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GEOTECHNICAL DESIGN REPORT ON
OSBORNE BRIDGE OVER FARNHAM BROOK, BRIDGE NO. 2634
MAINEDOT WIN 26111.00
PITTSFIELD, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

for
Fuss & O'Neill, Inc.
Manchester, New Hampshire

File No. 0208477-000
February 2026

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

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File No. 0208477-000

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
Osborne Bridge over Farnham Brook, Bridge No. 2634
MaineDOT WIN 26111.00
Pittsfield, Maine

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit herewith our report entitled, "Geotechnical Design Report, Osborne Bridge over Farnham Brook, Bridge No. 2634, MaineDOT WIN 26111.00, Pittsfield, Maine" (GDR). This GDR has been prepared in accordance with our proposed work scope and fee estimate document, dated 19 May 2023, and executed by you on 10 August 2023.

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 State Route 100 over Farnham Brook in Pittsfield, 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. 193+00 (south) to Sta. 194+75 (north).

PROJECT LOCATION, EXISTING SITE CONDITIONS, AND EXISTING BRIDGE STRUCTURE

The existing bridge carries northbound (NB) and southbound (SB) vehicular traffic on State Route 11/100 over Farnham Brook in Pittsfield, Maine, approximately 0.2 miles north of Webb Road. The project site adjacent to the roadway consists of a partially wooded area. Existing roadway grades within the limit of work generally slope downward from south to north and range from approximately El. 177 (south) to El. 175 (north). The bottom of Farnham Brook, within the limits of State Route 11/100, is approximately El. 167.

Based on our review of the historical plans for the subject project dated 17 February 1932, it is our understanding that the existing bridge consists of a two-span, cast-in-place concrete culvert. The invert of the existing culvert is approximately El. 167 based on the plan impacts complete (PIC) plans. The existing culvert was constructed on 8-in. diameter timber grillage. Historical bridge plans are included in Appendix C.

PROPOSED BRIDGE STRUCTURE

Based on our review of the PIC plans, it is our understanding that the bridge replacement alternative recommended by Fuss & O'Neill generally consists of the following:

- A 71-ft long x 13-ft span x 6-ft rise precast concrete box culvert. The culvert invert elevation will be at El. 165. The bottom of the culvert structure will bear at approximately El. 164.
- 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.
- The PIC plans show the upstream and downstream ends of the culvert will be beveled.
- The proposed culvert will be shifted to approximately 5.5 ft north of the existing culvert.
- Vertical profile of Route 100 will roughly match the existing profile.

Existing and proposed site conditions are shown on Figure 2.

Geologic Setting

According to the Surficial Geology of the Pittsfield Quadrangle, surficial deposits mapped within the site and nearby vicinity consist of swamp/tidal marsh deposits consisting of peat, silt, clay and sand, and marine deposits consisting of silt and clay. The Surficial Materials Map indicates no mapped bedrock data at the site. According to the Reconnaissance Bedrock Geology of the Pittsfield Quadrangle, 1971, bedrock at the site is Silurian in age and mapped as interbedded phyllite and metasiltstone of the Waterville Formation.

Geotechnical Field Investigation

Haley & Aldrich completed a geotechnical field investigation (investigation) at the site between 21 August and 24 August 2023. Two borings, designated BB-PFB-103 and BB-PFB-104, were drilled at/near the existing bridge. Note, borings BB-PFB-101 and BB-PFB-102 were drilled for the replacement of the adjacent Farnham bridge culvert.

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 Haley & Aldrich by taping distances from existing site features. Ground surface elevations at completed boring locations were determined by Haley & Aldrich using topographic information provided by Fuss & O'Neill. 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 (NEBC) of Hermon, Maine, using a Mobile Drill B-53 track-mounted drill rig. The borings were advanced to depths ranging from approximately 32 to 52 ft below ground surface (BGS) using solid-stem augers to advance through bituminous concrete and concrete at the surface, and then cased-washed drilling methods with 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. Haley & Aldrich reviewed the hammer calibration report provided by NEBC, confirmed that the hammer was calibrated within 12 months of when drilling was completed, and confirmed the hammer efficiency factor. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.76; 76 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 borings BB-PFB-103 and BB-PFB-104 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.

Borings were terminated in soil; therefore, bedrock was not cored.

A Haley & Aldrich geologist was present on-site full time to monitor the drilling, document the soil and groundwater conditions encountered, determine sampling intervals, and prepare boring logs. Soil samples were collected and preserved in plastic containers and are currently being stored in our storage facility in Portland. The soil 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 marine soils and glacial till. Refer to Table II for a detailed summary of the soil units and thicknesses encountered in each boring. A general description of each soil unit is provided separately, below. Detailed soil descriptions are provided on the boring logs included Appendix A. Refer to the interpretive subsurface profile on Figure 3 for a graphical representation of the interpretive subsurface conditions present along the State Route 11/100 centerline.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	10	Medium dense to very dense SAND with various amounts of silt and gravel; very dense GRAVEL; stiff SILT with various amounts of sand and gravel. *(encountered in all borings)*
Organic Deposit	2	Hard sandy SILT, trace organics; medium stuff CLAY, trace roots, organic silt layer. *(encountered in all borings)*
Marine Deposit	>20 to 38	Soft to stiff SILT; medium stiff to hard CLAY with various amounts of silt; medium dense to dense fine to coarse SAND with various amounts of silt and gravel. *(encountered in all borings)*
Glacial Till	>2	Very dense, SAND with various amounts of silt and gravel. (encountered in boring BB-PFB-104)

Please note that soil 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.

GROUNDWATER CONDITIONS

Groundwater observation wells were not installed in either of the completed borings. As a result, long-term static water levels at the site were not 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 6 to 7 ft BGS (El. 168.5 to El. 169).
- water levels measured in each boring during or after drilling was approximately 10.2 to 11.2 ft BGS (El. 164.3 to El. 164.8).
- Surveyed stream water level El. 168.4, as shown in the PIC plans.

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 Farnham 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 investigation to assist in soil classification and determination of engineering soil properties. Geotechnical laboratory testing was performed by GeoTesting Express in Acton, Massachusetts. 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 D6913	Fill	2	AASHTO Classification: A-1-b USCS Classification: SM, GM
		Marine Deposit	2	AASHTO Classification: A-2-4, A-4 USCS Classification: SM
Atterberg Limits	ASTM D4318	Marine Deposit	3	Liquid Limit: 36<LL<40 Plastic Limit: 22<PL<27 Plasticity Index: 13<PI<14 Liquidity Index: 0.2<LI<0.9 Water Content: 30%<WC<34%
One-Dimensional Consolidation	ASTM D2435	Marine Deposit	1	Sample determined to be too disturbed to provide accurate consolidation parameters

Notes:
[1] LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; LI = Liquidity Index; WC = Water Content

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 PIC plans (El. 164.0), the thickness of "typical" bedding details (i.e., 1 ft; see below), and the subsurface conditions encountered in the borings, very soft to medium stiff marine soils (i.e., silt, silty clay) are anticipated to be present at the proposed subgrade level (i.e., bottom of bedding/excavation). It is not anticipated that construction of the box culvert and prepared subgrade will require removal of bedrock.

We recommend the following bedding detail:

- The culvert structure should be placed on a 1-ft thick layer of Underdrain Backfill Material, Type C, (MaineDOT Item 203.55, Culvert Bedding Stone). Although a 2-ft thick bedding layer detail is frequently selected for clay subgrades, based on discussions with MaineDOT, we recommend reducing this to a 1-ft thick bedding layer to minimize excavation into marine clay and below groundwater. Stabilization/Reinforcement Geotextile (MaineDOT Standard Specification 722.01) should be placed below and wrapped around the sides of the Underdrain Backfill Material. 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 (marine soils). If over-excavation is required, we recommend that the area be backfilled with Underdrain Backfill Material. Based on the subsurface conditions encountered in the borings, over-excavation of organic soils at the culvert subgrade may 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 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 1.5 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 1.7 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 80 pounds per cubic inch (pci) be used to design the culvert.

SETTLEMENT

Based on our review of plan and profile drawings included in the PIC plans and as stated above, the proposed culvert will be shifted to approximately 5.5 ft north of the existing culvert. The vertical alignments of State Route 11/100 will roughly match existing site conditions. The 5.5 ft directly south of the proposed culvert (in the footprint of the existing culvert) will be subject to approximately 10 ft of new fill, which will result in consolidation settlement.

The computer analysis software, Settle3 Version5, developed by Rocscience, Inc., was used to evaluate settlement due to the shift in culvert horizontal alignment. One incremental consolidation test was performed on the in-situ marine deposits to determine compressibility parameters. Upon reviewing the consolidation test results, we determined that the tested sample was too disturbed to provide accurate compressibility parameters. The following marine deposit compressibility parameters were therefore assumed in the settlement analyses based on other projects in the same region where similar marine deposits were present:

- Overconsolidation Ratio (OCR) = 1
- Virgin Compression Ratio (CR) = 0.19
- Recompression Ratio (RR) = 0.020
- Secondary Compression Ratio ($C_{\alpha\varepsilon}$) = 0.004

- Coefficient of Consolidation (c_v) = 2x10^{-6} ft^2/sec
- The following stages were assumed in Settle3:
  - Existing Embankment and Culvert Construction (reference stage): Existing embankment constructed (10 ft high with embankment fill unit weight of 125 pcf); existing culvert constructed (SER I dead load of 0.856 ksf per Fuss & O'Neill).
  - Proposed Culvert Construction: Existing culvert replaced with off-alignment proposed culvert (SER I dead load of 0.640 ksf per Fuss & O'Neill); embankment fill placed over portion of existing culvert footprint (10 ft high with embankment fill unit weight of 125 pcf)

Based on the above assumptions, we estimate that up to approximately 1¾ in. of primary consolidation settlement will occur post-construction. This settlement will generally occur after final paving, depending on sequencing of the Farnham and Osborne culvert construction. We anticipate that most (more than 90%) of the primary consolidation will occur within the first year following construction. Note that this settlement will occur directly to the south of the proposed culvert over the footprint of the existing culvert. We do not anticipate settlement directly beneath the proposed culvert because the soil beneath the proposed culvert will experience a net unload. Additionally, we estimate that approximately 1.5 in. of secondary compression will occur in this location over the culvert design life of 75 years. The total post-construction settlement over the culvert design life is therefore approximately 3¼ in.

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.

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 12 and 13 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 marine soils) can be accomplished using normal earth-excavating equipment. Note that cobbles were encountered in the existing fill. In addition, the existing timber grillage will be encountered during excavation. Bedrock was not encountered within 32 ft BGS and 52 ft BGS is borings BB-PFB-103 and BB-PFB-104, respectively.

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 surveyed water level in Farnham Brook (El. 168.4) and the proposed subgrade level (El. 163), we anticipate that excavations will typically extend up to approximately 6 ft below the water level in Farnham 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 Farnham Brook.

SUBGRADE PREPARATION

As discussed in previous sections of this GDR, we anticipate that soils present at the excavation subgrade will likely consist of very soft to medium stiff marine soils (i.e., silt and silty clay). 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:

- Use caution removing the existing timber grillage to avoid soil disturbance in the natural bearing soils.
- 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

STATE OF MAINE
ERIN A. FORCE
No. 12207
LICENSED
PROFESSIONAL ENGINEER

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

Enclosures:

- Table I – Subsurface Exploration Location Data
- Table II – Subsurface Exploration Subsurface Data
- Figure 1 – Project Locus
- Figure 2 – Exploration Location Plan
- Figure 3 – Interpretive Subsurface Profile
- Appendix A – Test Boring Logs
- Appendix B – Laboratory Test Results
- Appendix C – Historic Bridge Drawings
- Appendix D – Calculations

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Hanson, Lindley S. and Dabney W. Caldwell, Surficial Geology of the Pittsfield Quadrangle, Maine, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File No. 86-35, 1986.

Locke, Daniel B., Surficial Materials, Pittsfield Quadrangle, Maine, Maine Geological Survey, Department of Conservation, Augusta, Maine. Open File No. 00-53, 2000.

TABLES

TABLE I
Subsurface Exploration Location Data
Osborne Bridge over Farnham Brook, Bridge No. 2634
MaineDOT WIN 26111.00
Pittsfield, Maine

Haley & Aldrich, Inc. File No.: 0208477-000

Test Boring No.[1]	Ground Surface Elevation (ft)[3]	Station[4]	Offset Distance (ft) & Direction[5]
BB-PFB-103	175.5	194+19	6.7 LT
BB-PFB-104	175.0	194+70	7.1 RT

Notes:
1 Test boring locations are shown on Figure 2, Exploration Location Plan.
2 As-drilled location of test borings were determined by Haley & Aldrich, Inc. by taping distances from existing site features.
3 Ground surface elevations at test boring locations were estimated by Haley & Aldrich, Inc. using topographic information provided by Fuss & O'Neill, are measured in ft, and reference the North American Vertical Datum of 1988 (NAVD 88).
4 Station and offset information shown were determined by Haley & Aldrich, are approximate and are rounded to the nearest ft.
5 LT = offset distance toward left direction; RT = offset distance toward right direction.

	Individual	Date
Prepared By:	EMH	4/21/2025
Checked By:	JAD	5/5/2025
Reviewed By:	EAF	5/1/2025

TABLE II
Subsurface Exploration Subsurface Data
Osborne Bridge over Farnham Brook, Bridge No. 2634
MaineDOT WIN 26111.00
Pittsfield, Maine

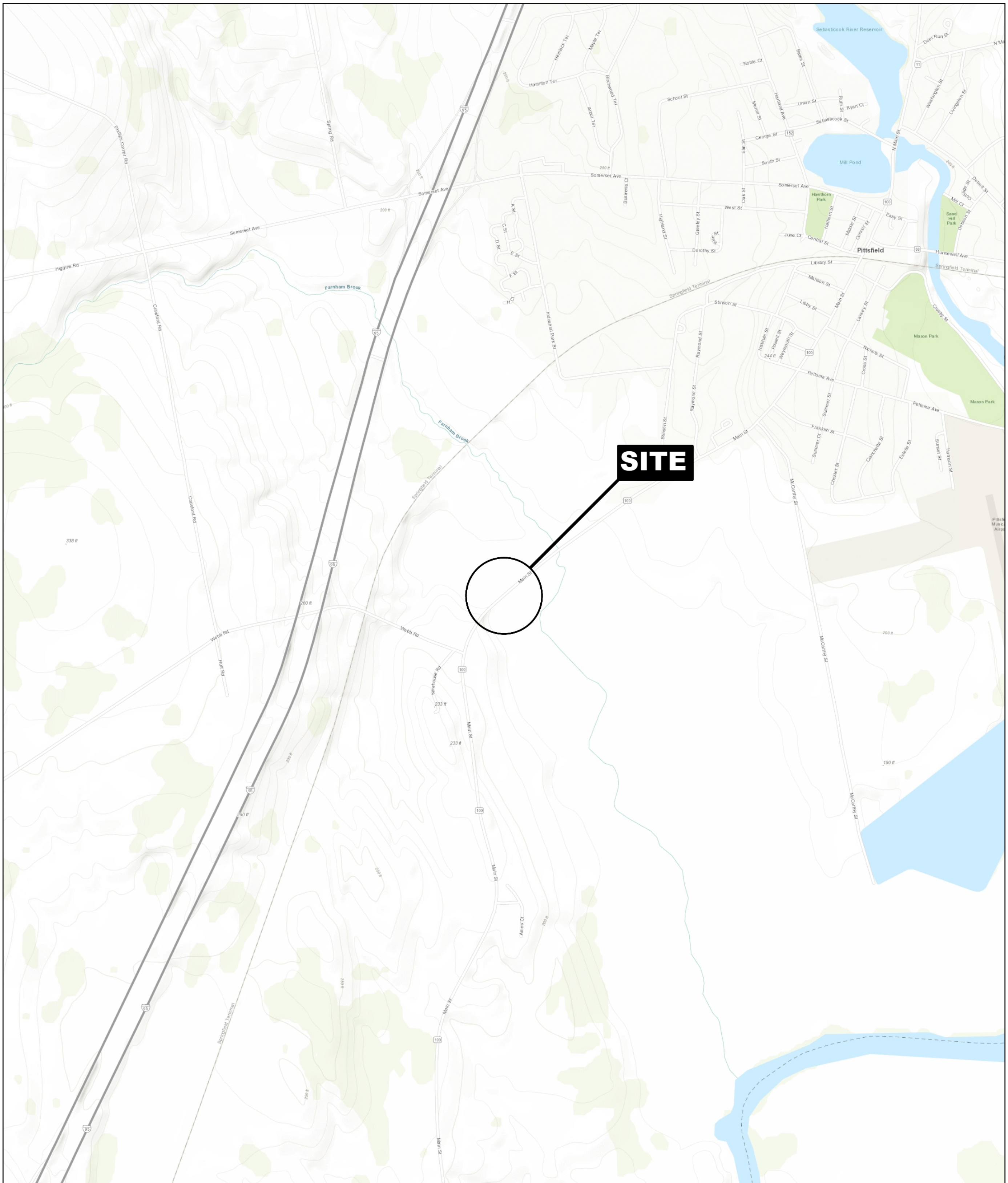
Haley & Aldrich, Inc. File No.: 0208477-000

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Stratigraphy Data[2]												Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration[2] (ft)
		Fill[3]			Organic Deposits			Marine Deposits[4]			Glacial Till[4]				
		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)	Thickness (ft)		
BB-PFB-103	175.5	0.0	175.5	10.0	10.0	165.5	2.0	12.0	-2.0	> 20.0	NE	NE	NE	32.0	143.5
BB-PFB-104	175.0	0.0	175.0	10.0	10.0	165.0	2.0	12.0	-2.0	38.0	50.0	-48.0	> 2.0	52.0	123.0

Notes:
[1] Test boring locations are shown on Figure 2, Exploration Location Plan.
[2] Ground surface elevations at test boring locations were estimated by Haley & Aldrich, Inc. using topographic information provided by Fuss & O'Neill, are measured in feet (ft) and reference the North American Vertical Datum of 1988 (NAVD 88).
[3] Samples were not collected in the upper 5 or 4 feet in test borings BB-PFB-103 and BB-PFB-104, respectively due to drilling difficulties.
[4] "> 20.0" indicates total thickness of stratum is greater than value shown; "NE" indicates stratum was not encountered in test boring.

	Individual	Date
Prepared By:	EMH	4/21/2025
Checked By:	JAD	5/5/2025
Reviewed By:	EAF	5/1/2025

FIGURES



SITE COORDINATES: 44°46'12"N, 69°24'02"W

MAP SOURCE: USGS

HALEY ALDRICH

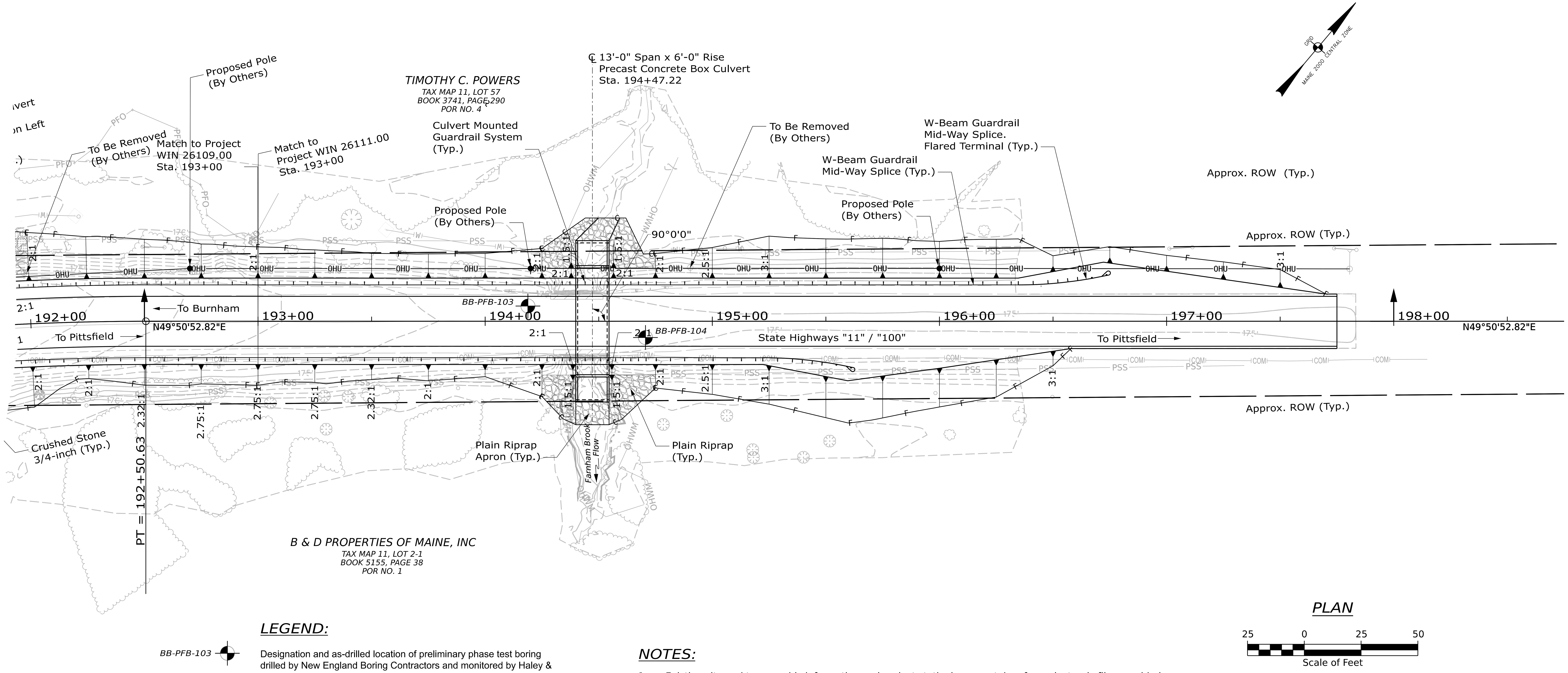
OSBORNE BRIDGE OVER FARNHAM BROOK, BRIDGE NO. 2634
MAINEDOT WIN 26111.00
PITTSFIELD, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 2,000 FEET
FEBRUARY 2026

FIGURE 1

0208477_000_LOCUS HALEYALDRICH\EHUNSTEIN



LEGEND:

BB-PFB-103 — Designation and as-drilled location of preliminary phase test boring drilled by New England Boring Contractors and monitored by Haley & Aldrich, Inc. in August 2023.

Approximate location and orientation of interpretive subsurface profile

Key

BB = Bridge boring

NOTES:

1. *Existing site and topographic information and project stationing were taken from electronic files provided by Fuss & O'Neill on January 29, 2026.*
2. *As-drilled locations of test borings were determined in the field by Haley & Aldrich by taping from existing site features.*
3. *Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).*
4. *Refer to Appendix A for test boring logs.*
5. *Borings BB-PFB-101 and BB-PFB-102 were drilled for the adjacent Farnham bridge.*

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2611100

BRIDGE NO. 2634 | WIN 26111.00 | BRIDGE PLANS

	BY	DATE
PROJ. MANAGER B. NICHOLS		
DESIGN-DETAILED E. HUNSTEIN	K. POST	1/25
CHECKED-REVIEWED E. HUNSTEIN	E. FORCE	2/26
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

OSBORNE BRIDGE
FARNHAM BROOK
PITTSFIELD SOMERSET COUNTY

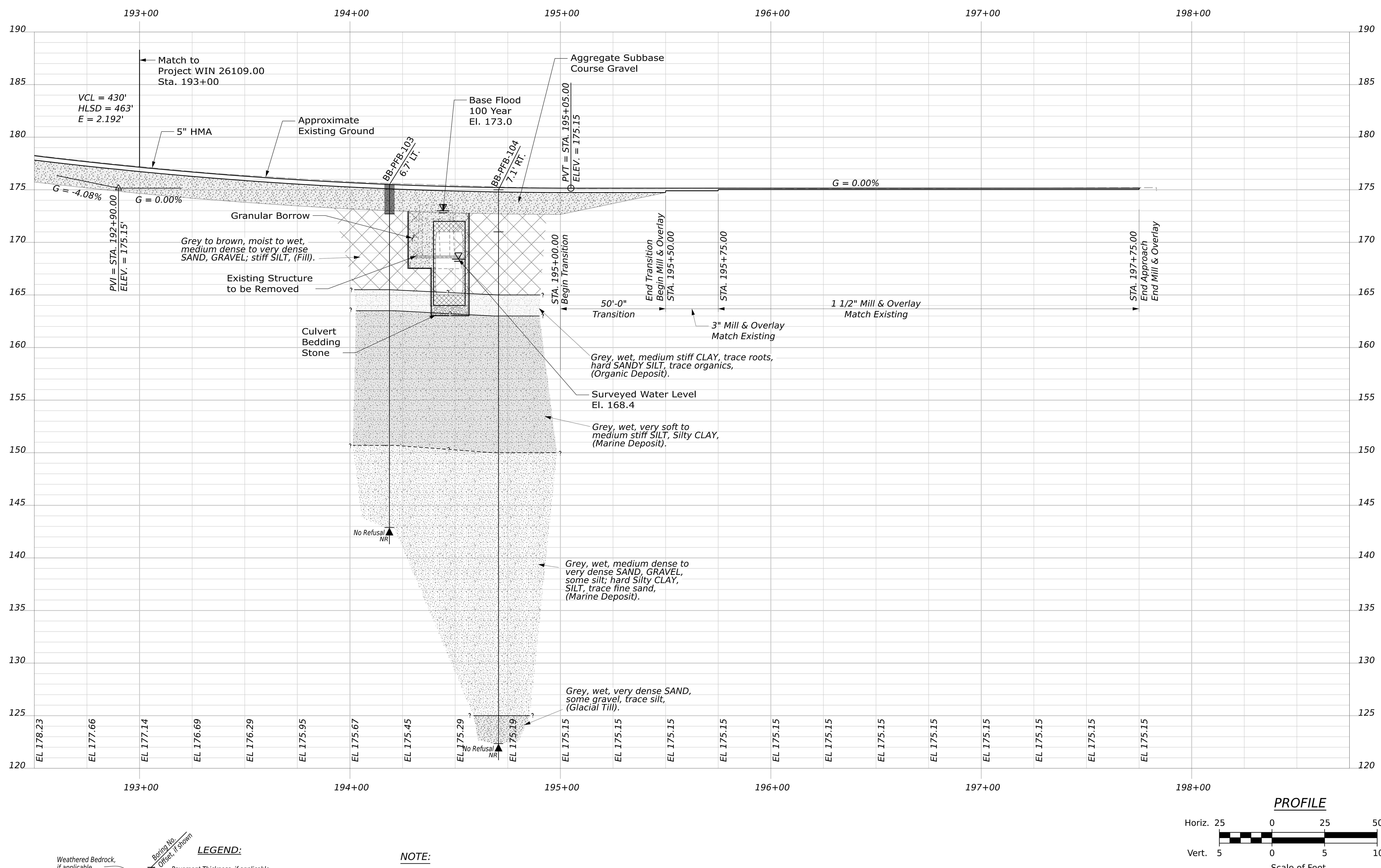
EXPLORATION LOCATION PLAN

HALEY ALDRICH

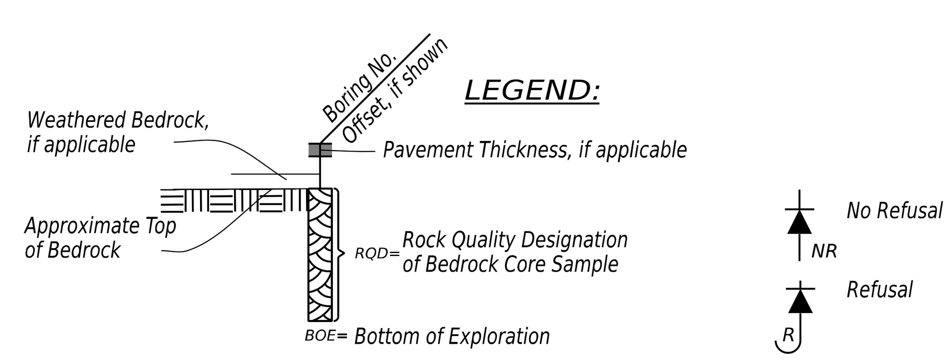
SHEET NUMBER
2
OF

Username: kpostolowski Date: 2/4/2026

Filename: \\haleyaldrich.com\share\CF\Projects\0208477\CAD\0208477_001_Boring Plan.dgn



LEGEND:



NOTE:

1. This generalized interpretive subsurface profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more information refer to the exploration logs.
2. Boring offset is based on the proposed Route 100 baseline.
3. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
4. Test borings were monitored in the field by Haley & Aldrich, Inc.
5. Refer to the preliminary geotechnical design report for test boring logs and rock core photographs.

HALEY ALDRICH

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

02611100

WIN 26111.00

BRIDGE PLANS

BRIDGE NO. 2634

SIGNATURE

P.E. NUMBER

DATE

PROJ. MANAGER	B. NICHOLS	
DESIGN-DETAILED	E. HUNSTEIN	K. POST 1/25
CHECKED-REVIEWED	E. HUNSTEIN	E. FORCE 2/26
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

OSBORNE BRIDGE
FARNHAM BROOK
PITTSFIELD SOMERSET COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF

Username: kpostolowski Date: 2/5/2026

Filename: \\haleyaldrich.com\share\CF\Projects\0208476\CAD\Client Files\2026-1\2026-0129 updated microstation plans\0202476-77_Profiles_Route100.dgn

APPENDIX A
Test Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations (in this order, if applicable):

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Osborne Bridge No. 2634
Location: Pittsfield, Maine

Boring No.: BB-PFB-103
WIN: 26111.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	175.5	**Auger ID/OD:**	--
Operator:	B. Gome	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	HW-140#/30"; SS-140#/30"
Date Start/Finish:	8-23-2023/8-24-2023	**Drilling Method:**	HW to 32 ft	**Core Barrel:**	--
Boring Location:	194+19, 7 ft LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	11.2 ft
Hammer Efficiency Factor:	0.76	**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							0			No Samples collected from 0 to 5 ft. Unable to determine asphalt thickness. Metal rebar encountered at approximately 1.5 ft. Used SSA to penetrate. Solid material, possibly concrete from 1.2 to 2.8 ft. Drill action indicates dense material with occasional obstruction from 2.8 to 5.0 ft.	
							0				
							16	172.7		2.8	
							6			Note: Advanced SSA to 5.0 ft.	
							29				
5	1D	24/6	5.0 - 7.0	11/11/12/6	23	29	36			Grey-brown, wet, medium dense, Gravelly fine to coarse SAND, little silt, well-graded, (Fill). PID Reading = 0 ppm	G#744587 A-1-b, SM
							43				
	2D	24/10	7.0 - 9.0	21/29/61/31	90	114	37			Brown grading to grey slightly mottled, moist, very dense, Silty SAND, trace clay, poorly-graded, (Fill). Grey-brown changing to orange-brown, wet, very dense, Sandy fine to coarse GRAVEL, little silt, well-graded, oxidized, (Fill). PID Reading = 0 ppm	G#744588 A-1-b, GM
							159				
	3D	24/10	9.0 - 11.0	17/36/3/3	39	49	149			Similar to above, except dense, (Fill). PID Reading = 0 ppm	
10							65	165.5		10.0 Grey to grey-brown, wet, hard, fine Sandy SILT, trace organics (roots, peat fibers, depositional structure), (Organic Deposit).	
	4D	24/4	11.0 - 13.0	14/36/41/49	77	98	77			Similar to 3D, (Organic Deposit).	
							51	163.5		12.0	
							65				
							62				
15	5D	24/24	15.0 - 17.0	WOH/24"			21			Dark grey, wet, very soft to medium stiff, SILT, (Marine Deposit)	G#744583 WC=30 LL=40 PL=27 PI=13
							26				
							29				
	V1		18.6 - 19.0	Su=760/165 psf			33	156.9		18.6 65 x 110 mm vane raw torque readings: V1: 320/70 in-lbs	
	V2		19.6 - 20.0	Su=875/190 psf			30			V2: 370/80 in-lbs	
20											
	6D / V3	24/24	22.0 - 24.0 / 22.6 - 23.0	WOR/24" / Su=380/95 psf						Dark grey, wet, soft to medium stiff, Silty CLAY, (Marine Deposit). 65 x 110 mm vane raw torque readings: V3: 160/40 in-lbs	
	V4		23.6 - 24.0	Su=830/165 psf						V4: 350/70 in-lbs	
25								150.7		24.8	

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: Osborne Bridge No. 2634		**Location:** Pittsfield, Maine		**Boring No.:** BB-PFB-103	**WIN:** 26111.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	175.5	**Auger ID/OD:**	--
Operator:	B. Gome	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	HW-140#/30"; SS-140#/30"
Date Start/Finish:	8-23-2023/8-24-2023	**Drilling Method:**	HW to 32 ft	**Core Barrel:**	--
Boring Location:	194+19, 7 ft LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	11.2 ft
Hammer Efficiency Factor: 0.76		**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	7D	24/14	25.0 - 27.0	17/12/15/35	27	34				Dark grey, wet, dense, Silty fine SAND, trace gravel, poorly-graded, (Marine Deposit).	G#744589 A-4, SM
								148.3		27.2	
30	8D	24/6	30.0 - 32.0	18/28/24/30	52	66				Grey, wet, very dense, fine to coarse SAND, little silt, trace fine to coarse gravel, well-graded, (Marine Deposit).	
								143.5		32.0	
										Bottom of Exploration at 32.0 feet below ground surface. No Refusal	
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: Osborne Bridge No. 2634
Location: Pittsfield, Maine

Boring No.: BB-PFB-104
WIN: 26111.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	175.0	**Auger ID/OD:**	--
Operator:	B. Gome	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	HW-140#/30"; SS-140#/30"
Date Start/Finish:	8-21-2023/8-21-2023	**Drilling Method:**	HW to 52 ft	**Core Barrel:**	--
Boring Location:	194+70, 7 ft RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	10.2 ft
Hammer Efficiency Factor:	0.76	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							0			Note: SSA refusal at approxiamtely 18 in. Moved over approximately 3 ft N, SSA refusal again at approximately 18 in. Moved 5 ft N. At third attempt, used SSA to advance to 4.0 ft. Solid material from 0 to 4 ft. Unable to determine asphalt thickness. Possible concrete beneath asphalt. Very hard/dense material. Used maximum downpressure.	
							0				
							19				
							49	171.0		4.0	
5	1D	24/20	4.0 - 6.0	2/3/4/4	7	9	29			Grey, moist to wet at 6 ft, stiff, SILT, trace clay, trace fine to coarse sand, slightly plastic, trace organics, slightly mottled from 5.5 to 6 ft, bonded, (Fill/Reworked Marine Deposit). PID Reading = 0 ppm	
							10				
	2D	24/14	6.0 - 8.0	4/4/8/5	12	15	28			Dark grey, wet, stiff, SILT, trace clay, trace coarse gravel, (Fill/ Reworked Marine Deposit). PID Reading = 0 ppm	
							49				
							29				
							39	165.0		Note: Wood/timber fragments in wash at approximately 10 ft. 10.0	
10	3D	24/18	10.0 - 12.0	WOH/1/3/2	4	5	Push			Blue-grey, wet, medium stiff, CLAY, dark brown organic silt layer, trace roots, (Organic Deposit). PID Reading = 0 ppm	G#744584 WC=34 LL=36 PL=22 PI=14
	4D	24/24	12.0 - 14.0	WOH/24"						Dark grey, wet, very soft, Silty CLAY, moderately plastic, (Marine Deposit). PID Reading = 0 ppm	G#744585 WC=32 LL=38 PL=25 PI-13
	V1		14.6 - 15.0	Su=405/45 psf						65 x 130 mm vane raw torque readings: V1: 170/20 in-lbs	
15	V2		15.6 - 16.0	Su=425/70 psf						V2: 180/30 in-lbs	
	U1	24/24	17.0 - 19.0							U1: Shelby Tube collected 17.0-19.0 ft.	C#IP-1
	V3		19.6 - 20.0	Su=235/80 psf						55 x 110 mm vane raw torque readings: V3: 60/20 in-lbs	
20	V4		20.6 - 21.0	Su=700/155 psf						V4: 180/40 in-lbs	
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: Osborne Bridge No. 2634
Location: Pittsfield, Maine

Boring No.: BB-PFB-104
WIN: 26111.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	175.0	**Auger ID/OD:**	--
Operator:	B. Gome	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	HW-140#/30"; SS-140#/30"
Date Start/Finish:	8-21-2023/8-21-2023	**Drilling Method:**	HW to 52 ft	**Core Barrel:**	--
Boring Location:	194+70, 7 ft RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	10.2 ft
Hammer Efficiency Factor: 0.76		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/10	25.0 - 27.0	10/12/11/13	23	29	42	150.0		25.0 Dark grey, wet, medium dense, fine SAND, little silt, trace medium to coarse sand, poorly-graded, (Marine Deposit).	
							51				
							55				
							49				
							53				
30	6D	24/10	30.0 - 32.0	12/19/11/12	30	38	70			Dark grey, wet, dense, fine to medium SAND, some silt, some gravel, poorly-graded, (Marine Deposit).	G#744590 A-2-4, SM
							75				
							68				
							72				
							69				
35	7D	24/2	35.0 - 37.0	9/15/16/24	31	39	65			Similar to 6D, (Marine Deposit).	
							72				
							95	137.6		37.4	
							108				
							115				
40	8D	24/22	40.0 - 42.0	26/40/32/39	72	91				Dark grey, wet, very dense, Sandy fine to coarse GRAVEL, little silt, (Marine Deposit).	
								133.6		41.4 Dark grey, wet, hard, Silty CLAY, (Marine Deposit).	
								131.5		43.5	
45	9D	24/18	45.0 - 47.0	18/24/27/28	51	65				Dark grey, wet, very dense, Sandy fine to coarse GRAVEL, little silt, (Marine Deposit).	
								128.5		46.5 Dark grey, wet, hard, SILT, trace fine sand, (Marine Deposit).	
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: Osborne Bridge No. 2634
Location: Pittsfield, Maine

Boring No.: BB-PFB-104
WIN: 26111.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	175.0	**Auger ID/OD:**	--
Operator:	B. Gome	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	HW-140#/30"; SS-140#/30"
Date Start/Finish:	8-21-2023/8-21-2023	**Drilling Method:**	HW to 52 ft	**Core Barrel:**	--
Boring Location:	194+70, 7 ft RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	10.2 ft
Hammer Efficiency Factor:	0.76	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/16	50.0 - 52.0	48/88/62/49	150	190		125.0		50.0 Dark grey, wet, very dense, fine to coarse SAND, some fine to coarse gravel, trace silt, well-graded, (Glacial Till). Note: Roller bit split. Unable to advance boring after 10D sample.	
								123.2		51.8	
								123.0		Dark grey, wet, very dense, fine to coarse SAND, trace silt, poorly-graded, moderately bonded, (Glacial Till). 52.0	
										Bottom of Exploration at 52.0 feet below ground surface. No Refusal	
55											
60											
65											
70											
75											

Remarks:

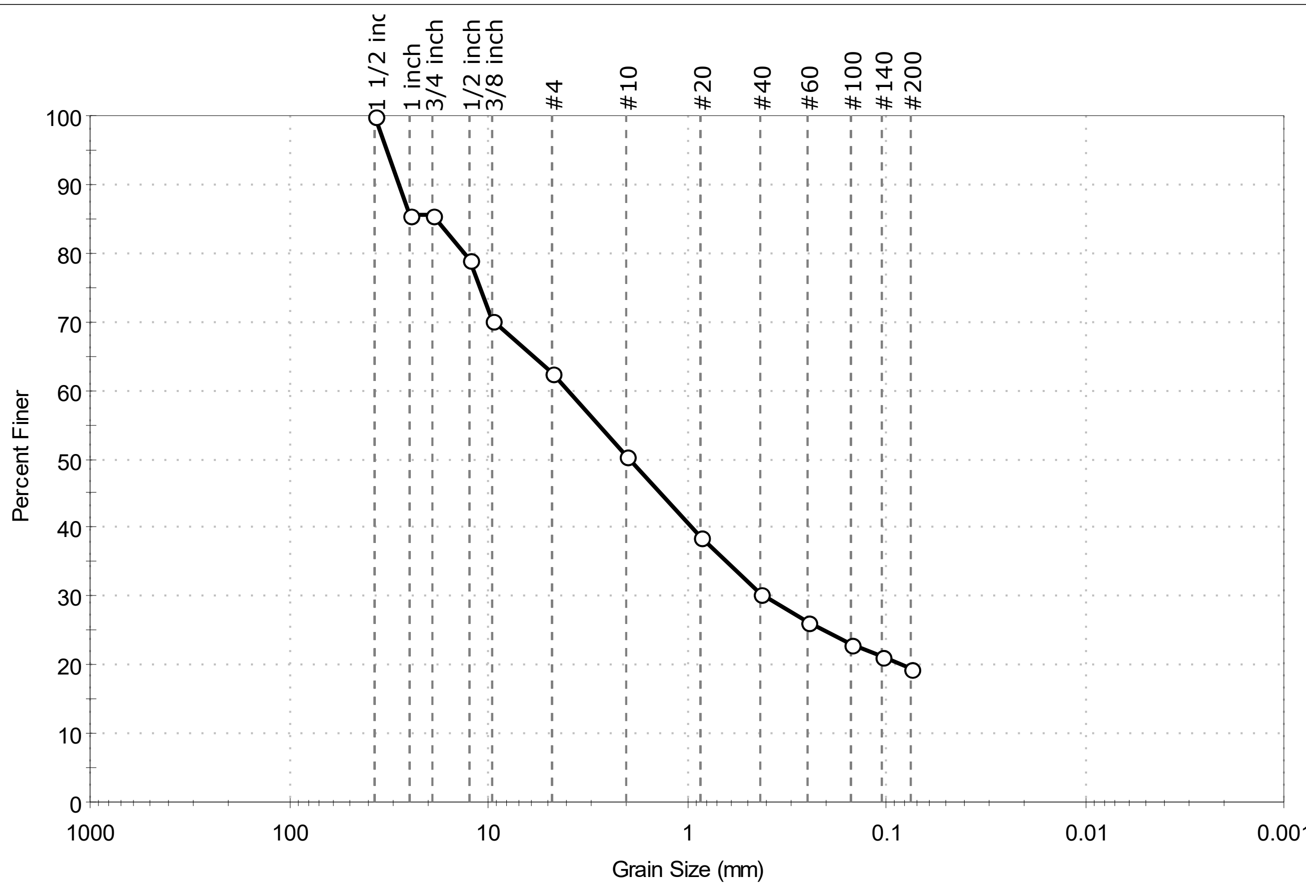
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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

APPENDIX B
Laboratory Test Results

Client:	Haley & Aldrich, Inc.				
Project:	Osborne Bridge Culvert Replace				
Location:	Pittsfield, ME			Project No:	GTX-318151
Boring ID:	BB-PFB-103	Sample Type:	jar	Tested By:	ckg
Sample ID:	1D	Test Date:	11/21/23	Checked By:	jsc
Depth :	5-7'	Test Id:	744587		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	37.4	43.3	19.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	85		
3/4 inch	19.00	85		
1/2 inch	12.50	79		
3/8 inch	9.50	70		
#4	4.75	63		
#10	2.00	50		
#20	0.85	39		
#40	0.42	30		
#60	0.25	26		
#100	0.15	23		
#140	0.11	21		
#200	0.075	19		

Coefficients

D_{85} =18.4522 mm	D_{30} =0.4019 mm
D_{60} =3.9398 mm	D_{15} =N/A
D_{50} =1.9290 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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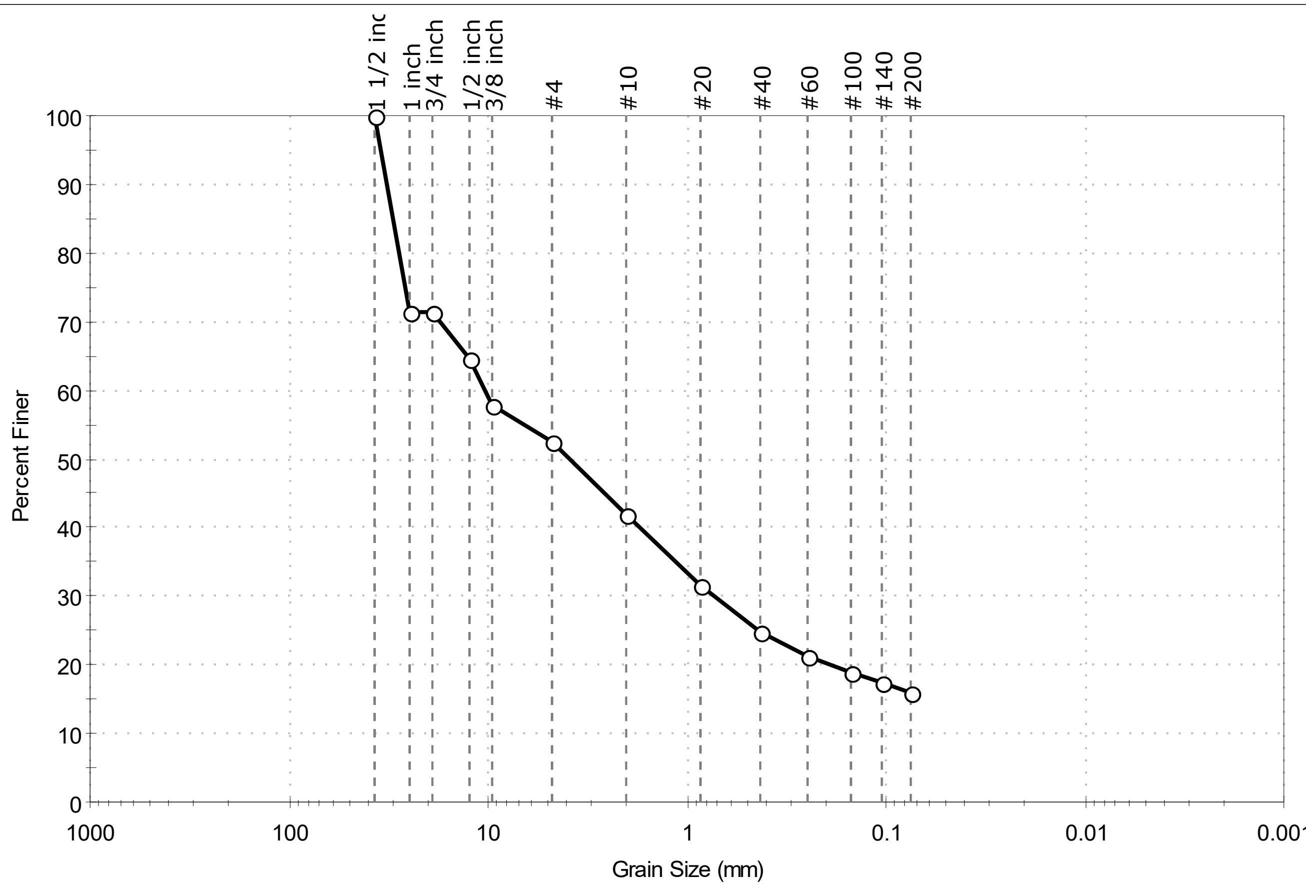
Client:	Haley & Aldrich, Inc.		
Project:	Osborne Bridge Culvert Replace		
Location:	Pittsfield, ME	Project No:	GTX-318151
Boring ID: BB-PFB-103	Sample Type: jar	Tested By:	ckg
Sample ID: 2D	Test Date: 11/21/23	Checked By:	jsc
Depth : 7-9'	Test Id: 744588		

Test Comment: ---
Visual Description: Moist, dark brown silty gravel with sand
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	47.6	36.4	16.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	71		
3/4 inch	19.00	71		
1/2 inch	12.50	65		
3/8 inch	9.50	58		
#4	4.75	52		
#10	2.00	42		
#20	0.85	32		
#40	0.42	25		
#60	0.25	21		
#100	0.15	19		
#140	0.11	17		
#200	0.075	16		

Coefficients

D_{85} =30.2976 mm | D_{30} =0.7247 mm
D_{60} =10.3468 mm | D_{15} =N/A
D_{50} =3.8962 mm | D_{10} =N/A
C_u =N/A | C_c =N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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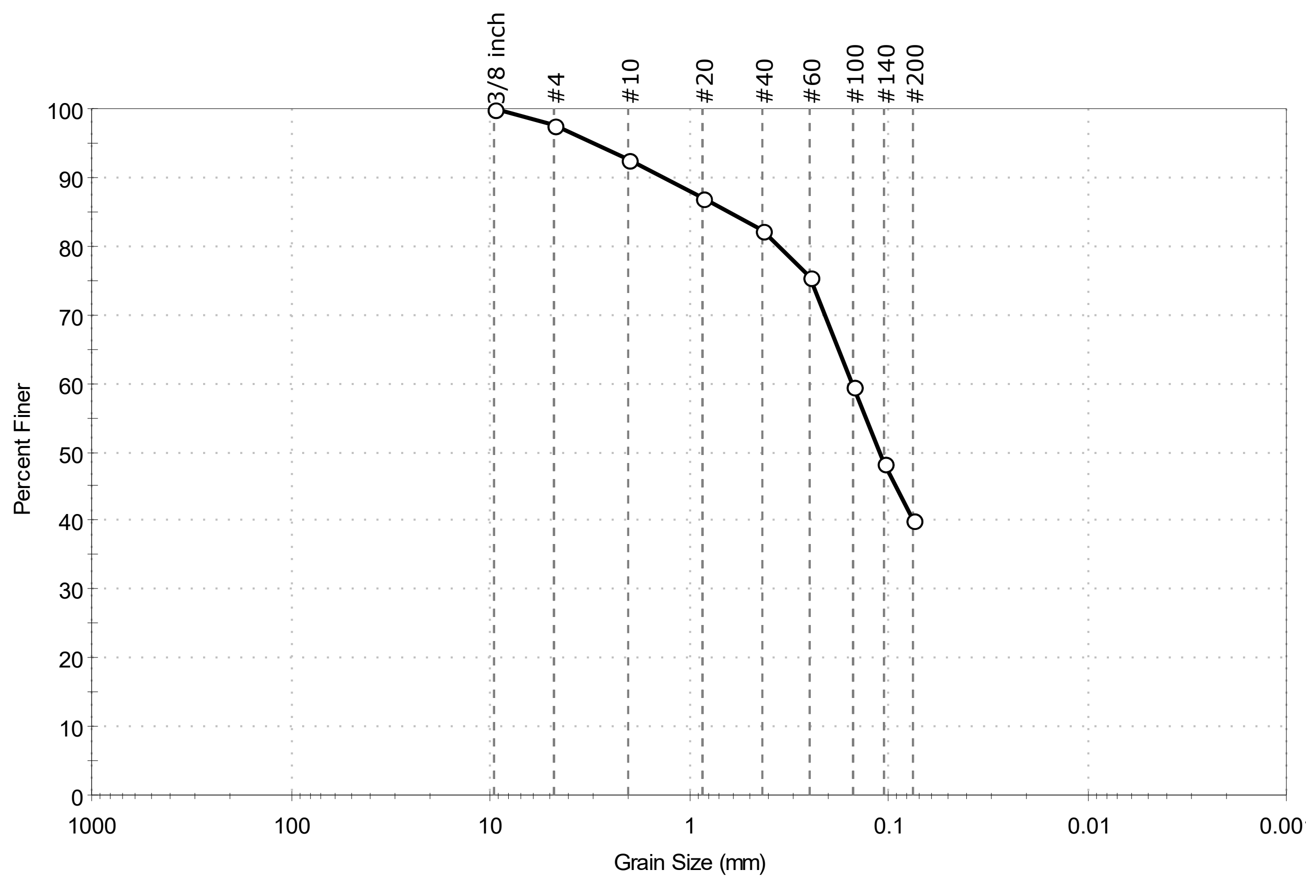
GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.		
Project:	Osborne Bridge Culvert Replace		
Location:	Pittsfield, ME	Project No:	GTX-318151
Boring ID:	BB-PFB-103	Sample Type: jar	Tested By: ckg
Sample ID:	7D	Test Date: 11/21/23	Checked By: jsc
Depth :	25-27'	Test Id: 744589	
Test Comment:	---		
Visual Description:	Moist, gray silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	2.4	57.3	40.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/8 inch	9.50	100		
#4	4.75	98		
#10	2.00	93		
#20	0.85	87		
#40	0.42	82		
#60	0.25	75		
#100	0.15	60		
#140	0.11	48		
#200	0.075	40		

Coefficients

D_{85} =0.6285 mm D_{30} =N/A
D_{60} =0.1524 mm D_{15} =N/A
D_{50} =0.1112 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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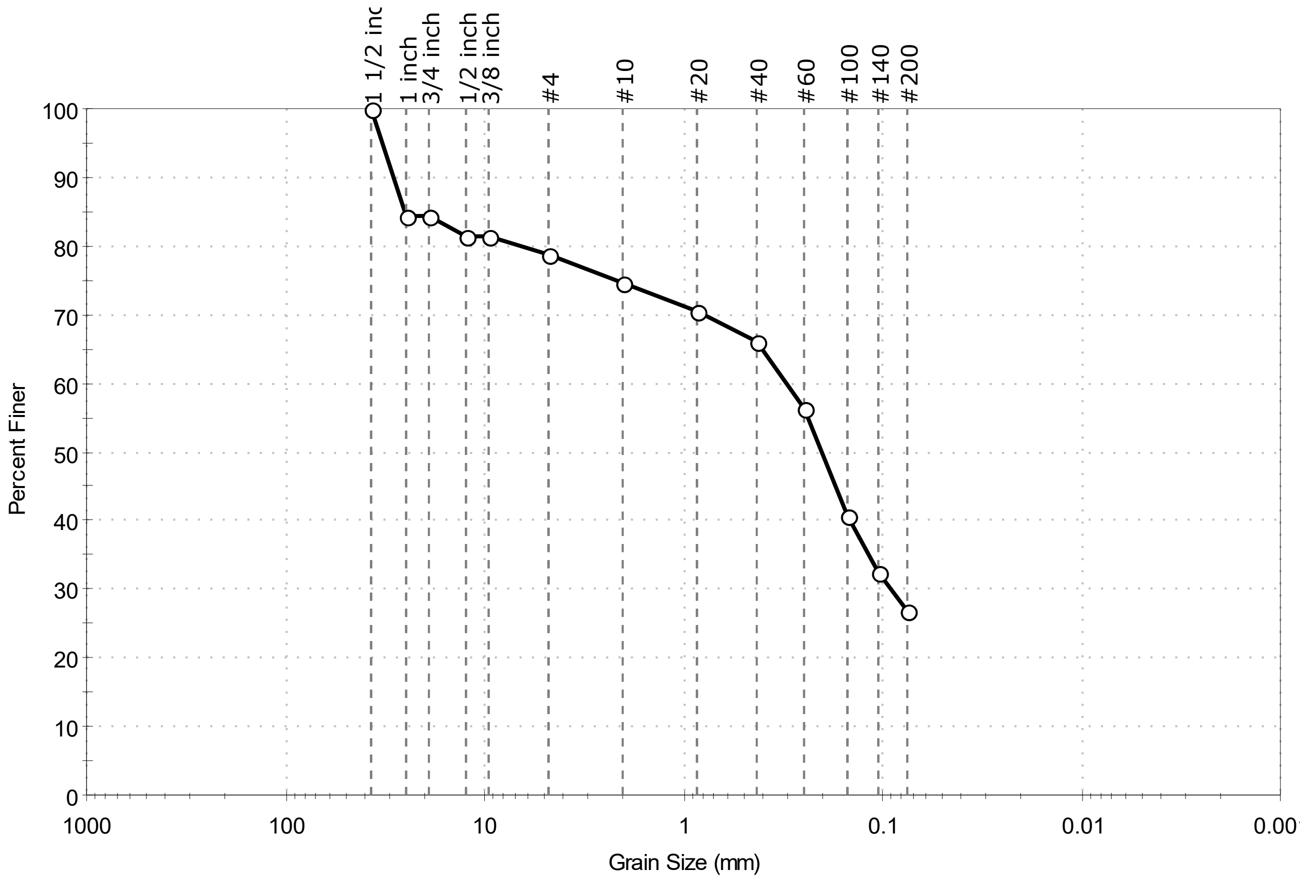
Client: Haley & Aldrich, Inc.			
Project: Osborne Bridge Culvert Replace			
Location: Pittsfield, ME		Project No:	GTX-318151
Boring ID: BB-PFB-104	Sample Type: jar	Tested By:	ckg
Sample ID: 6D	Test Date: 11/21/23	Checked By:	jsc
Depth : 30-32'	Test Id: 744590		
Test Comment: ---			
Visual Description: Moist, gray silty sand with gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	21.3	51.9	26.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	84		
3/4 inch	19.00	84		
1/2 inch	12.50	82		
3/8 inch	9.50	82		
#4	4.75	79		
#10	2.00	75		
#20	0.85	71		
#40	0.42	66		
#60	0.25	56		
#100	0.15	41		
#140	0.11	32		
#200	0.075	27		

Coefficients

D_{85} =25.4511 mm
D_{30} =0.0916 mm
D_{60} =0.3047 mm
D_{15} =N/A
D_{50} =0.2028 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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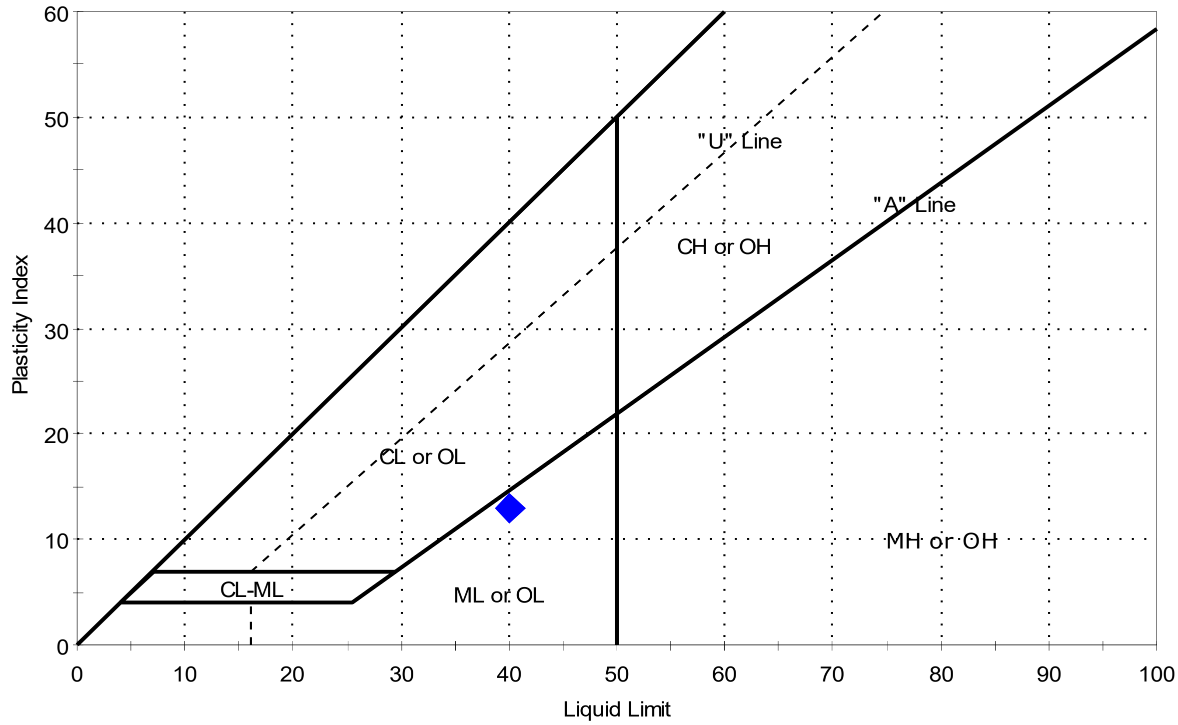
Client:	Haley & Aldrich, Inc.		
Project:	Osborne Bridge Culvert Replace		
Location:	Pittsfield, ME	Project No:	GTX-318151
Boring ID: BB-PFB-103	Sample Type: jar	Tested By:	cam
Sample ID: 5D	Test Date: 11/27/23	Checked By:	jsc
Depth : 15-17'	Test Id: 744583		
Test Comment:	---		
Visual Description:	Moist, gray silt		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	5D	B-PFB-10	15-17'	30	40	27	13	0.2	

Sample Prepared using the WET method

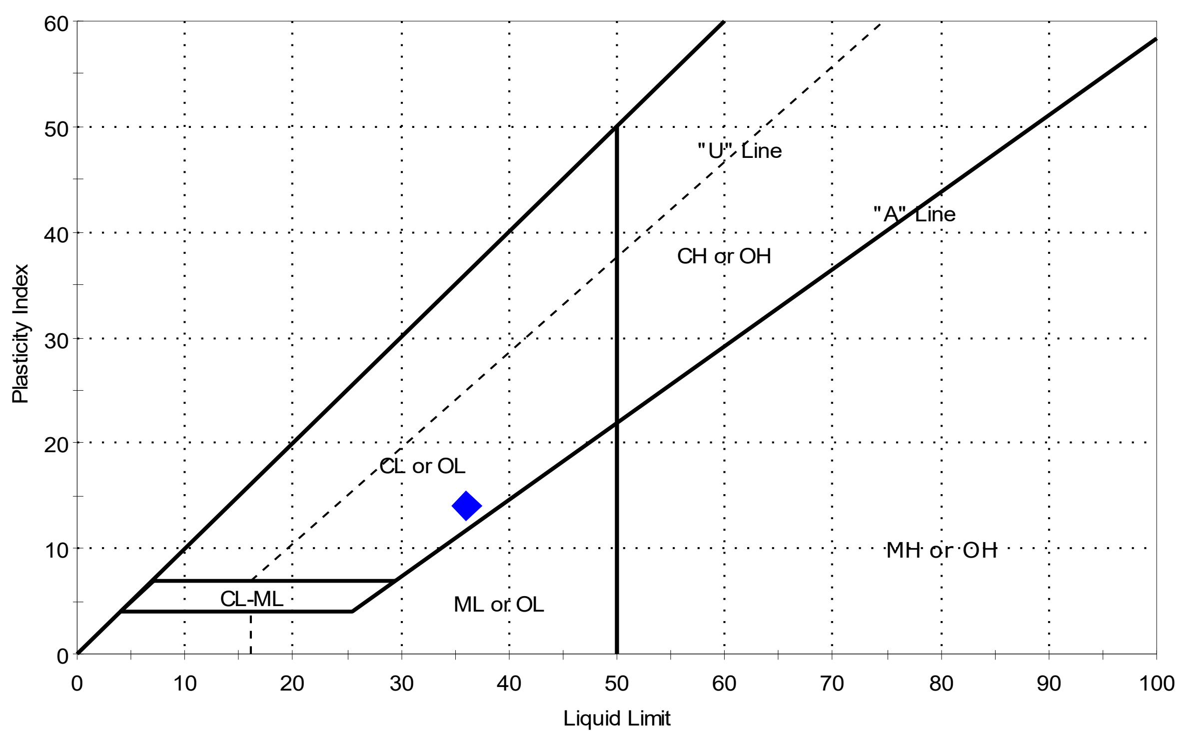
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.
Project:	Osborne Bridge Culvert Replace
Location:	Pittsfield, ME
Project No:	GTX-318151
Boring ID:	BB-PFB-104
Sample Type:	jar
Tested By:	cam
Sample ID:	3D
Test Date:	11/27/23
Checked By:	jsc
Depth :	10-12'
Test Id:	744584
Test Comment:	---
Visual Description:	Moist, brownish gray clay
Sample Comment:	---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-PFB-10	10-12'	34	36	22	14	0.9	

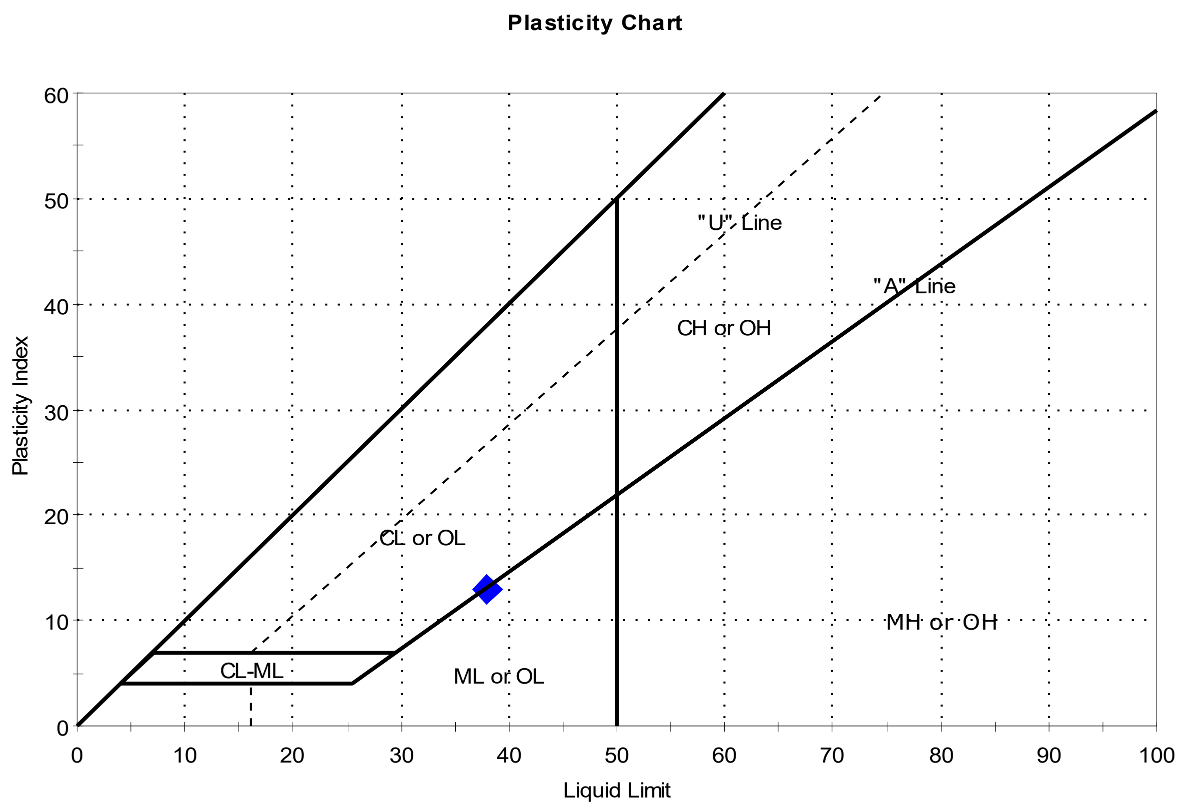
Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

GeoTesting EXPRESS

Client: Haley & Aldrich, Inc.			
Project: Osborne Bridge Culvert Replace			
Location: Pittsfield, ME		Project No:	GTX-318151
Boring ID: BB-PFB-104	Sample Type: jar	Tested By:	cam
Sample ID: 4D	Test Date: 11/27/23	Checked By:	jsc
Depth : 12-14'	Test Id: 744585		
Test Comment: ---			
Visual Description: Moist, gray silty clay			
Sample Comment: ---			

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-PFB-10	12-14'	32	38	25	13	0.6	

Sample Prepared using the WET method

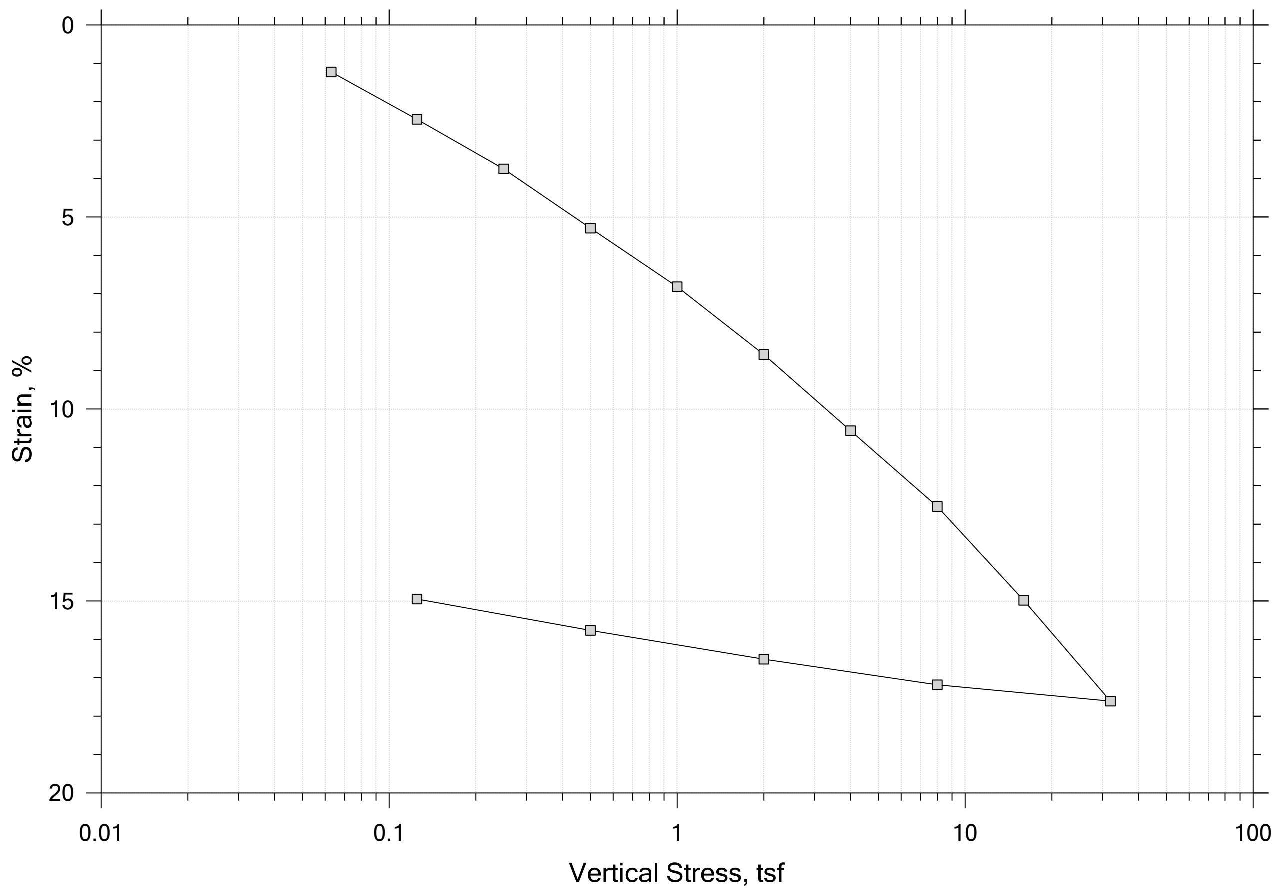
Dry Strength: VERY HIGH

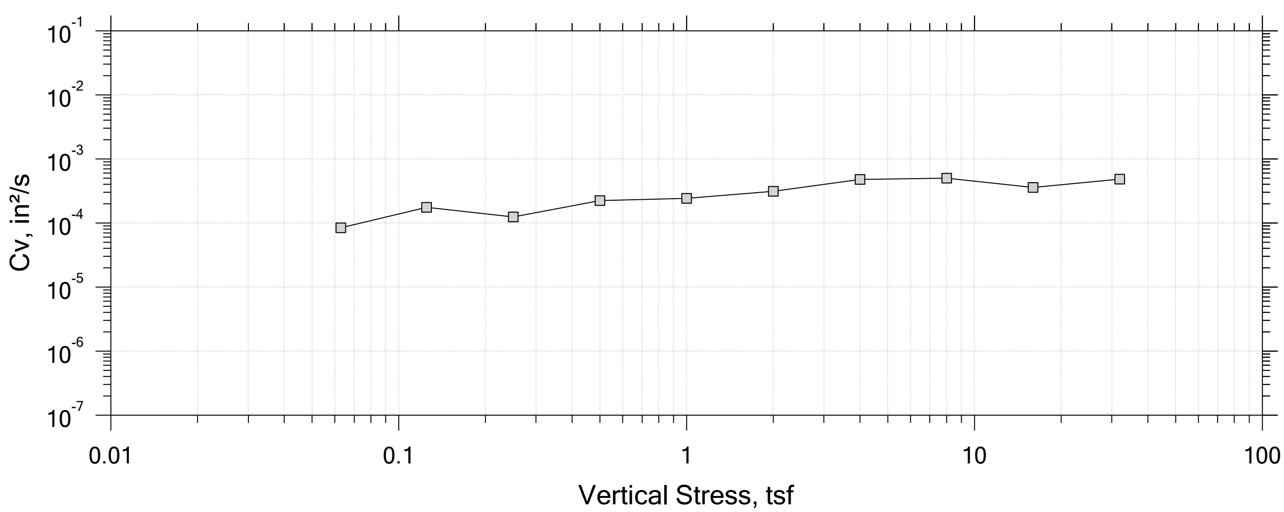
Dilatancy: SLOW

Toughness: LOW

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report





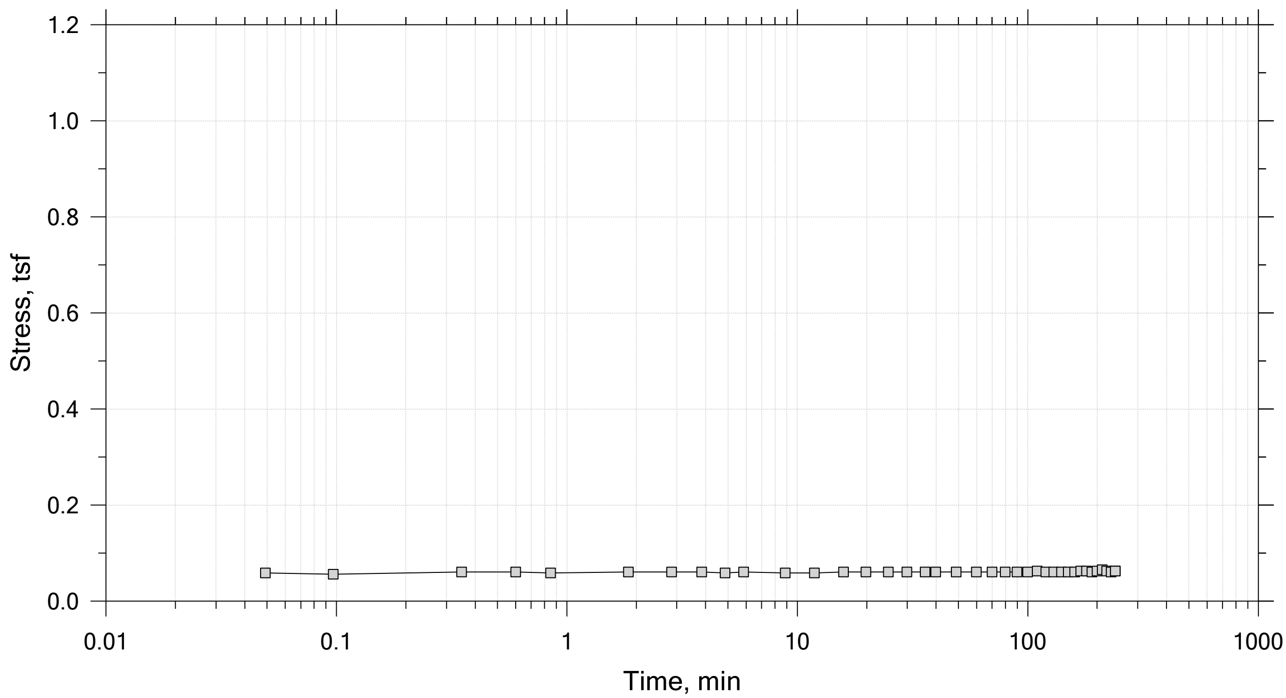
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		
	Displacement at End of Increment		

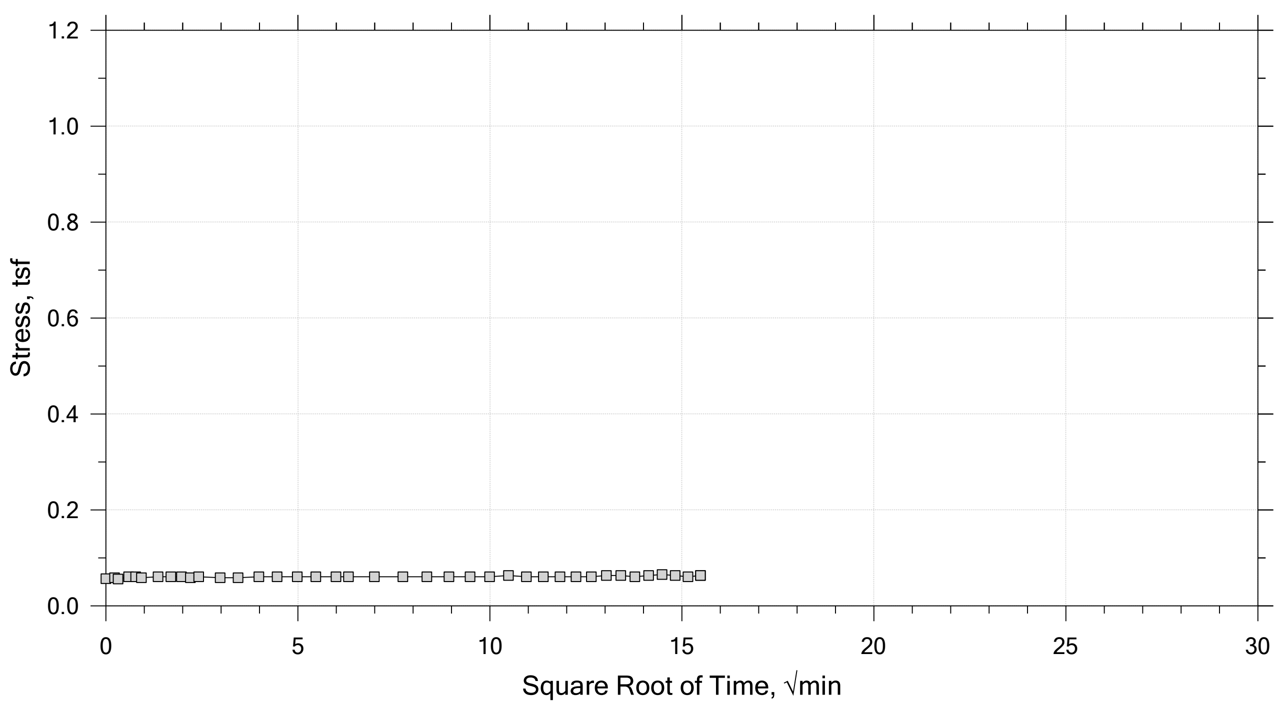
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 14

Constant Volume Step

Stress: 0.0629 tsf





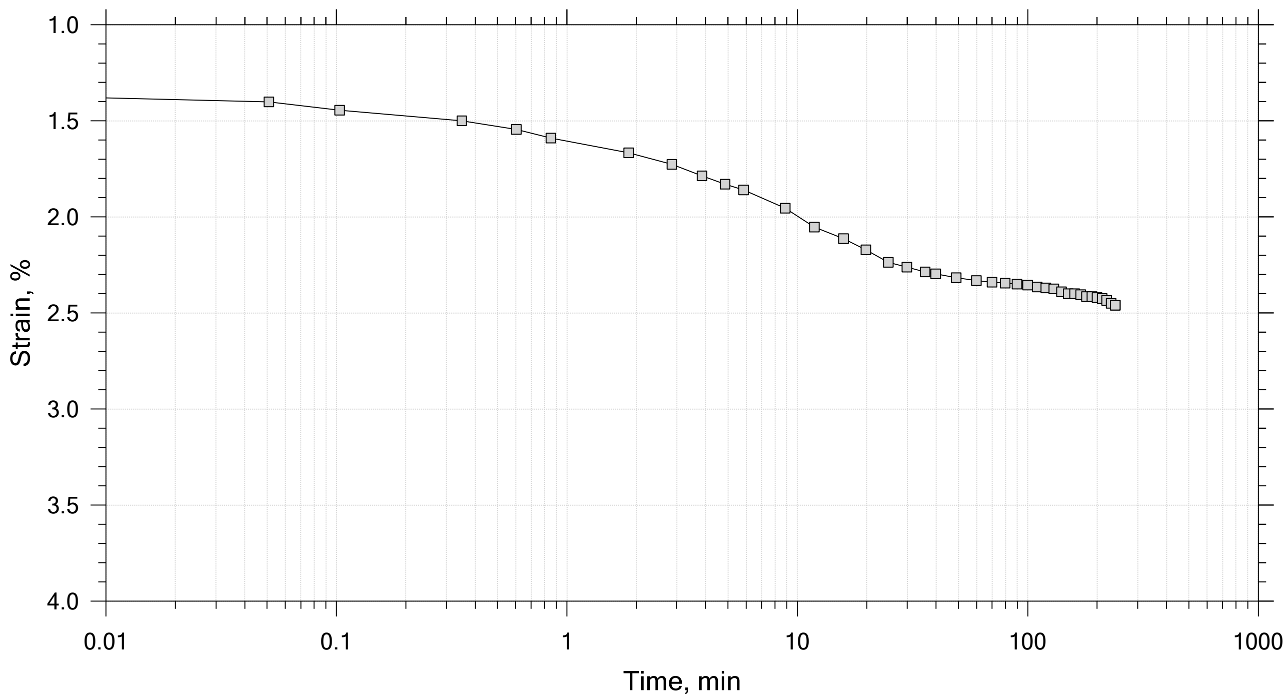
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

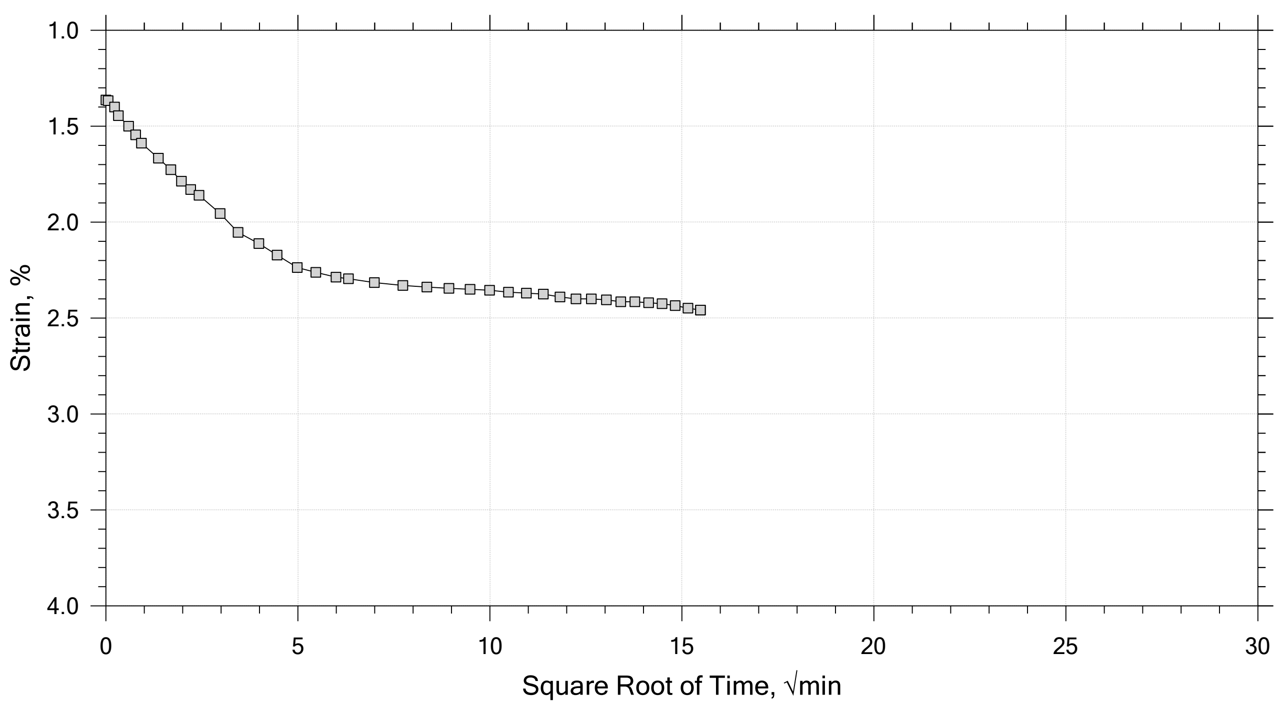
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14

Constant Load Step

Stress: 0.125 tsf





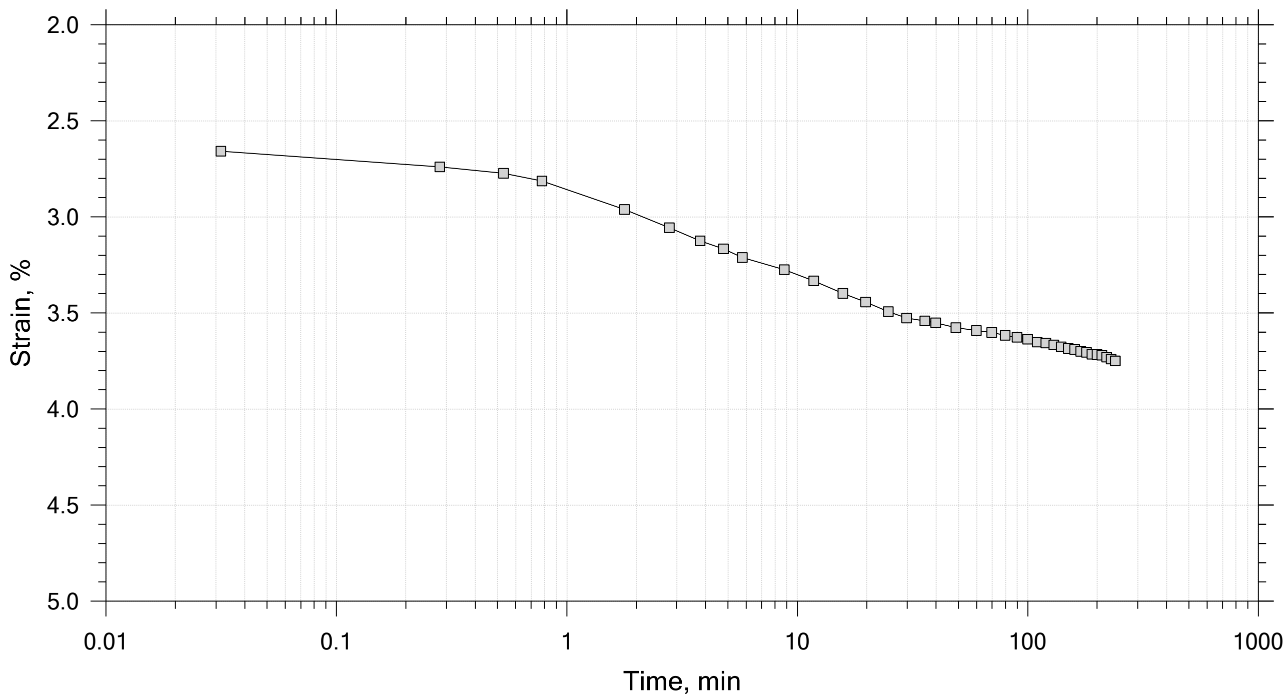
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

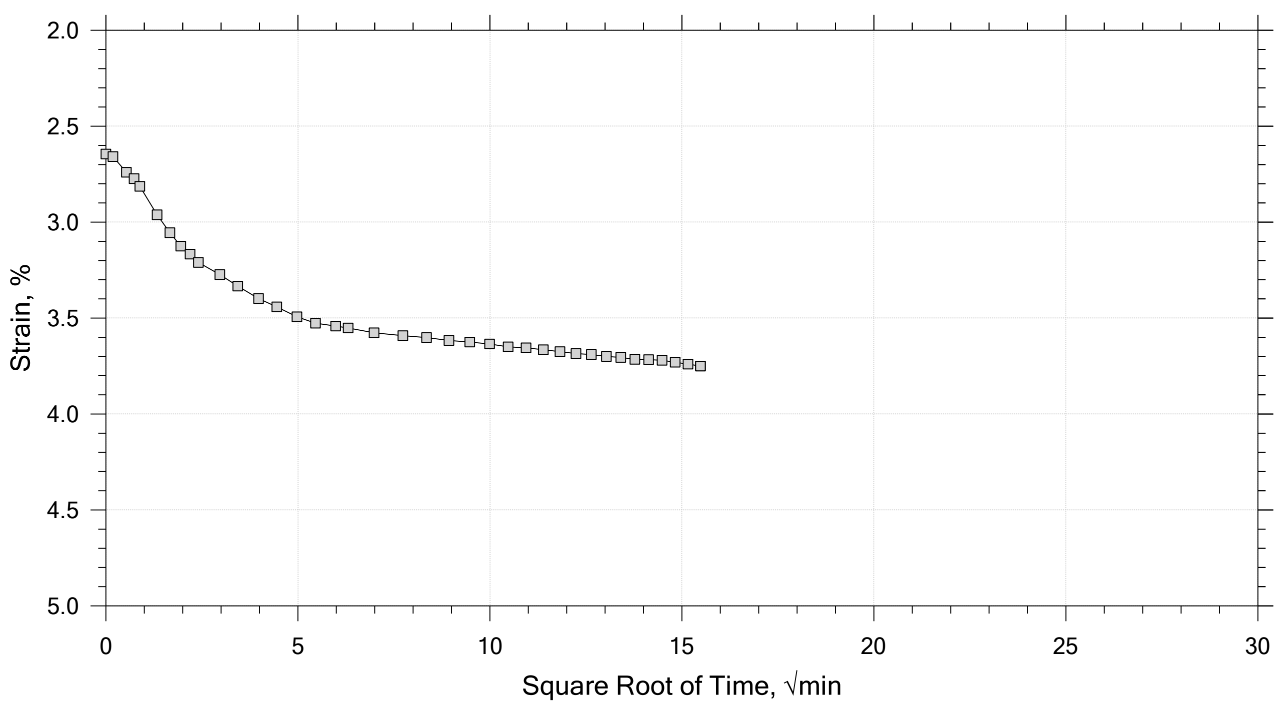
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14

Constant Load Step

Stress: 0.25 tsf





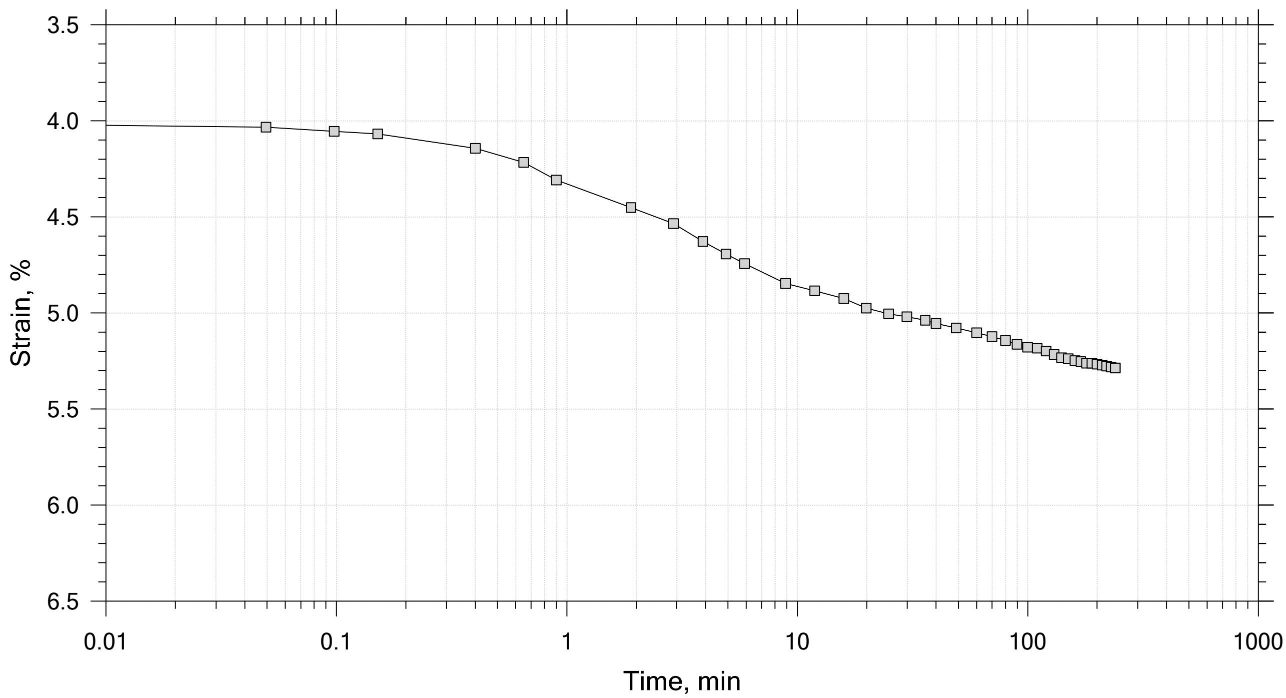
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

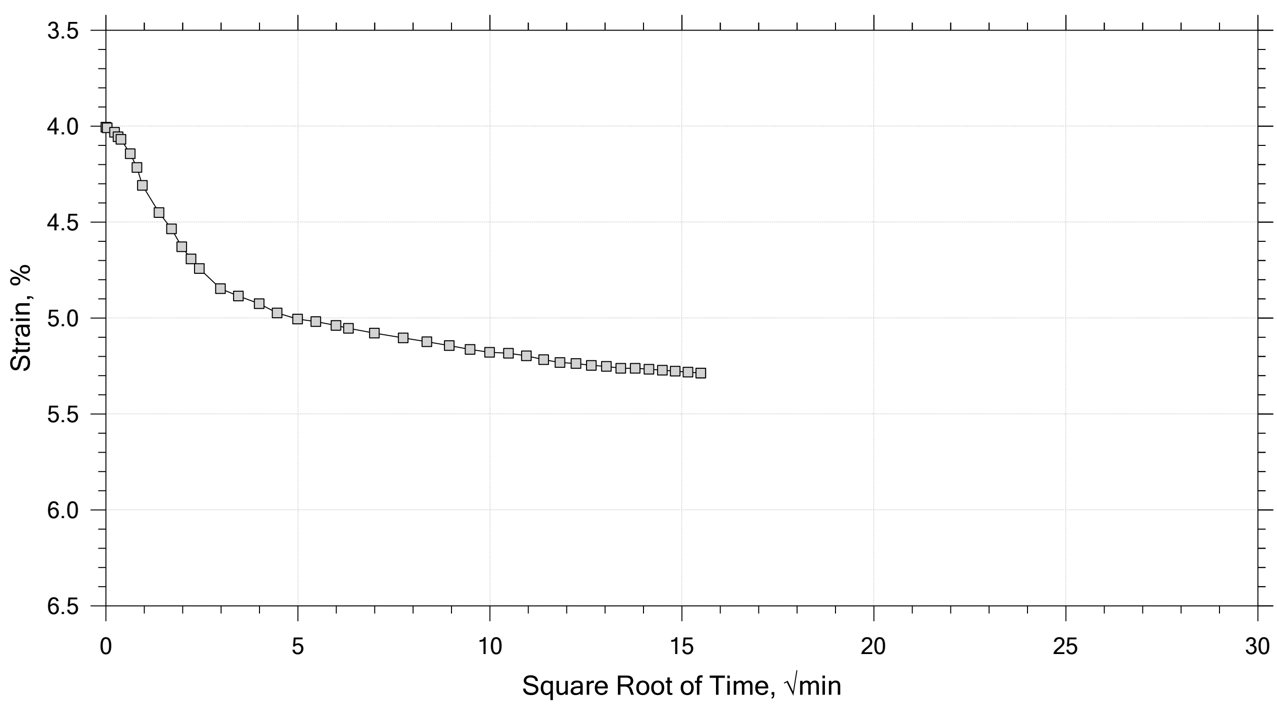
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14

Constant Load Step

Stress: 0.5 tsf





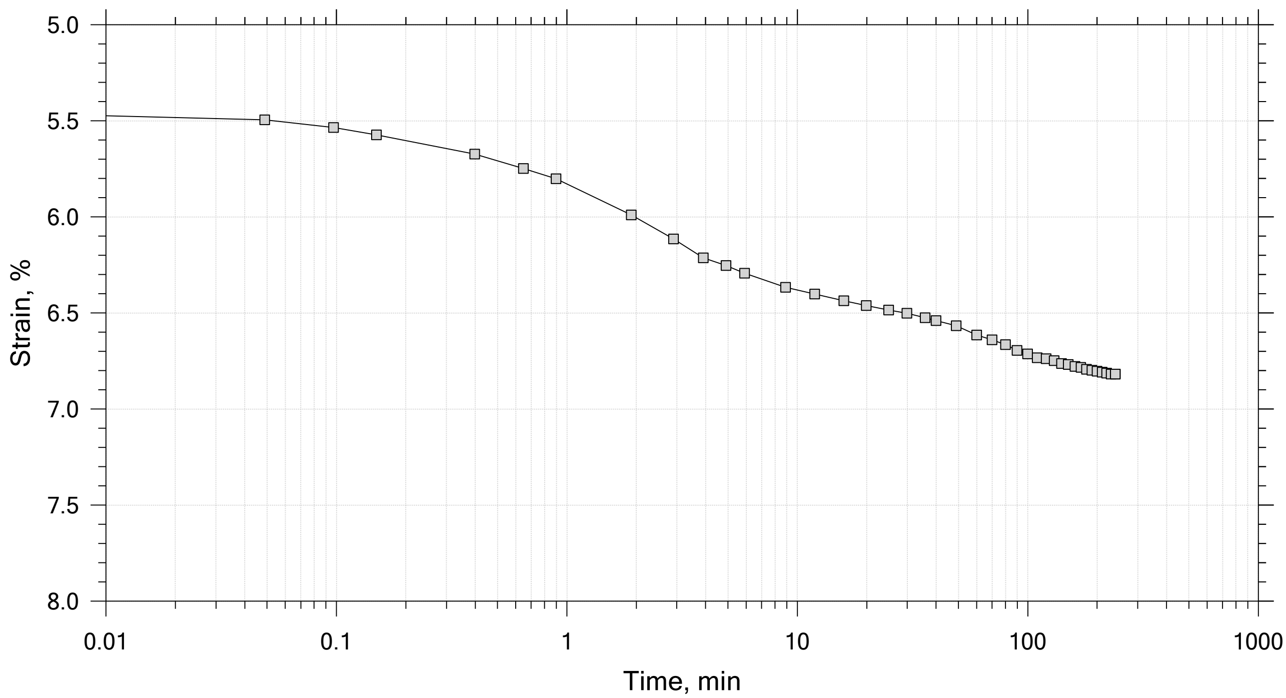
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

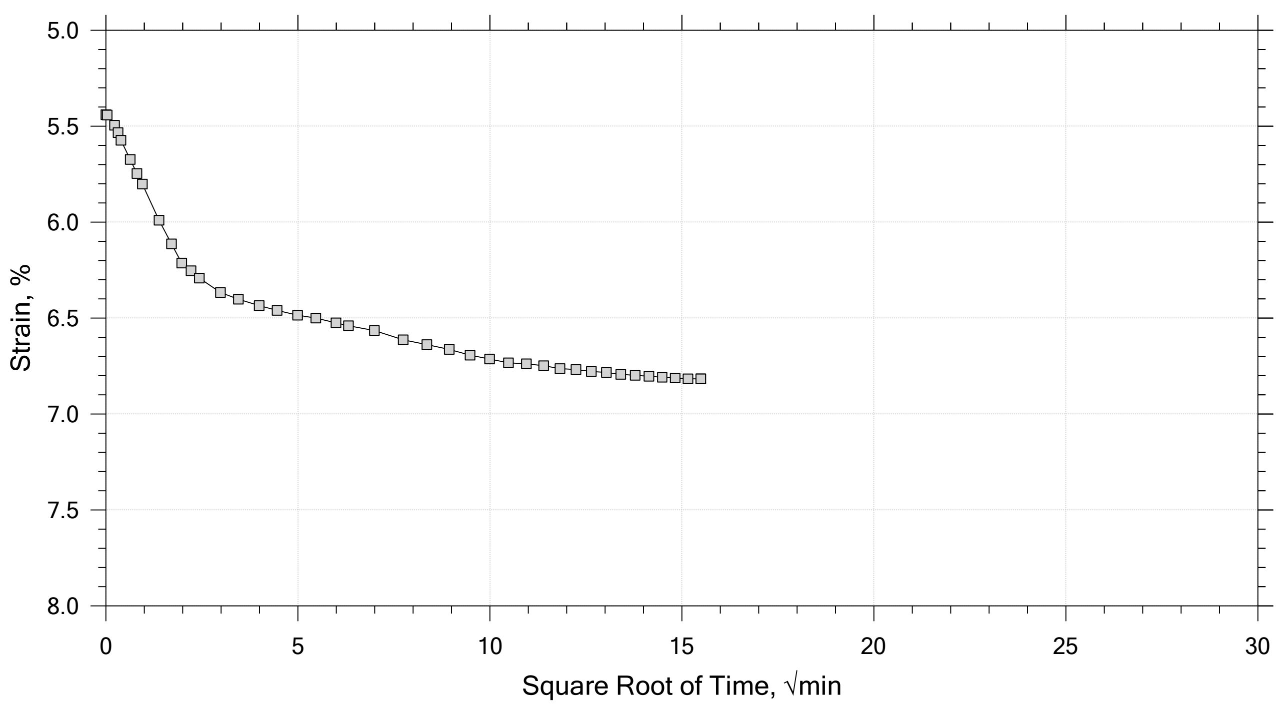
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14

Constant Load Step

Stress: 1 tsf

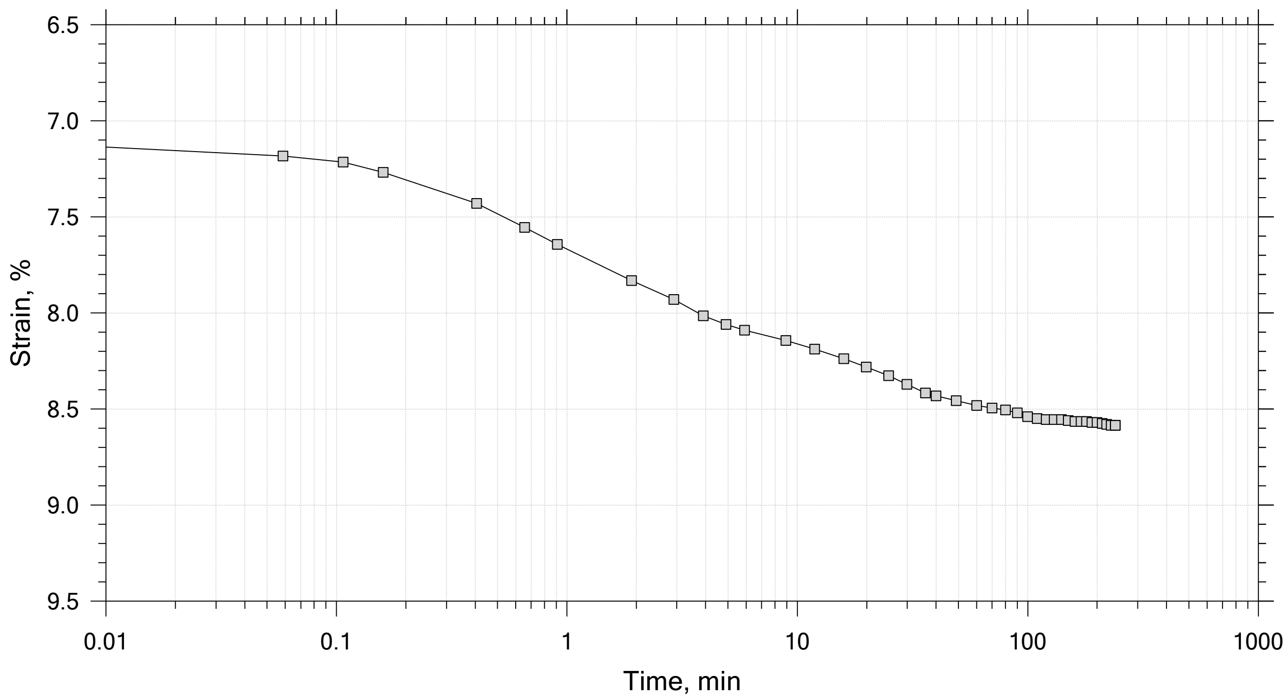


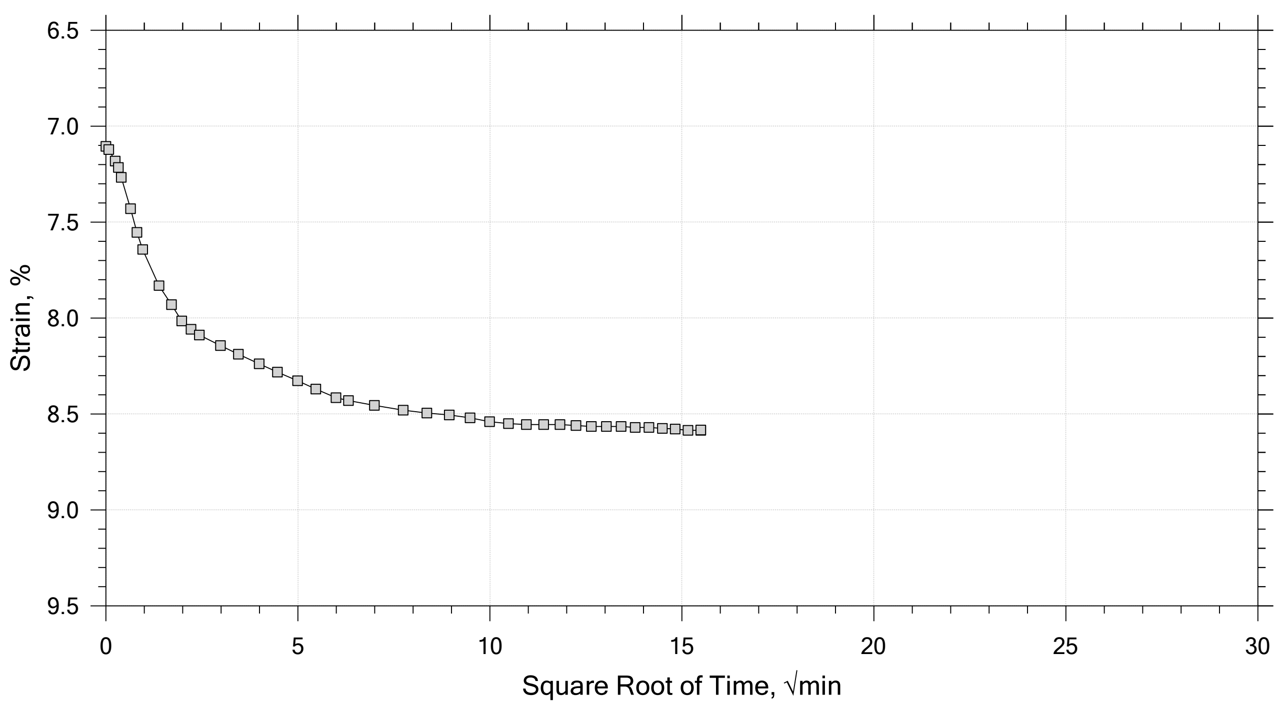


GeoTesting EXPRESS A Sercel Business			
	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14
Constant Load Step
Stress: 2 tsf





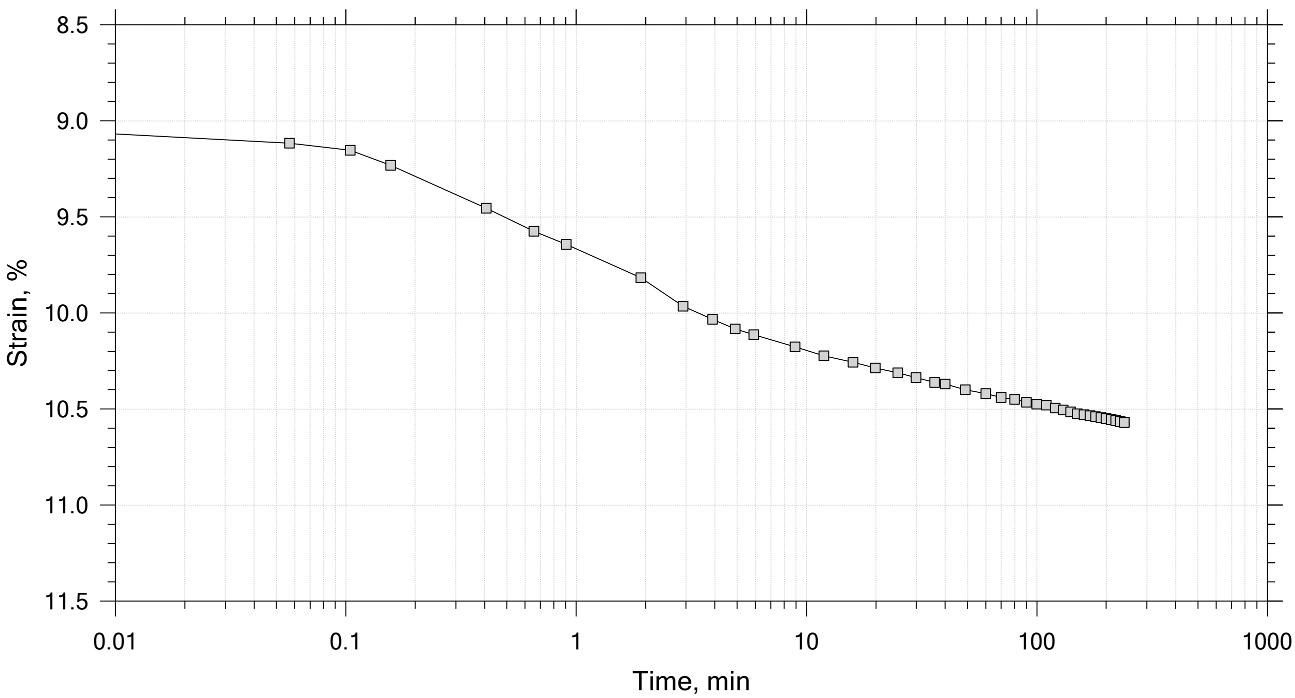
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

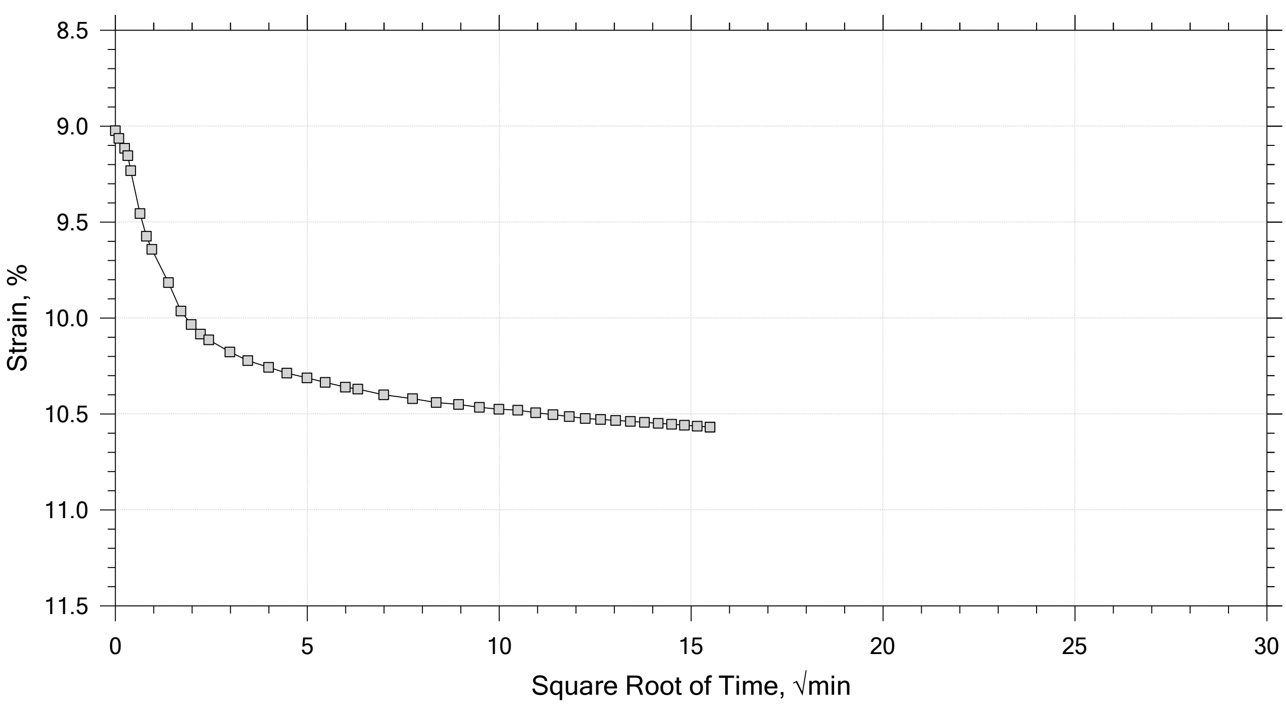
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14

Constant Load Step

Stress: 4 tsf





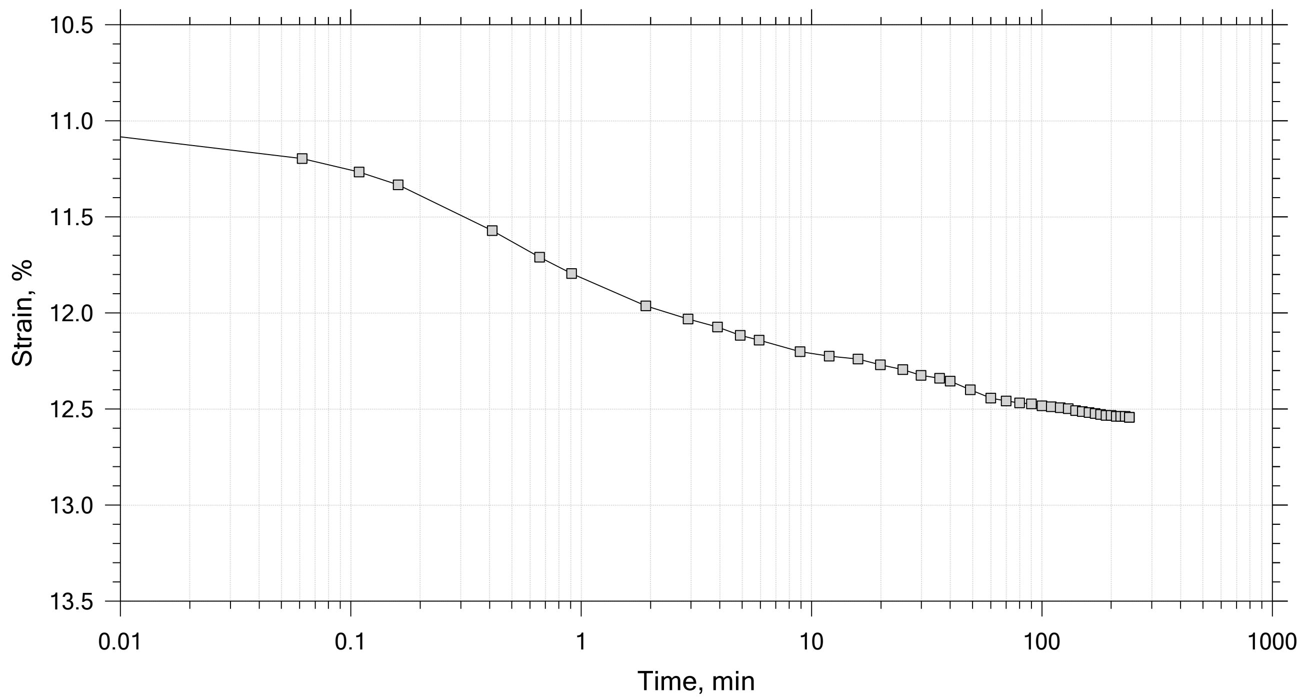
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

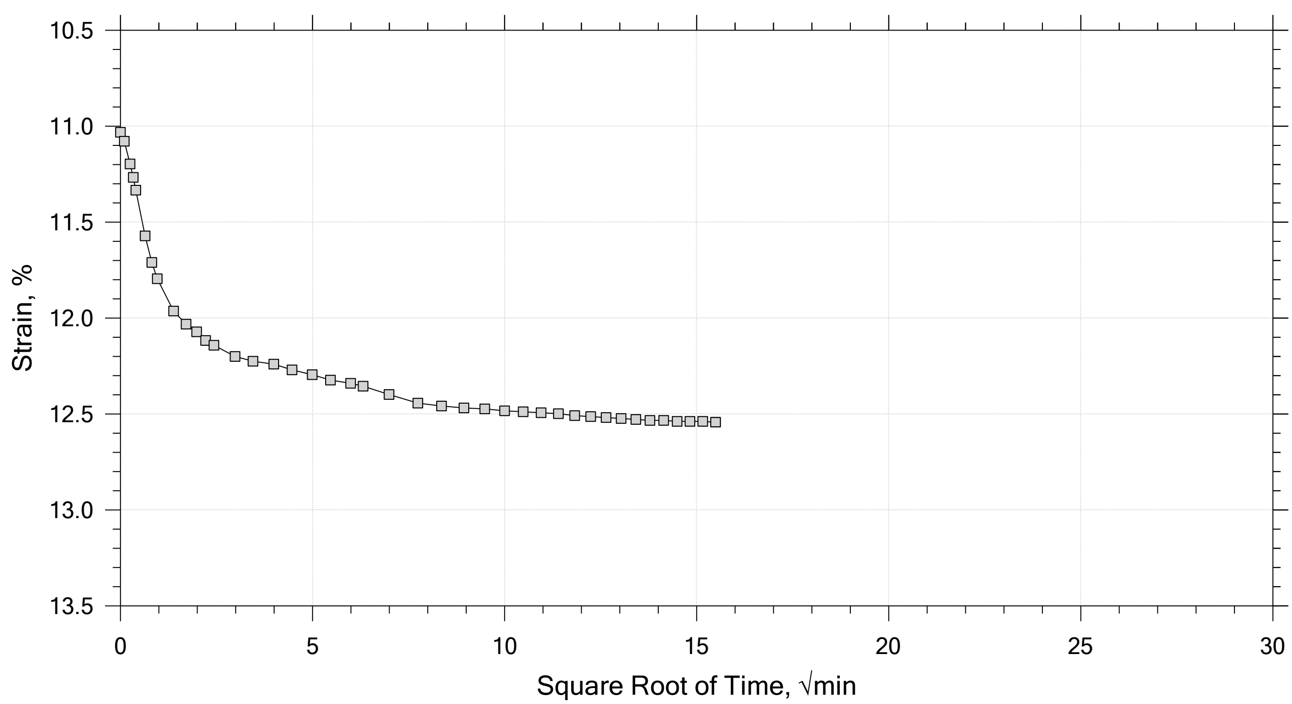
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 8 tsf





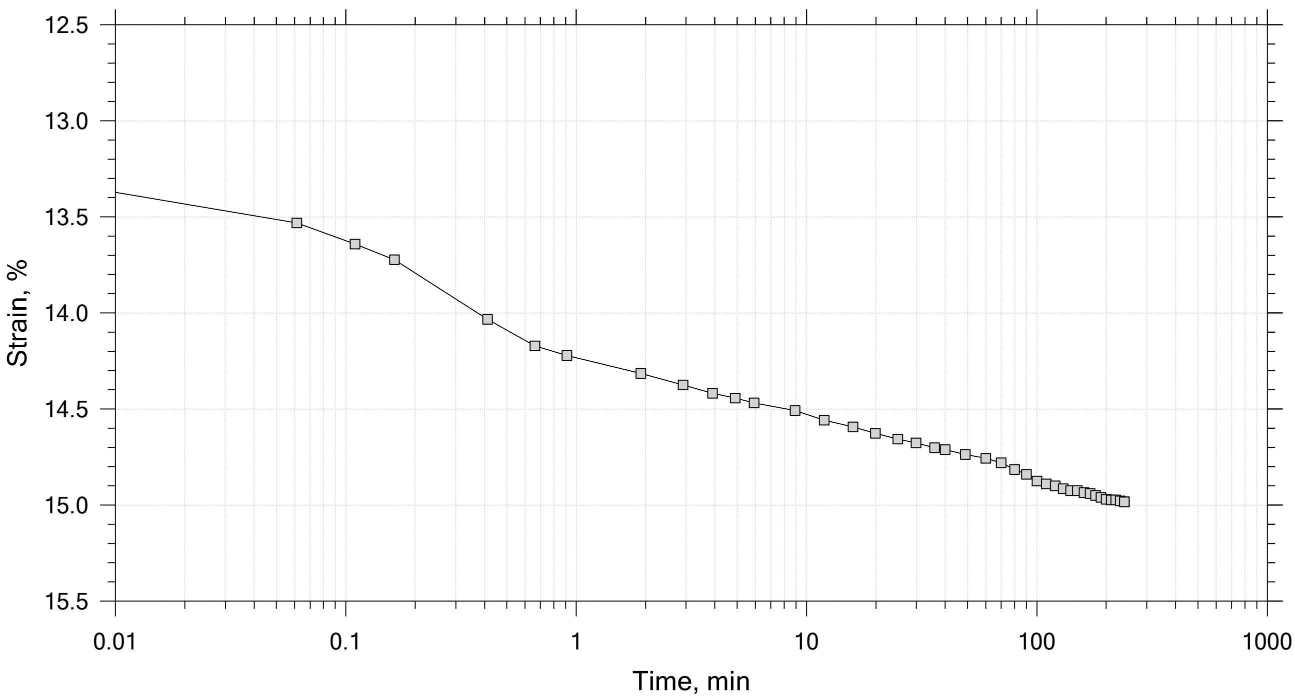
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

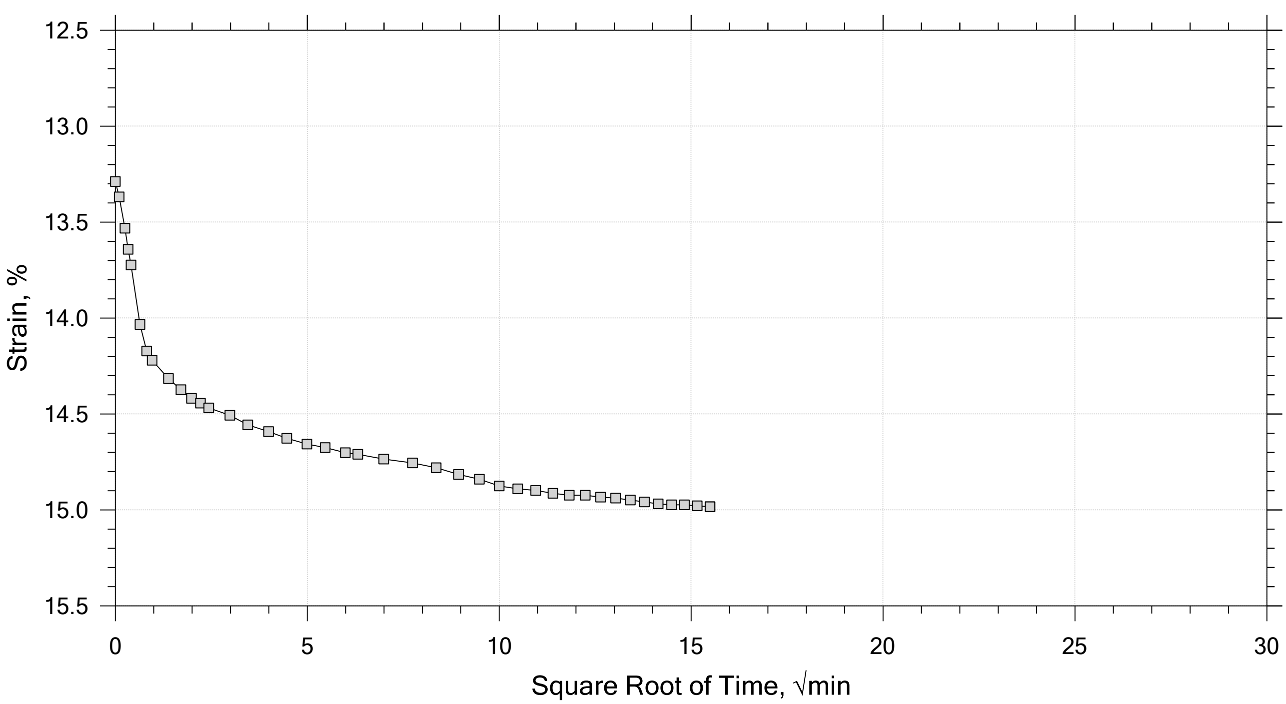
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14

Constant Load Step

Stress: 16 tsf





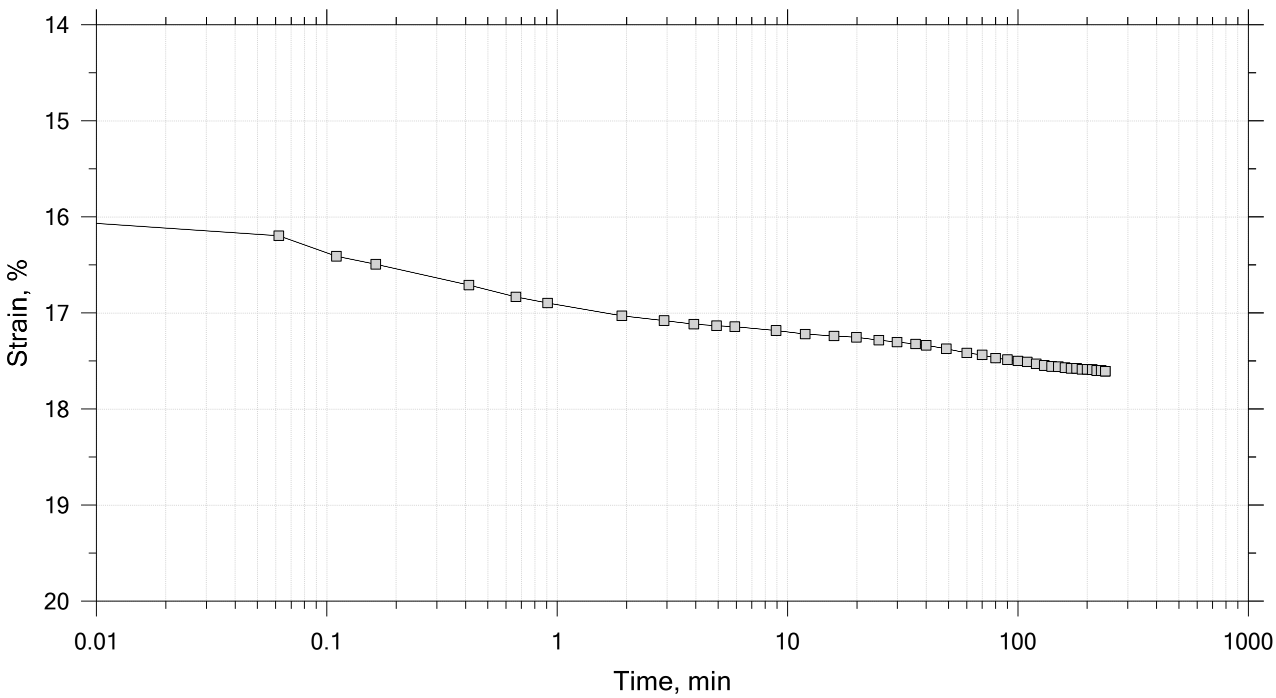
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

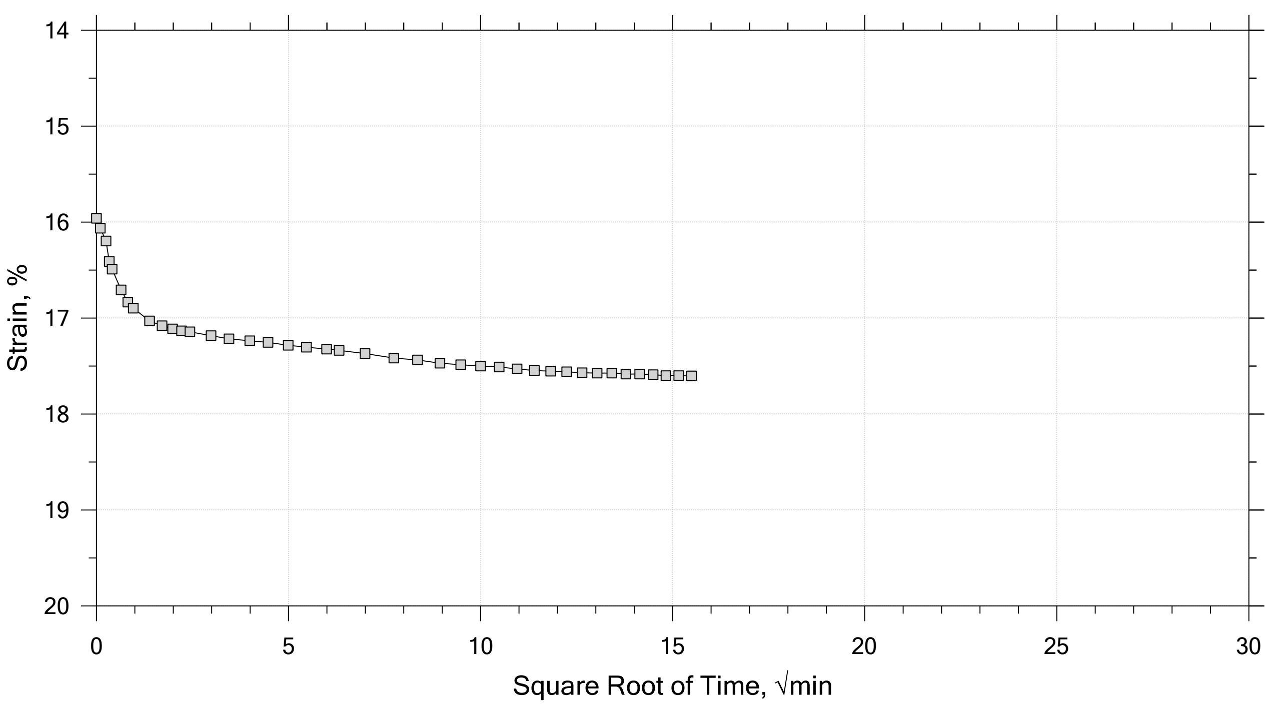
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14

Constant Load Step

Stress: 32 tsf





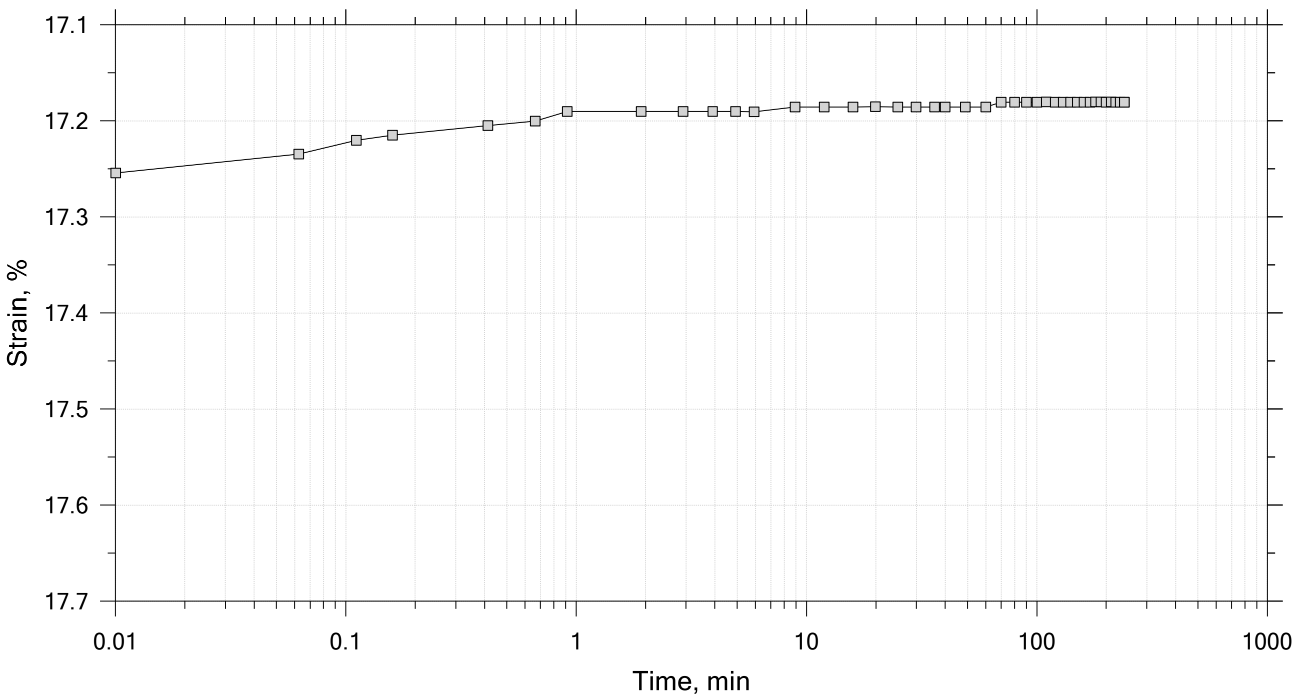
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

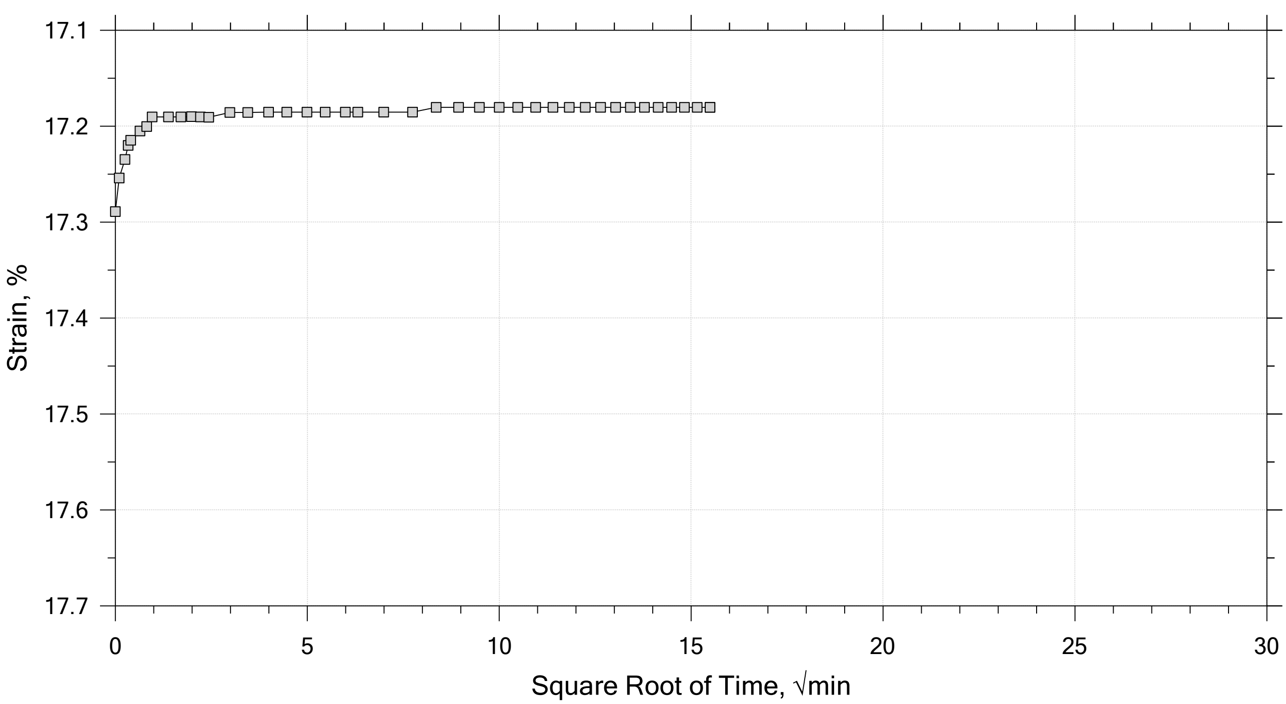
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14

Constant Load Step

Stress: 8 tsf





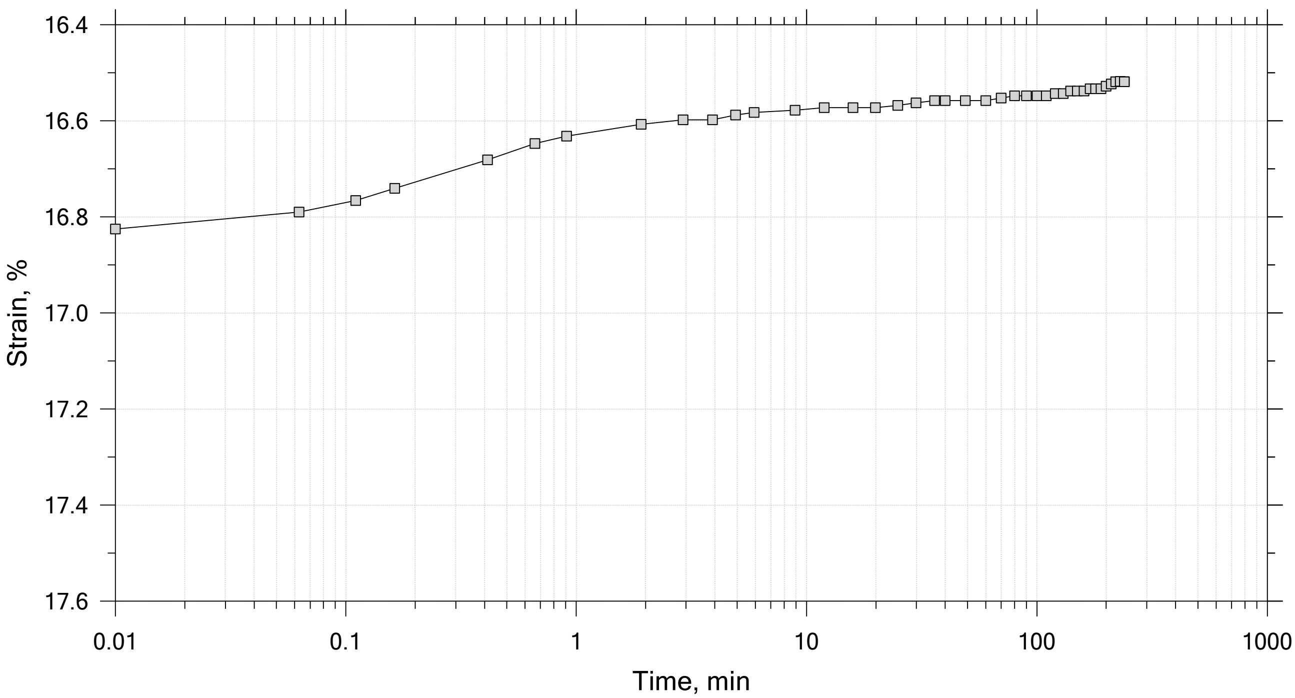
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

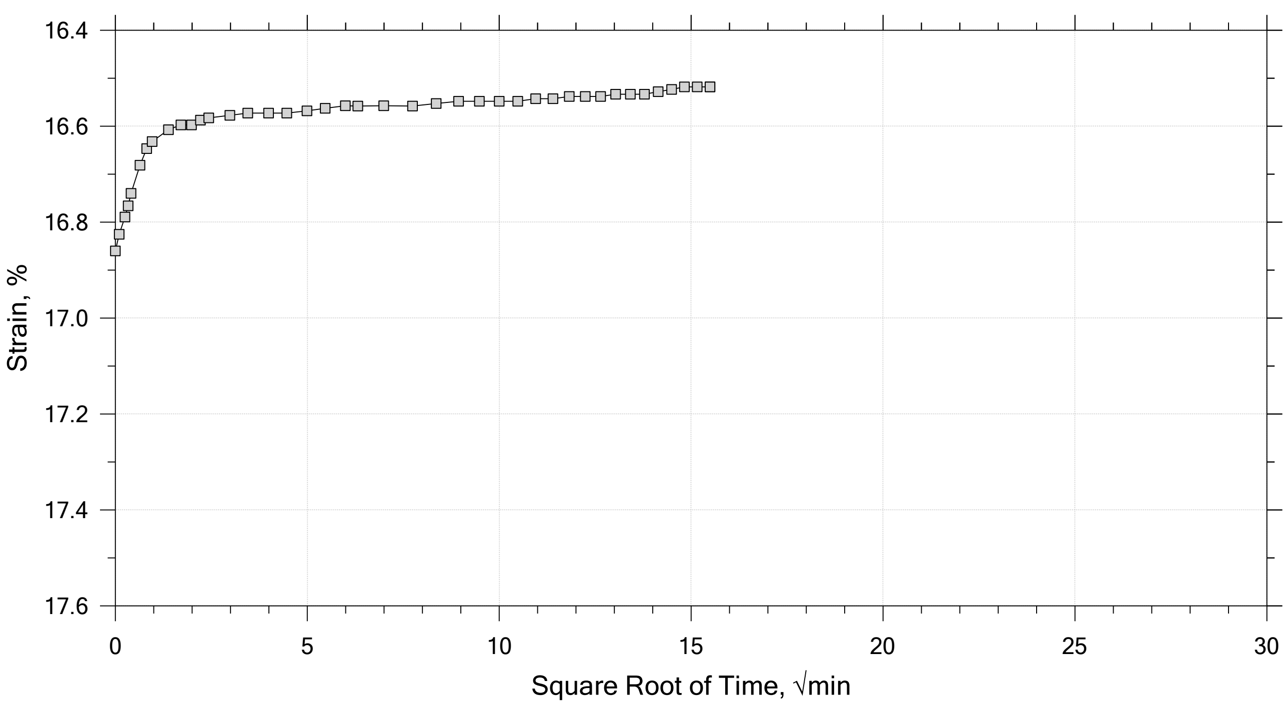
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14

Constant Load Step

Stress: 2 tsf





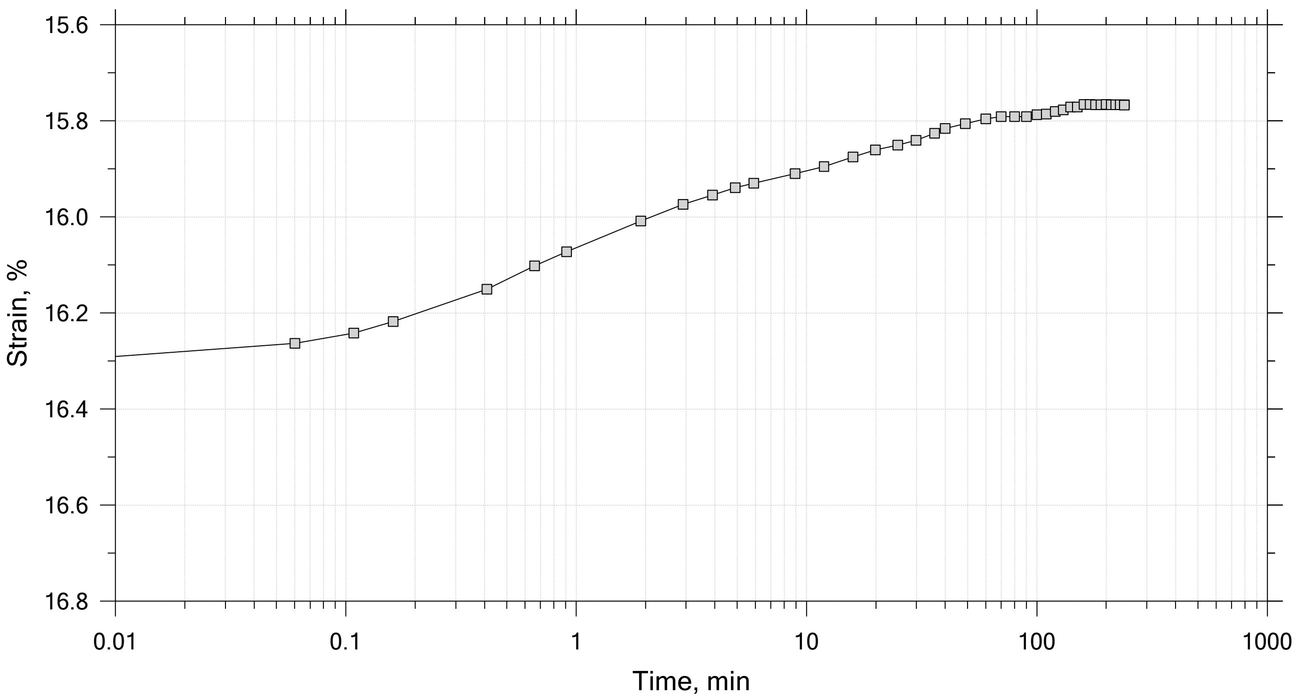
	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
GeoTesting EXPRESS A Sercel Business	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

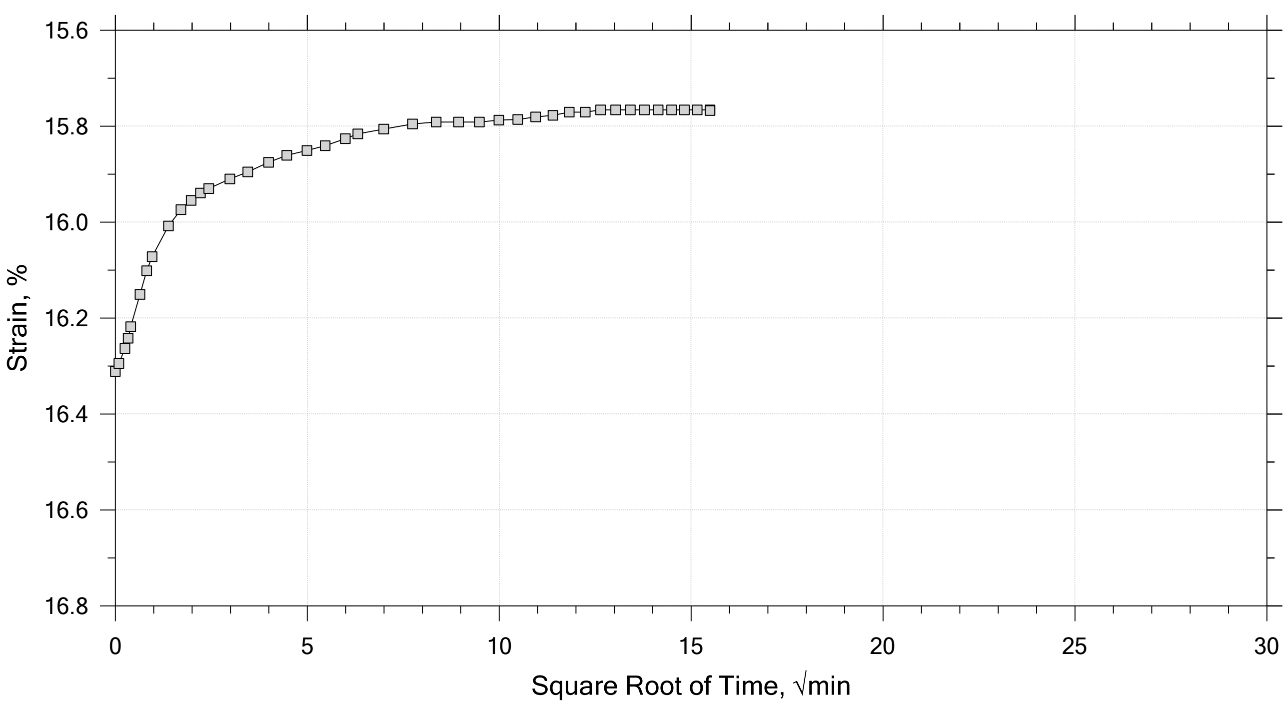
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14

Constant Load Step

Stress: 0.5 tsf





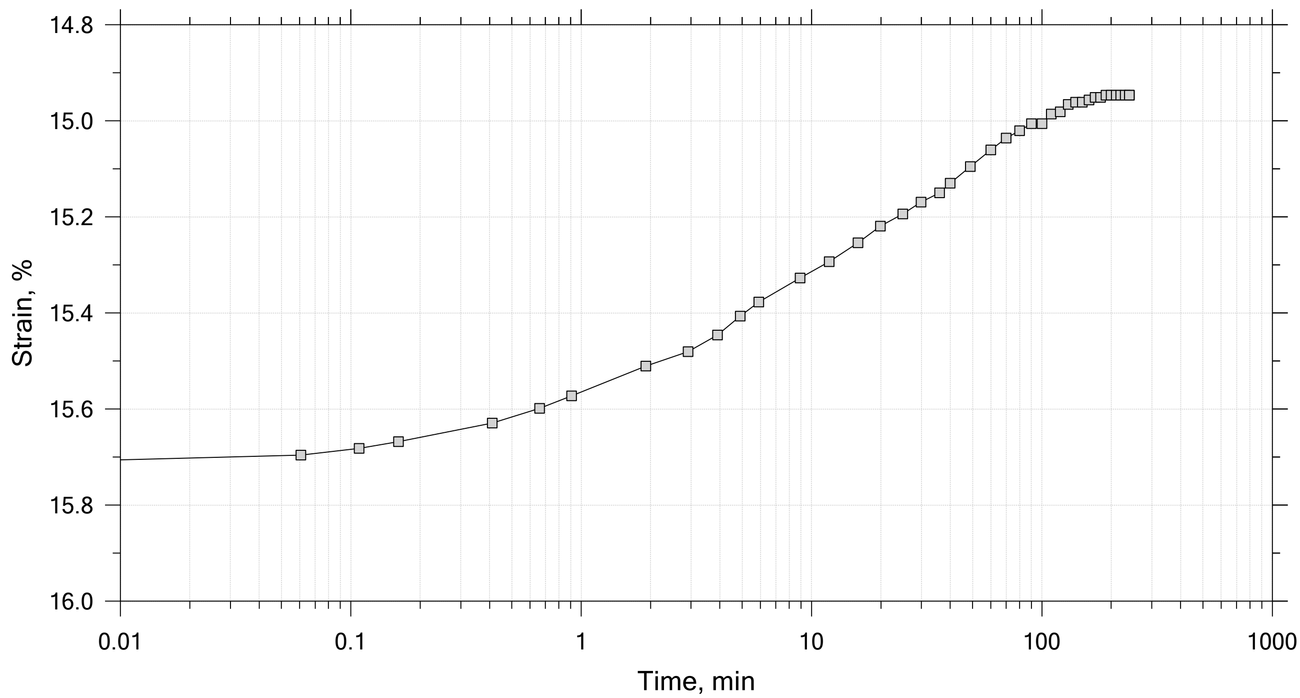
GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

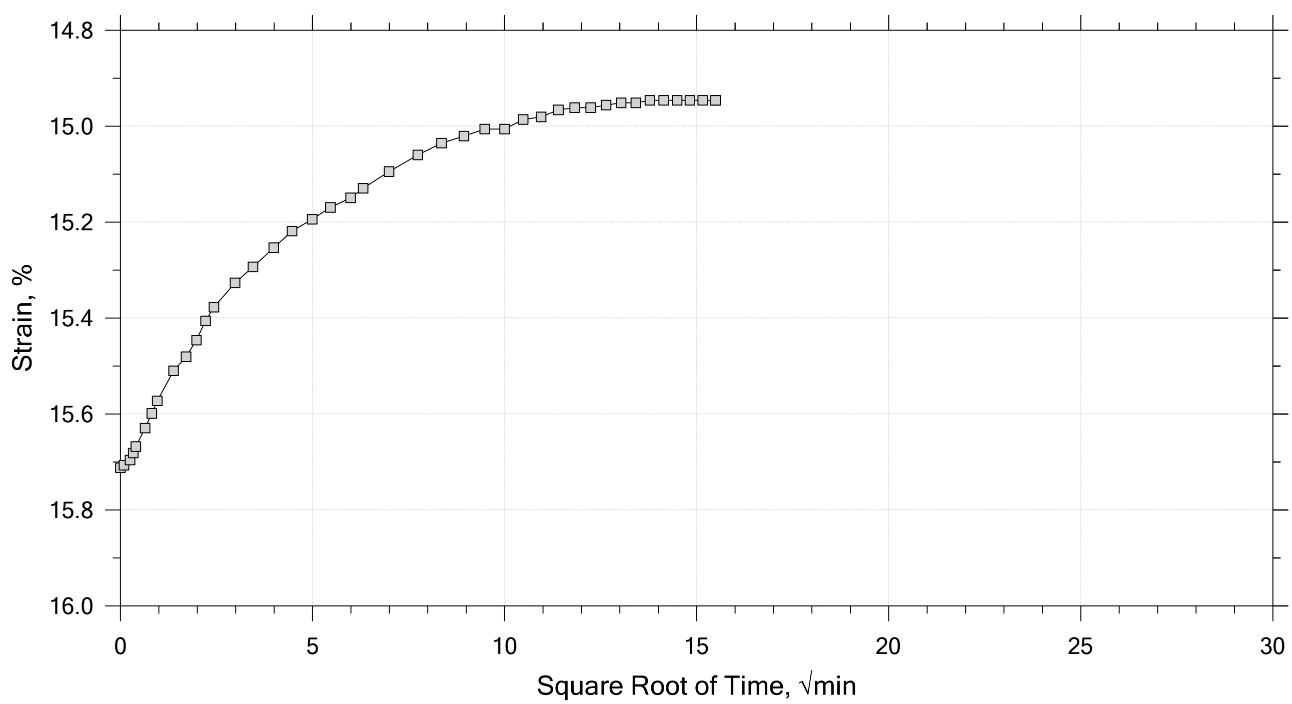
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 14

Constant Load Step

Stress: 0.125 tsf





GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.69	Liquid Limit: ---
Initial Height: 1.00 in	Initial Void Ratio: 0.816	Plastic Limit: ---
Final Height: 0.86 in	Final Void Ratio: 0.562	Plasticity Index: ---

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E14698	RING		E11203
Mass Container, gm	8.28	111.31	111.31	8.15
Mass Container + Wet Soil, gm	209.12	266.62	255.56	150.97
Mass Container + Dry Soil, gm	160.94	230.68	230.68	126.34
Mass Dry Soil, gm	152.66	119.37	119.37	118.19
Water Content, %	31.56	30.10	20.84	20.84
Void Ratio	---	0.82	0.56	---
Degree of Saturation, %	---	99.44	100.00	---
Dry Unit Weight, pcf	---	92.644	107.73	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test.
Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS A Sercel Business	Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
	Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: TX-012, Swell Pressure = 0.0629 tsf		

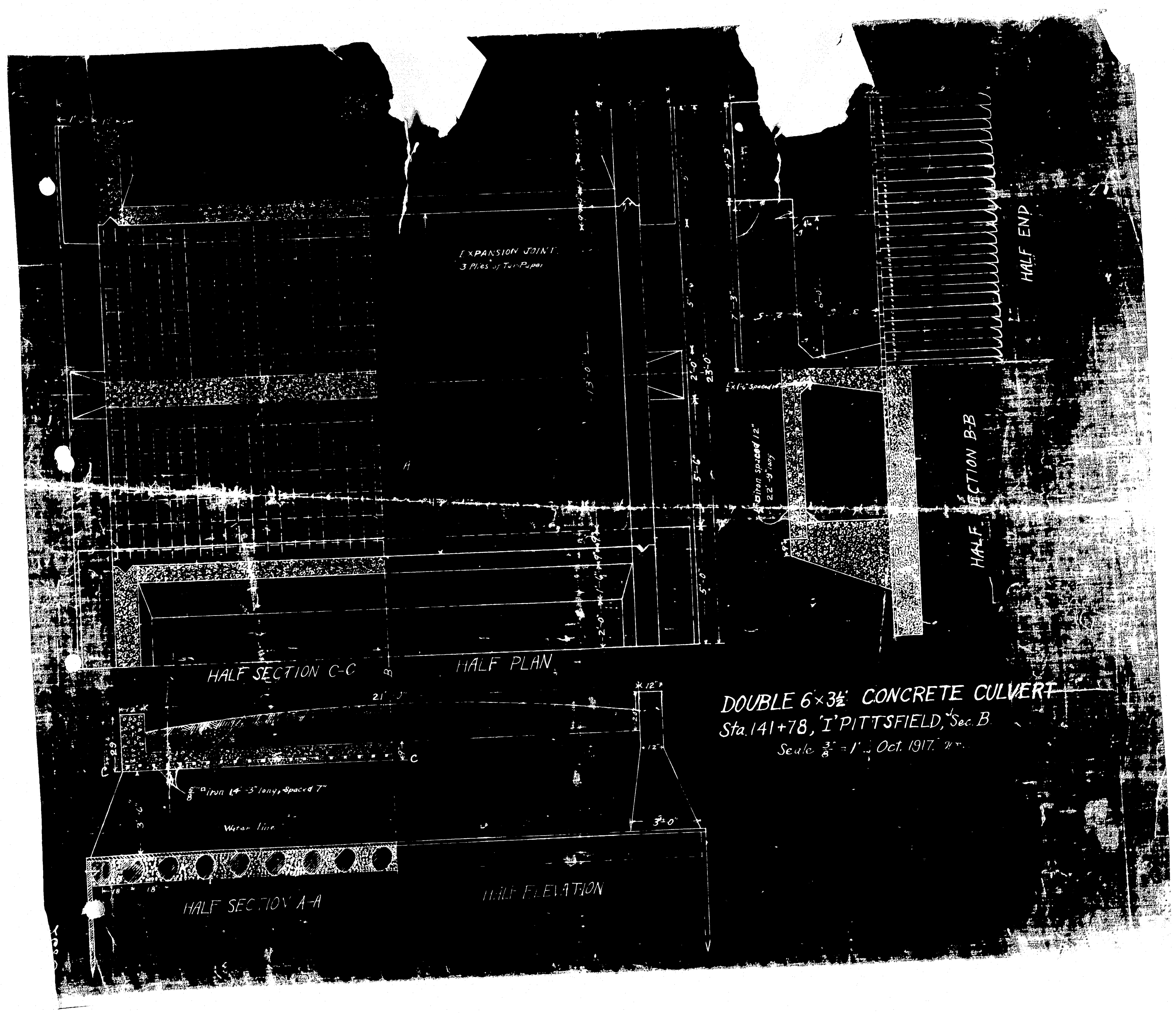
One-Dimensional Consolidation by ASTM D2435 - Method B

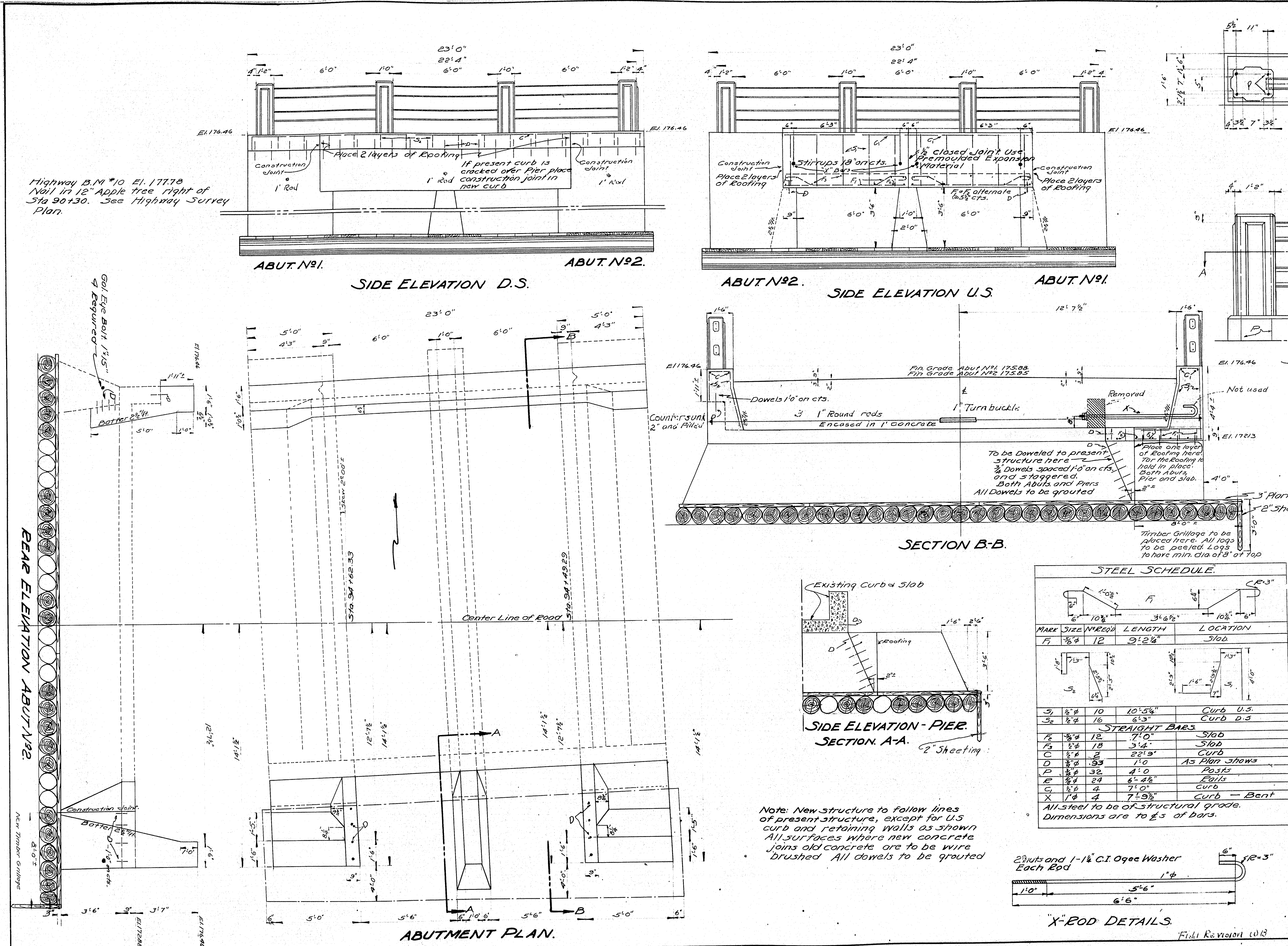
Square Root of Time Coefficients

Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv in²/s	Mv 1/tsf	k ft/day
1	0.0629	0.01226	0.793	1.23	41.612	8.39e-05	1.95e-01	3.06e-04
2	0.125	0.02459	0.771	2.46	19.356	1.76e-04	1.99e-01	6.54e-04
3	0.250	0.03750	0.748	3.75	26.834	1.24e-04	1.03e-01	2.39e-04
4	0.500	0.05287	0.720	5.29	14.414	2.23e-04	6.15e-02	2.57e-04
5	1.00	0.06818	0.692	6.82	12.943	2.41e-04	3.06e-02	1.38e-04
6	2.00	0.08584	0.660	8.58	9.596	3.14e-04	1.77e-02	1.04e-04
7	4.00	0.1057	0.624	10.6	6.035	4.79e-04	9.92e-03	8.90e-05
8	8.00	0.1254	0.588	12.5	5.517	5.01e-04	4.93e-03	4.63e-05
9	16.0	0.1498	0.544	15.0	7.327	3.59e-04	3.05e-03	2.05e-05
10	32.0	0.1761	0.496	17.6	5.124	4.83e-04	1.64e-03	1.48e-05
11	8.00	0.1718	0.504	17.2	0.914	2.64e-03	1.77e-04	8.74e-06
12	2.00	0.1652	0.516	16.5	1.819	1.34e-03	1.10e-03	2.78e-05
13	0.500	0.1577	0.529	15.8	7.046	3.53e-04	5.01e-03	3.31e-05
14	0.125	0.1495	0.544	14.9	32.384	7.82e-05	2.19e-02	3.21e-05

Project: Osborne Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318151
Boring No.: BB-PFB-104	Tested By: jlw	Checked By: trm
Sample No.: U1	Test Date: 7/10/25	Depth: 17-19'
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray silt		
Remarks: TX-012, Swell Pressure = 0.0629 tsf		
Displacement at End of Increment		

APPENDIX C
Historic Bridge Drawings


DOUBLE 6'×3½' CONCRETE CULVERT
Sta. 141+78, "I" PITTSFIELD, Sec. B
Scale ⅜" = 1' — Oct. 1917.
HALF SECTION C-C
HALF PLAN
HALF END
HALF SECTION B-B
HALF SECTION A-A
HALF ELEVATION
EXPANSION JOINT
3 Plies of Tar Paper
Water Line



RAIL DETAILS.

Curb and slab to be cast together. Steel for Posts to be set in curb. Precast rail bars in lengths of 6'5". Place rail bars in position with ends projecting into post forms 2½". Wrap ends 6" with 2 thicknesses of roofing felt. Fold in ends and when post forms are removed cut away all exposed felt. Panels on posts to be ⅞" thick. Chamfer all exposed edges of concrete ½" unless otherwise indicated.

STEEL SCHEDULE.

MARK	SIZE	Nº REQ'D	LENGTH	LOCATION
F₁	⅝"φ	12	9'-2¼"	Slab.
S₁	½"φ	10	10'-5¼"	Curb U.S.
S₂	½"φ	16	6'-3"	Curb D.S.
		STRAIGHT BARS.		
F₂	⅝"φ	12	7'-0"	Slab
F₃	½"φ	18	3'-4"	Slab
C	½"φ	2	22'-9"	Curb
D	¾"φ	93	1'-0	As Plan shows
P	¾"φ	32	4'-0	Posts
R	⅝"φ	24	6'-4½"	Rails
C₁	½"φ	4	7'-0"	Curb
X	1"φ	4	7'-9½"	Curb — Bent

All steel to be of structural grade. Dimensions are to ℄s of bars.

BRIDGE	2634
TOWN	13-35

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
OSBORNE BRIDGE OVER OSBORNE BROOK IN THE TOWN OF PITTSFIELD. SOMERSET COUNTY.
BRIDGE DETAILS.
SHEET 1 of 1 AUGUSTA ME FEB 17, 1932.

18-63

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 8-21-2023 to 8-24-2023 by New England Boring Contractors (monitored by Haley & Aldrich, Inc.).
2. PIC plan set dated March 2025.

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 11.5 ft below ground surface based on scaling from the PIC plans.
4. Footing size considered: 13 ft width x 71 ft long.
5. Footing will bear on granular borrow. Native soil beneath the granular borrow will be cohesive Marine Deposits.
6. Soil properties for the marine deposits will be 115 pcf (unit weight) and 575 (undrained strength).

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The factored bearing resistance for the Strength Limit State is 1.7 ksf

Service Limit State

The factored bearing resistance for the Service Limit State is 1.5 ksf for 1 in. settlement.

Modulus of Subgrade Reaction

The recommended modulus of subgrade reaction is 80 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.

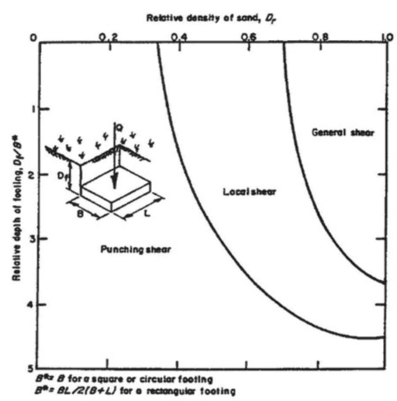


Figure C10.6.3.1.2b-1—Modes of Bearing Capacity Failure for Footings in Sand

INPUT PARAMETERS FOR BEARING CAPACITY

ϕ (deg.) =	0
γ_q (pcf) =	125
γ_f (pcf) =	115
c (psf) =	575
D_f (ft) =	11.50
D_w (ft) =	10
B (ft) =	13.0
e_B (ft) =	0.00
L (ft) =	71.0
e_L (ft) =	0.0
RF or 1/FS =	0.45
Depth Corr., (Y/N)?	N

CALCULATIONS FOR BEARING CAPACITY

B_{eff} = B' (ft) =	13.0
L_{eff} = L' (ft) =	71.0
$N_f = f(\phi)$ =	1.0
$N_c = f_1(\phi)$ =	5.1
$N_q = f_2(\phi)$ =	1.0
$Ng = f_3(\phi)$ =	0.0
s_c =	1.04
s_q =	1.00
s_g =	1.00
d_q =	1.00
C_{wq} =	0.50
C_{wg} =	0.50
N_{cm} =	5.33
N_{qm} =	1.00
N_{gm} =	0.00
q_n or q_{ult} (psf) =	3,782
q_n or q_{ult} (ksf) =	3.8
RF×q_n or q_{ult}/FS (ksf) =	1.7

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.

HALEY ALDRICH	CALCULATIONS	File No.	0208477-000
		Sheet	4 of 6
Client	Fuss & O'Neill, Inc.	Date	5-Feb-2025
Project	Osborne Culvert Replacement	Computed by	EMH
Subject	Bearing Resistance And Subgrade Modulus Calculation	Checked by	JAD/TPJ

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	Bearing Resistance (ksf) 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: 1.5 ksf

HALEY ALDRICH	CALCULATIONS	File No.:	0208477-000
		Sheet:	5 of 6
Client:	Fuss & O'Neill, Inc.	Date:	5-Feb-2025
Project:	Osborne Culvert Replacement	Computed by:	EMH
Subject:	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	JAD/TPJ

BACKGROUND INFORMATION FROM BOWLES 1977

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

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

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

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

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

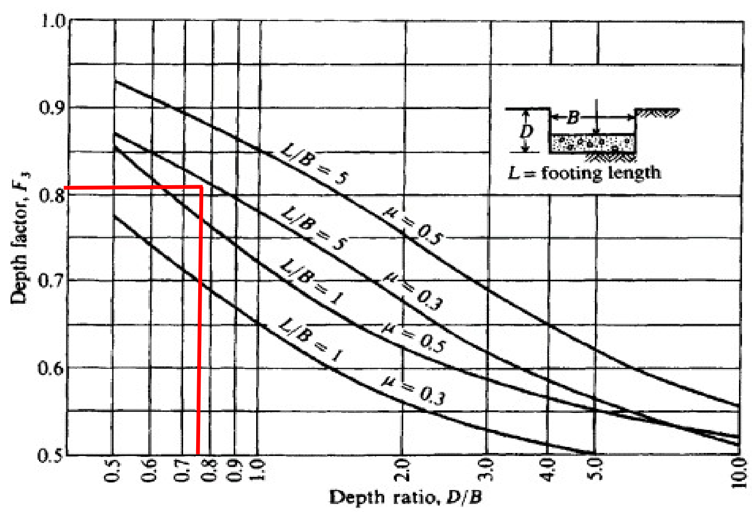
TABLE 2-7
Values or value ranges for Poisson's ratio μ

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 =	0.87
L/B =	5.5
v =	0.4

HALEY ALDRICH	CALCULATIONS	File No.:	0208477-000
		Sheet:	6 of 6
Client:	Fuss & O'Neill, Inc.	Date:	5-Feb-2025
Project:	Osborne Culvert Replacement	Computed by:	EMH
Subject:	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	JAD/TPJ

INPUT PARAMETERS FOR SUBGRADE MODULUS

Width of Footing, B (ft) =	13	
Length of Footing, L (ft) =	71	
Depth of Footing, D (ft) =	11.3	
Elastic Modulus, E_s (MPa) =	10	from Bowles Table 2-8
Elastic Modulus, E_s (ksf) =	209	
Poisson's Ratio, v_S =	0.4	from Bowles Table 2-7

CALCULATIONS FOR SUBGRADE MODULUS

E's (ft^2/k) =	0.004022025	from Bowles Equation 2-65
Thickness of Stratum, H (ft) =	14	approx. thickness bottom of culvert to bottom of soft clay
N = H/B =	1.1	For corner of footing
M = L/B =	5.5	For corner of footing
I_1 =	0.125	from Bowles Equation 5-16a
I_2=	0.125	from Bowles Equation 5-16b
Steinbrenner Influence Factor, I_s =	0.17	from Bowles Equation 5-16c
D/B =	0.87	
Depth Influence Factor, I_F =	0.81	from Bowles Table 5-7

Modulus of Subgrade Reaction, k_s (pci) =	82	from Bowles Equation 9-7

Recommend k_s = 80 pci

SETTLEMENT

PROBLEM STATEMENT & OBJECTIVE

Determine the magnitude of primary settlement and secondary compression due to the shift of the proposed culvert location.

EXECUTIVE SUMMARY

Approximately 1.75 in. of primary consolidation settlement and 1.5 in. secondary compression is predicted to occur adjacent to the proposed culvert. Settlement is not anticipated directly beneath the proposed culvert.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.

AVAILABLE INFORMATION

1. Boring logs dated 8-21-2023 to 8-24-2023 by New England Boring Contractors (monitored by Haley & Aldrich, Inc.).
2. PIC plan set dated March 2025.
3. Existing culvert loading information provided by Fuss & O'Neill on November 17, 2023. Proposed culvert loading information povided by Fuss & O'Neill on September 17, 2025.

ASSUMPTIONS

1. Consolidation parameters of the compressible soil were assumed based on data from the Brewer-Eddington project:

Consolidation Parameter	Assumed value	
Overconsolidation ratio, OCR	1	
Compression Ratio, CR	0.19	
Recompression Ratio, RR	0.02	
Coefficient of consolidation, c_v	2.0E-06	ft^2/sec
Coeff. of secondary compression, $C_{\varepsilon\alpha}$	0.004	
Culvert Design Life	75	years

2. The total thickness of the compressible layer beneath the culvert is 14 ft. The soil is double-drained.
3. Bottom of culvert is approx. 10 ft below finished grade. Total soil stress at bottom of culvert = 1,250 psf (10 ft*125 pcf).
4. Culvert loads provided by Fuss & O'Neill are shown below. Settlement analysis considered SER I dead load pressures.

<u>Proposed Culvert Bearing Pressures:</u>

Bearing Pressure:

Uniform Bearing Pressure: (Service I) $$q_{ser.I} := \frac{(DC_{ser.I} + DW_{ser.I} + EV_{ser.I} + LL_{ser.I})}{W_{culv} \cdot L_{bot}} = 0.81 \text{ ksf}$$

Uniform Bearing Pressure: (Strength I) $$q_{str.I} := \frac{(DC_{str.I} + DW_{str.I} + EV_{str.I} + LL_{str.I})}{W_{culv} \cdot L_{bot}} = 1.117 \text{ ksf}$$

Uniform Bearing Pressure from Dead Load: (Service I) $$q_{ser.I\_DL} := \frac{(DC_{ser.I} + DW_{ser.I} + EV_{ser.I})}{W_{culv} \cdot L_{bot}} = 0.640 \text{ ksf}$$

Uniform Bearing Pressure from Dead Load: (Strength I) $$q_{st.I\_DL} := \frac{(DC_{str.I} + DW_{str.I} + EV_{str.I})}{W_{culv} \cdot L_{bot}} = 0.82 \text{ ksf}$$

<u>Existing Culvert Bearing Pressures:</u>

Bearing Pressure:

Uniform Bearing Pressure: (Service I) $$q_{ser} := \frac{(DC_{ser} + DW_{ser} + LL_{ser} + EV_{ser})}{W_{bridge} \cdot L_{cribbing}} = 1.104 \text{ ksf}$$

Uniform Bearing Pressure: (Strength I) $$q_{I} := \frac{(DC_{I} + DW_{I} + LL_{I} + EV_{I})}{W_{bridge} \cdot L_{cribbing}} = 1.533 \text{ ksf}$$

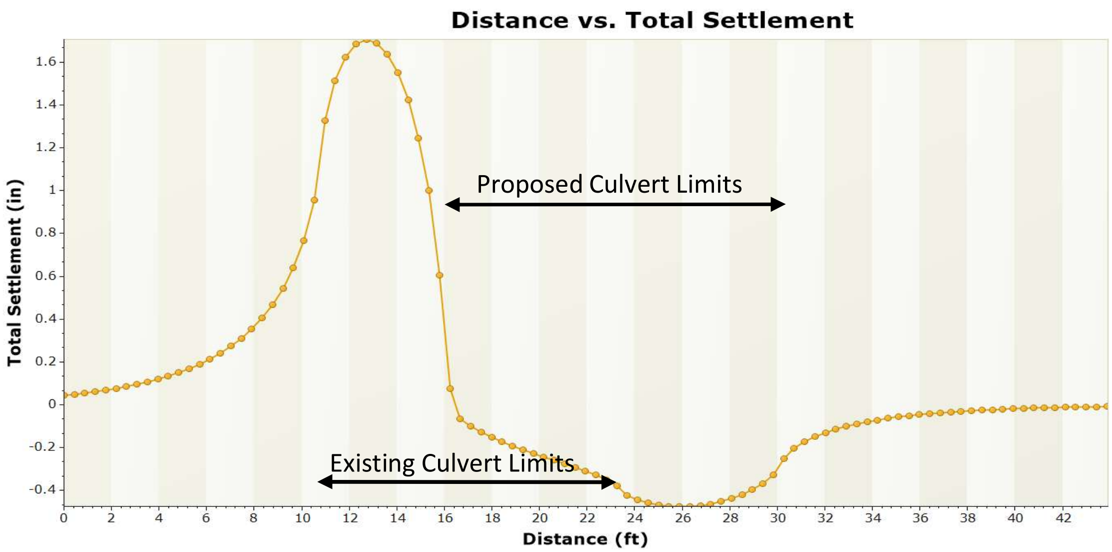
Uniform Bearing Pressure from Dead Load: (Service I) $$q_{ser\_DL} := \frac{(DC_{ser} + DW_{ser} + EV_{ser})}{W_{bridge} \cdot L_{cribbing}} = 0.856 \text{ ksf}$$

Uniform Bearing Pressure from Dead Load: (Strength I) $$q_{I\_DL} := \frac{(DC_{I} + DW_{I} + EV_{I})}{W_{bridge} \cdot L_{cribbing}} = 1.099 \text{ ksf}$$

	CALCULATIONS	File No.:	0208477-000
		Sheet:	2 of 2
Client:	Fuss & O'Neill, Inc.	Date:	12-Jan-2026
Project:	Osborne Culvert Replacement	Computed by:	EMH
Subject:	Settlement Evaluation	Checked by:	MMB

CALCULATIONS

From Settle3:



Maximum Primary Settlement	1.7	in

Note: Refer to Settle3 output for primary settlement calculation. The maximum predicted settlement is located immediately adjacent to the proposed culvert. Settlement directly beneath the culvert is negligible.

Time Rate of Consolidation Calculation

Coefficient of consolidation, c_v	2.00E-06	ft^2/sec
	63.07	ft^2/year
Compressible Layer Thickness, H	14	ft
Max. Drainage Path Length, Hdr	7	ft
Time Factor @ 90% Consol., T_{90}	0.848	
Time to 90% Consolidation, t_{90}	0.66	years

Secondary Compression Calculation

Coeff. of secondary compression, $C_{\varepsilon\alpha}$	0.004	
Culvert Design Life	75	years
Secondary Compression	0.12	ft
	1.38	in

Settle3 Analysis Information

2026-0112-HAI-Osborne Culvert Settlement.d6

Project Settings

Document Name	2026-0112-HAI-Osborne Culvert Settlement.d6.s3z
Author	EMH
Company	Haley & Aldrich, Inc.
Date Created	11/15/2023, 8:31:11 AM
Last saved with Settle3 version	5.018
Comments	
Stage 1 is current conditions. Stage 2 is after the new culvert and fill is placed.	
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches

Advanced Settings

Start of secondary consolidation (% of primary)	95
Min. stress for secondary consolidation (% of initial)	1
Reset time when load changes for secondary consolidation	No
Minimum settlement ratio for subgrade modulus	0.9
Use average poisson's ratio to calculate layered stresses	
Update Cv in each time step (improves consolidation accuracy)	
Ignore negative effective stresses in settlement calculations	
Add field points to load edges	

Soil Profile

Layer Option	Horizontal Soil Layers
Vertical Axis	Elevation
Ground Elevation (ft)	0

Stage Settings

Stage #	Name
1	Stage 1
2	Stage 2

Results (relative to Stage: Stage 1)

Time taken to compute: 0.878779 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	0	0
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	0
Effective Stress XX [ksf]	0	0
Effective Stress YY [ksf]	0	0
Total Stress ZZ [ksf]	0	0
Total Stress XX [ksf]	0	0
Total Stress YY [ksf]	0	0
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0
Pore Water Pressure [ksf]	0	0
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0	0
Over-consolidation Ratio	0	0
Void Ratio	0	0
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0

Stage: Stage 2

Data Type	Minimum	Maximum
Total Settlement [in]	-0.477237	1.70438
Total Consolidation Settlement [in]	-0.477237	1.70438
Virgin Consolidation Settlement [in]	0	1.70438
Recompression Consolidation Settlement [in]	-0.477237	0
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	-0.646811	0.394126
Loading Stress XX [ksf]	-0.606186	0.422847
Loading Stress YY [ksf]	-0.388322	0.17673
Effective Stress ZZ [ksf]	-0.646811	0.394126
Effective Stress XX [ksf]	-0.606186	0.422847
Effective Stress YY [ksf]	-0.388322	0.17673
Total Stress ZZ [ksf]	-0.646811	0.394126
Total Stress XX [ksf]	-0.606186	0.422847
Total Stress YY [ksf]	-0.388322	0.17673
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.454735	0.221396
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.454735	0.221396
Total Strain	-0.00831137	0.0315097
Pore Water Pressure [ksf]	0	0
Degree of Consolidation [%]	-1.42109e-14	1.42109e-14
Pre-consolidation Stress [ksf]	0	0.394013
Over-consolidation Ratio	0	1.60357
Void Ratio	-0.0661704	0.0174539
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	-6.38171e-05	0.00219094

Loads

1. Polygonal Load: "Polygonal Load 8"

Label	Polygonal Load 8
Load Type	Flexible
Area of Load	3970.67 ft2
Load	1.25 ksf
Elevation	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
168.217	24.003
34.468	24.003
34.468	-5.542
169.507	-5.542

2. Polygonal Load: "Polygonal Load 9"

Label	Polygonal Load 9
Load Type	Flexible
Area of Load	4491.99 ft2
Load	1.25 ksf
Elevation	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
182.288	-5.542
333.584	-5.542
333.584	24.003
180.802	24.003

3. Polygonal Load: "Polygonal Load 10"

Label	Polygonal Load 10
Load Type	Flexible
Area of Load	422.231 ft2
Load	0.64 ksf
Elevation	0 ft
Installation Stage	Stage 2

Coordinates

X [ft]	Y [ft]
174.041	24.003
174.041	-5.542
188.18	-5.54161
188.484	24.003

4. Polygonal Load: "Polygonal Load 2"

Label	Polygonal Load 2
Load Type	Flexible
Area of Load	374.719 ft2
Load	0.856 ksf

Advanced Staging

Stage	Load Factor	Bottom Elevation [ft]
Stage 1	1	0
Stage 2	0	0

Coordinates

X [ft]	Y [ft]
169.507	-5.542
182.288	-5.542
180.802	24.003
168.217	24.003

5. Polygonal Load: "Polygonal Load 11"

Label	Polygonal Load 11
Load Type	Flexible
Area of Load	200.526 ft2
Load	-1.25 ksf
Elevation	0 ft
Installation Stage	Stage 2

Coordinates

X [ft]	Y [ft]
188.484	24.003
180.802	24.003
182.288	-5.542
188.18	-5.54161

6. Polygonal Load: "Polygonal Load 1"

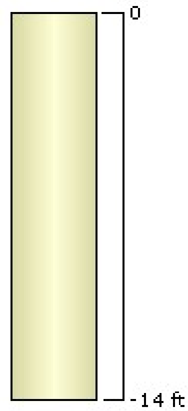
Label	Polygonal Load 1
Load Type	Flexible
Area of Load	153.014 ft2
Load	1.25 ksf
Elevation	0 ft
Installation Stage	Stage 2

Coordinates

X [ft]	Y [ft]
169.507	-5.542
174.041	-5.542
174.041	24.003
168.217	24.003

Soil Layers

Layer #	Type	Thickness [ft]	Elevation [ft]
1	Soft Marine Deposit	14	0



Soil Properties

Property	Soft Marine Deposit
Color	
Unit Weight [kips/ft3]	0.115
Saturated Unit Weight [kips/ft3]	0.115
K0	1
Primary Consolidation	Enabled
Material Type	Non-Linear
Cce	0.19*
Cre	0.02
e0	1.1
OCR	1
Undrained Su A [kips/ft2]	0
Undrained Su S	0.2
Undrained Su m	0.8
Piezo Line ID	1
* Base value only. Refer to Stage Factor section.	

Stage Factors, Soft Marine Deposit

Primary Consolidation - Non-Linear		
	Cce	
Stage	**Factor**	**Value**
Stage 1	1	0.19
Stage 2	1	0.19

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft3

Piezometric Line Entities

ID	Elevation (ft)
1	0 ft

Query

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Query Point 1	171.451, 9.2305	Auto: 31

Query Lines

Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	Query Line 1	158.137, 9.2305	202, 9.2305	100	Auto: 31

Field Point Grid

Number of points 989
Expansion Factor 2

Grid Coordinates

X [ft]	Y [ft]
409.975	121.861
409.975	-100.383
-41.923	-100.383
-41.923	121.861

LATERAL EARTH PRESSURES

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.:	0208477-000
		Sheet:	2 of 2
Client:	Fuss & O'Neill, Inc.	Date:	4/21/2025
Project:	Osborne Culvert Replacement	Computed by:	EMH
Subject:	Lateral Earth Pressure Coefficients	Checked by:	JAD

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	