$

Eq. 10.6.3.1.2a-10 has been verified to cover a range of friction angle, ϕ_f, of 32 degrees to 42 degrees, and a range of D_f/B of 1 to 8. Depth correction factor values beyond this range have not been verified at this time.

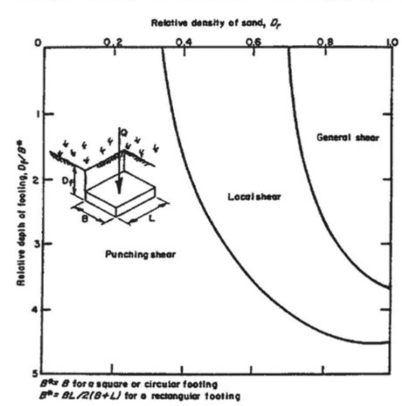


Figure C10.6.3.1.2b-1—Modes of Bearing Capacity Failure for Footings in Sand

HALEY ALDRICH	CALCULATIONS	File No.:	207786
		Sheet:	3 of 6
Client	Fuss & O'Neill	Date:	24-Jul-2023
Project	Dixmont Crocker Brook Culvert Replacement WIN 026097.00	Computed by:	EMH
Subject	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	MMB

INPUT PARAMETERS FOR BEARING CAPACITY

ϕ (deg.) =	32
γ_q (pcf) =	125
γ_f (pcf) =	125
c (psf) =	0
D_f (ft) =	14.80
D_w (ft) =	10
B (ft) =	24.0
e_B (ft) =	0.00
L (ft) =	77.0
e_L (ft) =	0.0
RF or 1/FS =	0.45
Depth Corr., (Y/N)?	N

CALCULATIONS FOR BEARING CAPACITY

B_{eff} = B' (ft) =	24.0
L_{eff} = L' (ft) =	77.0
$N_f = f(\phi)$ =	3.3
$N_c = f_1(\phi)$ =	35.5
$N_q = f_2(\phi)$ =	23.2
$Ng = f_3(\phi)$ =	30.2
s_c =	1.20
s_q =	1.19
s_g =	0.88
d_q =	1.00
C_{wq} =	0.84
C_{wg} =	0.50
N_{cm} =	42.73
N_{qm} =	27.72
N_{gm} =	26.43
q_n or q_{ult} (psf) =	62,790
q_n or q_{ult} (ksf) =	62.8
RF×q_n or q_{ult}/FS (ksf) =	28.3

Notes:
1. Refer to background page for definition of input parameters.
2. Analysis does not consider inclined load and inclined load adjustment factors, nor does it adjust for footings near slopes.
3. RF = resistance factor (e.g., as in AASHTO LRFD); FS is factor of safety if using allowable stress design.
4. e_B and e_L are the vertical load eccentricities in the B and L directions, respectively. Check code guidance for maximum vertical load eccentricities allowed.
5. B_{eff} and L_{eff} are the effective footing dimensions considering vertical load eccentricity and are equal to B-2e_B and L-2e_L, respectively.
6. RF×q_n and q_n/FS are the factored bearing resistance and the allowable bearing capacity, respectively.
7. Footing settlement should be checked separately.

BEARING RESISTANCE AT THE SERVICE LIMIT STATE

AASHTO Section 10.6.2.6 - Bearing Resistance at the Service Limit State

The use of presumptive values shall be based on the knowledge of geological conditions at or near the structure site... These bearing resistances are settlement limited, e.g., 1.0-in., and apply only at the service limit state.'

Table C10.6.2.5.1-1

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

Presumptive Bearing at Service Limit State: 6 ksf

HALEY ALDRICH	CALCULATIONS	File No.:	207786
		Sheet:	5 of 6
Client:	Fuss & O'Neill	Date:	24-Jul-2023
Project:	Dixmont Crocker Brook Culvert Replacement WIN 026097.00	Computed by:	EMH
Subject:	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	MMB

BACKGROUND INFORMATION FROM BOWLES 1977

$$k_s = \frac{\Delta q}{\Delta H} = \frac{1}{BE_s' I_s I_F} \qquad (9\text{-}7)$$

$$E_s' = (1 - \mu^2)/E_s$$

$$I_1 = \frac{1}{\pi}\left[M \ln \frac{(1 + \sqrt{M^2+1})\sqrt{M^2+N^2}}{M(1+\sqrt{M^2+N^2+1})} + \ln \frac{(M + \sqrt{M^2+1})\sqrt{1+N^2}}{M + \sqrt{M^2+N^2+1}}\right] \qquad (a)$$

$$I_2 = \frac{N}{2\pi}\tan^{-1}\left(\frac{M}{N\sqrt{M^2+N^2+1}}\right) \quad (\tan^{-1} \text{ in radians}) \qquad (b)$$

$$I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2 \qquad (c)$$

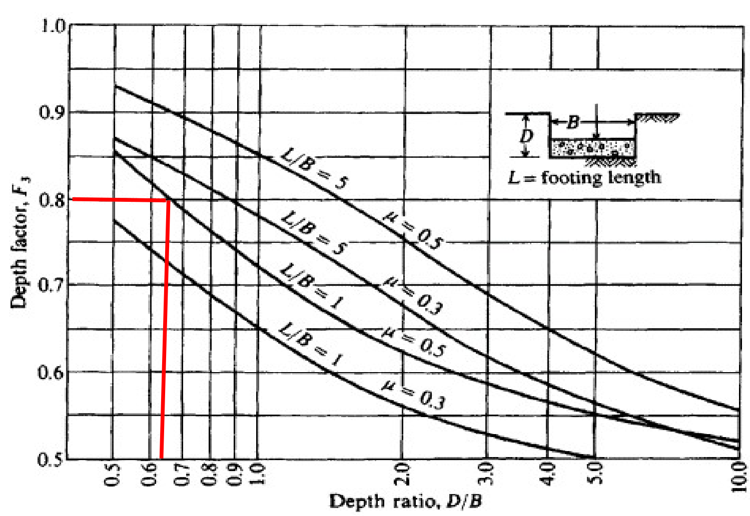
TABLE 2-7
Values or value ranges for Poisson's ratio μ

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand, gravelly sand	−0.1–1.00
commonly used	0.3–0.4
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15
Steel	0.33

4. Note that the stratum depth actually causing settlement is not at $H/B \to \infty$, but is at either of the following:
a. Depth $z = 5B$ where B = least total lateral dimension of base.
b. Depth to where a hard stratum is encountered. Take "hard" as that where E_s in the hard layer is about $10E_s$ of the adjacent upper layer.

TABLE 2-8
Value range* for the static stress-strain modulus E_s for selected soils (see also Table 5-6)
Field values depend on stress history, water content, density, and age of deposit

Soil	E_s, MPa
Clay	
Very soft	2–15
Soft	5–25
Medium	15–50
Hard	50–100
Sandy	25–250
Glacial till	
Loose	10–150
Dense	150–720
Very dense	500–1440
Loess	15–60
Sand	
Silty	5–20
Loose	10–25
Dense	50–81
Sand and gravel	
Loose	50–150
Dense	100–200
Shale	150–5000
Silt	2–20



D/B =	0.63
L/B =	3.2
v =	0.35

HALEY ALDRICH	CALCULATIONS	File No.:	207786
		Sheet:	6 of 6
Client:	Fuss & O'Neill	Date:	24-Jul-2023
Project:	Dixmont Crocker Brook Culvert Replacement WIN 026097.00	Computed by:	EMH
Subject:	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	MMB

INPUT PARAMETERS FOR SUBGRADE MODULUS

Width of Footing, B (ft) =	24	
Length of Footing, L (ft) =	77	
Depth of Footing, D (ft) =	15	
Elastic Modulus, E_s (MPa) =	100	from Bowles Table 2-8
Elastic Modulus, E_s (ksf) =	2089	
Poisson's Ratio, v_S =	0.35	from Bowles Table 2-7

CALCULATIONS FOR SUBGRADE MODULUS

E's (ft^2/k) =	0.000420158	from Bowles Equation 2-65
Thickness of Stratum, H = 5*B = (ft) =	120	from Bowles Chapter 5
N = H/B =	5.0	For corner of footing
M = L/B =	3.2	For corner of footing
I_1 =	0.552	from Bowles Equation 5-16a
I_2=	0.084	from Bowles Equation 5-16b
Steinbrenner Influence Factor, I_s =	0.59	from Bowles Equation 5-16c
D/B =	0.63	
Depth Influence Factor, I_F =	0.8	from Bowles Table 5-7

Modulus of Subgrade Reaction, k_s (pci) =	121	from Bowles Equation 9-7

Recommend k_s = 120 pci

Lateral Earth Pressures

HALEY ALDRICH

HALEY ALDRICH	CALCULATIONS	File No.:	207786-000
		Sheet:	1 of 2
Client:	MaineDOT	Date:	9/16/2024
Project:	Dixmont Crocker Brook Culvert Replacement WIN 26097.00	Computed by:	EMH
Subject:	Lateral Earth Pressure Coefficients	Checked by:	NAS

PROBLEM STATEMENT & OBJECTIVE

Calculate the at-rest and active lateral earth pressure coefficients for the proposed culvert replacement.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.
2. MaineDOT Bridge Design Guide (MaineDOT BDG), August 2003, with interim revisions through June 2018.

ASSUMPTIONS

1. Per the MaineDOT BDG, culverts are to be backfilled with granular underwater backfill (Soil Type 4).
2. Backfill is free-draining, so hydrostatic pressures do not develop. Backfill is normally consolidated.
3. Backslope angle behind inlet/outlet walls is unknown (see Sheet 2 for K_a values based on varying backslope angles).
4. Active lateral earth coefficients (K_a) assume the inlet/outlet walls are free to rotate.
5. At-rest lateral earth coefficients (K_o) assume walls do not rotate.

CALCULATIONS

Inputs

Total Unit Weight, ϒ (pcf) =	125	pcf	Soil Type 4, BDG Table 3-3
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, BDG Table 3-3
Backslope Angle, β =	0	degrees	Assumes level backfill (see note below)
Backface of Wall Angle to Horizontal, θ =	90	degrees	
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, BDG Table 3-3

At-Rest Lateral Earth Pressure Coefficient, K_o

$K_o = 1-\sin\phi$ AASHTO LRFD Eq. 3.11.5.2-1

K_o = 0.47

Static Active Lateral Earth Pressure Coefficient, K_a

$K_a = \sin^2(\theta+\phi')/r(\sin^2\theta\sin(\theta-\delta))$ AASHTO LRFD Eq. 3.11.5.3-1

where $r = [1 + \sqrt{(\sin(\phi+\delta)\sin(\phi-\beta)/(\sin(\theta-\delta)\sin(\theta+\beta))}]^2$

K_a = 0.27 (For level backfill. See Sheet 2 for K_a values based on varying backslope angles).

HALEY ALDRICH	**CALCULATIONS**	File No.:	207786-000
		Sheet:	2 of 2
Client:	MaineDOT	Date:	9/16/2024
Project:	Dixmont Crocker Brook Culvert Replacement WIN 26097.00	Computed by:	EMH
Subject:	Lateral Earth Pressure Coefficients	Checked by:	NAS

Backfill Slope Angle Behind Wall, β (degrees)	K_a	
0	0.27	Level backfill
1	0.28	
2	0.28	
3	0.28	
4	0.29	
5	0.29	
6	0.30	
7	0.30	
8	0.30	
9	0.31	
10	0.31	
11	0.32	
12	0.32	
13	0.33	
14	0.33	
15	0.34	
16	0.35	
17	0.35	
18	0.36	
19	0.37	
20	0.38	
21	0.38	
22	0.39	
23	0.41	
24	0.42	
25	0.43	
26	0.45	
26.6	0.46	2H:1V backfill
27	0.47	
28	0.49	
29	0.52	
30	0.55	
31	0.61	
32	0.79	

Frost Protection

OBJECTIVE:

Evaluate maximum depth of frost penetration based on soil and groundwater conditions, as well as geographic site location.

REFERENCES:

1. MaineDOT Bridge Design Guide, August 2003, with June 2018 updates
2. Boring logs BB-DCB-101 through BB-DCB-103
3. Laboratory testing performed by R.W. Gillespie & Associates, Inc.

EVALUATION:

1. Gather relevant information from soil samples taken below the proposed culvert:

Boring No.	Sample No.	Depth	USCS	Moisture Content (%)
BB-DCB-102	4D	7.5 ft to 9.5 ft	SM	4.9
BB-DCB-102	7D	15.0 ft to 17.0 ft	GM	8.4
BB-DCB-103	3D	5.0 ft to 7.0 ft	SM	13.1
BB-DCB-103	7D	13.0 ft to 15.0 ft	SM	14.3

2. Per MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1700°F - days.

3. Estimate range in frost penetration depth using MaineDOT Bridge Design Guide Table 5-1, the design freezing index above, and a "design" moisture content of the coarse- and fine-grained soil (i.e., assume average of available moisture content results).

4. For coarse-grained soil with water content of approx. 10 percent (average of four moisture content results from above), per Table 5-1, maximum depth of frost is approximately 7.3 ft (87.5 in.).
Subgrade soils are primarily coarse-grained, therefore fine-grained soil was not considered here.

Recommend culvert be founded at least 7.3 ft below the lowest adjacent ground surface exposed to freezing.

CALCULATIONS	File No.:	0207786-000
	Sheet:	2 of 2
Client: Fuss & O'Neill	Date:	16-Aug-2024
Project: Dixmont Crocker Brook Culvert Replacement WIN 26097.00	Computed by:	EMH
Subject: Frost Penetration Depth Evaluation	Checked by:	EAF

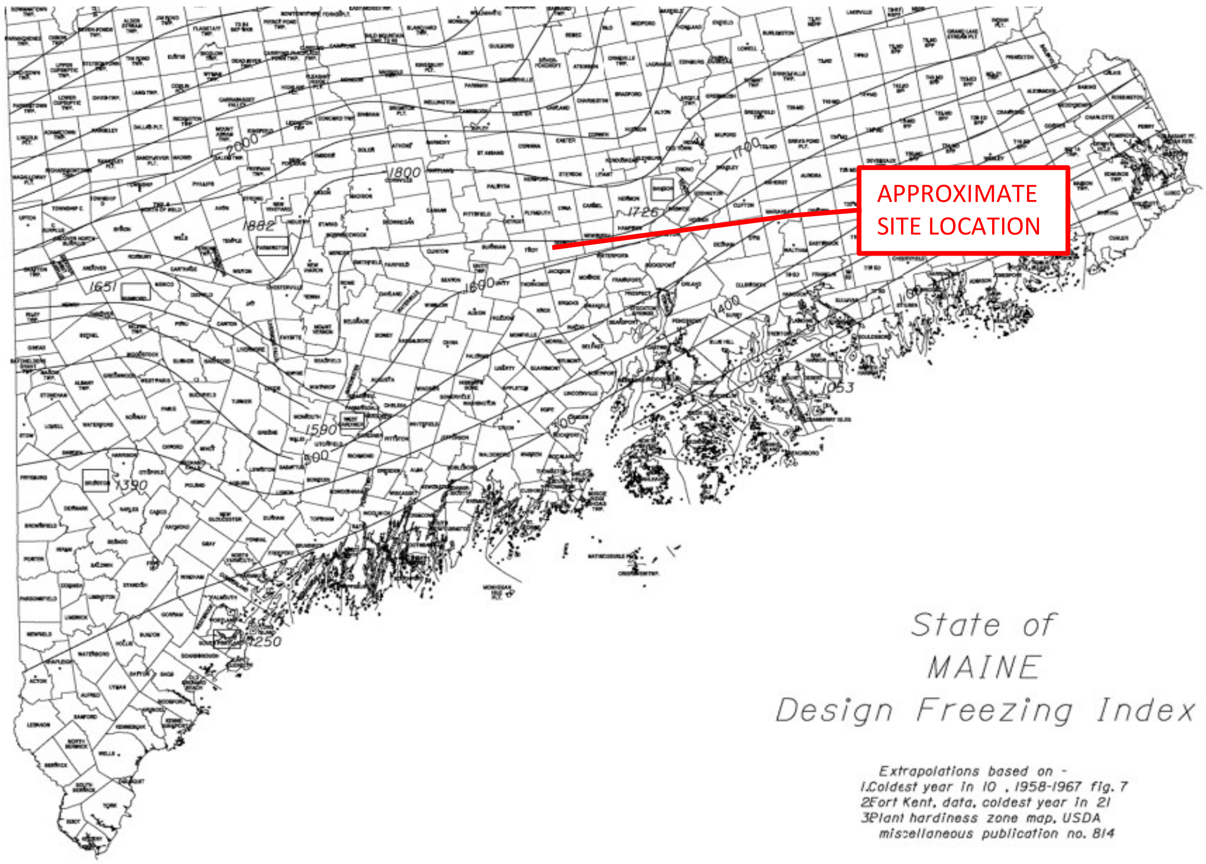


Table 5-1 Depth of Frost Penetration

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