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GEOTECHNICAL DESIGN REPORT ON
FARNHAM BRIDGE OVER FARNHAM BROOK, BRIDGE NO. 2274
MAINEDOT WIN 26109.00
PITTSFIELD, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

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

File No. 0208476-000
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Attention: Shannon Beaumont, P.E.
Senior Project Manager

Subject: Geotechnical Design Report
Farnham Bridge over Farnham Brook, Bridge No. 2274
MaineDOT WIN 26109.00
Pittsfield, Maine

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit herewith our report entitled, "Geotechnical Design Report, Farnham Bridge over Farnham Brook, Bridge No. 2274, MaineDOT WIN 26109.00, 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. 188+25 (south) to Sta. 193+00 (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 0.1 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. 192 (south) to El. 177 (north). The bottom of Farnham Brook, within the limits of State Route 11/100, is approximately El. 167.5.

Based on our review of the historical plans for the subject project dated May 1932, it is our understanding that the existing bridge consists of a single-span, cast-in-place concrete culvert. The invert of the existing culvert is approximately El. 167.5 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 75.63-ft long x 13-ft span x 6-ft rise precast concrete box culvert without special fill along the culvert bottom. The culvert invert elevation will be at El. 167.6. The bottom of the culvert structure will bear at approximately El. 166.6.
- 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 downstream end of the culvert will be beveled. It is our understanding that the upstream end of the culvert will include wingwalls that are 19.17-ft long (west) and 7.67-ft long (east).
- The proposed culvert will be skewed from the existing culvert location. The western inlet will be approximately 10 ft south of the existing culvert, and the eastern inlet will remain at approximately the same location as existing.
- 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 18 August and 25 August 2023. Two borings, designated BB-PFB-101 and BB-PFB-102, were drilled at/near the existing bridge.

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 39 to 44 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 through the existing fill and at 5-ft intervals thereafter 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 boring BB-PFB-102 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 4.7 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. 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	12 to 15	Medium dense to very dense SAND or GRAVEL with various amounts of silt. Probable cobbles and boulders encountered. *(encountered in all borings)*
Marine Deposit	>27	Soft to hard SILT or Silty CLAY with various amounts of sand; medium dense to dense fine to coarse SAND with various amounts of silt and gravel. *(encountered in all borings)*

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 11 to 15 ft BGS (El. 167.0 to 168.5).
- water levels measured in each boring during or after drilling was approximately 5.6 to 11ft BGS (El. 171.0 to 173.9).
- Surveyed stream water level El. 168.5, 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	3	AASHTO Classification: A-1-a, A-1-b USCS Classification: GW-GM, SM
		Marine Deposit	2	AASHTO Classification: A-2-4 USCS Classification: SM
Atterberg Limits	ASTM D4318	Marine Deposit	3	Liquid Limit: 31<LL<40 Plastic Limit: 21<PL<29 Plasticity Index: 10<PI<13 Water Content: 25%<WC<31%
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; 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.

SEISMIC DESIGN CONSIDERATIONS

In accordance with AASHTO LRFD Article 3.10.1 and 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 for the culvert. The culvert wingwalls, however, were designed considering seismic effects.

Due to the nature and thickness of the overburden soils encountered in the borings, we recommend the site be considered "Site Class D." The peak seismic ground acceleration coefficient modified by F_{pga}, A_s, was determined to be equal to 0.117 g based on the geographic site location and the recommended "Site Class D" designation using the United States Geological Survey (USGS) seismic design web service, which is based on a seismic event having a 7 percent probability of exceedance in 75 years (approximate 1,000 year return period).

CULVERT DESIGN RECOMMENDATIONS

Anticipated Subgrade Conditions and Bedding Detail

Based on the proposed bottom of culvert elevation in the draft PIC plans (El. 166.6), the thickness of "typical" bedding details (i.e., 1 ft; see below), and the subsurface conditions encountered in the borings, medium stiff to 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 Culvert Bedding Stone. 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 Culvert Bedding Stone. 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 2.1 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 130 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 western inlet of the proposed culvert will be approximately 10 ft south of the existing culvert, and the eastern inlet will remain at approximately the same location as existing. The vertical alignment of State Route 11/100 will roughly match existing site conditions. This results in a wedge directly north of the proposed culvert (in the footprint of the existing culvert) that will be subject to approximately 14.5 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 to 2
- 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 (14.5 to 16.5 ft high with embankment fill unit weight of 125 pcf); existing culvert constructed (SER I dead load of 1.603 ksf per Fuss & O'Neill).
  - Proposed Culvert Construction: Existing culvert replaced with off-alignment proposed culvert (SER I dead load of 0.742 ksf per Fuss & O'Neill); embankment fill placed over portion of existing culvert footprint (14.5 to 16.5 ft high with embankment fill unit weight of 125 pcf).

Based on the above assumptions, we estimate that less than ½ in. of primary consolidation settlement will occur post-construction. We anticipate that most (more than 90 percent) of the primary consolidation will occur within the first year following construction. Note that this settlement will occur directly to the north of the proposed culvert over the footprint of the existing culvert. We do not anticipate settlement directly beneath the proposed culvert. 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 estimated total post-construction settlement over the culvert design life is therefore less than 2 in.

Lateral Earth Pressure

Per the 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.

WINGWALL DESIGN RECOMMENDATIONS

Based on discussions and multiple design iterations with Fuss & O'Neill and MaineDOT, we recommend that the culvert wingwalls be constructed as geosynthetic-reinforced Precast Concrete Block Gravity Walls (MaineDOT Item 672.10; e.g., Redi-Rock). Recommendations for external stability of the walls are provided herein.

The walls should be designed by the vendor in accordance with Standard Specification 672 Precast Concrete Block Gravity Walls and AASHTO LRFD.

Mechanically Stabilized Earth Retaining Walls with steel reinforcing strips (MaineDOT Item 677.20) were also considered but not selected due to corrosion considerations.

External Stability

External stability (bearing, sliding, overturning and eccentricity) evaluations were completed for the wingwalls using the computer program MSEW+ (Update 2021.13) developed by ADAMA Engineering, Inc. Internal stability of the walls was not evaluated and will be evaluated by the wall vendor's engineer. Because the internal stability of the walls was not evaluated, the type of reinforcing material (e.g. geogrid, geosynthetic, metal strips, etc.) does not impact the results of our external stability evaluations for the retaining walls.

External stability evaluations were performed considering the following geometry:

- A maximum wall height, H, of 12.2 ft (including embedment).
- The reinforcement length, L, was set equal to the wall height (i.e., L/H = 1.0).
- A minimum wall embedment of 3.25 ft. Note that 1 ft of non-frost susceptible material should be placed beneath the wingwalls.
- Bottom of the wall at El. 164.4
- A 2H:1V backslope that is approximately 5 ft tall.
- A 250 psf live load surcharge was assumed to act over the entire roadway width (beyond the backslope).

External stability evaluations under seismic loading considered a peak seismic ground acceleration coefficient modified by F_{pga}, A_s, was determined to be equal to 0.117 g, corresponding to "Site Class D" as mentioned above.

Bearing Resistance

Bearing resistance calculations were completed in accordance with AASHTO LRFD Sections 11.10.5.4 and 10.6.3.1 for the culvert wingwalls based on the subgrade conditions present at the bottom of the walls as summarized above. Bearing capacity evaluations were performed and accounted for the presence of the relatively thin layer soft marine clay soils directly beneath the walls as well as stiffer marine soils several feet below, in accordance with the methodology outlined in Meyerhoff and Hanna (1978). Recommended bearing resistances for the wingwalls are as follows:

- The wingwalls should be evaluated at the Service Limit State not to exceed a presumptive bearing resistance equal to 3 ksf in accordance with AASHTO LRFD Table C10.6.2.6.1-1.
- Strength Limit State bearing resistance should not exceed 3.4 ksf, which is based on a resistance factor (ϕ_f) equal to 0.65 (AASHTO LRFD Table 11.5.7-1 for Mechanically Stabilized Earth Walls, which are defined by LRFD as soil-retaining systems employing tensile reinforcements in the soil mass). Bearing resistance calculations assumed a reinforcement length, L, equal to the wall height, H (i.e., L/H = 1.0) with the bottom of the wall set at El. 164.4 (i.e., 3.25 ft below the culvert invert elevation).
- Extreme Event Limit State bearing resistance should not exceed 4.7 ksf, which is based on a resistance factor (ϕ_f) equal to 0.9 (AASHTO LRFD Section 11.5.8) and the assumptions listed above.

A minimum Capacity Demand Ratio (CDR) of 1.0 is required for bearing resistance. Bearing resistance CDRs of 1.01 and 1.24 were achieved under static and seismic loading, respectively, for the wall conditions listed above. Please note that the reinforcement length needed to be increased from 0.7H (which is typical for reinforced modular block walls) to 1.0H to achieve acceptable CDR values.

Sliding Resistance

A minimum CDR of 1.0 is required for sliding resistance. A resistance factor of 1.0 for the strength and extreme event limit states was used in the evaluation in accordance with AASHTO LRFD Table 11.5.7-1 and Section 11.5.8. A foundation soil friction resistance of 32 degrees was used to evaluate sliding resistance due to the proposed 1-ft thick granular bedding material that will be placed below the walls. The CDRs for sliding along the reinforced and foundation soils interface were determined to be equal to 1.67 and 1.42 for the static and seismic cases, respectively.

Overturning Resistance and Eccentricity

A minimum CDR of 1.0 is required for overturning resistance. The location of the resultant eccentric load should be within the middle two-thirds (2/3) of the retaining wall base width and length for static conditions per AASHTO LRFD Article 11.6.3.3 and the middle eight-tenths (8/10) of the base width for seismic conditions per AASHTO LRFD Section 11.6.5.1. Based on our analyses, the computed static overturning CDR was 3.0, and the computed seismic overturning CDR was 2.0. The computed static eccentricity ratio (e/L) was determined to be equal to 0.10, and the computed seismic eccentricity ratio was determined to be equal to 0.23.

Global Stability

A series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 9.0 by Rocscience to evaluate the likelihood of global stability failures for the wingwalls. Spencer's method was used for all global stability analyses.

A soil profile model was developed based on the subsurface conditions encountered in the borings completed at the site. Based on the nature and consistency of the collected soil samples, the SPT N-values measured during soil sampling, and vane shear tests performed during drilling, physical and strength parameters were determined. Refer to Appendix A for the soil properties used. Additionally, a 250 psf live load surcharge was assumed to act within the limits of State Route 100.

The factor of safety for pseudo-static seismic load cases was calculated using a horizontal acceleration coefficient, k_h, equal to one half of the acceleration coefficient, A_s. A value of $A_s/2$ (0.059g) was selected in accordance with AASHTO LRFD guidance in Section 11.6.5.2.2 and based on a Seismic Site Class D. The reduction from A_s is due to soil slope flexibility and the fact that the peak ground acceleration during an earthquake only lasts for a very short period of time.

Based on the scope of the field investigation at the site, the scope of the laboratory testing program and the relatively uniform subsurface conditions encountered in the borings, it is our opinion that the subsurface stratigraphy and geotechnical parameters for the soils at the site are well defined. Therefore, the minimum required factor of safety for global stability of earth slopes per AASHTO LRFD and the BDG under static conditions is 1.3. The minimum required factor of safety for embankments subjected to pseudo-static seismic loading is 1.1 based on discussions with MaineDOT.

The minimum calculated static and pseudo-static factors of safety were determined to be 1.6 and 1.4, respectively, and are acceptable for both the static and pseudo-static load cases.

FROST PROTECTION

The minimum depth of embedment/cover for foundation or other below-grade structures needed for frost protection was evaluated in accordance with the requirements of BDG Section 5.2.1 based on a design freezing index equal to 1,750 freezing degree days and the subgrade material type (granular), and the subgrade material water content data presented above. Based on discussions with MaineDOT, culverts with flowing water do not need additional protection for frost. We recommend that the wingwalls bear a minimum of 4.25 ft below the lowest adjacent ground surface exposed to freezing. Riprap should not be considered as contributing to the overall thickness of soils required for frost protection, unless the voids between stones have been sufficiently filled with finer-grained material (e.g., void-filled riprap). In accordance with LRFD Section 11.10.2.2, as an alternative to locating the wall base below the depth of frost penetration, the soil within the depth of frost penetration below the wall can be removed and replaced with nonfrost-susceptible clean granular soil (e.g., underdrain backfill material).

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 and wingwalls. Based on existing ground surface elevations, the proposed culvert invert and bottom of wingwall elevations, and the anticipated subgrade conditions, excavation depths ranging between approximately 15 and 19 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 (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 44 ft BGS and 39 ft BGS is borings BB-PFB-101 and BB-PFB-102, 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. 168.5) and the proposed subgrade level (El. 163.4 to El. 165.6), we anticipate that excavations will typically extend up to approximately 3 to 5 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 medium stiff to 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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References

Griffin, John R., Bedrock Geology of the Pittsfield Quadrangle, Maine, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File No. 71-3.

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.

Meyerhof, G. G., and Hanna, A. M., "Ultimate bearing capacity of foundations on layered soils under included load." Canadian Geotechnical Journal, November 1978.

TABLES

TABLE I
Subsurface Exploration Location Data
Farnham Bridge No. 2274
MaineDOT WIN 26109.00
Pittsfield, Maine

Haley & Aldrich, Inc. File No.: 0208476-000

Test Boring No.[1]	Ground Surface Elevation (ft)[3]	Station[4]	Offset Distance (ft) & Direction[5]
BB-PFB-101	182.0	191+34	6 LT
BB-PFB-102	179.5	191+80	9 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	5/30/2025
Checked By:	CFE	6/9/2025
Reviewed By:	EAF	6/10/2025

TABLE II
Subsurface Exploration Subsurface Data
Farnham Bridge No. 2274
MaineDOT WIN 26109.00
Pittsfield, Maine

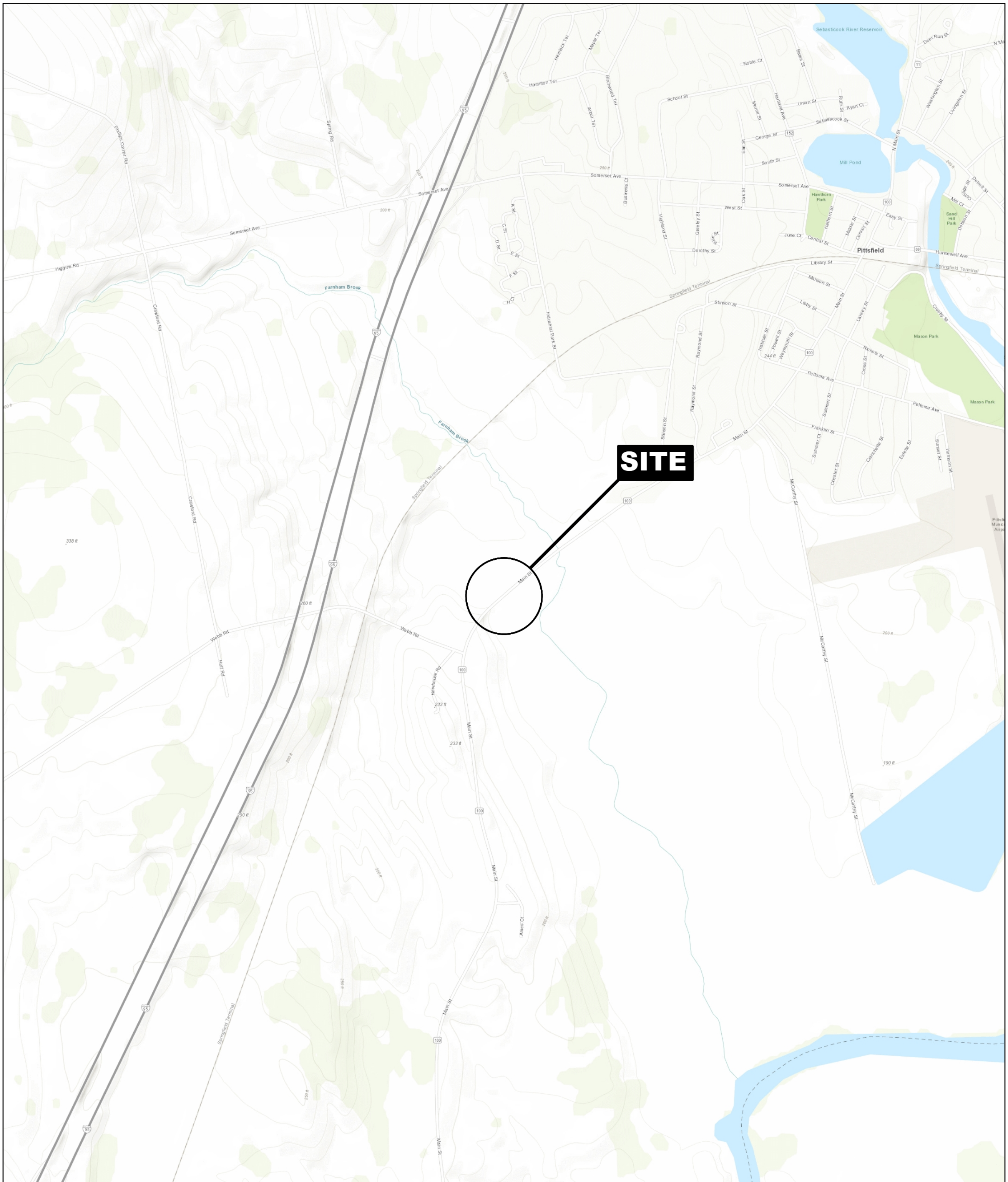
Haley & Aldrich, Inc. File No.: 0208476-000

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]			Marine Deposits[4,5]				
		Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)		
BB-PFB-101	182.0	0.0	182.0	15.0	15.0	167.0	> 29.0	44.0	138.0
BB-PFB-102	179.5	0.0	179.5	11.7	11.7	167.8	> 27.3	39.0	140.5

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 Bituminous concrete was encountered in the upper 2.1 ft of test boring BB-PFB-101 as well as the upper 1.6 ft of test boring BB-PFB-102
4 A 0.1 ft layer of organic silt was encountered in BB-PFP-102 from 11.7 to 11.8 ft BGS.
5 "> 29.0" indicates total thickness of stratum is greater than value shown.

	Individual	Date
Prepared By:	EMH	5/30/2025
Checked By:	CFE	6/9/2025
Reviewed By:	EAF	6/10/2025

FIGURES



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

MAP SOURCE: USGS

FARNHAM BRIDGE OVER FARNHAM BROOK, BRIDGE NO. 2274
MAINEDOT WIN 26109.00
PITTSFIELD, MAINE

PROJECT LOCUS

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

FIGURE 1

0208476_000_LOCUS HALEYALDRICH\EHUNSTEIN

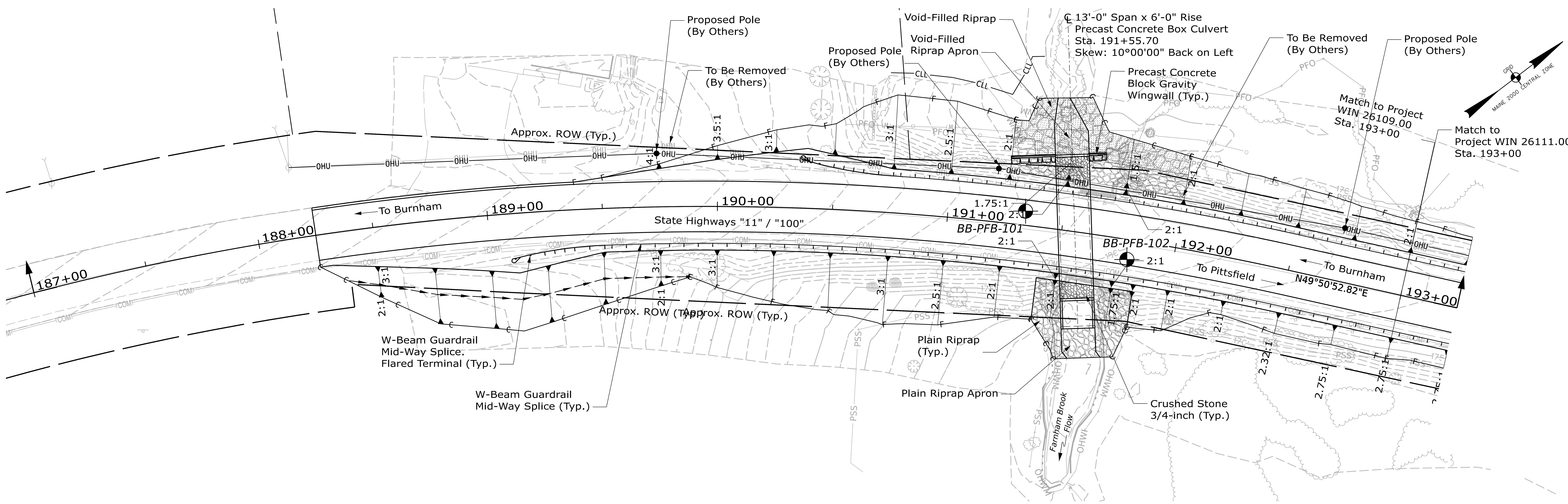
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2610900

WIN 26109.00

BRIDGE PLANS

BRIDGE NO. 2274



PLAN

25 0 25 50
Scale of Feet

LEGEND:

BB-PFB-101 — 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 (see Figure 3).

Key

BB = Bridge Boring

NOTES:

1. Existing ground surface elevations and stationing were taken from the 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. Refer to Apendix A and the boring log sheets for test boring logs and bedrock core photographs and Appendix B for laboratory test results.
4. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

HALEY ALDRICH

	BY	DATE
PROJ. MANAGER	B. NICHOLS	
DESIGN-DETAILED	E. HUNSTEIN	K. POST 1/26
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

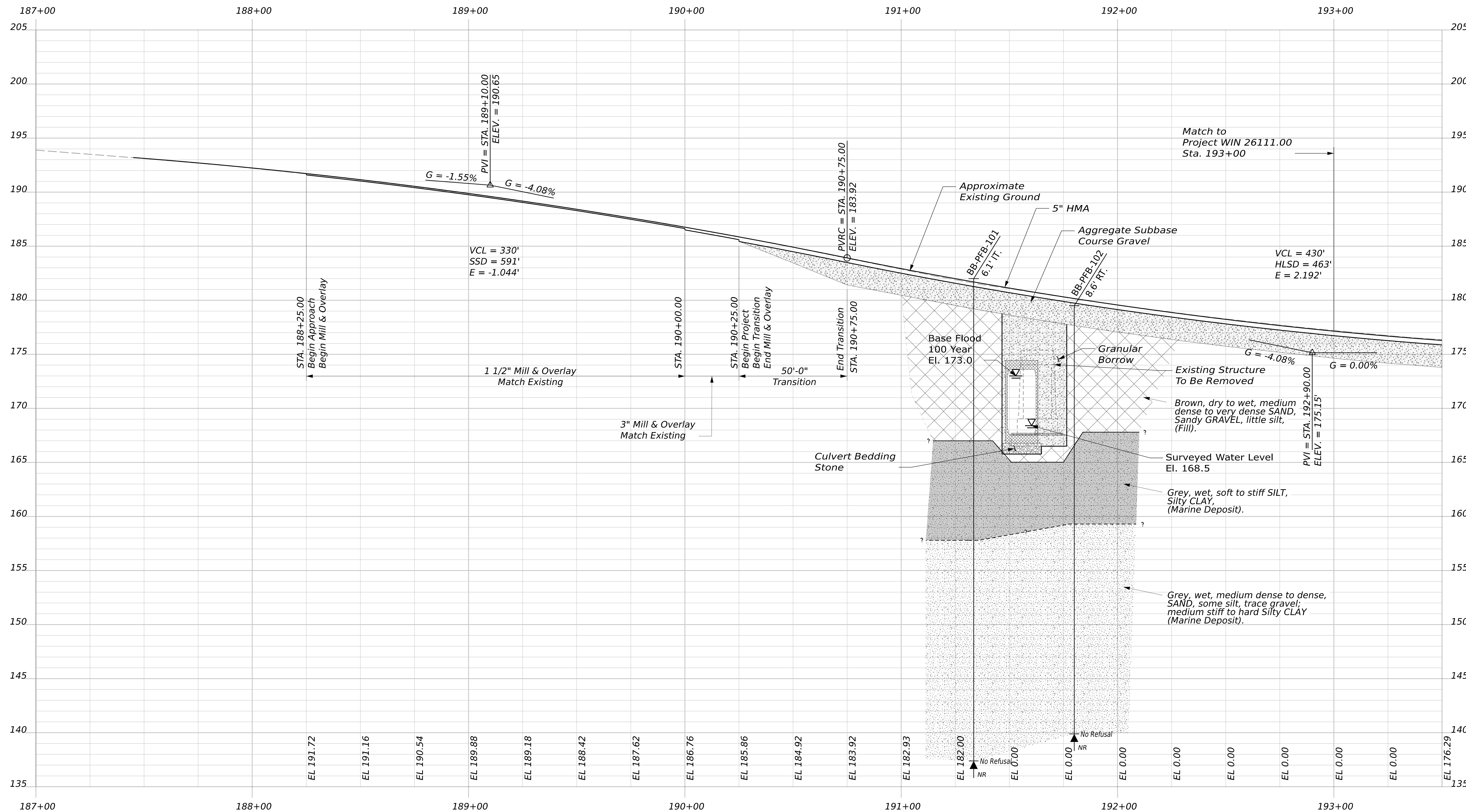
FARNHAM BRIDGE
FARNHAM BROOK

PITTSFIELD SOMERSET COUNTY

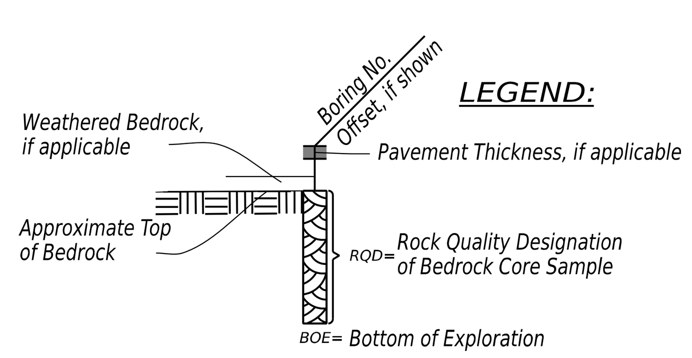
EXPLORATION LOCATION PLAN

SHEET NUMBER
2
OF

Username: kpostolowski Date: 2/4/2026



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.

PROFILE

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

HALEY ALDRICH

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
0 2610900
WIN 26109.00
BRIDGE PLANS
BRIDGE NO. 2274

	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

FARNHAM BRIDGE
FARNHAM BROOK
PITTSFIELD SOMERSET COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF

Username: kpostolowski Date: 2/4/2026

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Farnham Bridge No. 2274	**Boring No.:**	BB-PFB-101		
Location:	Pittsfield, Maine	**WIN:**	26109.00		
Driller:	New England Boring Contractors	**Elevation (ft.)**	182.0	**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:	8-24-2023/8-25-2023	**Drilling Method:**	HW to 43.5 ft	**Core Barrel:**	--
Boring Location:	191+34, 6 ft LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	11 ft (Approx.)
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							1			Asphalt and Concrete 0 to 2.1 ft. Note: No samples taken 0 to 5.0 ft.	
							1				
							3	179.9		2.1 — Note: Drill action indicates probable cobbles and boulders from 2.1 to 4.2 ft .	
							33				
5	1D	24/16	5.0 - 7.0	16/23/16/15	39	49	47			Brown, dry, dense, fine to coarse SAND, some gravel, little silt, well-graded, (Fill).	G#744574 A-1-b, SM
							81				
	2D	24/2	7.0 - 9.0	25/31/28/27	59	75	93			Similar to 1D above, except wet, very dense, possibly pushed a cobble, (Fill).	
							85				
	3D	24/1	9.0 - 11.0	7/14/12/6	26	33	89			Broken rock fragments and pieces, possibly pushed a cobble, (Fill).	
10							12				
	4D	24/0	11.0 - 13.0	5/4/5/8	9	11	43			No Recovery Note: Possibly pushing a cobble.	
							31				
							29			Note: Lost water from approximately 13.0 to 15.0 ft. Drill action indicates possible cobbles.	
							28				
15	5D	24/8	15.0 - 17.0	1/1/6/5	7	9	27	167.0		15.0 — Grey-brown, wet, stiff, SILT, (Marine Deposit).	G#744571 WC=25 LL=40 PL=29 PI=11
							25				
							19			Note: Lost all drill water at approximately 19.0 ft.	
							22				
							24				
20											
	6D	24/18	23.0 - 25.0	WOR/WOR/11/19	11	14				Similar to 5D, except soft to stiff, (Marine Deposit).	
	V1		23.6 - 24.0	Su=270/80 psf						55 x 110 mm vane raw torque readings:	
25	V2		24.6 - 25.0	Su=1,395/310 psf				157.8		V1: 70/20 in-lbs V2: 360/80 in-lbs — 24.2	

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: Farnham Bridge No. 2274
Location: Pittsfield, Maine

Boring No.: BB-PFB-101
WIN: 26109.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	182.0	**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:	8-24-2023/8-25-2023	**Drilling Method:**	HW to 43.5 ft	**Core Barrel:**	--
Boring Location:	191+34, 6 ft LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	11 ft (Approx.)
Hammer Efficiency Factor:	0.76	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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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										Brown-grey, wet, stiff, Silty CLAY, some fine sand, (Marine Deposit).	
								154.0		28.0	
30	7D	24/12	30.0 - 32.0	16/16/19/22	35	44				Grey-brown, wet, dense, fine to coarse SAND, some silt, some gravel, poorly-graded, well bonded, (Marine Deposit).	G#744575 A-2-4, SM
35								147.0		35.0	
	8D	24/12	35.0 - 37.0	1/2/4/8	6	8				Dark grey, wet, medium stiff, CLAY, (Marine Deposit).	G#744572 WC=25 LL=36 PL=23 PI=13
40	9D	24/20	40.0 - 42.0	5/11/19/28	30	38				Similar to S8, except hard, (Marine Deposit).	
	10D	24/16	42.0 - 44.0	6/10/27/42	37	47				Dark grey, wet, hard, Silty CLAY with occasional fine sand seams, (Marine Deposit).	
								138.5		43.5	
								138.0		Grey, wet, dense, fine SAND, little silt, (Marine Deposit). 44.0	
										Bottom of Exploration at 44.0 feet below ground surface.	
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:	Farnham Bridge No. 2274	**Boring No.:**	BB-PFB-102		
Location:	Pittsfield, Maine	**WIN:**	26109.00		
Driller:	New England Boring Contractors	**Elevation (ft.)**	179.5	**Auger ID/OD:**	--
Operator:	B. Gome	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	R. Estes/H. Hollauer	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	HW-140#/30"; SS-140#/30"
Date Start/Finish:	8-18-2023/8-21-2023	**Drilling Method:**	HW to 39 ft	**Core Barrel:**	--
Boring Location:	191+80, 9 ft RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	5.6 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										Bituminous Concrete (19 in.)	
	1D	24/13	1.6 - 3.6	5/10/11/12	21	27		177.9		1.6 Brown, dry, medium dense, Sandy fine to coarse GRAVEL, trace silt, well-graded, (Fill). PID Reading = 4.7 ppm	G#744576 A-1-a, GW-GM
							74				
							68				
	2D	5/4	4.0 - 4.4	50(5")			41			Similar to S1, except very dense, SPT refusal on cobble, (Fill). PID Reading = 3.0 ppm	G#744577 A-1-a, GW-GM
5	3D	24/12	5.0 - 7.0	37/19/17/17	36	46	81			Brown, moist, dense, Sandy fine to coarse GRAVEL, little silt, few cobbles, well-graded, (Fill). PID Reading = 2.9 ppm	
							174				
	4D	24/6	7.0 - 9.0	3/13/11/8	24	30	84			Similar to S3, except medium dense, trace silty clay, (Fill). PID Reading = 2.9 ppm	
							67				
	5D	24/2	9.0 - 11.0	19/12/16/31	28	35	11			Similar to S3, except dense, (Fill). Note: Encountered wood from 9.6 to 10.5 ft. PID Reading = 3.1 ppm	
10							9				
	6D	24/15	11.0 - 13.0	5/5/5/5	10	13	21	167.8		11.7	
							17	167.7		Dark brown ORGANIC SILT 11.8	
							33			Grey-brown to grey mottled, wet, stiff, Silty CLAY, (Marine Deposit). PID Reading = 3.4 ppm Note: Attempt vane at 13.0 ft, unable to push vane.	
15	7D V1		15.0 - 17.0 15.6 - 16.0	WOH(24") Su=1,085/215 psf			10			Grey, wet, stiff to medium stiff, Lean CLAY, (Marine Deposit). 55 x 110 mm vane raw torque readings: V1: 280/55 in-lbs V2: 210/45 in-lbs	G#744573 WC=31 LL=31 PL=21 PI=10
	V2		16.6 - 17.0	Su=815/175 psf			6				
							7				
	U1	24/24	18.2 - 20.2	Push			7				C#IP-1
							10				
20	8D	24/24	20.2 - 22.2	13/12/12/23	24	30	21	159.3		20.2 Grey, wet, medium dense, fine SAND, little silt, poorly-graded, (Marine Deposit). PID Reading = 0 ppm Note: Vane refusal at 20.6 ft. Grey, medium dense, fine to coarse SAND, little silt, trace fine gravel, moderately bonded, no odor, (Marine Deposit).	
							15				
							22				
							26				
25							39				

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: Farnham Bridge No. 2274		**Boring No.:**	BB-PFB-102
Location: Pittsfield, Maine		**WIN:**	26109.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	179.5	**Auger ID/OD:**	--
Operator:	B. Gome	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	R. Estes/H. Hollauer	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	HW-140#/30"; SS-140#/30"
Date Start/Finish:	8-18-2023/8-21-2023	**Drilling Method:**	HW to 39 ft	**Core Barrel:**	--
Boring Location:	191+80, 9 ft RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	5.6 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	9D	24/10	25.0 - 27.0	14/20/15/10	35	44	40			Grey, wet, dense, fine to coarse SAND, some silt, trace fine gravel, poorly-graded, no odor, (Marine Deposit). PID Reading = 0 ppm	G#744578 A-2-4, SM
							35				
							30				
							24				
							27				
30	10D	24/6	30.0 - 32.0	WOH/10/17/20	27	34	25			Grey, wet, dense, fine to coarse SAND, little silt, trace fine to coarse gravel, trace clay, well-graded, moderately bonded, no odor, (Marine Deposit). PID Reading = 0 ppm	
							31				
							36				
							89				
							96				
35	11D	24/6	35.0 - 37.0	2/3/3/2	6	8				Gravel wash, advanced casing to 37.0 ft and resampled. PID Reading = 0 ppm	
	12D	24/12	37.0 - 39.0	12/11/24/37	35	44				Grey to dark grey, wet, dense, fine SAND, little silt, trace medium to coarse sand, poorly-graded, no odor, (Marine Deposit). PID Reading = 0 ppm	
								140.5		39.0 **Bottom of Exploration at 39.0 feet below ground surface.**	
40											
45											
50											

Remarks:

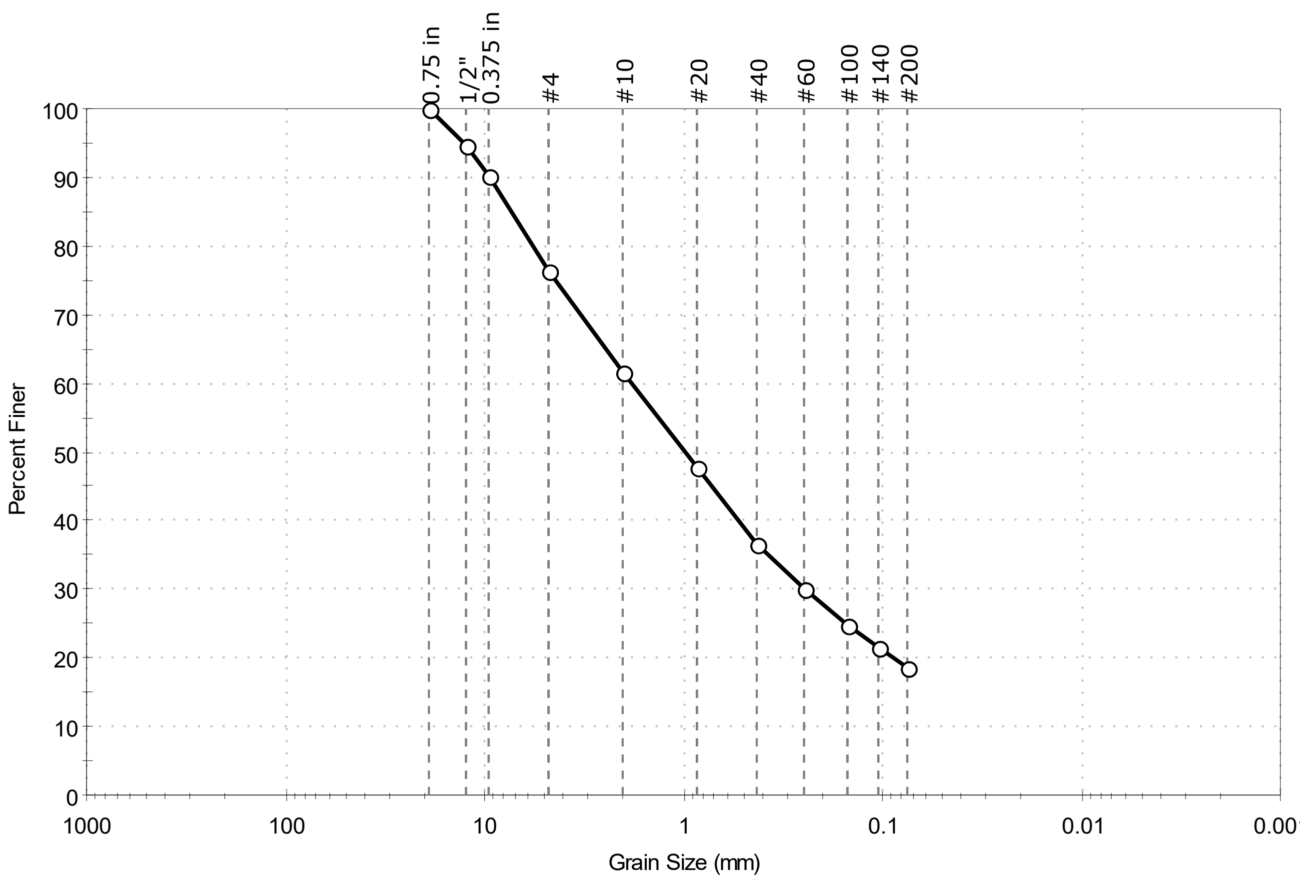
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* 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.

APPENDIX B
Laboratory Test Results

Client:	Haley & Aldrich, Inc.		
Project:	Farnham Bridge Culvert Replace		
Location:	Pittsfield, ME	Project No:	GTX-318150
Boring ID: BB-PFB-101	Sample Type: jar	Tested By:	ckg
Sample ID: 1D	Test Date: 11/22/23	Checked By:	jsc
Depth : 5-7'	Test Id: 744574		
Test Comment:	---		
Visual Description:	Moist, brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	23.6	57.7	18.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
1/2"	12.50	95		
0.375 in	9.50	90		
#4	4.75	76		
#10	2.00	62		
#20	0.85	48		
#40	0.42	37		
#60	0.25	30		
#100	0.15	25		
#140	0.11	21		
#200	0.075	19		

Coefficients

D_{85} =7.3272 mm
D_{30} =0.2457 mm
D_{60} =1.8082 mm
D_{15} =N/A
D_{50} =0.9688 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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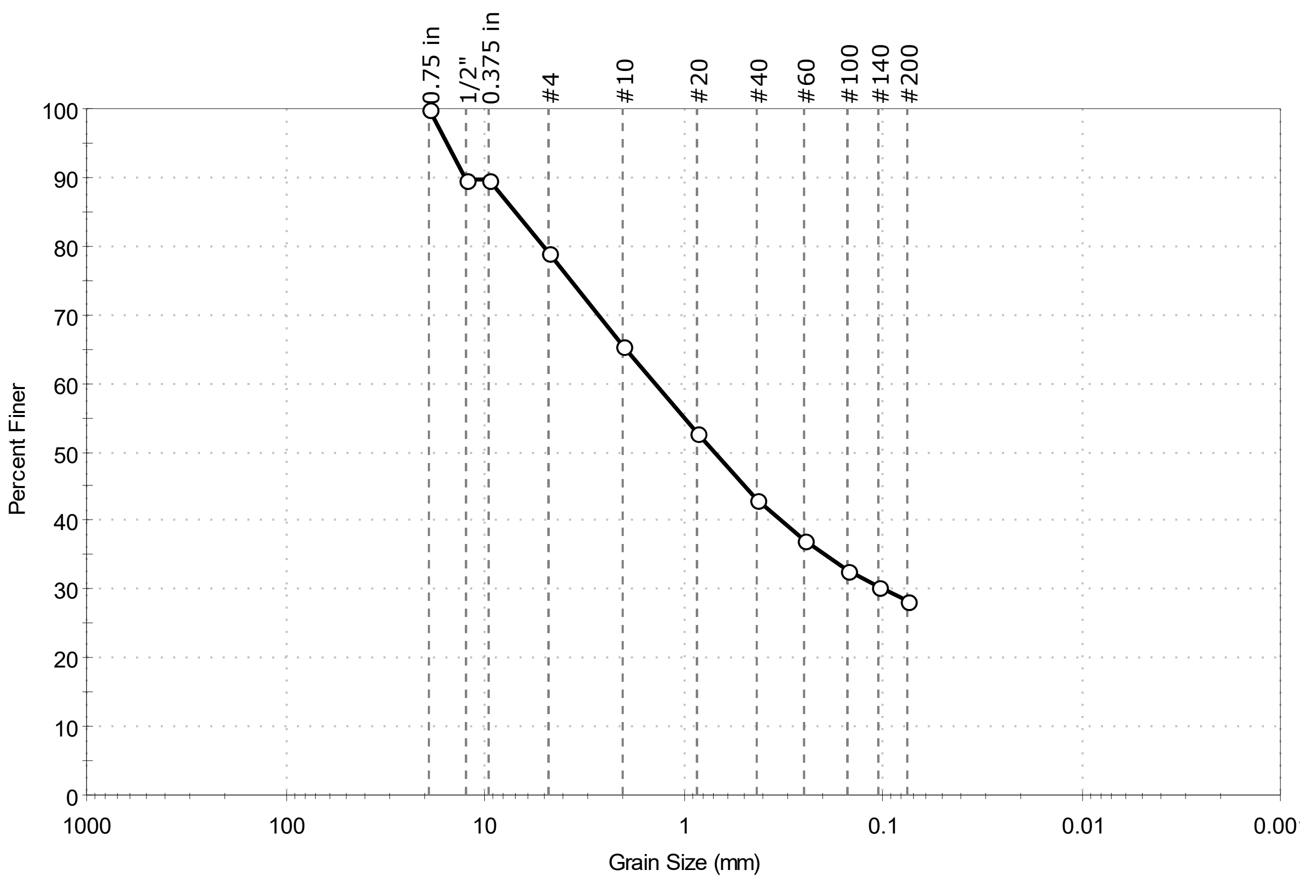
Client:	Haley & Aldrich, Inc.		
Project:	Farnham Bridge Culvert Replace		
Location:	Pittsfield, ME	Project No:	GTX-318150
Boring ID: BB-PFB-101	Sample Type: jar	Tested By:	ckg
Sample ID: 7D	Test Date: 11/22/23	Checked By:	jsc
Depth : 30-32'	Test Id: 744575		

Test Comment: ---
Visual Description: Moist, brownish gray silty sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	21.0	50.7	28.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
1/2"	12.50	90		
0.375 in	9.50	90		
#4	4.75	79		
#10	2.00	65		
#20	0.85	53		
#40	0.42	43		
#60	0.25	37		
#100	0.15	33		
#140	0.11	30		
#200	0.075	28		

Coefficients

D_{85} =7.0334 mm
D_{30} =0.1001 mm
D_{60} =1.3889 mm
D_{15} =N/A
D_{50} =0.6950 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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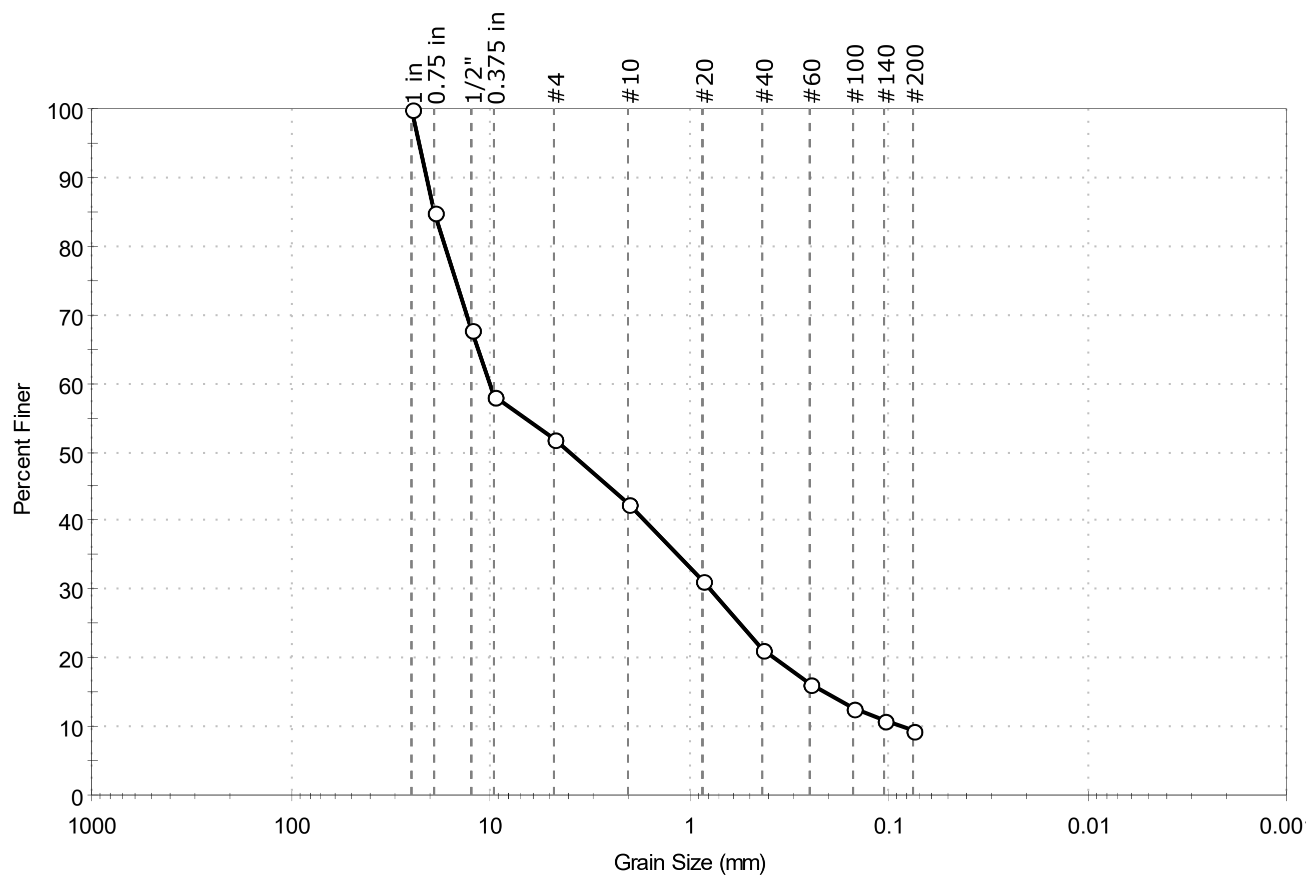
Client:	Haley & Aldrich, Inc.		
Project:	Farnham Bridge Culvert Replace		
Location:	Pittsfield, ME	Project No:	GTX-318150
Boring ID: BB-PFB-102	Sample Type: jar	Tested By:	ckg
Sample ID: 1D	Test Date: 11/22/23	Checked By:	jsc
Depth : 1.6-3.6'	Test Id: 744576		
Test Comment:	---		
Visual Description:	Moist, dark brown sandy gravel with silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	48.0	42.4	9.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	85		
1/2"	12.50	68		
0.375 in	9.50	58		
#4	4.75	52		
#10	2.00	42		
#20	0.85	31		
#40	0.42	21		
#60	0.25	16		
#100	0.15	13		
#140	0.11	11		
#200	0.075	9.6		

Coefficients

D_{85} =19.0431 mm | D_{30} =0.7746 mm

D_{60} =10.0068 mm | D_{15} =0.2097 mm

D_{50} =3.9846 mm | D_{10} =0.0841 mm

C_u =118.987 | C_c =0.713

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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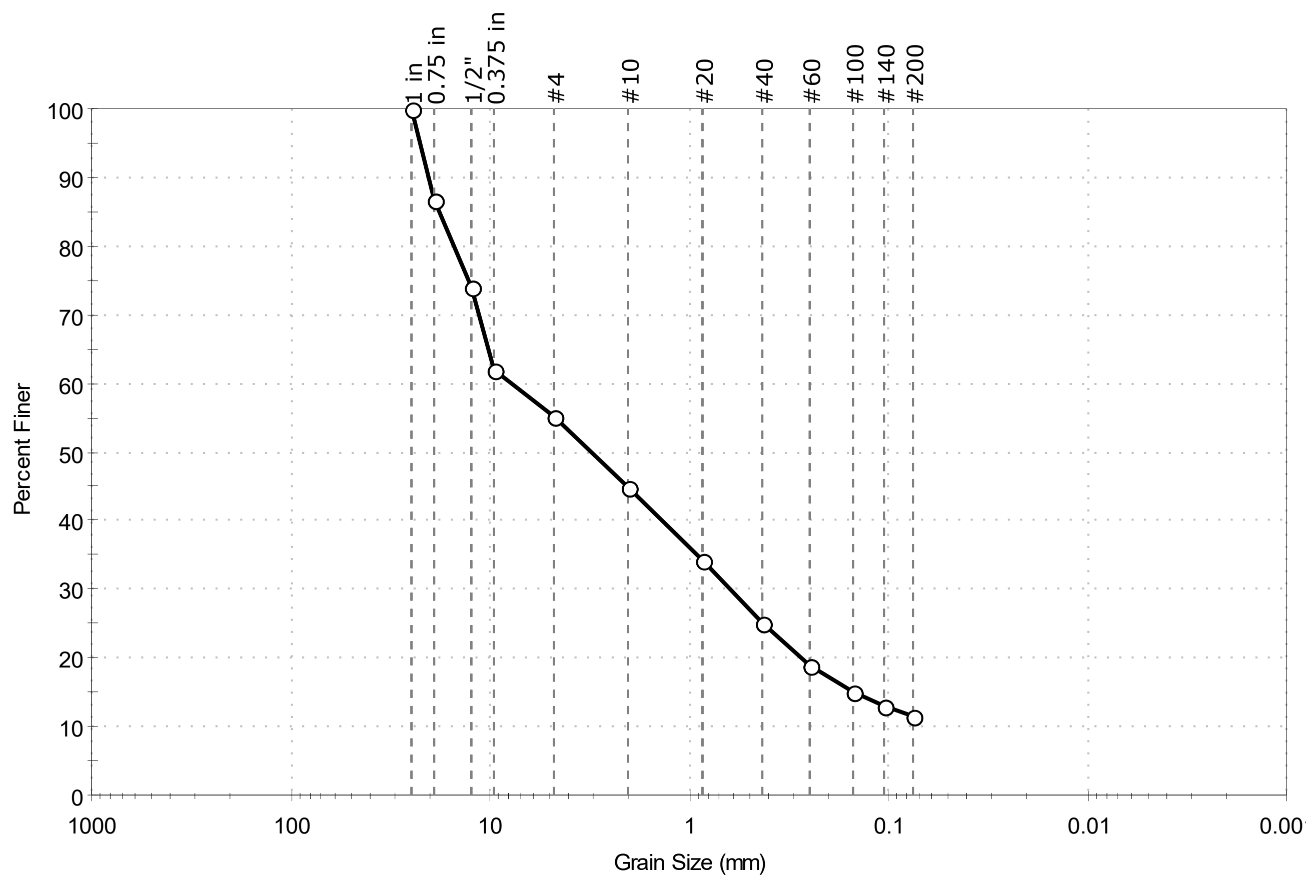
Client:	Haley & Aldrich, Inc.			
Project:	Farnham Bridge Culvert Replace			
Location:	Pittsfield, ME		Project No:	GTX-318150
Boring ID:	BB-PFB-102	Sample Type: jar	Tested By:	ckg
Sample ID:	3D	Test Date: 11/22/23	Checked By:	jsc
Depth :	5-7'	Test Id: 744577		

Test Comment: ---

Visual Description: Moist, dark brown sandy gravel with silt

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	44.9	43.7	11.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	87		
1/2"	12.50	74		
0.375 in	9.50	62		
#4	4.75	55		
#10	2.00	45		
#20	0.85	34		
#40	0.42	25		
#60	0.25	19		
#100	0.15	15		
#140	0.11	13		
#200	0.075	11		

Coefficients

D_{85} =17.9515 mm | D_{30} =0.6219 mm

D_{60} =7.8367 mm | D_{15} =0.1507 mm

D_{50} =3.1099 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-a (0))

Sample/Test Description

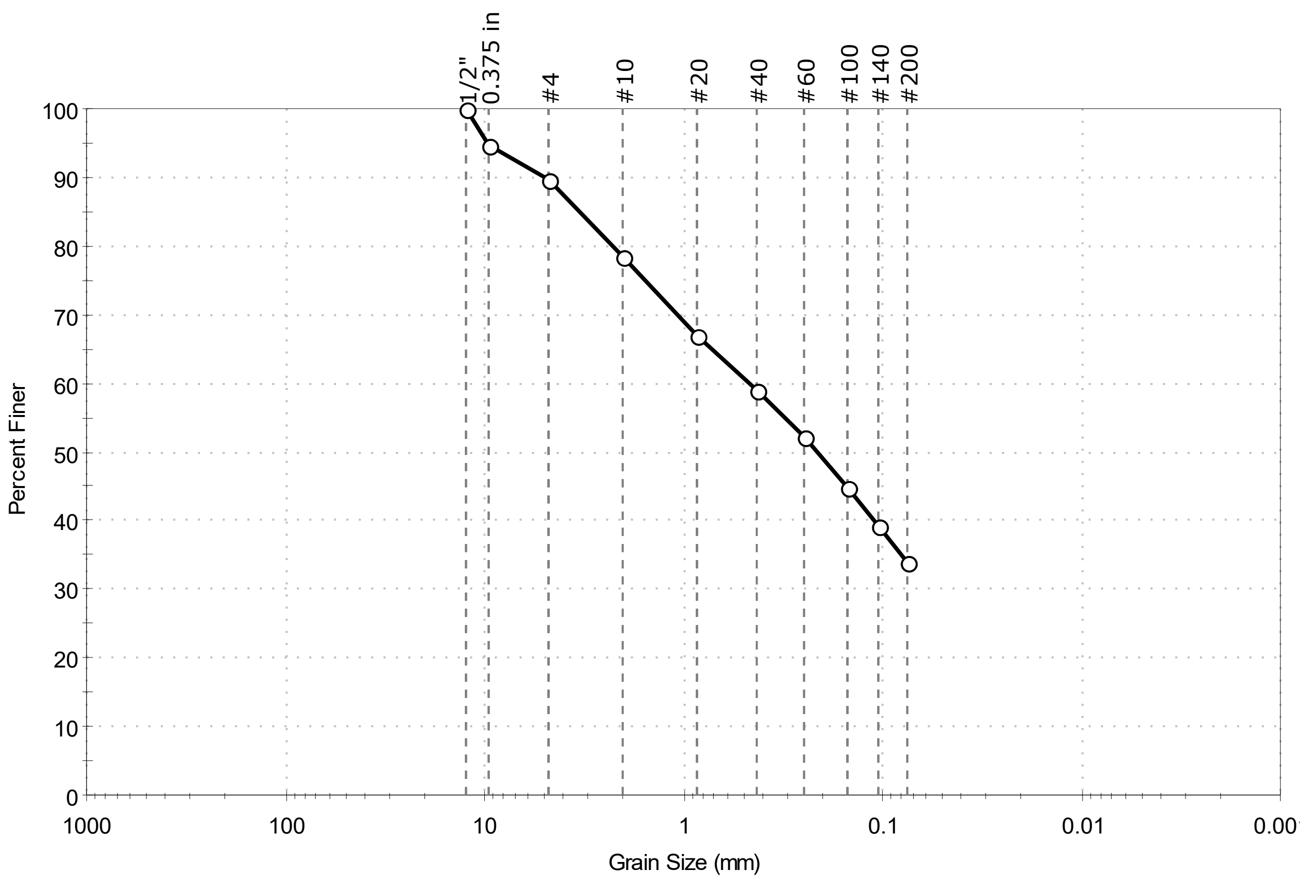
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 11/28/2023 10:06:34 AM

Client: Haley & Aldrich, Inc.			
Project: Farnham Bridge Culvert Replace			
Location: Pittsfield, ME		Project No: GTX-318150	
Boring ID: BB-PFB-102	Sample Type: jar	Tested By: ckg	
Sample ID: 9D	Test Date: 11/22/23	Checked By: jsc	
Depth : 25-27'	Test Id: 744578		
Test Comment: ---			
Visual Description: Moist, grayish brown silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	10.2	55.8	34.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1/2"	12.50	100		
0.375 in	9.50	95		
#4	4.75	90		
#10	2.00	78		
#20	0.85	67		
#40	0.42	59		
#60	0.25	52		
#100	0.15	45		
#140	0.11	39		
#200	0.075	34		

Coefficients

D_{85} =3.2955 mm D_{30} =N/A

D_{60} =0.4620 mm D_{15} =N/A

D_{50} =0.2130 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

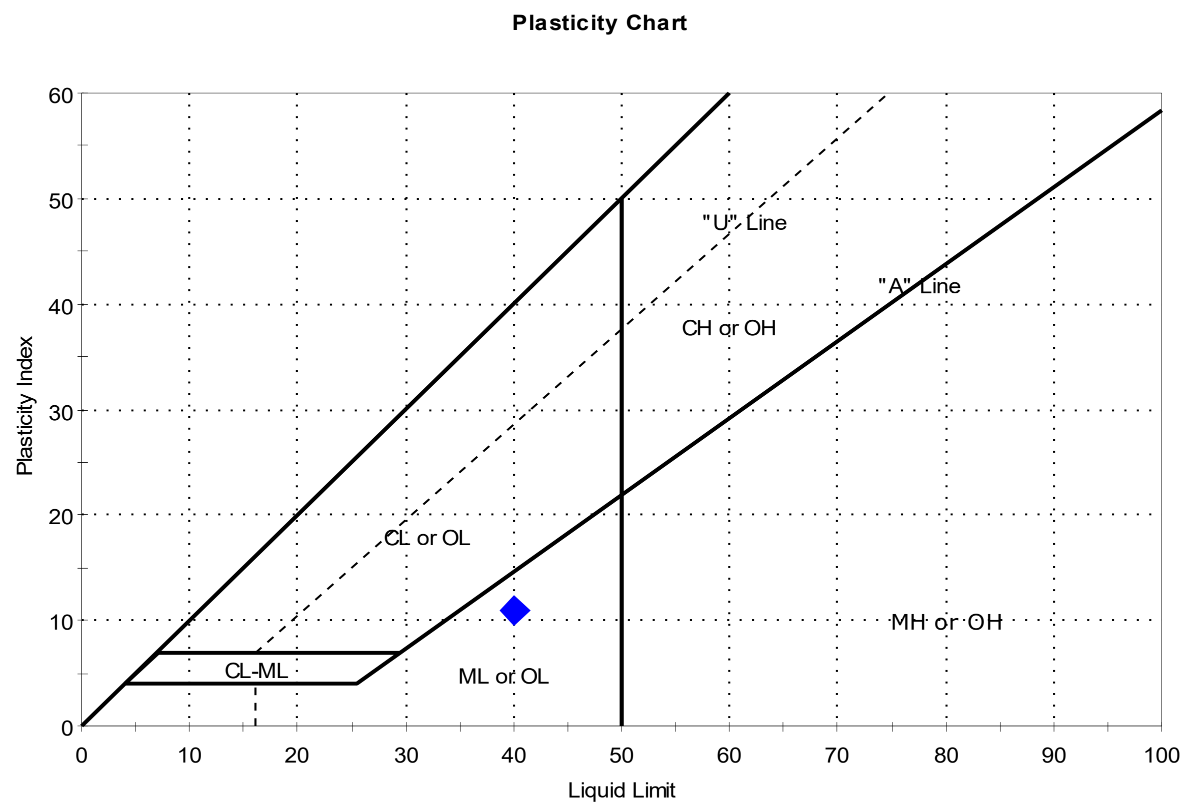
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 11/28/2023 10:06:35 AM

Client:	Haley & Aldrich, Inc.				
Project:	Farnham Bridge Culvert Replace				
Location:	Pittsfield, ME			Project No:	GTX-318150
Boring ID:	BB-PFB-101	Sample Type:	jar	Tested By:	cam
Sample ID:	5D	Test Date:	11/27/23	Checked By:	jsc
Depth :	15-17'	Test Id:	744571		
Test Comment:	---				
Visual Description:	Moist, grayish brown 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'	25	40	29	11	-0.4	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

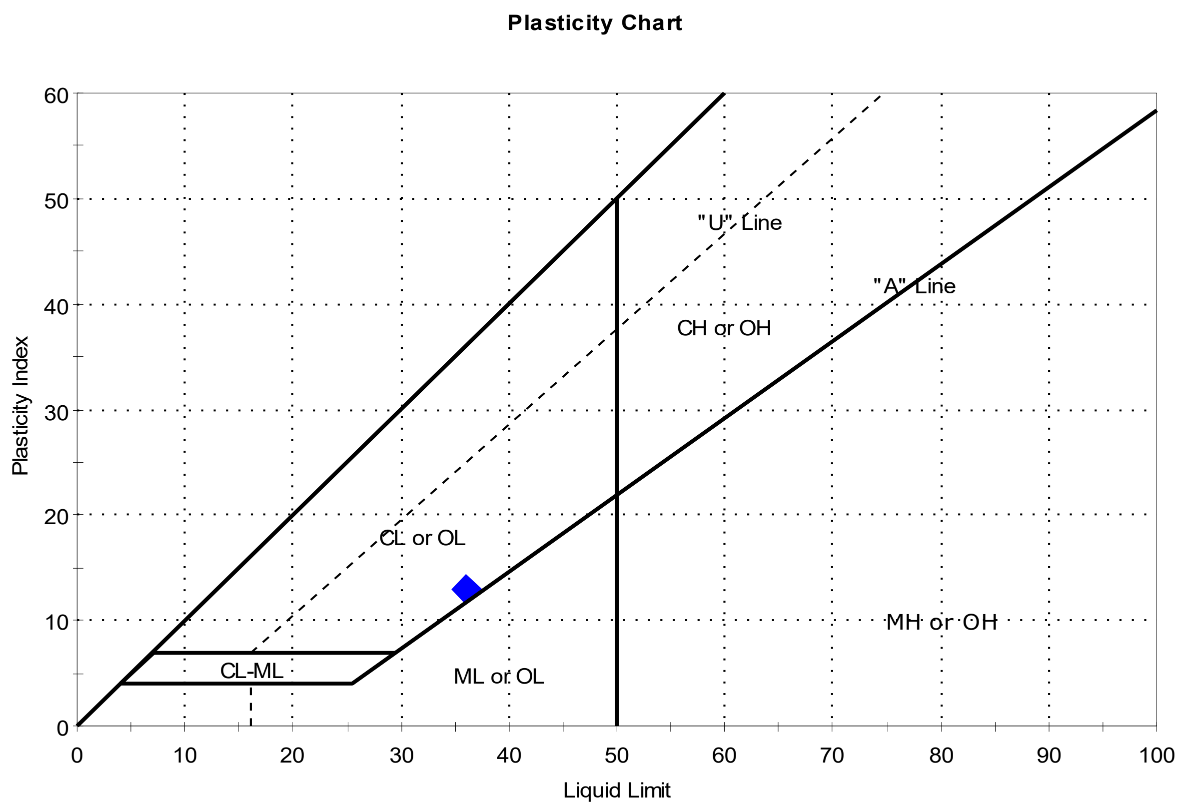
Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	Farnham Bridge Culvert Replace				
Location:	Pittsfield, ME			Project No:	GTX-318150
Boring ID:	BB-PFB-101	Sample Type:	jar	Tested By:	cam
Sample ID:	8D	Test Date:	11/27/23	Checked By:	jsc
Depth :	35-37'	Test Id:	744572		

Test Comment: ---
Visual Description: Moist, 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
◆	8D	B-PFB-10	35-37'	25	36	23	13	0.1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

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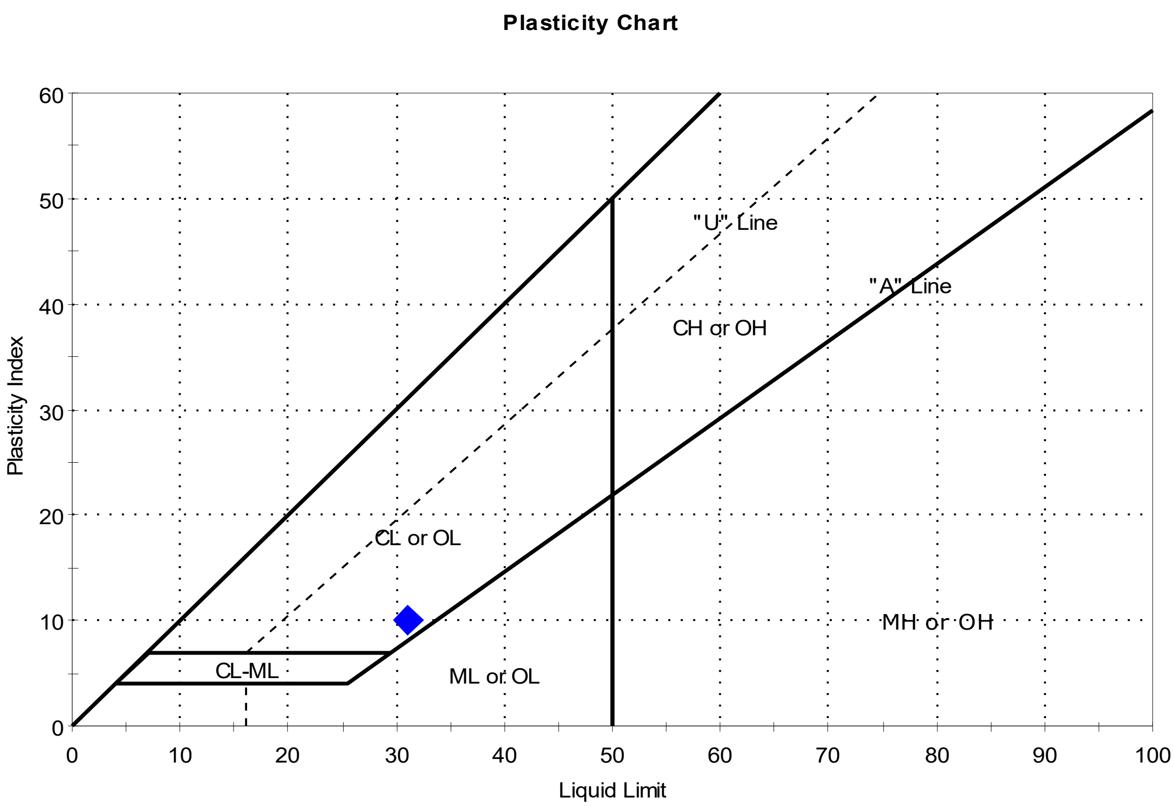
Client:	Haley & Aldrich, Inc.		
Project:	Farnham Bridge Culvert Replace		
Location:	Pittsfield, ME	Project No:	GTX-318150
Boring ID:	BB-PFB-102	Sample Type: jar	Tested By: cam
Sample ID:	7D	Test Date: 11/22/23	Checked By: jsc
Depth :	15-17'	Test Id: 744573	
Test Comment:	---		
Visual Description:	Moist, 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
◆	7D	B-PFB-10	15-17'	31	31	21	10	1	

Sample Prepared using the WET method

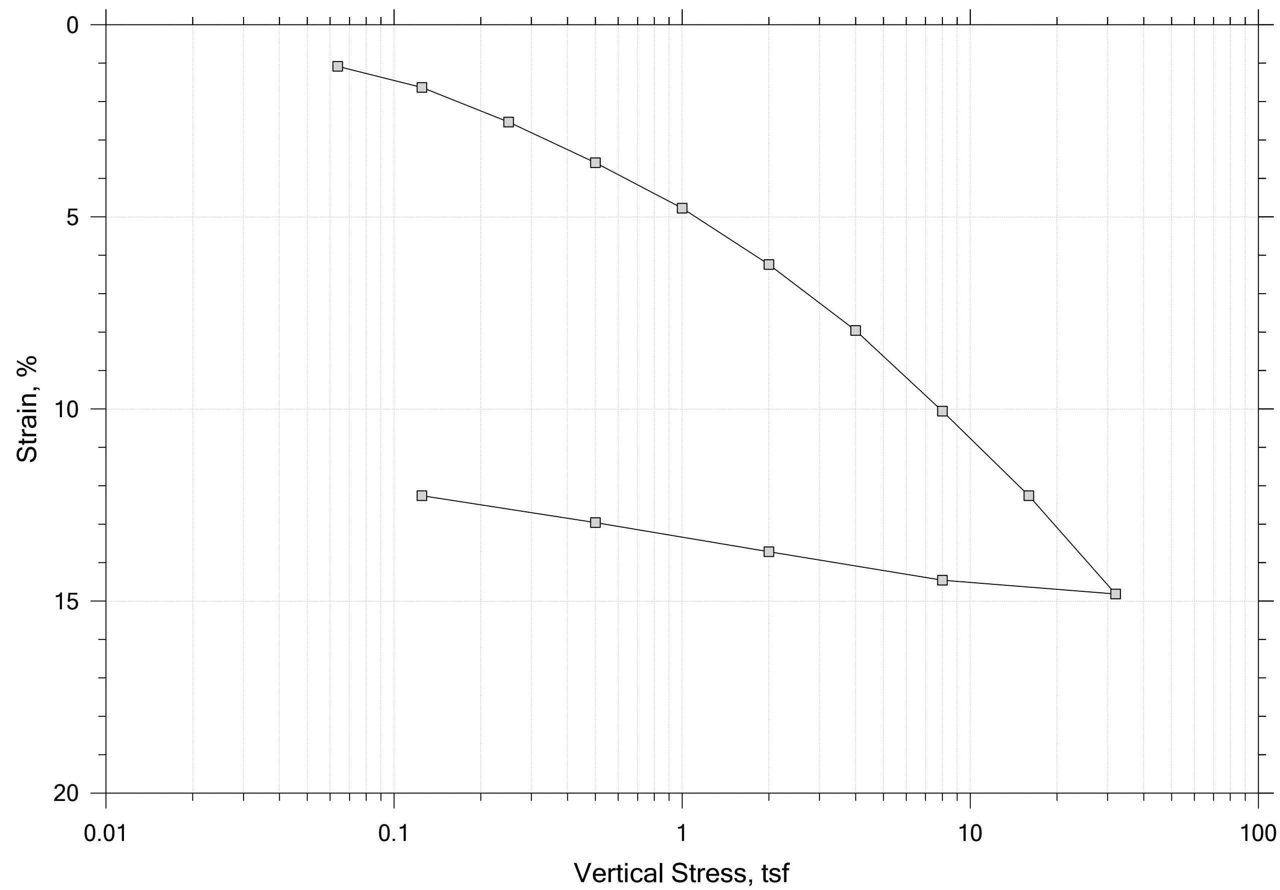
Dry Strength: VERY HIGH

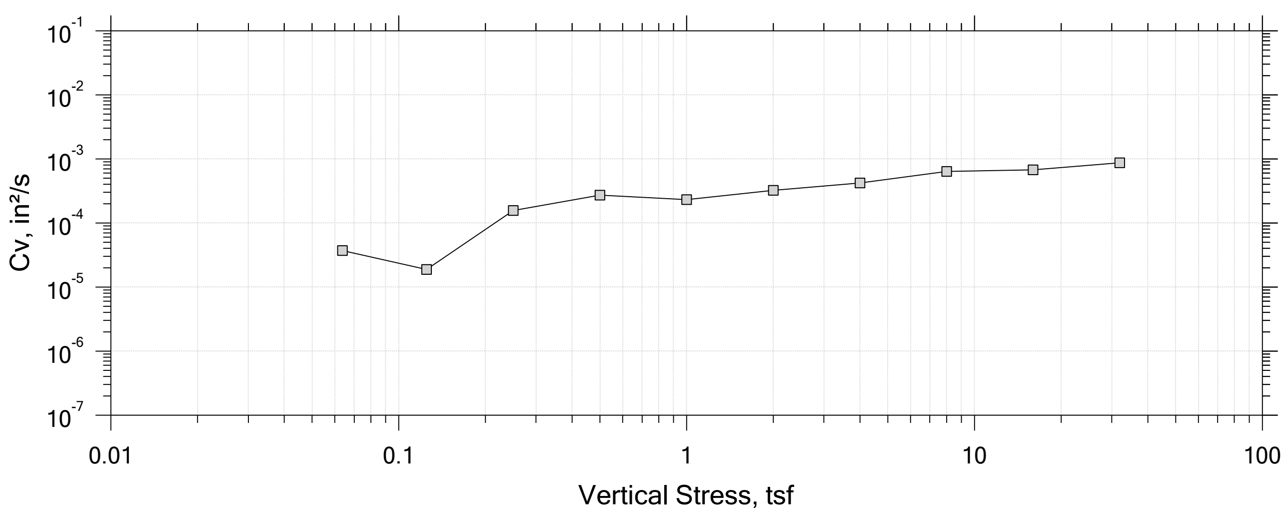
Dilatancy: SLOW

Toughness: LOW

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

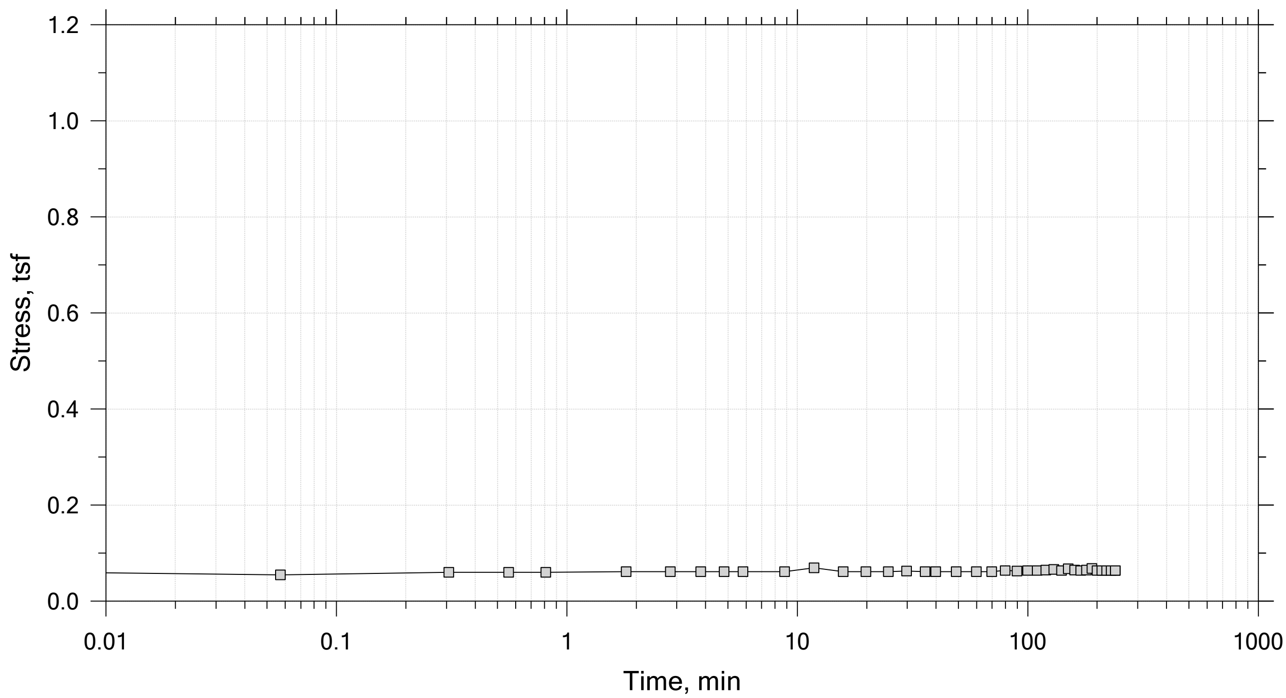


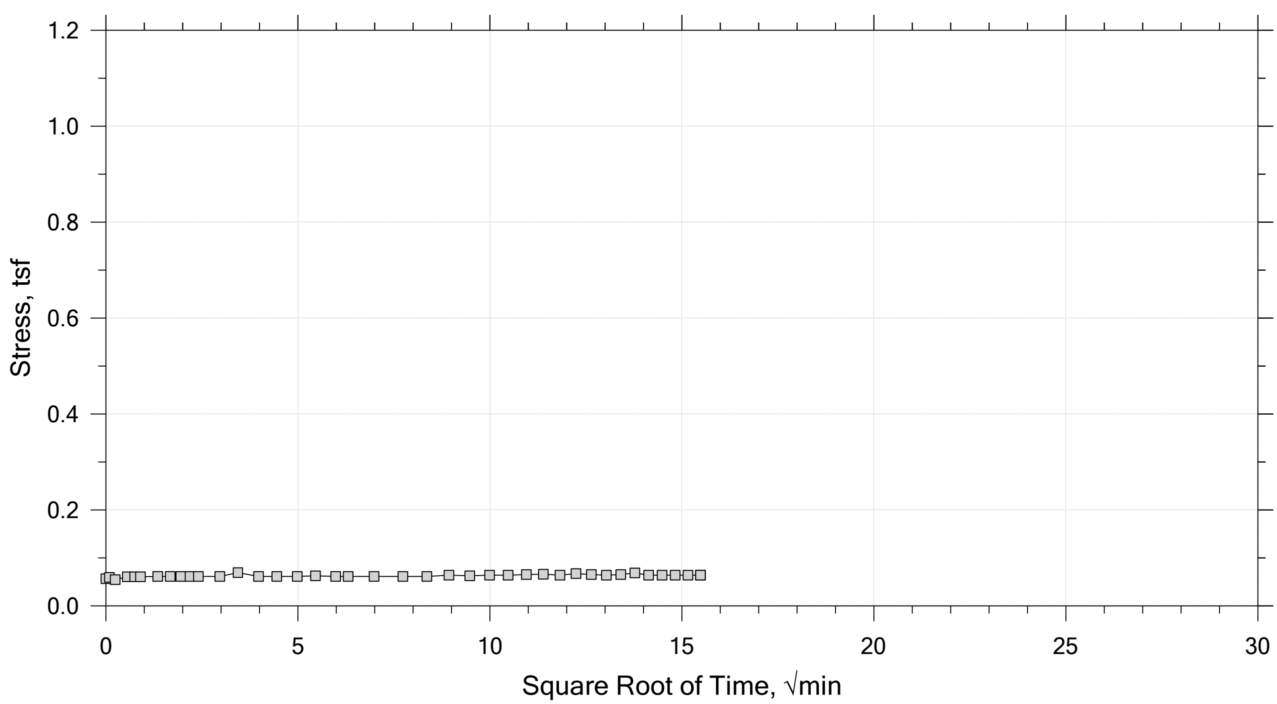


Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: TX-019 Swell Pressure = 0.0637 tsf		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 14
Constant Volume Step
Stress: 0.0637 tsf





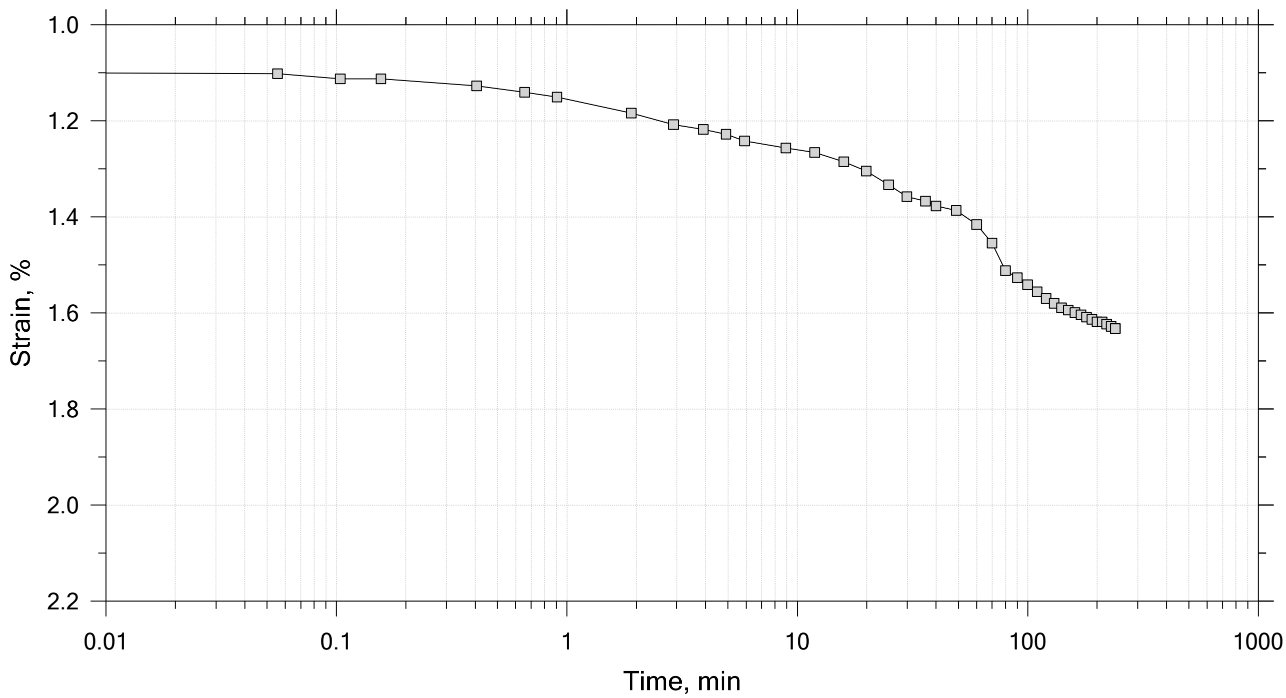
GeoTesting EXPRESS A Sercel Business	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

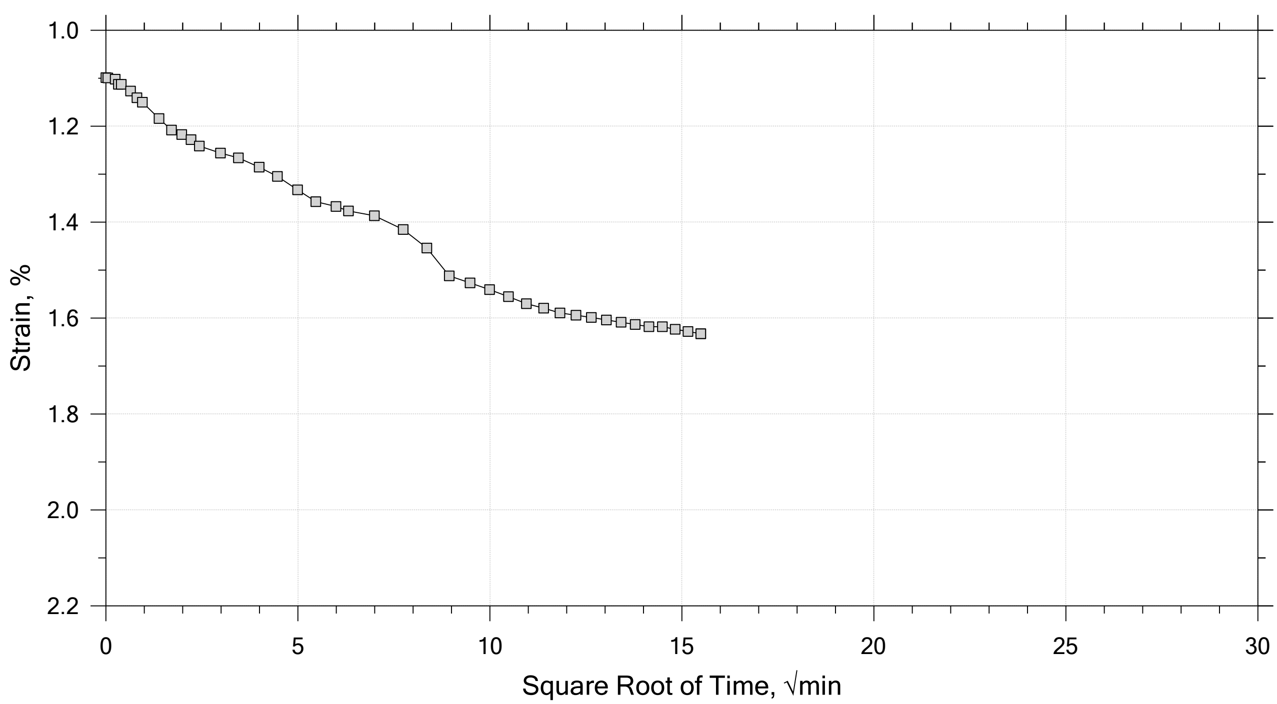
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14

Constant Load Step

Stress: 0.125 tsf





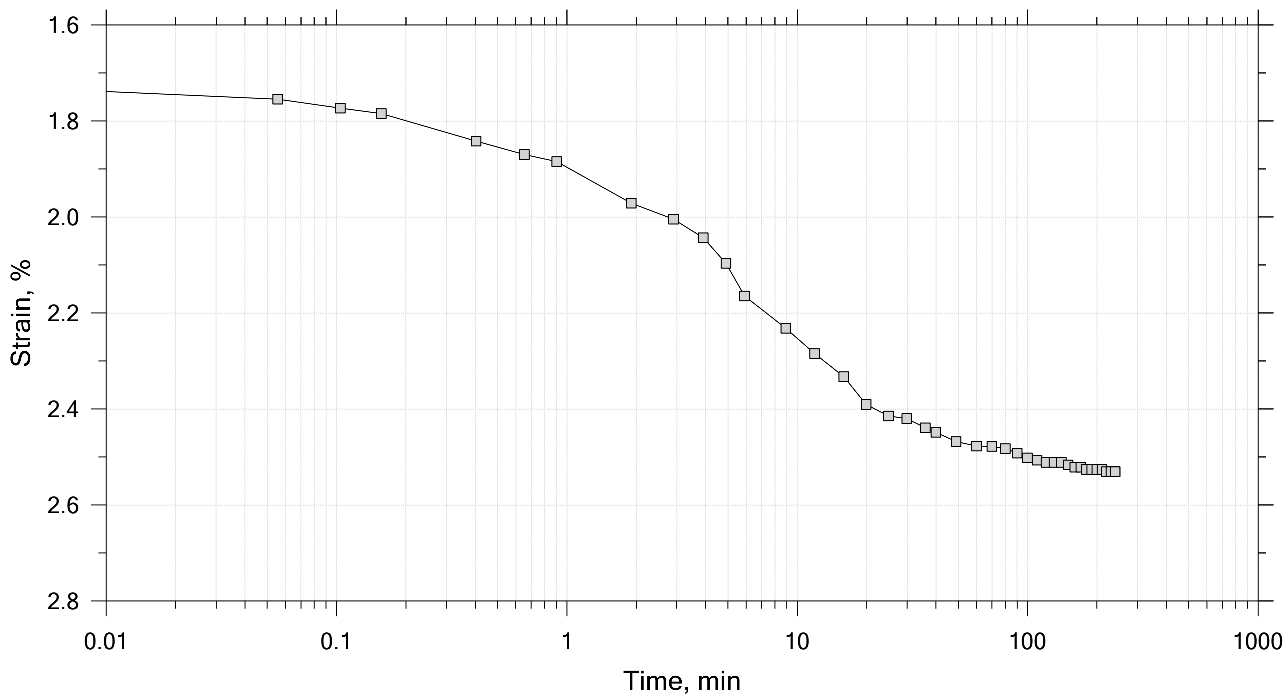
GeoTesting EXPRESS A Sercel Business			
	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

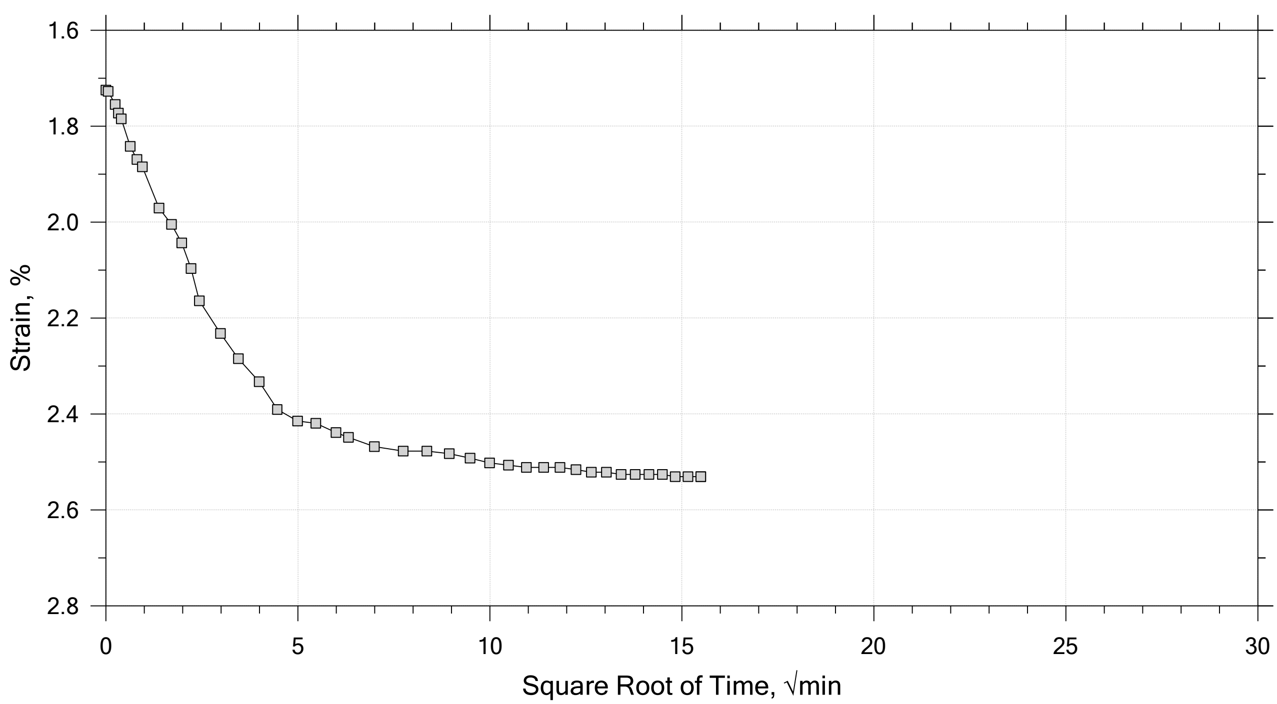
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14

Constant Load Step

Stress: 0.25 tsf





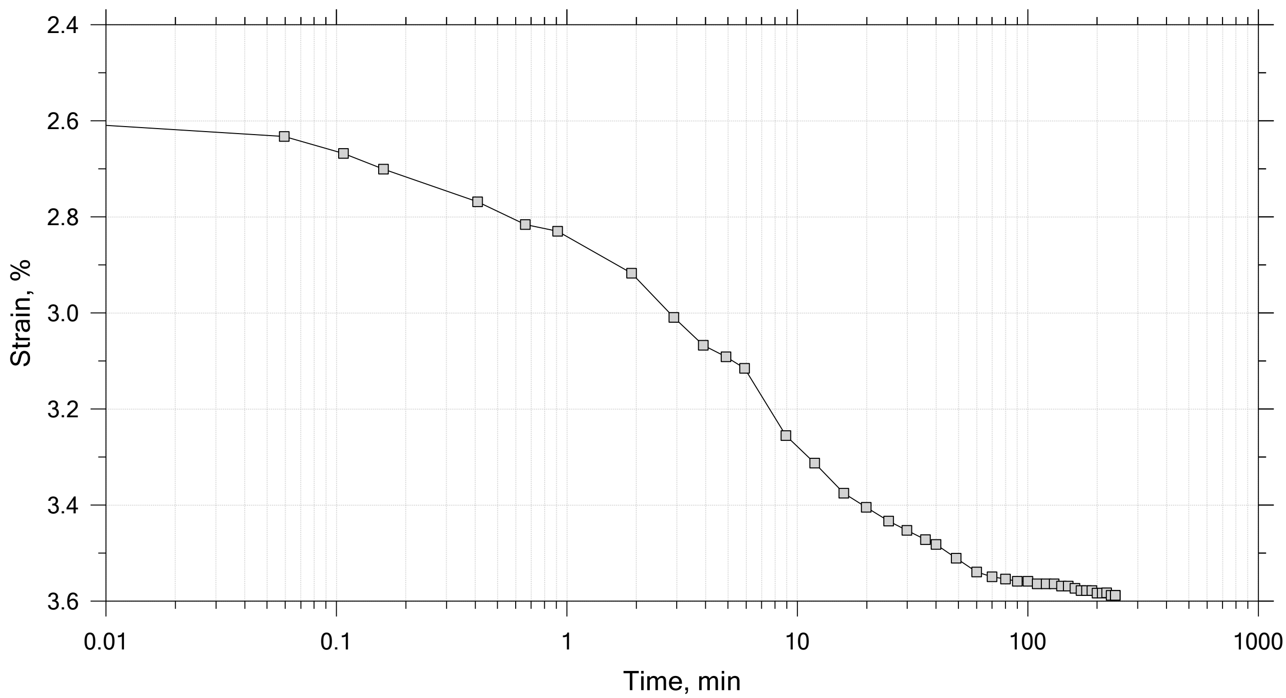
GeoTesting EXPRESS A Sercel Business	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

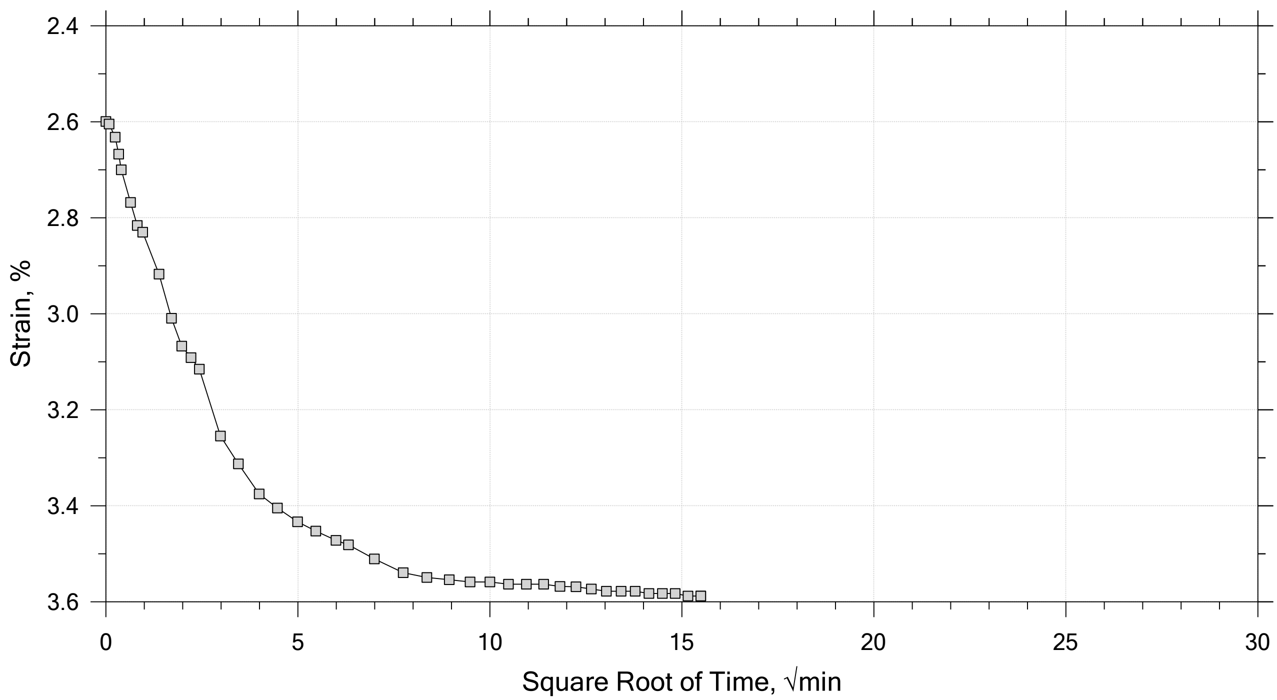
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14

Constant Load Step

Stress: 0.5 tsf





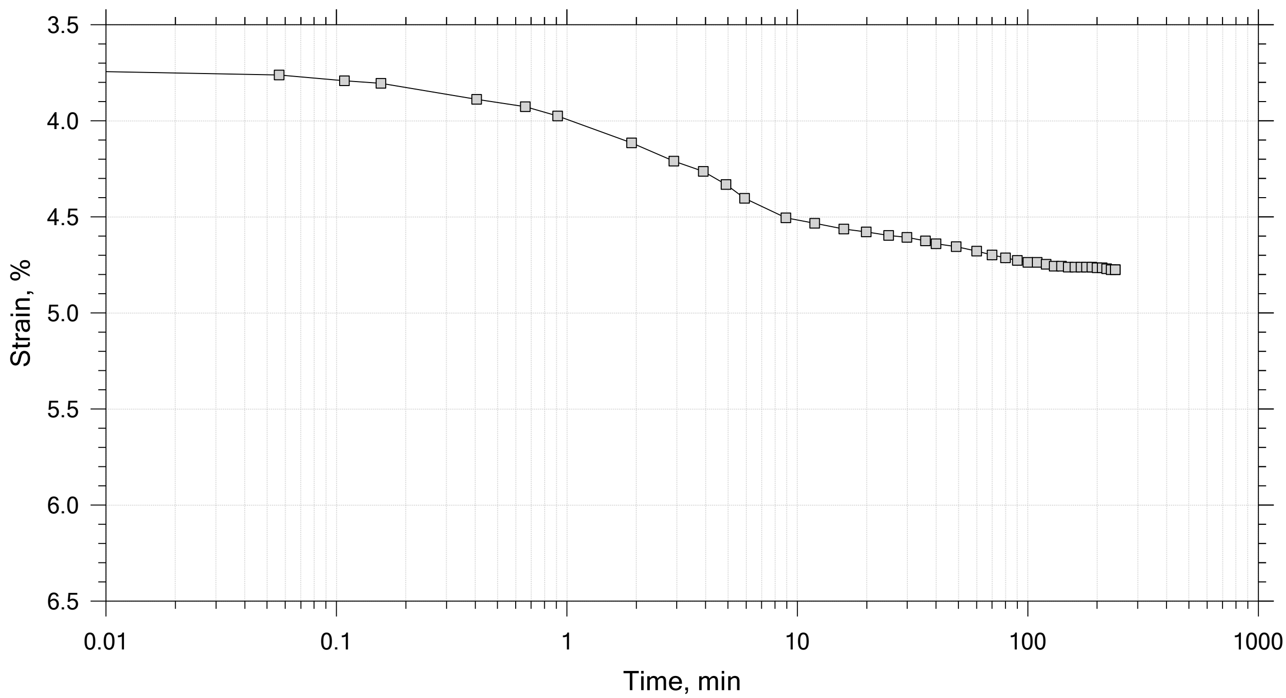
GeoTesting EXPRESS A Sercel Business	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

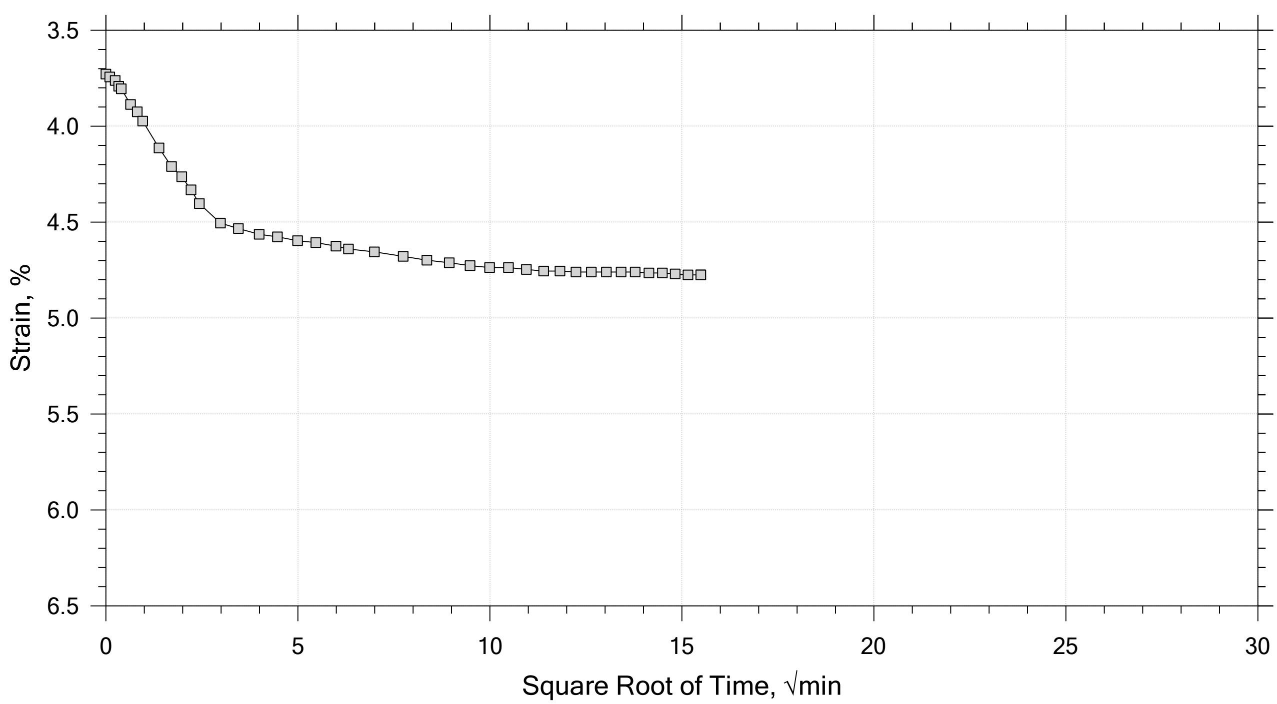
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14

Constant Load Step

Stress: 1 tsf





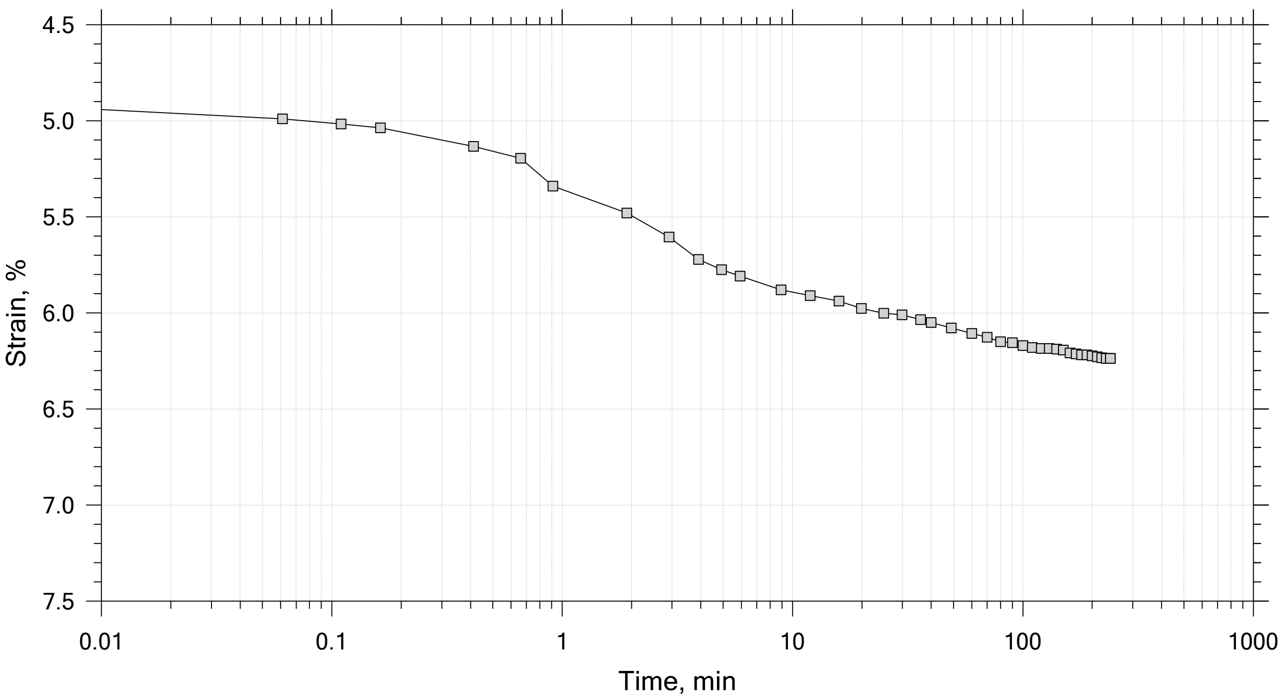
GeoTesting EXPRESS A Sercel Business			
	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

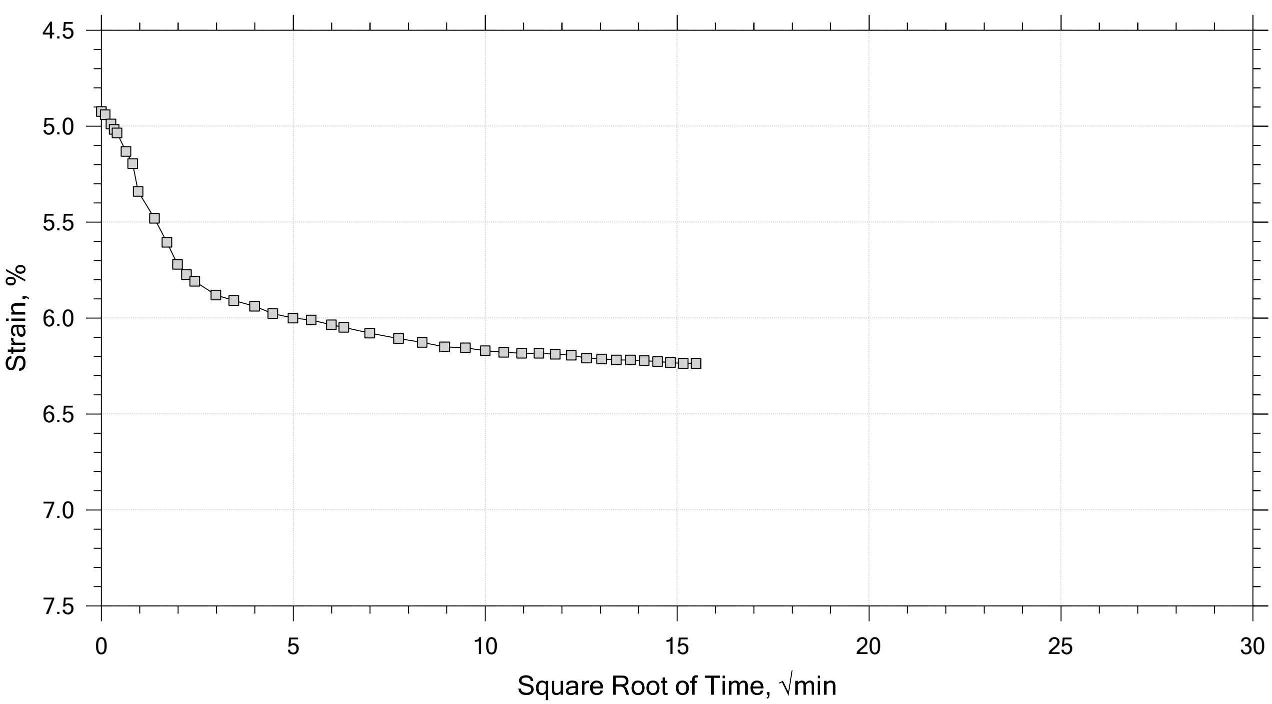
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14

Constant Load Step

Stress: 2 tsf





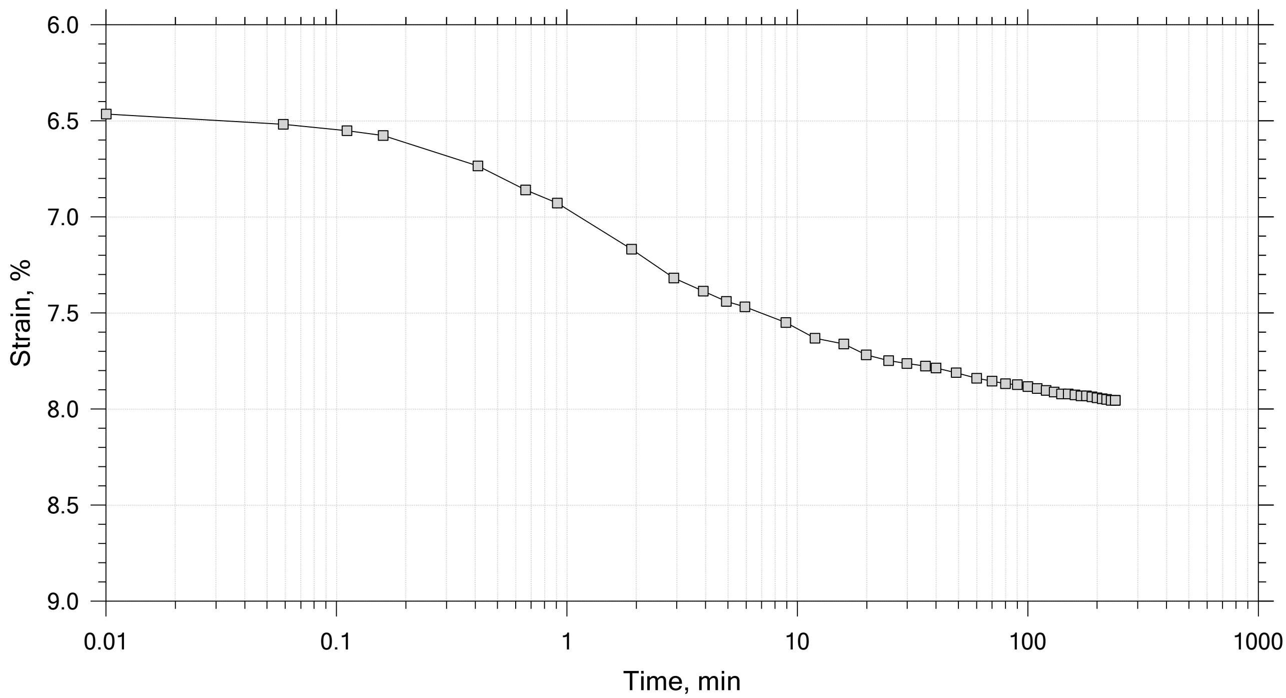
GeoTesting EXPRESS A Sercel Business			
	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

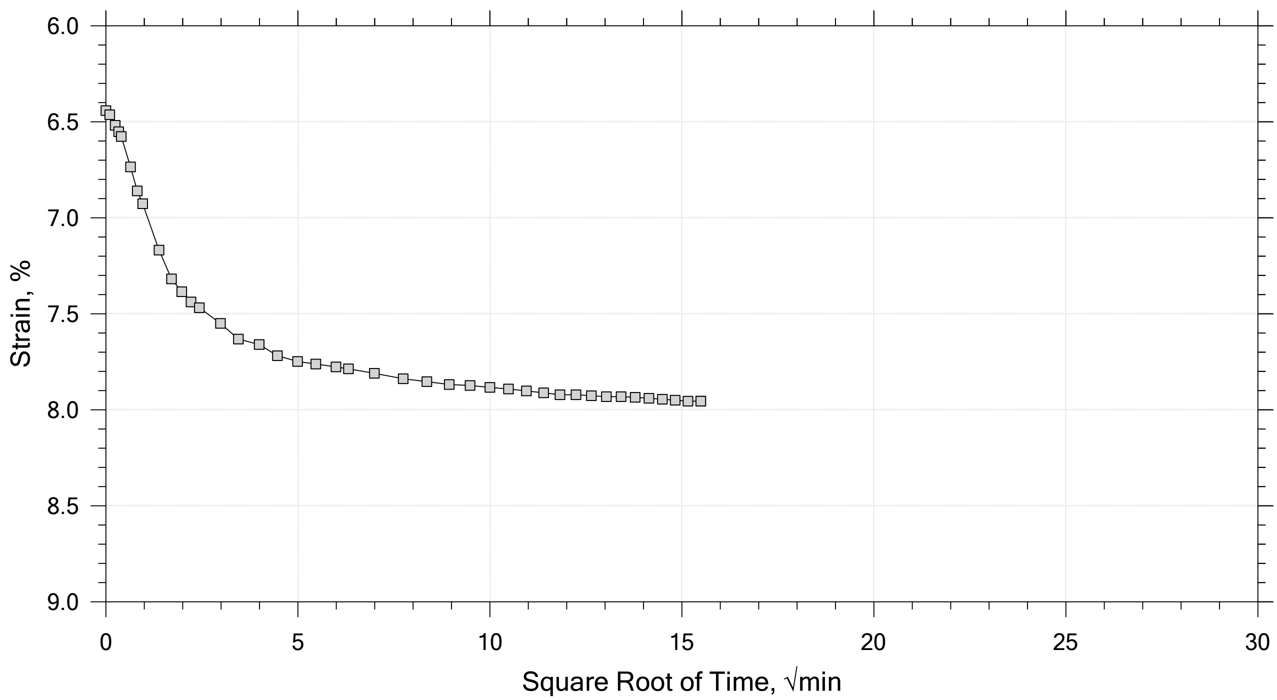
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14

Constant Load Step

Stress: 4 tsf





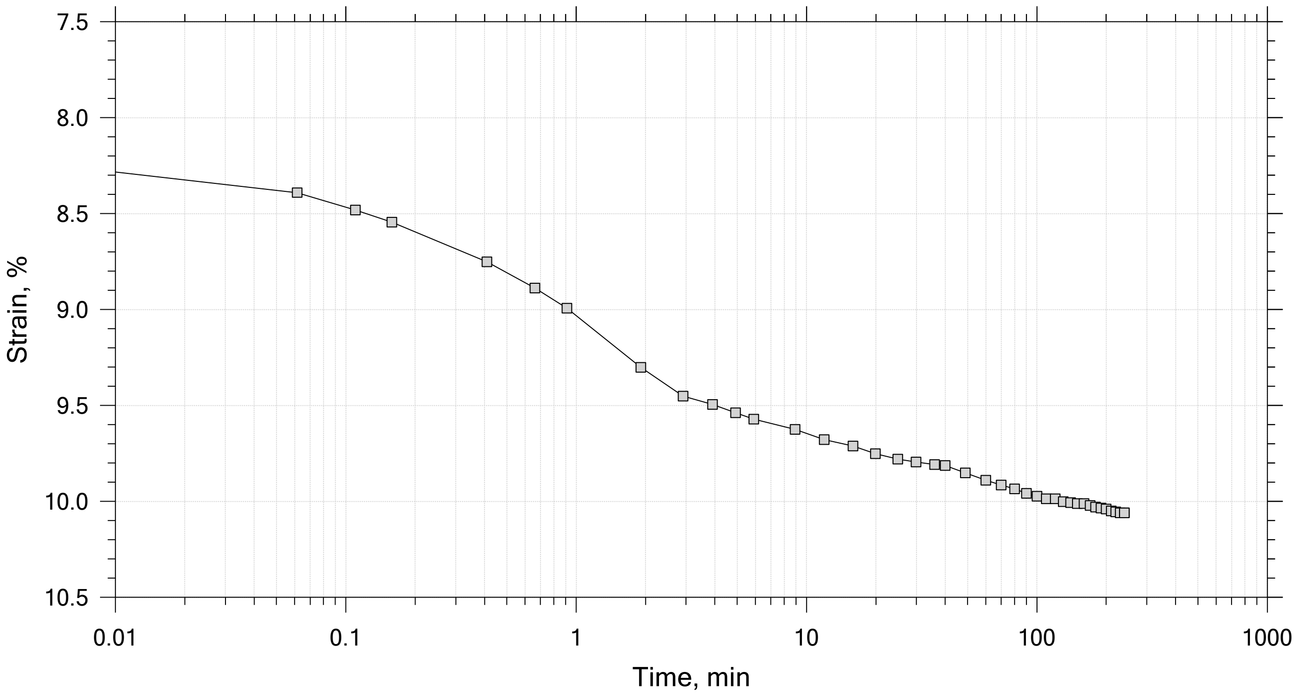
GeoTesting EXPRESS A Sercel Business	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

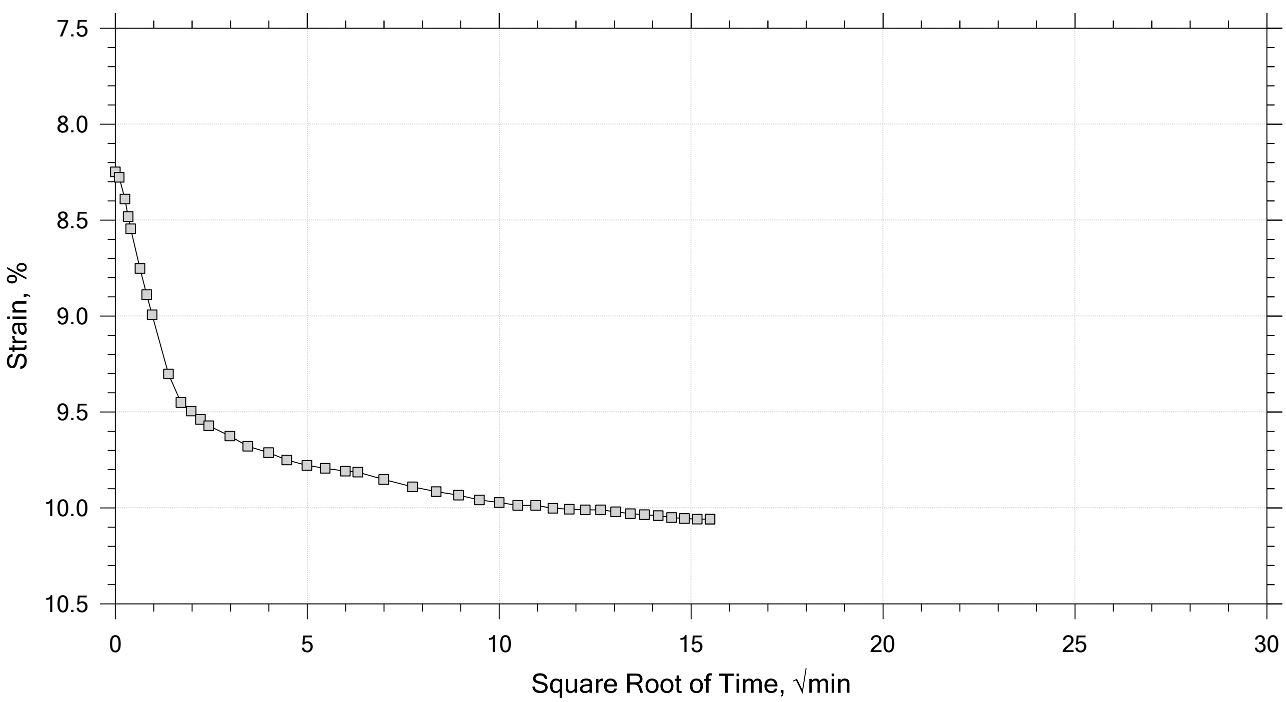
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 8 tsf





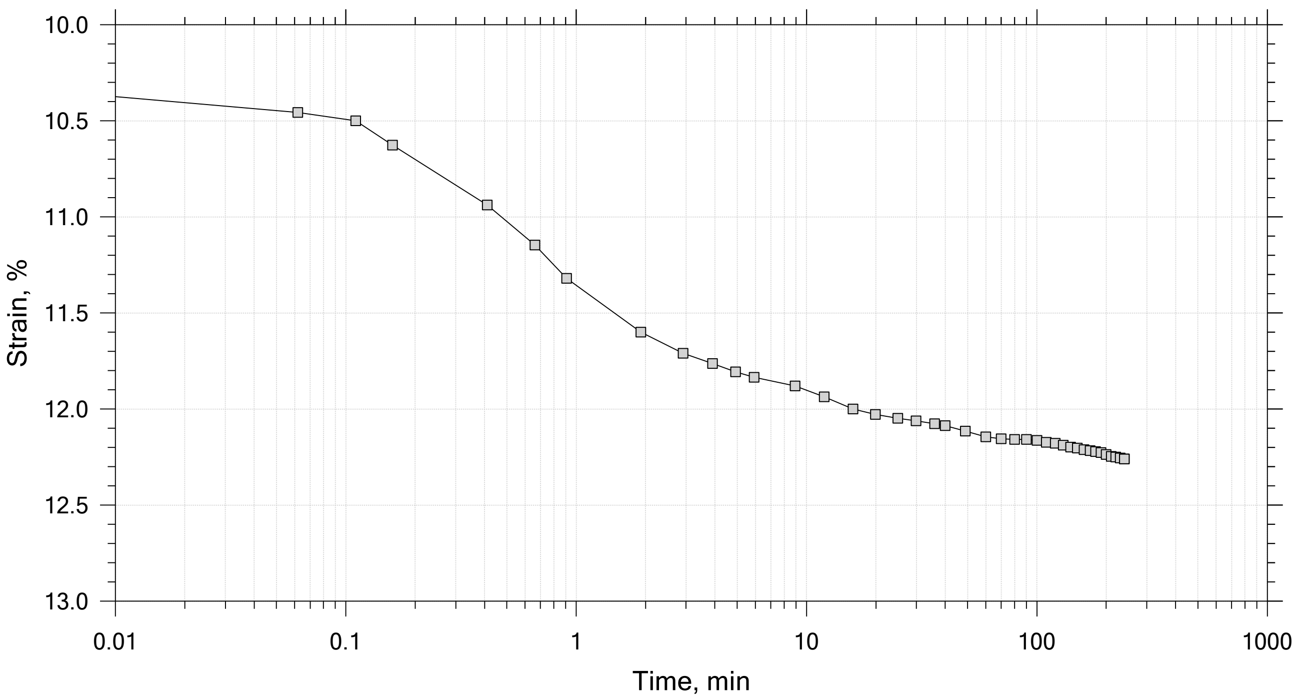
Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: TX-019 Swell Pressure = 0.0637 tsf		

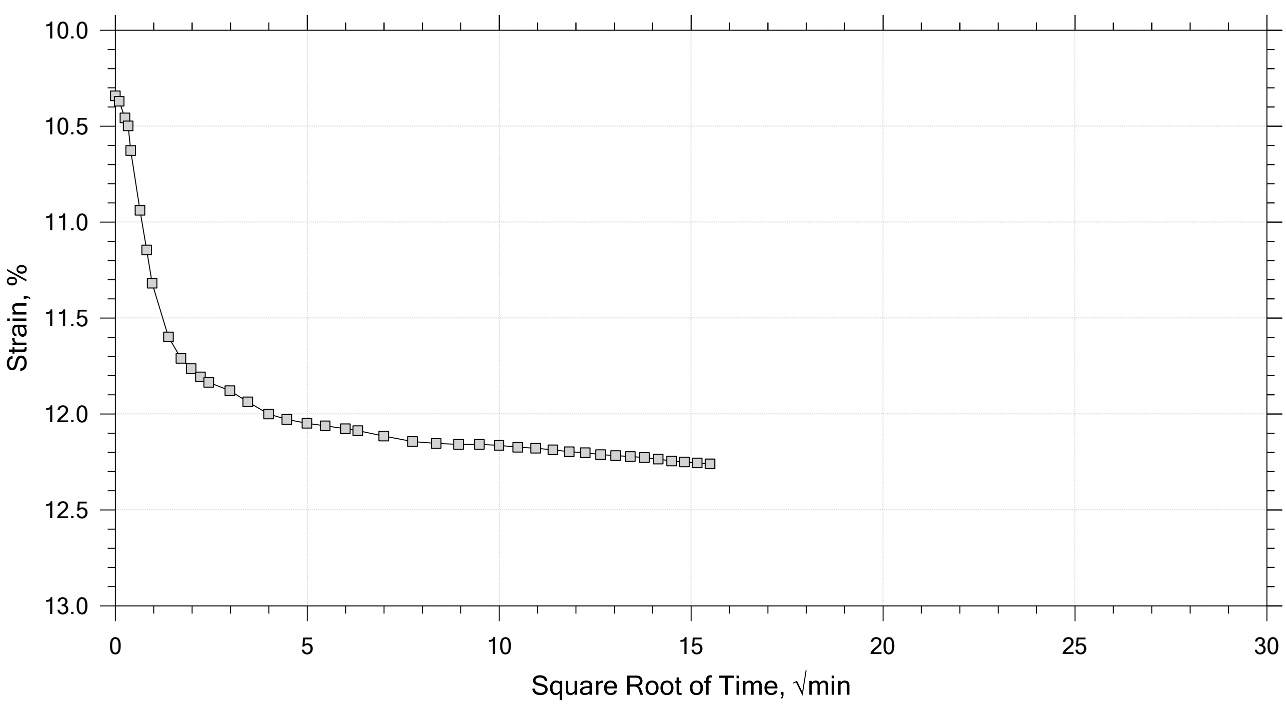
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14

Constant Load Step

Stress: 16 tsf





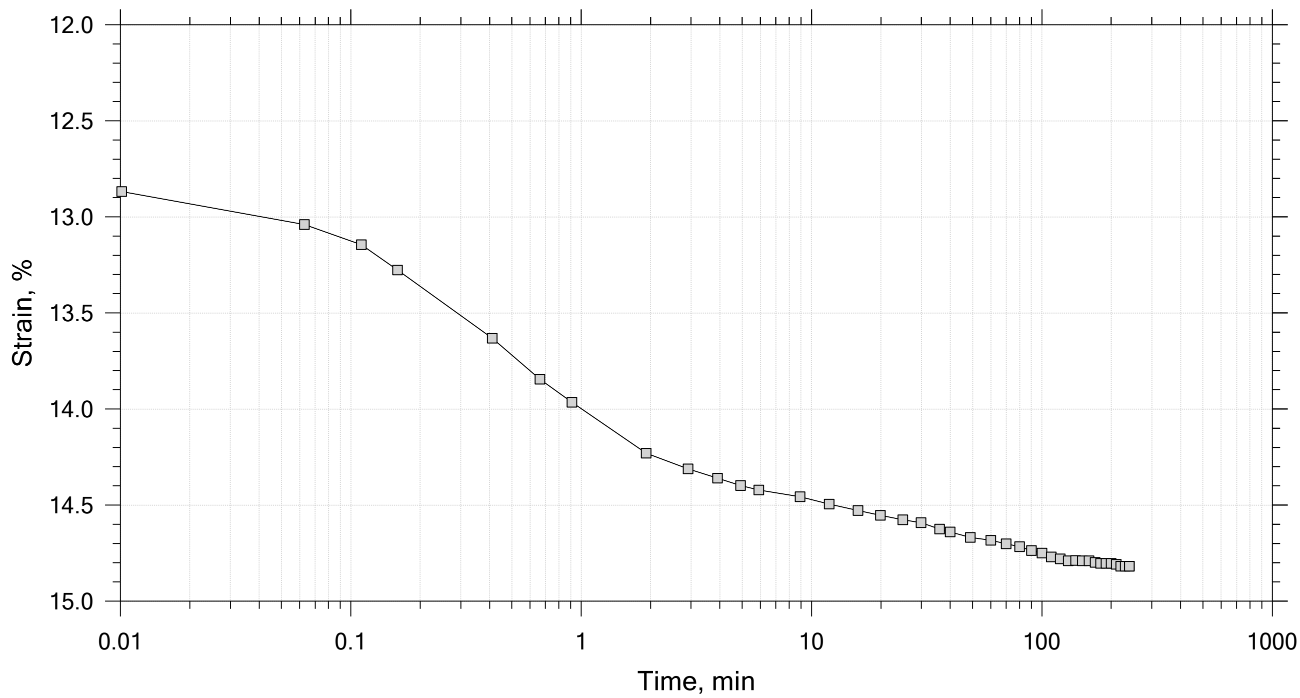
GeoTesting EXPRESS A Sercel Business	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

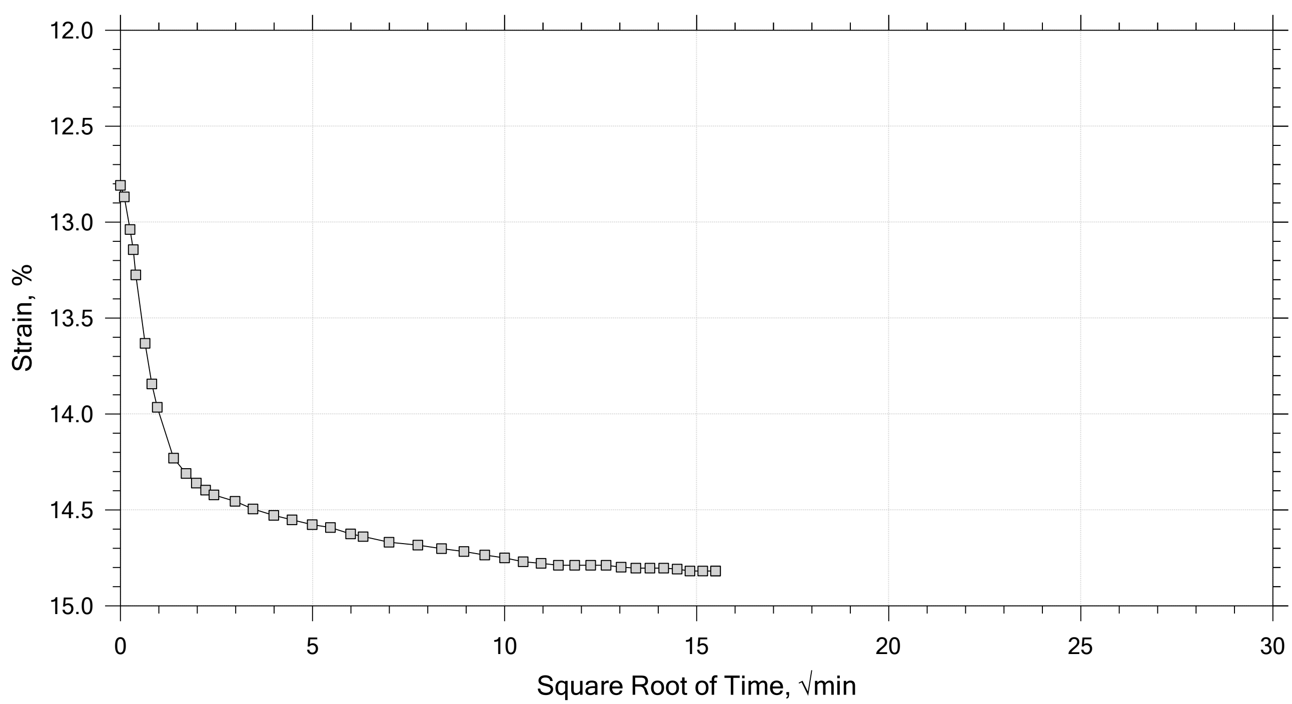
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14

Constant Load Step

Stress: 32 tsf

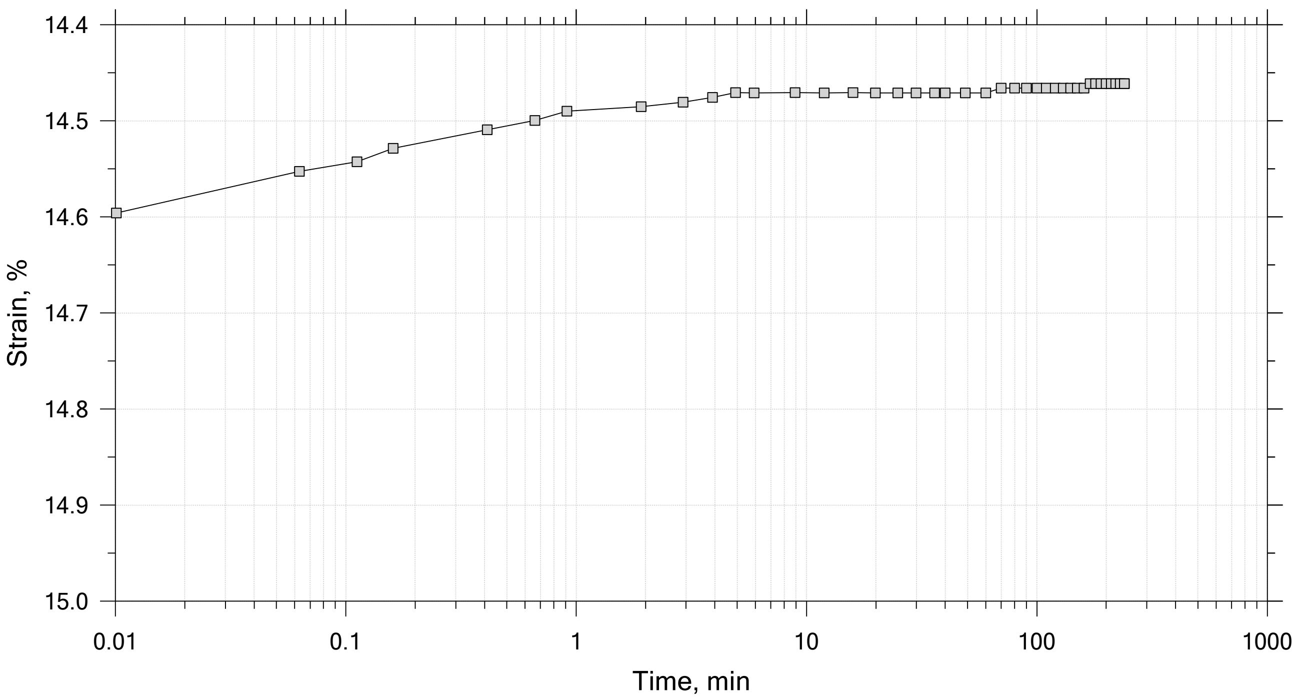


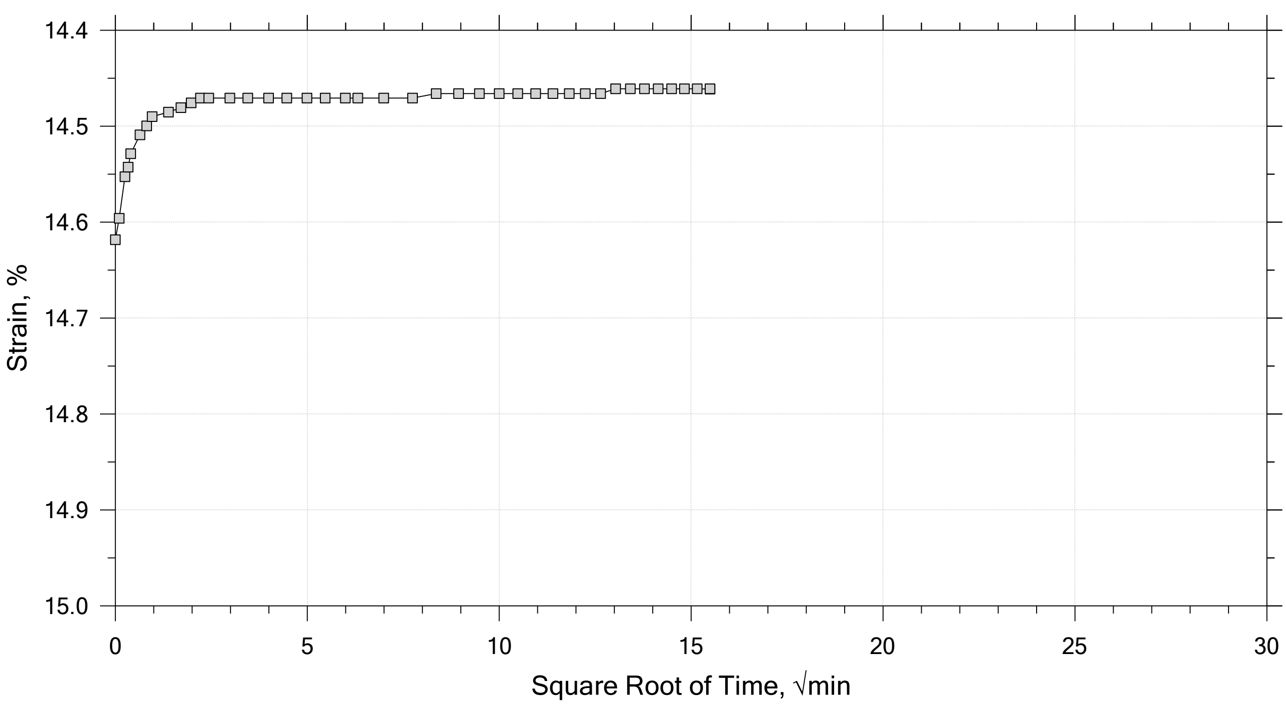


GeoTesting EXPRESS A Sercel Business			
	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 8 tsf





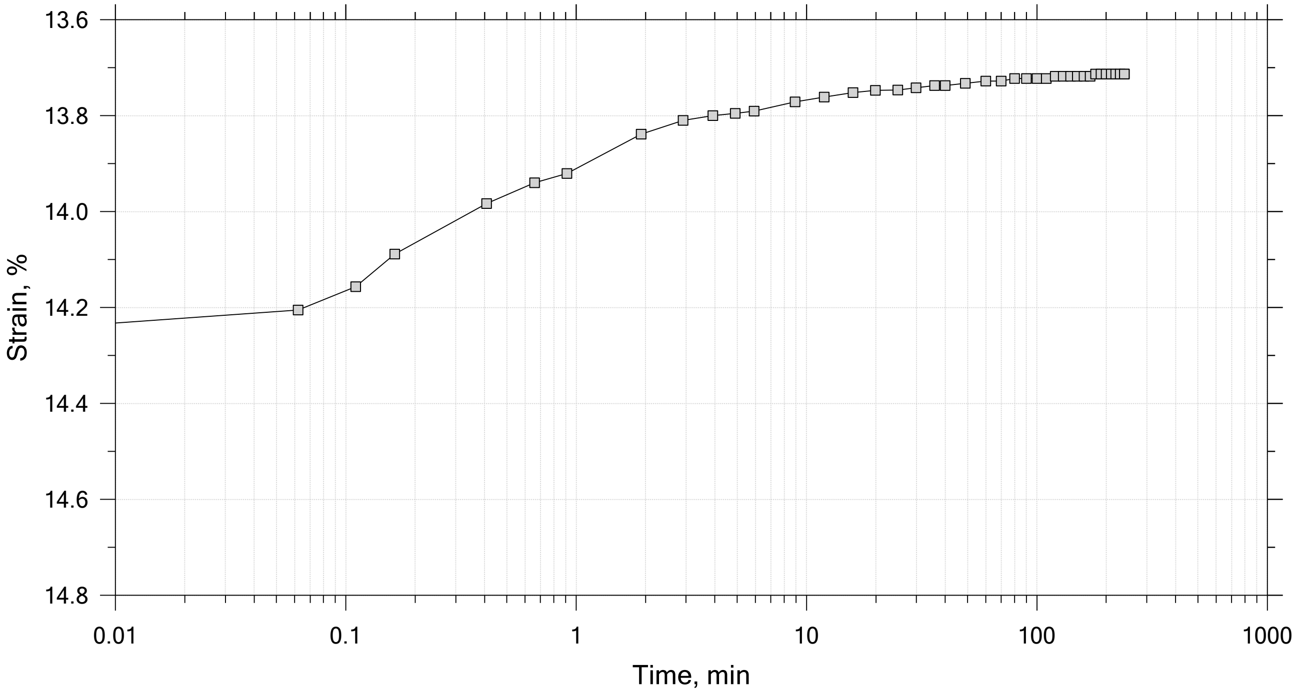
GeoTesting EXPRESS A Sercel Business			
	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

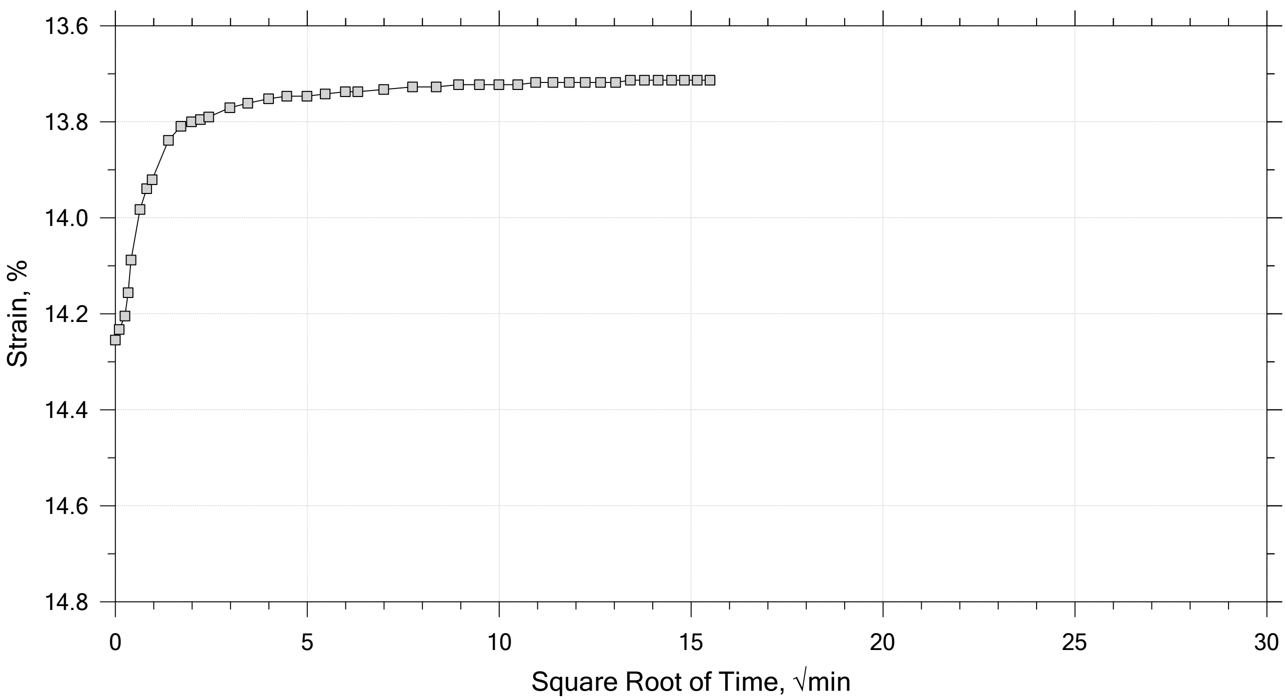
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14

Constant Load Step

Stress: 2 tsf





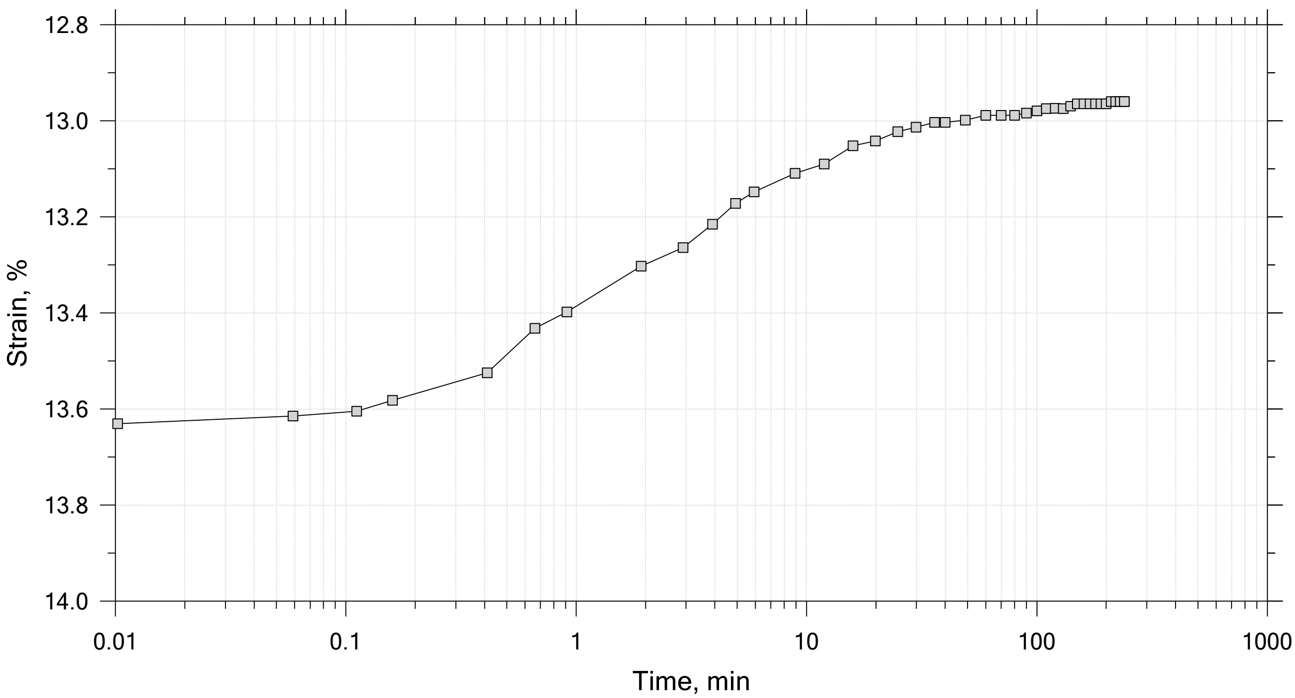
GeoTesting EXPRESS A Sercel Business	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

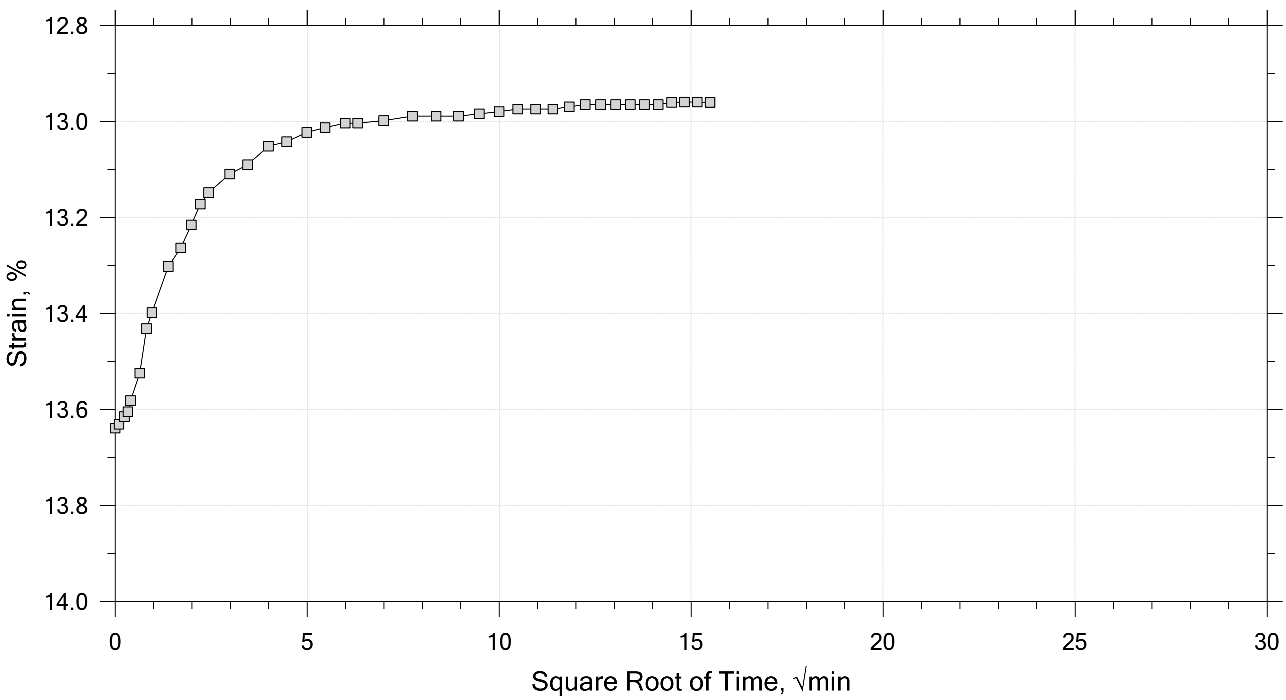
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14

Constant Load Step

Stress: 0.5 tsf

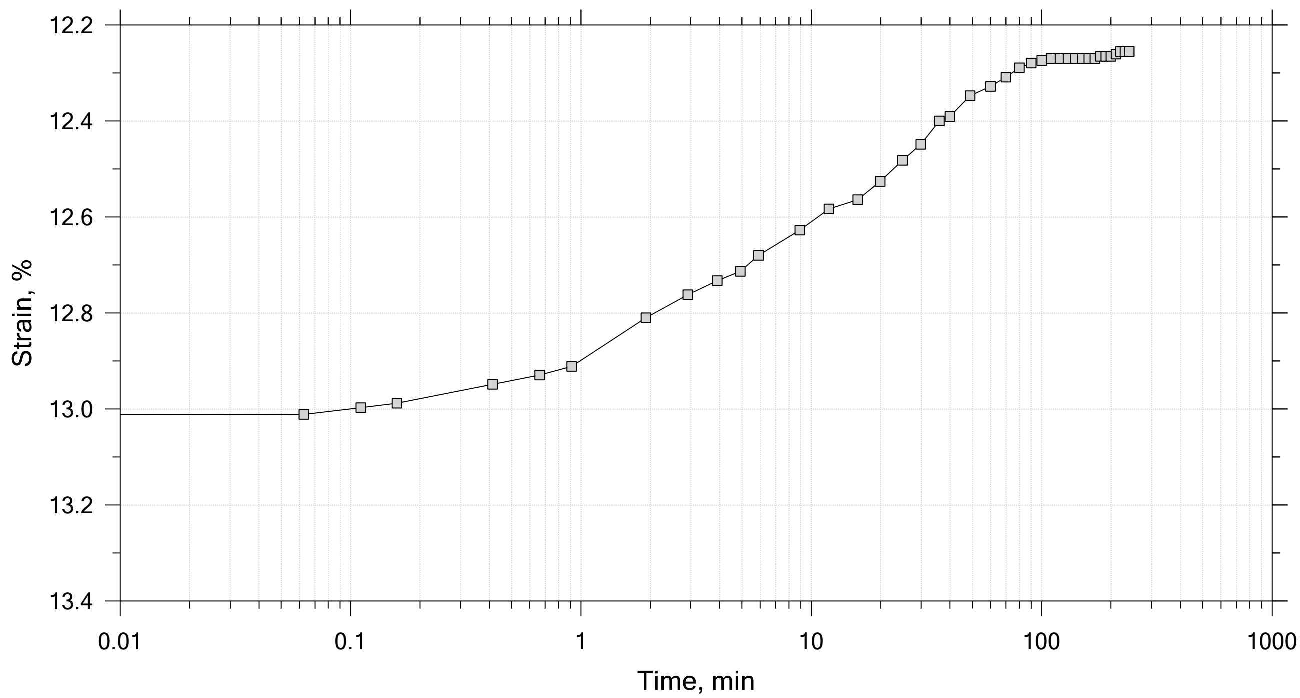


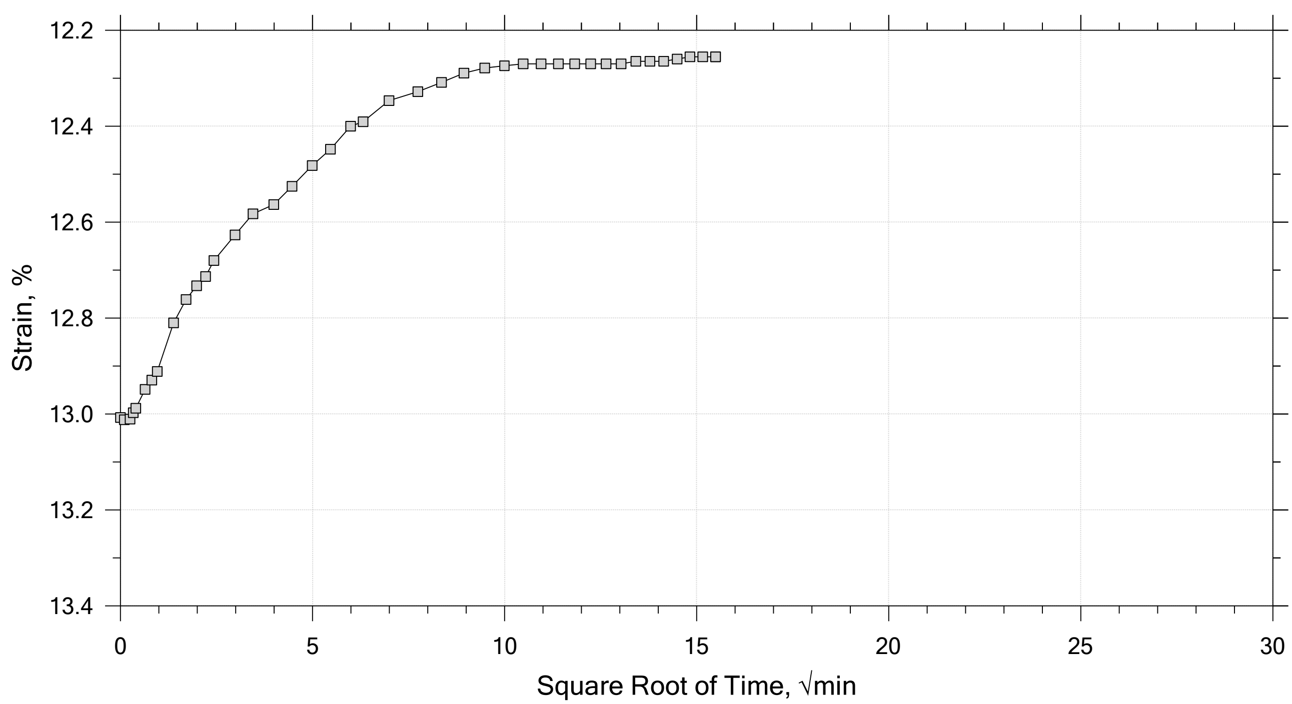


GeoTesting EXPRESS A Sercel Business	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 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: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.70	Liquid Limit: ---
Initial Height: 1.00 in	Initial Void Ratio: 0.671	Plastic Limit: ---
Final Height: 0.88 in	Final Void Ratio: 0.471	Plasticity Index: ---

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E13010	RING		E12835
Mass Container, gm	8.27	111.1	111.1	8.27
Mass Container + Wet Soil, gm	185.4	272.8	263.65	159.71
Mass Container + Dry Soil, gm	146.85	241	241	137.22
Mass Dry Soil, gm	138.58	129.9	129.9	128.95
Water Content, %	27.82	24.49	17.44	17.44
Void Ratio	---	0.67	0.47	---
Degree of Saturation, %	---	98.44	100.00	---
Dry Unit Weight, pcf	---	100.81	114.56	---

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.

Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: TX-019 Swell Pressure = 0.0637 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

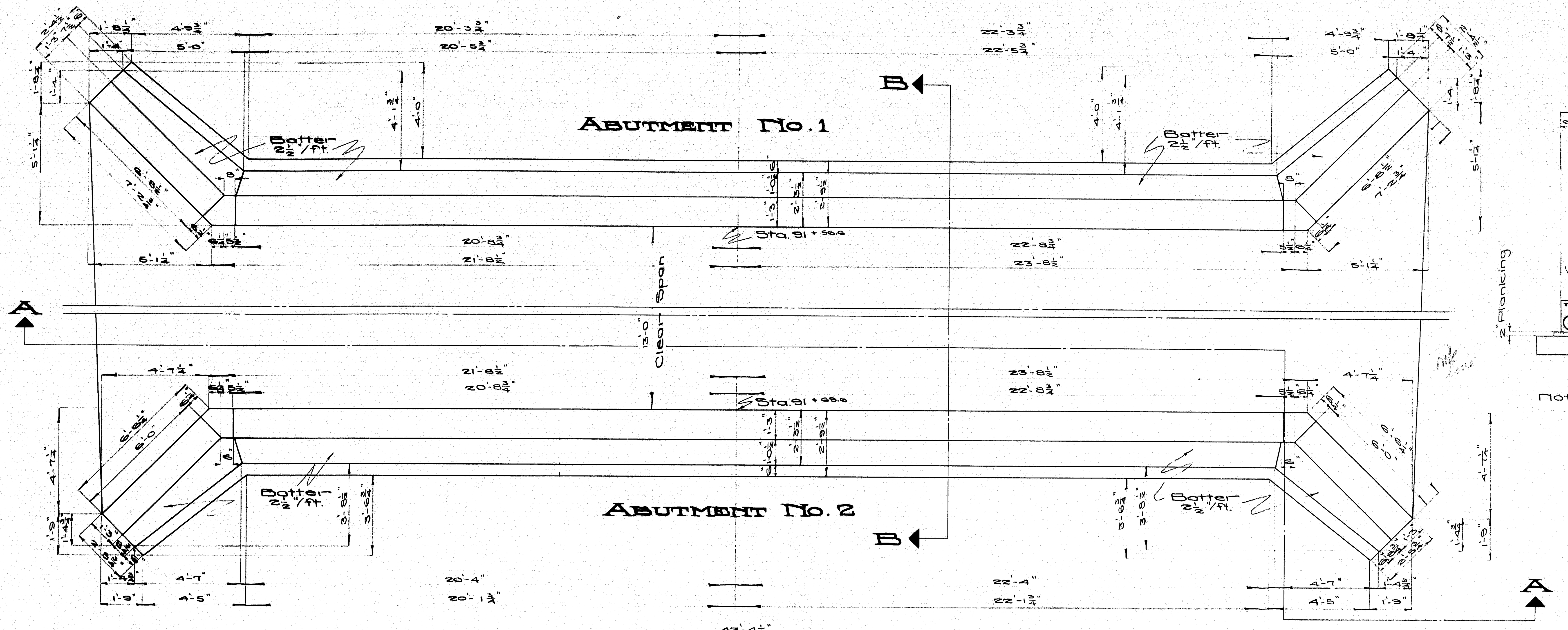
Square Root of Time Coefficients

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	0.0637	0.01077	0.653	1.08	94.695	3.69e-05	1.69e-01	1.17e-04
2	0.125	0.01633	0.644	1.63	182.797	1.88e-05	9.07e-02	3.19e-05
3	0.250	0.02531	0.629	2.53	21.719	1.56e-04	7.18e-02	2.10e-04
4	0.500	0.03588	0.611	3.59	12.276	2.70e-04	4.23e-02	2.14e-04
5	1.00	0.04775	0.591	4.78	14.054	2.31e-04	2.37e-02	1.03e-04
6	2.00	0.06237	0.567	6.24	9.796	3.22e-04	1.46e-02	8.82e-05
7	4.00	0.07955	0.538	7.96	7.224	4.22e-04	8.59e-03	6.79e-05
8	8.00	0.1006	0.503	10.1	4.574	6.40e-04	5.26e-03	6.30e-05
9	16.0	0.1226	0.466	12.3	4.153	6.72e-04	2.75e-03	3.46e-05
10	32.0	0.1482	0.424	14.8	3.033	8.71e-04	1.60e-03	2.61e-05
11	8.00	0.1446	0.430	14.5	1.606	1.60e-03	1.49e-04	4.47e-06
12	2.00	0.1371	0.442	13.7	3.009	8.67e-04	1.25e-03	2.02e-05
13	0.500	0.1296	0.455	13.0	8.126	3.27e-04	5.02e-03	3.07e-05
14	0.125	0.1226	0.466	12.3	44.424	6.07e-05	1.88e-02	2.14e-05

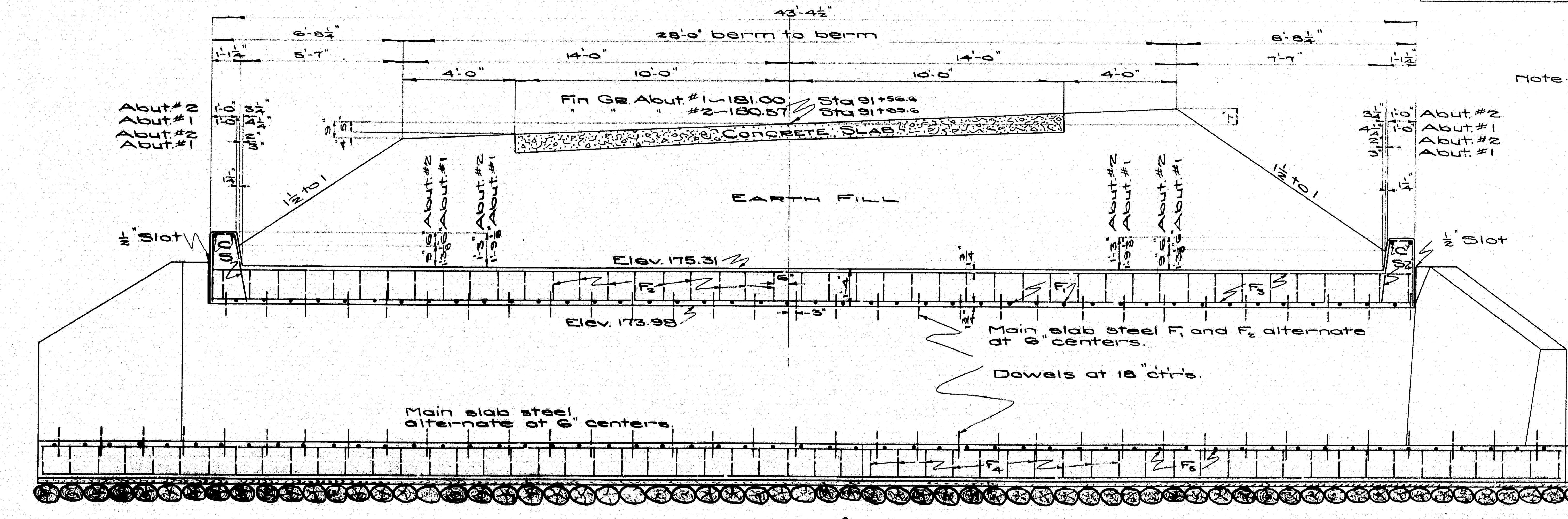
GeoTesting EXPRESS A Sercel Business			
	Project: Farnham Bridge Culvert Rep.	Location: Pittsfield, ME	Project No.: GTX-318150
	Boring No.: BB-PFB-102	Tested By: jlw	Checked By: trm
	Sample No.: U1	Test Date: 7/10/25	Depth: 18.2-20.2'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-019 Swell Pressure = 0.0637 tsf		
	Displacement at End of Increment		

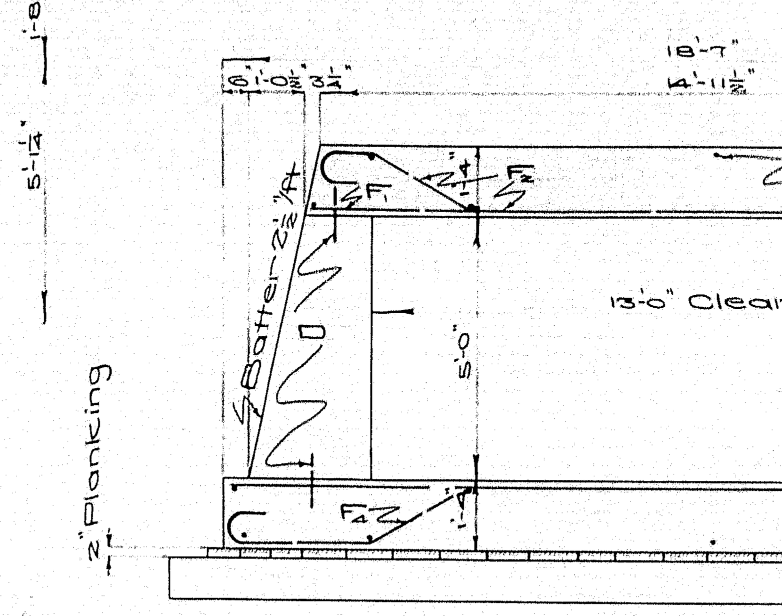
APPENDIX C
Historic Bridge Drawings

Part Plan



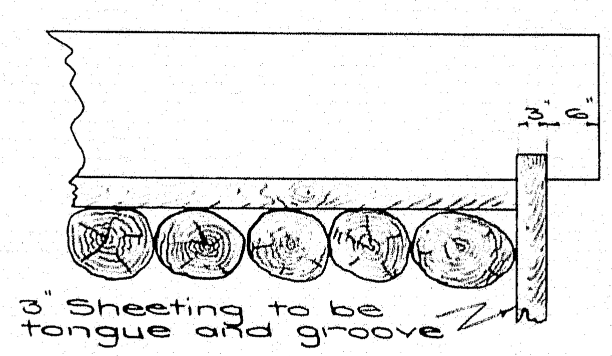
Section and View A-A





Section B-B

Note— Logs to be peeled, to have an 8" minimum diameter at the top and to project 12" beyond the floor slab.



Detail of Sheeting at End of Slab

Note— Cover the ½" slots between substructure and curbs on the back side with two layers of heavy roofing felt. Felt to be 10" wide. Coat surface of concrete and back side of each layer as applied with hot tar or asphalt. The area to be covered by the felt is to be recessed ¼" by nailing thin strips to the forms before concrete is placed.

Note— Highway B.M. #10 Elev. 177.78, nail in 12" apple tree at right of Sta. 90+30. For curve and grade data see Highway Plans. (140 B)

Bridge 2274
Town—13-35

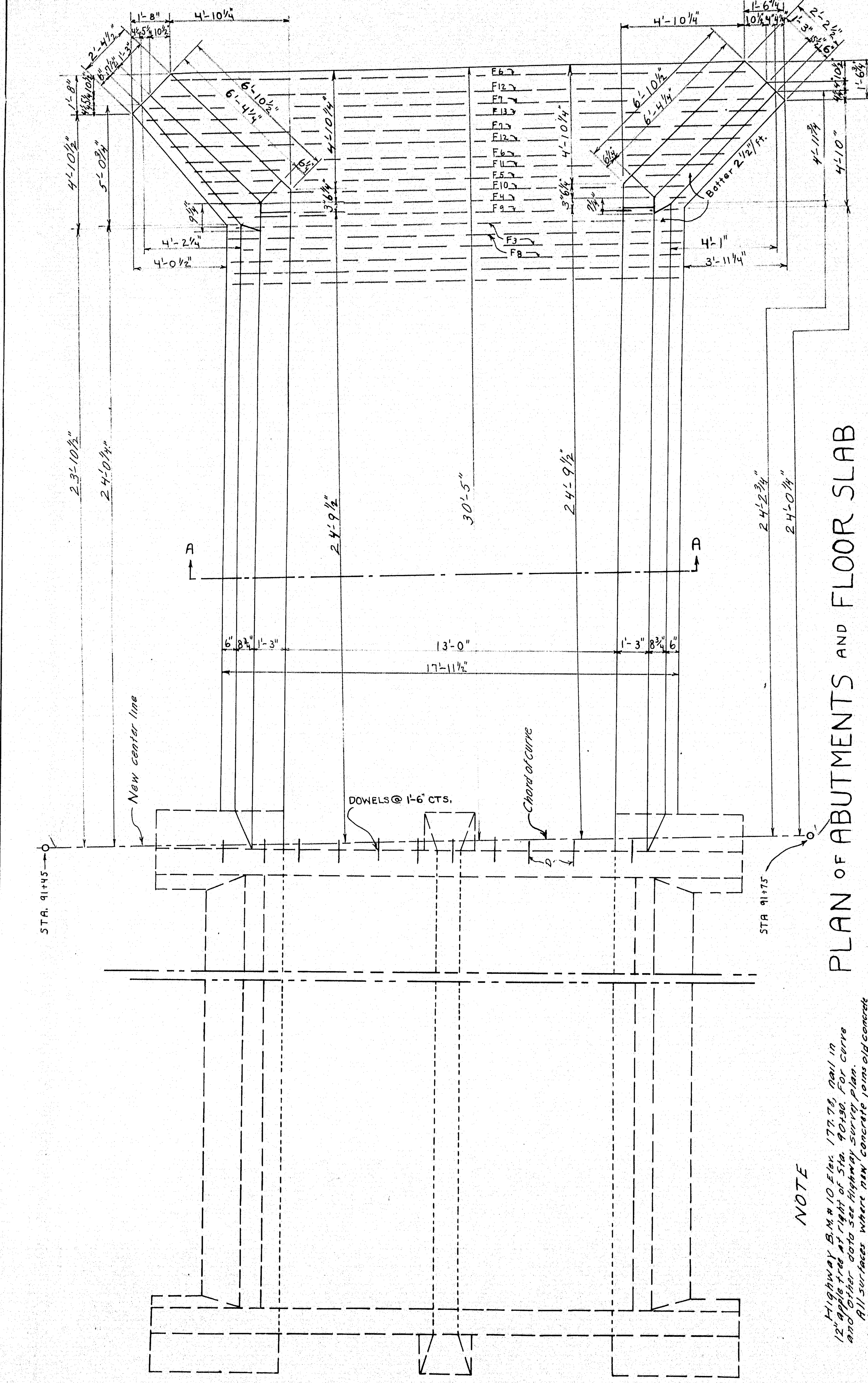
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION

FARNHAM BRIDGE
OVER
FARNHAM BROOK
IN THE TOWN OF
PITTSFIELD
SOMERSET COUNTY

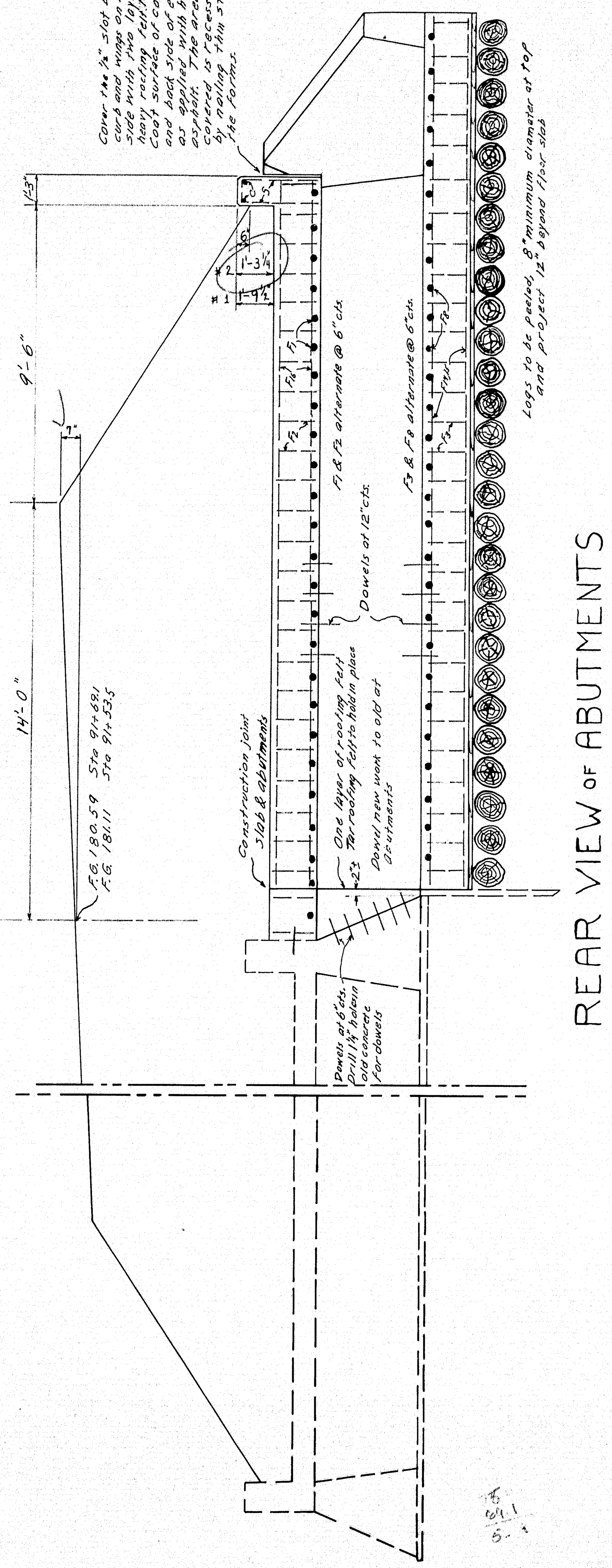
BRIDGE DETAILS

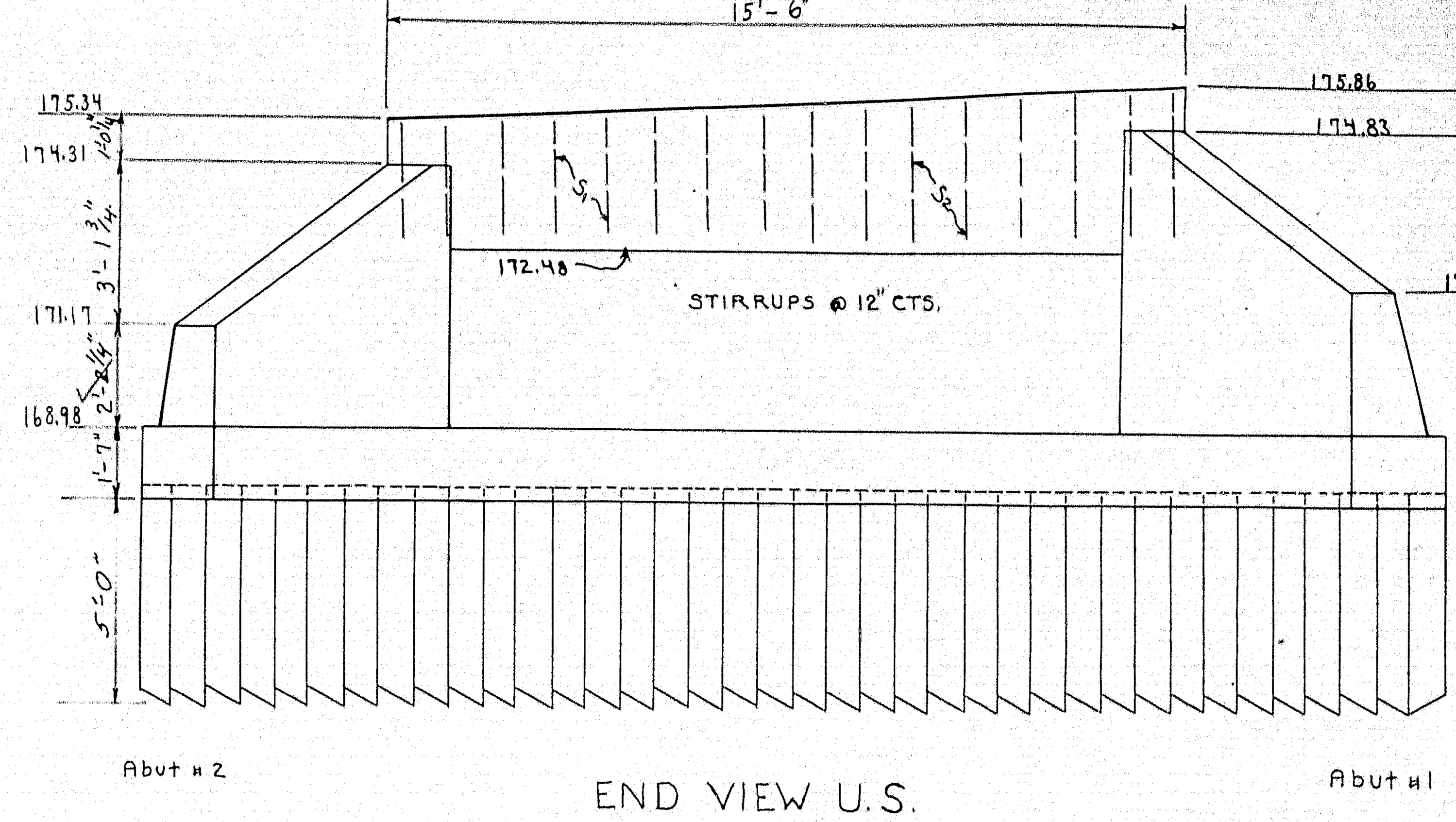
Sheet 1 of 2 Augusta, Me. May 19

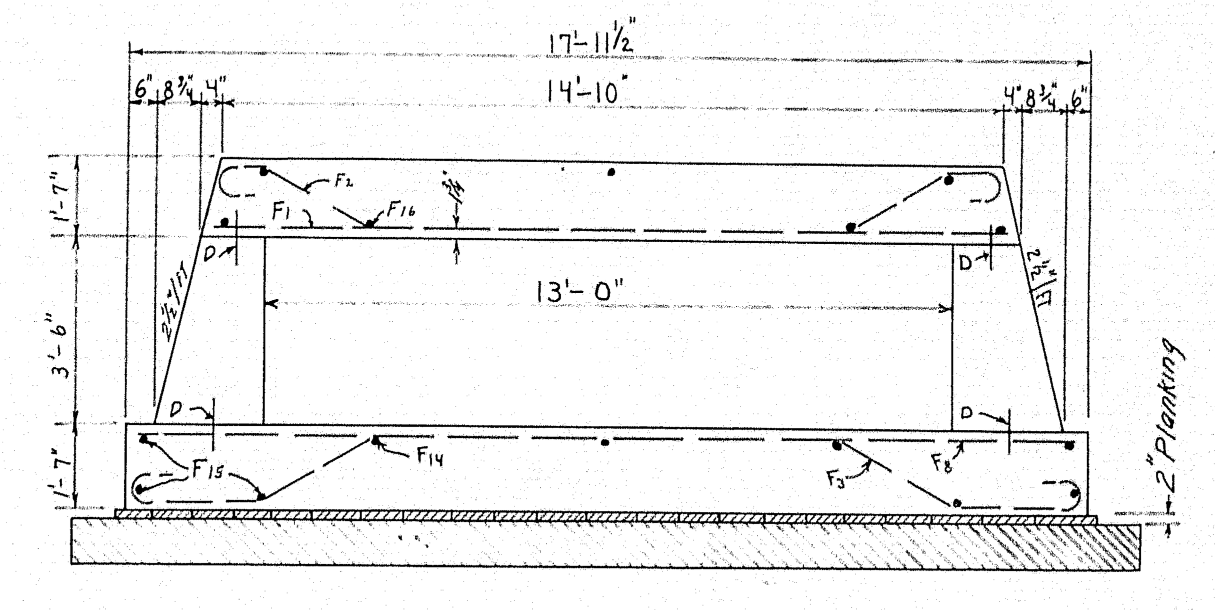
14-1



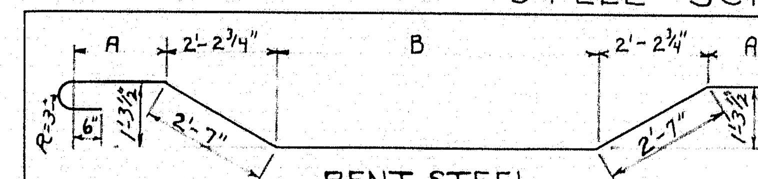
NOTE
Highway B.M. # 10 Elev. 177.74 Nail in 12" apple tree at right of Sta. 90+30. For curve and other data see highway survey plan. All surfaces where new concrete joins old concrete to be roughened. All dowels placed in old concrete to be grouted.







STEEL SCHEDULE



BENT STEEL

MARK	SIZE	NUMBER	LENGTH A	LENGTH B	TOTAL LENGTH	LOCATION
F2	7/8 Φ	26	7¼"	8'-6½"	17'-5¾"	TOP SLAB
F3	"	23	2'-0¾"	8'-6½"	20'-4¾"	FLOOR "
F4	"	1	2'-0¾"	10'-4½"	22'-2¾"	" "
F5	"	1	2'-0¾"	11'-11½"	23'-9¾"	" "
F6	"	2	2'-0¾"	13'-6½"	25'-4¾"	" "
F7	"	2	2'-0¾"	14'-9½"	26'-7¾"	" "
S1	½" Φ	8			6'-3"	CURB
S2	"	8			6'-10"	"

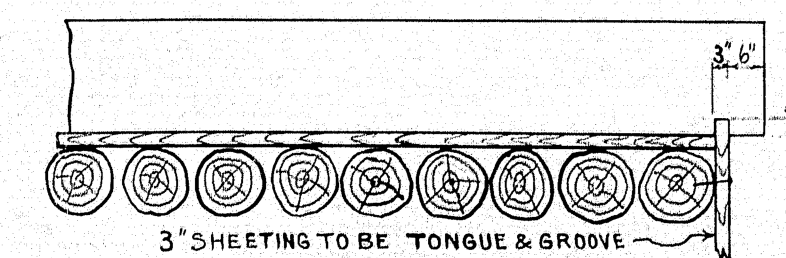
S1: 1'-0" × 2'-7½"

S2: 1'-0" × 2'-11"

STRAIGHT STEEL

MARK	SIZE	NUMBER	LENGTH	LOCATION
F1	7/8" Φ	25	15'-2"	TOP SLAB
F8	"	23	17'-7½"	FLOOR "
F9	"	1	18'-8"	" "
F10	"	1	20'-4"	" "
F11	"	1	21'-11"	" "
F12	"	2	23'-6"	" "
F13	"	1	25'-1½"	" "
F14	½" Φ	3	29'-3"	" "
F15	"	6	31'-0"	" "
F16	"	7	23'-4"	TOP "
C	"	2	15'-3"	CURB
D	¾" Φ	140	1'-0"	SLABS & ABUTS.

Dimensions are to center of bars
Steel to be plain round structural grade



DETAIL OF SHEETING AT END OF SLAB

TOWN 13-35
BRIDGE 2274

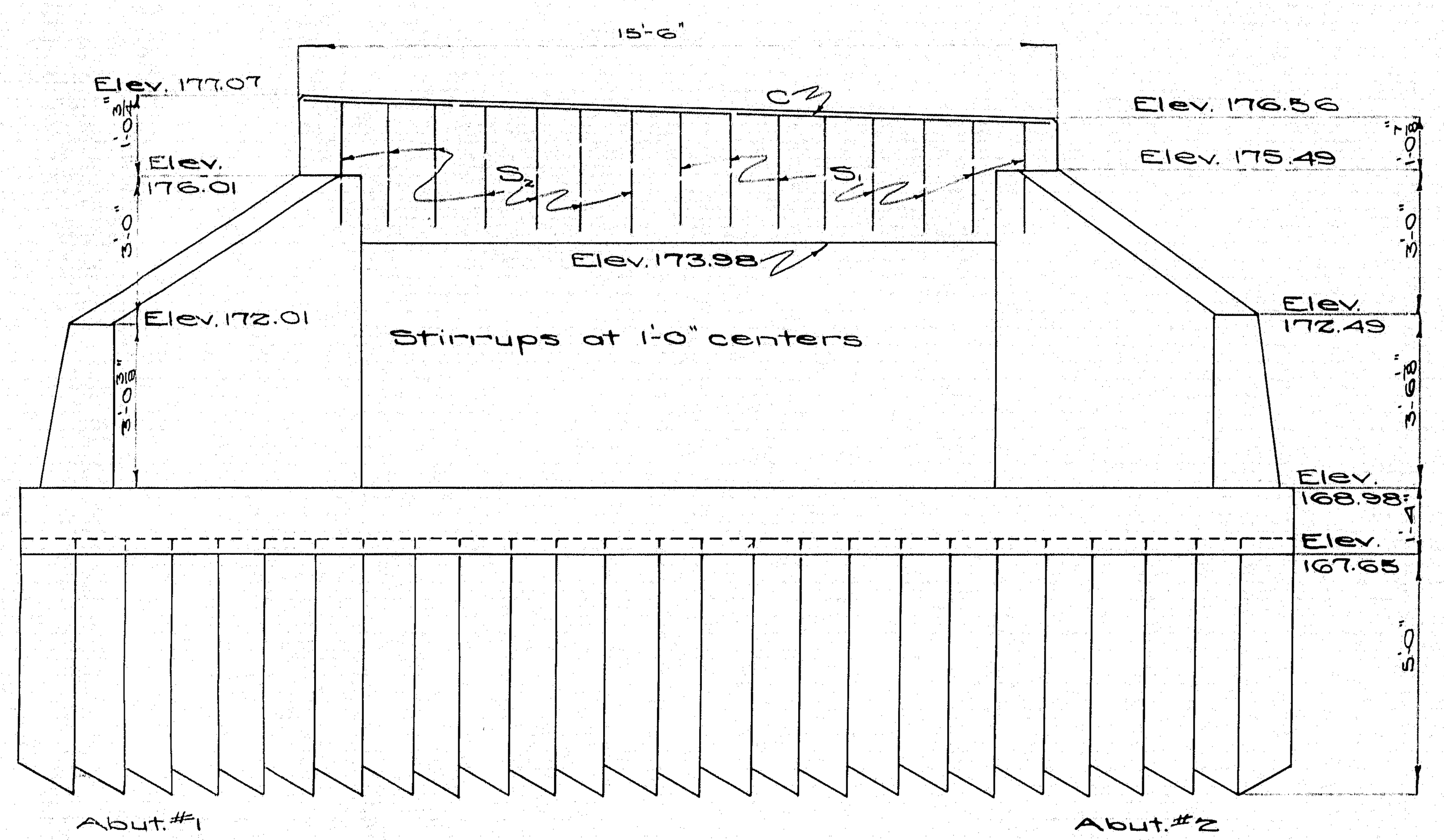
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION

FARNHAM BRIDGE
OVER
FARNHAM BROOK
IN THE TOWN OF
PITTSFIELD
SOMERSET COUNTY
BRIDGE DETAILS

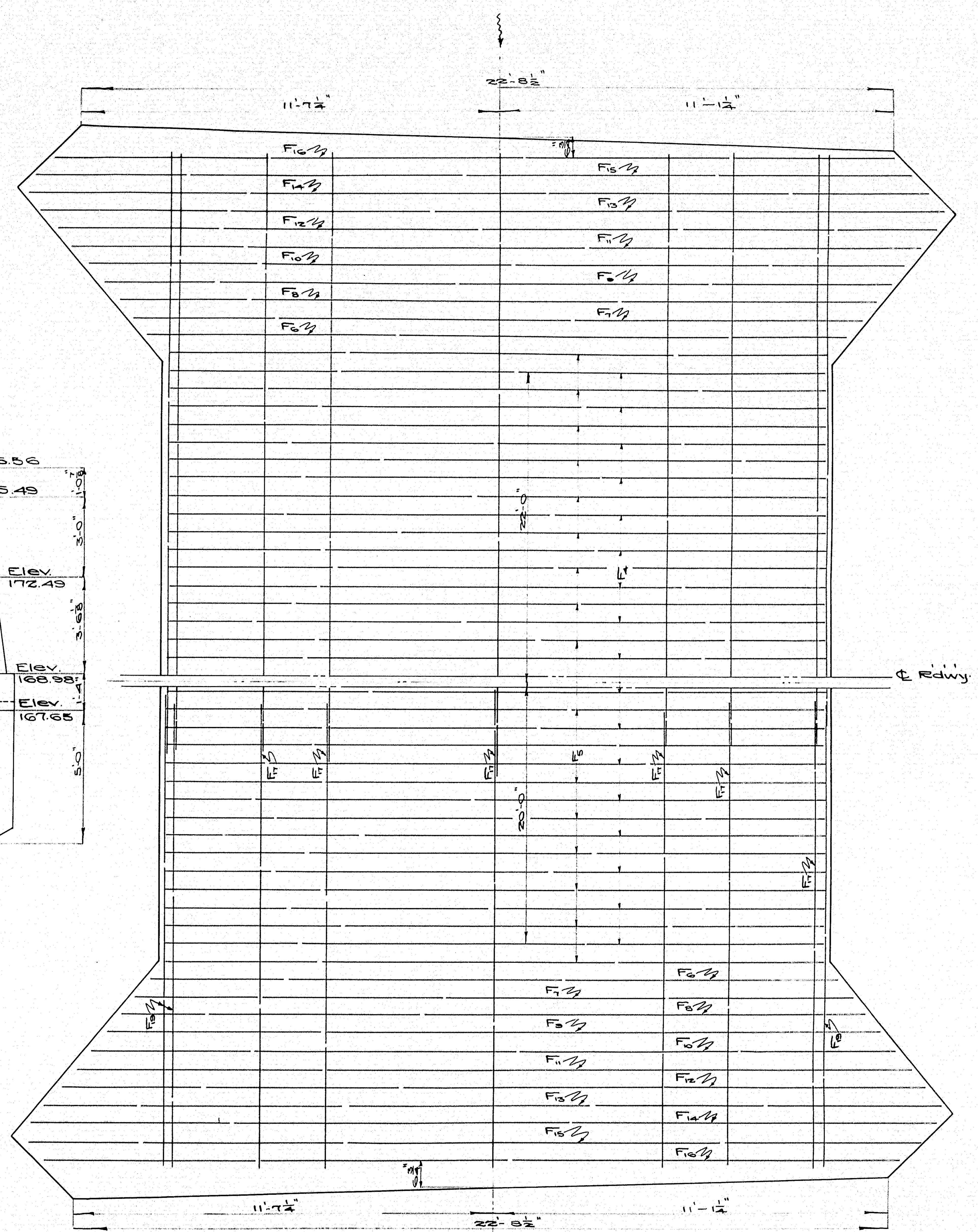
SHEET 1 OF 1 AUGUSTA, ME. FEB. 26, 1932

14-199A

Checked

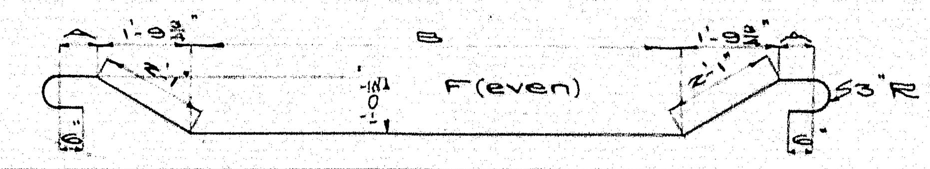


END VIEW D.S.

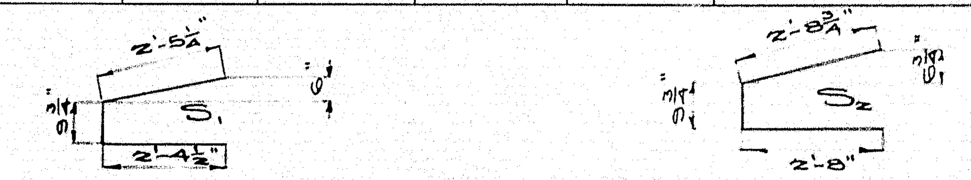


STEEL PLAN FLOOR SLAB

STEEL SCHEDULE



MARK	SIZE	NO. R'Q'D	LEN. A	LEN. B	TOT. LENGTH	LOCATION
F_2	⅞"Φ	44	0'-7"	9'-4½"	17'-3¼"	Top Slab
F_4	⅞"Φ	43	2'-4¾"	9'-4½"	20'-10¾"	Floor "
F_6	⅞"Φ	2	2'-4¾"	9'-10½"	21'-4¾"	" "
F_8	⅞"Φ	2	2'-4¾"	11'-9½"	23'-3¾"	" "
F_{10}	⅞"Φ	2	2'-4¾"	13'-5"	24'-11¼"	" "
F_{12}	⅞"Φ	2	2'-4¾"	15'-1½"	26'-7¾"	" "
F_{14}	⅞"Φ	2	2'-4¾"	16'-0"	27'-6¼"	" "
F_{16}	⅞"Φ	2	2'-4¾"	14'-6"	26'-0¼"	" "



S_1 ½"Φ 16 R'q'd. 5'-7½" long — Curbs.

S_2 ½"Φ 14 R'q'd. 6'-2½" long — Curbs.

STRAIGHT BARS

MARK	SIZE	NO. R'Q'D.	LENGTH	LOCATION
* F_1	⅞"Φ	——	15'-2"	Straighten 18 F_5(old) and cut 15'-2" long. {Use 25 origonal F_1 15'-2" long.
F_3	½"Φ	3	16'-4¾"	Top Sl. (lap with 29'-3" bars)
F_{3A}	½"Φ	4	22'-3¾"	" " (" " 23'-4" old bars)
F_5	⅞"Φ	21	18'-5"	Fl. Sl. (Use 23 old 17'-7½" bars F_8)
* F_7	⅞"Φ	——	20'-2"	Use origonal F_{10} - 20'-4" and straighten and cut origonal F_8 to 20'-2"
* F_9	⅞"Φ	——	22'-0"	Use origonal F_{11} - 21'-11" and straighten and cut 1 origonal F_8 to 22'-0"
* F_{11}	⅞"Φ	——	23'-6"	Use 2 orig. F_{12} - 23'-6" long.
F_{13}	⅞"Φ	1	25'-1"	Use orig F_{13} - 25'-1½"
* F_{15}	⅞"Φ	——	24'-6"	Use 2 origonal F_1 straighten and cut off one hook.
F_{17}	½"Φ	6	25'-6¾"	Fl. Slb. (lap with 6 31'-0" bars, old F_{18})
F_{19}	½"Φ	6	29'-4"	" "
* C	½"Φ	——	15'-4"	Use 2 origonal F_{16} - 23'-4" cut to 15'-4" Curbs (Use 2 old C-bars
* D	¾"Φ	Use 140	of old bars	1'-0" long. "D"

Note —
All steel to be plain bars of structural grade.
All dimensions are to centers of bars.
* — On hand.

Town — 13-35
Bridge 2274

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION

FARNHAM BRIDGE
OVER
FARNHAM BROOK
IN THE TOWN OF
PITTSFIELD
SOMERSET COUNTY
BRIDGE DETAILS

Sheet 2 of 2 Augusta, Me. 1932

14-2c

0 1 2 3 4 5 INCHES

APPENDIX D
Calculations

SEISMIC SITE CLASS

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using available subsurface SPT N information.

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered in the two borings (BB-PFB-101 and BB-PFB-102), recommend a Site Class D for design.

REFERENCES

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

AVAILABLE INFORMATION

1. Boring logs dated 8-18-2023 to 8-25-2023 by New England Boring Contractors (monitored by Haley & Aldrich, Inc.).

ASSUMPTIONS

1. Where SPT N, Vs or su data was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N, Vs or su for the extended profile was then assumed based on the available information.
2. Boring BB-PFB-102 Sample 5D was tested by vane shear prior to taking the spoon, indicating the clay is medium stiff to stiff. Assume a SPT N value of 5 blows/foot, corresponding to the low end of "medium stiff" fine grained soil per the MaineDOT field card.

PROCEDURE

1. Check the site against the three categories of Site Class F, requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. Categorize the site using one of the following three methods (Method A, B, or C).

Method A
Average shear wave velocity for the upper 100 ft of the soil profile:

$$\overline{V}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of *i* th soil (ft/s).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

PROCEDURE

Method B
Average standard penetration test (SPT) for the upper 100 ft of the soil profile:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th soil layer not to exceed 100 ft (blows/ft).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

Method C
Average standard penetration test (SPT) for the cohesionless layers in the upper 100 ft of the soil profile:

$$\bar{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th cohesionless soil layer (blows/ft).
d_i = thickness of *i* th cohesionless soil layer (ft).
m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and m.

Average undrained shear strength for the cohesive layers in the upper 100 ft of the soil profile:

$$\bar{s}_u = \frac{\sum_{i=1}^{k} d_i}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

where

s_{ui} = undrained shear strength of *i* th cohesive soil layer (psf), not to exceed 5000 psf
d_i = thickness of *i* th cohesive soil layer (ft).
k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and k.

Based on the available information, Method B will be used for the seismic Site Class evaluation.

SITE CLASS DEFINITIONS

(Table from AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020)

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/s, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s$ < 1,200 ft/s, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10.0 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays ($H > 10.0$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25.0$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H >120$ ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile

$\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile

PI = plasticity index (ASTM D4318)

w = moisture content (ASTM D2216)

CALCULATIONS - METHOD B

Exploration ID: BB-PFB-101
Ground Surface El.: 182.0

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	6.0	176.0	Fill	7.0	39	0.179
2D	8.0	174.0	Fill	2.0	59	0.034
3D	10.0	172.0	Fill	2.0	26	0.077
4D	12.0	170.0	Fill	4.0	9	0.444
5D	16.0	166.0	Marine Deposit (Silt)	5.0	7	0.714
6D	24.0	158.0	Marine Deposit (Silt/Silty Clay)	8.0	11	0.727
7D	31.0	151.0	Marine Deposit (Sand)	7.0	35	0.200
8D	36.0	146.0	Marine Deposit (Clay)	3.5	6	0.583
9D	41.0	141.0	Marine Deposit (Clay)	3.5	30	0.117
10D	43.0	139.0	Marine Deposit (Clay/Sand)	58.0	37	1.568
			Totals =	100.0		4.644

N-bar (blows/ft) = 21.5
Site Class = D

Exploration ID: BB-PFB-102
Ground Surface El.: 179.5

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	2.6	176.9	Fill	3.8	21	0.181
2D	4.2	175.3	Fill	0.9	50	0.018
3D	6.0	173.5	Fill	2.3	36	0.064
4D	8.0	171.5	Fill	2.0	24	0.083
5D	10.0	169.5	Fill	2.7	28	0.096
6D	12.0	167.5	Organic Silt/Marine Deposit (Clay)	3.3	10	0.330
7D	16.0	163.5	Marine Deposit (Clay)	5.2	2	2.600
8D	21.2	158.3	Marine Deposit (Sand)	3.4	24	0.142
9D	26.0	153.5	Marine Deposit (Sand)	4.9	35	0.140
10D	31.0	148.5	Marine Deposit (Sand)	5.0	27	0.185
11D	36.0	143.5	Marine Deposit (Sand)	3.5	6	0.583
12D	38.0	141.5	Marine Deposit (Sand)	63.0	35	1.800
			Totals =	100.0		6.223

N-bar (blows/ft) = 16.1
Site Class = D

RESULTS SUMMARY & RECOMMENDATIONS

Boring Number	Parameter	Average Value	Site Class
BB-PFB-101	SPT N	21.5	D
BB-PFB-102	SPT N	16.1	D

Recommend a Site Class D for design.

BEARING RESISTANCE AND MODULUS OF SUBGRADE REACTION

PROBLEM STATEMENT & OBJECTIVE

Calculate the Strength and Service Limit bearing resistances of the proposed culvert and wingwalls 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.
4. Meyerhof & Hanna, "Ultimate bearing capacity of foundations on layered soils under inclined load," 1978.

AVAILABLE INFORMATION

1. Boring logs dated 8-18-2023 to 8-25-2023 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 for the culvert. Assume 1/3 B for wingwall eccentricity.
2. Groundwater is located at a depth of 5.6 ft below the finished roadway based on the highest encountered water levels in the boring logs.
3. Bottom of culvert is 14 ft below ground surface based on scaling from the PIC plans.
4. Culvert size considered: 13 ft width x 75.6 ft long. Wingwall size considered: 12.2 ft length (L/H=1) x 19.2 ft long.
5. Footings will bear on bedding stone. Native soil beneath the bedding stone will be cohesive Marine Deposits.
6. Soil properties for the marine deposits beneath the culvert will be 115 pcf (unit weight) and 725 psf (undrained strength). Stronger material beneath the soft clay will be ignored for the culvert.
6. Soil properties for the marine deposits beneath the culvert will be 115 pcf (unit weight) and 725 psf (undrained strength) for the upper soil layer and 1400 psf for the lower soil layer.
7. The stronger clay begins at El. 157.8 per boring BB-PFB-101. The bottom of the wingwall will be El. 164.4 (3.25 ft below the culvert invert).
8. Wingwalls have an eccentricity of 0.7 ft based on results from MSEW.

CONCLUSIONS AND RECOMMENDATIONS

Culvert Recommendations

Strength Limit State

The factored bearing resistance for the Strength Limit State is 2.1 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 130 pci

Wingwall Recommendations

Strength Limit State

The factored bearing resistance for the Strength Limit State is 3.4 ksf

Service Limit State

The factored bearing resistance for the Service Limit State is 3.0 ksf for 1 in. settlement.

Extreme Event Limit State

The factored bearing resistance for the Extreme Event Limit State is 4.6 ksf

HALEY ALDRICH	CALCULATIONS	File No.:	0208476-000
		Sheet:	2 of 7
Client	Fuss & O'Neill, Inc.	Date:	5-Feb-2026
Project	Farnham Culvert Replacement	Computed by:	EMH
Subject	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	MMB/TPJ

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.

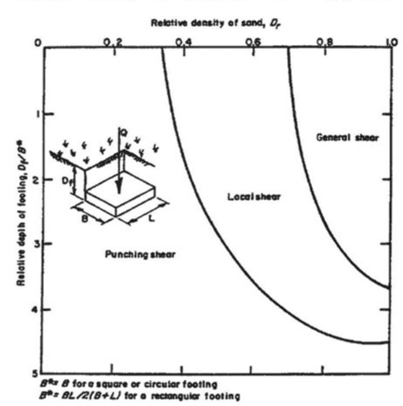


Figure C10.6.3.1.2b-1—Modes of Bearing Capacity Failure for Footings in Sand

LAYERED BEARING CAPACITY BACKGROUND INFORMATION FROM MEYERHOF AND HANNA (1978)

$$q_u = q_t + (q_b - q_t)\left(1 - \frac{H}{D}\right)^2 \geq q_t$$

where

q_u = ultimate bearing capacity
q_t = nominal bearing capacity in a thick bed of the upper soil layer
q_b = nominal bearing capacity in a thick bed of the lower soil layer
H = vertical distance from bottom of footing to the top of the lower soil layer
D ≈ footing width, B, for loose sand and clay per Meyerhof and Hanna (1978)

HALEY ALDRICH	CALCULATIONS	File No.:	0208476-000
		Sheet:	4 of 7
Client	Fuss & O'Neill, Inc.	Date:	5-Feb-2026
Project	Farnham Culvert Replacement	Computed by:	EMH
Subject	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	MMB

INPUT PARAMETERS FOR BEARING CAPACITY

	Culvert	Wingwall qt	Wingwall qb
ϕ (deg.) =	0	0	0
γ_q (pcf) =	125	125	125
γ_f (pcf) =	115	115	115
c (psf) =	725	725	1400
D_f (ft) =	14.0	3.25	3.25
D_w (ft) =	5.6	0	0
B (ft) =	13.0	12.2	12.2
e_B (ft) =	0.00	0.70	0.70
L (ft) =	75.6	19.2	19.2
e_L (ft) =	0.0	0.0	0.0
Depth Corr., (Y/N)?	N	N	N

CALCULATIONS FOR BEARING CAPACITY

	Culvert	Wingwall qt	Wingwall qb
B_{eff} = B' (ft) =	13.0	10.8	10.8
L_{eff} = L' (ft) =	75.6	19.2	19.2
$N_f = f(\phi)$ =	1.0	1.0	1.0
$N_c = f_1(\phi)$ =	5.1	5.1	5.1
$N_q = f_2(\phi)$ =	1.0	1.0	1.0
$Ng = f_3(\phi)$ =	0.0	0.0	0.0
s_c =	1.03	1.11	1.11
s_q =	1.00	1.00	1.00
s_g =	1.00	1.00	1.00
d_q =	1.00	1.00	1.00
C_{wq} =	0.50	0.50	0.50
C_{wg} =	0.50	0.50	0.50
N_{cm} =	5.32	5.72	5.72
N_{qm} =	1.00	1.00	1.00
N_{gm} =	0.00	0.00	0.00
q_n or q_{ult} (psf) =	4,730	4,349	8,209
q_n or q_{ult} (ksf) =	4.7	4.3	8.2
H (ft) =	--	6.6	
D (ft) =	--	12.2	
q_u (ksf) =	--	5.2	
RF =	0.45	0.65	
RF x q_{ult} (ksf) =	2.1	3.4	

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

BEARING RESISTANCE AT THE SERVICE LIMIT STATE

AASHTO Section 10.6.2.6 - Bearing Resistance at the Service Limit State

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

Table C10.6.2.5.1-1

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	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	for culvert
	3	ksf	for wingwalls

BEARING RESISTANCE AT THE EXTREME EVENT LIMIT STATE (WINGWALLS ONLY)

From the Strength Limit State calculations, the nominal bearing resistance is the following:

q_{ult} = 5.2 ksf

Using a resistance factor of 0.9 from Section 11.5.8, the factored bearing resistance is the following:

q_R = 4.6 ksf

BACKGROUND INFORMATION FROM BOWLES 1977

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

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

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

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

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

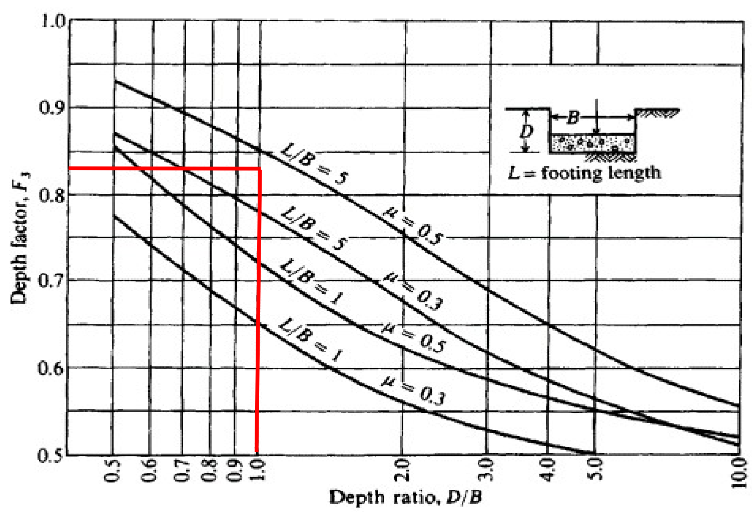
TABLE 2-7
Values or value ranges for Poisson's ratio μ

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 =	1.08
L/B =	5.8
v =	0.4

HALEY ALDRICH	CALCULATIONS	File No.:	0208476-000
		Sheet:	7 of 7
Client:	Fuss & O'Neill, Inc.	Date:	5-Feb-2026
Project:	Farnham Culvert Replacement	Computed by:	EMH
Subject:	Bearing Resistance And Subgrade Modulus Calculation	Checked by:	MMB/TPJ

INPUT PARAMETERS FOR SUBGRADE MODULUS

Width of Footing, B (ft) =	13	
Length of Footing, L (ft) =	75.6	
Depth of Footing, D (ft) =	14.0	
Elastic Modulus, E_s (MPa) =	15	from Bowles Table 2-8
Elastic Modulus, E_s (ksf) =	313	
Poisson's Ratio, v_S =	0.4	from Bowles Table 2-7

CALCULATIONS FOR SUBGRADE MODULUS

E's (ft^2/k) =	0.00268135	from Bowles Equation 2-65
Thickness of Stratum, H (ft) =	13	approx. thickness bottom of culvert to bottom of soft clay
N = H/B =	1.0	For corner of footing
M = L/B =	5.8	For corner of footing
I_1 =	0.113	from Bowles Equation 5-16a
I_2=	0.123	from Bowles Equation 5-16b
Steinbrenner Influence Factor, I_s =	0.15	from Bowles Equation 5-16c
D/B =	1.08	
Depth Influence Factor, I_F =	0.83	from Bowles Table 5-7

Modulus of Subgrade Reaction, k_s (pci) =	130	from Bowles Equation 9-7

Recommend k_s = 130 pci

SETTLEMENT

HALEY ALDRICH	CALCULATIONS	File No.:	0208476-000
		Sheet:	1 of 2
Client:	Fuss & O'Neill, Inc.	Date:	22-Dec-2025
Project:	Farnahm Culvert Replacement	Computed by:	EMH
Subject:	Settlement Evaluation	Checked by:	MMB

PROBLEM STATEMENT & OBJECTIVE

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

EXECUTIVE SUMMARY

Approximately 0.25 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 8-18-2023 to 8-25-2023 by New England Boring Contractors (monitored by Haley & Aldrich, Inc.).
2. PIC plan set dated March 2025.
3. Existing Farnham culvert loading information provided by Fuss & O'Neill on Sepetmber 17, 2025. Proposed Farnham culvert loading information povided by Fuss & O'Neill on March 10, 2025.

ASSUMPTIONS

1. Consolidation parameters of the compressible soil were assumed based on data from the Brewer-Eddington project. OCR was assumed based on encountered consistency of the clay/silt in the borings.

Consolidation Parameter	Assumed value	
Overconsolidation ratio, OCR	1 to 2	
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 12.6 ft. The soil is double-drained.
3. Bottom of culvert is approx. 14 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.

Proposed Farnham Culvert Bearing Pressures:

Bearing Pressure:

Uniform Bearing Pressure: (Service I)

$$q_{ser.I} := \frac{(DC_{ser.I} + DW_{ser.I} + EV_{ser.I} + LL_{ser.I})}{W_{culv} \cdot L_{bot}} = 0.864 \text{ ksf}$$

Uniform Bearing Pressure: (Strength I)

$$q_{str.I} := \frac{(DC_{str.I} + DW_{str.I} + EV_{str.I} + LL_{str.I})}{W_{culv} \cdot L_{bot}} = 1.165 \text{ ksf}$$

Uniform Bearing Pressure from Dead Load: (Service I)

$$q_{ser.I\_DL} := \frac{(DC_{ser.I} + DW_{ser.I} + EV_{ser.I})}{W_{culv} \cdot L_{bot}} = 0.742 \text{ ksf}$$

Uniform Bearing Pressure from Dead Load: (Strength I)

$$q_{st.I\_DL} := \frac{(DC_{str.I} + DW_{str.I} + EV_{str.I})}{W_{culv} \cdot L_{bot}} = 0.951 \text{ ksf}$$

Existing Farnham Culvert Bearing Pressures:

Bearing Pressure:

Uniform Bearing Pressure: (Service I)

$$q_{ser} := \frac{(DC_{ser} + DW_{ser} + LL_{ser} + EV_{ser})}{W_{bot\_slab} \cdot L_{cribbing}} = 1.743 \text{ ksf}$$

Uniform Bearing Pressure: (Strength I)

$$q_{I} := \frac{(DC_{I} + DW_{I} + LL_{I} + EV_{I})}{W_{bot\_slab} \cdot L_{cribbing}} = 2.314 \text{ ksf}$$

Uniform Bearing Pressure from Dead Load: (Service I)

$$q_{ser\_DL} := \frac{(DC_{ser} + DW_{ser} + EV_{ser})}{W_{bot\_slab} \cdot L_{cribbing}} = 1.603 \text{ ksf}$$

Uniform Bearing Pressure from Dead Load: (Strength I)

$$q_{I\_DL} := \frac{(DC_{I} + DW_{I} + EV_{I})}{W_{bot\_slab} \cdot L_{cribbing}} = 2.068 \text{ ksf}$$

CALCULATIONS

From Settle3:

Maximum Primary Settlement	0.14	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	12.6	ft
Max. Drainage Path Length, Hdr	6.3	ft
Time Factor @ 90% Consol., T_{90}	0.848	
Time to 90% Consolidation, t_{90}	0.53	years

Secondary Compression Calculation

Coeff. of secondary compression, $C_{\varepsilon\alpha}$	0.004	
Culvert Design Life	75	years
Secondary Compression	0.11	ft
	1.30	in

Settle3 Analysis Information

2025-1222-HAI-Farnham Culvert Settlement-d3

Project Settings

Document Name	2025-1222-HAI-Farnham Culvert Settlement-d3.s3z
Author	EMH
Company	Haley & Aldrich, Inc.
Date Created	6/4/2025, 8:52:45 AM
Last saved with Settle3 version	5.018
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 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]	-0.842714	0.142126
Total Consolidation Settlement [in]	-0.842714	0.142126
Virgin Consolidation Settlement [in]	0	0.113554
Recompression Consolidation Settlement [in]	-0.842714	0.0292375
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	-1.37868	0.321995
Loading Stress XX [ksf]	-1.22088	0.360594
Loading Stress YY [ksf]	-1.18243	0.79386
Effective Stress ZZ [ksf]	-1.37868	0.321995
Effective Stress XX [ksf]	-1.22088	0.360594
Effective Stress YY [ksf]	-1.18243	0.79386
Total Stress ZZ [ksf]	-1.37868	0.321995
Total Stress XX [ksf]	-1.22088	0.360594
Total Stress YY [ksf]	-1.18243	0.79386
Modulus of Subgrade Reaction (Total) [ksf/ft]	-1.16243	0.205022
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-1.16243	0.205022
Total Strain	-0.0464211	0.108533
Pore Water Pressure [ksf]	0	0
Degree of Consolidation [%]	-1.42109e-14	1.42109e-14
Pre-consolidation Stress [ksf]	0	2.62449
Over-consolidation Ratio	0	417.875
Void Ratio	-0.227919	0.0974843
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	-0.0113796	0.000951794

Loads

1. Polygonal Load: "New Culvert"

Label	New Culvert
Load Type	Flexible
Area of Load	744.177 ft2
Load	0.742 ksf
Elevation	0 ft
Installation Stage	Stage 2

Coordinates

X [ft]	Y [ft]
123.326	126.294
124.816	76.916
139.924	74.488
138.351	124.341

2. Polygonal Load: "Existing Culvert"

Label	Existing Culvert
Load Type	Flexible
Area of Load	773.048 ft2
Load	1.603 ksf

Advanced Staging

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

Coordinates

X [ft]	Y [ft]
127.125	76.449
143.204	74.007
150.001	122.749
134.901	124.788

3. Polygonal Load: "New Fill"

Label	New Fill
Load Type	Flexible
Area of Load	370.551 ft2
Load	1.81 ksf
Elevation	0 ft
Installation Stage	Stage 2

Coordinates

X [ft]	Y [ft]
138.351	124.322
139.925	74.505
143.204	74.007
150.001	122.749

4. Polygonal Load: "Removed Fill 16.5 ft"

Label	Removed Fill 16.5 ft
Load Type	Flexible
Area of Load	342.199 ft2
Load	2.06 ksf

Advanced Staging

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

Coordinates

X [ft]	Y [ft]
123.326	126.279
124.814	76.9505
127.125	76.449
134.901	124.788

5. Polygonal Load: "Existing/Proposed South Embankment"

Label	Existing/Proposed South Embankment
Load Type	Flexible
Area of Load	3764.25 ft2
Load	2.06 ksf
Elevation	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
88.493	129.843
47.063	132.568
44.176	86.747
124.814	76.9505
123.326	126.279

6. Polygonal Load: "Existing/Proposed North Embankment"

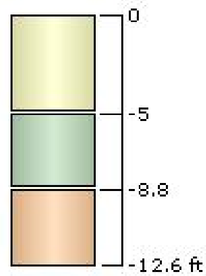
Label	Existing/Proposed North Embankment
Load Type	Flexible
Area of Load	3748.4 ft2
Load	1.81 ksf
Elevation	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
150.001	122.749
143.204	74.007
217.574	61.594
227.764	108.422

Soil Layers

Layer #	Type	Thickness [ft]	Elevation [ft]
1	OC SILT	5	0
2	NC SILT	3.8	-5
3	OC Silty CLAY	3.8	-8.8



Soil Properties

Property	OC SILT	NC SILT	OC Silty CLAY
Color			
Unit Weight [kips/ft3]	0.115	0.115	0.115
Saturated Unit Weight [kips/ft3]	0.115	0.115	0.115
K0	1	1	1
Primary Consolidation	Enabled	Enabled	Enabled
Material Type	Non-Linear	Non-Linear	Non-Linear
Cce	0.19	0.19	0.19
Cre	0.02	0.02	0.02
e0	1.1	1.1	1.1
OCR	1*	1	1*
Undrained Su A [kips/ft2]	0	0	0
Undrained Su S	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8
Piezo Line ID	1	1	1

* Base value only. Refer to Stage Factor section.

Stage Factors, OC SILT

Primary Consolidation - Non-Linear		
	OCR	
Stage	**Factor**	**Value**
Stage 1	1	1
Stage 2	2	2

Stage Factors, OC Silty CLAY

Primary Consolidation - Non-Linear		
	OCR	
Stage	**Factor**	**Value**
Stage 1	1	1
Stage 2	2	2

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	143.552, 117.897	Auto: 55

Query Lines

Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	Query Line 1	129.74, 156.288	133.607, 42.612	50	Auto: 55

Field Point Grid

Number of points	1038
Expansion Factor	2

Grid Coordinates

X [ft]	Y [ft]
270.044	174.848
270.044	19.314
1.896	19.314
1.896	174.848

LATERAL EARTH PRESSURES

HALEY ALDRICH	CALCULATIONS	File No.:	0208476-000
		Sheet:	1 of 2
Client:	Fuss & O'Neill, Inc.	Date:	5/30/2025
Project:	Farnham Culvert Replacement	Computed by:	EMH
Subject:	Lateral Earth Pressure Coefficients	Checked by:	MMB

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.:	0208476-000
		Sheet:	2 of 2
Client:	Fuss & O'Neill, Inc.	Date:	5/30/2025
Project:	Farnham Culvert Replacement	Computed by:	EMH
Subject:	Lateral Earth Pressure Coefficients	Checked by:	MMB

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	

WINGWALL EXTERNAL STABILITY

PROBLEM STATEMENT AND OBJECTIVE

Determine the Capacity Demand Ratios (CDRs) for bearing resistance, sliding resistance, and overturning for the proposed retaining walls.

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.

AVAILABLE INFORMATION

1. Draft PIC Plans dated March 2025.
2. Email correspondance with Fuss & O'Neill.

ASSUMPTIONS

1. The following wall geometry was considered:
   Wall Height, H = 12.2 ft
   Reinforcement Length, L = 12.2 ft (i.e., L/H = 1.0)
   Wall Embedment, E = 3.2 ft
   A 5 ft tall, 2H:1V backslope at the top of the wall.
   A 250 psf live load surcharge acting over the roadway width.
2. The wall will bear on granular fill placed over marine deposit soils. The granular fill will have a friction angle of 32 degrees.
3. An ultimate bearing capacity of 5.2 ksf was calculated independently (see bearing resistance calculation). Groundwater assumed at base of the wall.
4. MSE wall facia assumed to be segmental panels.
5. Seismic analysis will consider PGA = 0.073 and Fpga = 1.6 (As = PGA * Fpga = 0.117), corresponding to Site Class D.

RESULTS

Analysis	Wall Height (ft)	Reinforcement Length (ft)	Bearing CDR	Sliding CDR	Overturning CDR	Eccentricity (e/L)
Static	12.2	12.2	1.01	1.67	2.99	0.10
Seismic			1.24	1.42	2.03	0.23

Notes:
1. CDR = Capacity Demand Ratio.
2. All bearing, sliding, and overturning CDRs are greater than 1 (minimum bearing, sliding, and overturning CDR = 1)
3. Maximum permissible bearing static eccentricity ratio = 0.33; maximum permissible bearing seismic eccentricity ratio = 0.4.
4. MSEW+ program used (AASHTO LRFD 2017-2020).
5. Geogrid modeled for internal reinforcement to simulate the length of the MSE wall. Internal stability was not considered.
6. Analysis performed for external stability only (assume specialty wall designer to check internal stability)
7. The following static resistance factors were assumed:
   Bearing Resistance: 0.65 for MSE Walls, (LRFD Table 11.5.7-1)
   Sliding Resistance: 1.0 (LRFD Table 11.5.7-1)
7. The following seismic resistance factors were assumed:
   Bearing Resistance: 0.9 for MSE Walls, (LRFD Section 11.5.8)
   Sliding Resistance: 1.0 (LRFD Section 11.5.8)

AASHTO 2017-2020
Pittsfield - Farnham

MSEW+: Update # 2021.13

PROJECT IDENTIFICATION

Title: Pittsfield - Farnham
Project Number: 208476
Client: F&O
Designer: EMH
Station Number:

Description:

Company's information:

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.....Farnham MSEW-D4.BENp

Original date and time of creating this file: 10/14/2025

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	32.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ²

Factored bearing capacity resistance of foundation is given: q-ult-static = 5200.0 lb/ft²., q-ult-sesmic = 5200.0 lb/ft².

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.3073 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 61.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3198 (eq. 17 is utilized to calculate Ka for all batters)
(For external stability user specified δ = 21.30°)

BEARING CAPACITY

Bearing capacity is controlled by general shear.
Bearing capacity factors (calculated by MSEW): Nc = N/A N γ= N/A

SEISMICITY (using AASHTO 2017-2020)

Peak ground acceleration coeff., A = PGA = 0.073 and Site Factor, Fpga = 1.600. Maximum ground acceleration coeff., As = 0.117
Design acceleration coefficient in Internal Stability: Kh = Am = 0.117
Design acceleration coefficient in External Stability: Kh_d = 0.117 => Kh = Am = 0.117

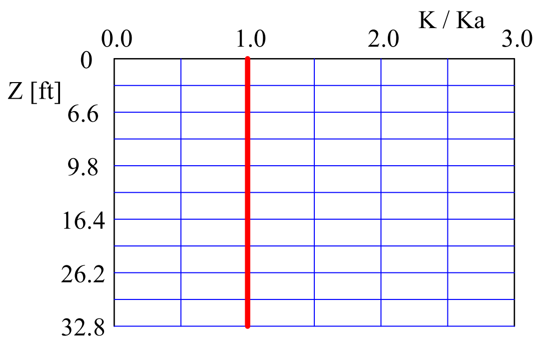
Kae (Kh > 0) = 0.4312 Kae (Kh = 0) = 0.3198
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geogrids
(Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	68519.0				
Durability reduction factor, RFd	2.00				
Installation-damage reduction factor, RFid	3.00				
Creep reduction factor, RFc	5.00	N/A	N/A	N/A	N/A
CDR for strength	N/A				
Coverage ratio, Rc	1.000				
Friction angle along geogrid-soil interface, ρ	21.33				
Pullout resistance factor, F*	$0.80 \cdot \tan\phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

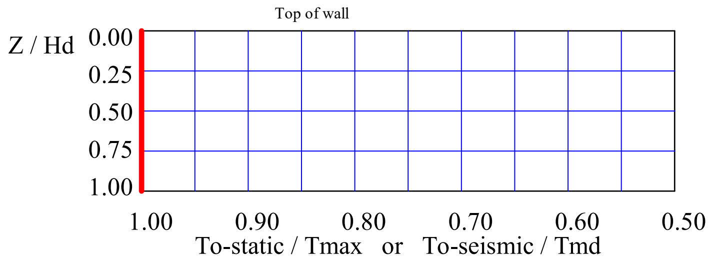
Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.00 ft. Horizontal distance to Center of Gravity of panel is 0.50 ft.
Average unit weight of panel is γ_f = 152.78 lb/ft ³

Z / Hd	To-static / Tmax or To-seismic / Tmd
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



Connection strength, T-lot, is related to T-ult

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Durability reduction factor, RFd	2.00	N/A	N/A	N/A	N/A
Creep reduction factor, RFc	5.00	N/A	N/A	N/A	N/A
Capacity Demand Ratio: connection break, CDR	N/A	N/A	N/A	N/A	N/A
Capacity Demand Ratio: connection pullout, CDR	N/A	N/A	N/A	N/A	N/A
CRu = Tult-connection/Tult-geogrid	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd 12.20 [ft] { Embedded depth is E = 3.20 ft, and height above top of finished bottom grade is H = 9.00 ft }

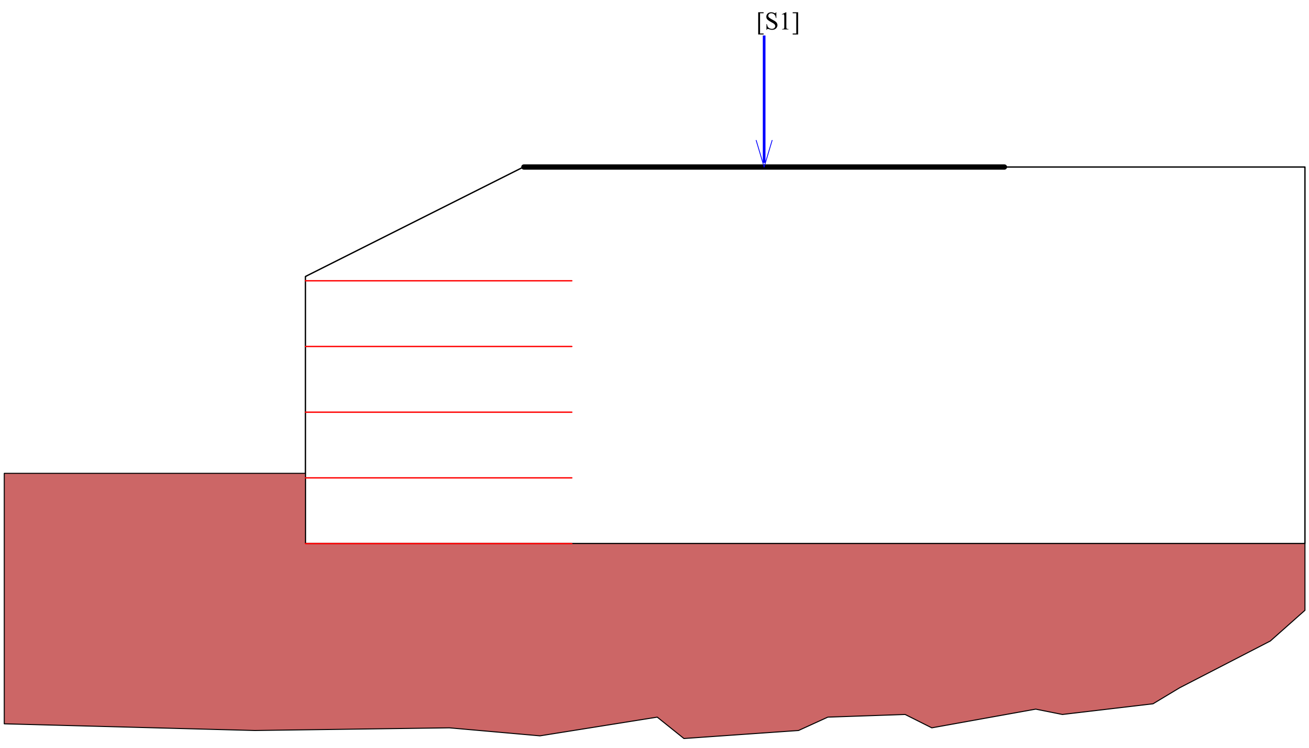
Soil in front of wall is Horizontal.

Batter, ω 0.0 [deg]
Backslope, β 26.6 [deg]
Backslope rise 5.0 [ft] Broken back equivalent angle, I = 11.58° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

OTHER EXTERNAL LOAD(S)
[S1] Strip Load, Qv-d = 0.0 and Qv-l = 250.0 [lb/ft ²].
Footing width, b=22.0 [ft]. Distance of center of footing from wall face, d = 21.0 [ft] @ depth of 0.0 [ft] below soil surface.

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 [ft]

AASHTO 2017-2020 – Load and Resisting Factors

INTERNAL STABILITY

Load factor for vertical earth pressure, EV:	$\gamma_{p\text{-}EV}$	1.35	
Load factor for earthquake loads, EQ:	$\gamma_{p\text{-}EQ}$	1.00	
Load factor for live load surcharge, LS: (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75	
Load factor for dead load surcharge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50	
Resistance factor for reinforcement tension	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.20
Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.20
Resistance factor for geosynthetic pullout	ϕ	0.90	1.20

EXTERNAL STABILITY

		Static		Combined Static/Seismic
Load factor for vertical earth pressure, EV				
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.00
Load factor for earthquake loads, EQ (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

		Static	Combined Static/Seismic
Resistance factor for shear resistance along common interfaces			
Reinforced Soil and Foundation	ϕ_{τ}	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_{τ}	1.00	1.00

		Static	Combined Static/Seismic
Resistance factor for bearing capacity of shallow foundation	ϕ_b	0.65	0.90

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.01, factored bearing load = 3351 lb/ft².

Foundation Interface: Direct sliding, CDR = 1.665, Eccentricity, e/L = 0.1014, CDR-overturning = 2.99

GEOGRID #	Elevation [ft]	Length [ft]	Type #	CONNECTION CDR [connection break]	Geogrid strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.00	12.20	1	5.25	1.945	15.735	1.040	0.1014	---
2	3.00	12.20	1	3.15	1.166	6.747	1.152	0.0535	---
3	6.00	12.20	1	4.28	1.585	6.123	1.286	0.0069	---
4	9.00	12.20	1	6.68	2.476	5.729	1.434	-0.0450	---
5	12.00	12.20	1	21.07	7.804	8.904	1.497	-0.1346	---

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, CDR = 1.24, factored bearing load = 3759 lb/ft².

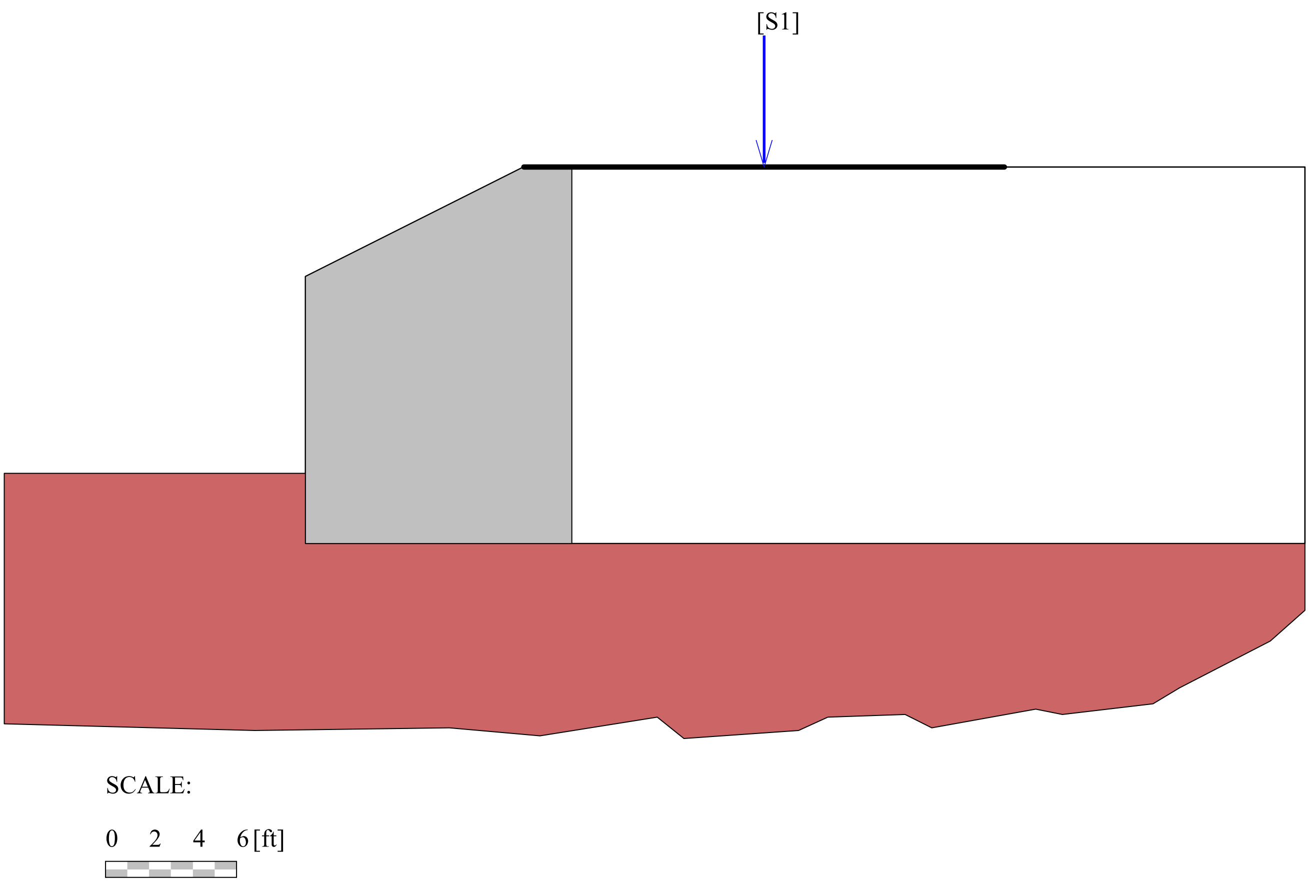
Foundation Interface: Direct sliding, CDR = 1.423, Eccentricity, e/L = 0.2287, Fs-overturning = 2.03

GEOGRID #	Elevation [ft]	Length [ft]	Type #	CONNECTION CDR [connection break]	Geogrid strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.00	12.20	1	8.97	3.323	17.861	0.889	0.2287	---
2	3.00	12.20	1	5.49	2.032	8.369	0.970	0.1640	---
3	6.00	12.20	1	7.38	2.733	7.234	1.059	0.1031	---
4	9.00	12.20	1	11.26	4.171	6.148	1.138	0.0403	---
5	12.00	12.20	1	31.20	11.556	6.171	1.110	-0.0513	---

BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Given factored bearing resistance, q-n)			
Factored bearing resistance. q-n	3380	4680	[lb/ft ²]
Factored bearing load, σ_V	3351.0	3759	[lb/ft ²]
Eccentricity, e	0.71	1.20	[ft]
Eccentricity, e/L	0.058	0.098	
CDR calculated	1.01	1.24	
Base length	12.20	12.20	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Static: Unfactored R = 26230.23 [lb/ft], L = 12.20, Unfactored e = 0.58 [ft], and Sigma = 2377.73 [lb/ft ²]
Seismic: Unfactored R = 26652.73 [lb/ft], L = 12.20, Unfactored e = 1.17 [ft], and Sigma = 2701.04 [lb/ft ²]



DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.665 and CDR-seismic = 1.423

#	Geogrid Elevation [ft]	Geogrid Length [ft]	CDR Static	CDR Seismic	Geogrid Type #	Product name
1	0.00	12.20	1.040	0.889	1	---
2	3.00	12.20	1.152	0.970	1	---
3	6.00	12.20	1.286	1.059	1	---
4	9.00	12.20	1.434	1.138	1	---
5	12.00	12.20	1.497	1.110	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.1014, e/L seismic = 0.2287; Overturning: CDR-static = 2.99, CDR-seismic = 2.03

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.00	12.20	0.1014	0.2287	1	---
2	3.00	12.20	0.0535	0.1640	1	---
3	6.00	12.20	0.0069	0.1031	1	---
4	9.00	12.20	-0.0450	0.0403	1	---
5	12.00	12.20	-0.1346	-0.0513	1	---

GLOBAL STABILITY

CALCULATIONS		File No.:	208476-000
		Sheet:	1 of 1
Client:	Fuss & O'Neill	Date:	6-Jan-2026
Project:	Farnham Culvert Replacement	Computed by:	EMH
Subject:	Global Stability for Culvert Wingwalls	Checked by:	TPJ

PROBLEM STATEMENT AND OBJECTIVE

Calculate the global stability minimum factor of safety for culvert wingwalls for static analyses.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. Maine Department of Transportation Bridge Design Guide (BDG) 2003, with updates to June 2018.
3. Slide version 9.0 by RocScience.

AVAILABLE INFORMATION

1. Boring logs dated 8-18-2023 to 8-25-2023 by New England Boring Contractors (monitored by Haley & Aldrich, Inc.).
2. PIC plan set dated March 2025.

ASSUMPTIONS

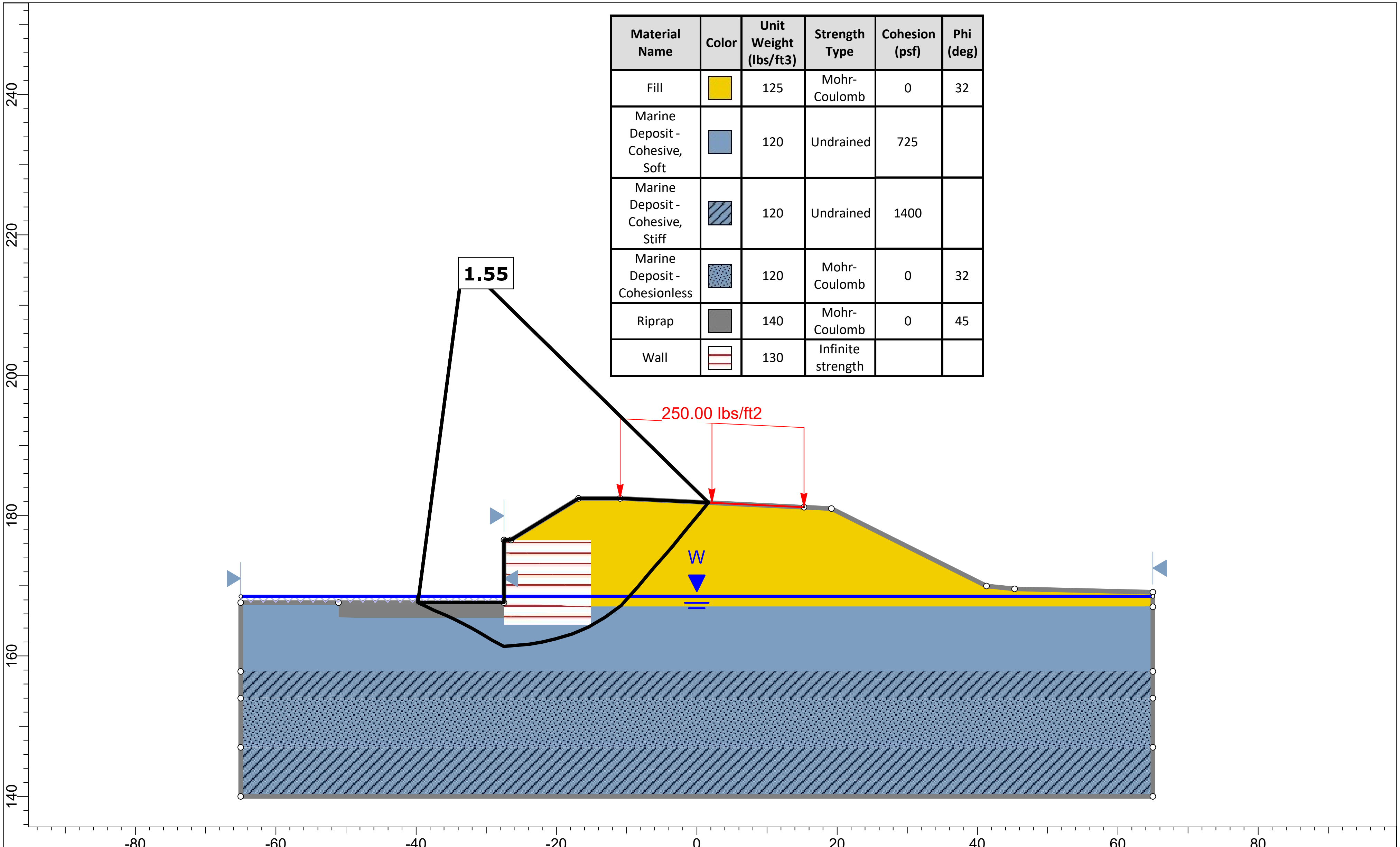
1. Water level will be modeled at El. 168.5 based on the surveyed water level at the culvert.
2. Refer to the Slide outputs on the following pages for material properties used in the global stability analysis.
3. Seismic cases will have a seismic force of As/2 (0.117g/2) = 0.059 g based on the seismic site class calculations.
4. A 250 psf traffic surcharge will be modeled within the limits of the roadway.
5. Analyses will be performed using Spencer's method considering non-circular failure surfaces.
6. The wingwall will be modeled with infinite strength (i.e., only external wall stability evaluated).

RESULTS AND CONCLUSIONS

Analysis	Minimum Calculated Factor of Safety
Static	1.55
Pseudostatic	1.42

Based on AASHTO LRFD Section 11.6.3.7, an acceptable resistance factor where the geotechnical parameters and subsurface stratigraphy are well defined is 0.75 (F.S. = 1/0.75 = 1.3) for static conditions.

Per discussions with MAineDOT, the preferred acceptable factor of safety for pseudostatic conditions is 1.1.

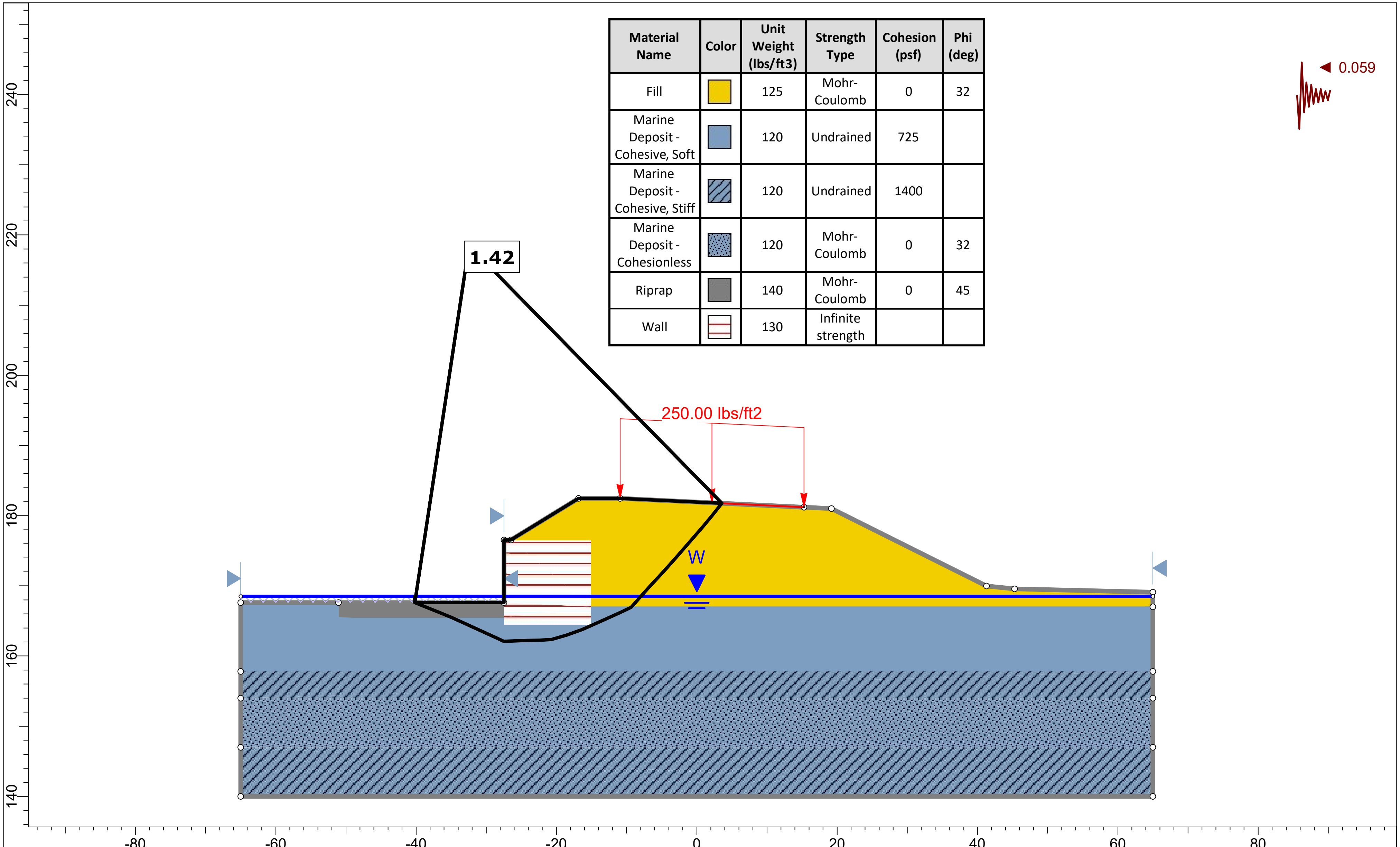


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		125	Mohr-Coulomb	0	32
Marine Deposit - Cohesive, Soft		120	Undrained	725	
Marine Deposit - Cohesive, Stiff		120	Undrained	1400	
Marine Deposit - Cohesionless		120	Mohr-Coulomb	0	32
Riprap		140	Mohr-Coulomb	0	45
Wall		130	Infinite strength		

rocscience

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	EMH	*Company*	Haley & Aldrich, Inc.
Date	12/22/2025, 3:12:08 PM	*File Name*	2025-1222-Pittsfield Farnham Global Stabiliy-d1.slmd

SLIDEINTERPRET 9.026



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		125	Mohr-Coulomb	0	32
Marine Deposit - Cohesive, Soft		120	Undrained	725	
Marine Deposit - Cohesive, Stiff		120	Undrained	1400	
Marine Deposit - Cohesionless		120	Mohr-Coulomb	0	32
Riprap		140	Mohr-Coulomb	0	45
Wall		130	Infinite strength		

rocscience

SLIDEINTERPRET 9.026

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Pseudostatic
Drawn By	EMH	*Company*	Haley & Aldrich, Inc.
Date	12/22/2025, 3:12:08 PM	*File Name*	2025-1222-Pittsfield Farnham Global Stabiliy-d1.slmd

FROST PROTECTION

HALEY ALDRICH	CALCULATIONS	File No.:	0208476-000
		Sheet:	1 of 2
Client:	Fuss & O'Neill	Date:	30-May-2025
Project:	Farnham Culvert Replacement	Computed by:	EMH
Subject:	Frost Penetration Depth Evaluation	Checked by:	MMB

OBJECTIVE:

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

REFERENCES:

1. MaineDOT Bridge Design Guide, August 2003, with June 2018 updates
2. Boring logs BB-PFB-101 and BB-PFB-102
3. Laboratory testing performed by GeoTesting Express.

EVALUATION:

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

Boring No.	Sample No.	Depth	USCS	Moisture Content (%)
BB-PFB-101	5D	15.0 ft to 17.0 ft	ML	25.0
BB-PFB-101	8D	35.0 ft to 37.0 ft	CL	25.0
BB-PFB-102	7D	15.0 ft to 17.0 ft	CL	31.0

2. Per MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1750°F - days.

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

4. For fine-grained soil with water content of approx. 27 percent (average of water content results above), per Table 5-1, maximum depth of frost is approximately 4.25 ft (51 in.).
Subgrade soils directly beneath the proposed culvert are primarily fine-grained, therefore coarse-grained soil was not considered.

Recommend culvert be founded at least 4.25 ft below the lowest adjacent ground surface exposed to freezing.

HALEY ALDRICH	CALCULATIONS	File No.:	0208476-000
		Sheet:	2 of 2
Client:	Fuss & O'Neill	Date:	30-May-2025
Project:	Farnham Culvert Replacement	Computed by:	EMH
Subject:	Frost Penetration Depth Evaluation	Checked by:	MMB

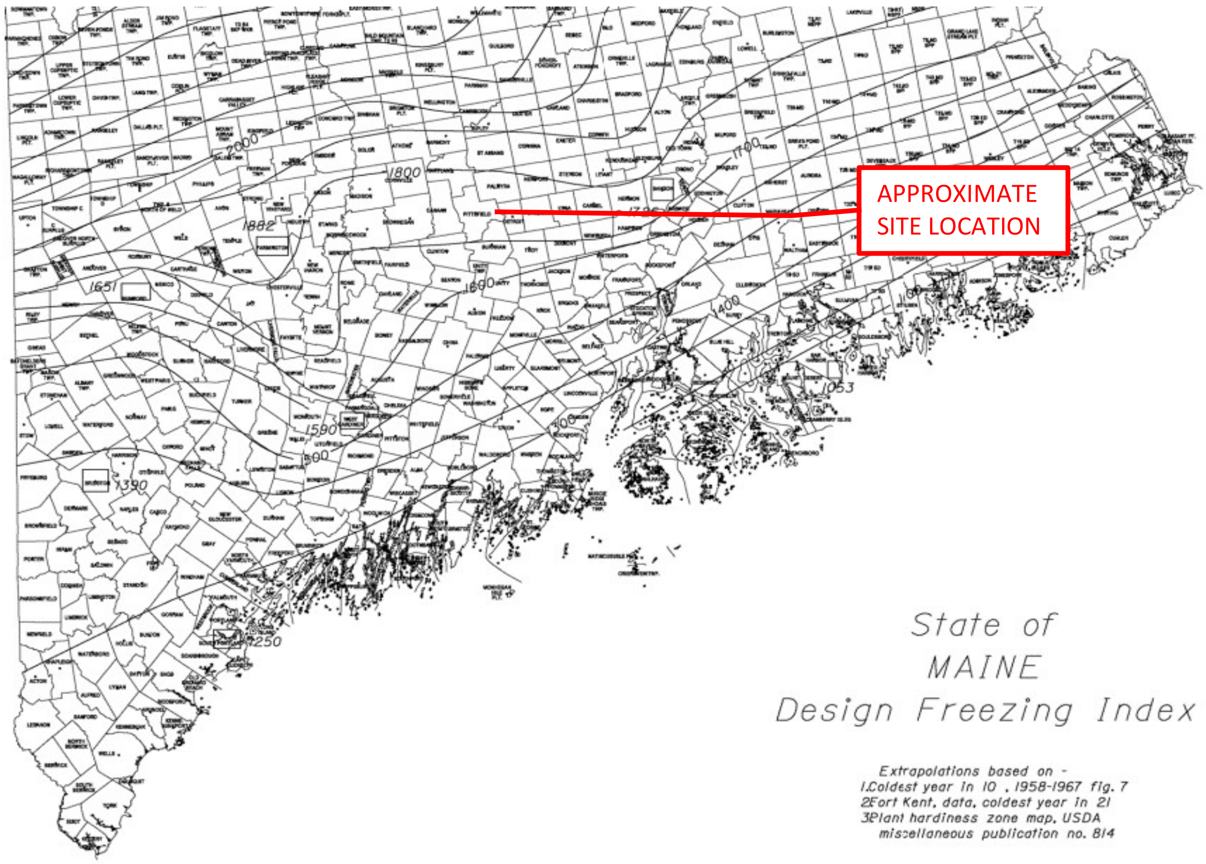


Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0