

WAGGON BRIDGE OVER
WILSON STREAM
REPLACEMENT

WIN 26234.00

City of Monmouth
Kennebec County
Maine

SUMMARY OF
FINAL
GEOTECHNICAL
ASSESSMENT

August 2025

PREPARED FOR

**Maine Department of
Transportation**

16 State House Station
Augusta, Maine 04333

PREPARED BY

HNTB Corporation

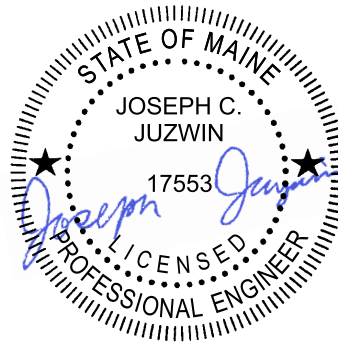
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Waggon Bridge Over Wilson Stream Replacement

Monmouth, Maine

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August 29, 2025



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Prepared For:

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1.0 PROJECT DESCRIPTION AND SCOPE

1.1 Project Background

The following report summarizes the final geotechnical assessment completed by HNTB for the replacement of the Waggon Bridge, carrying Waugan Road over Wilson Stream in Monmouth, Kennebec County, Maine. A Project Location Map is included as Figure 1.

The existing single-span structure carries a single travel lane over Wilson Stream. It is proposed that the existing structure will be replaced along its current alignment with a new single-span structure supported by integral abutments. The proposed structure will accommodate two lanes of traffic. Approach embankments will be widened to accommodate the new bridge width.

1.2 Scope of Services

HNTB's **Geotechnical and Foundation Services group** is providing geotechnical design services for the replacement of the Waggon Bridge. The overall objective of this study is to characterize the subsurface conditions within the project limits and to develop geotechnical design criteria and foundation recommendations for support of the proposed replacement structure and approach embankments.

A preliminary subsurface exploration program was executed by HNTB's **partner** Schonewald Engineering Associates, Inc. (Schonewald), in coordination with HNTB. The final assessment completed by HNTB included the following scope of services:

- Reviewed historical geotechnical data for the project site.
- Implemented a subsurface exploration and laboratory testing program to gather additional information where needed.
- Analyzed the resulting data collected to identify subsurface conditions that impact the design and construction of the project.
- Established engineering design parameters based on the available borings.
- Conducted geotechnical analyses and provided final recommendations and design parameters for the support of the proposed bridge and construction of the approach embankments.
- Established seismic site classification

1.3 Existing Site Conditions

At the project site, Waugan Road is generally oriented in a north/south direction carrying two undivided travel lanes, one in each direction. The roadway tapers to a single bi-directional lane across the bridge where Wilson Stream crosses underneath at a slight skew in a northeast-southwest direction. The roadway profile across the bridge and approaches is relatively flat and slightly elevated above the surrounding terrain area on embankment. The top of existing bridge deck is approximately 3 feet above normal water elevation, which results in regular flooding. The maximum depth of Wilson Stream is approximately 10 feet.

The existing Waggon Bridge was constructed in 1980 and consists of a single, 39-foot steel multi-girder span with steel plate deck and asphalt surface. The span is supported on full-height stacked stone masonry abutments with concrete caps, forming a roughly U-shaped channel in the stream.

The east side of north approach and west side of south approach gradually slope down into Wilson Stream, while northwest and southeast quadrants slope to floodplain.

1.4 Proposed Improvements

The existing structure will be replaced along its current alignment with a new, approximately 64-foot-long single-span two-lane concrete structure supported on integral abutments. The integral abutments will be set behind the existing concrete abutments and are anticipated to be supported on H-pile foundations driven to top of rock. The ground surface in front of abutments will be regraded to improve channel geometry and armored for scour protection. Approach embankments will be widened immediately behind the abutments to accommodate the added lane. In-line wingwalls will be constructed with the abutments.

1.5 Survey Control

All horizontal coordinates presented in this report are provided in feet and reference the North American Datum of 1983 (NAD 83), Maine West State Plane coordinate system. Elevations are provided in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

2.0 GEOLOGY AND SITE CONDITIONS

2.1 General Site Geology

The project is located within the New England physiographic province, a region characteristically composed of gneiss, schist, marble, quartzite, and slate, with frequent folding, faulting, and igneous intrusions. The region has been subject to glaciation, resulting in a large number of lakes and streams, and surficial geology composed primarily of glacial deposits (glacial till, glaciofluvial deposits and glacial lake deposits).

Existing geologic mapping available for the project site includes Soil Survey data provided by the USDA Natural Resource Conservation Service (NRCS) and bedrock and surficial geology mapping prepared by the Maine Geological Survey (MGS) for the Wayne and Winthrop quadrangles. Excerpts from these maps are included in Figure 2 and Figure 3, respectively

NRCS mapping indicates that the project site consists of Togus fibrous peat and Scantic silt loam, both originating from glacial meltwaters through the site. Surficial geology mapping indicates that the bridge is located in wetland deposits containing muck, peat, silt, and clay in poorly drained areas. Immediately beyond the wetland deposits, surficial materials are identified as belonging to the Presumpscot Formation, consisting of glaciomarine silt and clay with sand deposited on the sea floor during the late-glacial marine submergence. Bedrock geology mapping shows the site is located within the Sangerville Formation, consisting of medium-grained, medium-gray biotite-muscovite-quartz-feldspar granofels interbedded with biotite-muscovite-quartz-feldspar schist.

3.0 SUBSURFACE EXPLORATION AND TESTING

3.1 Subsurface Exploration

A subsurface exploration program was performed for this project by New England Boring Contractors of Derry, NH under the supervision of Schonewald on behalf of HNTB. A series of two borings were performed at the project site, one at each proposed abutment location. The borings were advanced by cased wash boring methods, using 4.0-inch (HW-size) and 3.0-inch (NW-size) inside diameter steel casing. Standard Penetration Test (SPT) sampling was conducted in general accordance with ASTM D1586 by driving a 1-3/8 inch ID split spoon sampler with a 140-pound hammer dropped 30 inches to obtain samples at 5 foot intervals. An automatic hammer was utilized, and a current calibration report for the hammer was obtained by Schonewald as verification of hammer efficiency factor. Each sample was removed from the sampler in the field, examined, and classified in accordance with the Burmister Soil Identification System. Sampling was advanced to top of rock, and a minimum of 10 feet of NQ2-sized rock core was collected from each boring location. Where Presumpscot Formation cohesive soils were encountered, SPT sampling was supplemented with in-situ vane shear testing to provide an estimate of undisturbed and remolded shear strength. The Boring Location Plan is included as Figure 4. Draft boring logs prepared by Schonewald are included in Appendix 1.

3.2 Laboratory Testing

A geotechnical laboratory testing program was performed by Geotesting Express of Acton, Massachusetts to verify the visual-manual field classifications and to aid in determination of the engineering design parameters. Testing included index tests to aid in identification and classification of soils, and corrosion testing for use in evaluating corrosivity of steel substructure elements. The complete laboratory results are presented in Appendix 2. Laboratory testing consisted of twelve (12) tests for natural moisture content, four (4) standard grain size analyses with natural water content, five (5) tests for fines content, twelve (12) tests for Atterberg limits, two (2) tests for organic content, and two (2) undisturbed samples for consolidation and consolidated-undrained triaxial testing. The soil testing was performed in accordance with the following ASTM Standards:

Moisture Content	ASTM D2216
Grain Size Analysis	ASTM D6913
Percent Passing No. 200 Sieve	ASTM D1140
Atterberg Limits	ASTM D4318
Organic Content	ASTM D2974
One-Dimensional Consolidation	ASTM D4767
Consolidated-Undrained Triaxial	ASTM D4767

Select soil samples were tested for pH, minimum laboratory resistivity, chlorides content, and sulfate content by GeoTesting Express, Inc. of Acton, Massachusetts for determination of site subsurface corrosion potential. The corrosivity testing was performed in accordance with the following Standards:

pH	AASHTO T289
Sulfates	AASHTO T290
Chlorides	AASHTO T291
Minimum Laboratory Resistivity	AASHTO T288

In instances of inadequate material recovered, the Electrical Resistivity test per ASTM G57 was performed. A summary of the laboratory corrosion tests is presented in Table 3.2. The below results are not indicative of aggressive environmental conditions for corrosion of steel or concrete structure and foundation elements.

Table 3.2: Summary of Corrosion Test Results

Boring No.	Sample No.	Depth (ft)	pH	Resistivity (ohm-cm)	Sulfate (ppm)	Chloride (ppm)
BB-MWS-101	2D, 3D	4 – 6, 9 – 10.5	6.34	5,165	14	21

4.0 SUBSURFACE CONDITIONS

4.1 Stratigraphy

Based on the subsurface exploration, the following general subsurface strata were identified:

- **Fill:** The bridge site is overlain with up to 8 feet of loose, fine to medium grained granular fill consisting of varying portions of sand, gravel, and fines. In some instances, the fines were identified to be plastic based on field classifications.
- **Stream Alluvium:** In boring BB-MWS-101, below the fill stratum, a 6-foot-thick alluvium layer consisting of a very loose to loose sandy organic silt was encountered. The material is classified as being mottled in one sample and having fibrous peat and wood in others.
- **Marine Silt & Clay:** Beneath the Alluvium, a Presumpscot Formation Marine Silt and Clay stratum was encountered generally consisting of a grey Silty Clay, and Clay with some silt. This stratum was observed to have a thickness ranging from approximately 22 to 34 feet. The stratum ranged from very soft to soft. In boring BB-MWWS-102, approximately 6.5 feet of medium stiff desiccated “crust” was encountered at the top of the stratum.
- **Glacial Till:** A generally dense to very dense Glacial Till layer underlies the Marine Silt & Clay stratum consisting primarily of varying amounts of dark grey fine to coarse sand and gravel, with little to some silt. This stratum had a thickness ranging from approximately 4.5 feet to 11.5 feet, increasing in thickness from south to north.
- **Granofels Bedrock:** The top of bedrock was encountered at a depth of 40.2 feet in boring BB-MWS-101 and at 57.6 feet in BB-MWS-102 indicating a steep slope from one end of the bridge to the other. Bedrock was cored at both borings and was classified as a fine to medium grained Granofels.

The rock encountered was medium grey with visible bedding, hard and typically fresh with rough joints. Rock Quality Designation (RQD) values ranged from 0 to 77 percent and averaged approximately 36 percent.

Based on a review of boring data and as-built drawings, the pile foundations of the existing bridge abutments are anticipated to rely primarily on end bearing below the Presumpscot clay, in the glacial till or at top of rock. An interpretive subsurface profile is included in Figure 5.

4.2 Subsurface Material Properties

Geotechnical design parameters for soil and rock were developed for each stratum based on material descriptions, laboratory test results, standard published correlations, and engineering judgment. A summary of soil and rock design properties at the proposed bridge abutments and approaches, as interpreted from available borings and laboratory testing, are presented in Table 4.2 below. Foundations will terminate at the top of rock however general rock parameters were calculated for pile tip parameters.

Table 4.2: Engineering Properties of Soil

Layer	\overline{N}_{60} (bpf)	\overline{N}_{160} (bpf)	γ (pcf)	Φ (deg.)	S_u (psf)	K_{aw} (pci)	K_{bw} (pci)	G (ksi)	E (ksi)
1 – Fill	5	8	111	31	-	71	46	131	318
2 – Alluvium	4	5	108	30	-	52	37	121	290
3A – Marine Silt & Clay – Crust	5	7	115	-	600	-	-	135	324
3B – Marine Silt & Clay	1	1	110	31	536-137	-	-	68	162
4 – Glacial Till	92	49	126	40	-	233	117	465	1289

- Where:
- \overline{N}_{60} = Average SPT-N value of stratum, corrected for hammer efficiency, in blows per foot.
 - \overline{N}_{160} = Average SPT-N value of stratum, corrected for hammer efficiency and effective overburden pressure, in blows per foot.
 - γ = Total unit weight of soil, based on grain size and relative density per Bowles 4th Edition, Table 3-4.
 - Φ = Internal friction angle of soil, per multiple SPT-N value correlations for cohesionless material. Drained friction angle for Marine Silt and Clay from laboratory triaxial testing.
 - S_u = Undrained shear strength of soil, per in-situ vane shear testing.
 - K_{aw} = Subgrade modulus above water table, based on Figure 6.8.7-1, pp. 70 of API RP 2A.
 - K_{bw} = Subgrade modulus below water table, based on Figure 6.8.7-1, pp. 70 of API RP 2A.
 - G = Shear modulus, taken as $E/[2(1+\mu)]$ per Bowles 5th Edition, pg. 121, equation (a).
 - E = Elastic modulus, based on grain size and relative density as presented in AASHTO LRFD 9th Ed. Table C10.4.6.3-1.

Undrained shear strengths measured from field vane shear testing performed on site were found to vary from stiff closer to the desiccated crust layer to soft towards the bottom of the layer and is listed as such in the table above. Refer to Appendix 2 for full suite of subsurface design parameters.

5.0 FINAL DESIGN OF BRIDGE FOUNDATIONS

5.1 General Design Considerations

The proposed substructure configuration consists of integral abutments supported on vertical concrete stem walls. For final design, abutments founded on driven steel H-piles are recommended. A driveability

assessment was performed using GRLWEAP Version 14 by Pile Dynamics, Inc. to assess the feasibility of such piles. HP14x73 were assessed for driving. Based on this assessment, no driving issues are anticipated in the encountered subsurface conditions using common pile-driving hammers, indicating that pile design will be structurally limited based on combined axial compression and bending requirements. The required nominal driving resistance for abutment piles should be estimated based on a Strength limit state resistance factor of 0.65 for axial geotechnical compression resistance, assuming dynamic load testing with signal matching will be performed on a minimum of 2% of production piles per abutment. Calculations of the required nominal driving resistance shall also take into consideration settlement-induced downdrag, as presented below in Section 6.4. Refer to Appendix 2 for Driveability calculations. A lateral and structural assessment for pile axial and bending resistance was performed by the project structural engineer and is included in the structural calculations under separate cover.

5.2 Foundation Construction Considerations

In order to construct the new bridge foundations, the close proximity of the stream will need to be considered. It is anticipated that the new bridge abutments will be slightly offset behind the existing abutments and will be approximately 9 feet in height. To construct the abutments, excavation will need to be advanced below the water level likely by installing a cofferdam made of driven steel sheet piles keyed into the underlying Presumpscot Clay & Silt aquitard. It is anticipated the cofferdam shall be constructed around the entire perimeter of the abutment on both approaches to allow for excavation below water and subsequent abutment construction. Dewatering of the cofferdam will be required to allow for placement of abutment reinforcement and concrete pouring. Tremie seals are not recommended for integral abutments; sheet piles should be keyed into the clay aquitard. The cofferdam designer should consider seepage and bottom heave when specifying minimum embedment depth of cofferdam sheet piles.

It is expected all piles will be driven to the top of rock. In the event that piles are unable to be driven to the desired depth due to an obstruction, the obstruction shall be excavated, and area properly backfilled prior to pile driving again. Any pile relocation or modification in pile array shall be discussed and agreed with the engineer.

5.3 Dynamic Pile Load Testing Program

When driven piles are installed for the proposed abutments, an independent wave equation analysis should be performed by the contractor to verify that the proposed pile hammer is adequate to drive the piles to the required nominal resistances without subjecting the pile to stresses greater than 45 ksi. A separate wave equation analysis should be provided at each abutment, for a total of two (2) analyses as per MaineDOT Bridge Design Guide (BDG) Table 5-10. The first pile driven at each North and South abutment should be dynamically tested to confirm nominal pile resistance and develop a driving criterion for the remainder of the piles. The pile driving acceptance criteria should prevent pile damage and be structured so that the end of initial drive penetration resistance is between 3 and 15 blows per inch over the final 6 inches of driving. If the pile penetration is less than 0.5 inches over 10 consecutive blows, the driving may be terminated.

Relaxation may occur when piles are driven to and tip on poor quality bedrock resulting in a loss of pile resistance over time. Minimum 24 hour pile restrikes should be conducted on all test piles in order to ensure the required nominal pile resistance has been achieved and verify that pile relaxation has not occurred. Additional dynamic pile tests may be required if pile behavior varies significantly between adjacent piles.

6.0 FINAL DESIGN OF BRIDGE APPROACHES

6.1 Roadway Widening Considerations

To match the widened bridge, the existing tapered approaches are to be widened to two lanes. Due to the underlying very soft soils, special considerations will need to be taken in order to successfully widen the roadway. The placement of fill on the side slopes transverse to the roadway as well as longitudinal to the bridge abutment into the stream was assessed for global stability. A roadway profile raise of up to fifteen (15) inches was also checked for estimated settlement.

6.2 Stability Assessment

In analyzing global stability, limit equilibrium analyses were performed for critical cross-sections using the Slope/W module of GeoStudio 2021, distributed by Geo-Slope International Ltd. Subsurface conditions for global stability analysis were selected based on conditions at the nearest abutment borings and stability was evaluated both transverse to the roadway behind each abutment and parallel to the roadway in front of the proposed abutment. **Spencer's method of analysis has been used to perform all global stability analyses and satisfies both force and moment equilibrium which meets the requirements prescribed by AASHTO LRFD Bridge Design Specifications, 9th Ed. (AASHTO LRFD) Article C11.6.2.2 for slope stability analysis.** Global stability analysis was performed for long term and short term loading conditions using estimated drained and undrained soil strength design parameters. For all slope stability analysis, a 250 psf uniform live load surcharge was applied within proposed roadway limits, and a normal stream and groundwater table elevation of 170.59' was assumed. Analyses were also run with a lower stream elevation to assess water fluctuation impacts. Global stability analysis was performed in the Service-I loading condition. AASHTO LRFD requires a resistance factor of 0.75 for slopes not supporting structures and 0.65 for slopes supporting structures in order to satisfy global stability, which approximately equates to a 1.3 and 1.5 factor of safety, respectively.

To achieve the minimum factor of safety for side slopes on all bridge quadrants, a minimum side slope of 2H:1V with a 3-foot-thick surficial riprap layer starting at the road shoulder is required. This yields factors of safety at critical cross sections meeting or exceeding the required 1.3, eliminating the need for any sheetpiles or lightweight fill.

For the slopes in front of the integral abutment, existing conditions indicate a very steep to vertical slope from the abutment down to the stream bed. New slopes are to be constructed below the proposed bridge starting at approximately Elevation 169.5 feet with an inclination of 1.75H:1V to the stream bottom located at about Elevation 159 feet. The slopes were modeled with a minimum three-foot-thick surficial riprap layer to prevent scour and shallow sloughing. As the frame behavior of the proposed integral abutments and piles

will inherently resist slope failure through the abutment stems, a shear force equal to the active earth pressure of soil acting along an assumed 8-foot-deep integral abutment was incorporated into the assessment. This approach is assumed conservative considering that the piles, abutment, and bridge will behave as a stiff frame, which is capable of providing equal and opposite resistance up to the passive earth pressure. The analysis indicated that the north proposed abutment slope has a Factor of Safety of less than 1.5 for the drained condition. Understanding that the foundation is supported on piles driven to the top of rock, that piles are not relied upon for stability of the slope, and that the abutment is not directly supported by the embankment, there is little concern of foundation instability due to a slope failure. HNTB recommends acceptance of the front slope not directly supporting the bridge foundations with a stability factor of safety exceeding 1.3.

Global stability factors of safety are summarized in Table 6.2 below. Refer to Appendix 2 for global stability calculations.

Table 6.2: Slope Stability Factors of Safety

Location	Factor of Safety Undrained	Factor of Safety Drained
Side Slope - Abut 1 Sta 25+95 (Left)	1.40	1.3
Side Slope - Abut 1 Sta 25+95 (Right)	1.39	1.39
Side Slope - Abut 2 Sta 26+70 (Left)	1.34	1.34
Side Slope - Abut 2 Sta 26+70 (Right)	1.32	1.32
Longitudinal - Abutment 1	1.61	1.45
Longitudinal - Abutment 2	1.58	1.97

6.3 Settlement Assessment

A settlement assessment was conducted to evaluate the impact of the proposed fill placement in widened limits of embankment, as well as a potential roadway profile raise of up to 14 inches. Increase in grade varies from 8 inches at the crown of the existing roadway to approximately 4.5 feet on the farthest edges of the roadway. Settlement was assessed in Settle3 Version 5.016 by Rocscience with resulting estimated 15-year and total settlement magnitudes summarized in Table 6.3 and Table 6.4 for two critical sections on both sides of the proposed bridge. Secondary settlement estimates are included over a 75-year design life of the embankment. Refer to calculations for tables showing settlements divided into typical repaving cycles for better clarity on long-term settlement. Refer to Appendix 2 for settlement calculations.

Table 6.3: Estimated Settlement of Approach Embankments at Sta. 25+95.00

Location	Transverse Offset (ft)	Anticipated Settlement at 15 Years (in)	Total Anticipated Settlement (in)
Maximum Settlement	15 Left	<1.7"	< 3.4"
Left Guardrail	15 Left	<1.7"	< 3.4"
Center of Roadway	0	<0.9"	< 2.1"
Right Guardrail	15 Right	<0.5"	< 1.2"

Table 6.4: Estimated Settlement of Approach Embankments at Sta. 26+70.00

Location	Transverse Offset (ft)	Anticipated Settlement at 15 Years (in)	Total Anticipated Settlement (in)
Maximum Settlement	15 Right	<0.8"	< 1.6"
Left Guardrail	15 Left	<0.6"	< 1.1"
Center of Roadway	0	<0.7"	< 1.5"
Right Guardrail	15 Right	<0.8"	< 1.6"

6.4 Downdrag

Due to the soft soil conditions found at the proposed locations of the integral abutment piles, downdrag calculations were performed in accordance with AASHTO LRFD Article 3.11.8. The Settle3 models referenced in Section 6.3 were utilized to determine the extent of downdrag down to the point where settlement was less than 0.4 inches relative to the pile and considered materials above this point to be contributing to downdrag. An Ensoft APILE model was created to then model the unit skin friction of each unique soil layer to assist in the calculation of the total downdrag load for several driven steel HP14 pile types. Estimated downdrag loads for evaluated pile sizes are summarized below in Table 6.5.

Table 6.5: Summary of Unfactored Downdrag Loads per Pile Type (kips)

Abutment #	HP14x74	HP14x89	HP14x102	HP14x117
1	51.7	51.8	52.4	52.9
2	28.3	28.6	28.9	29.2

The required nominal pile driving resistance taking into consideration downdrag loads, R_{ndr} , should be calculated in accordance with the below equation per AASHTO LRFD Section C10.7.3.7, considering the downdrag loads indicated above in Table 6-5 and the factored axial compression load per pile as determined from the substructure structural design:

$$R_{ndr} = \frac{\sum \gamma_i Q_i}{\phi_{dyn}} + \frac{\gamma_p DD}{\phi_{dyn}} + R_{sdd}$$

Where:

- $\sum \gamma_i Q_i$ = The Strength limit state factored load per pile (kips) per substructure design.
- ϕ_{dyn} = Resistance factor for axial geotechnical compression, 0.65 per project recommendations and required dynamic testing.
- γ_p = Load factor for downdrag, 1.40 for Strength limit state analysis per AASHTO LRFD Table 3.4.1-2.
- DD = Downdrag load per pile (kips) as indicated above in Table 6.5.
- R_{sdd} = Side resistance which must be overcome during driving through the downdrag zone, taken as DD for this project (kips).

The nominal pile driving resistance should not exceed 90% of the nominal axial structural compression resistance of selected pile size.

7.0 SEISMIC CONDITIONS

7.1 Seismic Activity

Unlike the seismically active regions in the western United States where the North American and the Pacific plates meet, earthquakes in the northeast occur less frequently, are typically smaller in magnitude, more complex and less understood than plate boundary activity. The project site is situated nearly halfway between the plate boundaries to the west and the Mid-Atlantic Ridge to the east where the North American Plate diverges from the Eurasian Plate. Seismic events occurring deep within these boundaries are known as interplate earthquakes. In the last few decades, a common explanation for the cause of these earthquakes is that ancient zones of weakness are being reactivated in the present-day stress field. Due to these uncertainties the level of seismic hazard in the northeast, as presented by USGS mapping, is based primarily on the past records of seismic activity rather than the location of geologically mapped faults.

According to USGS earthquake history, several significant earthquake epicenters have been recorded in the Northeast region. Table 7.1 summarizes some of the notable earthquakes of magnitude 4.0 or greater centered within 200 miles of the project site over the last 100 years, as reported by USGS earthquake archives.

Table 7.1: Historic Nearby Earthquakes

Approximate Epicenter Location	Date (month/day/year)	Magnitude	Approximate Distance from Project Site (miles)
Waterboro, Maine	10/16/2012	4.7	40
Black Brook, New York	4/20/2002	5.3	180
Peru, Maine	5/29/1983	4.2	27
Sanbornton, New Hampshire	1/19/1982	4.5	92
Altona, New York	6/9/1975	4.2	185
Saint-Augustin-de-Woburn, Quebec	6/15/1973	4.8	187
Scarborough, Maine	4/26/1957	4.4	47
Sangerville, Maine	12/28/1947	4.5	70
Tamworth, New Hampshire	12/24/1940	5.6	66
Dannemora, New York	12/20/1940	5.3	185
Warrensburg, New York	4/15/1934	4.5	193
Ossipee, New Hampshire	4/20/1931	4.7	164
South Paris, Maine	10/9/1925	4.0	23

7.2 Seismic Evaluation

Seismic design of all proposed structures shall be in accordance with the AASHTO LRFD Bridge Design Specifications, AASHTO LRFD Seismic Design Guidelines, and MaineDOT BDG Chapter 5. For determining seismic loads and liquefaction criteria, values for the peak ground acceleration coefficient (PGA) and the short- and long-period spectral acceleration coefficients (S_s and S_1 , respectively) were obtained from the USGS “U.S. Seismic Design Maps” online system. Values obtained are based on a 7% probability of exceedance over a 75-year period (1033-year return period) per 2002 USGS seismic data and 2009 AASHTO design criteria, with the project latitude and longitude in Monmouth, Maine used as the location of interest. These values were compared and found to be consistent with the corresponding values provided in the 1000-year return period seismic response maps in AASHTO LRFD Figures 3.10.2.1-1 through 3.10.2.1-3.

The given base acceleration coefficients correspond to the peak ground acceleration at the top of rock (Site Class B). Given the soft soil, the project site meets the criteria given for Site Class E per AASHTO LRFD Table 3.10.6.1. The recommended design base acceleration coefficients, site factors, and maximum ground acceleration coefficients are presented below in Table 7.2. Based on the Site Response Spectrum, per AASHTO LRFD Section 3.10.6, the proposed bridge is located in Seismic Zone 2. Refer to Appendix 2 for site class determination calculations.

Table 7.2: Seismic Response Spectrum

Acceleration Coefficient	At Top of Rock	Site Factor	At Ground Surface
Peak Ground Acceleration	PGA= 0.083g	$F_{pga}= 2.5$	$A_s= 0.208g$
0.2 Second Period	$S_s= 0.170g$	$F_a= 2.5$	$S_{DA}= 0.424g$
1.0 Second Period	$S_1= 0.046g$	$F_v= 3.5$	$S_{D1}= 0.162g$

7.3 Liquefaction

Liquefaction is a phenomenon whereby a soil substantially loses strength in response to an applied cyclic stress, typically associated with earthquake loading. This temporary loss of soil strength causes the soil to behave like a liquid, impacting bearing capacity and lateral stiffness. Liquefaction induced ground movement can cause serious damage to structures. Per AASHTO LRFD Section 10.5.4.2, a liquefaction assessment shall be performed if a site within Seismic Zone 3 and 4, and for sites in Seismic Zone 2 on sites with loose to very loose saturated sands. Due to the site falling under Seismic Zone 2 with loose alluvium sands, an assessment was performed based on the SPT N-values reported in the borings and the results of the lab testing. The calculations yielded liquefaction factors of safety above 1.0, confirming there is no concern for liquefaction under the design seismic event for this site.

8.0 LIMITATIONS OF REPORT

The conclusions and recommendations contained in this report are based upon the subsurface data obtained during this exploration and on details stated in this report. The validity of the conclusions and

recommendations contained in this report are necessarily limited by, among other things, the scope of field exploration and by the number of borings. Therefore, given the nature of this subsurface study, there is a possibility that actual conditions encountered will differ from those discussed in this report. Should conditions arise which differ from those described in this report, HNTB should be notified immediately and provided with all information when available regarding subsurface conditions.

As part of the geotechnical recommendations presented in this report, HNTB makes no warranty as to the absence or presence of any environmental hazard or waste present on any property evaluated hereunder and all reports generated here to are qualified as being based upon existing data reasonably available to HNTB and not subject to independent verification. HNTB is not responsible for any latent defects that could not be reasonably discovered during the performance of its services and makes no legal representations whatsoever concerning any matter, including but not limited to, the ownership of any property or the interpretation of any law. These limitations form a material part of this report and are considered incorporated by reference therein. No warranty for the contents of this report, neither expressed nor implied, is made except that professional services were performed in accordance with generally accepted principles and practices.

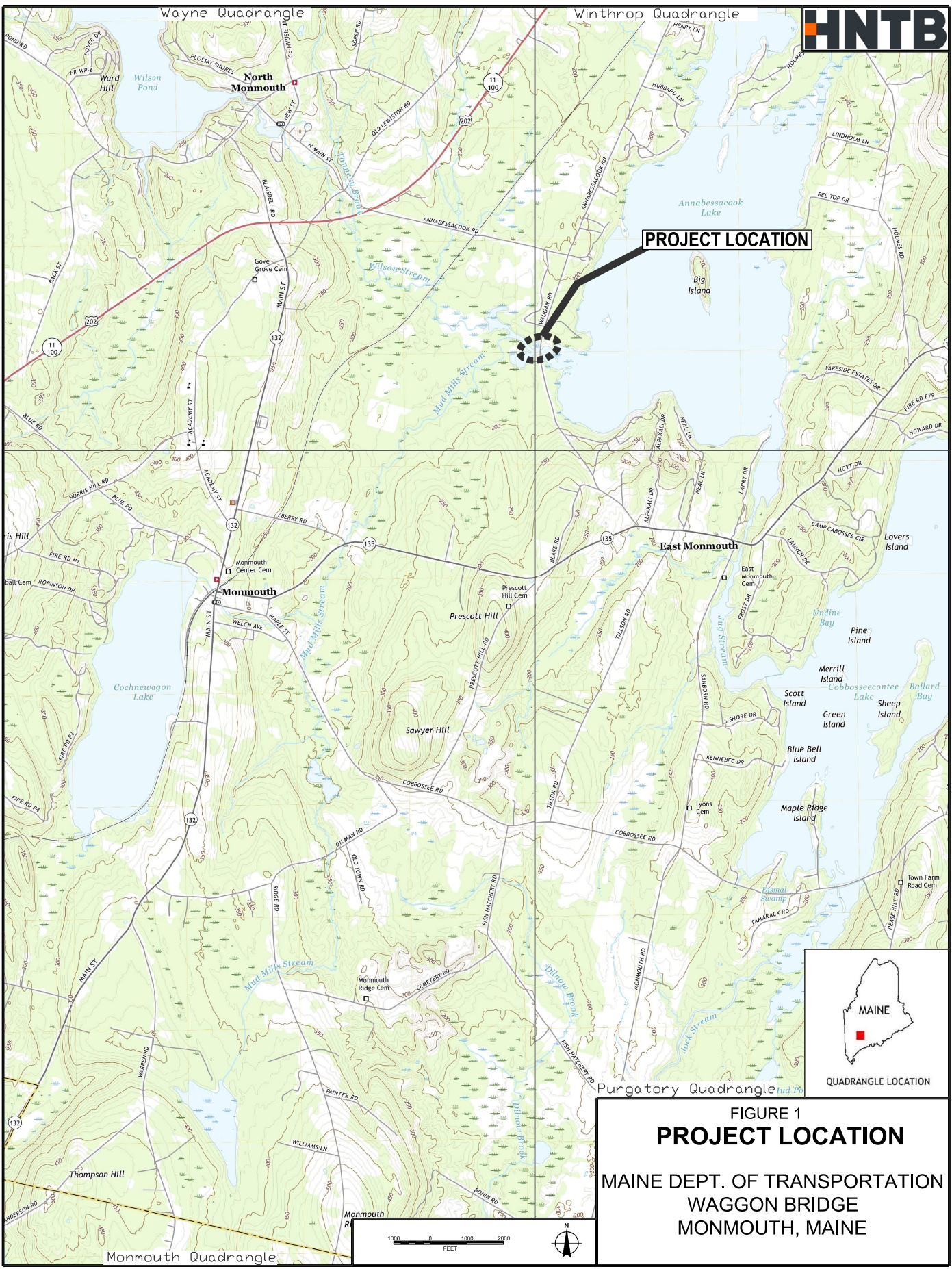
9.0 REFERENCES

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11. United States Geological Survey. “USGS Seismic Design Web Services”. Available online at <https://earthquake.usgs.gov/ws/designmaps/>. Last accessed 11/30/2022.

FIGURE 1

PROJECT LOCATION MAP



PROJECT LOCATION



QUADRANGLE LOCATION

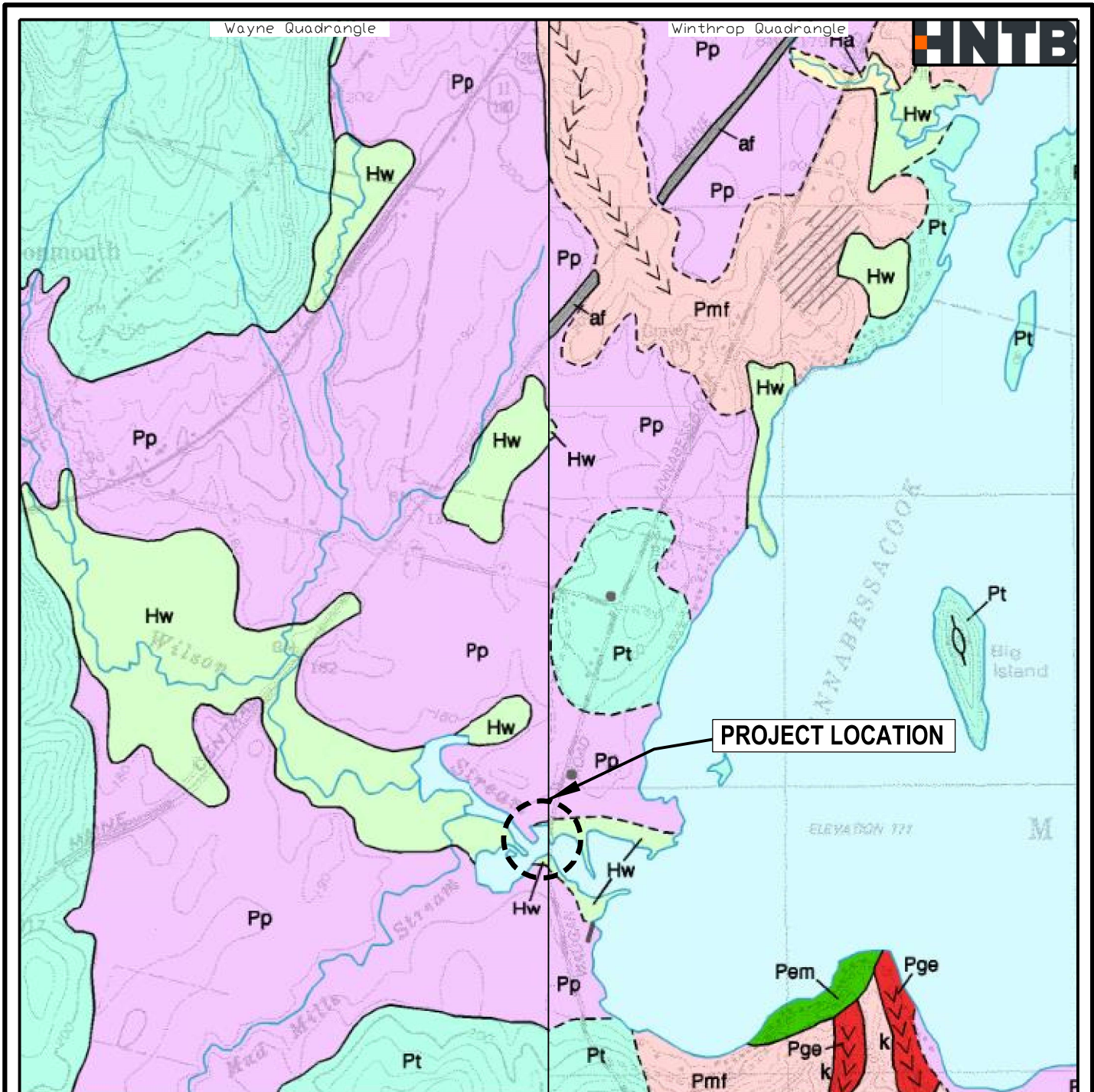
**FIGURE 1
PROJECT LOCATION**

**MAINE DEPT. OF TRANSPORTATION
WAGGON BRIDGE
MONMOUTH, MAINE**

Basemaps: U.S.G.S. Monmouth, Wayne, Winthrop, and Purgatory, ME Quadrangles US Topo, 2021

FIGURE 2

SURFICIAL GEOLOGY MAP



LEGEND

- Ha** Stream alluvium - Sand, silt, gravel, and organic sediment. Deposited on flood plains of streams. Unit includes some wetland areas.
- Hw** Wetland deposits - Muck, peat, silt, and clay in poorly drained areas.
- Pp** Presumpscot Formation - Glaciomarine silt, clay, and sand deposited on the late-glacial sea floor.
- Pmf** Marine-fan deposit - Sand and gravel deposited on the sea floor at the glacier margin during the late-glacial marine submergence.
- Pt** Till - Loose to very compact, poorly sorted, massive to weakly stratified mixture of sand, silt, and gravel-size rock debris deposited by glacial ice. Locally includes lenses of waterlaid sand and gravel.
- Pge** Esker deposits - Ridges of sand and gravel deposited by glacial meltwater streams in subglacial tunnels.
- Pem** End Moraine - Ridge of glacial till deposited at the margin of the late Wisconsinan ice sheet when it stood at the south end of Annabessacook Lake.

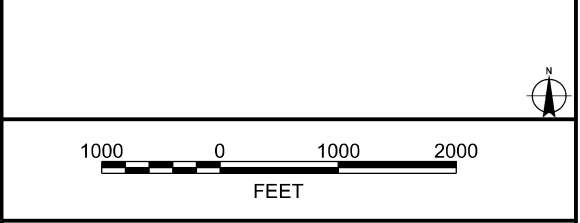
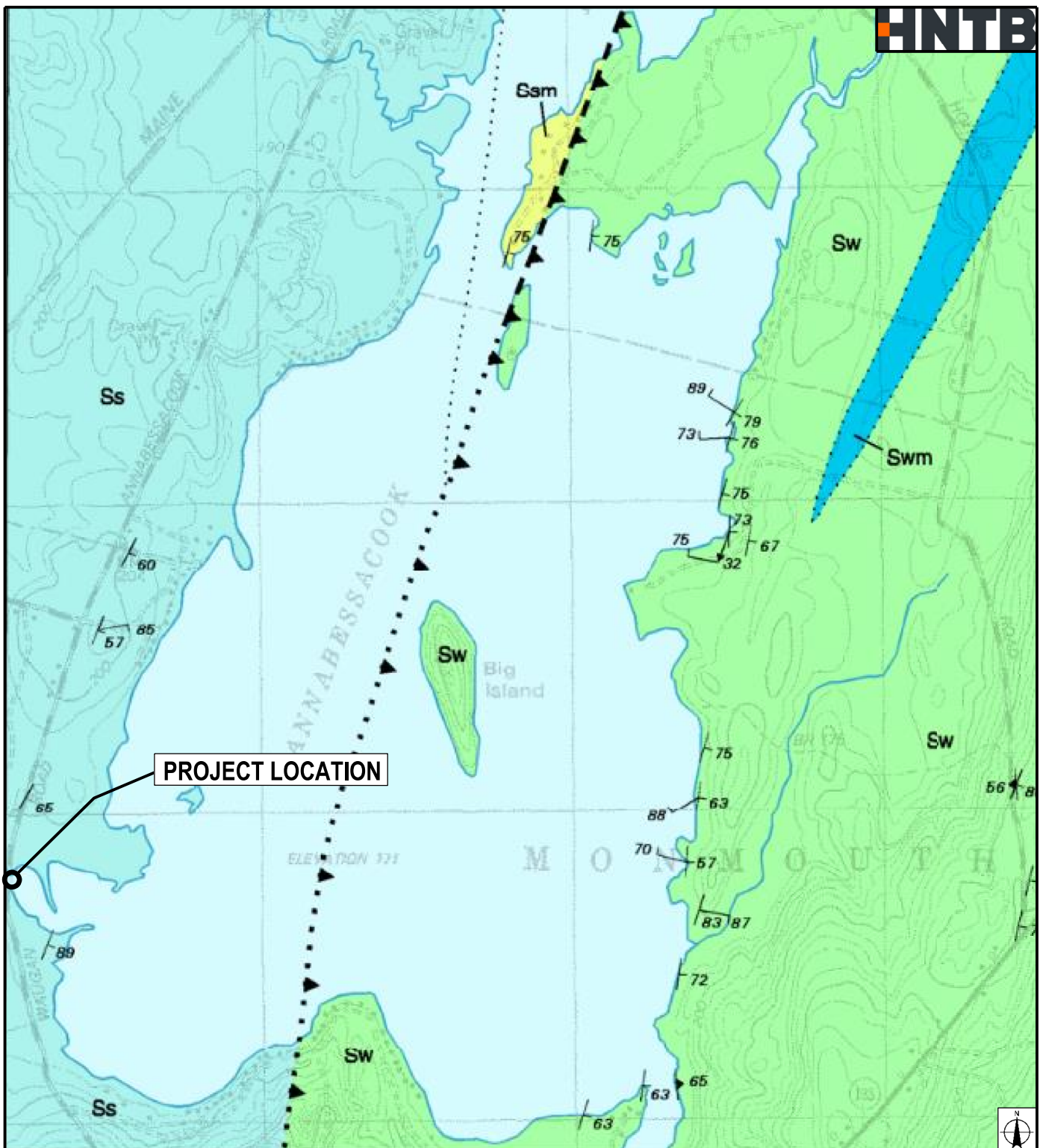


FIGURE 2
SURFICIAL GEOLOGY MAP
 MAINE DEPT. OF TRANSPORTATION
 WAGGON BRIDGE
 MONMOUTH, MAINE

FIGURE 3

BEDROCK GEOLOGY MAP



LEGEND



Ss Sangerville Formation - Medium-grained biotite-muscovite-quartz-feldspar schist and light gray biotite-muscovite-quartz-feldspar granofels or greenish-gray calc-silicate granofels. Beds typically from 2-20 cm, although some beds exceed 1 meter in thickness. Calc-silicate lenses range from a few centimeters to one meter in length and up to 10 cm in width and contain green calc-silicate minerals and often orange grossular garnet.



Sw Waterville Formation - Thinly bedded, medium-grained biotite-quartz-feldspar granofels commonly interlayered on a scale of 2-10 cm with medium-grained, green and white calc-silicate rock. Muscovite-biotite schists and aluminosilicate-bearing schists are common, although these rock types are subordinate to the granofels and calc-silicate rock. Quartz laminae of 1-2 mm thickness are common.



Ssm Marble and calc-silicate rock - Medium-grained, dark gray to black marble with variable amounts of calc-silicate minerals, interlayered on a scale of 2-10 cm with medium-grained, green and white calc-silicate layers that commonly contain calcite, and medium-grained biotite-quartz-feldspar granofels.



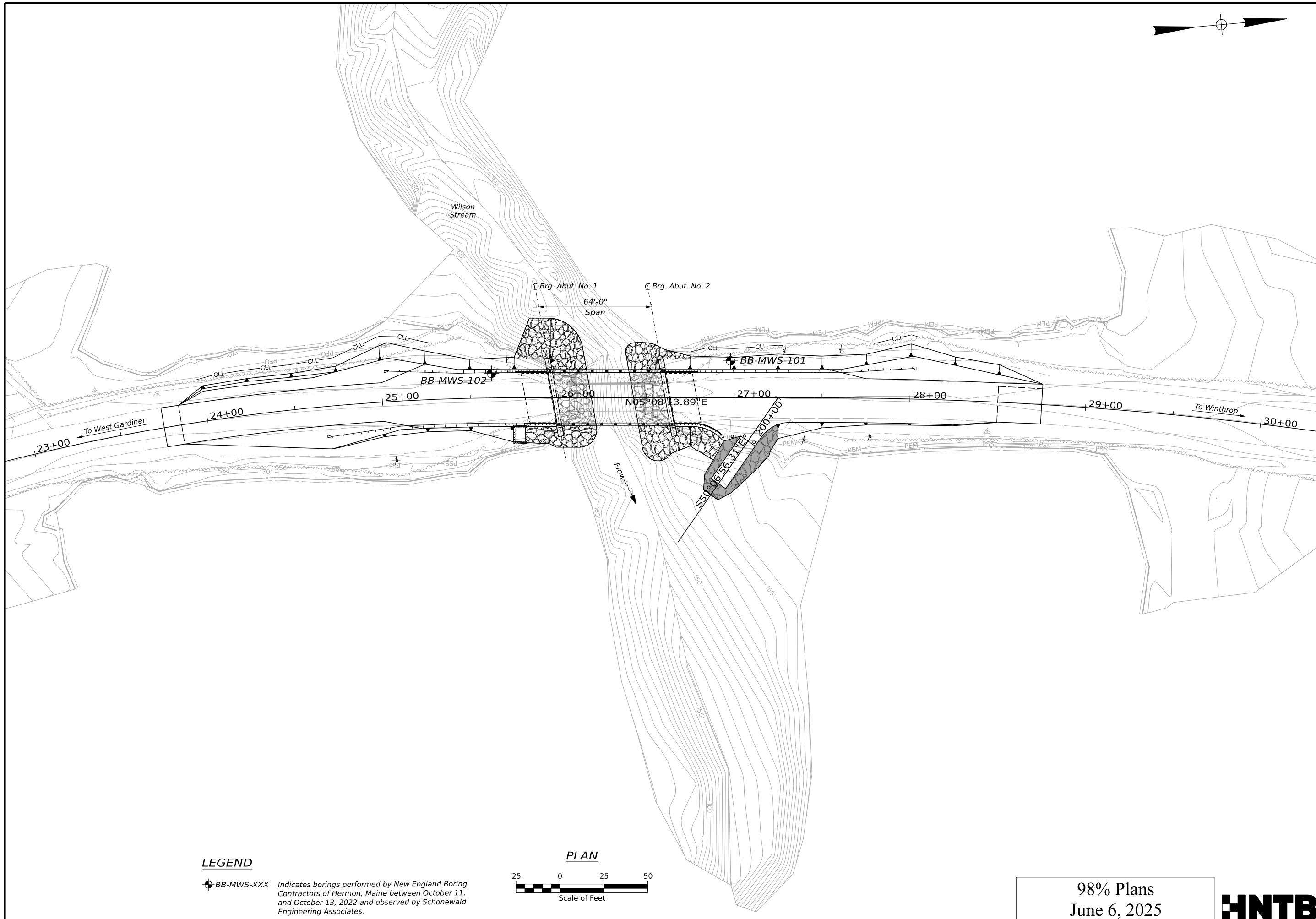
Swm Marble and calc-silicate rock - Medium-grained, dark gray to black marble with variable amounts of calc-silicate minerals, interlayered on a scale of 2-10 cm with medium-grained, green and white calc-silicate layers that commonly contain calcite, and medium-grained, biotite-quartz-feldspar granofels.



FIGURE 3
BEDROCK GEOLOGY MAP
 MAINE DEPT. OF TRANSPORTATION
 WAGGON BRIDGE
 MONMOUTH, MAINE

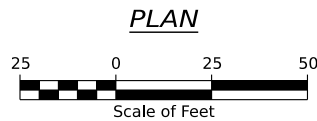
FIGURE 4

BORING LOCATION PLAN



LEGEND

BB-MWS-XXX Indicates borings performed by New England Boring Contractors of Hermon, Maine between October 11, and October 13, 2022 and observed by Schonewald Engineering Associates.



98% Plans
June 6, 2025



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2623400
WIN 026234.00
BRIDGE NO. 0487 BRIDGE PLANS

PROJ. MANAGER	BY	DATE	SIGNATURE
Trevor Gibson	C. Tabin	06/2025	
DESIGNED-DETAILED	M. Rowicki	06/2025	
CHECKED-REVIEWED	J. Jaquin	06/2025	
DESIGNED-DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

**MONMOUTH
WAGGON BRIDGE**

BORING LOCATION PLAN

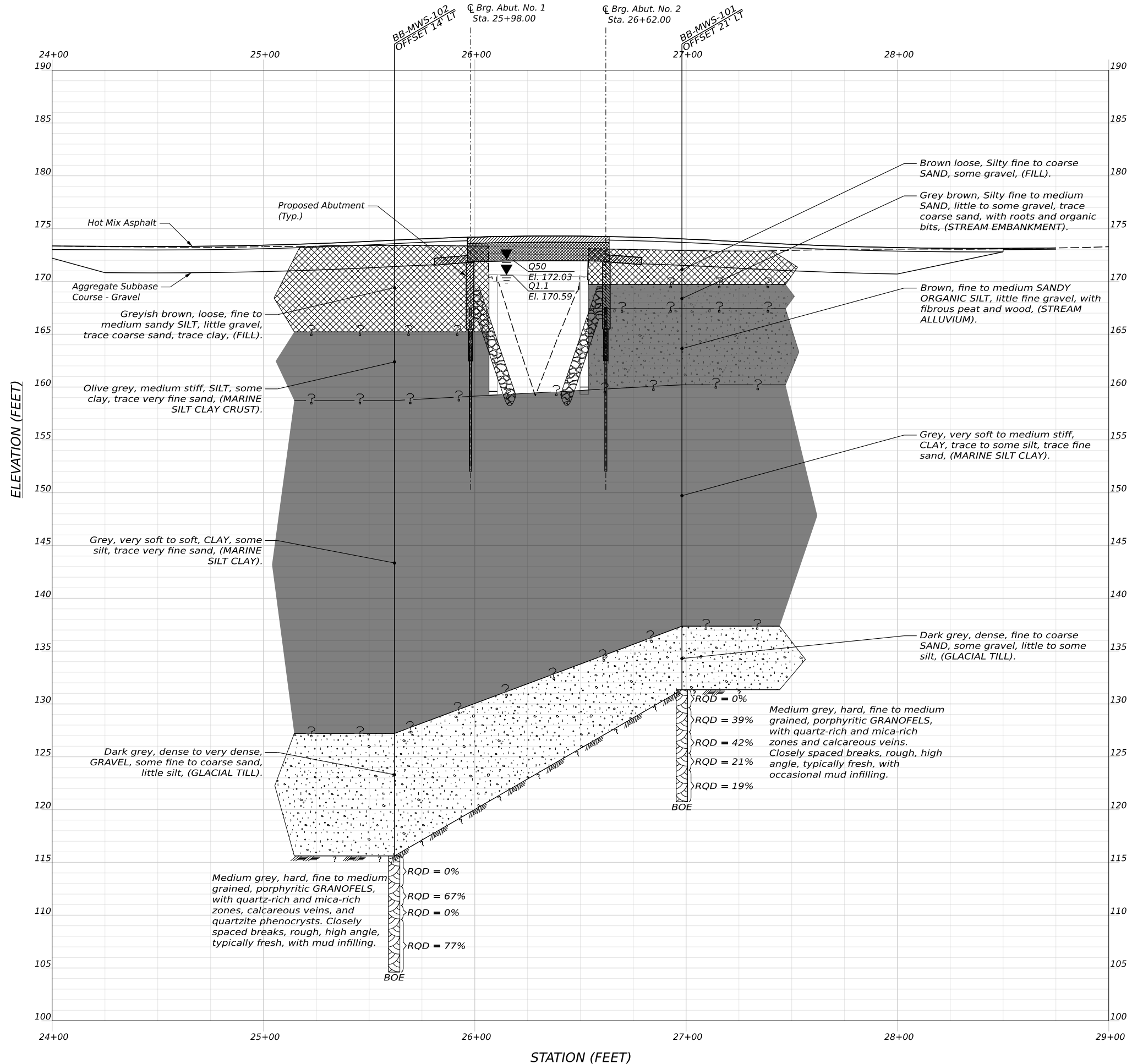
SHEET NUMBER

6

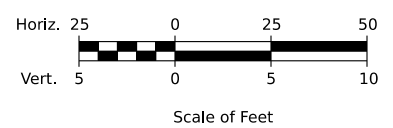
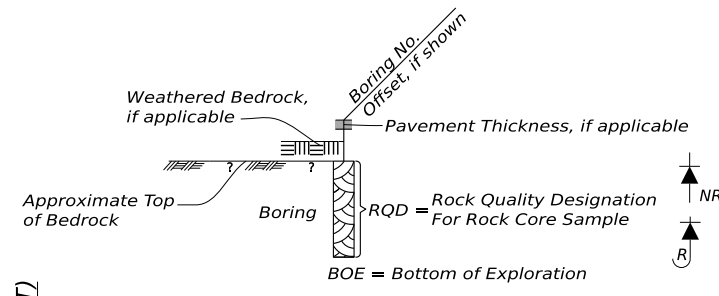
OF 28

FIGURE 5

SUBSURFACE PROFILE



LEGEND



NOTES:

1. This Generalized Interpretive Soil Profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.
2. Existing abutment linework shown screened for clarity.

STATE OF MAINE DEPARTMENT OF TRANSPORTATION 2623400 BRIDGE NO. 0487 WIN 026234.00 BRIDGE PLANS
MONMOUTH WAGGON BRIDGE INTERPRETIVE SUBSURFACE PROFILE
SHEET NUMBER 7 OF 28

98% Plans
June 6, 2025



APPENDIX 1

GEOTECHNICAL DATA REPORT



**GEOTECHNICAL DATA REPORT
WAGGON BRIDGE #0487 REPLACEMENT
WAUGAN ROAD OVER WILSON STREAM
MONMOUTH, MAINE
MAINEDOT WIN 26234.00**

CORRECTED

PREPARED FOR:

HNTB Corporation
South Portland, Maine

PREPARED BY:

Schonewald Engineering Associates, Inc. (SchonewaldEA)
Cumberland, Maine 04021

**July 23, 2023 – UPDATED to include MaineDOT Key to Soil Descriptions (Attachments Pg 4) and
Station and Offset to Boring Logs (Attachments Pgs 5 through 10)
December 2022**

SchonewaldEA Project No. 22-016

VIA EMAIL

December 28, 2022
Project No. 22-016 (Waggon)

Mr. Joe Juzwin, P.E.
Mr. Josh Olund, P.E.
HNTB Corporation
82 Running Hill Road, Suite 201
South Portland, ME 04106

Re: Geotechnical Data Report
Waggon Bridge #0487 Replacement
Waugan Road over Wilson Stream
Monmouth, Maine
MaineDOT WIN 26234.00

Dear Joe and Josh:

Schonewald Engineering Associates, Inc. (SchonewaldEA) has prepared this geotechnical data report to transmit field- and laboratory-generated data to support HNTB's design work associated with the replacement of Waggon Bridge that carries Waugan Road over Wilson Stream in Monmouth, Maine (MaineDOT WIN 26234.00). SchonewaldEA's work on this project has been completed under Subconsultant Task Order Agreement No. 586.01 with HNTB that is dated September 9, 2022.

SchonewaldEA completed the following items to support HNTB's design effort:

- Retained New England Boring Contractors (NEBC) to provide drilling and traffic control services, and completed two (2) test borings;
- Coordinated the above field effort, including site reconnaissance; utility clearance; retaining the drilling subcontractor; and coordinating field logistics;
- Observed and logged the test borings on a full-time basis;
- Retained and coordinated a qualified geotechnical laboratory to complete a soils testing program appropriate for the subsurface conditions encountered and proposed design of the replacement bridge;
- Provided draft boring logs and laboratory test results as the project progressed;
- Prepared a boring location plan, boring logs, and photo sheets of the bedrock core;
- Prepared this data report to transmit the boring location plan, boring logs, bedrock core photo sheets, and the laboratory test report; and
- Delivered the remaining soil samples and rock core obtained in the borings to HNTB.

SUBSURFACE EXPLORATION PROGRAM

SchonewaldEA retained New England Boring Contractors of Hermon, Maine to provide drilling and traffic control services for this project. Two test borings were drilled at the bridge site to evaluate subsurface conditions. The test borings were designated BB-MWS-101 and -102 and were located at the two corners of the existing bridge that were accessible, allowed the drill rig and crew to be off the roadway proper, and that did not block the stop signs from roadway traffic. The test borings were drilled between October 11 and 13, 2022 and were observed and logged by SchonewaldEA.

The approximate locations of the test borings are depicted on the Boring Location Plan that is included as Attachment 1. The base plan for the Boring Location Plan was taken from a plan entitled "Waugan Bridge, Wilson Stream, Monmouth, Kennebec [County], General Plan," prepared by HNTB, progress print dated November 2022; original scale 1 inch = 50 feet. The borings were located in the field by taping

from prominent site features depicted on the survey base plan; the locations should be considered only as accurate as the method implies. The ground surface elevation at each borehole that is noted on the boring log was approximated from the ground surface topography depicted on the base plan.

The test borings were advanced using standard cased wash boring techniques. Each boring was extended through overburden to refusal and 10 feet of NQ2 (N-size, double-barrel core barrel) bedrock core was obtained. The logs of the test borings are included as Attachment 2. Photographs of the bedrock core obtained in the test borings are included as Attachment 3.

Standard Penetration Tests (SPTs) were completed and split-spoon soil samples were obtained near the ground surface and then typically at five-foot intervals to the bottom of each boring. SPTs were performed using an auto hammer that had been calibrated in general conformance with MaineDOT policy. The hammer efficiency factor is recorded on the boring logs. In lieu of SPT'ing / split-spoon sampling, vane shear tests were completed in soft marine silt-clay soils where encountered in the test borings. Vane shear tests were completed using MaineDOT's Geonor vane (65 mm by 130 mm) and the associated soil shear strength was determined using a vane constant developed by MaineDOT. Vane shear testing was completed in accordance with MaineDOT's standard field procedures that conform to the ASTM standard. Thin-walled undisturbed tube samples of the soft marine silt-clay were obtained using a fixed-piston sampler and were obtained, packed/ sealed, and handled/ transported in accordance with generally accepted best field practices.

Groundwater levels observed within the borings are noted on the boring logs, along with the conditions (e.g., stabilization times) under which the groundwater measurements were obtained. The boreholes were backfilled using drill cuttings, supplemented by manufactured sand and gravel.

GEOTECHNICAL LABORATORY TESTING PROGRAM

Representative soil samples obtained in the test borings were submitted to the GeoTesting Express (GTX) geotechnical laboratory in Acton, Massachusetts for a soil testing program to support the design for the new structure. The actual testing program was developed by HNTB's geotechnical group based on draft logs of the test borings provided by SchonewaldEA. The laboratory testing program is summarized in the following table.

Boring No.	Sample No.	Sample Depth (ft, BGS)	Tests Performed
UNDISTURBED TUBE SAMPLES			
BB-MWS-101	U1	24-26	tube opening, Atterberg limits, moisture content, % -#200, consolidation (CRS)
BB-MWS102	U1	19-21	tube opening, Atterberg limits, moisture content, % -#200, consolidation (CRS), 3-point CIU shear strength
BB-MWS-102	U2	29-31	tube opening, Atterberg limits, moisture content, % -#200, consolidation (CRS), 3-point CIU shear strength
SPLIT-SPOON JAR SAMPLES			
BB-MWS-101	2D	4-6	organic content, AASHTO corrosion series (see Note 1)
BB-MWS-101	3D-A	10.5-11	grain size, organic content
BB-MWS-101	4D	14-16	Atterberg limits, moisture content, %-#200
BB-MWS-101	5D	19-21	Atterberg limits, moisture content
BB-MWS-101	6D	29-31	Atterberg limits, moisture content

Boring No.	Sample No.	Sample Depth (ft, BGS)	Tests Performed
SPLIT-SPOON JAR SAMPLES (cont'd)			
BB-MWS-102	1D	2-4	Atterberg limits
BB-MWS-102	3D	9-11	Atterberg limits, moisture content
BB-MWS-102	4D	14.5-16.5	Atterberg limits, moisture content
BB-MWS-102	5D	24-26	Atterberg limits, moisture content
BB-MWS-102	6D	34-36	Atterberg limits, moisture content
BB-MWS-102	7D	39-41	Atterberg limits, moisture content, %-#200
BB-MWS-102	10D	54-56	grain size

Note 1: Corrosion series was performed on a composite sample of BB-MWS-101 2D (4-6 ft) and 3D (9-11 ft). There was insufficient sample volume to complete the resistivity test by AASHTO Method T288, therefore, SchonewaldEA directed GTX to proceed with resistivity test by ASTM Method G57; pH, sulfates, and chlorides were determined by AASHTO method.

Laboratory test results are summarized on the test boring logs included as Attachment 2. The laboratory test report that includes methodology/ test standards is included as Attachment 4.

SchonewaldEA appreciates the opportunity to be of service to HNTB and MaineDOT. If you have any questions regarding the work completed by SchonewaldEA or the attached data, please call/reply at your convenience.

Sincerely,
Schonewald Engineering Associates, Inc.



Isabel V. (Be) Schonewald, P.E.
President

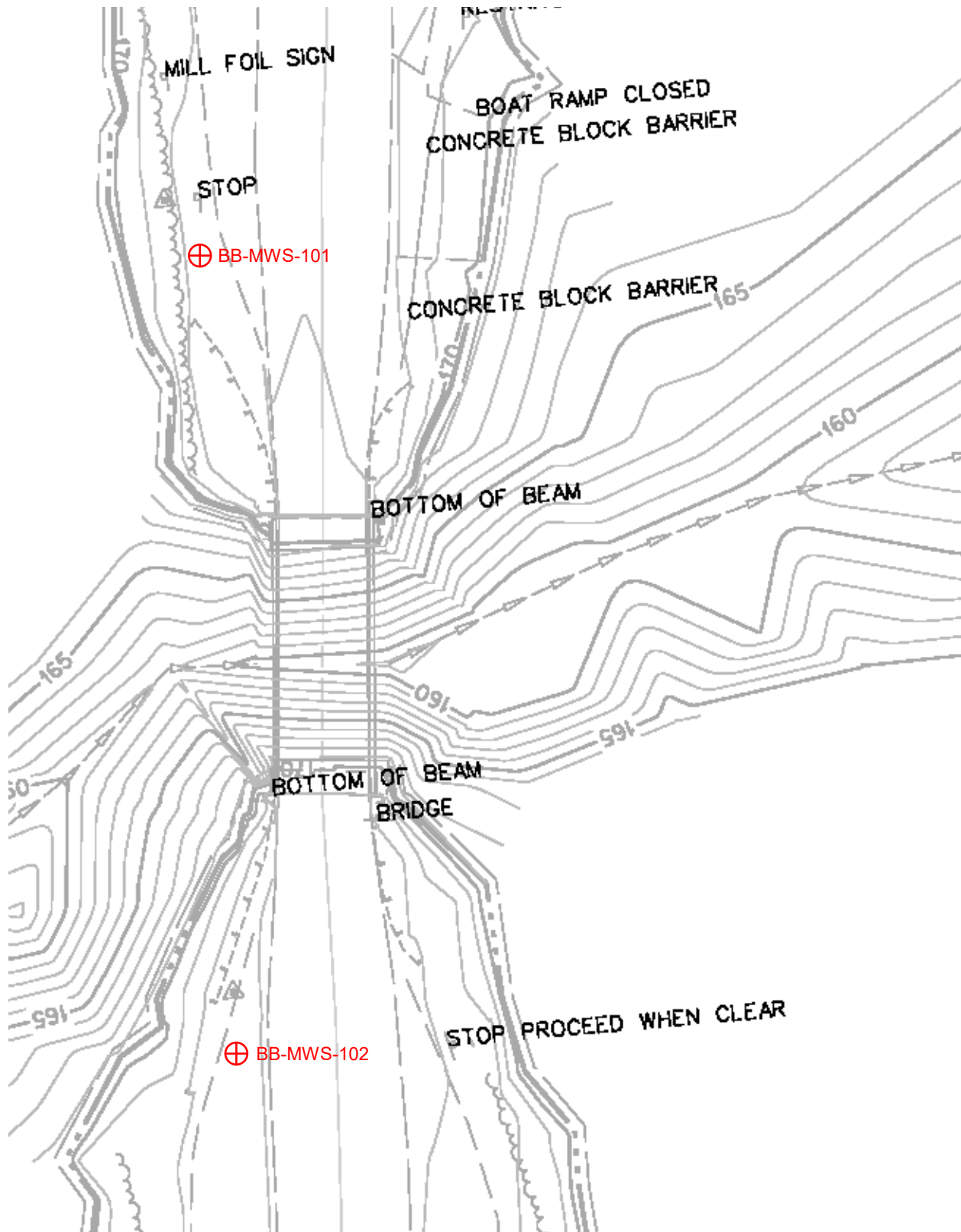
Attachments

**GEOTECHNICAL DATA REPORT
WAGGON BRIDGE #0487 REPLACEMENT
WAUGAN ROAD OVER WILSON STREAM
MONMOUTH, MAINE
MAINEDOT WIN 26234.00**

ATTACHMENTS

DESCRIPTION	ATTACHMENT PAGES
ATTACHMENT 1 - BORING LOCATION PLAN	2
ATTACHMENT 2 - MAINEDOT KEY TO SOIL DESCRIPTIONS & BORING LOGS	4-10
ATTACHMENT 3 - BEDROCK CORE PHOTOGRAPHS	12-13
ATTACHMENT 4 - RESULTS OF LABORATORY TESTS ON SOIL SAMPLES	15-50

ATTACHMENT 1
BORING LOCATION PLAN



SCHONEWALD
ENGINEERING
ASSOCIATES, INC.

PROJECT NO.: 22-016
DATE: DEC 2022
DRAWN BY: IVS
APPROX. SCALE: 1" = 25'

**BORING LOCATION PLAN
WAGGON BRIDGE #0487
WAUGAN ROAD OVER WILSON STREAM
MONMOUTH, MAINE
MAINEDOT WIN 26234.00**

BASE PLAN TAKEN FROM PLAN ENTITLED "WAUGAN BRIDGE, WILSON STREAM, MONMOUTH, KENNEBEC, GENERAL PLAN," PREPARED BY HNTB, DATED 11/22, ORIGINAL SCALE 1" = 50'



Figure No.:

1

ATTACHMENT 2
MAINEDOT KEY TO SOIL DESCRIPTIONS
&
BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER 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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	
		(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.	
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, Gravelly sands, little or no fines	
		(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	
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):				Desired Rock 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				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	

Driller: New England Boring Contractors	Elevation (ft.): 172	Auger ID/OD: n/a
Operator: Enos/ Gomm	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: Schonewald	Rig Type: Mobile Drill B-53 track (NEBC-23)	Hammer Wt./Fall: 140 lbs/ 30 in
Date Start/Finish: 10/11/22; 1515-10/12/22; 1120	Drilling Method: cased wash boring	Core Barrel: NQ2
Boring Location: Station 26+98, 21 ft LT	Casing ID/OD: HW(4.0/4.5) 14'/NW(3.0/3.5) 40.5'	Water Level*: 3.4 ft (open, end)

Hammer Efficiency Factor: 0.859 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plasticity Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0												
	1D	24/19	2.0 - 4.0	3-1-2-2	3	4			169.0		sand and gravel (shoulder) Brown, moist, Silty fine to coarse SAND, some gravel, (MISC FILL). Grading at 3.0 ft to: -----3.0	
5	2D	24/18	4.0 - 6.0	2-2-1/12"	<3				166.7		1D: Dark grey brown, moist, Silty fine to medium SAND, little to some gravel, trace coarse sand, with roots and organic bits, (ORIGINAL STREAM EMBANKMENT). Dark grey brown, moist, Silty fine to medium SAND, little to some gravel, trace coarse sand, with roots and organic bits, (ORIGINAL STREAM EMBANKMENT). Changing at 5.3 ft to: -----5.3	GTX#693270 ORG CONTENT 5.8% NO GTX# CORROSION SERIES (COMPOSITE W 3D)
	MV										2D: Dark brown, saturated, fine to medium Sandy ORGANIC SILT, little fine gravel with fibrous peat and wood, (STREAM ALLUVIUM). Grey, fine Sandy SILT in tip of spoon.	REFER TO 2D
10	3D	24/20	9.0 - 11.0	WOH-3-2/12"	<5						MV: Unable to push vane at 9.0 ft. 3D: Grey, mottled, SILT, some fine sand, (STREAM ALLUVIUM). Changing at 10.5 ft to: -----10.5	
	3D-A										3D-A: Brown, ORGANIC SILT, trace to little fine SAND, with fibrous peat and wood, (STREAM ALLUVIUM).	GTX#693270 ORG CONTENT 11.3% GTX#693267 G=A-4(0)
15	4D	24/22	14.0 - 16.0	PUSH THRU VANE	--						Grey, medium stiff, Silty CLAY, trace very fine sand, (MARINE SILT-CLAY). V1: 19.5 / 2.5 ft-lbs (65 mm x 130 mm vane raw torque readings) V2: 19.5 / 2 ft-lbs (65 mm x 130 mm vane raw torque readings)	GTX#693246 WC=48.5% GTX#693237 LL=51 PL=31 PI=20 GTX#693259 %#200=98.4
	V1		14.6 - 15.0	Su=536 /69 psf								
	V2		15.6 - 16.0	Su=536 /55 psf								
20	5D	24/24	19.0 - 21.0	PUSH THRU VANE	--						Dark grey with black, soft, CLAY, some silt, (MARINE SILT-CLAY). V3: 13.5 / 1.5 ft-lbs (65 mm x 130 mm vane raw torque readings) V4: 11 / 1 ft-lbs (65 mm x 130 mm vane raw torque readings)	GTX#693246 WC=44.2% GTX#693238 LL=41 PL=24 PI=17
	V3		19.6 - 20.0	Su=371 /41 psf								
	V4		20.6 - 21.0	Su=302 /27 psf								
25	U1	24/22	24.0 - 26.0	PISTON SAMPLER	--						Grey, CLAY, some silt, (MARINE SILT-CLAY).	GTX#693246 WC=39.9% GTX#693234

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Waggon Bridge #0487 Waugan Rd over Wilson Stream Location: Monmouth, ME	Boring No.: BB-MWS-101 WIN: 26234.00
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Driller: New England Boring Contractors	Elevation (ft.): 172	Auger ID/OD: n/a
Operator: Enos/ Gomm	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: Schonewald	Rig Type: Mobile Drill B-53 track (NEBC-23)	Hammer Wt./Fall: 140 lbs/ 30 in
Date Start/Finish: 10/11/22; 1515-10/12/22; 1120	Drilling Method: cased wash boring	Core Barrel: NQ2
Boring Location: Station 26+98, 21 ft LT	Casing ID/OD: HW(4.0/4.5) 14"/NW(3.0/3.5) 40.5'	Water Level*: 3.4 ft (open, end)

Hammer Efficiency Factor: 0.859	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25												LL=30 PL=20 PI=10 GTX#693259 % #200=98.9 GTX#CRC-3 CRS CONSOL
30	6D V5 V6	24/12	29.0 - 31.0 29.6 - 30.0 30.6 - 31.0	PUSH THRU VANE Su=220 /27 psf Su=220 /27 psf	--						Grey, very soft, CLAY, some silt, (MARINE SILT-CLAY). V5: 8 / 1 ft-lbs (65 mm x 130 mm vane raw torque readings) V6: 8 / 1 ft-lbs (65 mm x 130 mm vane raw torque readings)	GTX#693246 WC=40.1% GTX#693239 LL=38 PL=20 PI=18
35	MV 7D	24/12	34.0 - 36.0	WOR-WOH-4-5	--				137.4		MV: Unable to push vane at 34.0 ft. Grey, CLAY, some silt, (MARINE SILT-CLAY) (TRANSITION). Silty fine SAND in bottom of sample.	
40	8D R1 R2	14/11 17/12 28/28	39.0 - 40.2 40.5 - 41.9 41.9 - 44.2	17-63-50/2" RQD = 0% RQD = 39%	>113				131.8		Dark grey, fine to coarse SAND, some gravel, little to some silt, (GLACIAL TILL). Top of bedrock at Elev. 131.8 ft. R1: Bedrock: Medium grey, fine to medium grained, porphyritic GRANOFELS, with quartz-rich and mica-rich zones and calcareous veins; high angle and folded remnant bedding visible; hard, typically fresh. Typically high angle with lesser low angle, closely spaced breaks; undulating, rough, typically fresh to slightly discolored, and open, with occasional mud infilling. R1 highly broken. (SANGERVILLE FORMATION) Core times: 2:10/ -- min:sec/ft. ROCK QUALITY = VERY POOR R2: Similar to R1, except not highly broken. Core times: --/ 1:35/ -- min:sec/ft. ROCK QUALITY = POOR R3: Similar to R2. Core times: --/ 1:45/ -- min:sec/ft. ROCK QUALITY = POOR R4: Similar to R2. Core times: 1:55/ -- min:sec/ft. ROCK QUALITY = VERY POOR R5: Similar to R2, except bottom 1.3 ft highly broken. Core times: --/ 1:20/ 1:35/ -- min:sec/ft.	GTX#693268 G=A-1-b(0)
45	R3 R4 R5	24/20 19/17 36/35	44.2 - 46.2 46.2 - 47.8 47.8 - 50.8	RQD = 42% RQD = 21% RQD = 19%								
50												

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Waggon Bridge #0487 Waugan Rd over Wilson Stream Location: Monmouth, ME	Boring No.: BB-MWS-101 WIN: 26234.00
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Driller: New England Boring Contractors	Elevation (ft.): 172	Auger ID/OD: n/a
Operator: Enos/ Gomm	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: Schonewald	Rig Type: Mobile Drill B-53 track (NEBC-23)	Hammer Wt./Fall: 140 lbs/ 30 in
Date Start/Finish: 10/11/22; 1515-10/12/22; 1120	Drilling Method: cased wash boring	Core Barrel: NQ2
Boring Location: Station 26+98, 21 ft LT	Casing ID/OD: HW(4.0/4.5) 14'/NW(3.0/3.5) 40.5'	Water Level*: 3.4 ft (open, end)

Hammer Efficiency Factor: 0.859 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) W_c = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf)
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140 lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50									121.2		ROCK QUALITY = VERY POOR	
											Bottom of Exploration at 50.8 feet below ground surface.	
55												
60												
65												
70												
75												

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Waggon Bridge #0487 Waugan Rd over Wilson Stream Location: Monmouth, ME	Boring No.: <u>BB-MWS-102</u> WIN: <u>26234.00</u>
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Driller: New England Boring Contractors	Elevation (ft.): 172.5	Auger ID/OD: n/a
Operator: Enos/ Gomm	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: Schonewald	Rig Type: Mobile Drill B-53 track (NEBC-23)	Hammer Wt./Fall: 140 lbs/ 30 in
Date Start/Finish: 10/12/22; 1145-10/13/22; 1130	Drilling Method: cased wash boring	Core Barrel: NQ2
Boring Location: Station 25+62, 14 ft LT	Casing ID/OD: HW(4.0/4.5) 44.5'/NW(3.0/3.5) 57.6'	Water Level*: 6.0 ft (open, end)

Hammer Efficiency Factor: 0.859 Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions:
D = Split Spoon Sample R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample Attempt SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
U = Thin Wall Tube Sample HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
V = Field Vane Shear Test, PP = Pocket Penetrometer WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
MV = Unsuccessful Field Vane Shear Test Attempt WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0												
	1D	24/14	2.0 - 4.0	8-3-4-3	7	10						
	2D	24/6	4.0 - 6.0	8-3-4-4	7	10						
5												
	MV								164.5			
	3D	24/7	9.0 - 11.0	3-3-2-2	5	7						
10												
	MV 4D	24/21	14.5 - 16.5	1/12"-1-1	--				158.0			
15												
	U1	24/24	19.0 - 21.0	PISTON SAMPLER								
20												
	5D	24/24	24.0 - 26.0	PUSH THRU VANE								
25												

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Waggon Bridge #0487 Waugan Rd over Wilson Stream Location: Monmouth, ME	Boring No.: BB-MWS-102 WIN: 26234.00
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Driller: New England Boring Contractors Operator: Enos/ Gomm Logged By: Schonewald Date Start/Finish: 10/12/22; 1145-10/13/22; 1130 Boring Location: Station 25+62, 14 ft LT	Elevation (ft.): 172.5 Datum: NAVD88 Rig Type: Mobile Drill B-53 track (NEBC-23) Drilling Method: cased wash boring Casing ID/OD: HW(4.0/4.5) 44.5'/NW(3.0/3.5) 57.6'	Auger ID/OD: n/a Sampler: Standard Split-Spoon Hammer Wt./Fall: 140 lbs/ 30 in Core Barrel: NQ2 Water Level*: 6.0 ft (open, end)
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Hammer Efficiency Factor: 0.859 <small>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</small>	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/> <small>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</small>	<small>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 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected</small>
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Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	V1 V2		24.6 - 25.0 25.6 - 26.0	Su=316 /27 psf Su=220 /41 psf							V2: 8 / 1.5 ft-lbs (65 mm x 130 mm vane raw torque readings)	LL=31 PL=21 PI=10
30	U2	24/24	29.0 - 31.0	PISTON SAMPLER							Grey, CLAY, some silt, (MARINE SILT-CLAY).	GTX#693248 WC=37.7% GTX#693236 LL=31 PL=21 PI=10 GTX#693259 % #200=99.7 GTX#-- 3-pt CIU GTX#CRC-1 CRS CONSOL
35	6D V3 V4	24/18	34.0 - 36.0 34.6 - 35.0 35.6 - 36.0	PUSH THRU VANE Su=275 /27 psf Su=343 /41 psf							Dark grey with black, soft, CLAY, some silt, (MARINE SILT-CLAY). V3: 10 / 1 ft-lbs (65 mm x 130 mm vane raw torque readings) V4: 12.5 / 1.5 ft-lbs (65 mm x 130 mm vane raw torque readings)	GTX#693248 WC=46.9% GTX#693244 LL=44 PL=23 PI=21
40	7D V5 MV	24/24	39.0 - 41.0 39.6 - 40.0	VANE/12"-WOR/12" Su=137* /69 psf							Grey, CLAY, some silt, (MARINE SILT-CLAY). V5: 5 / 2.5 ft-lbs (65 mm x 130 mm vane raw torque readings) *gritty - possible sand lens/layer Unable to push vane at 40.0 ft.	GTX#693248 WC=40.5% GTX#693245 LL=36 PL=22 PI=14 GTX#693259 % #200=99.8
45	8D	24/5	44.0 - 46.0	3-9-8-7	17	24	NW				Unable to push vane at 44.0 ft. Dark grey, medium dense, GRAVEL, little fine to coarse sand, little silt, trace clay; possible transition	
									126.5		-----	46.0
50	9D	24/8	49.0 - 51.0	17-18-13-11	31	44	--				Dark grey, dense, GRAVEL, some fine to coarse sand, little silt, (GLACIAL TILL).	

Remarks:

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> US CUSTOMARY UNITS				Project: Waggon Bridge #0487 Waugan Rd over Wilson Stream Location: Monmouth, ME		Boring No.: <u>BB-MWS-102</u>					
Driller: New England Boring Contractors		Elevation (ft.): 172.5		Auger ID/OD: n/a							
Operator: Enos/ Gomm		Datum: NAVD88		Sampler: Standard Split-Spoon							
Logged By: Schonewald		Rig Type: Mobile Drill B-53 track (NEBC-23)		Hammer Wt./Fall: 140 lbs/ 30 in							
Date Start/Finish: 10/12/22; 1145-10/13/22; 1130		Drilling Method: cased wash boring		Core Barrel: NQ2							
Boring Location: Station 25+62, 14 ft LT		Casing ID/OD: HW(4.0/4.5) 44.5'/NW(3.0/3.5) 57.6'		Water Level*: 6.0 ft (open, end)							
Hammer Efficiency Factor: 0.859		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>									
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 Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
50						51	114.9		<p>Dark grey, very dense, Silty GRAVEL, some fine to coarse sand, (GLACIAL TILL).</p> <p>Top of bedrock at Elev. 114.9 ft. R1: Bedrock: Medium grey, fine to medium grained, porphyritic GRANOFELS, with occasional quartz-rich and mica-rich zones, calcareous veins, and quartzite phenocrysts; high angle remnant bedding visible; hard, typically fresh. Highly broken with evidence of wide mud-infilled fractures. (SANGERVILLE FORMATION) Core times: 2:15/ 1:55/ -- min:sec/ft. ROCK QUALITY = VERY POOR R2: Similar rock as R1. Typically moderately dipping, closely to moderately spaced breaks; undulating, rough, typically fresh, and open with mud infilling. Core times: 1:40/ -- min:sec/ft. ROCK QUALITY = FAIR R3: Similar rock as R1. Highly broken; one long, very wide, mud-infilled, near- vertical fracture. Core times: --/ 2:10/ -- min:sec/ft ROCK QUALITY = VERY POOR R4: Similar rock as R1 with garnets in lower 3 feet of core. Typically moderately dipping, moderately spaced breaks; undulating, rough, typically fresh, and open with mud infilling. Very wide, mud-infilled fracture from 65.6 to 65.7 ft. Core times: --/ 1:50/ 2:05/ 2: 30/ 2:10/ -- min:sec/ft. ROCK QUALITY = GOOD</p> <p>Bottom of Exploration at 68.6 feet below ground surface.</p>	<p>GTX#693269 G=A-4(0)</p>	
55	10D	24/6	54.0 - 56.0	16-17-22-33	39	56	38				
							66				
							64				
							101				
	R1	33/18	57.7 - 60.5	RQD = 0%			--				
							62				
							115				
60	R2	21/21	60.5 - 62.3	RQD = 67%							
	R3	17/17	62.2 - 63.6	RQD = 0%							
	R4	60/60	63.6 - 68.6	RQD = 77%							
65											
70											
75											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.								Page 3 of 3			
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.								Boring No.: BB-MWS-102			

ATTACHMENT 3
BEDROCK CORE PHOTOGRAPHS



Core box containing wetted core from test boring BB-MWS-101 (Box 1 of 1); left side of core box (top portion of cores). Slots from top to bottom:

- 1) BB-MWS-101, R1/R2;
- 2) BB-MWS-101, R3/R4;
- 3) empty;
- 4) empty.



Core box containing wetted core from test boring BB-MWS-101 (Box 1 of 1); right side of core box (bottom portion of cores). Slots from top to bottom:

- 1) BB-MWS-101, R2/R3;
- 2) BB-MWS-101, R4/R5;
- 3) empty;
- 4) empty.

SCHONEWALD
ENGINEERING
ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
WAGGON BRIDGE #0487
WAUGAN ROAD OVER WILSON STREAM
MONMOUTH, MAINE
MAINEDOT WIN 26234.00

Sheet No.:

1 of 2



Core box containing wetted core from test boring BB-MWS-102 (Box 1 of 1); left side of core box (top portion of cores). Slots from top to bottom:

- 1) BB-MWS-102, R1/R2;
- 2) BB-MWS-102, R4;
- 3) empty;
- 4) empty.



Core box containing wetted core from test boring BB-MWS-102 (Box 1 of 1); right side of core box (bottom portion of cores). Slots from top to bottom:

- 1) BB-MWS-102, R2/R3;
- 2) BB-MWS-102, R4;
- 3) empty;
- 4) empty.

SCHONEWALD
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ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
WAGGON BRIDGE #0487
WAUGAN ROAD OVER WILSON STREAM
MONMOUTH, MAINE
MAINEDOT WIN 26234.00

Sheet No.:

2 of 2

ATTACHMENT 4
RESULTS OF LABORATORY TESTS ON SOIL SAMPLES

TABULATION OF SOIL TESTING
(GTX PROJECT NO. 316327)
(listed in order of test report presentation)

Boring No.	Sample No.	Sample Depth (ft., BGS)	GTX TEST ID / NO.	Tests Completed (Method)
BB-MWS-101	4D	14-16	693246	moisture content (ASTM D2216)
BB-MWS-101	5D	19-21	693246	moisture content (ASTM D2216)
BB-MWS-101	6D	29-31	693246	moisture content (ASTM D2216)
BB-MWS-101	U1	24-26	693246	moisture content (ASTM D2216)
BB-MWS-102	3D	9-11	693248	moisture content (ASTM D2216)
BB-MWS-102	4D	14.5-16.5	693248	moisture content (ASTM D2216)
BB-MWS-102	5D	24-26	693248	moisture content (ASTM D2216)
BB-MWS-102	6D	34-36	693248	moisture content (ASTM D2216)
BB-MWS-102	7D	39-41	693248	moisture content (ASTM D2216)
BB-MWS-102	U1	19-21	693248	moisture content (ASTM D2216)
BB-MWS-102	U2	29-31	693248	moisture content (ASTM D2216)
BB-MWS-101	2D, 3D	4-6, 9-11	693270	moisture, ash, organic matter (ASTM D2974)
BB-MWS-101	3D-A	10.5-11	693270	moisture, ash, organic matter (ASTM D2974)
BB-MWS-101	2D, 3D	4-6, 9-11	not provided	ph (AASHTO T289)
BB-MWS-101	2D, 3D	4-6, 9-11	not provided	lab soil resistivity (ASTM G57)
BB-MWS-101	4D	14-16	693259	finer content [% passing #200 sieve] (ASTM D1140)
BB-MWS-101	U1	24-26	693259	finer content [% passing #200 sieve] (ASTM D1140)
BB-MWS-102	7D	39-41	693259	finer content [% passing #200 sieve] (ASTM D1140)
BB-MWS-102	U1	19-21	693259	finer content [% passing #200 sieve] (ASTM D1140)
BB-MWS-102	U2	29-31	693259	finer content [% passing #200 sieve] (ASTM D1140)
BB-MWS-101	3D-A	10.5-11	693267	grain size w/o hydrometer (ASTM D6913)
BB-MWS-101	8D	39-40.2	693268	grain size w/o hydrometer (ASTM D6913)
BB-MWS-102	10D	54-56	693269	grain size w/o hydrometer (ASTM D6913)
BB-MWS-101	4D	14-16	693237	Atterberg Limits (ASTM D4318)
BB-MWS-101	5D	19-21	693238	Atterberg Limits (ASTM D4318)
BB-MWS-101	6D	29-31	693239	Atterberg Limits (ASTM D4318)
BB-MWS-101	U1	24-26	693234	Atterberg Limits (ASTM D4318)
BB-MWS-102	1D	2-4	693240	Atterberg Limits (ASTM D4318)
BB-MWS-102	3D	9-11	693241	Atterberg Limits (ASTM D4318)
BB-MWS-102	4D	14.5-16.5	693242	Atterberg Limits (ASTM D4318)
BB-MWS-102	5D	24-26	693243	Atterberg Limits (ASTM D4318)
BB-MWS-102	6D	34-36	693244	Atterberg Limits (ASTM D4318)
BB-MWS-102	7D	39-41	693245	Atterberg Limits (ASTM D4318)
BB-MWS-102	U1	19-21	693235	Atterberg Limits (ASTM D4318)
BB-MWS-102	U2	29-31	693236	Atterberg Limits (ASTM D4318)
BB-MWS-102	U1	19-21	not provided	3-point CIU triaxial test (ASTM D4767)
BB-MWS-102	U2	29-31	not provided	3-point CIU triaxial test (ASTM D4767)
BB-MWS-101	U1	24-26	CRC-3	constant rate of strain consolidation (ASTM D4186)
BB-MWS-102	U1	19-21	CRC-2	constant rate of strain consolidation (ASTM D4186)
BB-MWS-102	U2	29-31	CRC-1	constant rate of strain consolidation (ASTM D4186)
BB-MWS-101	2D, 3D	4-6, 9-11	TEi-Testing Services	Chloride Method B (AASHTO T291)
BB-MWS-101	2D, 3D	4-6, 9-11	TEi-Testing Services	Sulfates [Soluble] (AASHTO T290)



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	11/08/22
Depth :	---	Tested By:	ckg
		Checked By:	bfs
		Test Id:	693246

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-MWS-101	4D	14-16'	Moist, olive gray silt	48.5
BB-MWS-101	5D	19-21'	Moist, gray clay	44.2
BB-MWS-101	6D	29-31'	Moist, olive gray clay	40.1
BB-MWS-101	U- 1	24-26'	Wet, gray clay	39.9

Notes: Temperature of Drying : 110° Celsius



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	11/08/22
Depth :	---	Tested By:	ckg
		Checked By:	bfs
		Test Id:	693248

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-MWS-102	3D	9-11'	Moist, olive gray silt	31.7
BB-MWS-102	4D	14.5-16.5'	Wet, olive gray silt	57.8
BB-MWS-102	5D	24-26'	Wet, olive gray clay	41.0
BB-MWS-102	6D	34-36'	Wet, gray clay	46.9
BB-MWS-102	7D	39-41'	Wet, olive gray clay	40.5
BB-MWS-102	U- 1	19-21'	Wet, gray silt	49.9
BB-MWS-102	U- 2	29-31'	Wet, gray clay	37.7

Notes: Temperature of Drying : 110° Celsius



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	11/15/22
Depth :	---	Test Id:	693270
		Tested By:	cam
		Checked By:	bfs

Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content, %	Ash Content, %	Organic Matter, %
BB-MWS-101	2D, 3D	4-6', 9-10.5'	Moist, dark grayish brown silt with organics	64	94.2	5.8
BB-MWS-101	3D-A	10.5-11'	Moist, dark brownish gray silt with organics	140	88.7	11.3

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass; dried to a constant mass at temperature of 105° C
 Ash content and organic matter determined by Method C; dried to constant mass at temperature 440° C



Client:	Schonewald Engineering Associates, Inc.
Project Name:	Waggon Bridge, Wilson Stream
Project Location:	Monmouth, ME
GTX #:	316327
Test Date:	11/15/22
Tested By:	NLB
Checked By:	bfs

pH by AASHTO T 289

Boring ID	Sample ID	Depth, ft	Description	pH
BB-MWS-101	2D, 3D	4-6, 9-10.5	Moist, dark grayish brown silt with organics	6.34



Client:	Schonewald Engineering Associates, Inc.
Project:	Waggon Bridge, Wilson Stream
Location:	Monmouth, ME
GTX#:	316327
Test Date:	11/15/22
Tested By:	nlb
Checked By:	bfs

**Laboratory Measurement of Soil Resistivity Using
the Wenner Four-Electrode Method by ASTM G57
(Laboratory Measurement)**

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, (ohm-cm) ⁻¹
BB-MWS-101	2D, 3D	4-6, 9-10.5	Moist, dark grayish brown silt with organics	5,165	1.94E-04

Notes: Test Equipment: Nilsson Model 400 Soil Resistance Meter, MC Miller Soil Box
 Water added to sample to create a thick slurry prior to testing (saturated condition).
 Electrical Conductivity is calculated as inverse of Electrical Resistivity (per ASTM G57)
 Test conducted in standard laboratory atmosphere: 68-73 F



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	11/11/22
Depth :	---	Test Id:	693259
		Tested By:	ckg
		Checked By:	bfs

Amount of Material Passing #200 Sieve - ASTM D1140

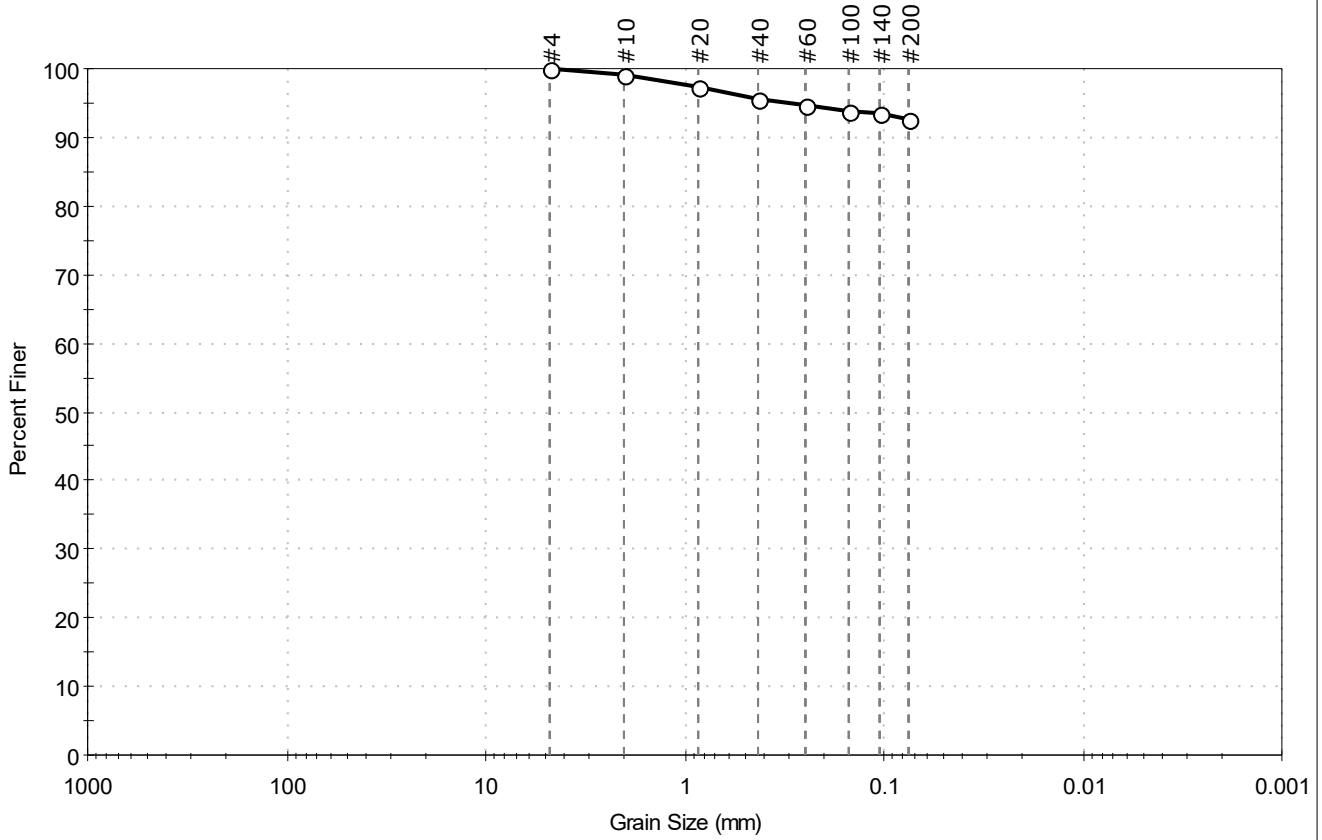
Boring ID	Sample ID	Depth	Visual Description	Fines, %
BB-MWS-101	4D	14-16'	Moist, olive gray silt	98.4
BB-MWS-101	U-1	24-26'	Wet, gray clay	98.9
BB-MWS-102	7D	39-41'	Wet, olive gray clay	99.8
BB-MWS-102	U-1	19-21'	Wet, gray silt	99.8
BB-MWS-102	U-2	29-31'	Wet, gray clay	99.7

Notes: Tests performed using Method B - washing using a wetting agent
 Dry mass of test specimen was determined directly



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-101	Sample Type:	jar
Sample ID:	3D-A	Test Date:	11/11/22
Depth:	10.5-11'	Test Id:	693267
Test Comment:	---		
Visual Description:	Moist, dark brownish gray silt with organics		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	7.3	92.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	97		
#40	0.42	96		
#60	0.25	95		
#100	0.15	94		
#140	0.11	93		
#200	0.075	93		

<u>Coefficients</u>	
D ₈₅ = N/A	D ₃₀ = N/A
D ₆₀ = N/A	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

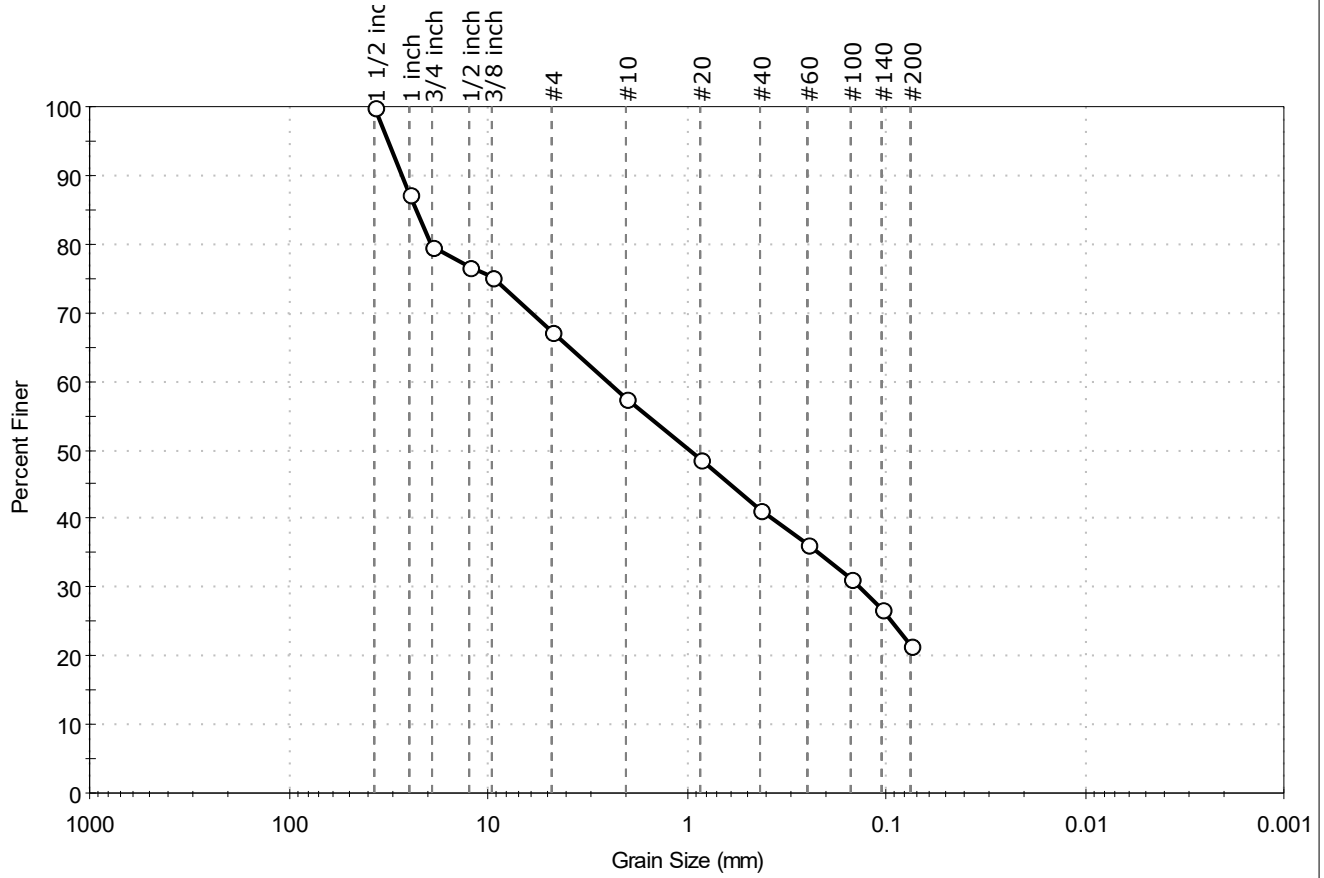
<u>Classification</u>	
ASTM	N/A
AASHTO	Silty Soils (A-4 (0))

<u>Sample/Test Description</u>
Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-101	Sample Type:	jar
Sample ID:	8D	Test Date:	11/11/22
Depth :	39-40.2'	Test Id:	693268
Test Comment:	---		
Visual Description:	Moist, olive gray silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	32.8	45.7	21.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	87		
3/4 inch	19.00	80		
1/2 inch	12.50	77		
3/8 inch	9.50	75		
#4	4.75	67		
#10	2.00	58		
#20	0.85	49		
#40	0.42	41		
#60	0.25	36		
#100	0.15	31		
#140	0.11	27		
#200	0.075	22		

<u>Coefficients</u>	
D ₈₅ = 23.0130 mm	D ₃₀ = 0.1365 mm
D ₆₀ = 2.4719 mm	D ₁₅ = N/A
D ₅₀ = 0.9615 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

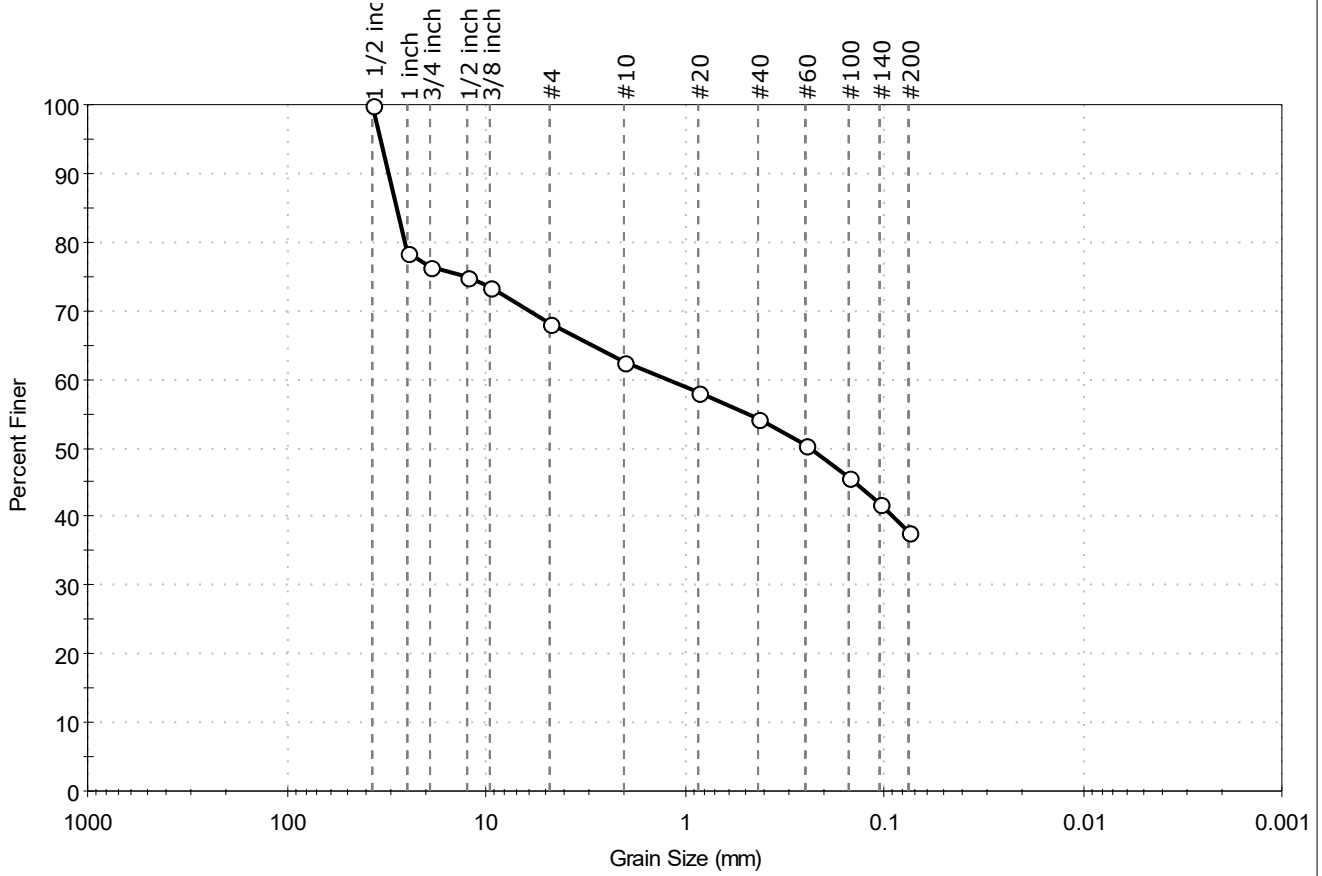
<u>Classification</u>	
ASTM	N/A
AASHTO	Stone Fragments, Gravel and Sand (A-1-b (0))

<u>Sample/Test Description</u>
Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	jar
Sample ID:	10D	Test Date:	11/11/22
Depth :	54-56'	Test Id:	693269
Test Comment:	---		
Visual Description:	Moist, olive gray silty gravel with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	31.8	30.4	37.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	78		
3/4 inch	19.00	76		
1/2 inch	12.50	75		
3/8 inch	9.50	73		
#4	4.75	68		
#10	2.00	63		
#20	0.85	58		
#40	0.42	54		
#60	0.25	50		
#100	0.15	46		
#140	0.11	42		
#200	0.075	38		

<u>Coefficients</u>	
D ₈₅ = 28.3103 mm	D ₃₀ = N/A
D ₆₀ = 1.2074 mm	D ₁₅ = N/A
D ₅₀ = 0.2373 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

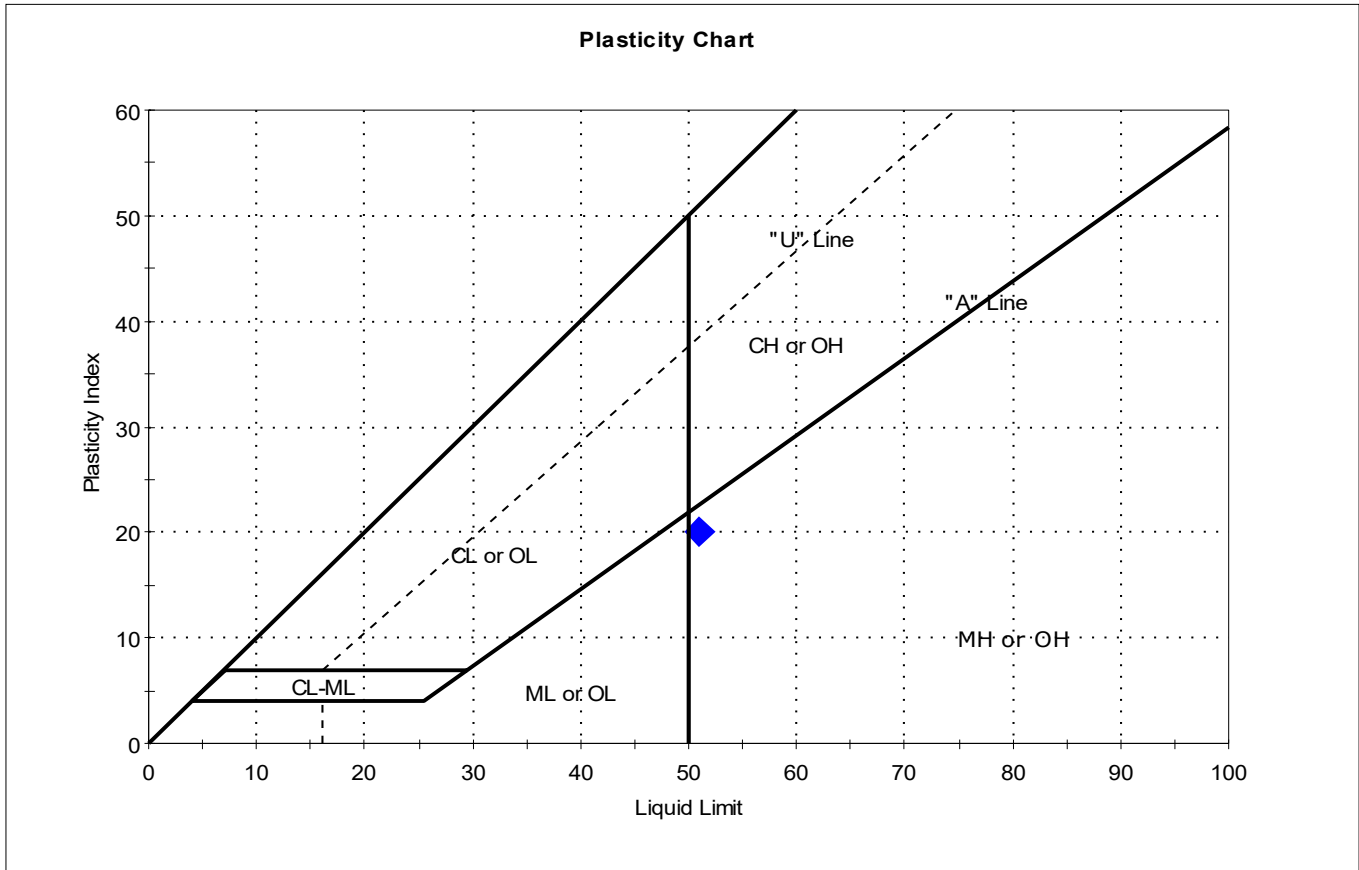
<u>Classification</u>	
ASTM	N/A
AASHTO	Silty Soils (A-4 (0))

<u>Sample/Test Description</u>
Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-101	Sample Type:	jar
Sample ID:	4D	Test Date:	11/14/22
Depth:	14-16'	Checked By:	bfs
		Test Id:	693237
Test Comment:	---		
Visual Description:	Moist, olive gray silt		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	BB-MWS-101	14-16'	49	51	31	20	0.9	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

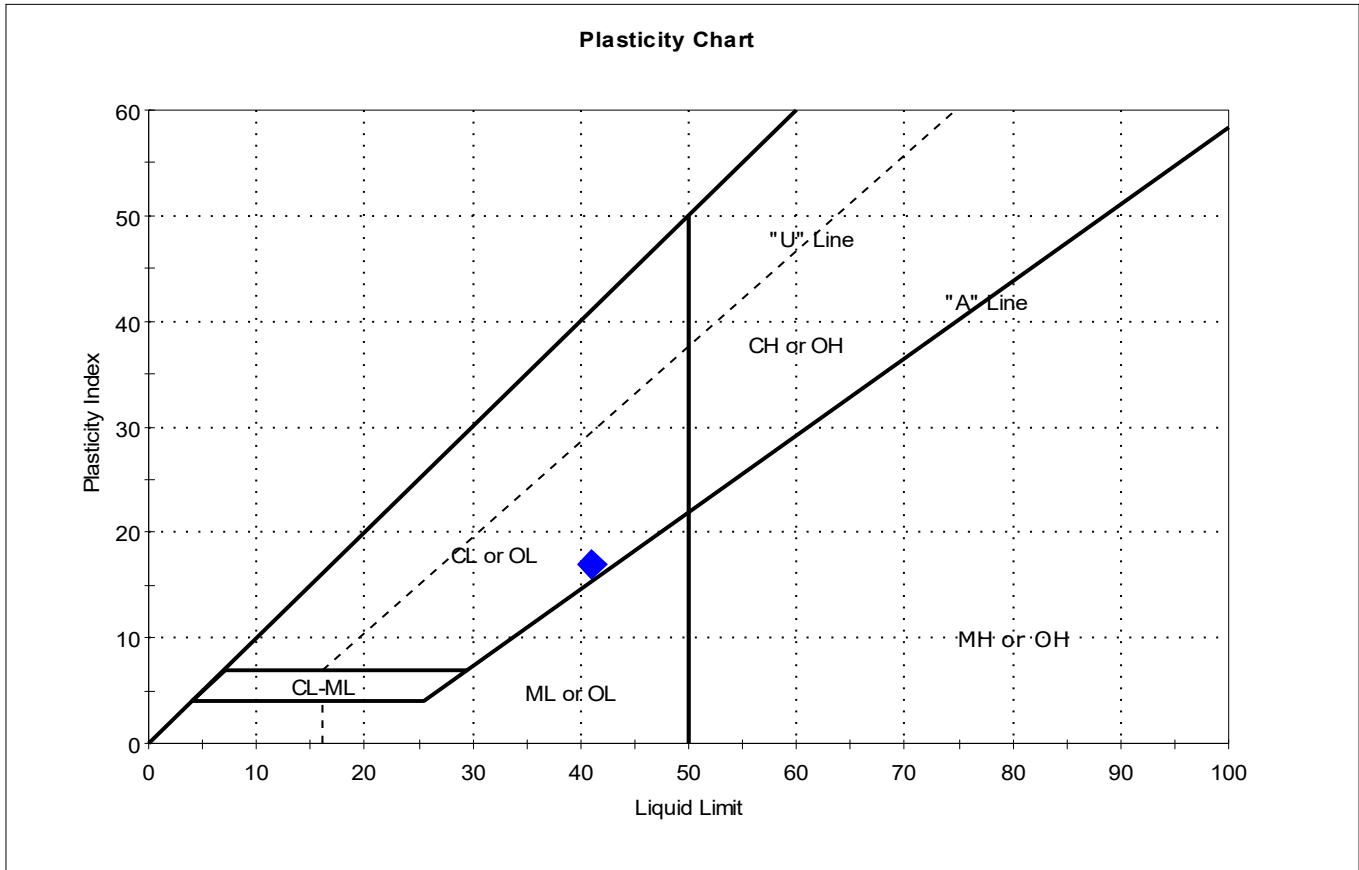
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-101	Sample Type:	jar
Sample ID:	5D	Test Date:	11/14/22
Depth :	19-21'	Checked By:	bfs
		Test Id:	693238
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
◆	5D	BB-MWS-101	19-21'	44	41	24	17	1.2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

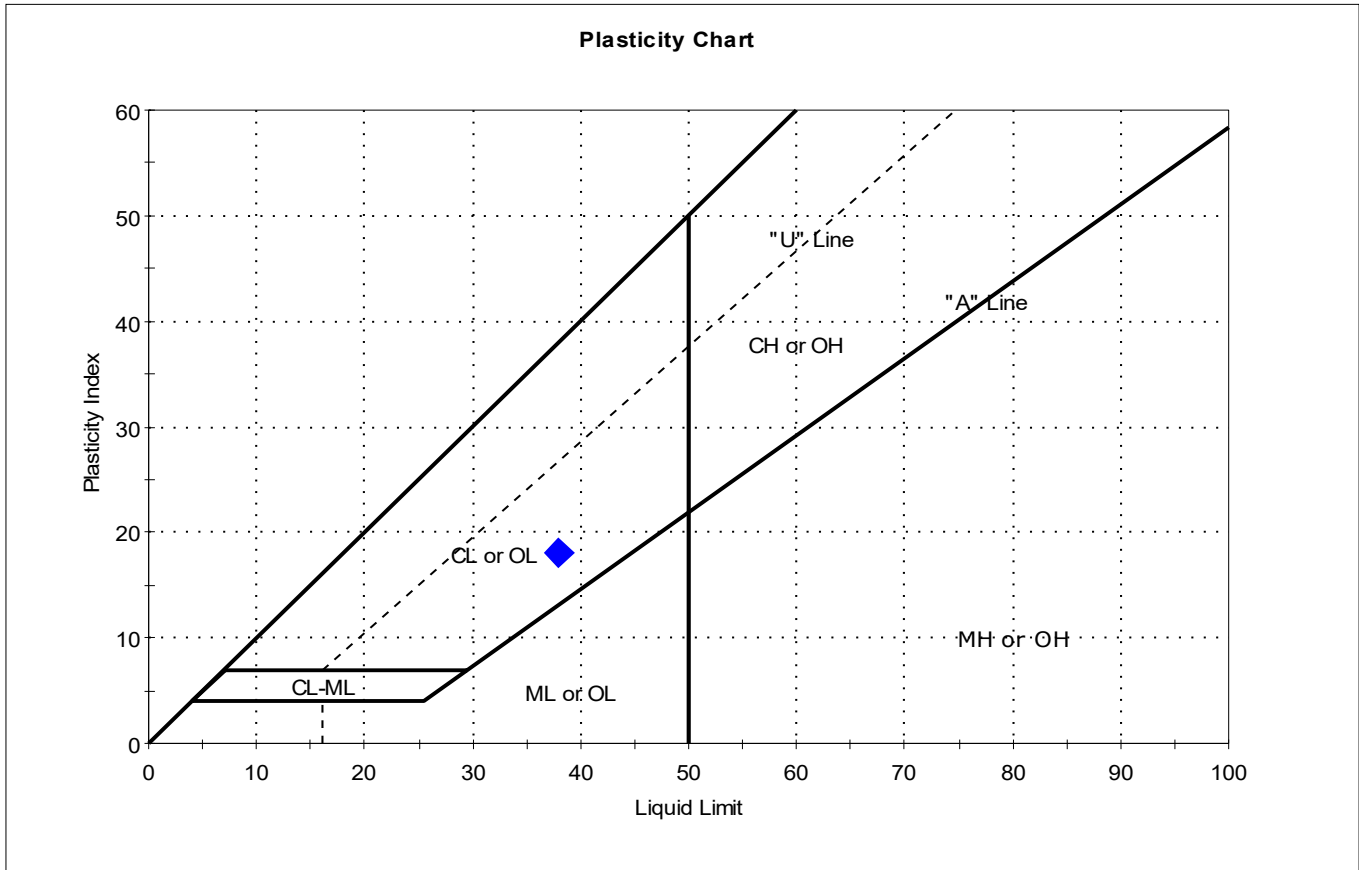
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-101	Sample Type:	jar
Sample ID:	6D	Test Date:	11/15/22
Depth :	29-31'	Test Id:	693239
Test Comment:	---		
Visual Description:	Moist, olive 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
◆	6D	BB-MWS-101	29-31'	40	38	20	18	1.1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

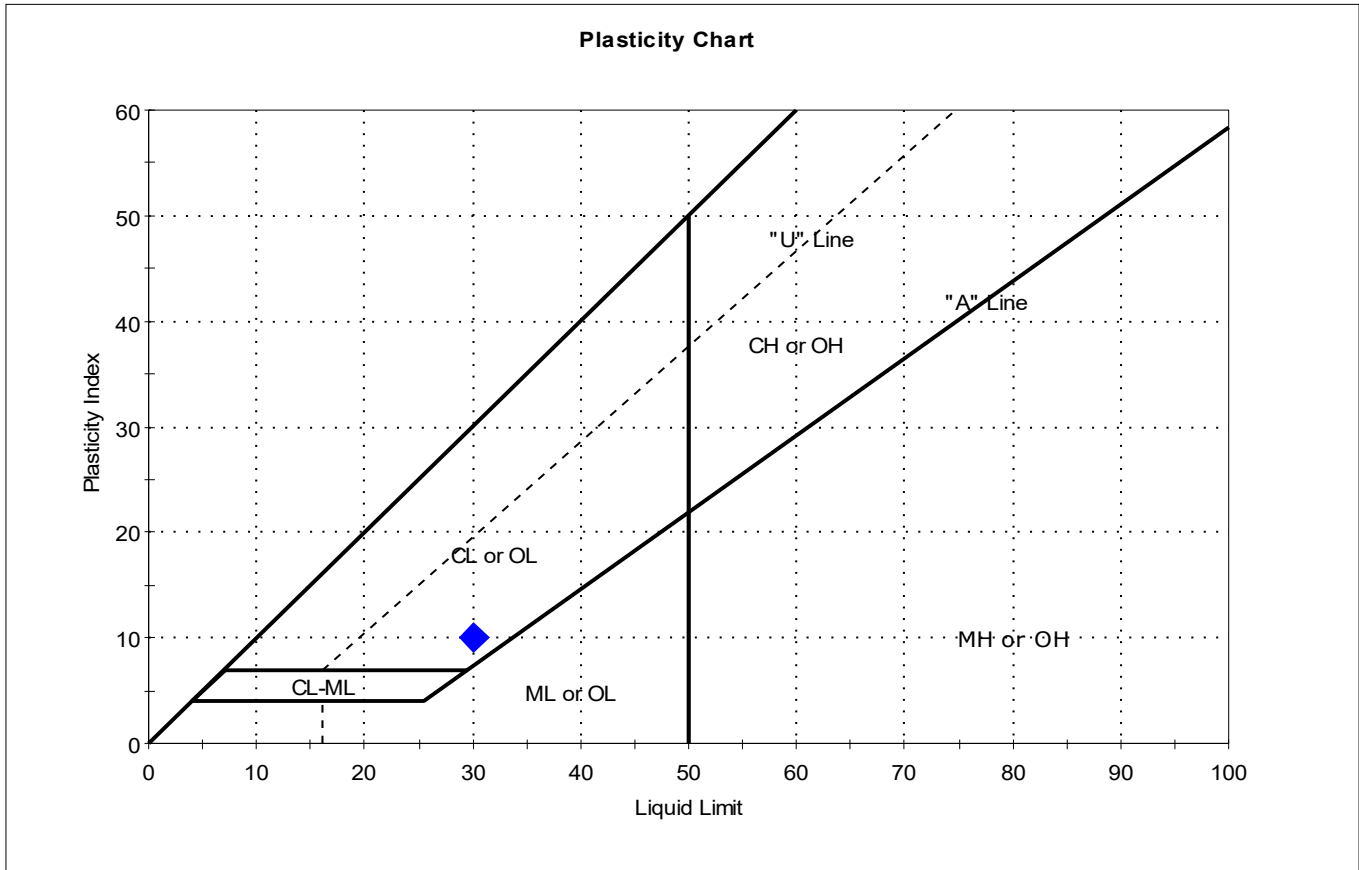
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-101	Sample Type:	tube
Sample ID:	U-1	Test Date:	11/14/22
Depth:	24-26'	Test Id:	693234
Test Comment:	---		
Visual Description:	Wet, 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
◆	U-1	BB-MWS-101	24-26'	40	30	20	10	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

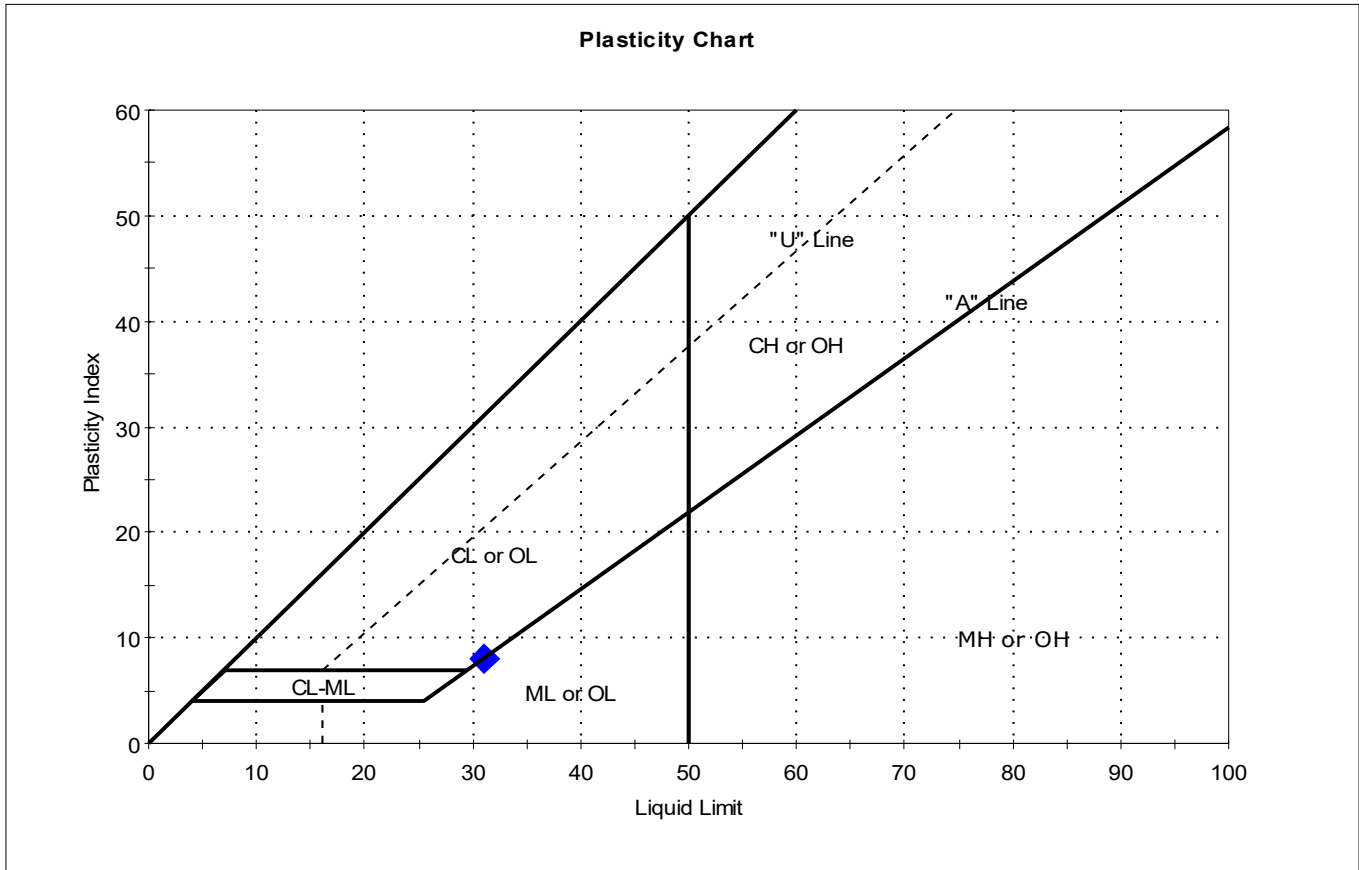
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	jar
Sample ID:	1D	Test Date:	11/15/22
Depth :	2-4'	Test Id:	693240
Test Comment:	---		
Visual Description:	Moist, olive 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
◆	1D	BB-MWS-102	2-4'	20	31	23	8	-0.4	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

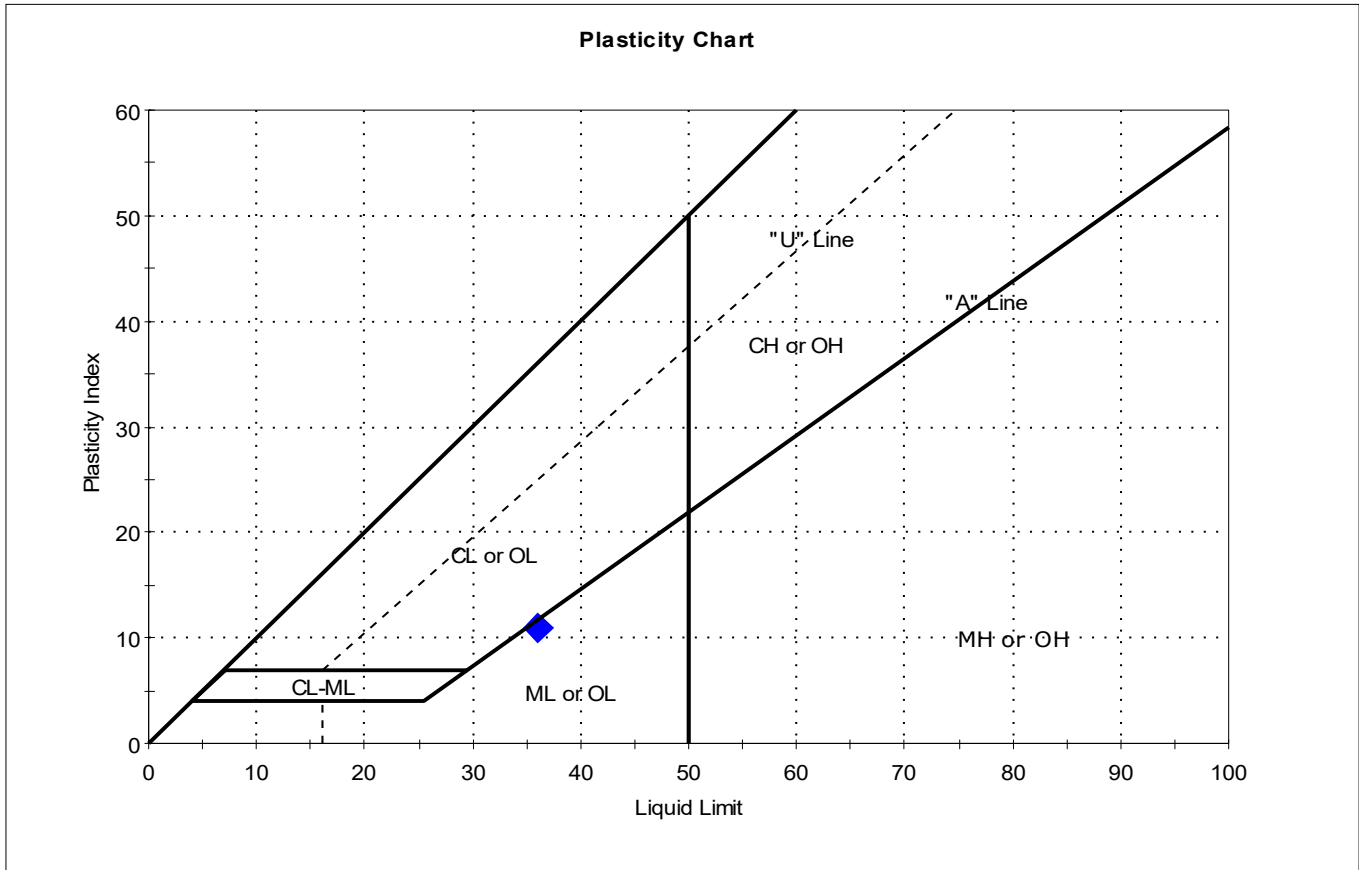
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	jar
Sample ID:	3D	Test Date:	11/11/22
Depth :	9-11'	Test Id:	693241
Test Comment:	---		
Visual Description:	Moist, olive gray silt		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	BB-MWS-102	9-11'	32	36	25	11	0.6	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

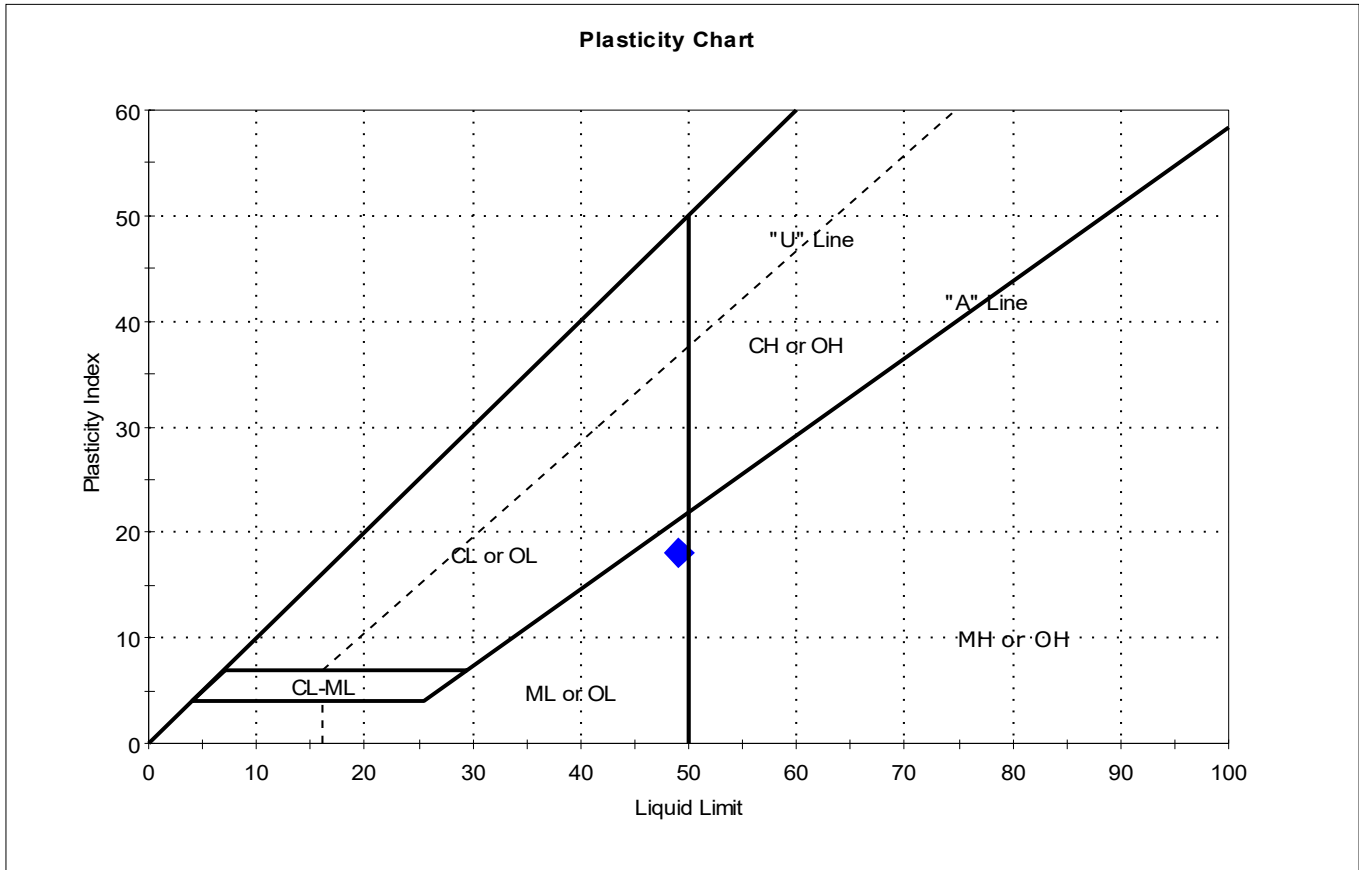
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	jar
Sample ID:	4D	Test Date:	11/14/22
Depth:	14.5-16.5'	Test Id:	693242
Test Comment:	---		
Visual Description:	Wet, olive gray silt		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	BB-MWS-102	14.5-16.5	58	49	31	18	1.5	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

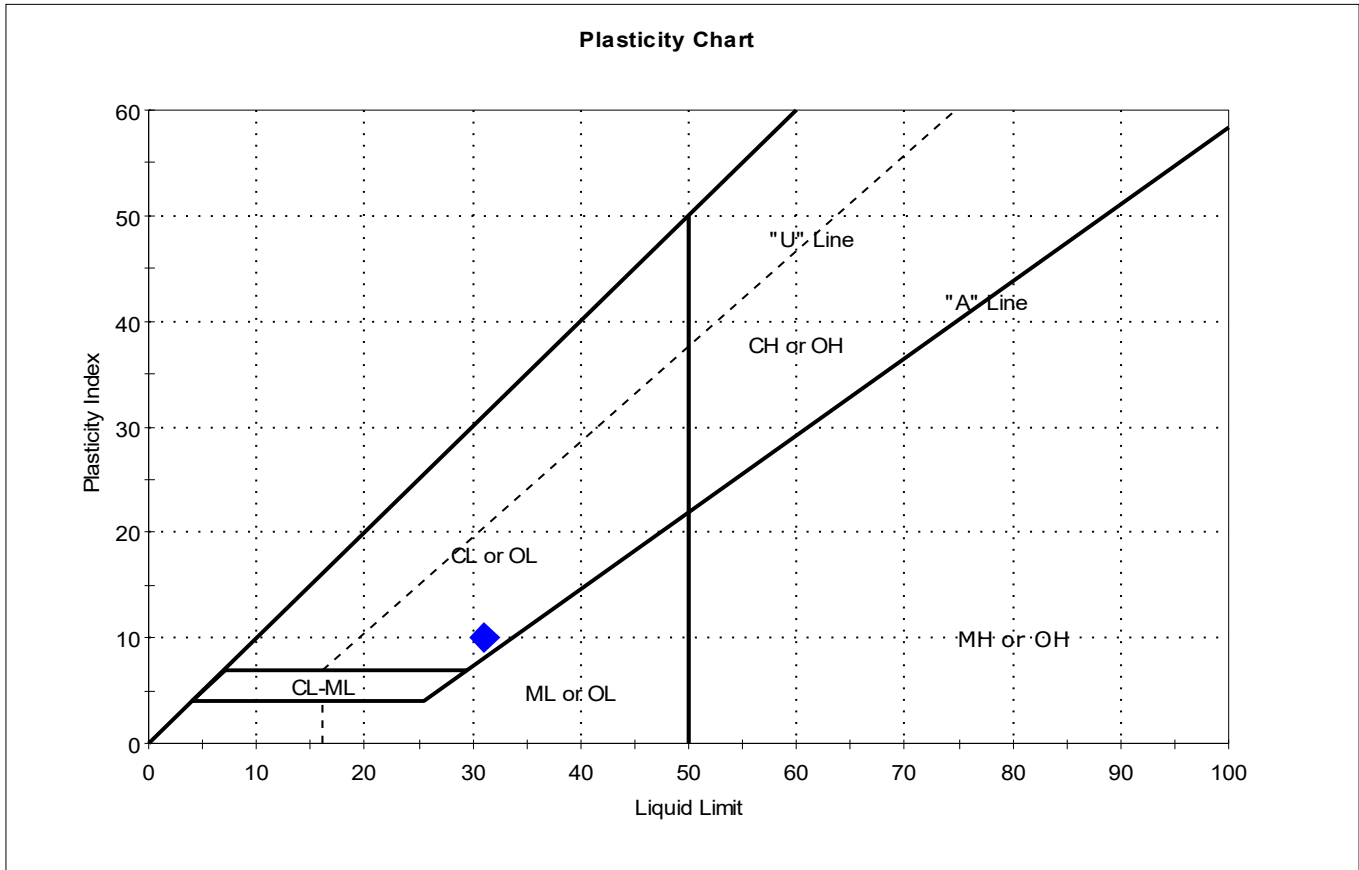
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	jar
Sample ID:	5D	Test Date:	11/14/22
Depth :	24-26'	Checked By:	bfs
		Test Id:	693243
Test Comment:	---		
Visual Description:	Wet, olive 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
◆	5D	BB-MWS-102	24-26'	41	31	21	10	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

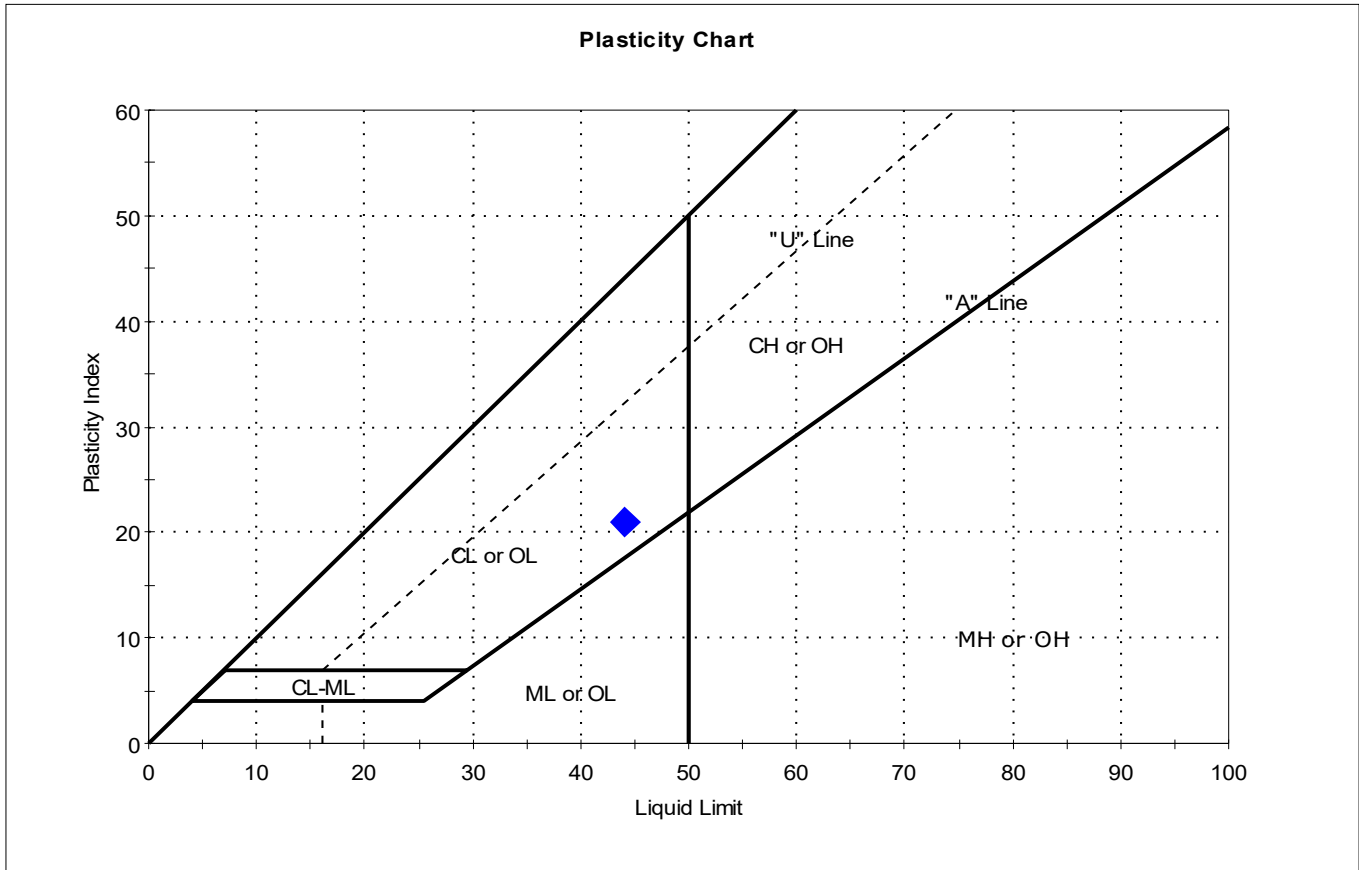
Dilatancy: SLOW

Toughness: LOW



Client: Schonewald Engineering Associates, Inc.	Project No: GTX-316327
Project: Waggon Bridge, Wilson Stream	
Location: Monmouth, ME	
Boring ID: BB-MWS-102	Sample Type: jar
Sample ID: 6D	Test Date: 11/14/22
Depth: 34-36'	Test Id: 693244
Tested By: cam	Checked By: bfs
Test Comment: ---	
Visual Description: Wet, 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
◆	6D	BB-MWS-102	34-36'	47	44	23	21	1.1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

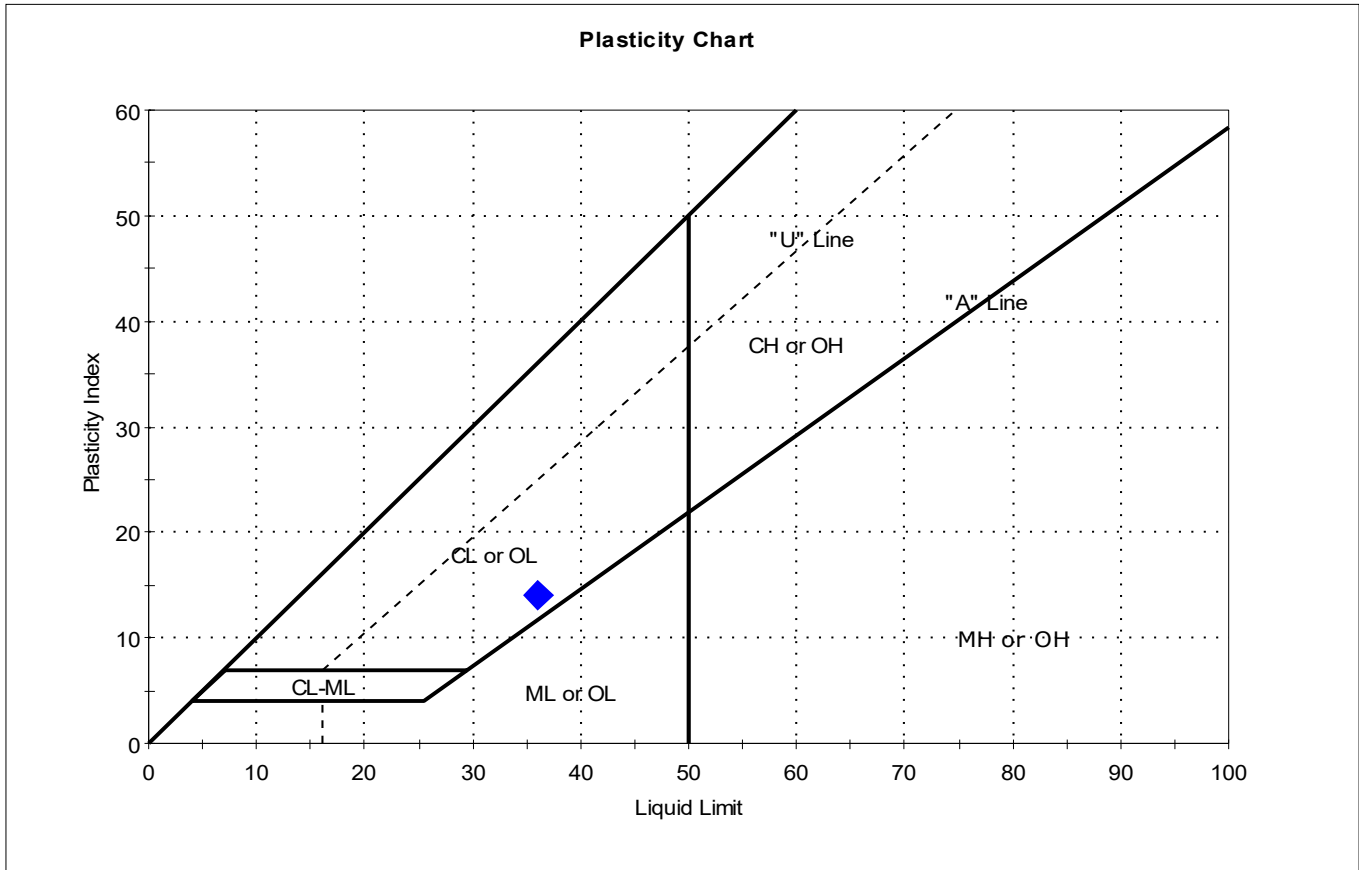
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	jar
Sample ID:	7D	Test Date:	11/14/22
Depth:	39-41'	Test Id:	693245
Test Comment:	---		
Visual Description:	Wet, olive 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	BB-MWS-102	39-41'	41	36	22	14	1.3	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

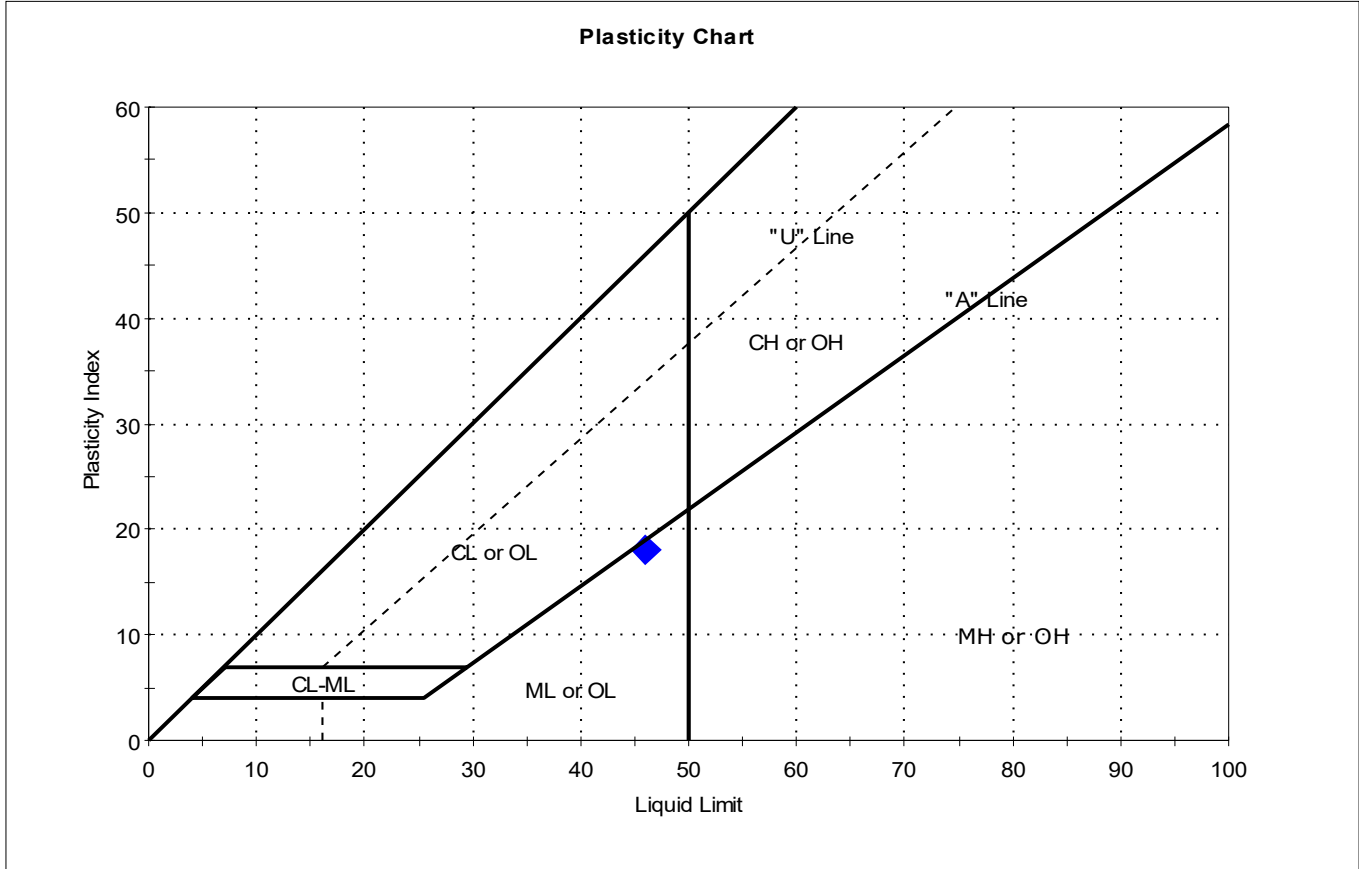
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	tube
Sample ID:	U-1	Test Date:	11/11/22
Depth :	19-21'	Checked By:	bfs
		Test Id:	693235
Test Comment:	---		
Visual Description:	Wet, gray silt		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	U-1	BB-MWS-102	19-21'	50	46	28	18	1.2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

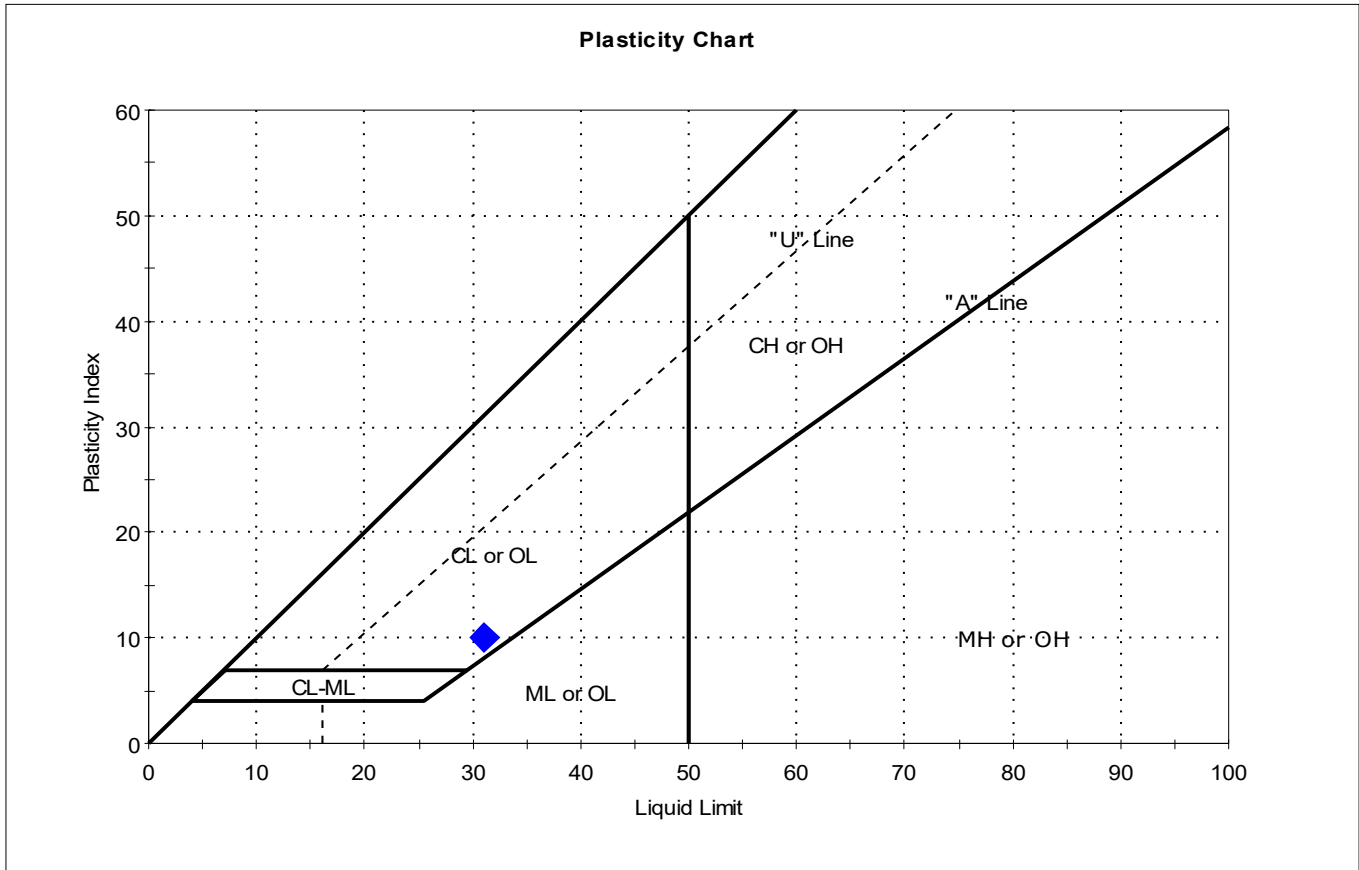
Dilatancy: SLOW

Toughness: LOW



Client:	Schonewald Engineering Associates, Inc.		
Project:	Waggon Bridge, Wilson Stream		
Location:	Monmouth, ME	Project No:	GTX-316327
Boring ID:	BB-MWS-102	Sample Type:	tube
Sample ID:	U-2	Test Date:	11/14/22
Depth :	29-31'	Test Id:	693236
Test Comment:	---		
Visual Description:	Wet, 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
◆	U-2	BB-MWS-102	29-31'	38	31	21	10	1.7	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW



Client: Schonwald Engineering Associates, Inc.

Project Name: Waggon Bridge, Wilson Stream

Project Location: Monmouth, ME

Project Number: GTX-316327

Tested By: trm

Checked By: njh

Boring ID: BB-MWS-102

Preparation: intact

Description: Wet, gray silt

Classification: ---

Group Symbol: ---

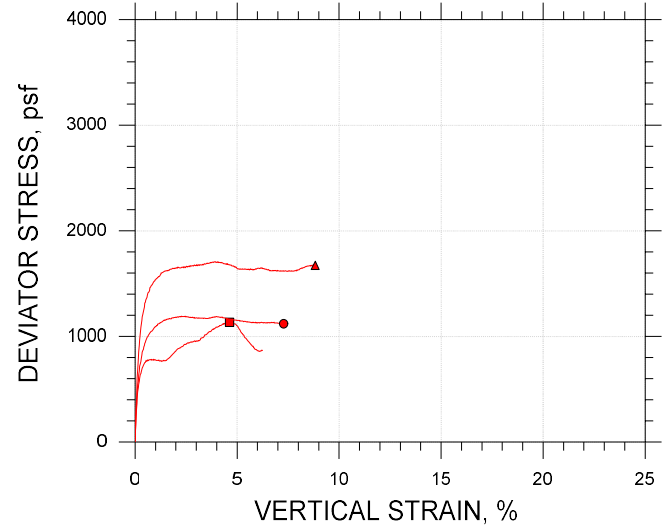
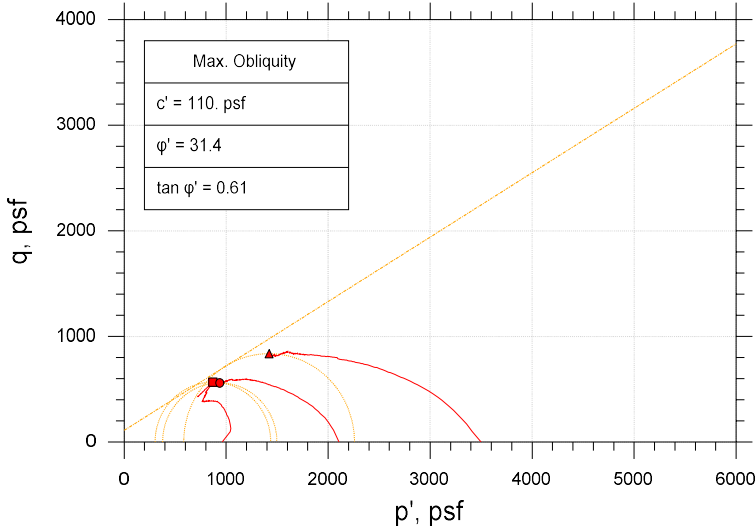
Liquid Limit: 46

Plastic Limit: 28

Plasticity Index: 18

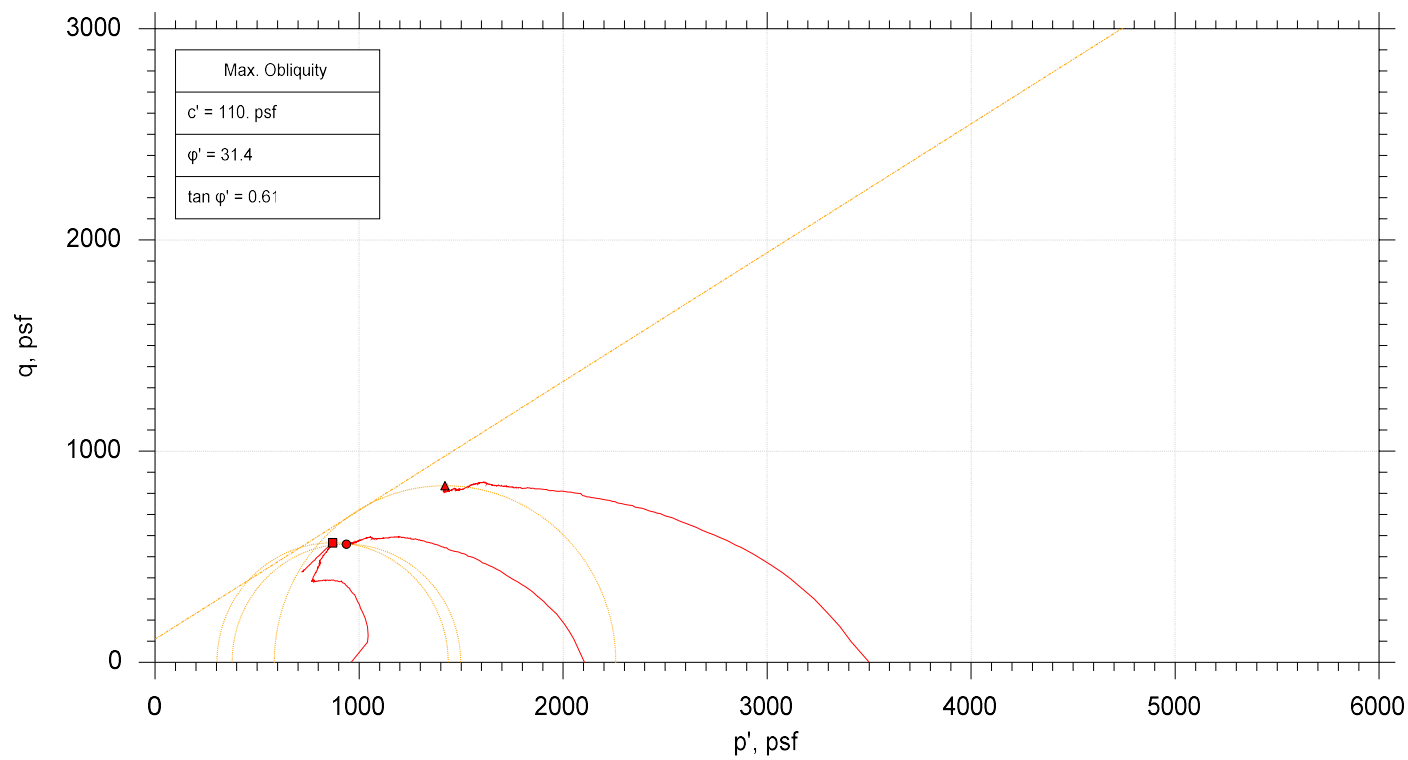
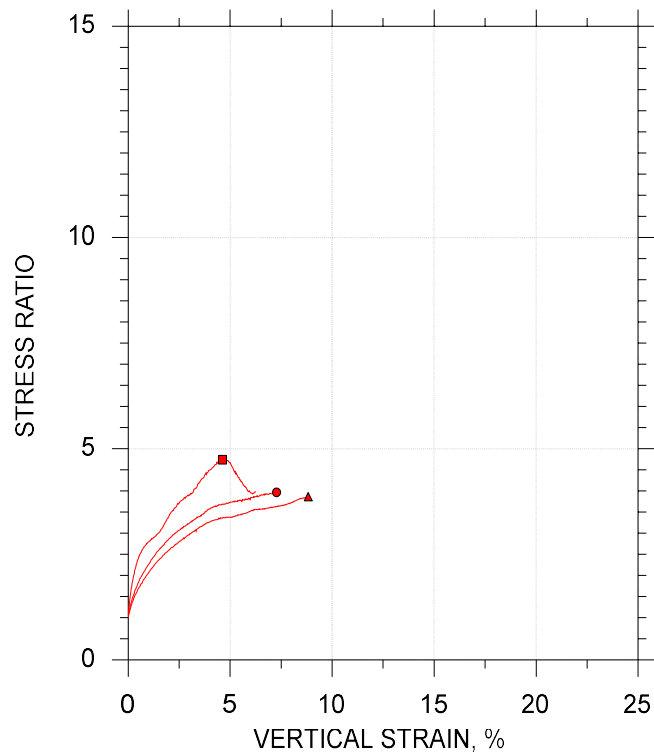
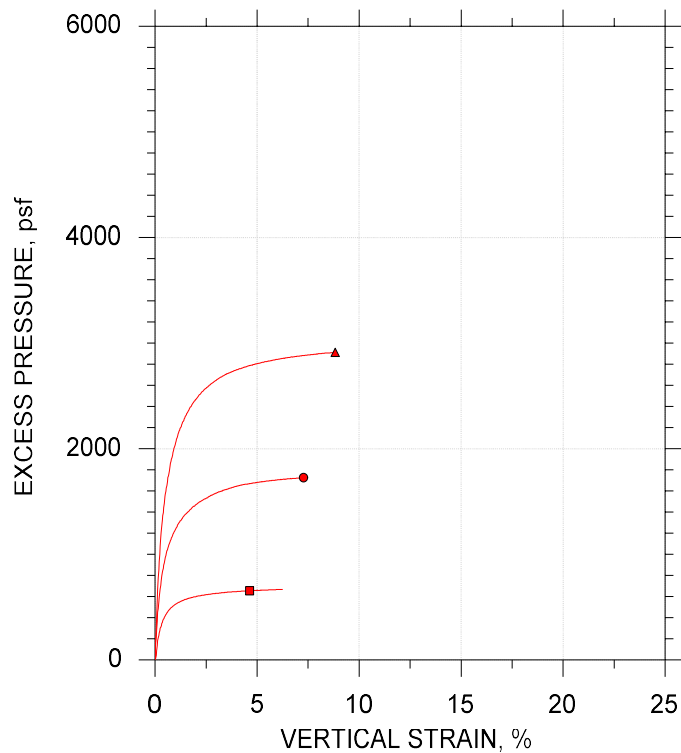
Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



Symbol	■	●	▲	
Sample ID	U-1	U-1	U-1	
Depth, ft	19-21'	19-21'	19-21'	
Test Number	CU-2-1	CU-2-2	CU-2-3	
Initial	Height, in	4.450	4.450	4.450
	Diameter, in	2.010	2.010	2.010
	Moisture Content (from Cuttings), %	47.5	48.8	48.7
	Dry Density, pcf	72.1	71.1	71.3
	Saturation (Wet Method), %	95.9	96.1	96.4
	Void Ratio	1.34	1.37	1.36
Before Shear	Moisture Content, %	47.7	47.1	41.6
	Dry Density, pcf	73.6	74.2	79.4
	Cross-sectional Area (Method A), in ²	3.126	3.087	2.974
	Saturation, %	100.0	100.0	100.0
	Void Ratio	1.29	1.27	1.12
	Back Pressure, psf	2.174e+004	2.174e+004	2.173e+004
Vertical Effective Consolidation Stress, psf	955.4	2088.	3460.	
Horizontal Effective Consolidation Stress, psf	959.4	2105.	3498.	
Vertical Strain after Consolidation, %	0.5419	1.454	3.412	
Volumetric Strain after Consolidation, %	1.870	4.037	8.070	
Time to 50% Consolidation, min	---	---	27.00	
Shear Strength, psf	566.9	560.2	837.1	
Strain at Failure, %	4.63	7.28	8.83	
Strain Rate, %/min	0.01600	0.01600	0.01600	
Deviator Stress at Failure, psf	1134.	1120.	1674.	
Effective Minor Principal Stress at Failure, psf	303.1	377.2	583.9	
Effective Major Principal Stress at Failure, psf	1437.	1498.	2258.	
B-Value	0.95	0.90	0.98	
Notes:	<ul style="list-style-type: none"> - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Atterberg Limits determined by ASTM D4318. - Deviator Stress includes membrane correction. - Values for c and ϕ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. 			
Remarks:				

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	U-1	CU-2-1	19-21'	trm	11/7/22	njh	11/14/22	316327-CU-2-1n.dat
●	U-1	CU-2-2	19-21'	trm	11/4/22	njh	11/14/22	316327-CU-2-2n.dat
▲	U-1	CU-2-3	19-21'	trm	11/7/22	njh	11/14/22	316327-CU-2-3n.dat

	Project: Waggon Bridge, Wilson Stream	Location: Monmouth, ME	Project No.: GTX-316327
	Boring No.: BB-MWS-102	Sample Type: intact	
	Description: Wet, gray silt		
	Remarks: TX-007		



Client: Schonewald Engineering Associates, Inc.

Project Name: Waggon Bridge, Wilson Stream

Project Location: Monmouth, ME

Project Number: GTX-316327

Tested By: trm

Checked By: njh

Boring ID: BB-MWS-102

Preparation: intact

Description: Wet, gray clay

Classification: ---

Group Symbol: ---

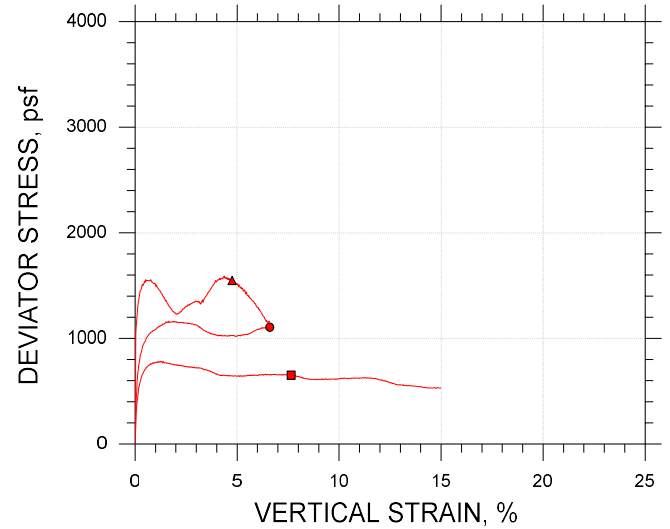
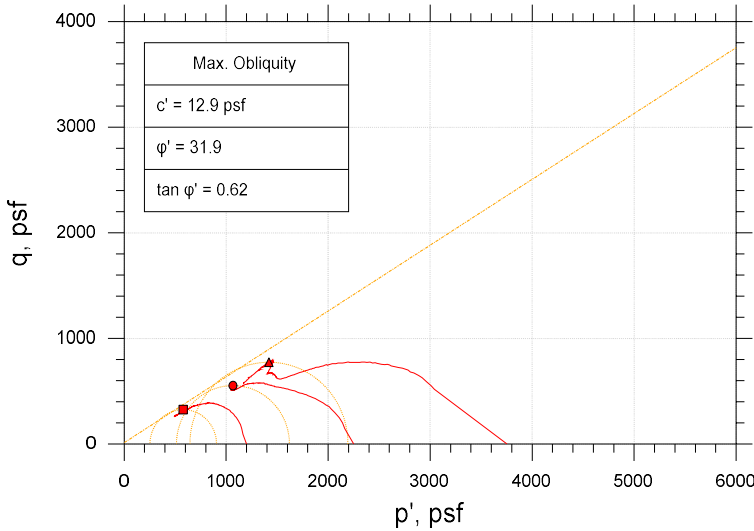
Liquid Limit: 31

Plastic Limit: 21

Plasticity Index: 10

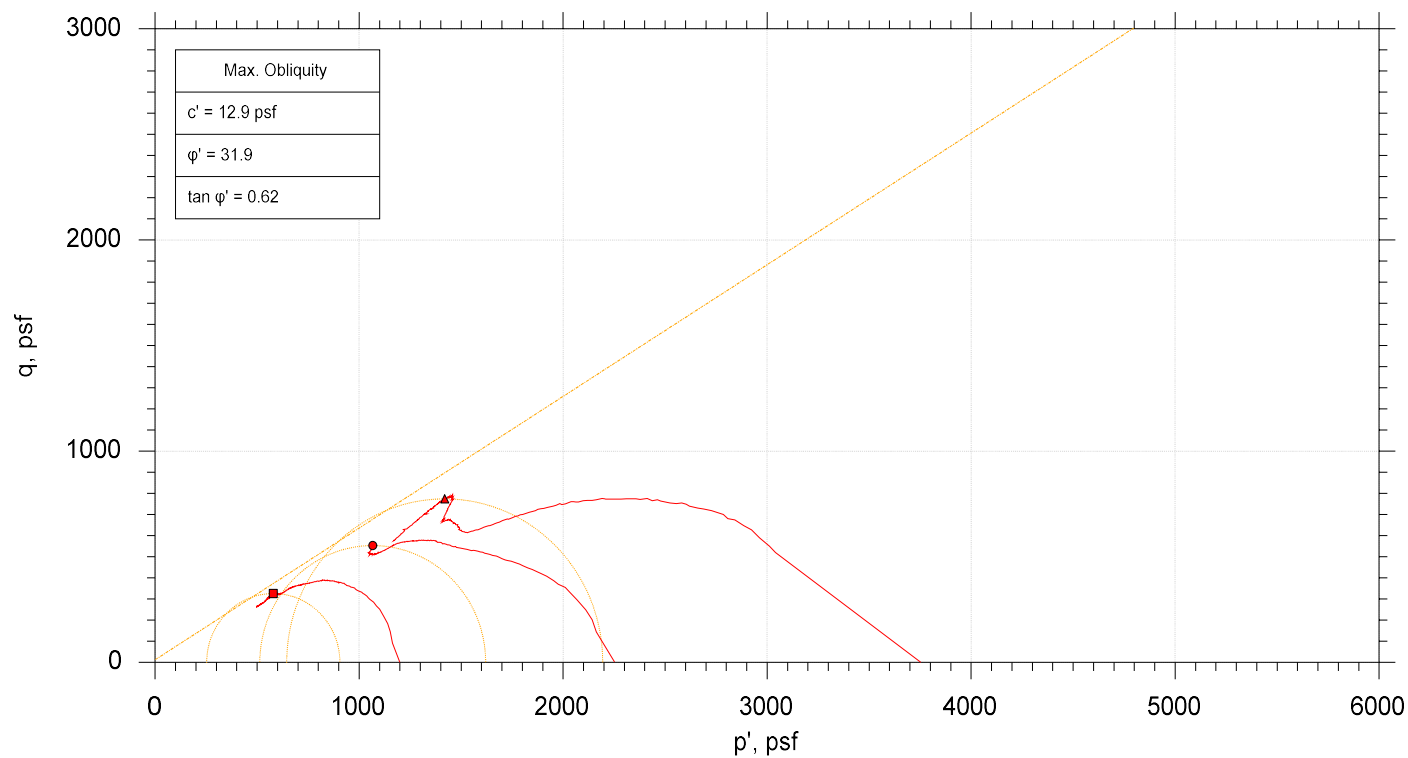
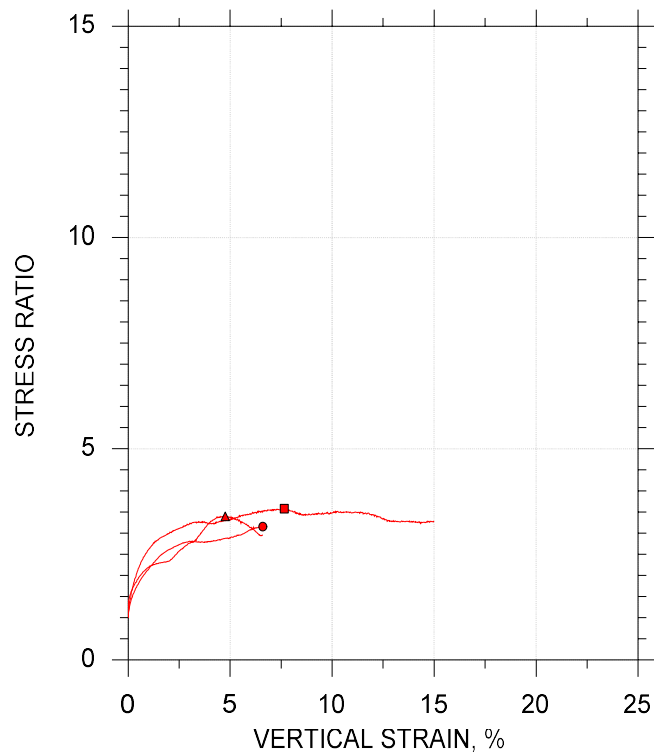
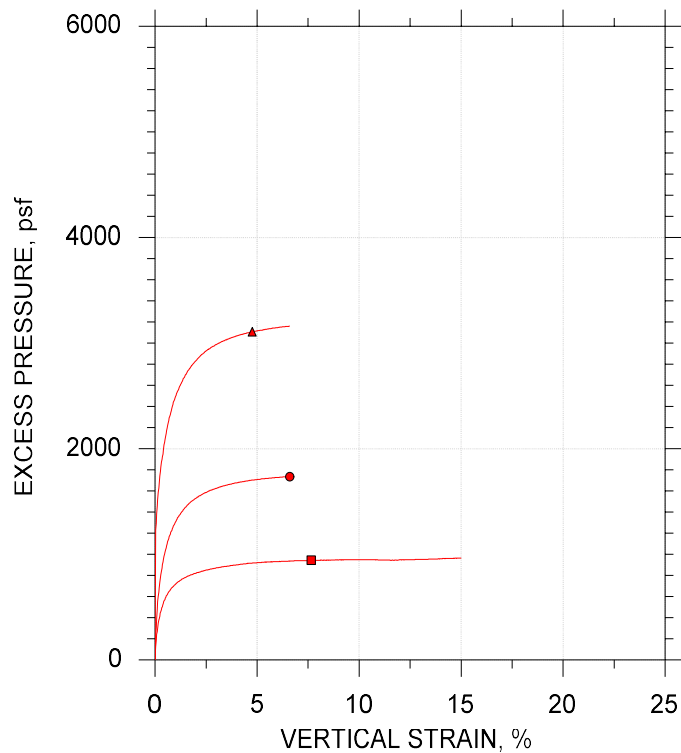
Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



Symbol	■	●	▲	
Sample ID	U-2	U-2	U-2	
Depth, ft	29-31'	29-31'	29-31'	
Test Number	CU-1-1	CU-1-2	CU-1-3	
Initial	Height, in	4.600	4.620	4.480
	Diameter, in	2.010	2.040	2.010
	Moisture Content (from Cuttings), %	37.7	39.1	37.3
	Dry Density, pcf	81.1	77.1	82.5
	Saturation (Wet Method), %	94.3	88.9	96.6
	Void Ratio	1.08	1.19	1.04
Before Shear	Moisture Content, %	36.2	40.4	34.8
	Dry Density, pcf	85.2	80.6	86.9
	Cross-sectional Area (Method A), in ²	3.068	3.197	3.087
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.978	1.09	0.940
	Back Pressure, psf	2.173e+004	2.174e+004	2.171e+004
Vertical Effective Consolidation Stress, psf	1190.	2226.	3726.	
Horizontal Effective Consolidation Stress, psf	1198.	2250.	3752.	
Vertical Strain after Consolidation, %	0.8209	2.291	2.221	
Volumetric Strain after Consolidation, %	2.551	4.603	4.438	
Time to 50% Consolidation, min	---	---	41.00	
Shear Strength, psf	326.5	553.6	774.6	
Strain at Failure, %	7.65	6.60	4.76	
Strain Rate, %/min	0.01600	0.01600	0.01600	
Deviator Stress at Failure, psf	653.0	1107.	1549.	
Effective Minor Principal Stress at Failure, psf	252.7	513.1	644.9	
Effective Major Principal Stress at Failure, psf	905.6	1620.	2194.	
B-Value	0.99	0.95	1.00	
Notes:	<ul style="list-style-type: none"> - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Atterberg Limits determined by ASTM D4318. - Deviator Stress includes membrane correction. - Values for c and ϕ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. 			
Remarks:				

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

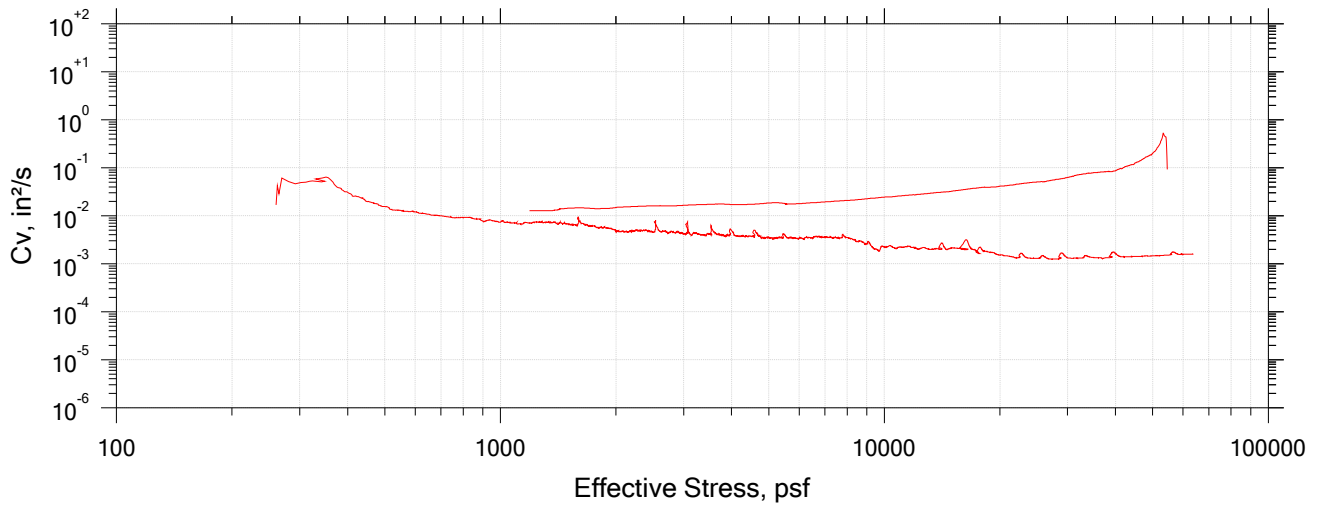
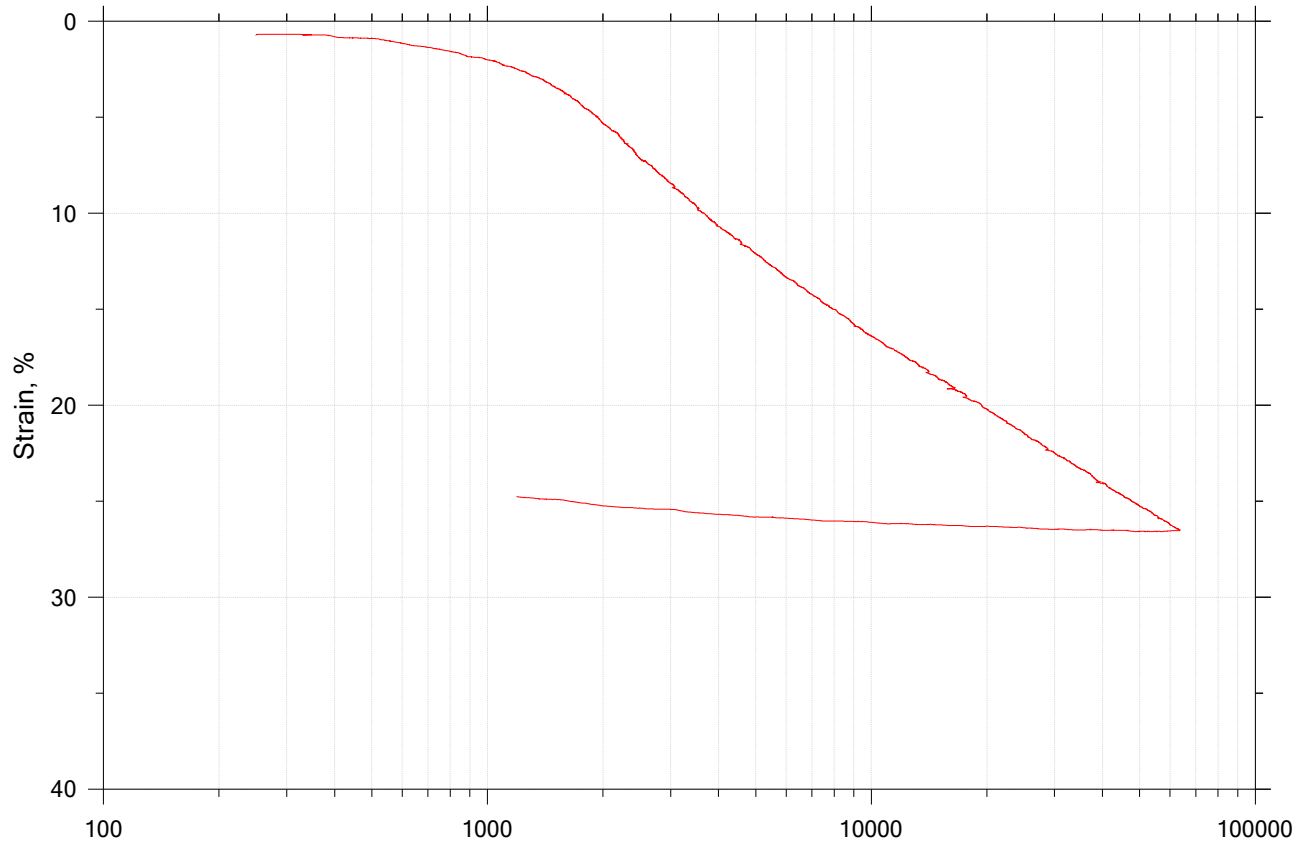



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	U-2	CU-1-1	29-31'	trm	11/3/22	njh	11/14/22	316327-CU-1-1n.dat
●	U-2	CU-1-2	29-31'	trm	11/4/22	njh	11/14/22	316327-CU-1-2n.dat
▲	U-2	CU-1-3	29-31'	trm	11/3/22	njh	11/14/22	316327-CU-1-3n.dat

	Project: Waggon Bridge, Wilson Stream	Location: Monmouth, ME	Project No.: GTX-316327
	Boring No.: BB-MWS-102	Sample Type: intact	
	Description: Wet, gray clay		
	Remarks: TX-018		

CRC Test

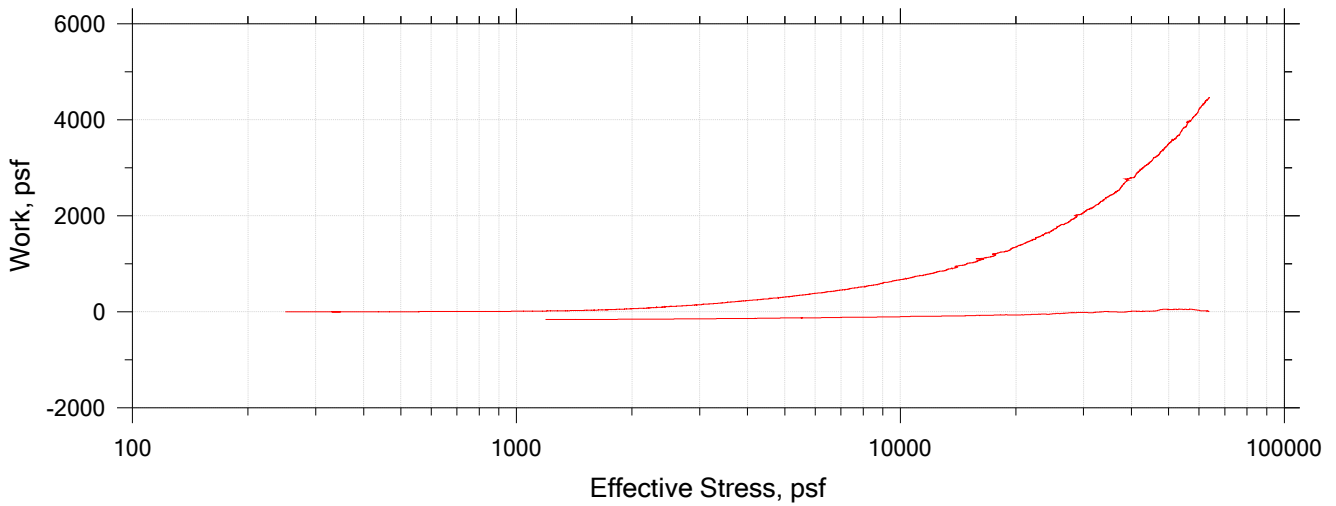
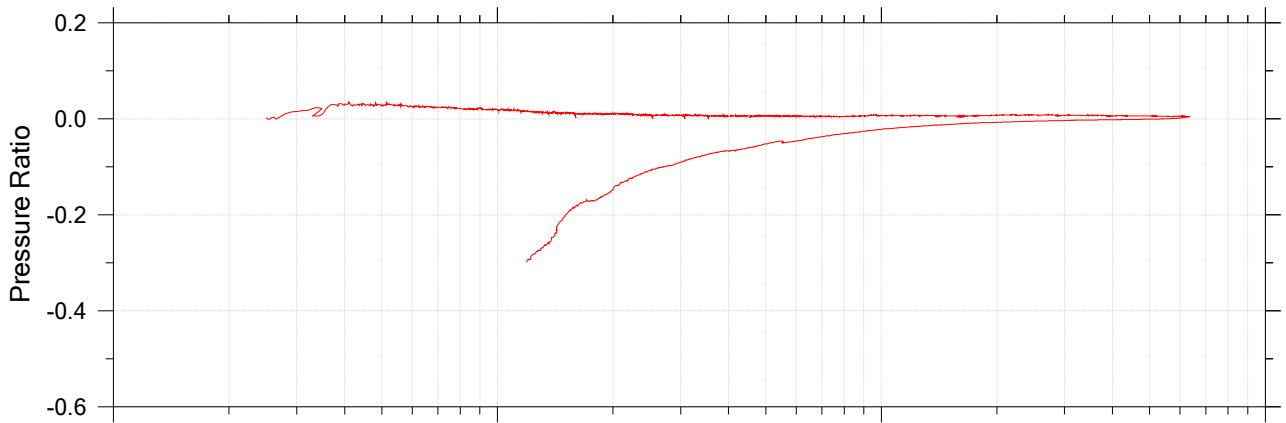
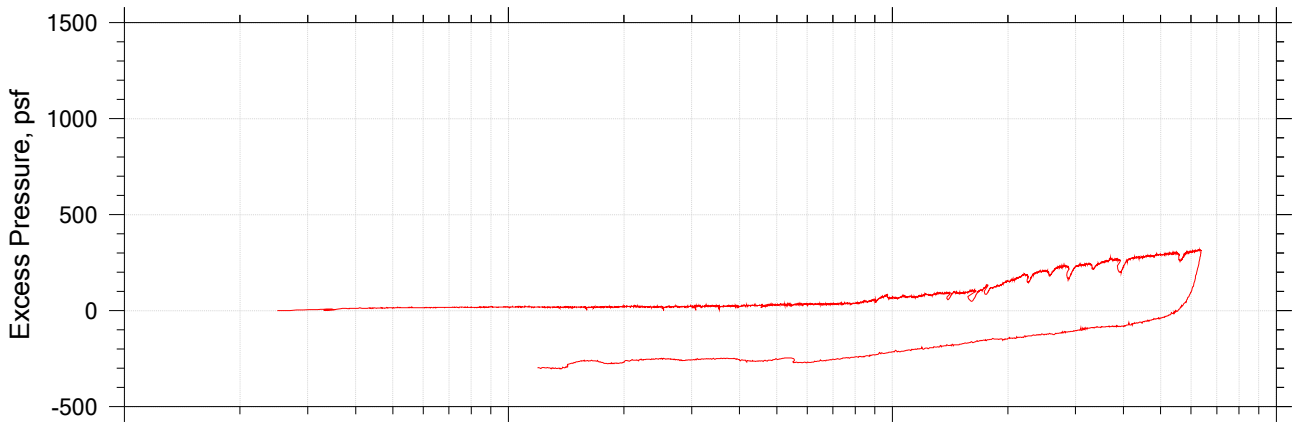
Summary




	Project Name: Waggon Bridge, Weston Street	Location: Manmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-101	Tester: md	Checker: njh
	Sample Number: U-1	Test Date: 11/03/22	Depth: 24-26'
	Test Number: CRC-3	Preparation: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System TX-003		

CRC Test

Pressure Curves




	Project Name: Waggon Bridge, Weston Street	Location: Manmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-101	Tester: md	Checker: njh
	Sample Number: U-1	Test Date: 11/03/22	Depth: 24-26'
	Test Number: CRC-3	Preparation: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System TX-003		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.72 (Estimated)	Liquid Limit: 30
Specimen Height, in: 1.00	Initial Void Ratio: 1.06	Plastic Limit: 20
Final Height, in: 0.75	Final Void Ratio: 0.547	Plasticity Index: 10

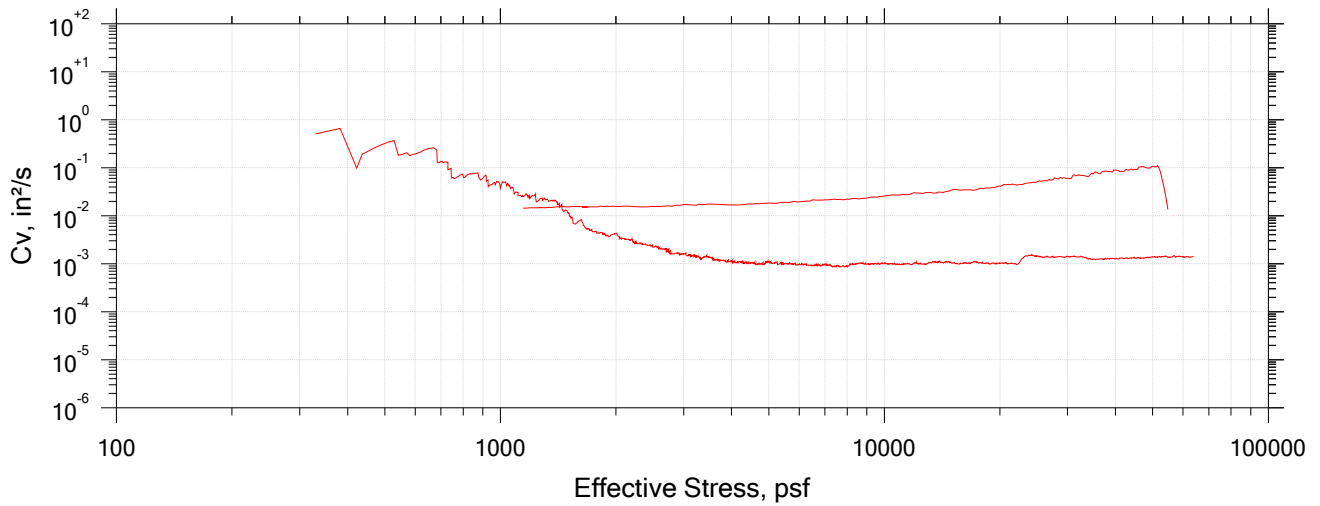
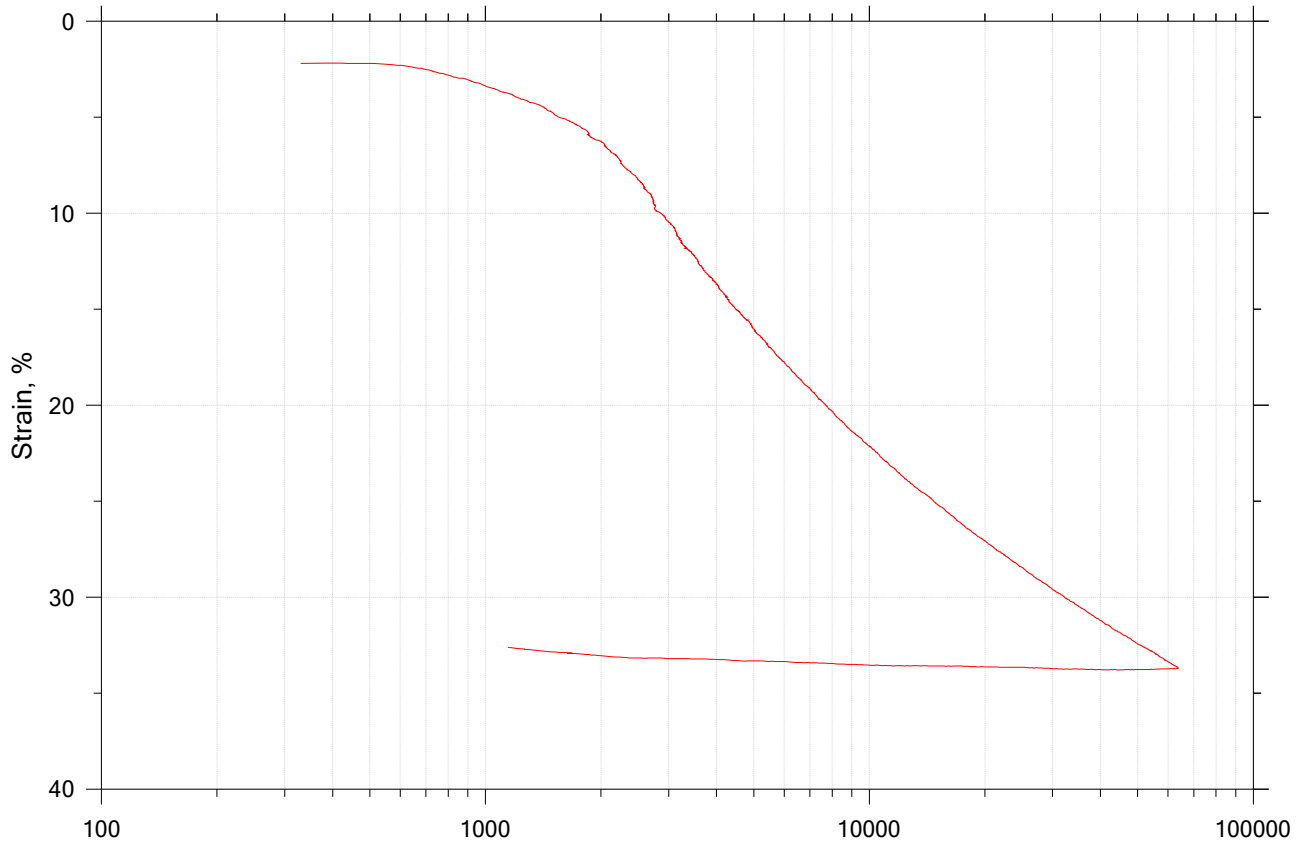
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-4797	---		E2955
Mass Container, gm	8.25	111.02	111.02	8.44
Mass Container + Wet Soil, gm	133.57	258.37	238.29	137.46
Mass Container + Dry Soil, gm	97.83	216.96	216.96	115.84
Mass Dry Soil, gm	89.58	105.94	105.94	107.4
Water Content, %	39.90	39.08	20.13	20.13
Void Ratio	---	1.06	0.55	---
Degree of Saturation, %	---	99.93	100.00	---
Dry Unit Weight, pcf	---	82.221	109.63	---


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 Name: Waggon Bridge, Weston Street	Location: Manmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-101	Tester: md	Checker: njh
	Sample Number: U-1	Test Date: 11/03/22	Depth: 24-26'
	Test Number: CRC-3	Preparation: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System TX-003		

CRC Test

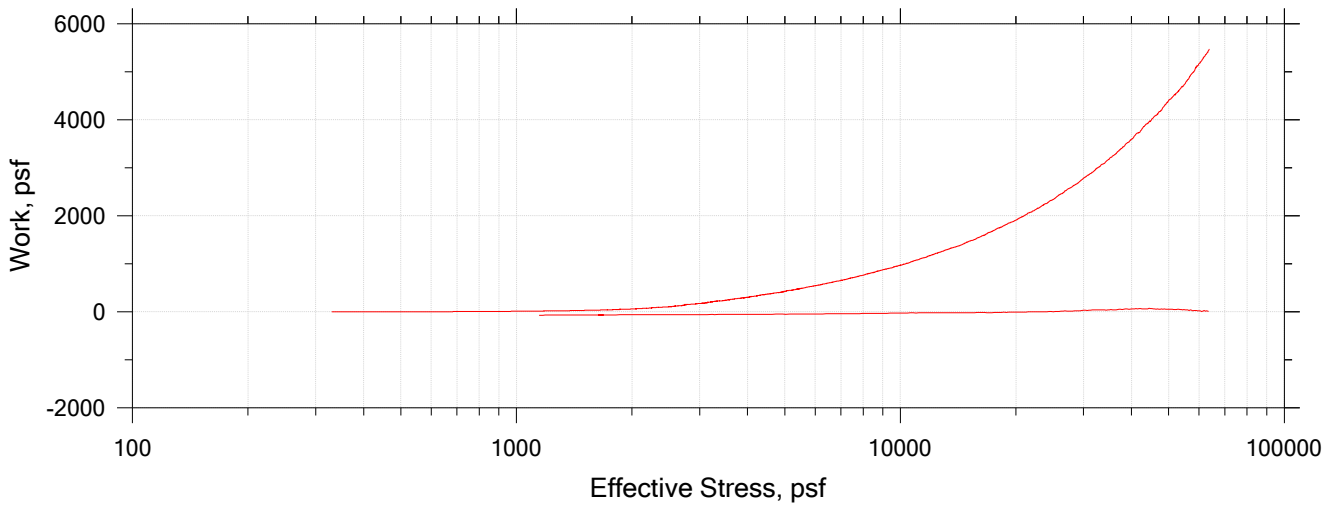
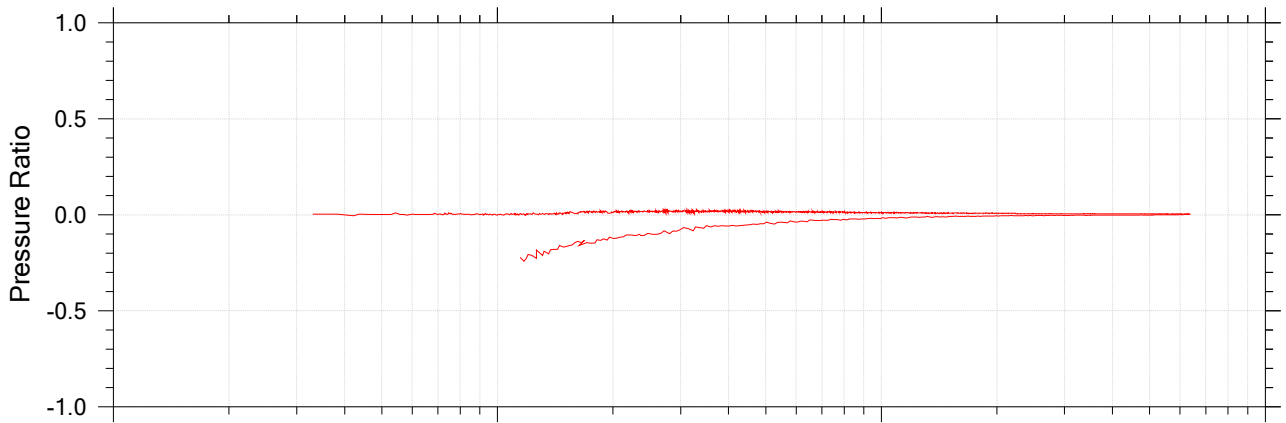
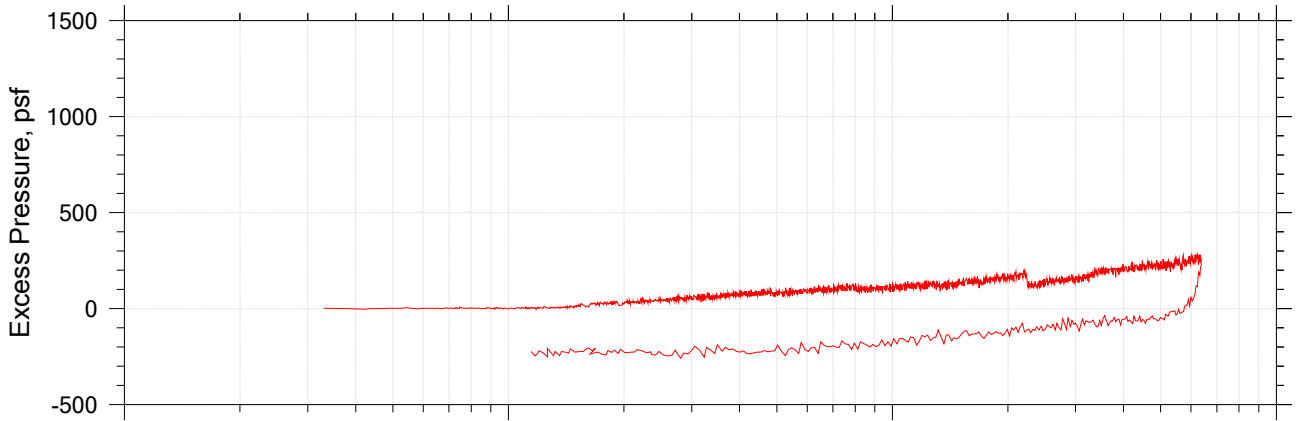
Summary




	Project Name: Waggon Bridge, Wilson Stream	Location: Monmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-102	Tester: md	Checker: njh
	Sample Number: U-1	Test Date: 11/03/22	Depth: 19-21'
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Wet, gray silt		
	Remarks: TX-008		

CRC Test

Pressure Curves




	Project Name: Waggon Bridge, Wilson Stream	Location: Monmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-102	Tester: md	Checker: njh
	Sample Number: U-1	Test Date: 11/03/22	Depth: 19-21'
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Wet, gray silt		
	Remarks: TX-008		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.70 (Estimated)	Liquid Limit: 46
Specimen Height, in: 1.00	Initial Void Ratio: 1.46	Plastic Limit: 28
Final Height, in: 0.68	Final Void Ratio: 0.671	Plasticity Index: 18

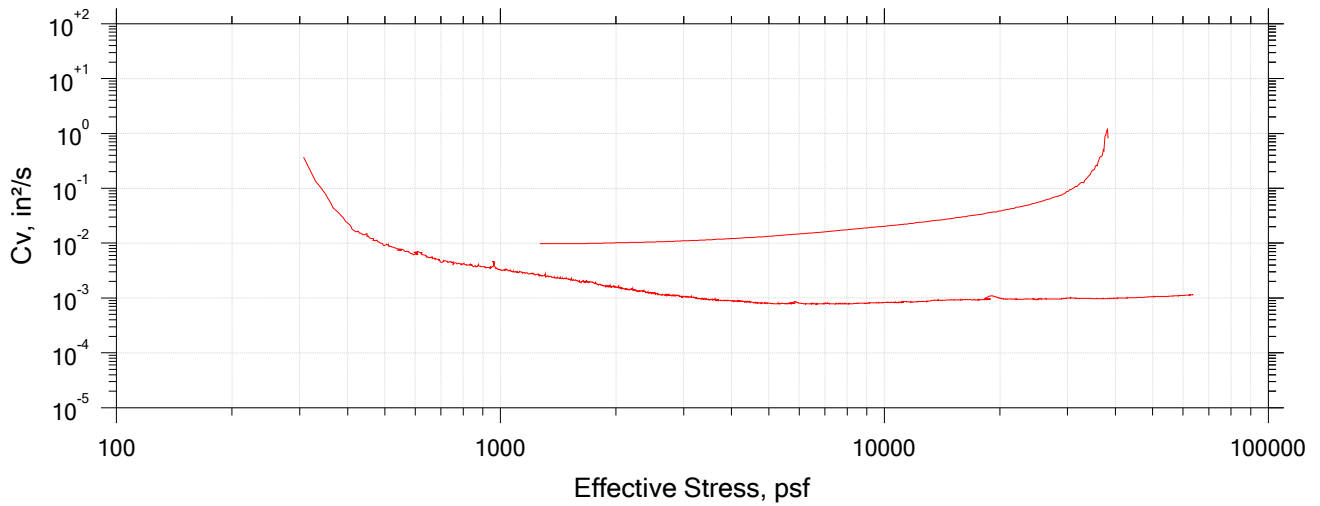
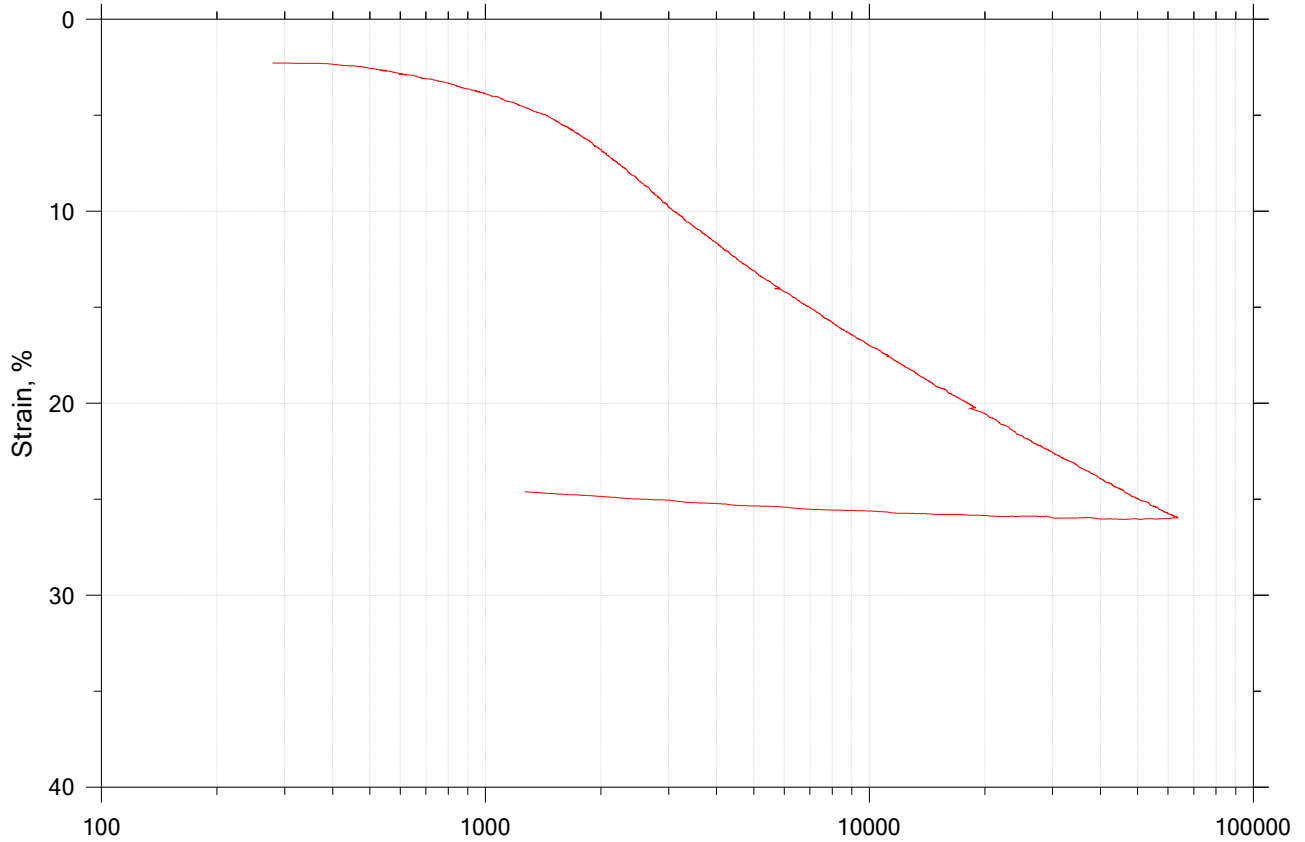
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-4873	---		E4256
Mass Container, gm	8.19	110.37	110.37	8.29
Mass Container + Wet Soil, gm	223.52	245.35	220.57	122.17
Mass Container + Dry Soil, gm	151.86	198.6	198.6	99.47
Mass Dry Soil, gm	143.67	88.234	88.234	91.18
Water Content, %	49.88	52.98	24.90	24.90
Void Ratio	---	1.46	0.67	---
Degree of Saturation, %	---	97.99	100.00	---
Dry Unit Weight, pcf	---	68.477	100.7	---


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 Name: Waggon Bridge, Wilson Stream	Location: Monmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-102	Tester: md	Checker: njh
	Sample Number: U-1	Test Date: 11/03/22	Depth: 19-21'
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Wet, gray silt		
	Remarks: TX-008		

CRC Test

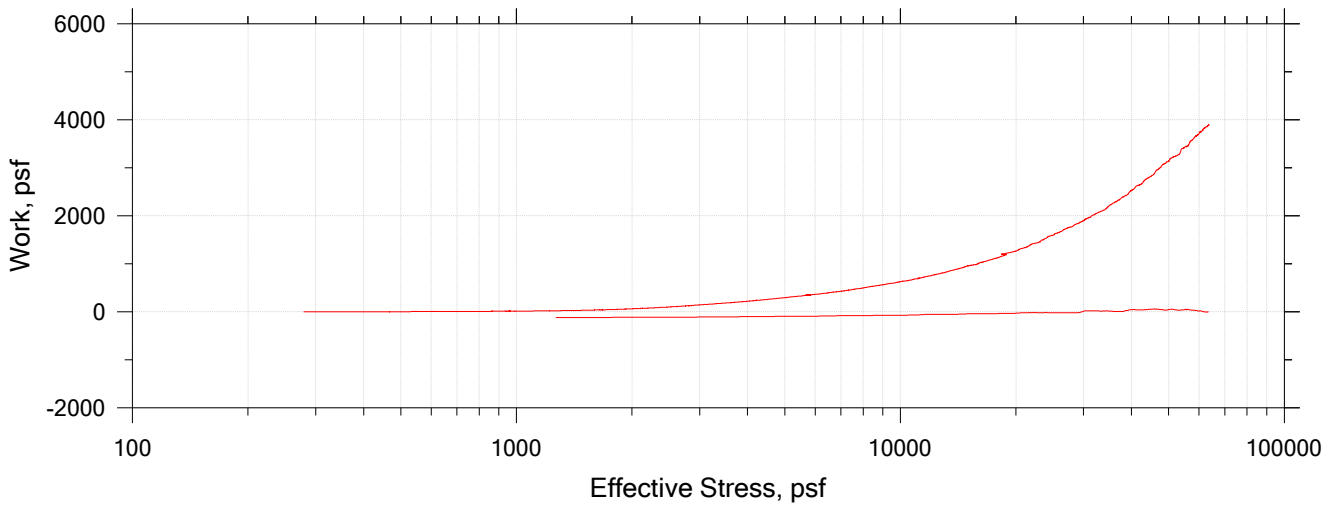
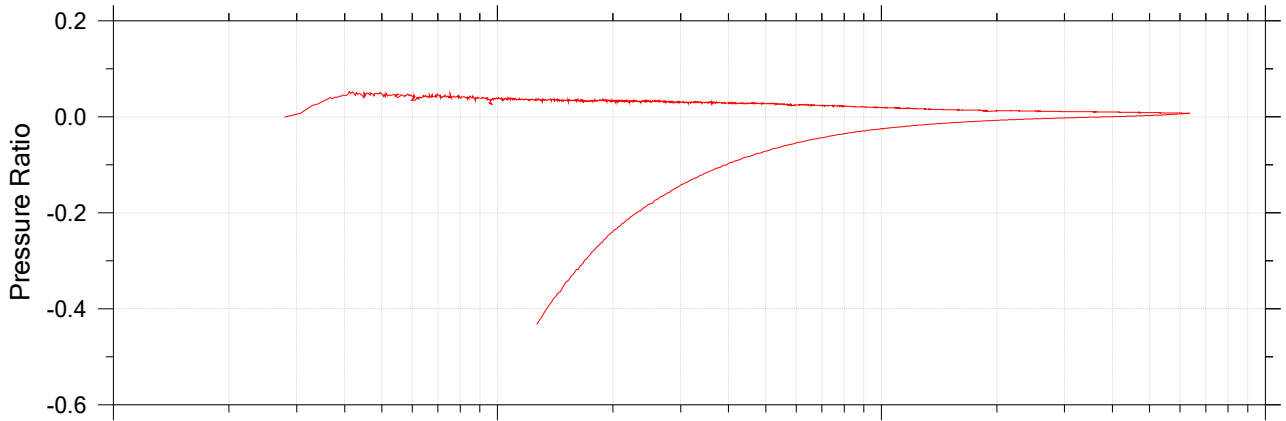
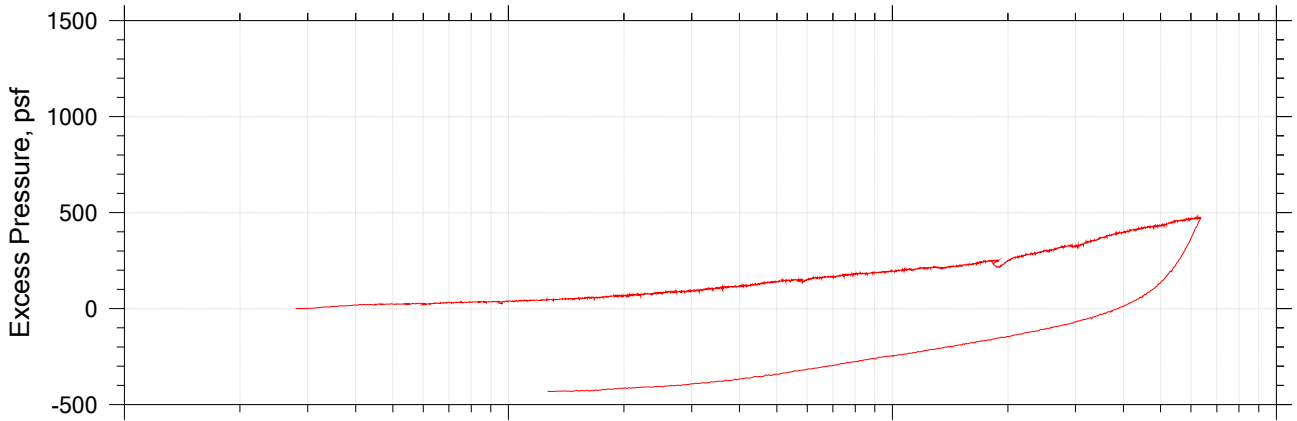
Summary




	Project Name: Waggon Bridge, Wilson Stream	Location: Monmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-102	Tester: md	Checker: njh
	Sample Number: U-2	Test Date: 11/03/22	Depth: 29-31'
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System JJ		

CRC Test

Pressure Curves




	Project Name: Waggon Bridge, Wilson Stream	Location: Monmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-102	Tester: md	Checker: njh
	Sample Number: U-2	Test Date: 11/03/22	Depth: 29-31'
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System JJ		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.73 (Estimated)	Liquid Limit: 31
Specimen Height, in: 1.00	Initial Void Ratio: 1.07	Plastic Limit: 21
Final Height, in: 0.77	Final Void Ratio: 0.592	Plasticity Index: 10

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-6069	---		E1161
Mass Container, gm	8.3	108.56	108.56	8.47
Mass Container + Wet Soil, gm	272.21	255.36	237.68	139.2
Mass Container + Dry Soil, gm	199.54	214.65	214.65	115.88
Mass Dry Soil, gm	191.24	106.09	106.09	107.41
Water Content, %	38.00	38.38	21.71	21.71
Void Ratio	---	1.07	0.59	---
Degree of Saturation, %	---	98.02	100.00	---
Dry Unit Weight, pcf	---	82.332	106.93	---

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 Name: Waggon Bridge, Wilson Street	Location: Monmouth, ME	Project Number: GTX-316327
	Boring Number: BB-MWS-102	Tester: md	Checker: njh
	Sample Number: U-2	Test Date: 11/03/22	Depth: 29-31'
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System JJ		

APPENDIX 2

CALCULATIONS

SOIL PARAMETERS



For	Maine DOT - Waggon	Job No	72998	Sheet No	
Made by	MLR	Check	JZ	Backchk	MLR
Date	10/28/2022	Date	5/20/2025	Date	5/20/2025

Soil Design Properties

South Abutment - Boring BB-MW-102

Layer	Top of Layer Elevation (ft)	Bottom of Layer Elevation (ft)	Depth to Top of Layer (ft)	Depth to Bot. of Layer (ft)	N (bpf)	N ₆₀ (bpf)	N ₁₆₀ (bpf)	Unit Weight γ (pcf)	Friction Angle ϕ' (deg)	Undrained Shear Strength c^* (psf)	e ₅₀ Major Principal Strain @50% (-)	K (Above GWT) (pci)	K (Below GWT) (pci)	Torsional Shear Stress (psf)	G (ksf)	E (ksf)
Layer 1 - Fill	173.4	165.2	0.0	8.2	5	5	8	111	31	626	-	71	46	125	131	318
Layer 3 - Marine Silt and Clay Crust	165.2	159.0	8.2	14.4	5	5	7	115	-	600	0.012	-	-	450	135	324
Layer 4 - Marine Silt and Clay	159.0	127.0	14.4	46.4	2	1	1	110	-	137-536	0.016	-	-	340	68	162
Layer 5 - Glacial Till	127.0	115.6	46.4	57.8	65	92	49	126	40	-	-	233	117	1200	465	1289

*Cohesion estimated from Vane Shear Test performed on New Borings.

North Abutment - Boring BB-MW-101

Layer	Top of Layer Elevation (ft)	Bottom of Layer Elevation (ft)	Depth to Top of Layer (ft)	Depth to Bot. of Layer (ft)	N (bpf)	N ₆₀ (bpf)	N ₁₆₀ (bpf)	Unit Weight γ (pcf)	Friction Angle ϕ' (deg)	Undrained Shear Strength c^* (psf)	e ₅₀ Major Principal Strain @50% (-)	K (Above GWT) (pci)	K (Below GWT) (pci)	Torsional Shear Stress (psf)	G (ksf)	E (ksf)
Layer 1 - Fill	173.0	169.7	0.0	3.3	5	5	8	111	31	-	-	71	46	80	131	318
Layer 2 - Alluvium	169.7	160.0	3.3	13.0	4	4	5	108	30	-	-	52	37	200	121	290
Layer 4 - Marine Silt and Clay	160.0	133.0	13.0	40.0	2	1	1	110	-	137-536	0.016	-	-	330	68	162
Layer 5 - Glacial Till	133.0	126.0	40.0	47.0	65	92	49	126	40	-	-	233	117	1200	465	1289

*Cohesion estimated from Vane Shear Test performed on New Borings.

Pile Models	Lateral	Axial	Tip
Layer 1	Sand (Reese)	Driven Pile (McVay)	Driven Pile (McVay)
Layer 2	Sand (Reese)	Driven Pile (McVay)	Driven Pile (McVay)
Layer 3	Clay (Soft; Matlock)	Driven Pile (McVay)	Driven Pile (McVay)
Layer 4	Clay (Soft; Matlock)	Driven Pile (McVay)	Driven Pile (McVay)
Layer 4	Sand (Reese)	Driven Pile (McVay)	Driven Pile (McVay)

Pile Tip Resistance:	Note 9	kips
GWT Elev.	170.59	ft

Q1.1 Water Elevation



For	Maine DOT - Waggon	Job No	72998	Sheet No	
Made by	MLR	Check	JZ	Backchk	MLR
Date	10/28/2022	Date	11/23/2022	Date	11/23/2022

References:

1. Bridge Software Institute - University of Florida. *FB-MultiPier Soil Parameter Table* . May 2011.
2. EnSoft Inc., LPile version 2016.9.26 User Manual
3. AASHTO. *LRFD Bridge Design Specifications* . Ninth Ed. 2020

Notes:

1. Friction angle and unit weight, determined per SPT evaluation and correlations provided in Reference 1.
2. Undrained shear strength of soils based on field vane testing performed in borings.
3. ϵ_{50} and ϵ_{100} determined as per reference 1 from undrained shear strength data.
4. k determined as per reference 1.
5. ν = Poisson's Ratio determined from Ref. 3:AASHTO- Table C10.4.6.3-1, pg. 10-20
6. E = Young's Modulus from SPT N values and Ref. 3 AASHTO Table C10.4.6.3-1, pg 10-20.
7. G = Shear Modulus determined as per $E / 2(1+\nu)$
8. Unit Skin Friction from Ensoft APILE model based on Friction Angles and undrained shear strengths summarized. Assumed HP14x73 sized piles.
9. Piles are presumed to tip on rock layer during driving, refer to rock properties. Piles are to be modeled as tipping on rock with no embedment into it.



For	Maine DOT - Waggon	Job No	72998	Sheet No	
Made by	MLR	Check	JZ	Backchk	MLR
Date	5/20/2025	Date	5/20/2025	Date	5/20/2025



Rock Design Parameters

Strata	Description	Thickness	Condition	γ_m (pcf)	RQD (%)	q_u (psf)	E_m (ksi)	K (dim)	G (ksi)	Poisson's (dim)	Torsional Shear Stress (psf)	Tip Resistance (kips)
Layer 6	Granofels Bedrock	-	Best Estimate	150	25	1330000	651	0.005	271	0.2	2000	626

Where:

Rock

- γ_m = Moist Unit Weight of Rock
- RQD = Rock Quality Designation of cores
- GSI = Geological Strength Index for Jointed Rocks
- q_u = Unconfined Compressive Strength of Rock
- E_i = Elastic Modulus of Intact Rock
- E_m = Elastic Modulus of Rock Mass
- u = Poisson's Ratio of Rock
- G = Shear Modulus of Rock

Pile Models	Lateral	Axial	Tip
Layer 6	Weak Rock (Reese)	Driven Pile (McVay)	Driven Pile (McVay)

GSI	40
-----	----

Notes:

1. The Unit Weight, γ_m , is based on past experience with similar rock. No rock testing performed for this project.
2. The Uniaxial Compressive Strength, q_u , is based on past experience with similar rock. No rock testing performed for this project. Assume 10,000 psi.
3. The Elastic Modulus of Rock Mass, E_m , and Poisson's Ratio, u, are based on past experience with similar rock and AASHTO LRFD 2020 (Table 10.4.6.5-1). No rock testing performed for this project.
4. Shear Modulus, G, is equal to $E_m / 2(1+u)$.
5. RQD was based on visual field classification by the inspector. GSI was based upon AASHTO LRFD 2020 (Figure 10.4.6.4-1).
6. Elastic Modulus of Rock Mass, E_m , was based upon AASHTO LRFD 2015 (Table 10.4.6.5-1). For E_m , upperbound value is minimum of the average plus 1STDEV and the maximum value, best estimate is the average, and lower bound is the maximum of the average minus 1STDEV and the minimum value.
7. Tip resistance of rock is based on the factored structural resistance of a uncorroded HP 14x73 pile with a resistance factor of 0.65 predicated on the fact that PDA testing will be performed. $\phi P_n = \phi \cdot A \cdot F_y$

SEISMIC SITE CLASS

GLOBAL STABILITY

Project: Waggon Bridge		Job No: 67328		Design Criteria Document:	
Client: MaineDOT		Discipline: Geotech		Calculation No: 2	
Name or Description of Calculation: Stability Assessment _PDR Level					
Calc. Rev. No.	Originator	Checker	Senior Technical Reviewer (if required)	Confirmation Required (Y/N)	
0	M. Rowicki	J. Zwetchkenbaum			
1	M. Rowicki	J. Zwetchkenbaum			
2	M. Rowicki	J. Zwetchkenbaum			
Calculation Objective:					
Perform a slope stability checks of proposed embankment widening on bridge approaches. Use Slope/W to perform assessment.					
Rev2 incorporated in June 2025 reassessed slopes per latest 60% design plans.					
Calculation Methodology/List of Assumptions:					
<u>General</u>					
<ul style="list-style-type: none"> - Slope/W was utilized to assess settlement using the Spencer analysis type. - Soil parameters and strata elevations were taken from previously summarized parameter sheet. - Side slopes (transverse) as well as the slope through the abutment (longitudinal) were checked. - For the side slope condition, Section at Sta. 25+95.00 and 26+70 were run as the governing conditions on either side of the bridge. Geometric working coordinates were extracted from the cross section and input into the program. - Piezometric line was placed at EL. 170.6 - A live load surcharge of 250 psf is applied to the entire roadway. - Bedrock is modeled as an impenetrable layer. - Both drained and undrained conditions were assessed. - Proposed Fill is assumed to be common borrow, modeled with a unit weight of 125 pcf and an internal friction angle of 34 degrees. Riprap is modeled as a 110 pcf, 42 degree material to a depth of 3 ft. a 24" thick subgrade gravel layer is modeled as a 125 pcf and 41 degree friction angle material. - For the Marine Silt & Clay, Vane Shear values were utilized for the undrained shear strength. As the vane shears (per boring logs) indicated a decrease in strength with depth, the S=f(datum) material model was specified for Su values started at 137 psf and ending at 536 psf at the bottom of the stratum. For the drained condition, a 31.5 degree friction angle was selected as the average of the two triaxial CU tests performed on the Marine Silt & Clay. The same value was conservative used on the crust layer. - Models were also assessed for only the Q1.1 as the lowest water elevation based on discussion with the hydraulics engineer. Non-tidal, dam-controlled waterway. 					
<u>Side Slope Model:</u>					
<ul style="list-style-type: none"> - Riprap added with a thickness of 3' along the entire slope from Elev. 172 ft. as dictated by Maine HNTB office. 					
<u>Abutment Slope Model:</u>					
<ul style="list-style-type: none"> - Abutment were specified as 8' tall and 4' wide per structures direction. It is modeled as a high strength material. 					

- Riprap is added with a 3' thickness along the entire slope with a 1' thick gravel base layer.

References/Inputs:

- See above

Attachments: (List each attachment following the subject calculation)

Attachment 1: Waggon Critical Cross Sections

Attachment 2: Slope/W Model Outputs of Proposed Conditions

- 25+95 Side Slope: Geometry
- 25+95 Side Slope: Left - Undrained
- 25+95 Side Slope: Right – Undrained
- 25+95 Side Slope: Left - Drained
- 25+95 Side Slope: Right – Drained

- 26+70 Side Slope: Geometry
- 26+70 Side Slope: Left - Undrained
- 26+70 Side Slope: Right – Undrained
- 26+70 Side Slope: Left - Drained
- 26+70 Side Slope: Right – Drained

- Longitudinal Section: Left - Undrained
- Longitudinal Section: Right Undrained
- Longitudinal Section: Left - Drained
- Longitudinal Section: Right Drained

Conclusions:Side Slopes:

Results for 2H:1V Slopes:



Location	Undrained Min Slope Stability Factor of Safety (FS)	Drained Min Slope Stability Factor of Safety (FS)
Abut 1 Sta 25+95 (Left)	1.399	1.301
Abut 1 Sta 25+95 (Right)	1.385	1.385
Abut 2 Sta 26+70 (Left)	1.337	1.337
Abut 2 Sta 26+70 (Right)	1.315	1.315

- All slopes meet minimum required FoS =1.3.

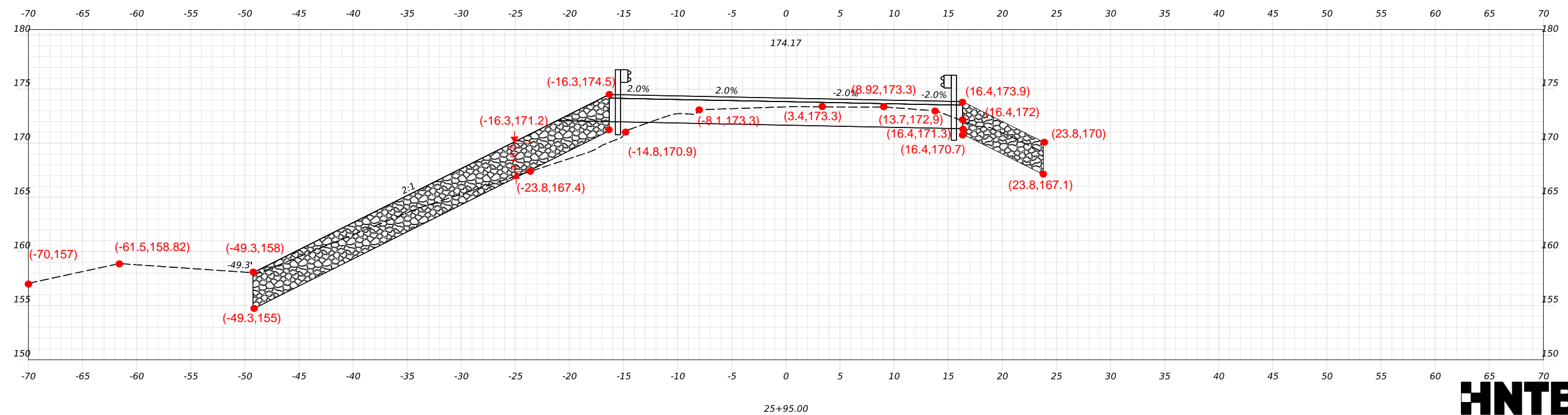
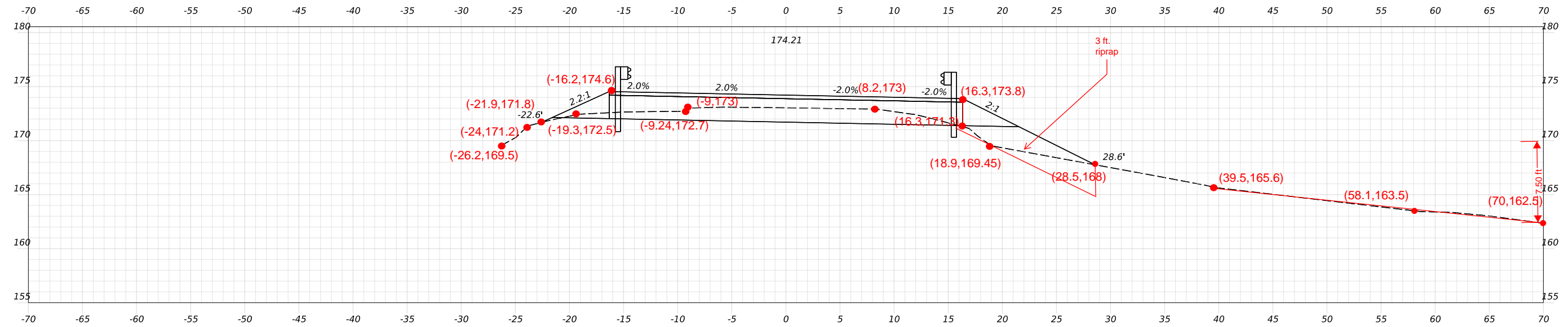
Abutment Slope:

Location	Undrained Min Slope Stability Factor of Safety (FS)	Drained Min Slope Stability Factor of Safety (FS)
Abut 1 (Left)	2.793	1.450
Abut 2 (Right)	2.433	1.970

- Slopes in front of or supporting an existing foundation are to have a minimum required FoS=1.4. One slope condition is less than this as 1.450 however the foundation is being supported on piles are driven to rock and the entire bridge system is working as a frame. There is no concern for a foundation instability caused by a slope failure through the integral abutment piles.

Document Check:	Name	Signature	Date
Originator:	M.Rowicki		6/2/25
Checker:	J. Zwetchkenbaum		6/5/24
BackChecker:	M.Rowicki	-	-
Updater:	No comment	-	-
Verifier:	No comment	-	-

Coordinates for Slope Stability Models



PROJ. MANAGER	TRACER	BY	DATE
DESIGNED-Detailed	C. Tabin	C. Tabin	05/2025
CHECKED-Reviewed	E. Davidson	J. Olund	05/2025
DESIGNED-Detailed			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

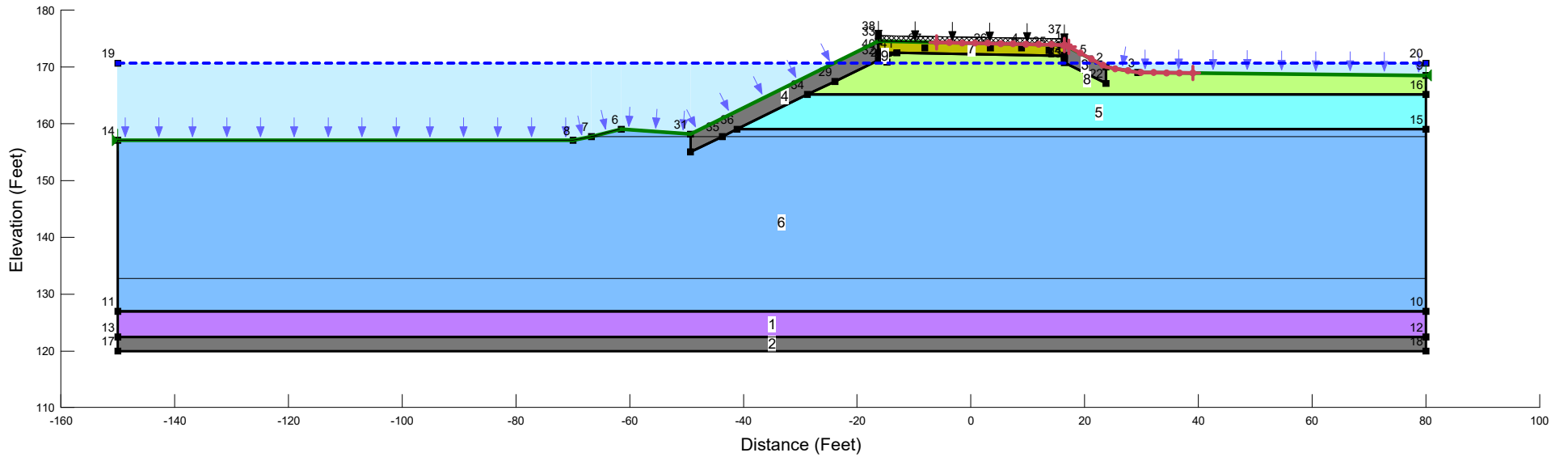
MONMOUTH WAGGON BRIDGE CROSS SECTION



Username: ctobin Date: 5/8/2025

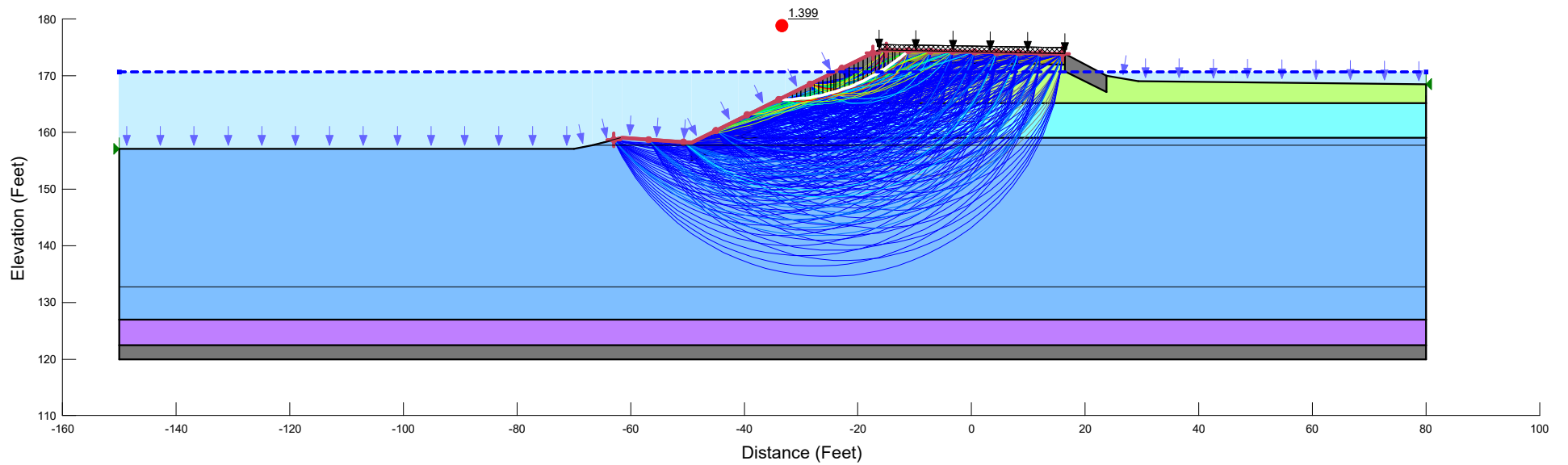
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Grey	Bedrock	Bedrock (Impenetrable)										1
Light Green	Existing Fill	Mohr-Coulomb	111						0	31	0	1
Purple	Glacial Till	Mohr-Coulomb	126						0	40	0	1
Light Blue	Marine Silt and Clay	S=f(datum)	110		536	-16	536	157.8				1
Cyan	Marine Silt and Clay Crust	Undrained (Phi=0)	115	600								1
Yellow-Green	Pavement & Subgrade	Mohr-Coulomb	125						0	42	0	1
Pink	Proposed Fill	Mohr-Coulomb	125						0	34	0	1
Dark Grey	Riprap	Mohr-Coulomb	110						0	42	0	1

Name: Slope Stability - Right
File Name: Transverse Section 25+95.00-3 ft Riprap_2to1_Undrained.gsz



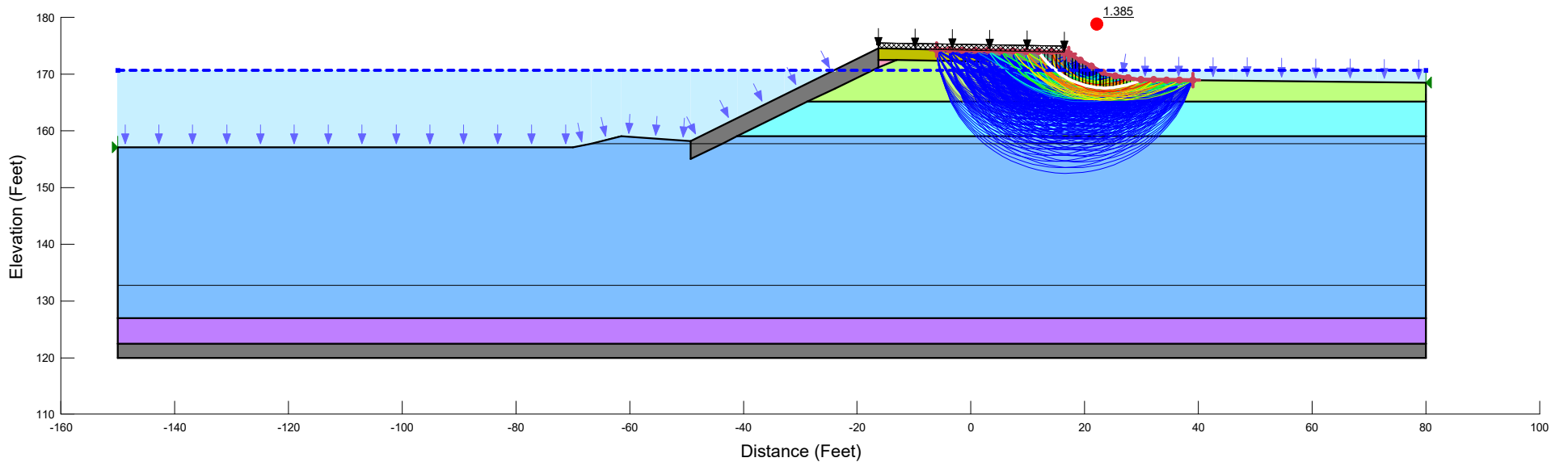
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■	Bedrock	Bedrock (Impenetrable)										1
■	Existing Fill	Mohr-Coulomb	111						0	31	0	1
■	Glacial Till	Mohr-Coulomb	126						0	40	0	1
■	Marine Silt and Clay	S=f(datum)	110		536	-16	536	157.8				1
■	Marine Silt and Clay Crust	Undrained (Phi=0)	115	600								1
■	Pavement & Subgrade	Mohr-Coulomb	125						0	42	0	1
■	Proposed Fill	Mohr-Coulomb	125						0	34	0	1
■	Riprap	Mohr-Coulomb	110						0	42	0	1

Name: Slope Stability - Left
File Name: Transverse Section 25+95.00-3 ft Riprap_2to1_Undrained.gsz



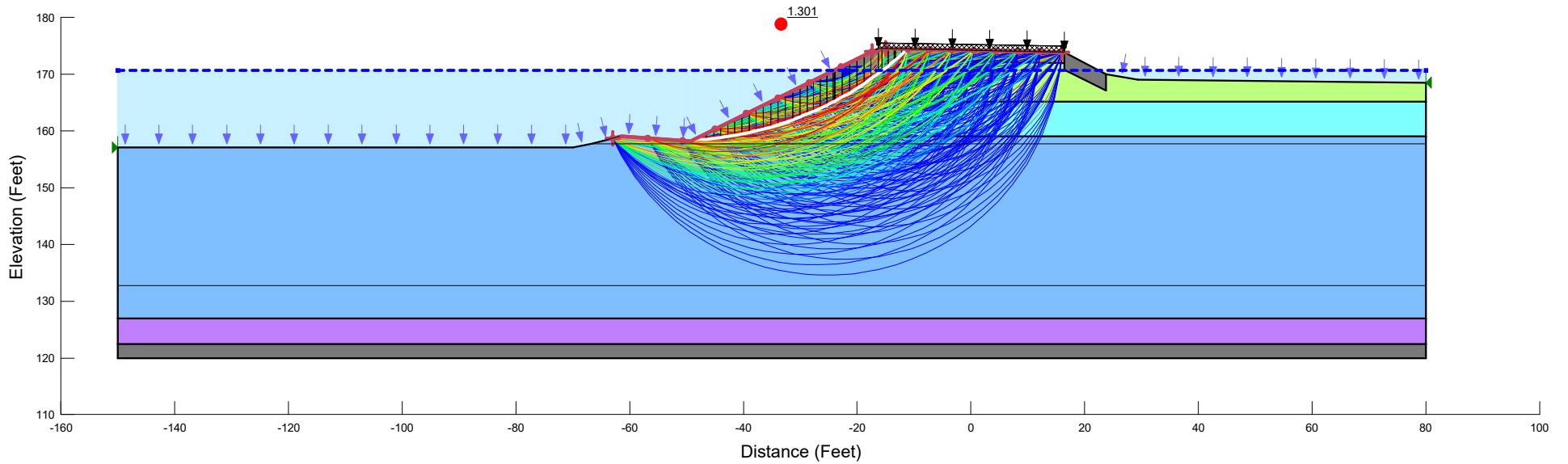
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■	Bedrock	Bedrock (Impenetrable)										1
■	Existing Fill	Mohr-Coulomb	111						0	31	0	1
■	Glacial Till	Mohr-Coulomb	126						0	40	0	1
■	Marine Silt and Clay	S=f(datum)	110		536	-16	536	157.8				1
■	Marine Silt and Clay Crust	Undrained (Phi=0)	115	600								1
■	Pavement & Subgrade	Mohr-Coulomb	125						0	42	0	1
■	Proposed Fill	Mohr-Coulomb	125						0	34	0	1
■	Riprap	Mohr-Coulomb	110						0	42	0	1

Name: Slope Stability - Right
File Name: Transverse Section 25+95.00-3 ft Riprap_2to1_Undrained.gsz



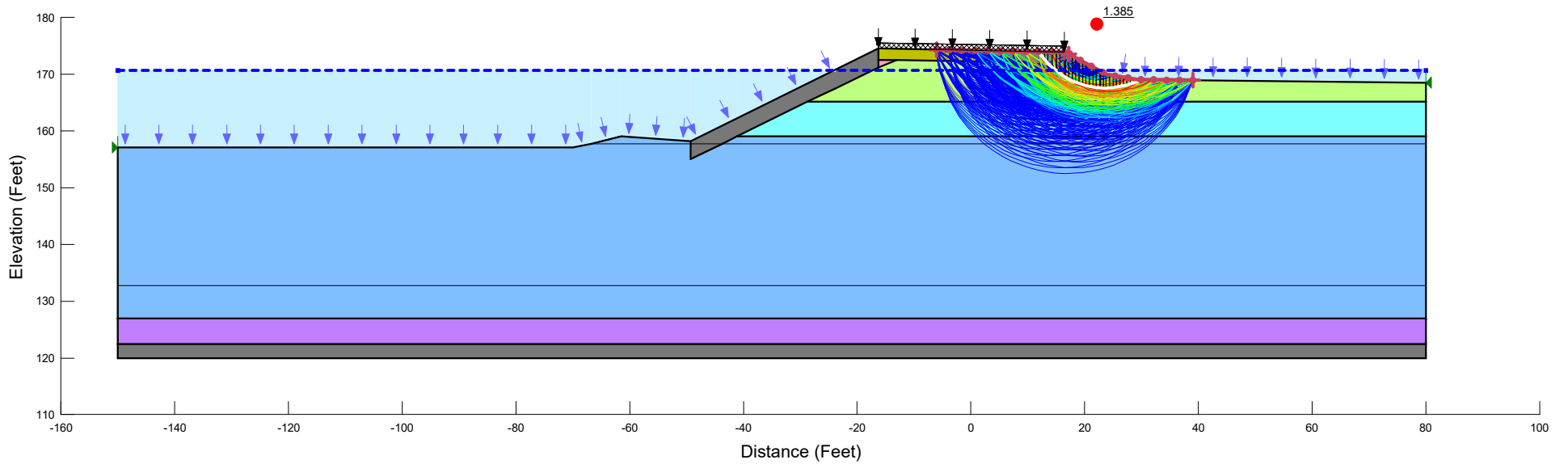
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■	Bedrock	Bedrock (Impenetrable)					1
■	Existing Fill	Mohr-Coulomb	111	0	31	0	1
■	Glacial Till	Mohr-Coulomb	126	0	40	0	1
■	Marine Silt and Clay	Mohr-Coulomb	110	0	31.5	0	1
■	Marine Silt and Clay Crust	Mohr-Coulomb	115	0	31.5	0	1
■	Pavement & Subgrade	Mohr-Coulomb	125	0	42	0	1
■	Proposed Fill	Mohr-Coulomb	125	0	34	0	1
■	Riprap	Mohr-Coulomb	110	0	42	0	1

Name: Slope Stability - Left
File Name: Transverse Section 25+95.00-3 ft Riprap_2to1_Drained.gsz



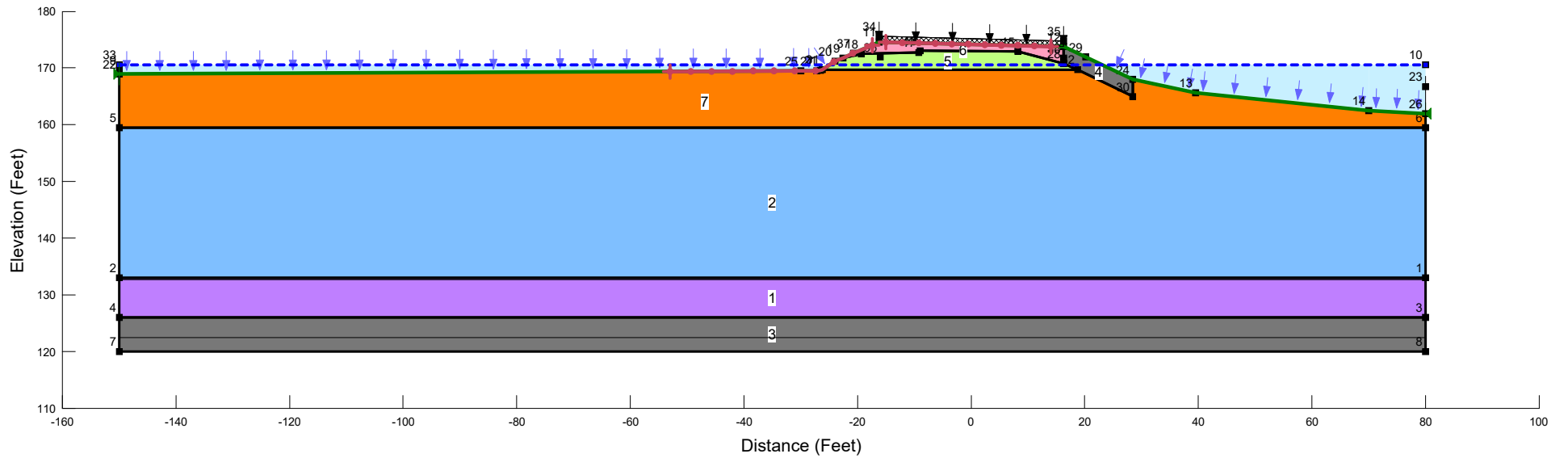
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Grey	Bedrock	Bedrock (Impenetrable)					1
Light Green	Existing Fill	Mohr-Coulomb	111	0	31	0	1
Purple	Glacial Till	Mohr-Coulomb	126	0	40	0	1
Light Blue	Marine Silt and Clay	Mohr-Coulomb	110	0	31.5	0	1
Cyan	Marine Silt and Clay Crust	Mohr-Coulomb	115	0	31.5	0	1
Olive Green	Pavement & Subgrade	Mohr-Coulomb	125	0	42	0	1
Pink	Proposed Fill	Mohr-Coulomb	125	0	34	0	1
Dark Grey	Riprap	Mohr-Coulomb	110	0	42	0	1

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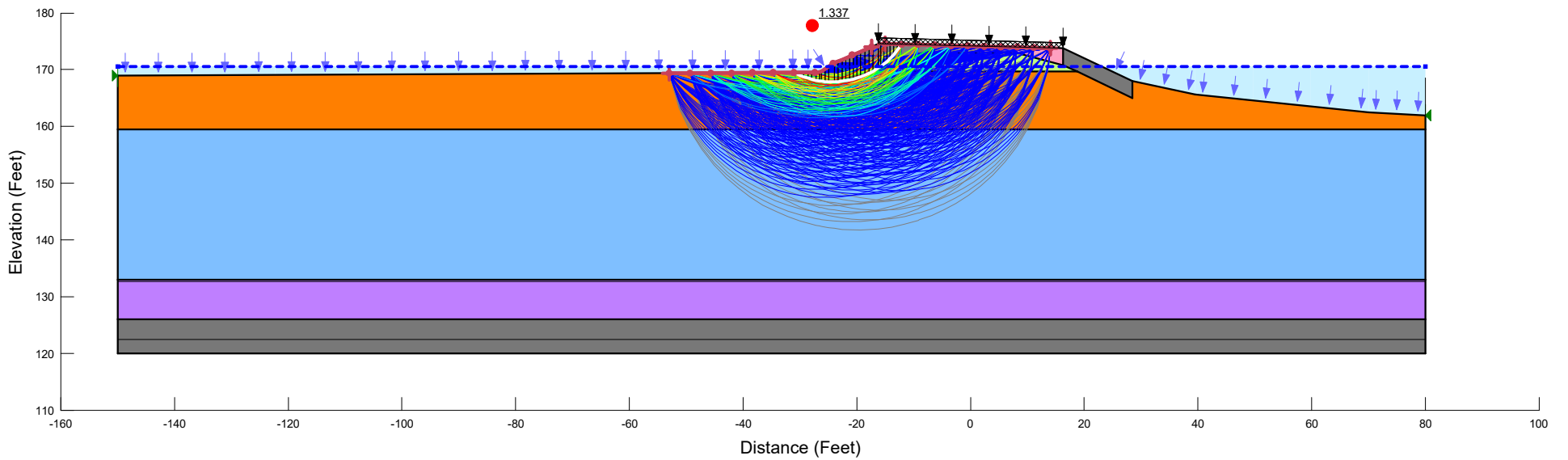
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Orange	Alluvium	Mohr-Coulomb	108	0	30	0	1
Dark Grey	Bedrock	Bedrock (Impenetrable)					1
Light Green	Existing Fill	Mohr-Coulomb	111	0	31	0	1
Purple	Glacial Till	Mohr-Coulomb	126	0	40	0	1
Light Blue	Marine Silt and Clay	Mohr-Coulomb	110	0	31.5	0	1
Pink	Proposed Fill	Mohr-Coulomb	125	0	34	0	1 </td
Dark Grey	Riprap	Mohr-Coulomb	110	0	42	0	1

Name: Slope Stability - Left
File Name: Transverse Section 26+70.00_3 ft Riprap_Drained.gsz



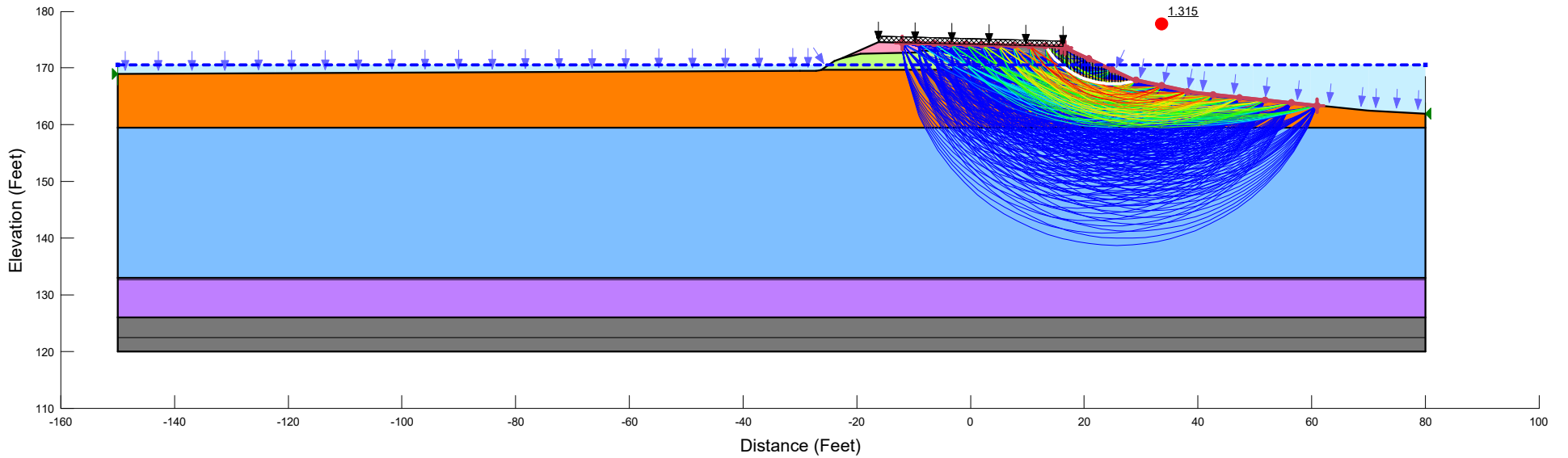
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Orange	Alluvium	Mohr-Coulomb	108					0	30	0	1
Dark Gray	Bedrock	Bedrock (Impenetrable)									1
Light Green	Existing Fill	Mohr-Coulomb	111					0	31	0	1
Purple	Glacial Till	Mohr-Coulomb	126					0	40	0	1
Blue	Marine Silt and Clay	S=f(datum)	110	536	-16	536	157.8				1
Pink	Proposed Fill	Mohr-Coulomb	125					0	34	0	1
Dark Gray	Riprap	Mohr-Coulomb	110					0	42	0	1

Name: Slope Stability - Left
File Name: Transverse Section 26+70.00_3 ft Riprap_Undrained.gsz



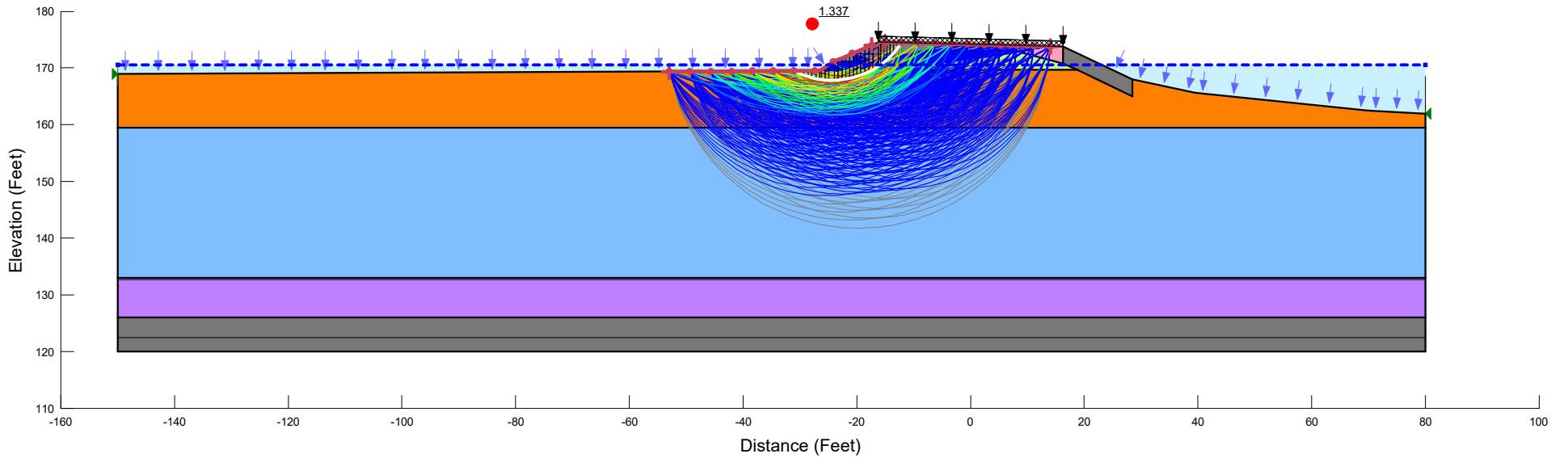
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Orange	Alluvium	Mohr-Coulomb	108					0	30	0	1
Grey	Bedrock	Bedrock (Impenetrable)									1
Light Green	Existing Fill	Mohr-Coulomb	111					0	31	0	1
Purple	Glacial Till	Mohr-Coulomb	126					0	40	0	1
Blue	Marine Silt and Clay	S=f(datum)	110	536	-16	536	157.8				1
Pink	Proposed Fill	Mohr-Coulomb	125					0	34	0	1
Dark Grey	Riprap	Mohr-Coulomb	110					0	42	0	1

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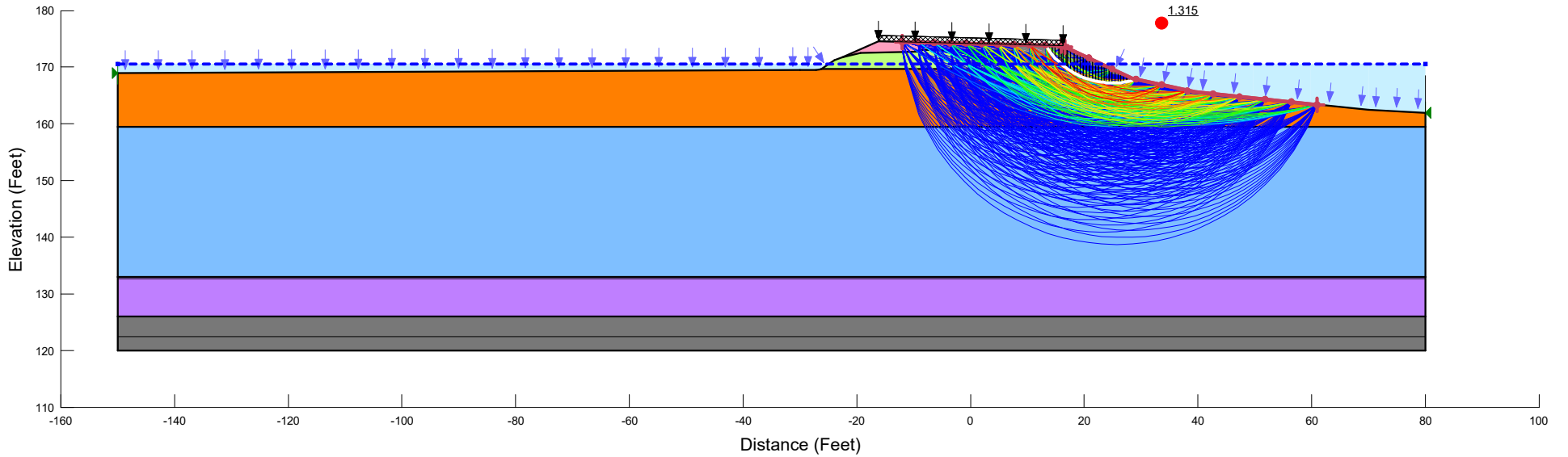
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Orange	Alluvium	Mohr-Coulomb	108					0	30	0	1
Dark Gray	Bedrock	Bedrock (Impenetrable)									1
Light Green	Existing Fill	Mohr-Coulomb	111					0	31	0	1
Purple	Glacial Till	Mohr-Coulomb	126					0	40	0	1
Blue	Marine Silt and Clay	S=f(datum)	110	536	-16	536	157.8				1
Pink	Proposed Fill	Mohr-Coulomb	125					0	34	0	1
Dark Gray	Riprap	Mohr-Coulomb	110					0	42	0	1







Name: Slope Stability - Left
File Name: Transverse Section 26+70.00_3 ft Riprap_Undrained.gsz



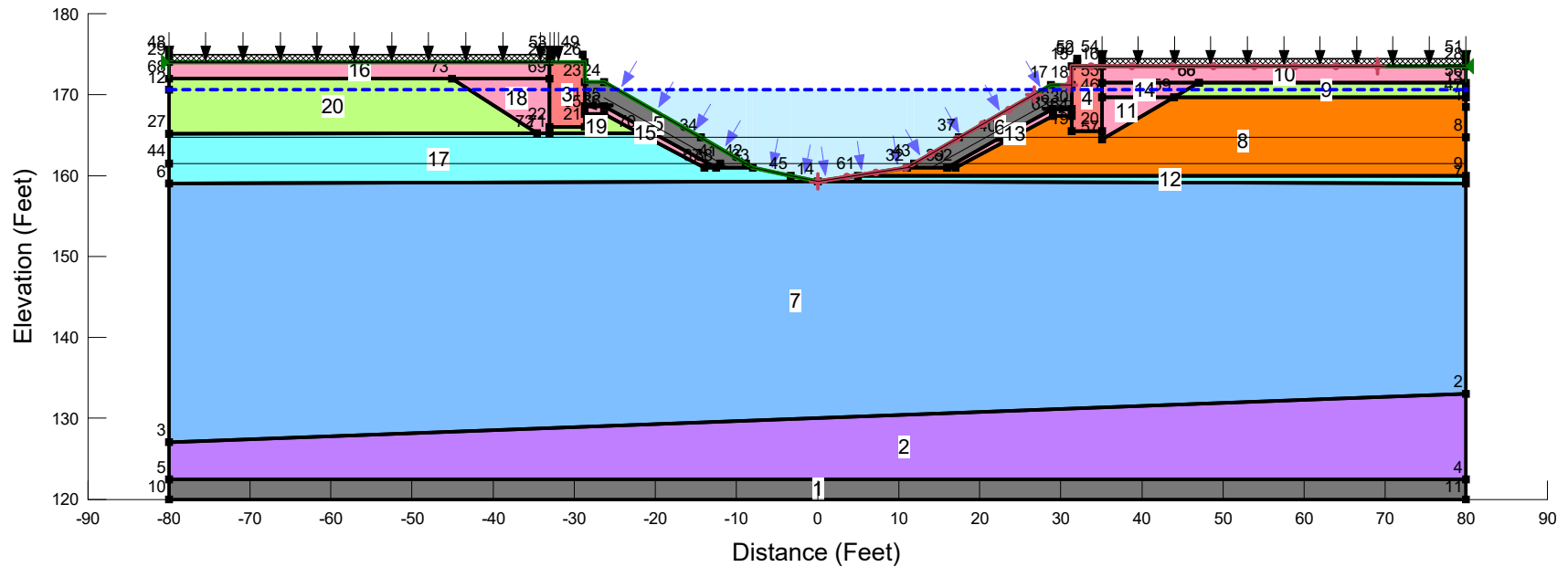
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Orange	Alluvium	Mohr-Coulomb	108					0	30	0	1
Grey	Bedrock	Bedrock (Impenetrable)									1
Light Green	Existing Fill	Mohr-Coulomb	111					0	31	0	1
Purple	Glacial Till	Mohr-Coulomb	126					0	40	0	1
Blue	Marine Silt and Clay	S=f(datum)	110	536	-16	536	157.8				1
Pink	Proposed Fill	Mohr-Coulomb	125					0	34	0	1
Dark Grey	Riprap	Mohr-Coulomb	110					0	42	0	1










Name: Slope Stability - Right
File Name: Transverse Section 26+70.00_3 ft Riprap_Undrained.gsz



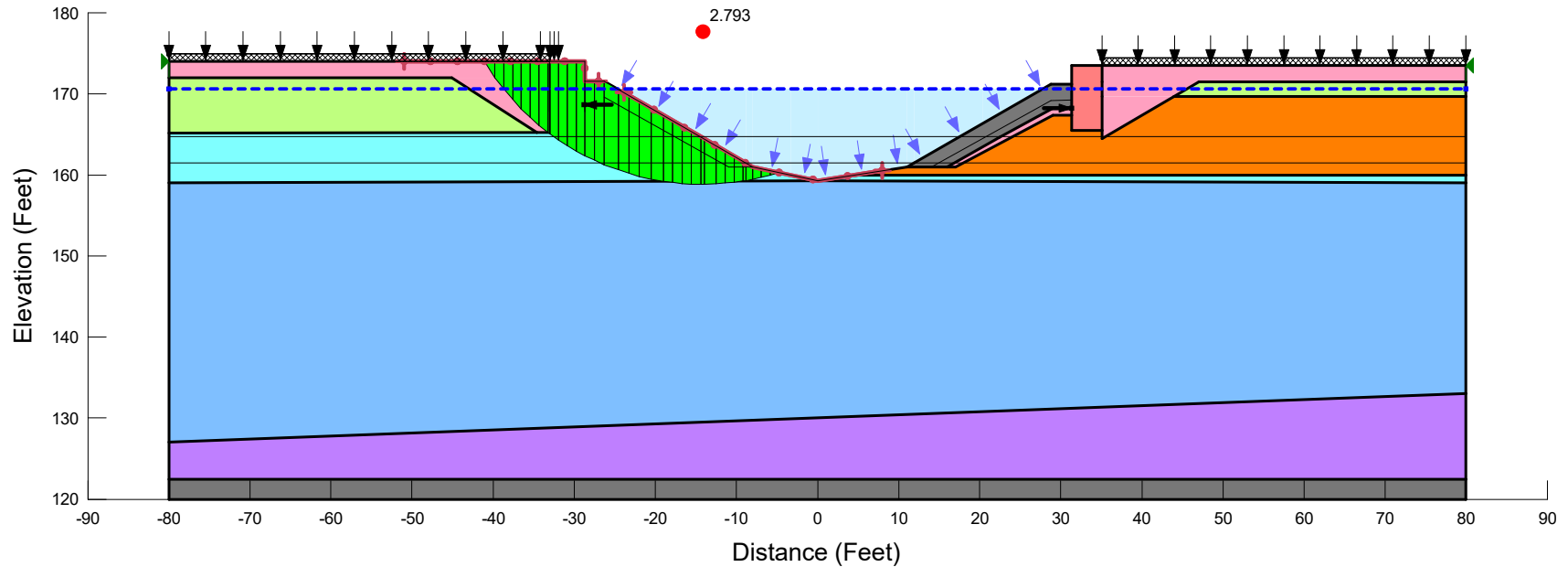
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	Abutment	High Strength	150								1
	Alluvium	Mohr-Coulomb	108					0	30	0	1
	Bedrock	Bedrock (Impenetrable)									1
	Existing Fill	Mohr-Coulomb	111					0	31	0	1
	Glacial Till	Mohr-Coulomb	126					0	40	0	1
	Marine Silt and Clay	S=f(datum)	110	536	-16	536	157.8				1
	Marine Silt and Clay Crust	Mohr-Coulomb	115					600	0	0	1
	Proposed Fill	Mohr-Coulomb	125					0	32	0	1
	Riprap	Mohr-Coulomb	110					0	42	0	1










Name: Slope Stability-Right
File Name: Longitudinal Section_Modified Final_June2025_Undrained.gsz



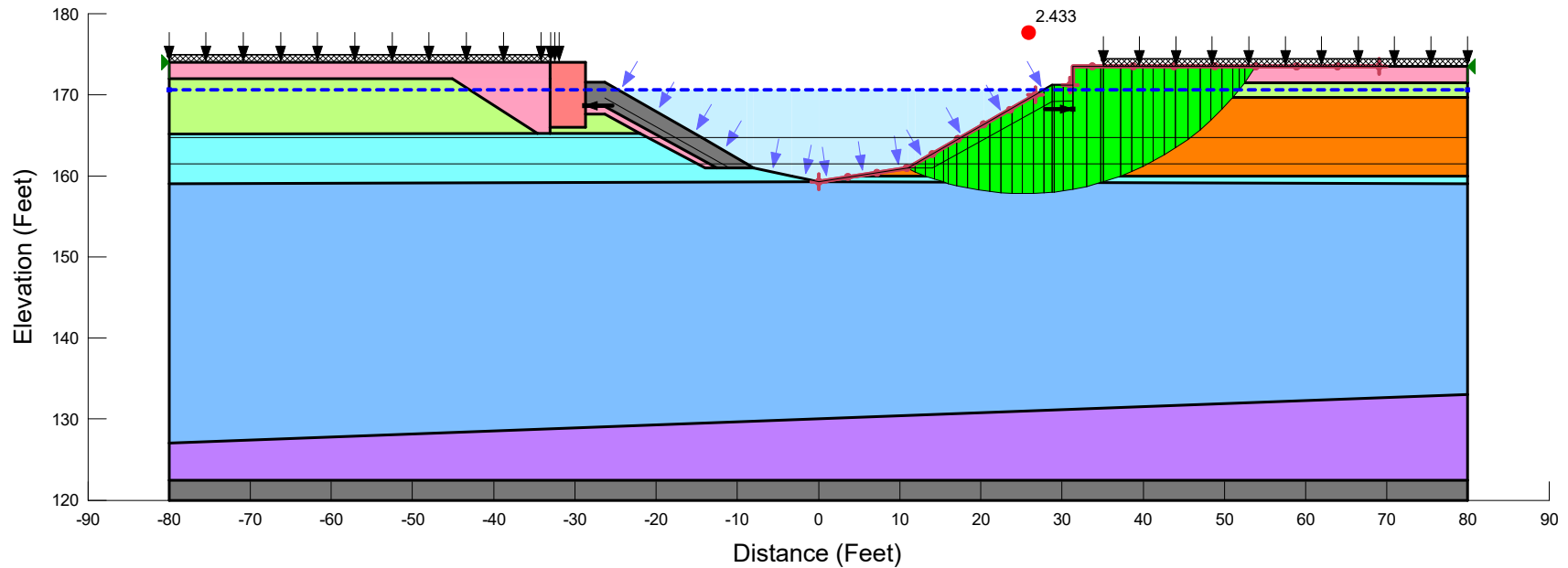
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	Abutment	High Strength	150								1
	Alluvium	Mohr-Coulomb	108					0	30	0	1
	Bedrock	Bedrock (Impenetrable)									1
	Existing Fill	Mohr-Coulomb	111					0	31	0	1
	Glacial Till	Mohr-Coulomb	126					0	40	0	1
	Marine Silt and Clay	S=f(datum)	110	536	-16	536	157.8				1
	Marine Silt and Clay Crust	Mohr-Coulomb	115					600	0	0	1
	Proposed Fill	Mohr-Coulomb	125					0	32	0	1
	Riprap	Mohr-Coulomb	110					0	42	0	1










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File Name: Longitudinal Section_Modified Final_June2025_Undrained.gsz



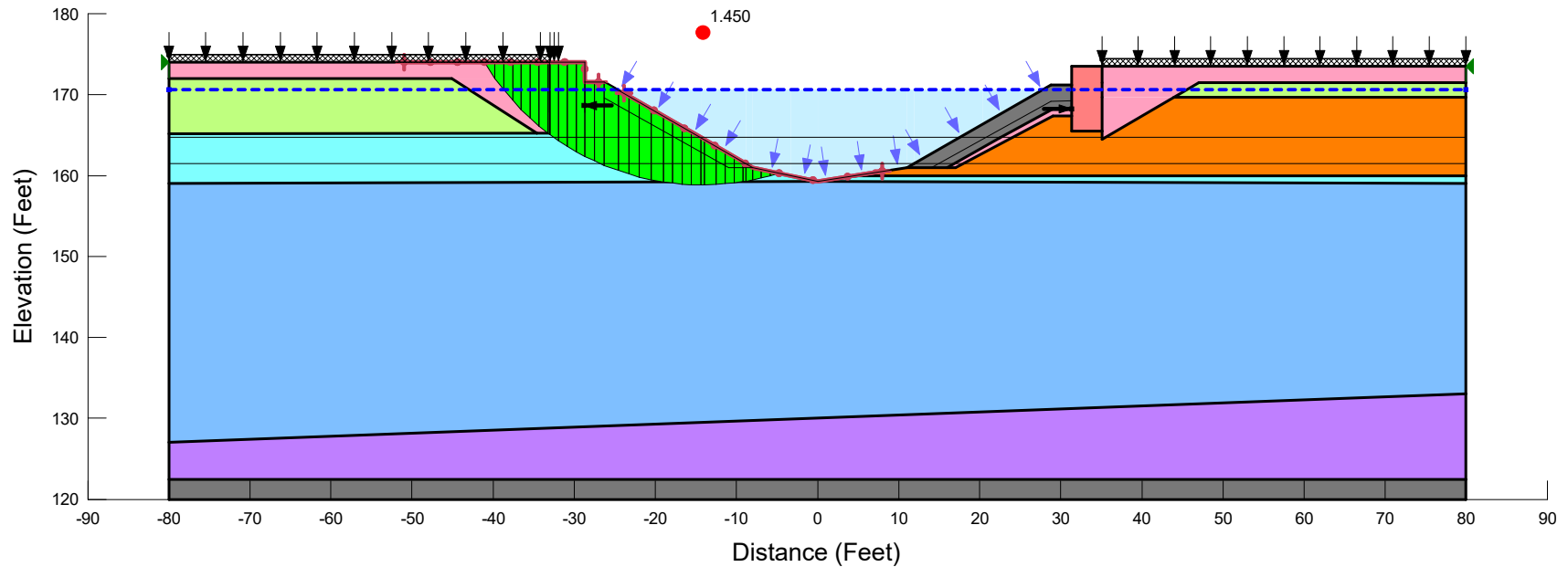
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	Abutment	High Strength	150								1
	Alluvium	Mohr-Coulomb	108					0	30	0	1
	Bedrock	Bedrock (Impenetrable)									1
	Existing Fill	Mohr-Coulomb	111					0	31	0	1
	Glacial Till	Mohr-Coulomb	126					0	40	0	1
	Marine Silt and Clay	S=f(datum)	110	536	-16	536	157.8				1
	Marine Silt and Clay Crust	Mohr-Coulomb	115					600	0	0	1
	Proposed Fill	Mohr-Coulomb	125					0	32	0	1
	Riprap	Mohr-Coulomb	110					0	42	0	1










Name: Slope Stability-Right
File Name: Longitudinal Section_Modified Final_June2025_Undrained.gsz



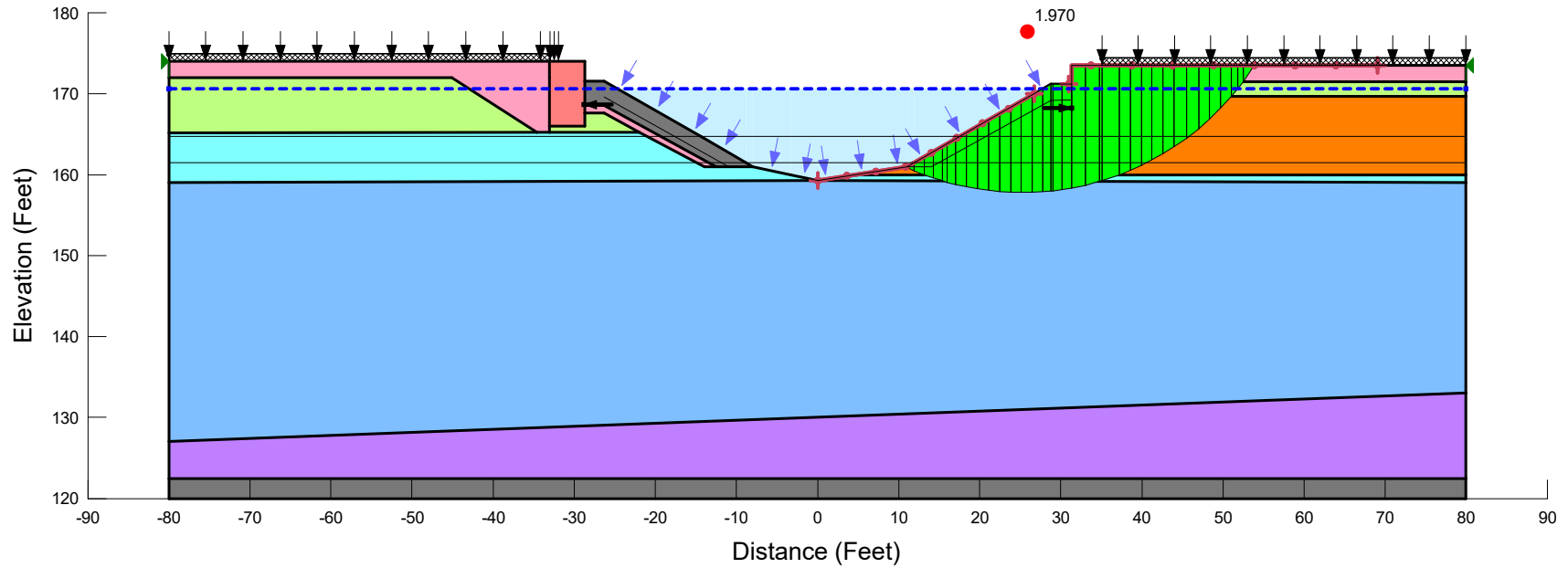
		Material Model	Weight (pcf)	Cohesion (psf)	Friction Angle (°)	(°)	Surface
	Abutment	High Strength	150				1
	Alluvium	Mohr-Coulomb	108	0	30	0	1
	Bedrock	Bedrock (Impenetrable)					1
	Existing Fill	Mohr-Coulomb	111	0	31	0	1
	Glacial Till	Mohr-Coulomb	126	0	40	0	1
	Marine Silt and Clay	Mohr-Coulomb	110	0	31.5	0	1
	Marine Silt and Clay Crust	Mohr-Coulomb	115	0	31.5	0	1
	Proposed Fill	Mohr-Coulomb	125	0	32	0	1
	Riprap	Mohr-Coulomb	110	0	42	0	1

Name: Slope Stability-Left
File Name: Longitudinal Section_Modified Final_June2025_Drained.gsz



		Material Model	Weight (pcf)	Cohesion (psf)	Friction Angle (°)	(°)	Surface
	Abutment	High Strength	150				1
	Alluvium	Mohr-Coulomb	108	0	30	0	1
	Bedrock	Bedrock (Impenetrable)					1
	Existing Fill	Mohr-Coulomb	111	0	31	0	1
	Glacial Till	Mohr-Coulomb	126	0	40	0	1
	Marine Silt and Clay	Mohr-Coulomb	110	0	31.5	0	1
	Marine Silt and Clay Crust	Mohr-Coulomb	115	0	31.5	0	1
	Proposed Fill	Mohr-Coulomb	125	0	32	0	1
	Riprap	Mohr-Coulomb	110	0	42	0	1

Name: Slope Stability-Right
File Name: Longitudinal Section_Modified Final_June2025_Drained.gsz



SETTLEMENT

Project: Waggon Bridge		Job No: 67328		Design Criteria Document:	
Client: MaineDOT		Discipline: Geotech		Calculation No: 2	
Name or Description of Calculation: Settlement Assessment _Final Design					
Calc. Rev. No.	Originator	Checker	Senior Technical Reviewer (if required)	Confirmation Required (Y/N)	
0	M. Rowicki	J. Zwetchkenbaum			
1	M. Rowicki	J. Zwetchkenbaum			
2	M. Rowicki	J. Zwetchkenbaum			
Calculation Objective: Perform a settlement assessment of the proposed grade increase across roadway and widened approaches.					
Calculation Methodology/List of Assumptions: <u>General</u> <ul style="list-style-type: none"> - Settle 3 was utilized to assess anticipated settlement due to proposed fill on critical sections on both sides of the proposed bridge. - Fill was placed on the approaches to the point of where the sheet piles will be driven in previous assessment. - Due to changing of slopes from 1.5:1 to 2:1, stability checks yield favorable results without the need for a sheetpile wall. As a full slope will be utilized, increased settlement magnitudes were checked. - Controlling cross sections were modeled in Settle 3 with the proposed embankment fill divided into specific geometric shapes to assist in the modeling. Refer to marked-up cross sections for geometrical assumptions and layer thicknesses. - Secondary settlement was considered over a 75 year timeline. 					
References/Inputs: <ul style="list-style-type: none"> - Summary of Soil Parameters - Cross Sections provided by Maine HNTB office. 					
Attachments: (List each attachment following the subject calculation) Attachment 1: Cross Section of Proposed Embankment (60% plans) Attachment 2: Settle 3 Model Results					

Conclusions:

Current settlement magnitudes with 2:1 side slopes @ Sta. 25+95.00

Max Settlement at 75 Years (in)				
Location	Left Side of Road	Center of Road	Right Side of Road	Max
Elastic	0.2	<0.1	<0.1	0.2
Consolidation	3.0	2.0	1.1	3.0
Total Settlement	3.4	2.1	1.2	3.4

Total Cumulative Settlement per Stage (in)			
Stage (Years)	Left Side of Road	Center of Road	Right Side of Road
0	0.2	<0.1	<0.1
1	0.7	0.3	0.2
2	0.9	0.4	0.2
5	1.2	0.6	0.3
15	1.7	0.9	0.5
30	2.3	1.3	0.7
45	2.7	1.7	0.9
60	3.1	1.9	1.0
75	3.4	2.1	1.2



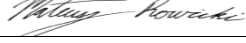


See plots for graphed data.

Current settlement magnitudes with 2:1 side slopes @ Sta. 26+70.00

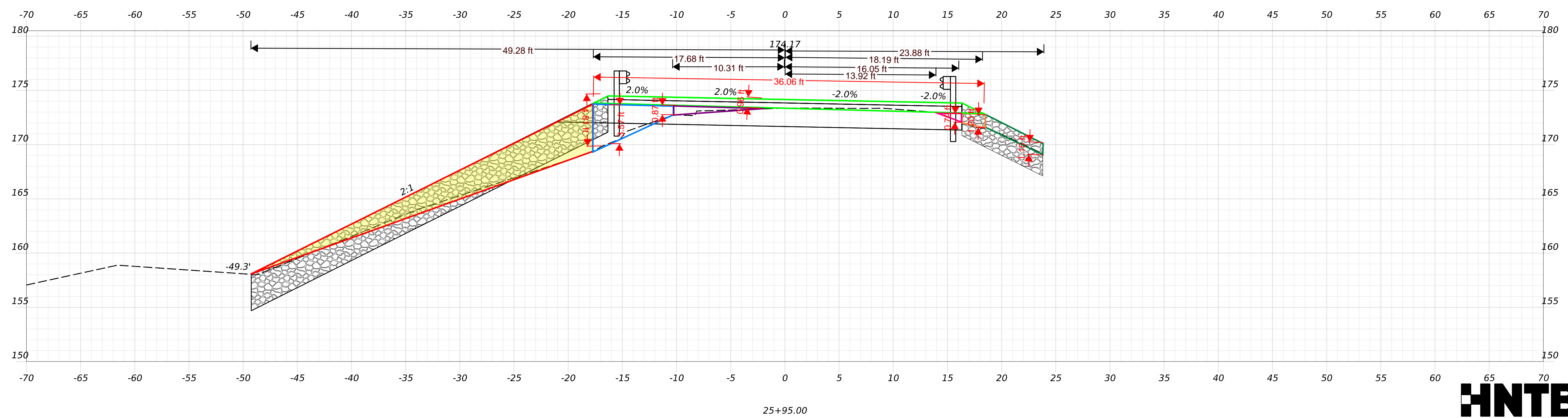
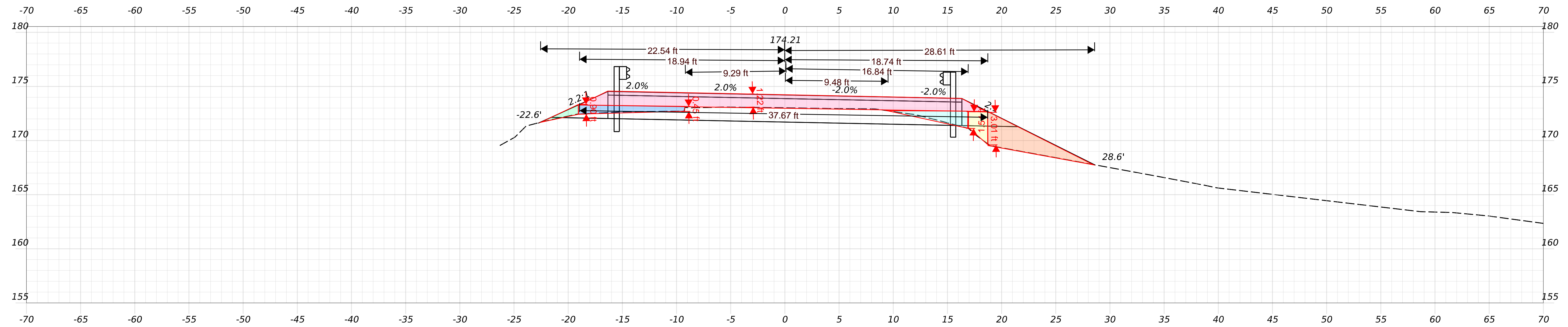
Max Settlement at 75 Years (in)				
Location	Left Side of Road	Center of Road	Right Side of Road	Max
Elastic	<0.1	<0.1	0.2	0.2
Consolidation	1.0	1.4	1.4	1.4
Total Settlement	1.1	1.5	1.6	1.6

Total Cumulative Settlement per Stage (in)			
Stage (Years)	Left Side of Road	Center of Road	Right Side of Road
0	<0.1	<0.1	0.2
1	0.2	0.2	0.3
2	0.2	0.3	0.4
5	0.3	0.4	0.5
15	0.6	0.7	0.8
30	0.8	1.1	1.1
45	0.9	1.2	1.3
60	1.0	1.4	1.5
75	1.1	1.5	1.6

See plots for graphed data.

Document Check:	Name	Signature	Date
Originator:	M.Rowicki		5/23/25
Checker:	J. Zwetchkenbaum		6/5/25
BackChecker:	M.Rowicki		6/5/25
Updater:	M.Rowicki		6/5/25
Verifier:	J. Zwetchkenbaum		6/5/25

Load Shapes and Dimensions for Settlement Model



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2623400
WIN
026234.00
BRIDGE NO. 0487
BRIDGE PLANS

SIGNATURE
P.E. NUMBER
DATE

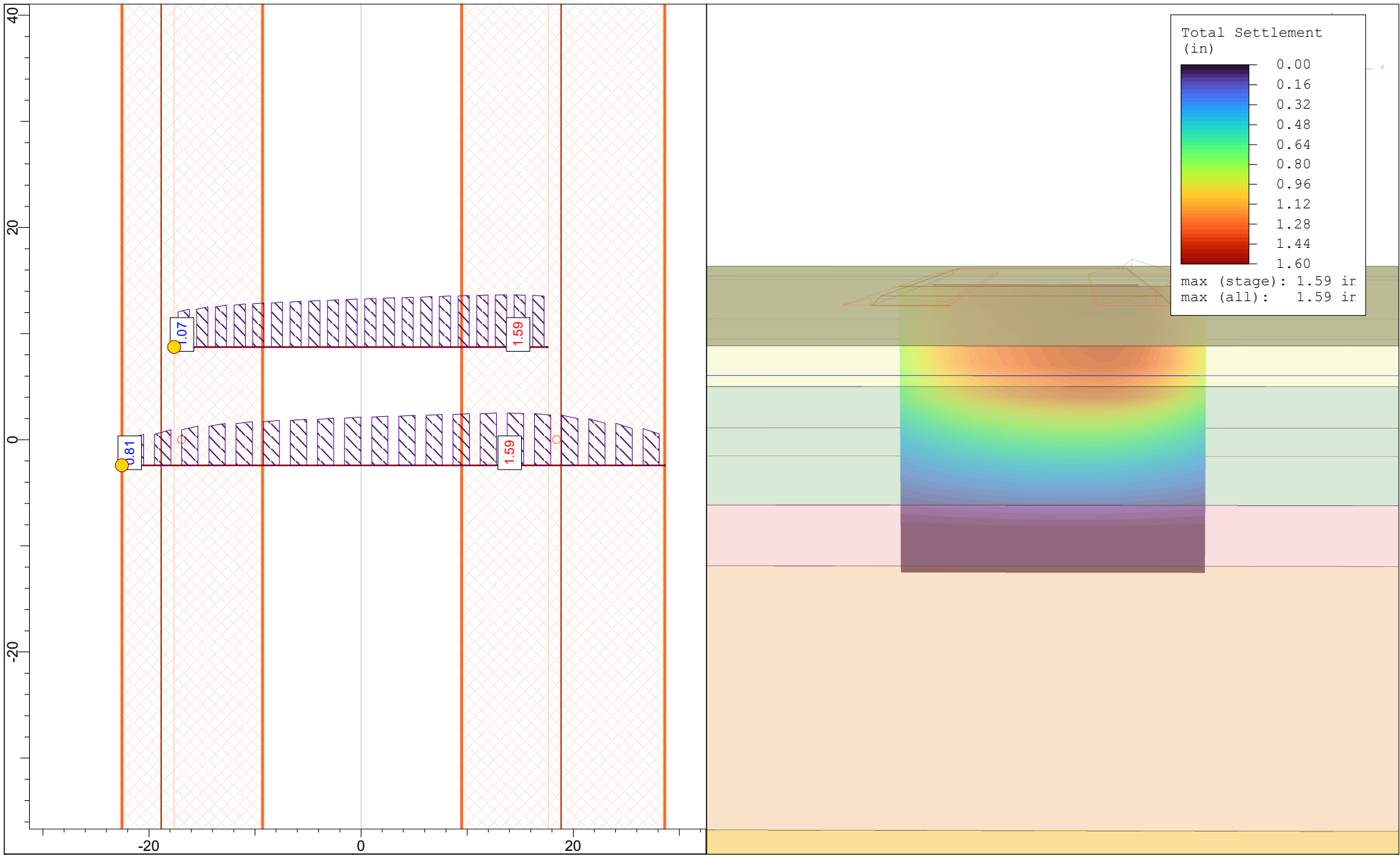
PROJ. MANAGER	BY	DATE
Trevor Gleason	C. Tobin	05/2025
CHECKED-REVIEWED	E. Davidson	05/2025
DESIGNED-DETAILED		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		


MONMOUTH
WAGGON BRIDGE
CROSS SECTION

SHEET NUMBER
1
OF 1

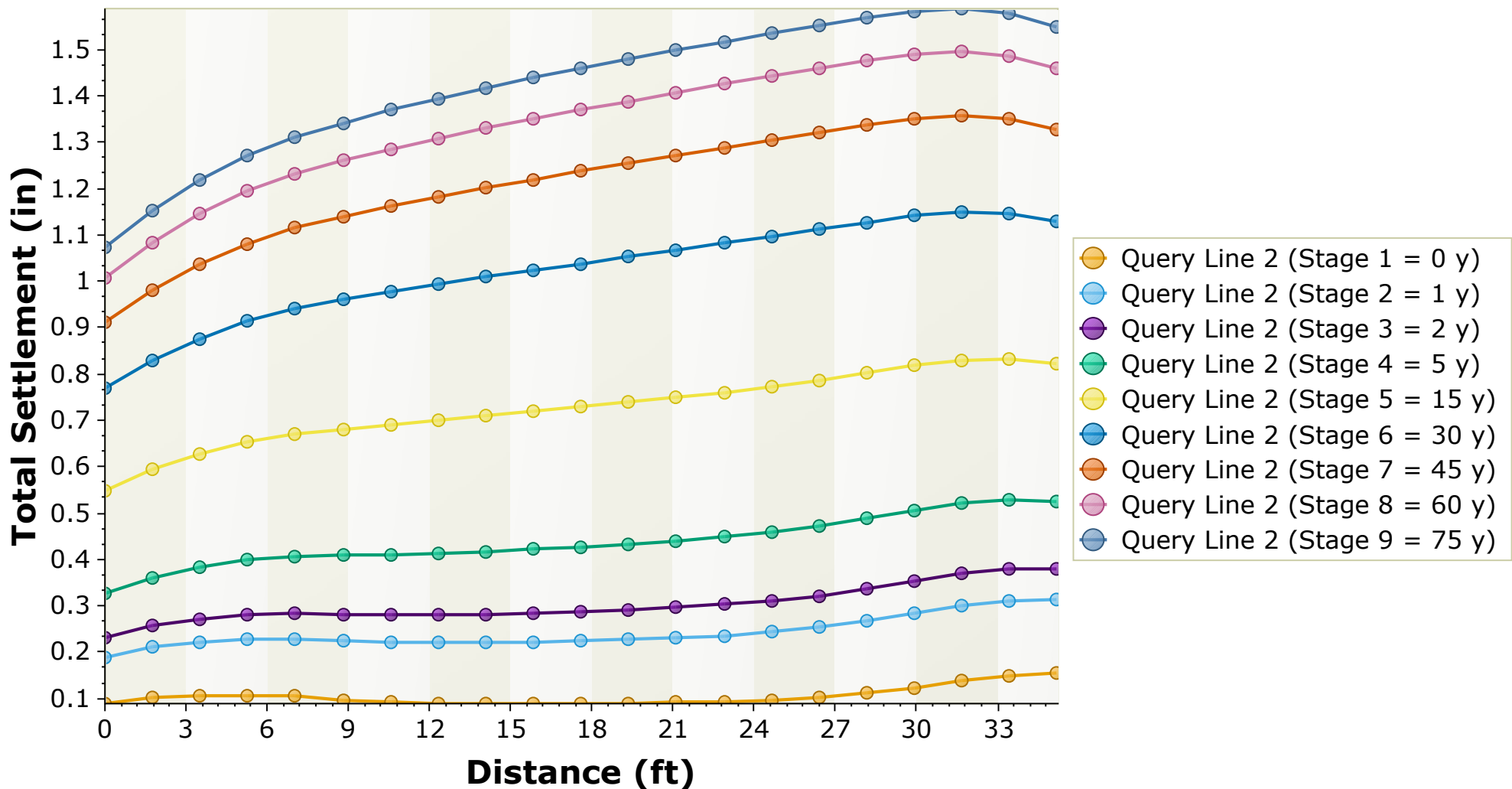


Username: ctoabin Date: 5/8/2025



	Project		Waggon Bridge Replacement	
	Analysis Description		Settlement Sta.26+70	
	Drawn By	M.Rowicki	Company	HNTB
	Date	8/25/2024, 10:51:54 AM	File Name	67328_Waugan Bridge Approaches_Sta.26+68.s3z

Distance vs. Total Settlement



Reference Stage: None
 Total Settlement at Elevation = 173 ft



Project	Waggon Bridge Replacement		
Analysis Description	Settlement Sta.26+70		
Drawn By	M.Rowicki	Company	HNTB
Date	8/25/2024, 10:51:54 AM	File Name	67328_Waugan Bridge Approaches_Sta.26+68.s3z

Waggon Bridge Replacement
Settlement Check @ 26+70
HNTB

Report Creation Date: 2025/05/22, 10:42:00

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Settle3 Analysis Information

Waggon Bridge Replacement

Project Settings

Document Name	67328_Waugan Bridge Approaches_Sta.26+68.s3z
Project Title	Waggon Bridge Replacement
Analysis	Settlement Sta.26+70
Author	M.Rowicki
Company	HNTB
Date Created	8/25/2024, 10:51:54 AM
Last saved with Settle3 version	5.021
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year

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)	173

Stage Settings

Stage #	Name	Time [years]
1	Stage 1	0
2	Stage 2	1
3	Stage 3	2
4	Stage 4	5
5	Stage 5	15
6	Stage 6	30
7	Stage 7	45
8	Stage 8	60
9	Stage 9	75

Results

Time taken to compute: 0.592369 seconds

Stage: Stage 1 = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.156154
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.67644
Effective Stress XX [ksf]	0.00208804	3.21184
Effective Stress YY [ksf]	-0.146547	3.2153
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0.00125752
Pore Water Pressure [ksf]	0	2.89173
Excess Pore Water Pressure [ksf]	0	0.350502
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.0145751	2.67164
Over-consolidation Ratio	1	1.72
Void Ratio	0	1.46
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-5.55112e-17	0.00958788

Stage: Stage 2 = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.314673
Total Consolidation Settlement [in]	-0.00145267	0.162755
Virgin Consolidation Settlement [in]	0	0.0421936
Recompression Consolidation Settlement [in]	-0.00145267	0.121367
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.67176
Effective Stress XX [ksf]	0.00208804	3.19275
Effective Stress YY [ksf]	-0.146547	3.19621
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.0023562	13.2041
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.00392425	26.1704
Total Strain	-2.0292e-05	0.00376781
Pore Water Pressure [ksf]	0	2.91306
Excess Pore Water Pressure [ksf]	0	0.16376
Degree of Consolidation [%]	0	10.4203
Pre-consolidation Stress [ksf]	0.0145751	2.67164
Over-consolidation Ratio	1	1.70005
Void Ratio	0	1.45954
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	53.9255
Undrained Shear Strength	-4.54769e-05	0.00976808

Stage: Stage 3 = 2 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.380984
Total Consolidation Settlement [in]	-0.00210654	0.230835
Virgin Consolidation Settlement [in]	0	0.100106
Recompression Consolidation Settlement [in]	-0.00210654	0.132188
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0.00129999
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.67131
Effective Stress XX [ksf]	0.00208804	3.18904
Effective Stress YY [ksf]	-0.146547	3.19251
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.00184925	10.9079
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.00271013	18.605
Total Strain	-3.02629e-05	0.00421194
Pore Water Pressure [ksf]	0	2.91675
Excess Pore Water Pressure [ksf]	0	0.154078
Degree of Consolidation [%]	0	14.6644
Pre-consolidation Stress [ksf]	0.0145751	2.67164
Over-consolidation Ratio	1	1.68822
Void Ratio	0	1.45926
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	61.3922
Undrained Shear Strength	-9.48985e-05	0.00976808

Stage: Stage 4 = 5 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.528618
Total Consolidation Settlement [in]	-0.00260429	0.379747
Virgin Consolidation Settlement [in]	0	0.236959
Recompression Consolidation Settlement [in]	-0.00260429	0.145159
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0.00406209
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.67157
Effective Stress XX [ksf]	0.00208804	3.18254
Effective Stress YY [ksf]	-0.146547	3.186
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.0012591	7.8962
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.0016151	11.383
Total Strain	-4.72173e-05	0.00481141
Pore Water Pressure [ksf]	0	2.92253
Excess Pore Water Pressure [ksf]	0	0.140856
Degree of Consolidation [%]	0	23.9673
Pre-consolidation Stress [ksf]	0.0145751	2.67164
Over-consolidation Ratio	1	1.67015
Void Ratio	0	1.45884
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	70.5249
Undrained Shear Strength	-0.000222814	0.00976808

Stage: Stage 5 = 15 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.831845
Total Consolidation Settlement [in]	-1.15624e-05	0.6864
Virgin Consolidation Settlement [in]	0	0.528176
Recompression Consolidation Settlement [in]	-1.15624e-05	0.163158
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0.00829893
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.68514
Effective Stress XX [ksf]	0.00208804	3.19909
Effective Stress YY [ksf]	-0.146547	3.20255
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.00075628	5.05549
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000877133	6.33726
Total Strain	-3.048e-06	0.00620881
Pore Water Pressure [ksf]	0	2.90362
Excess Pore Water Pressure [ksf]	0	0.121204
Degree of Consolidation [%]	0	42.9981
Pre-consolidation Stress [ksf]	0.0145751	2.68037
Over-consolidation Ratio	1	1.64527
Void Ratio	0	1.45824
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	81.2532
Undrained Shear Strength	-4.06287e-05	0.00976808

Stage: Stage 6 = 30 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.14927
Total Consolidation Settlement [in]	0	1.00752
Virgin Consolidation Settlement [in]	0	0.840832
Recompression Consolidation Settlement [in]	0	0.172611
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0.0109721
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.70686
Effective Stress XX [ksf]	0.00208804	3.23657
Effective Stress YY [ksf]	-0.146547	3.24004
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000535238	3.68381
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000594655	4.33275
Total Strain	3.77247e-06	0.00734231
Pore Water Pressure [ksf]	0	2.86372
Excess Pore Water Pressure [ksf]	0	0.0813058
Degree of Consolidation [%]	0	62.8726
Pre-consolidation Stress [ksf]	0.0145751	2.70211
Over-consolidation Ratio	1	1.62745
Void Ratio	0	1.45781
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	87.9473
Undrained Shear Strength	0	0.00976808

Stage: Stage 7 = 45 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.35649
Total Consolidation Settlement [in]	0	1.21442
Virgin Consolidation Settlement [in]	0	1.04242
Recompression Consolidation Settlement [in]	0	0.178353
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0.0138243
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.7332
Effective Stress XX [ksf]	0.00208804	3.26263
Effective Stress YY [ksf]	-0.146547	3.26609
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000450447	3.13331
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000492535	3.59926
Total Strain	3.77247e-06	0.00803036
Pore Water Pressure [ksf]	0	2.83606
Excess Pore Water Pressure [ksf]	0	0.0536422
Degree of Consolidation [%]	0	75.6773
Pre-consolidation Stress [ksf]	0.0145751	2.72852
Over-consolidation Ratio	1	1.61641
Void Ratio	0	1.45754
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	92.0627
Undrained Shear Strength	0	0.00976808

Stage: Stage 8 = 60 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.49428
Total Consolidation Settlement [in]	0	1.34975
Virgin Consolidation Settlement [in]	0	1.17385
Recompression Consolidation Settlement [in]	0	0.182103
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0.0182922
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.75097
Effective Stress XX [ksf]	0.00208804	3.27984
Effective Stress YY [ksf]	-0.146547	3.2833
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000407454	2.84959
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.00044294	3.24125
Total Strain	3.77247e-06	0.00847822
Pore Water Pressure [ksf]	0	2.81778
Excess Pore Water Pressure [ksf]	0	0.0353651
Degree of Consolidation [%]	0	84.0312
Pre-consolidation Stress [ksf]	0.0145751	2.74629
Over-consolidation Ratio	1	1.60922
Void Ratio	0	1.45737
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	94.7675
Undrained Shear Strength	0	0.00976808

Stage: Stage 9 = 75 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.58671
Total Consolidation Settlement [in]	0	1.43833
Virgin Consolidation Settlement [in]	0	1.25987
Recompression Consolidation Settlement [in]	0	0.184597
Immediate Settlement [in]	0	0.156154
Secondary Settlement [in]	0	0.0233547
Loading Stress ZZ [ksf]	5.0701e-11	0.403896
Loading Stress XX [ksf]	-0.0769235	0.712001
Loading Stress YY [ksf]	-0.146547	0.540021
Effective Stress ZZ [ksf]	5.0701e-11	2.76273
Effective Stress XX [ksf]	0.00208804	3.29119
Effective Stress YY [ksf]	-0.146547	3.29465
Total Stress ZZ [ksf]	5.0701e-11	5.56818
Total Stress XX [ksf]	0.00208804	6.09556
Total Stress YY [ksf]	-0.146547	6.09902
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000382958	2.68631
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00589668	28.7608
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000415548	3.04301
Total Strain	3.77247e-06	0.00877155
Pore Water Pressure [ksf]	0	2.80573
Excess Pore Water Pressure [ksf]	0	0.0233147
Degree of Consolidation [%]	0	89.5009
Pre-consolidation Stress [ksf]	0.0145751	2.75806
Over-consolidation Ratio	1	1.60452
Void Ratio	0	1.45725
Permeability [ft/y]	0	0.0346148
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	96.5505
Undrained Shear Strength	0	0.00976808

Loads

1. Fill Load: "Fill Load 1"

Label	Fill Load 1
Load Type	Flexible
Area of Load	3826 ft ²
Elevation	173 ft
Installation Stage	Stage 1 = 0 y

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
9.48	-100	0
16.84	-100	0.1925
18.74	-100	0.37625
28.61	-100	0
28.61	100	0
18.74	100	0.37625
16.84	100	0.1925
9.48	100	0

2. Fill Load: "Fill Load 2"

Label	Fill Load 2
Load Type	Flexible
Area of Load	2650 ft ²
Elevation	173 ft
Installation Stage	Stage 1 = 0 y

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-9.29	100	0.05625
-18.94	100	0.1125
-22.54	100	0
-22.54	-100	0
-18.94	-100	0.1125
-9.29	-100	0.05625

Embankments

1. Embankment: "Embankment Load 1"






Label	Embankment Load 1						
Center Line	(0, 100) to (0, -100.272)						
Near End Angle	90 degrees						
Far End Angle	90 degrees						
Number of Layers	1						
Base Width	37.7						
Layer	Stage	Left Bench Width (ft)	Left Angle (deg)	Height (ft)	Unit Weight (kips/ft3)	Right Angle (deg)	Right Bench Width (ft)
1	Stage 1 = 0 y	0	45	1.2	0.125	45	0

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Elevation [ft]	Drained at Bottom
1	Existing Fill	3.3	173	No
2	Alluvium	9.7	169.7	No
3	Marine Silt & Clay- Top	5	160	No
4	Marine Silt and Clay -Bot	22	155	No
5	Glacial Till	7	133	No

Soil Properties

Property	Existing Fill	Alluvium	Marine Silt and Clay-Bot	Glacial Till
Color				
Unit Weight [kips/ft3]	0.111	0.108	0.11	0.126
Saturated Unit Weight [kips/ft3]	0.115	0.11	0.115	0.131
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Disabled	Enabled
Es [ksf]	318	290	-	1289
Esur [ksf]	318	290	-	1289
Primary Consolidation	Disabled	Disabled	Enabled	Disabled
Material Type			Non-Linear	
Cce	-	-	0.1329	-
Cre	-	-	0.0128	-
e0	-	-	1.065	-
OCR	-	-	1	-
Cv [ft2/y]	-	-	10	-
Cvr [ft2/y]	-	-	10	-
B-bar	-	-	1	-
Secondary Consolidation	Disabled	Disabled	Mesri	Disabled
Cae/Cce	-	-	0.04	-
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1
Property	Marine Silt & Clay-Top			
Color				
Unit Weight [kips/ft3]	0.115			
Saturated Unit Weight [kips/ft3]	0.115			
K0	1			
Primary Consolidation	Enabled			
Material Type	Non-Linear			
Cce	0.197			
Cre	0.037			
e0	1.46			
OCR	1.72			
Cv [ft2/y]	10			
Cvr [ft2/y]	10			
B-bar	1			
Secondary Consolidation	Mesri			
Cae/Cce	0.04			
Undrained Su A [kips/ft2]	0			
Undrained Su S	0.2			
Undrained Su m	0.8			
Piezo Line ID	1			

Groundwater

Groundwater method
Water Unit Weight

Piezometric Lines
0.0624 kips/ft³

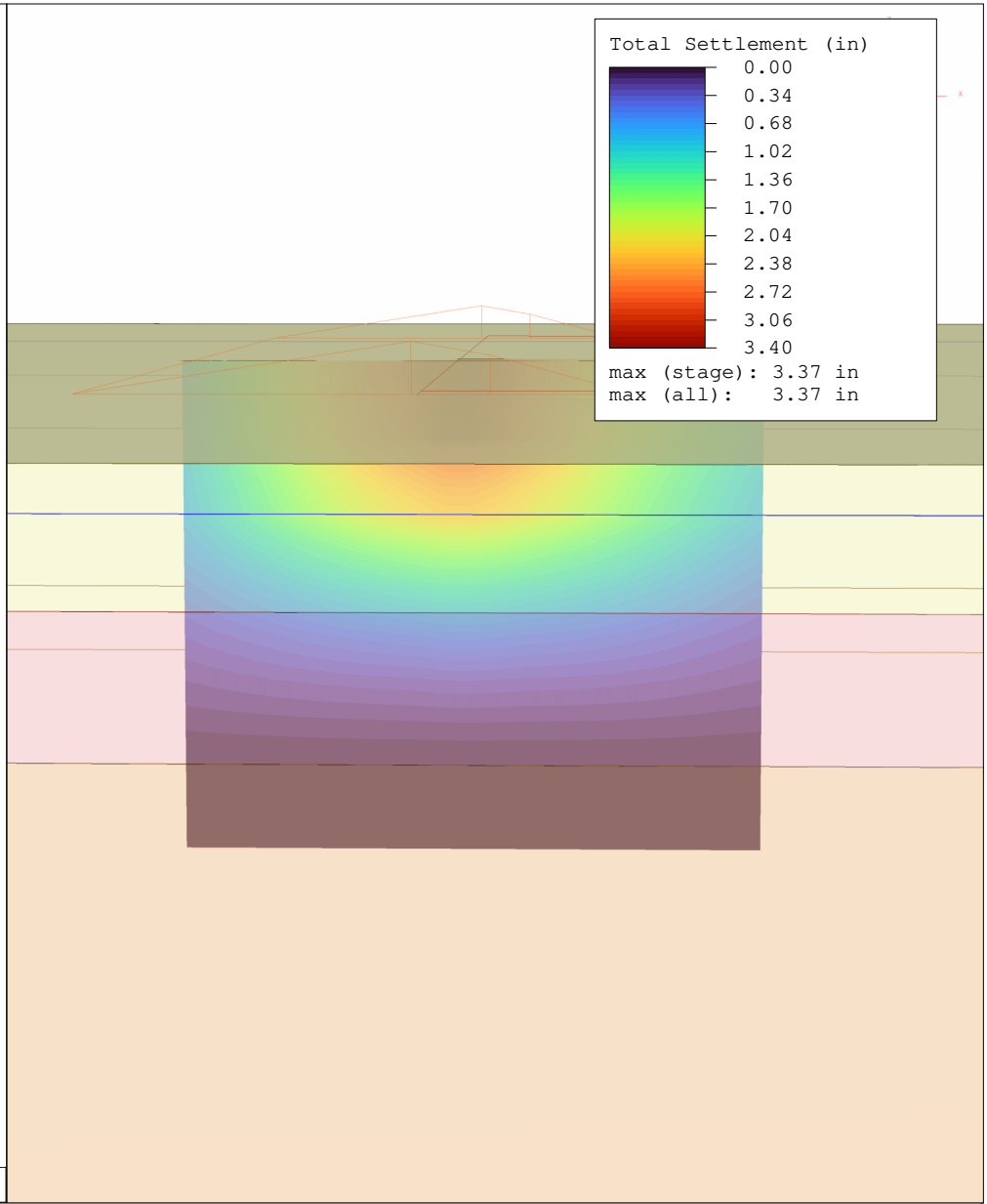
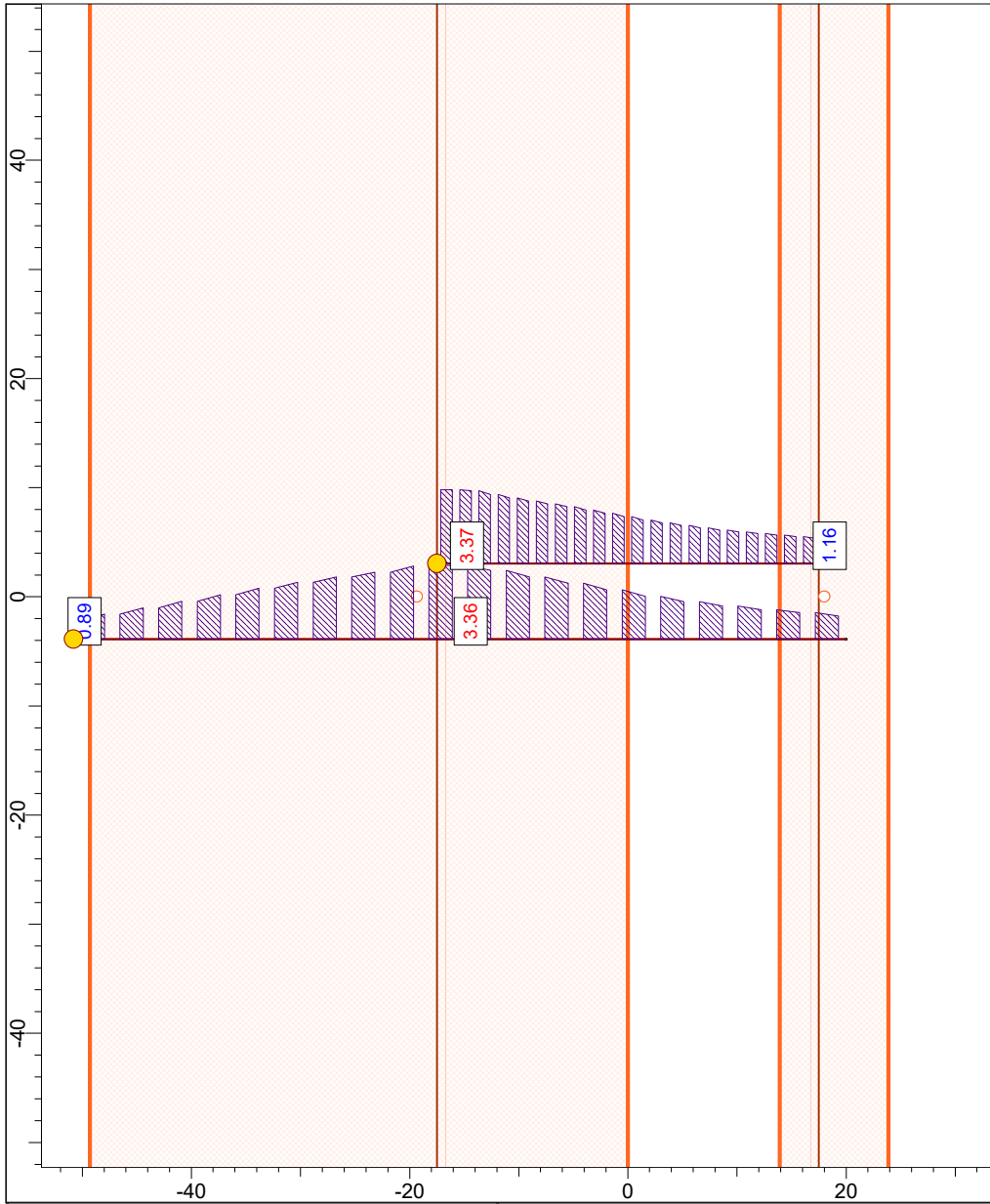
Piezometric Line Entities


ID	Elevation (ft)
1	170.59 ft

Query

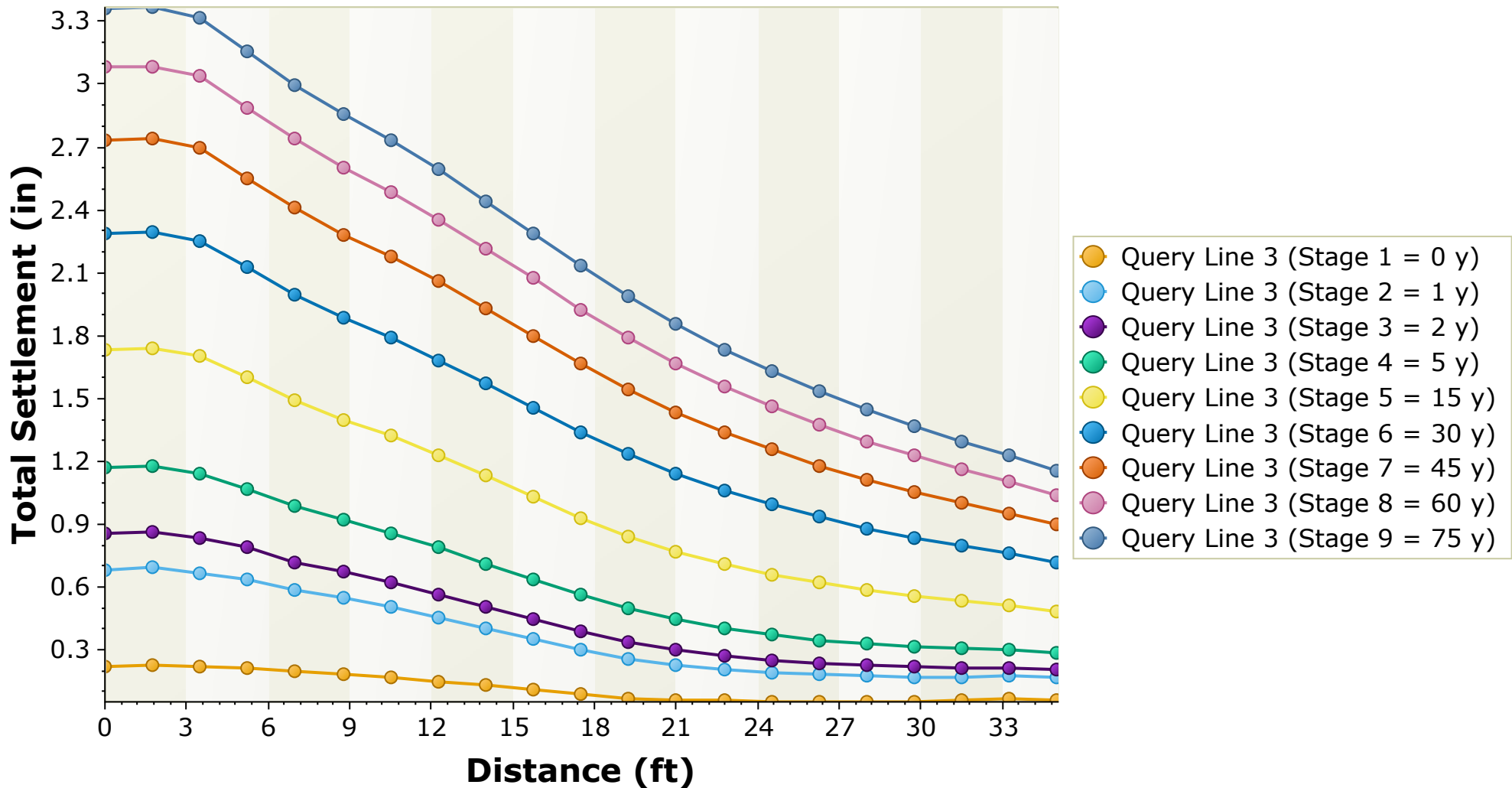
Query Lines

Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	Query Line 1	-22.54, -2.4324	28.61, -2.4324	20	Auto: 69
2	Query Line 2	-17.619, 8.714	17.579, 8.714	20	Auto: 69



	Project Waggon Bridge Replacement	
	Analysis Description Settlement - Sta.25+95	
	Drawn By M.Rowicki	Company HNTB
	Date 8/25/2024, 10:51:54 AM	File Name 67328_Waugan Bridge Approaches_Sta.25+95.s3z

Distance vs. Total Settlement



Reference Stage: None

Total Settlement at Elevation = 173.7 ft



<i>Project</i>	Waggon Bridge Replacement		
<i>Analysis Description</i>	Settlement - Sta.25+95		
<i>Drawn By</i>	M.Rowicki	<i>Company</i>	HNTB
<i>Date</i>	8/25/2024, 10:51:54 AM	<i>File Name</i>	67328_Waugan Bridge Approaches_Sta.25+95.s3z

Waggon Bridge Replacement
Settlement Check @ 25+95
HNTB

Report Creation Date: 2025/05/22, 11:02:50

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Settle3 Analysis Information

Waggon Bridge Replacement

Project Settings

Document Name	67328_Waugan Bridge Approaches_Sta.25+95.s3z
Project Title	Waggon Bridge Replacement
Analysis	Settlement - Sta.25+90
Author	M.Rowicki
Company	HNTB
Date Created	8/25/2024, 10:51:54 AM
Last saved with Settle3 version	5.021
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year

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)	173.7

Stage Settings

Stage #	Name	Time [years]
1	Stage 1	0
2	Stage 2	1
3	Stage 3	2
4	Stage 4	5
5	Stage 5	15
6	Stage 6	30
7	Stage 7	45
8	Stage 8	60
9	Stage 9	75

Results

Time taken to compute: 0 seconds

Stage: Stage 1 = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.228514
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	-1.0821e-05	3.4601
Effective Stress XX [ksf]	-0.031562	4.15633
Effective Stress YY [ksf]	0.029132	4.05791
Total Stress ZZ [ksf]	-1.0821e-05	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0.00202882
Pore Water Pressure [ksf]	0	3.66976
Excess Pore Water Pressure [ksf]	0	0.619755
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.0172631	3.45215
Over-consolidation Ratio	1	1.72
Void Ratio	0	1.46
Permeability [ft/y]	0	0.0308539
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-5.55112e-17	0.0153305

Stage: Stage 2 = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.69233
Total Consolidation Settlement [in]	-0.00353942	0.463816
Virgin Consolidation Settlement [in]	0	0.0370589
Recompression Consolidation Settlement [in]	-0.00353942	0.426757
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.45719
Effective Stress XX [ksf]	-0.031562	4.10083
Effective Stress YY [ksf]	0.029132	4.00241
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.00076016	12.1126
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.00111011	18.9115
Total Strain	-3.2165e-05	0.0112574
Pore Water Pressure [ksf]	0	3.7254
Excess Pore Water Pressure [ksf]	0	0.391427
Degree of Consolidation [%]	0	11.6143
Pre-consolidation Stress [ksf]	0.0172631	3.45215
Over-consolidation Ratio	1	1.72339
Void Ratio	0	1.46008
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-0.000122897	0.0253327

Stage: Stage 3 = 2 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.86636
Total Consolidation Settlement [in]	-0.0056369	0.611692
Virgin Consolidation Settlement [in]	0	0.0923946
Recompression Consolidation Settlement [in]	-0.0056369	0.519298
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0.0261537
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.45777
Effective Stress XX [ksf]	-0.031562	4.09328
Effective Stress YY [ksf]	0.029132	3.99485
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000602647	9.84251
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000808353	14.1533
Total Strain	-4.81837e-05	0.0136462
Pore Water Pressure [ksf]	0	3.73301
Excess Pore Water Pressure [ksf]	0	0.368883
Degree of Consolidation [%]	0	15.3172
Pre-consolidation Stress [ksf]	0.0172631	3.45215
Over-consolidation Ratio	1	1.71977
Void Ratio	0	1.45999
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-0.000206668	0.0253801

Stage: Stage 4 = 5 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.17606
Total Consolidation Settlement [in]	-0.00997266	0.885641
Virgin Consolidation Settlement [in]	0	0.295888
Recompression Consolidation Settlement [in]	-0.00997266	0.589753
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0.0619014
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.46084
Effective Stress XX [ksf]	-0.031562	4.07224
Effective Stress YY [ksf]	0.029132	3.97381
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000417514	7.03147
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000513296	9.19439
Total Strain	-9.33682e-05	0.0168288
Pore Water Pressure [ksf]	0	3.75437
Excess Pore Water Pressure [ksf]	0	0.328754
Degree of Consolidation [%]	0	22.177
Pre-consolidation Stress [ksf]	0.0172631	3.45294
Over-consolidation Ratio	1	1.71078
Void Ratio	0	1.45979
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-0.000379939	0.0254028

Stage: Stage 5 = 15 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.73627
Total Consolidation Settlement [in]	-0.00961272	1.40299
Virgin Consolidation Settlement [in]	0	0.756134
Recompression Consolidation Settlement [in]	-0.00961272	0.646943
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0.104762
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.47349
Effective Stress XX [ksf]	-0.031562	4.05377
Effective Stress YY [ksf]	0.029132	3.95534
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000237406	4.19546
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000267791	5.11474
Total Strain	-0.000133114	0.0206136
Pore Water Pressure [ksf]	0	3.77302
Excess Pore Water Pressure [ksf]	0	0.297959
Degree of Consolidation [%]	0	35.1318
Pre-consolidation Stress [ksf]	0.0172631	3.46559
Over-consolidation Ratio	1	1.69231
Void Ratio	0	1.45936
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-0.000608972	0.025415

Stage: Stage 6 = 30 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.29324
Total Consolidation Settlement [in]	-0.000599426	1.9341
Virgin Consolidation Settlement [in]	0	1.24613
Recompression Consolidation Settlement [in]	-0.000599426	0.687968
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0.131804
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.49307
Effective Stress XX [ksf]	-0.031562	4.1029
Effective Stress YY [ksf]	0.029132	4.00447
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000158515	3.13411
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000172342	3.71018
Total Strain	-2.9235e-05	0.0229993
Pore Water Pressure [ksf]	0	3.72366
Excess Pore Water Pressure [ksf]	0	0.248603
Degree of Consolidation [%]	0	48.4015
Pre-consolidation Stress [ksf]	0.0172631	3.48518
Over-consolidation Ratio	1	1.67395
Void Ratio	0	1.45893
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-0.000120761	0.0254215

Stage: Stage 7 = 45 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.73676
Total Consolidation Settlement [in]	0	2.36244
Virgin Consolidation Settlement [in]	0	1.6556
Recompression Consolidation Settlement [in]	0	0.706843
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0.147623
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.5098
Effective Stress XX [ksf]	-0.031562	4.14897
Effective Stress YY [ksf]	0.029132	4.05055
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.00012576	2.6249
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000134724	3.03748
Total Strain	8.14254e-09	0.0243947
Pore Water Pressure [ksf]	0	3.67754
Excess Pore Water Pressure [ksf]	0	0.20248
Degree of Consolidation [%]	0	59.1114
Pre-consolidation Stress [ksf]	0.0172631	3.5019
Over-consolidation Ratio	1	1.66025
Void Ratio	0	1.4586
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-0.000172964	0.0254253

Stage: Stage 8 = 60 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.0866
Total Consolidation Settlement [in]	0	2.70036
Virgin Consolidation Settlement [in]	0	1.98027
Recompression Consolidation Settlement [in]	0	0.720089
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0.160204
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.52344
Effective Stress XX [ksf]	-0.031562	4.18891
Effective Stress YY [ksf]	0.029132	4.09048
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000107979	2.32678
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000114734	2.65737
Total Strain	8.14254e-09	0.0253848
Pore Water Pressure [ksf]	0	3.63753
Excess Pore Water Pressure [ksf]	0	0.162477
Degree of Consolidation [%]	0	67.5566
Pre-consolidation Stress [ksf]	0.0172631	3.51555
Over-consolidation Ratio	1	1.64946
Void Ratio	0	1.45834
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.025428

Stage: Stage 9 = 75 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.365
Total Consolidation Settlement [in]	0	2.96649
Virgin Consolidation Settlement [in]	0	2.23607
Recompression Consolidation Settlement [in]	0	0.73042
Immediate Settlement [in]	0	0.228514
Secondary Settlement [in]	0	0.172997
Loading Stress ZZ [ksf]	-1.0821e-05	0.656589
Loading Stress XX [ksf]	-0.172468	0.84903
Loading Stress YY [ksf]	-0.152558	0.629642
Effective Stress ZZ [ksf]	0	3.53797
Effective Stress XX [ksf]	-0.031562	4.22126
Effective Stress YY [ksf]	0.029132	4.12284
Total Stress ZZ [ksf]	0	7.12986
Total Stress XX [ksf]	-0.031562	7.82609
Total Stress YY [ksf]	0.029132	7.72767
Modulus of Subgrade Reaction (Total) [ksf/ft]	-9.69874e-05	2.1338
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.0024114	34.1027
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000102535	2.41898
Total Strain	8.14254e-09	0.0261529
Pore Water Pressure [ksf]	0	3.6061
Excess Pore Water Pressure [ksf]	0	0.131041
Degree of Consolidation [%]	0	74.2077
Pre-consolidation Stress [ksf]	0.0172631	3.53019
Over-consolidation Ratio	1	1.64086
Void Ratio	0	1.45814
Permeability [ft/y]	0	0.0789783
Coefficient of Consolidation [ft ² /y]	0	10
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.0254302

Loads

1. Fill Load: "Fill Load 1"

Label	Fill Load 1
Load Type	Flexible
Area of Load	1992 ft ²
Elevation	173.7 ft
Installation Stage	Stage 1 = 0 y

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
13.92	-100	0
16.05	-100	0.1
18.19	-100	0.15
23.88	-100	0.15
23.88	100	0.15
18.19	100	0.15
16.05	100	0.1
13.92	100	0

2. Fill Load: "Fill Load 2"

Label	Fill Load 2
Load Type	Flexible
Area of Load	9860 ft ²
Elevation	173.7 ft
Installation Stage	Stage 1 = 0 y

Coordinates and Load

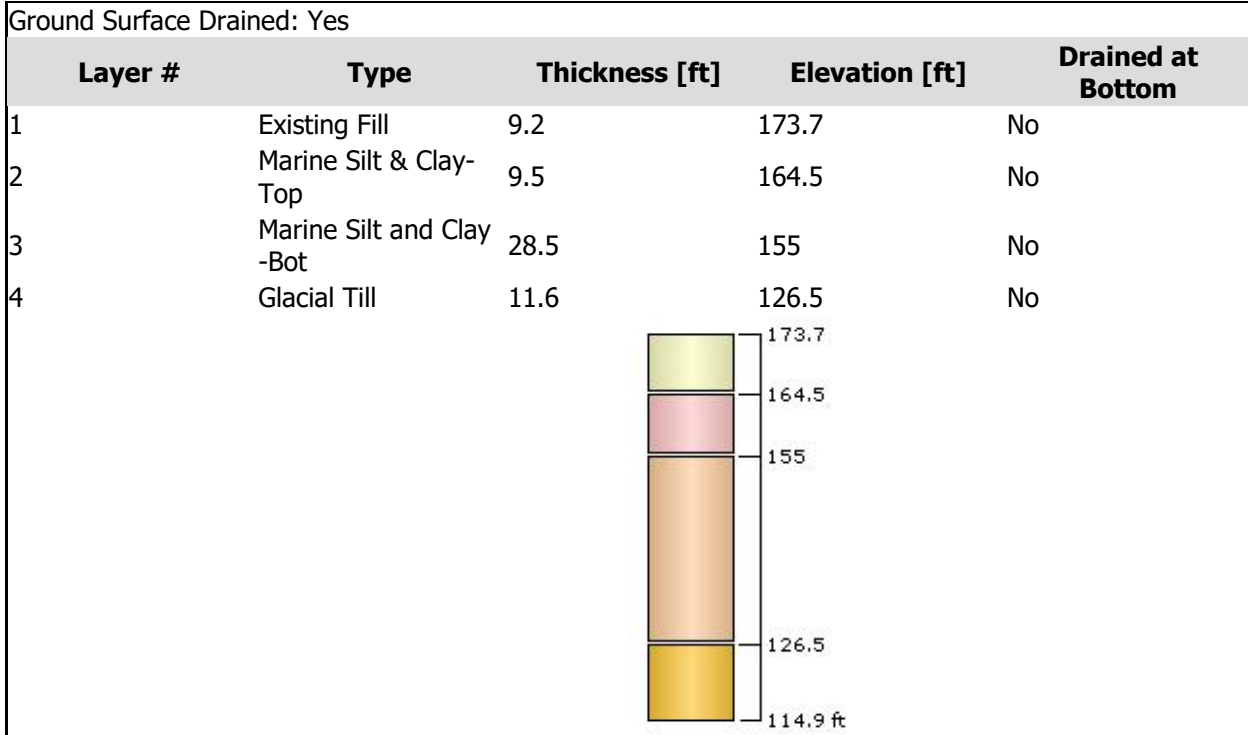
X [ft]	Y [ft]	Load Magnitude [ksf]
0	100	0.10875
-10.31	100	0.44625
-17.68	100	0.60125
-49.3	100	0
-49.3	-100	0
-17.68	-100	0.60125
-10.31	-100	0.44625
0	-100	0.10875

Embankments





1. Embankment: "Embankment Load 1"

Label		Embankment Load 1					
Center Line		(0, 100) to (0, -100.272)					
Near End Angle		90 degrees					
Far End Angle		90 degrees					
Number of Layers		1					
Base Width		35					
Layer	Stage	Left Bench Width (ft)	Left Angle (deg)	Height (ft)	Unit Weight (kips/ft3)	Right Angle (deg)	Right Bench Width (ft)
1	Stage 1 = 0 y	0	45	0.77	0.125	45	0

Soil Layers



Soil Properties

Property	Existing Fill	Marine Silt and Clay-Bot	Glacial Till	Marine Silt & Clay-Top
Color				
Unit Weight [kips/ft3]	0.111	0.11	0.126	0.115
Saturated Unit Weight [kips/ft3]	0.115	0.115	0.131	0.115
K0	1	1	1	1
Immediate Settlement	Enabled	Disabled	Enabled	Disabled
Es [ksf]	318	-	1289	-
Esur [ksf]	318	-	1289	-
Primary Consolidation	Disabled	Enabled	Disabled	Enabled
Material Type		Non-Linear		Non-Linear
Cce	-	0.1329	-	0.197
Cre	-	0.0128	-	0.037
e0	-	1.065	-	1.46
OCR	-	1	-	1.72
Cv [ft2/y]	-	10	-	10
Cvr [ft2/y]	-	10	-	10
B-bar	-	1	-	1
Secondary Consolidation	Disabled	Mesri	Disabled	Mesri
Cae/Cce	-	0.04	-	0.04
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method
Water Unit Weight

Piezometric Lines
0.0624 kips/ft³

Piezometric Line Entities

ID	Elevation (ft)
1	170.59 ft

Query

Query Lines

Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
2	Query Line 2	-50.8, -3.8778	20, -3.8778	20	Auto: 55
3	Query Line 3	-17.5, 3.03902	17.5, 3.03902	20	Auto: 55

DOWNDRAG

Calculations for:	MaineDOT Waggon	Job No.:	67328
Made by:	M.Rowicki	Date:	5/15/2025
Checked by:	JDZ	Date:	5/20/2025
Backchecked by:	M.Rowicki	Date:	5/21/2025



Pile Downdrag Loads

- Method Calculated downdrag per Article 3.11.8 of AASHTO LRFD 2020. Pre-2024 revised guidelines.
- Steps
1. Determined soil profile on each side, North & South, of the proposed bridge.
 2. Calculated settlement using Settle3 from proposed fill/embankment. Determine at what elevation settlement <0.4 inches.
 3. Referenced APILE axial calculations to obtain Unit Skin Friction in layers exceeding settlement criteria.
 4. Calculated downdrag (DD) by mutliplying length of pile by unit skin friction of layers.
 5. Calculate the downdrag driving load needed to be overcome using a resistance factor of 1.4

Pile Size	Width (in)	Depth (in)	Perimeter (ft)
HP14x74	14.6	13.6	4.70
HP14x89	14.7	13.8	4.75
HP14x102	14.8	14.0	4.80
HP14x117	14.9	14.2	4.85

Include Bitumen Slick Coat (Y/N)	N
Slick Coat Unit Skin (psf)	N/A

South Approach Strata (Sta. 25+95.00 - BB-MWS-102)

Elevation of <0.4" expected settlement: 135.5 (ft)

Soil Layer Subject to DD	Top Elev. (ft)	Bot. Elev. (ft.)	Thickness for DD (ft)	Unit Skin (psf)	HP14x74	HP14x89	HP14x102	HP14x117
					Est. Downdrag Load (kips)	Est. Downdrag Load (kips)	Est. Downdrag Load (kips)	Est. Downdrag Load (kips)
Marine Silt Clay Crust	165.5	159	6.5	450	13.7	13.9	14.0	14.2
Marine Silt Clay	159	129	23.5	340	38.0	38.0	38.4	38.8
Total DD per Pile - Unfactored (kips)					51.7	51.8	52.4	52.9
Max Factored DD per pile (kips)					72.4	72.6	73.3	74.1

North Approach Strata (Sta. 26+70.00 - BB-MWS-101)

Elevation of <0.4" expected settlement: 145 (ft)

Soil Layer Subject to DD	Top Elev. (ft)	Bot. Elev. (ft.)	Thickness for DD (ft)	Unit Skin (psf)	HP14x74	HP14x89	HP14x102	HP14x117
					Est. Downdrag Load (kips)	Est. Downdrag Load (kips)	Est. Downdrag Load (kips)	Est. Downdrag Load (kips)
Stream Alluvium	165.5	160	5.5	200	5.2	5.2	5.3	5.3
Marine Silt Clay	159.5	135	14.5	340	23.2	23.4	23.7	23.9
Total DD per Pile - Unfactored (kips)					28.3	28.6	28.9	29.2
Max Factored DD per pile (kips)					39.7	40.1	40.5	40.9

Notes:

Refer to Axial Calculations for Unit Skin Friction results

Elevation of <0.4" obtained from Settle3D Calculations, refer to caluclations for modeling information.

Resistance Factor for Downdrag is 1.4 for Strength, consistent with alpha Tomlinson Method. For minimum factor uses 0.25.

Resistance Factor for dynamic driving of piles is 0.65 per AASHTO Table 10.5.5.2.3-1 assuming PDA will be performed on piles.

Pile Driving Resistance per AASHTO LRFD C10.7.3.7

$$R_{ndr} = \frac{\sum \gamma_i Q_i}{\phi_{dyn}} + \frac{\gamma_p DD}{\phi_{dyn}} + R_{Sdd}$$

R_{ndr} = nominal pile driving resistance required (kips)
 $\sum \gamma_i Q_i$ = factored load per pile (kips)
 ϕ_{dyn} = resistance factor
 γ_p = load factor for downdrag
 DD = downdrag load per pile (kips)
 R_{Sdd} = skin friction which must be overcome during driving through downdrag zone (kips)

Nominal Driving Resistance of piles to be (Demand from FBMP plus Factored DD above, all divided by 0.65, plus Unfactored DD above).

Nominal Driving Resistance to not exceed Factored Structural Resistance of Pile = 626 kips (for HP14x74)

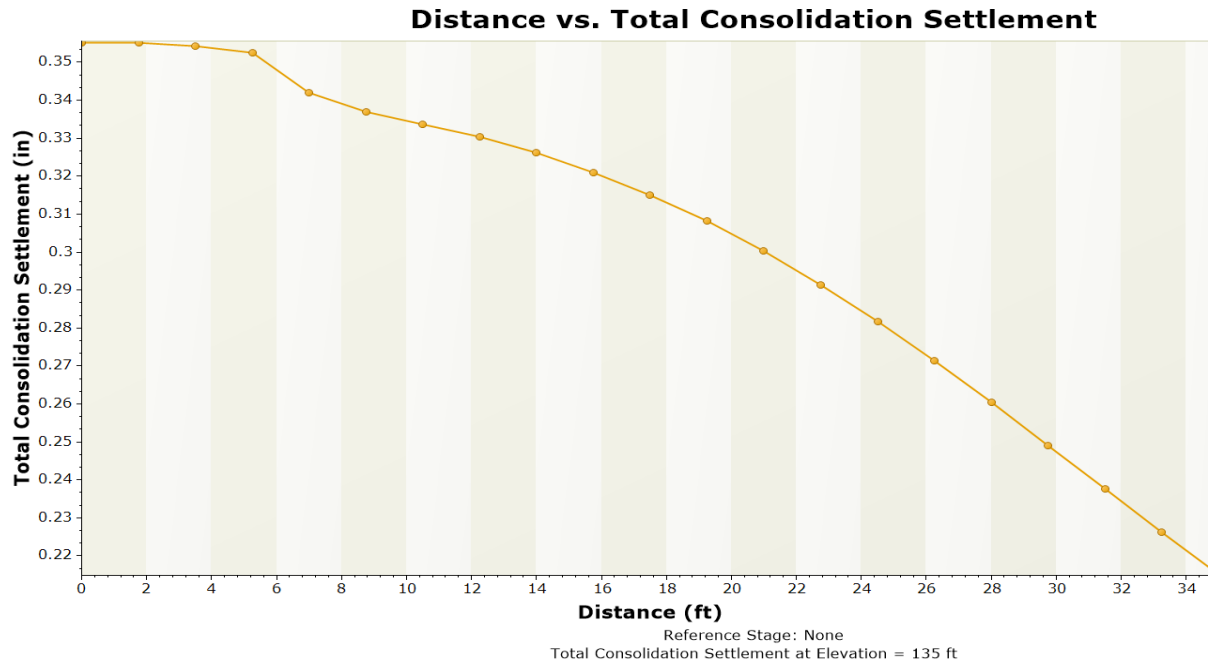
Calculations for:	MaineDOT Waggon	Job No.:	67328
Made by:	M.Rowicki	Date:	5/15/2025
Checked by:	JDZ	Date:	5/20/2025
Backchecked by:	M.Rowicki	Date:	5/21/2025



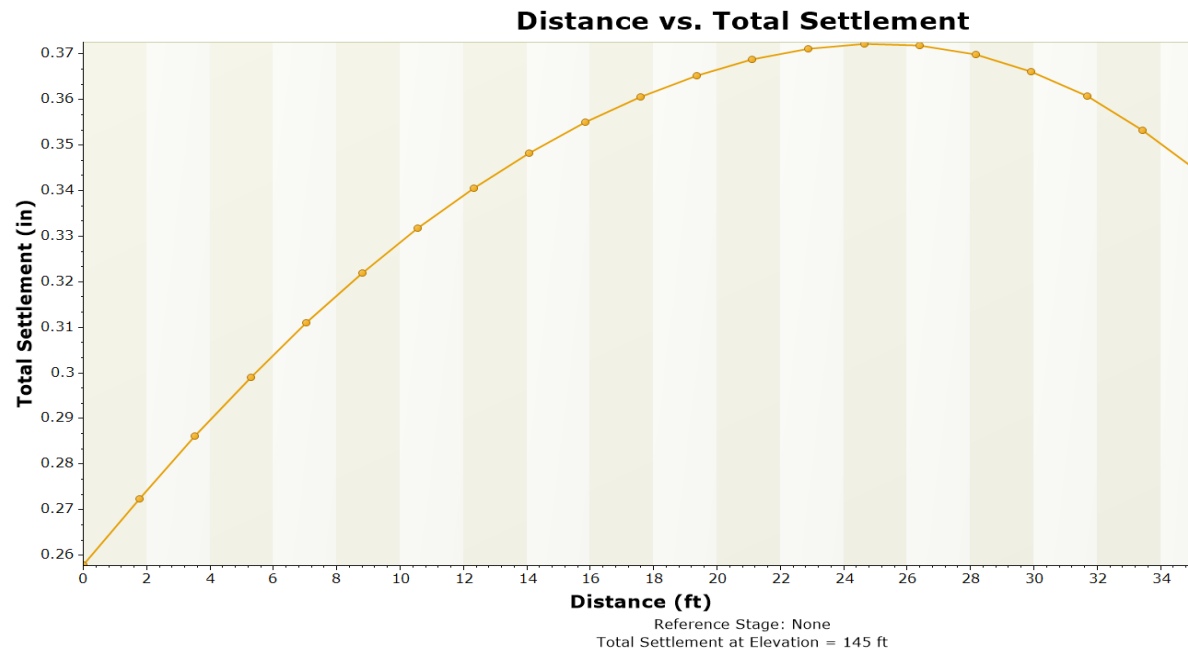
Settlement Curves where settlement <0.4"

Refer to settlement calculations for modeling and cross section details.

Sta. 25+95 - Settlement @ Elev. 135.5 ft.

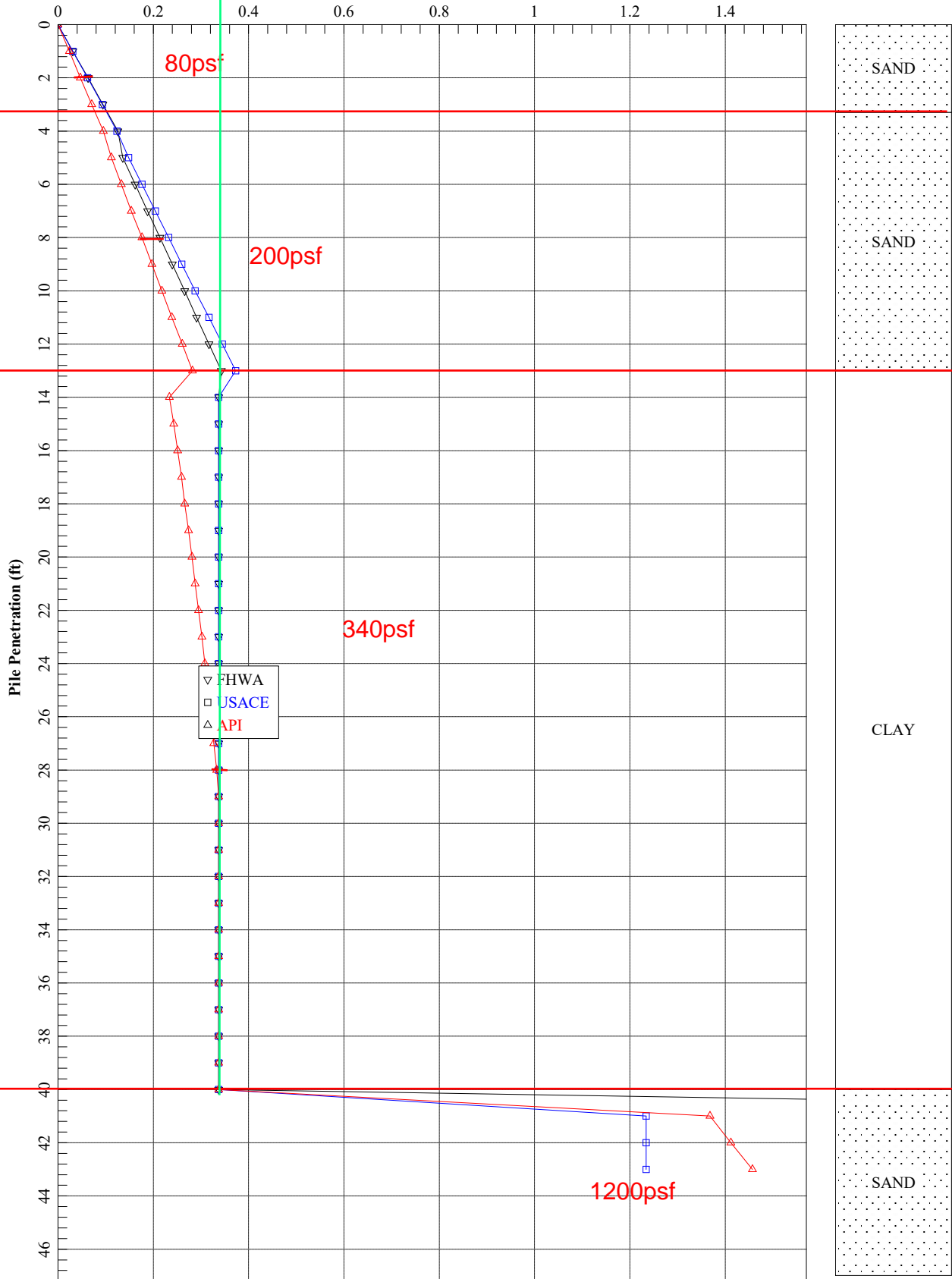


Sta. 26+70 - Settlement @ Elev. 145 ft.

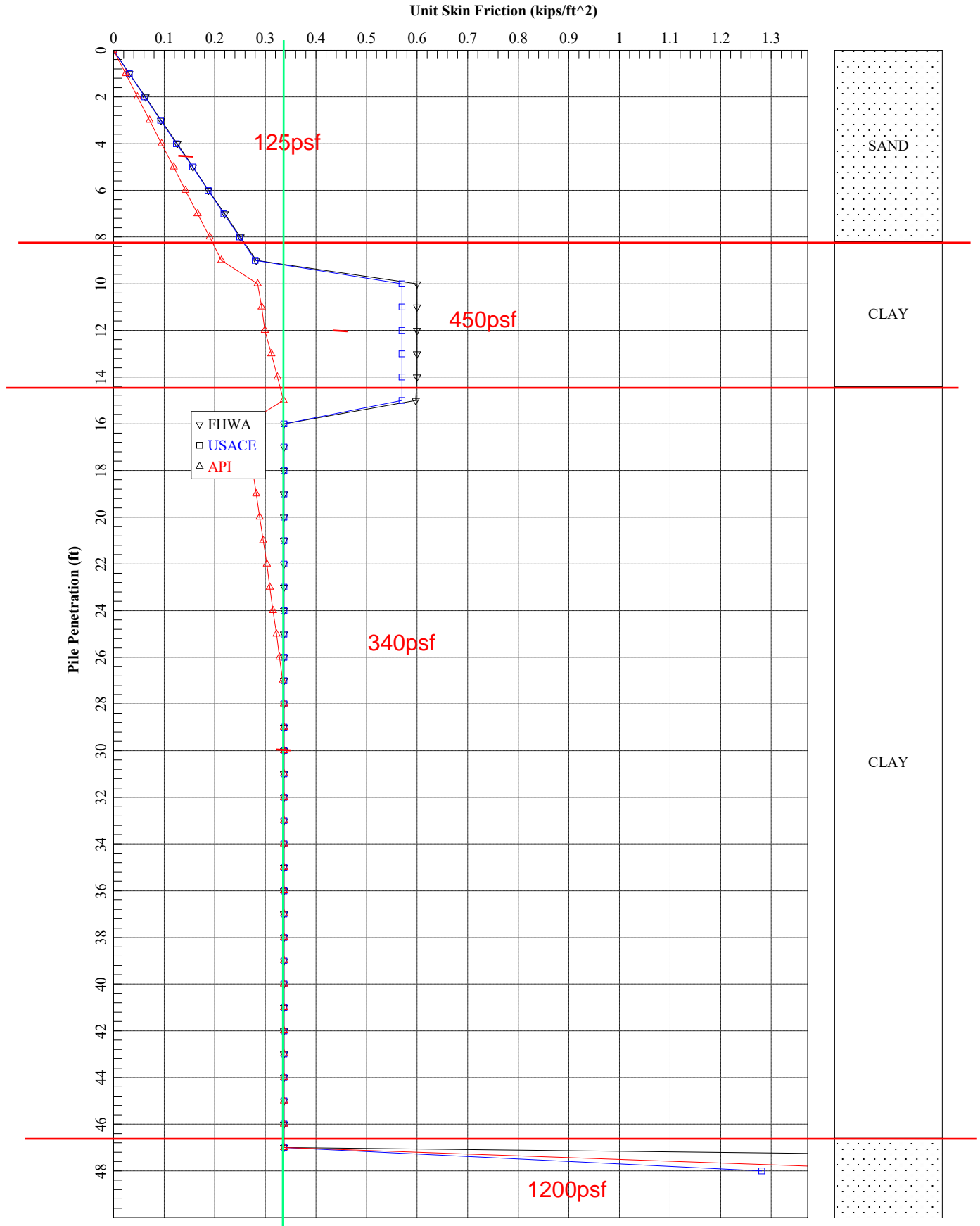


North - 101

Unit Skin Friction (kips/ft²)



South - 102



DRIVEABILITY

Project: Waggon Bridge	Job No: 67328	Design Criteria Document:
Client: MaineDOT	Discipline: Geotech	Calculation No: 0

Name or Description of Calculation: Waggon Driveability Calcs

Calc. Rev. No.	Originator	Checker	Senior Technical Reviewer (if required)	Confirmation Required (Y/N)
1	M. Rowicki	B. Materek		

Calculation Objective:

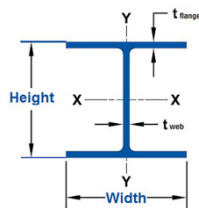
Check the driveability of a common Maine DOT pile size for the proposed integral abutment.

Calculation Methodology/List of Assumptions:

General:

- Utilized soil parameters put together from two borings.
- An APILE model was developed to estimate the unit skin and unit tip resistances
- An HP14x73 pile has been assumed. This is one of four typical piles the Maine Bridge manual specifies for integral abutments. Tip area input was only the area of the web and flanges and not the large block area of the HP section given that soft cohesive soils are present at the anticipated tip elevations.
- GRL Weap 14 was used to assess drivability.
- No design has been advanced as of this moment so the main purpose of the assessment is to determine if a common size is drivable and not to check a specific capacity.
- Shaft dampening: Clay=0.2 s/ft ; Sand=0.04 s/ft ; Toe=0.15 s/ft

HP14 x73 (Structural steel and H-piles)



Section description	Product group	Shape	Section Modulus	Moment of Inertia	Width	Height	Thickness flange	Thickness web	Weight single	Weight	Coating 2 sides	Coating area
			in ³ /ft cm ³ /m	in ⁴ /ft cm ⁴ /m	inch mm	inch mm	inch mm	inch mm	lbs/ft kg/m	lbs/ft ² kg/m ²	ft ² /ft m ² /m	ft ² /ft m ² /m
HP14 x73	Structural steel and H-piles	H-pile	107.0	729.0	14.60	13.60	0.505	0.505	73.0		6.96	
			5,753	99,551	371	345	12.8	12.8	108.62		2.13	

References/Inputs:





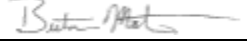
- 1) AASHTO LRFD Bridge Design Specifications, 9th Edition.

Attachments: (List each attachment following the subject calculation)

- 1) Summary of Drivability Assessment
- 2) Soil Parameters
- 3) Apile Output

Conclusions:

Results of assessment indicate that piles can be driven to required capacities without overstressing the piles during driving. The assessment indicates approximately 4 blows per inch (BPI) at a depth of 31' to achieve a 257 kip ultimate resistance assuming no losses. Driving stresses are shown as not exceeding 24 ksi which does not exceed $0.9 * 45$ ksi yield stress of a pile.

Document Check:	Name	Signature	Date
Originator:	M.Rowicki		11/9/22
Checker:	B. Materek		7/20/23
BackChecker:	M.Rowicki		7/20/23
Updater:	M.Rowicki		7/20/23
Verifier:	B. Materek		7/20/23

Project: MaineDOT Monmouth Bridges
Structure: Waugan Bridge
Location: Integral Abut Piles
Boring: BB-MWS-101 & 102

Pile Type: HP
Diameter/Size: 14x73
Wall Thickness: n/a
Steel Area: 21.40 in²

Estimated Pile Length: 31 feet
Estimated Pile Embedment ¹: 30 feet
Yield Stress: 50 ksi
Maximum Allowable Stress: 40 ksi

Soil Strength Loss Assumed: 70 %

Required Driving Resistance: N/A kips

Depth of Blow Count Limitation: None
Depth of Comp. Stress Limitation: None

Driving Limitation(s): None

- Notes:**
1. Top of piles assumed at top of ground surface. Embedment estimated using APILE.
 2. Wave equation analyses in the computer program GRLWEAPTM Version 14 by Pile Dynamics, Inc. was used to evaluate driveability. Default hammer parameters, including energy and efficiency provided in the GRLWEAP software, and manufacturer recommended cushion information was modeled in the analysis.
 3. The range of blows per foot considered acceptable was 25 bpf to 120 bpf (practical refusal).

GRL WEAP Output

Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 0.700/0.700

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	4.0	0.8	3.2	0.0	0.000	0.000	3.94	0.0	IP-2
10.0	15.1	9.9	5.2	2.2	23.727	10.371	3.94	15.2	IP-2
15.0	25.0	21.1	3.9	4.6	23.859	7.863	3.94	14.6	IP-2
20.0	30.8	26.9	3.9	5.8	23.877	6.979	3.94	14.4	IP-2
25.0	39.3	35.4	3.9	7.9	23.502	5.297	3.94	14.2	IP-2
30.0	50.4	46.5	3.9	10.2	22.986	4.171	3.94	14.1	IP-2
31.0	195.0	50.2	144.7	38.9	22.990	0.736	3.94	13.7	IP-2

Total Number of Blows: 152 (starting at penetration 5.0 ft)
Driving Time (min): 5 3 3 2 2 1 1 1 1 1
@Blow Rate (b/min): 30 40 50 60 70 80 90 100 110 120
Driving Time for continuously running hammer; any wait time not included.

Waugan Bridge
MaineDOT

Project: MainDOT Monmouth Bridges
Project No.: 67328
Structure: Waggon Bridge

Originator: MLR
Checker: BAM
Backchecker: MLR
Updater: MLR
Verifier: BAM

Date: 11/9/2022
Date: 7/20/2023
Date: 7/20/2023
Date: 7/20/2023
Date: 7/21/2023

Sheet No.: 1

Driveability Results



Project: MaineDOT Monmouth Bridges
Structure: Waugan Bridge
Location: Integral Abut Piles
Boring: BB-MWS-101 & 102

GRL WEAP Output

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	Ct ksi	Mx ksi	C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	5.4	0.8	4.6	0.0	0.000	0.000	3.94	0.0	0.0	IP-2	
10.0	17.4	9.9	7.4	2.5	23.724	9.883	3.94	15.4	IP-2		
15.0	26.6	21.1	5.6	5.0	23.853	7.539	3.94	14.6	IP-2		
20.0	32.5	26.9	5.6	6.1	23.872	6.655	3.94	14.4	IP-2		
25.0	41.0	35.4	5.6	8.2	23.499	4.989	3.94	14.2	IP-2		
30.0	52.1	46.5	5.6	10.5	22.987	3.938	3.94	14.1	IP-2		
31.0	257.0	50.2	206.8	52.9	22.987	1.110	3.94	13.5	IP-2		

Pile Type: HP
Diameter/Size: 14x73
Wall Thickness: n/a
Steel Area: 21.40 in²

Hammer: ICE IP-2
Hammer Energy: 17.358 kip-ft
Stroke: 3.94 ft
Hammer Cushion: GRL WEAP Default
Pile Cushion: None

Estimated Pile Length: 31 feet
Estimated Pile Embedment ¹: 30 feet
Yield Stress: 50 ksi
Maximum Allowable Stress: 40 ksi

Soil Strength Loss Assumed: No

Required Driving Resistance: N/A kips

Depth of Blow Count Limitation: None
Depth of Comp. Stress Limitation: None

Driving Limitation(s): None

Total Number of Blows: 166 (starting at penetration 5.0 ft)

Driving Time (min): 5 4 3 2 2 2 1 1 1 1

@Blow Rate (b/min): 30 40 50 60 70 80 90 100 110 120

Driving Time for continuously running hammer; any wait time not included.

- Notes:**
1. Top of piles assumed at top of ground surface. Embedment estimated using APILE.
 2. Wave equation analyses in the computer program GRLWEAP™ Version 14 by Pile Dynamics, Inc. was used to evaluate driveability. Default hammer parameters, including energy and efficiency provided in the GRLWEAP software, and manufacturer recommended cushion information was modeled in the analysis.
 3. The range of blows per foot considered acceptable was 25 bpf to 120 bpf (practical refusal).

Waugan Bridge
MaineDOT

Project: MainDOT Monmouth Bridges
Project No.: 67328
Structure: Waugan Bridge

Originator: MLR
Checker: BAM
Backchecker: MLR
Updater: MLR
Verifier: BAM

Date: 11/9/2022
Date: 7/20/2023
Date: 7/20/2023
Date: 7/20/2023
Date: 7/21/2023

Sheet No.: 1

Driveability Results





For	Maine DOT - Waugan	Job No	72998	Sheet No	
Made by	MLR	Check	JZ	Backchk	MLR
Date	10/28/2022	Date	11/23/2022	Date	11/23/2022

Soil Properties

Layer	Top of Layer Elevation (ft)	Bottom of Layer Elevation (ft)	Depth to Top of Layer (ft)	Depth to Bot. of Layer (ft)	N (bpf)	N ₆₀ (bpf)	N1 ₆₀ (bpf)	γ (pcf)	φ' (deg)	c* (psf)	K _(aw) (pci)	K _(bw) (pci)	G(ksf)	E(ksf)
Layer 1 - Fill	171.5	164.8	0	7	5	5	8	111	31		71	46	131	318
Layer 2 - Alluvium	164.8	161.5	7.0	13.5	4	4	5	108	30		52	37	121	290
Layer 3 - Marine Silt and Clay Crust	161.5	157.8	13.5	18	5	5	7	115	-	600	-	-	135	324
Layer 4 - Marine Silt and Clay	157.8	133.0	18.0	30.5	2	1	1	110	-	137-536	-	-	68	162
Layer 5 - Glacial Till	133.0	122.5	30.5	34.5	65	92	49	126	40		233	117	465	1289

*Cohesion estimated from Vane Shear Test performed on New Borings.

