

GEOTECHNICAL DESIGN REPORT

MERRILLS CORNER BRIDGE NO. 0714 OVER SHEPARDS RIVER

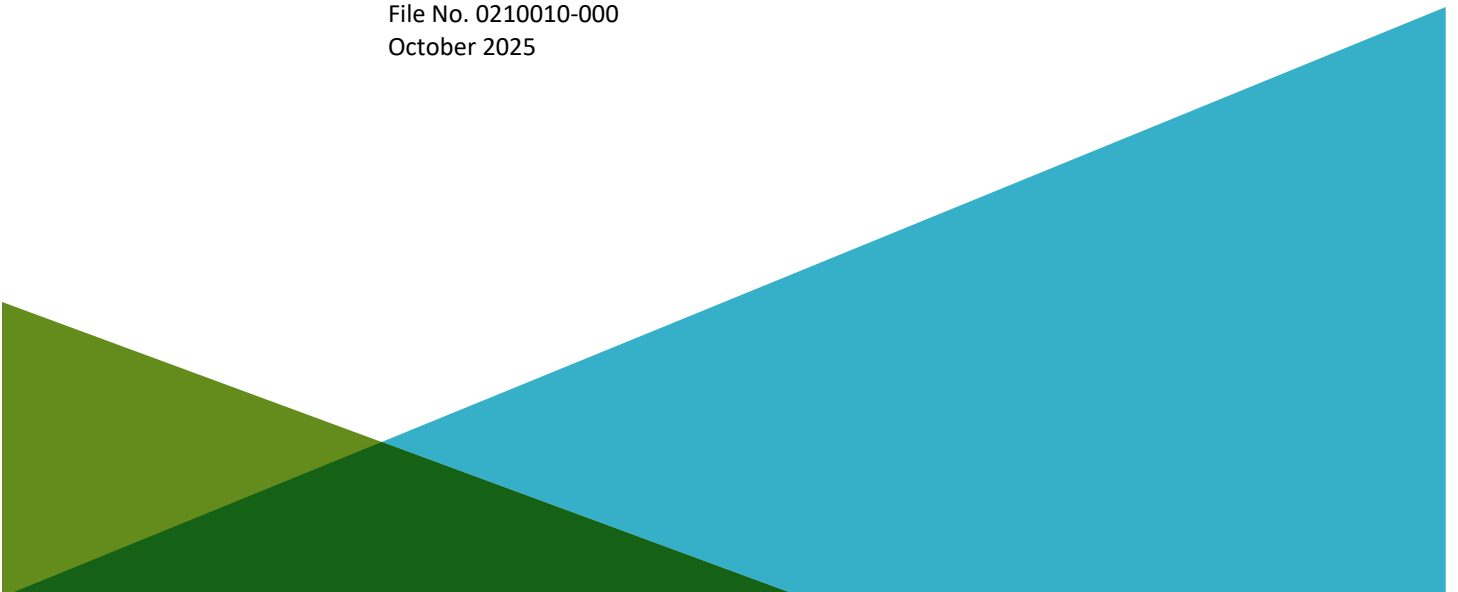
MAINEDOT WIN 27530.00

OLD COUNTY ROAD, BROWNFIELD, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

for
Maine Department of Transportation
Augusta, Maine

File No. 0210010-000
October 2025





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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Geotechnical Design Report
Merrills Corner Bridge No. 0714 over Shepards River
MaineDOT WIN 27530.00
Old County Road, Brownfield, Maine

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit herewith our report entitled, "Geotechnical Design Report, Merrills Corner Bridge No. 0714 over Shepards River, MaineDOT WIN 27530.00, Old County Road, Brownfield, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our proposed work scope and fee estimate document, dated 24 January 2024 and executed by you on 16 February 2024 (Bridge Program Assignment Letter #5) and the provisions of our Multi-PIN Project Contract Number 20200623000000000766, dated 27 August 2020. This GDR supersedes the Preliminary Geotechnical Design Report, which was issued to you on 28 February 2025.

Introduction

This GDR presents the results of geotechnical field investigation and laboratory testing program, technical evaluations, and geotechnical design recommendations completed by Haley & Aldrich on behalf of the Maine Department of Transportation (MaineDOT) for the proposed replacement bridge that will carry vehicular traffic on Old County Road over Shepards River in Brownfield, Maine (see Figure 1, Project Locus).

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

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

The proposed limit of work extends from Station (Sta.) 1+25 (south) to Sta. 3+75 (north).

PROJECT LOCATION, EXISTING SITE CONDITIONS, AND EXISTING BRIDGE STRUCTURE

The existing bridge carries southbound (SB) and northbound (NB) vehicular traffic on Old County Road over Shepards River in Brownfield, Maine, approximately 400 ft south of the intersection of Hampshire Road and Old County Road. The project site consists of a semi-wooded area. Existing roadway grades within the limit of work are generally flat and range from approximately El. 479 (south) to El. 472 (north). The bottom of Shepards River, which flows from west to east, within the limits of Old County Road, is approximately El. 463.

Based on our review of the final Preliminary Design Report (PDR), received on 31 October 2024 from the MaineDOT, it is our understanding that the existing bridge was originally constructed in 1930 (superstructure replaced in 1993 and repaired in 2008) and consists of an approximate 25-ft long, 15-ft wide (curb-to-curb), single-span structure supported by dry-laid stone founded on timber grillage (Abutment No. 1; south) and concrete encased stacked stone (Abutment No. 2; north). The superstructure consists of steel rolled girders with a timber deck.

In addition, based on our review of the bridge inspection reports (inspection reports) and the PDR, it is our understanding that the steel superstructure beams are in poor condition with moderate to heavy section loss, along with rusting and 100 percent paint failure. Additionally, the existing abutments are in critical condition. The northern abutment has been undermined in some sections, and the southern abutment has large voids in the stones and some of the timber grillage is exposed. The existing abutments have a critical condition scour rating. The PDR indicates that the south approach has settled approximately 2 inches (in.).

It is our understanding that historic bridge drawings are not available.

PROPOSED BRIDGE STRUCTURE

Based on our review of the PDR, it is our understanding that MaineDOT has developed and evaluated multiple replacement alternatives. Each alternative considered several factors, including but not limited to, overall project cost, maintenance of traffic, environmental impacts, and future structural maintenance. The on-alignment bridge replacement alternative recommended by MaineDOT in the PDR consists of a 51-ft long, single-span, skewed bridge, which is supported on integral abutments founded on steel H-piles bearing in/on bedrock.

Based on our review of the draft Plans, Specifications, and Estimate (PS&E) plans that were provided by MaineDOT on 22 October 2025, we understand the following:

- The bridge superstructure will be constructed using precast, prestressed solid slab beams and will be approximately 27-ft wide (curb-to-curb), which consists of two, 11-ft wide travel lanes and two, 2-½-ft wide shoulders.
- The proposed vertical curve across the replacement bridge will be slightly higher than the existing; therefore, a slight raise in grade (i.e., less than 1 ft) is anticipated.

- Old Country Road will be closed during construction, which will require an approximate 3- to 5-mile-long detour around the site.

Existing and proposed site conditions are shown in Figure 2, Boring Location Plan and Figure 3, Interpretive Subsurface Profile.

Geologic Setting

The surficial geology map (Maine Geological Survey, Surficial Geology of the Brownfield Quadrangle, Open-File No. 97-48, dated 1997) indicates near-surface soils present in the vicinity of the site consist of stream alluvium, which generally consists of sand, silt, gravel, organic material, and/or Shepards River valley deposit soils, which generally consists of sand and gravel.

The bedrock geology map (Maine Geological Survey, Bedrock Geology of the Portland 1:100,000 Quadrangle, Maine and New Hampshire, Open-File No. 98-1, dated 1998), indicates that bedrock at the site consists of foliated or lineated biotite granite.

Geotechnical Field Investigation

Haley & Aldrich completed a geotechnical field investigation at the site between 20 and 22 March 2024. Three borings, designated BB-BSR-101, BB-BSR-101A, and BB-BSR-102, were drilled adjacent to and on either side (south and north) of the existing bridge. Boring BB-BSR-101 was terminated at approximately 54 ft below ground surface (BGS) in glacial deposit soils due to broken casing. After boring BB-BSR-101 was terminated, the drill rig moved approximately 3 ft south and a second boring (BB-BSR-101A) was drilled so that subsurface information could be obtained below the depth that boring BB-BSR-101 was terminated.

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

Borings BB-BSR-101 and BB-BSR-101A were drilled by MaineDOT, using a CME 45C truck-mounted drill rig. Boring BB-BSR-102 was drilled by Seaboard Drilling, LLC. (Seaboard) working under contract to MaineDOT, using an Acker AD-2 truck-mounted drill rig. The borings were advanced to depths ranging from approximately 54 to 71 ft BGS using cased-washed drilling methods and 3-in. (NW-size) and 4-in. (HW-size) inside diameter (ID) steel casing.

Soil samples were generally collected continuously from ground surface to approximately 16 to 18 ft BGS and at standard 5-ft intervals thereafter, by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb automatic hammer dropped from a height of 30 in., as indicated on the boring logs. The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the boring logs. The uncorrected SPT N-value (N-uncorrected) is defined as the total

number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. Both drill rigs were equipped with a calibrated automatic hammer per MaineDOT requirements. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6. The energy-corrected SPT N-values are also provided on the boring logs. The theoretical hammer efficiency factor for each drill rig is presented below.

Drill Rig	Boring	Hammer Efficiency Factor	Theoretical Hammer Efficiency (percent)
CME 45C (MaineDOT)	BB-BSR-101 and BB-BSR-101A	0.962	96.2
Acker AD-2 (Seaboard)	BB-BSR-102	0.966	96.6

Unsaturated soil samples recovered from the borings were screened in the field with a photoionization detector (PID) for the presence of volatile organic compounds (VOCs). The measured PID readings are shown on the boring logs.

Approximately 10 to 13 ft of bedrock was sampled (cored) in borings BB-BSR-101A and BB-BSR-102. The bedrock was cored using a 2-in. (NQ-size) ID diamond-tipped core barrel.

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

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

Generalized Subsurface Conditions

The subsurface conditions present at the site generally consist of fill soils overlying naturally-deposited alluvial and glacial deposit soils and bedrock. Refer to Table II for a detailed summary of the geologic units and encountered thicknesses in each boring. A general description of each soil/bedrock unit is provided separately, below. Please note that glacial deposits encountered in the borings included glaciofluvial, glaciolacustrine, and glacial till soils. Because these deposits do not occur sequentially with increasing depth BGS in the borings and because the differences between the granular and cohesive portions of the deposits are subtle, their descriptions have been generalized for presentation purposes below. Detailed soil and bedrock descriptions are provided on the boring logs included in Appendix A. Refer to Figure 3 for a graphical representation of the subsurface conditions present along the proposed bridge alignment.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Bituminous Concrete	0.4	An approximate 0.4-ft thick layer of bituminous concrete was encountered at the ground surface in each boring.
Fill	5 to 10	Medium dense to very dense SAND with varying amounts of silt and gravel. <i>(encountered in borings BB-BSR-101 and BB-BSR-102 only)</i>
Alluvial Deposit	5	Medium stiff to very stiff Sandy SILT and ORGANIC SILT; dense fine SAND with some medium sand and little silt. <i>(encountered in boring BB-BSR-101)</i>
Glacial Deposit	12 to 47	Medium dense to very dense SAND with varying amounts of silt and gravel; very stiff to hard CLAY with varying amounts of silt, sand, and gravel; occasional cobble layers were encountered. <i>(encountered in each boring)</i>
Bedrock	Bedrock was encountered in borings BB-BSR-101A and BB-BSR-102. Top of bedrock surface encountered at depths ranging from approximately 57 to 61 ft BGS (El. 413 to El. 416) and slopes down from north to south.	

Please note that soil descriptions provided on the boring logs do not represent actual field conditions other than at specific boring locations. The actual conditions encountered between boring locations may vary from those described herein.

BEDROCK CONDITIONS

As stated previously, approximately 10 to 13 ft of bedrock was cored in borings BB-BSR-101A and BB-BSR-102. The sampled and recovered bedrock generally consisted of hard to very hard, fresh to slightly weathered, white, black, and grey, fine to coarse grained, biotite GRANODIORITE and biotite GRANITE. Primary joints were observed dipping at horizontal to low angles and are very close to moderately spaced, tight to open, and joint surfaces were rough. An intrusion of very hard, fresh, greyish black, aphanitic, BASALT was encountered in boring BB-BSR-102.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for the bedrock encountered in the borings drilled at the site ranged from 0 to 93 percent (average = 64 percent), indicating variable rock quality; from very poor to excellent in accordance with the MaineDOT Geotechnical Section “Key to Soil and Rock Descriptions and Terms Field Identification Information” document, dated May 2024.

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

GROUNDWATER CONDITIONS

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

- Recovered soil samples from each boring were visually observed to be “wet” at approximately 6 to 10 ft BGS (El. 463 to El. 468).
- The river water levels were measured approximately 8.4 ft below the existing bridge deck.

In addition, current flood levels in Shepards River, as determined by MaineDOT and reported in the draft PS&E plans, are summarized below.

Discharge	Headwater Elevation
Q _{1.1}	466.3
Q ₂₅	473.2
Q ₅₀	473.5
Q ₁₀₀	473.8

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

Geotechnical Laboratory Testing Program

A geotechnical laboratory testing program was undertaken on disturbed soil samples during the geotechnical field investigation to assist in soil classification and determination of engineering soil properties. Geotechnical laboratory testing was performed by the MaineDOT Central Laboratory in Bangor, Maine. All laboratory testing was performed in accordance with applicable ASTM International testing procedures. A summary of the lab testing results is provided below.

Laboratory Test	ASTM Test Designation	Geologic Unit	No. of Tests	Range in Test Results ¹
Moisture Content	ASTM D2216-98	Fill	3	2.8% < WC < 21.0%
		Alluvial Deposit	2	2.8% < WC < 44.2%
		Glacial Deposit	9	2.8% < WC < 44.2%
Organic Content	ASTM D2974	Fill	1	OC = 7.4%
		Alluvial Deposit	1	OC = 2.9%
Atterberg Limits	ASTM D4318-98	Fill	1	NP
		Alluvial Deposit	1	NP
Grain Size	ASTM D422	Fill	3	<u>AASHTO Classification:</u> A-2-4, A-1-b <u>USCS Classification:</u> SM, SC-SM, SW-SM
		Alluvial Deposit	2	<u>AASHTO Classification:</u> A-4, A-2-4 <u>USCS Classification:</u> ML, SM
		Glacial Deposits	9	<u>AASHTO Classification:</u> A-2-4, A-4, A-1-b <u>USCS Classification:</u> SM, CL, SW-SM

Note:

¹ NP = Non-plastic; WC = Moisture Content; OC = Organic Content

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

Geotechnical Evaluations and Design Recommendations

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

- American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Ninth Edition, 2020, referred to herein as AASHTO LRFD.

- MainedOT Bridge Design Guide (BDG), August 2003, with Interim Revisions through June 2018, referred to herein as BDG.

Engineering calculations that support the design recommendations included herein are provided for reference in Appendix C.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Site class was determined in accordance with AASHTO LRFD Section 3.10.3.1, using Method B. In instances where SPT N-values were equal to 0 (i.e., weight of rod or weight of hammer), were greater than 100 blows per foot (bpf) or where bedrock was present, default values of 1 and 100 bpf were used, respectively.

Based on the nature and thickness of the overburden soils and depth to bedrock at the site, as determined from the borings, we recommend the site be considered “Seismic Site Class D.” Spectral accelerations were determined based on the geographic site location and the recommended “Site Class D” designation using the United States Geological Survey (USGS) Seismic Design Web Service, which is based on the AASHTO recommended response spectra for a 7 percent probability of exceedance in 75 years (approximately 1,000-year return period). The recommended seismic design parameters are summarized below.

Design Parameter ¹	Design Value
Site factor for short-period range of acceleration response spectrum, F_a =	1.600
Site factor for long-period range of acceleration response spectrum, F_v =	2.400
Site factor at zero-period on acceleration response spectrum, F_{pga} =	1.595
Horizontal response spectral acceleration coeff. at 0.2-s period on rock, S_s (g) =	0.199
Horizontal response spectral acceleration coeff. at 1.0-s period on rock, S_1 (g) =	0.049
Peak seismic ground acceleration coeff. on rock, PGA (g) =	0.102
Horizontal response spectral acceleration coeff. at 0.2-s period modified by F_a , S_{Ds} (g) =	0.318
Horizontal response spectral acceleration coeff. at 1.0-s period modified by F_v , S_{D1} (g) =	0.118
Peak seismic ground acceleration coefficient modified by F_{pga} , A_s (g) =	0.163

Note:

¹ g = gravitational acceleration constant

In accordance with AASHTO LRFD Section 3.10.6, the site falls within Seismic Zone 1 based on the calculated value of S_{D1} (i.e., $S_{D1} < 0.15g$ = Seismic Zone 1 from AASHTO LRFD Table 3.10.6.1).

Based on our review of the soil conditions encountered in the borings and the results of the laboratory testing, it is our opinion that the potential for saturated granular soils present at the site to liquefy during the design earthquake event is low.

APPROACH EMBANKMENT DESIGN CONSIDERATIONS

Elastic and Consolidation Settlement

As stated above, existing site grades along Old County Road will be raised a maximum of approximately 1 ft to meet the proposed finish grades. Because of the limited amount of raise-in-grade height along Old County Road, and because compressible soils (e.g., soft organic and/or cohesive soils) were not encountered in the borings, we anticipate that any elastic settlement that does occur will be negligible and will likely occur rapidly during and immediately after fill placement and backfilling operations are completed, prior to final paving.

Global Embankment Stability

Embankment construction using normal weight earthfill can cause excessive vertical and lateral strains, potentially resulting in a shear failure of the foundation soil and subsequent failure of the embankments. A series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 9.0 to evaluate the likelihood of global stability failures of proposed embankments.

Due to the downward-sloping roadway grade behind Abutment No. 1 (south to north), we performed static and pseudo-static stability evaluations in the longitudinal direction (i.e., parallel to the existing roadway) through Abutment No. 1. Similar evaluations were also conducted through Abutment No. 2 (north to south), as the existing fill layer behind Abutment No. 2 is thicker than behind Abutment No. 1, which could pose a greater risk of instability.

Based on our review of the draft PS&E grading plans and the subsurface conditions encountered in the borings, it is our opinion that deep-seated rotational global stability failures in the transverse direction (i.e., perpendicular to the existing roadway) are unlikely. This assessment is supported by the proposed embankment side slope fill heights (maximum fill height of 1.6 ft) and the embankment side slope inclinations (7H:1V at Abutment No. 1 and 2H:1V at Abutment No. 2) being less than the soil's internal friction angle. Because of this, transverse global stability evaluations were not completed.

Soil descriptions, correlations with SPT N-values, geotechnical laboratory test results, and our experience were used to select soil properties (i.e., unit weight, friction angle, and undrained shear strength) for use in completing stability analyses. The physical and strength properties used to complete the stability evaluations are presented below.

Geologic Unit	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
New Fill (Granular Borrow)	125	30	0
Riprap	140	45	0
Existing Fill	125	28	0
Alluvial Deposit (Cohesive)	120	0	400

Geologic Unit	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
Alluvial Deposit (Cohesionless)	125	34	0
Glacial Deposits (Cohesive)	130	0	4,000
Glacial Deposits (Cohesionless)	130	40	0
Bedrock	Infinite Strength		

In addition, a 250 psf live load surcharge was assumed to act over the embankment width. Please note that abutment piles, which provide a resisting force, were not included in our evaluations.

The pseudo-static stability evaluations were completed using a horizontal acceleration coefficient, k_h , equal to 0.08g (i.e., one-half of the acceleration coefficient, A_s). A value of $A_s/2$ was selected in accordance with AASHTO LRFD guidance in Section 11.6.5.2.2; the reduction from A_s is due to soil slope flexibility and the fact that the peak ground acceleration during an earthquake lasts only for a very short period of time.

Circular failure surfaces were assessed using Spencer's and Bishop's methods of analysis. Non-circular failure surfaces were assessed using Spencer's method of analysis.

The calculated lowest global stability factors of safety values at Abutment Nos. 1 and 2 in the longitudinal direction are summarized below.

Location	Orientation	Calculated Lowest Factor of Safety	
		Static	Pseudo-Static
Abutment No. 1	Longitudinal	1.51	1.29
Abutment No. 2		1.50	

The minimum factor of safety required for static stability evaluations is 1.3 where the geotechnical parameters and subsurface stratigraphy are well defined, based on the requirements of AASHTO LRFD Article 11.6.3.7. It is our opinion that the geotechnical parameters and subsurface stratigraphy are well defined at this site. Per recent MaineDOT guidance, it is our understanding that the minimum required factor of safety for embankments subjected to pseudo-static (seismic) loading is 1.1.

The results indicate that the proposed embankments in the longitudinal direction meet the minimum required factors of safety under static and pseudo-static loading conditions assuming standard embankment construction and the use of normal weight earthfill.

ABUTMENT FOUNDATION SUPPORT

As discussed above and as shown on the interpretive subsurface profile (Figure 3), the subsurface conditions along the proposed bridge alignment consist primarily of fill overlying alluvial and glacial deposit soils, and bedrock. We consider the glacial till soils and bedrock suitable to support the proposed bridge abutments. Based on the depth to the suitable foundation bearing strata, scour considerations, and the anticipated magnitude of the factored pile loads, it is our opinion that driven steel H-piles are the most practicable and cost-effective foundation alternative.

Based on discussions with MaineDOT, it is our understanding that MaineDOT prefers the use of HP14x89 steel H-piles to support the bridges' integral abutments.

Corrosion and Deterioration

The geotechnical engineering design of the proposed piles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. Based on our visual observation of the soil samples and our experience on similar projects with similar soil conditions, it is our opinion that the in-situ soils have low corrosive potential. Because of this, the net factored pile resistances recommended herein do not include a reduction in pile cross sectional area for steel degradation.

Static Axial Compressive Pile Resistance

As discussed previously, it is our opinion that steel H-piles will need to be driven to/into bedrock to provide the minimum required axial compressive resistance, primarily through end bearing resistance, considering the nature, consistency, and thickness of overburden soils (i.e., fill, alluvial, and glacial soils).

Structural Resistance

In accordance with AASHTO LRFD Section 10.7.3, a distinction is made between piles driven to "soft" bedrock and those that are driven to "hard" bedrock. Based on our experience on similar projects with similar bedrock conditions, the sampling, recovery, and RQD measurements summarized herein, observation of the recovered bedrock core samples, and the results of laboratory testing, it is our opinion that the bedrock present at the abutments be classified as "hard." The geotechnical resistance of piles driven to end bearing in "hard" bedrock is controlled by the structural resistance as discussed in AASHTO LRFD Section 10.7.3.2.3. The structural resistance factor (Section 6.5.4.2) for axial resistance of piles in compression and subject to damage due to severe driving ("hard" bedrock bearing stratum) is 0.5. In addition, resistance factors for Service (SER) and Extreme Event (EE) Limit State loading are 1.0. Therefore, the nominal and factored structural resistances of HP14x89 steel H-piles at the SER, STR, and EE Limit States are summarized below.

Steel H-pile Section	Nominal Structural Resistance (kips)	Factored Structural Resistance (kips)		
		SER ($\phi = 1.0$)	STR ($\phi = 0.5$)	EE ($\phi = 1.0$)
HP14x89	1,305	1,305	653	1,035

Drivability Resistance

The design of the proposed abutment piles also included consideration of drivability resistance in accordance with AASHTO LRFD Section 10.7.8. The drivability evaluations were conducted using the computer program GRL WEAP 14 v. 14.1.20.1 (WEAP) developed by GRL Engineers, Inc. The drivability analyses were conducted to determine 1) if the piles could be impact driven through overburden soils to bedrock and 2) what nominal resistance could be achieved using hammer sizes typical of local pile driving contractors without damaging or overstressing the piles while keeping the penetration resistance below 15 blows per inch (bpi), which is the upper limit of penetration resistance typically allowed by the MaineDOT Standard Specifications.

The drivability evaluations were conducted assuming that a Delmag D25-32 single acting diesel hammer with a maximum rated energy equal to 66,340 ft-lbs. will be used to install the HP14x89 steel H-piles. The Delmag D25-32 hammer has a ratchet style fuel pump with four settings. The maximum fuel setting was used for the drivability analyses. Stroke height, drivability resistance, maximum compressive stress, and penetration resistance are summarized below.

Steel H-pile Section	Pile Hammer	Pile Hammer Fuel Setting (stroke, ft)	Drivability Resistance (kips)		Maximum Compressive Stress (ksi)	Penetration Resistance (bpi)
			Nominal	Factored ($\phi_{dyn} = 0.65$)		
HP14x89	Delmag D25-32	max (11.0)	800	520	44	9

As shown in the above table, the drivability resistance for HP14x89 steel H-piles is controlled by the maximum compressive driving stress.

Please note that the drivability results summarized above are based on an assumed pile-hammer size, fuel setting, stroke, and the assumption that the piles penetrate through the entire soil overburden and end bear in/on bedrock.

Summary of Axial Compressive Pile Resistances

The factored axial compressive pile resistance is controlled by the lesser of the STR Limit State drivability and structural resistances since the piles will be installed using impact hammers and the minimum nominal resistance will be confirmed in the field using dynamic testing. The factored resistances (structural and drivability) are summarized below.

Steel H-pile Section	Factored Structural Resistance (kips)	Factored Drivability Resistance (kips)	Governing Factored Resistance (kips)
HP14x89	653	520	520

As summarized above, the factored STR Limit State axial compressive pile resistance is controlled by the drivability resistance. Therefore, we recommend that net factored STR Limit State axial compressive resistances equal to 520 kips for HP14x89 steel H-piles be used to support the bridge abutments. Please note that based on our discussions with you, it is our understanding that the maximum factored STR compressive pile load is approximately 263 kips, is less than the governing factored resistance presented above.

MaineDOT standard practice is to dynamically test (during construction) a minimum of one pile per abutment and not less than 2 percent of the total number of piles anticipated to confirm that the minimum required nominal resistance has been achieved unless variable subsurface or sloping bedrock conditions are present. The top of bedrock surface encountered in borings drilled at Abutment No. 1 and Abutment No. 2 varied from approximately El. 413 to El. 416, respectively, which in our opinion, does not reflect highly variable or sloping bedrock conditions in the longitudinal direction. Because of this, we recommend that one pile at each abutment be dynamically tested during the end of initial drive (EOID) and beginning of restrike (BOR) during construction and prior to the installation of production piles.

We also recommend that minimum pile spacing, clearance, and embedment (into the pile cap) meet the requirements of AASHTO LRFD Section 10.7.1.2. The pile tips should also be protected using cast steel driving shoes to prevent damage when driving through the very dense glacial soils and to/into bedrock.

Lateral Pile Analysis

As stated above, both abutments are being designed as integral abutments; therefore, lateral pile analyses are required to investigate the effect that thermal displacements and axial compressive loading have on bending moments and combined stresses in the piles.

Based on our discussions with MaineDOT, we understand that the axial compressive pile loads, and thermal deflections are equal for both abutments. Given this and considering that the soils at the top of piles at Abutment No. 1 are slightly less dense than those at Abutment No. 2, we performed lateral pile analyses for Abutment No. 1 only.

Haley & Aldrich completed lateral pile analyses in accordance with the requirements of Bridge Design Guide Section 5.4.2.4 and the recommendations included in the “Integral Abutment Bridge Design Guidelines” by the Vermont Agency of Transportation (VTrans) using the computer program LPile v. 2022 (LPile) developed by Ensoft, Inc. using the soil profiles and soil properties summarized below.

Stratum	Soil Model	Top of Stratum Elevation	Stratum Thickness (ft)	Effective Unit Weight (pcf)	Friction Angle (ϕ , degrees)	Undrained Shear Strength (psf)	Modulus (k, pci)	Strain Factor (ϵ_{50})
Alluvial Deposit	Sand (Reese)	464.6	0.7	62.6	34	NA	160	NA
Glacial Till/ Glaciofluvial Deposit	Sand (Reese)	463.9	34	67.6	40	NA	160	NA
Glaciolacustrine Deposit	Stiff Clay w/o Free Water (Reese)	429.9	16.8	67.6	NA	4,000	NA	0.005

Steel H-pile head boundary conditions included a factored STR axial compressive pile load of 263 kips, a pile head deflection corresponding to the thermal deflection of 0.19 in. (expansion and contraction cases) in weak-axis bending, and a fixed-head condition (zero rotation condition). The ground surface levels used in the evaluations were El. 471.7 (expansion case; proposed finish grade behind the abutment) and El. 468.6 (contraction case; proposed top of shelf in front of the abutment). HP14x89 steel H-pile properties were used for weak-axis bending with a 50 kips per square inch (ksi) yield strength. The results of the lateral pile analyses are summarized below.

Pile Section	Case	Max. Factored Axial Comp. Pile Load (kips)	Shear Force Resulting in 0.19 in. of Pile Head Displacement (kips)	Max. Moment at Pile Head (kip-ft)	Depth Below Pile Head to Fixity ¹ (ft)	Total Stress at Pile Head (ksi)	Unbraced Length	
							Top Segment (L1, ft)	Bottom Segment (L2, ft)
HP14x89	Exp.	262.9	74.7	162.5	16.1	54.2	2.8	9.1
	Cont.		59.3	141.6	16.7	48.5	3.0	9.4

Note:

¹ Taken as the depth along the pile to the second point of zero deflection.

The lateral pile analysis results presented above were provided to MaineDOT for review and to determine if a plastic hinge analysis would be required. MaineDOT analyzed the results and determined that a plastic hinge did not form and that the HP14x89 steel H-piles oriented for weak-axis bending are suitable to resist the applied loads and displacements.

Estimated Pile Tip Elevations and Pile Lengths

As discussed previously, the piles are expected to develop most of their axial compressive resistance through end bearing in/on bedrock. For estimating pile length quantities, we recommend the following:

Substructure	Approximate Bottom of Pile Cap Elevation	Approximate Top of Pile Elevation	Approximate Top of Bedrock Elevation ¹	Approximate Pile Tip Elevation ²	Approximate In-Place Pile Length (ft)
Abutment No.1	466	468	413	412	56
Abutment No. 2	465	467	416	415	52

Notes:

¹ Based on top of bedrock elevations encountered in borings BB-BSR-101A and BB-BSR-102.

² Based on the assumption that the piles penetrate 1 ft into bedrock.

Please note that the recommended in-place pile lengths presented above do not account for an additional 5 ft of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and pile driving equipment. We recommend that the Contractor evaluate the subsurface conditions present at the site and estimate actual delivered pile lengths at each substructure location, which will be submitted to the Resident for review prior to ordering in accordance with the requirements of the Standard Specifications.

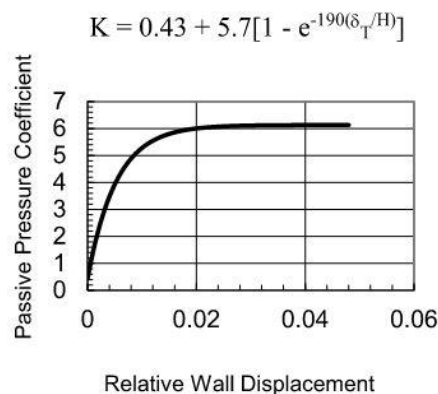
Based on the recommended pile lengths summarized above, we anticipate that minimal pile splicing will be needed. The actual number of pile splices may vary and is dependent on the actual order lengths proposed by the Contractor and the final pile tip elevations. Please note that Standard Specification Section 501.047 limits the total number of field splices to two for pile lengths ranging between 35 and 79 ft and also requires that the pile piece of lesser length be placed at the tip of the pile (i.e., the first part of the pile that enters the ground).

ABUTMENT BREASTWALL AND WINGWALL DESIGN

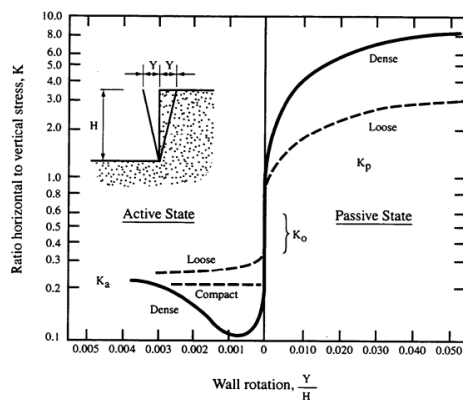
Design recommendations for abutment breastwalls and wingwalls are provided below.

- Drainage:
 - The abutment breastwall and wingwall designs should include a drainage system to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the structure. We recommend that drainage be provided in accordance with the requirements of BDG Section 5.4.2.13.
- Lateral Earth Pressures:
 - Recommendations summarized below are based on the following:

- Abutment breastwall and wingwalls are backfilled with a free-draining material (i.e., Soil Type 4, BDG Table 3-3; total unit weight = 125 pcf; internal angle of friction = 32 degrees; interface friction angle = 24 degrees).
- The abutment breastwall and wingwall backwalls are vertical.
- Adequate drainage is provided, as recommended herein and in accordance with the requirements of the BDG, to eliminate the potential for unbalanced hydrostatic pressures to develop.
- Backfill surfaces behind the abutments that are inclined at approximately 12 percent (Abutment No. 1) and -11 percent (Abutment No. 2).
- Integral abutments should be designed using a static passive lateral earth pressure coefficient (k_p) calculated using the “movement-dependent” coefficient in accordance with AASHTO LRFD Article C3.11.5.4 and Federal Highway Administration (FHWA) NHI-06-089. We recommend using the more stringent methodology developed by the Massachusetts Department of Transportation (MassDOT) or FHWA NHI-06-089, Figure 10-4 as outlined below. We also recommend that the k_p value used be no less than 3.25 (calculated using Rankine theory).



Plot of Passive Pressure Coefficient, K vs. Relative Wall Displacement, δ_r/H (MassDOT).



Plot of Passive Pressure Coefficient, K vs. Relative Wall Displacement, γ/H (FHWA).

Magnitude of Wall Rotation to Reach Failure

Soil type and condition	Rotation, Y/H	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

- We recommend a load factor for passive earth pressure equal to 1.5 be used to calculate factored passive lateral earth pressures consistent with MaineDOT standard practice.
- Wingwalls:
 - The Coulomb active earth pressure coefficients should be used for wall designs that are “gravity-shaped” or short-heeled, cantilever-types where the top of the stem wall interferes with the shear zone. In addition, Rankine active earth pressure coefficients should be used for long-heeled cantilever-type walls. Coulomb and Rankine active earth pressure coefficients are provided below for various backslope angles (e.g., horizontal, 2H:1V, 1.75H:1V).

Backslope Angle (degrees)	Rankine Active Earth Pressure Coefficient	Coulomb Active Earth Pressure Coefficient
0 (horizontal)	0.31	0.27
26.6 (2H:1V)	0.46	0.46
29.7 (1.75H:1V)	0.56	0.54

- In general, we recommend that walls be designed for a live load surcharge equivalent to the earthfill height summarized in AASHTO LRFD Tables 3.11.6.4-1 and 3.11.6.4-2. A uniform lateral load equal to the surcharge times the lateral earth pressure coefficient should also be applied to abutments and walls to account for the live load surcharge.

Frost Protection

The minimum depth of embedment/cover for both abutments were evaluated in accordance with Section 5.2.1 of the BDG. Based on a design freezing index equal to 1,380 freezing degree-days, we recommend that the abutments bear a minimum of 6.5 ft below the lowest adjacent ground surface exposed to freezing.

Limitations

This GDR is prepared for the exclusive use of MaineDOT relative to the Merrills Corner Bridge replacement project in Brownfield, Maine. There are no intended beneficiaries other than MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the GDR. Use of this GDR by any person or entity other than MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from MaineDOT and from Haley & Aldrich indicating that the GDR is adequate for such other use. Use of this GDR by such other person or entity without the written authorization of MaineDOT and Haley & Aldrich shall be at such other person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

The analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this GDR.

We understand that this GDR will be included as a reference document in the package that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the Contract Documents (CDs; plans, Standard Specifications, Special Provisions) and that the information contained in the CDs takes precedence over the information provided in this GDR.

Closure

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

Sincerely yours,

HALEY & ALDRICH, INC.



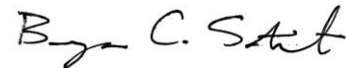
Tucker P. Jones, P.E.

Project Geotechnical Engineer



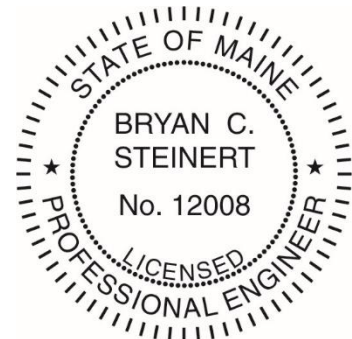
Nathan A. Sherwood, P.E.

Senior Project Manager



Bryan C. Steinert, P.E.

Senior Associate



Enclosures:

- Table I – Subsurface Exploration Location Data
- Table II – Subsurface Exploration Subsurface Data
- Table III – Subsurface Exploration Bedrock Core Data
- Figure 1 – Project Locus
- Figure 2 – Boring Location Plan
- Figure 3 – Interpretive Subsurface Profile
- Appendix A – Boring Logs and Bedrock Core Photographs
- Appendix B – Geotechnical Laboratory Test Results
- Appendix C – Geotechnical Calculations

References

1. Map entitled, "Surficial Geology of the Brownfield Quadrangle, Maine," Open-File No. 97-48, prepared by P. Thompson Davis and William R. Holland, Maine Geological Survey, Department of Conservation, dated 1997.
2. Map entitled, "Bedrock Geology of the Portland 1:100,000 Quadrangle, Maine and New Hampshire," Open-File No. 98-1, prepared by Henry N. Berry IV and Arthur M. Hussey II, Maine Geological Survey, Department of Conservation, dated 1998.

TABLE I

Subsurface Exploration Location Data
Merrills Corner Bridge No. 0714 over Shepards River
MaineDOT WIN 27530.00
Old County Road, Brownfield, Maine

Haley & Aldrich, Inc. File No.: 0210010-000

Boring No. ¹	Ground Surface Elevation ² (ft)	Station ³	Offset Distance (ft) & Direction ⁴	Horizontal Coordinates ⁵	
				Northing (Y)	Easting (X)
BB-BSR-101	473.9	2+12.2	1.7 LT	402,141	833,124
BB-BSR-101A	473.9	2+10.3	1.7 LT	402,139	833,125
BB-BSR-102	473.0	2+71.7	3.5 RT	402,200	833,112

Notes:

¹ Boring locations are shown on Figure 2, Boring Location Plan.

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

³ Station and offset information shown is considered approximate, is relative to project baselines, and was determined by MaineDOT.

⁴ LT = offset distance left of baseline (looking upstation); RT = offset distance right of baseline (looking upstation).

⁵ As-drilled coordinates of borings were determined by MaineDOT using GPS survey equipment, are measured in feet and reference the NAD83, Maine 2000 West Zone coordinate system.

	Individual	Date
Prepared By:	T. Jones	8/6/2024
Checked By:	N. Sherwood	10/10/2024
Reviewed By:	B. Steinert	1/28/2025

TABLE II
Subsurface Exploration Subsurface Data
Merrills Corner Bridge No. 0714 over Shepards River
MaineDOT WIN 27530.00
Old County Road, Brownfield, Maine

Haley & Aldrich, Inc. File No.: 0210010-000

Boring No. ¹	Ground Surface Elevation ² (ft)	Stratigraphic Data ^{3,4} (ft)												Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration (ft)
		Bituminous Concrete Thickness (ft)	Fill			Alluvial Deposit			Glacial Deposits ⁵			Bedrock			
			Depth to Top (ft)	Elevation of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elevation of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elevation of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elevation of Top (ft)		
BB-BSR-101	473.9	0.4	0.4	473.5	4.8	5.2	468.7	4.8	10.0	463.9	>44.0	--	--	54.0	419.9
BB-BSR-101A	473.9	--	--	--	--	--	--	--	< 49.0	> 424.9	> 11.8	60.8	413.1	71.0	402.9
BB-BSR-102	473.0	0.4	0.4	472.6	9.6	NE	NE	NE	10.0	463.0	47.0	57.0	416.0	70.5	402.5

Notes:

¹ Boring locations are shown on Figure 2, Boring Location Plan.

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

³ "NE" indicates stratum was not encountered in boring; "--" indicates boring not drilled deep enough or sampling not completed to determine presence of stratum; ">44.0/<44.0" indicates boring not drilled deep enough to determine full thickness of stratum; actual thickness could be greater than value shown.

⁴ Boring BB-BSR-101A was advanced to a depth of 49 ft without sampling. First sample was taken at a depth of 49 ft in the Glacial Deposit soils.

⁵ "Glacial Deposits" refers to "Glaciofluvial", "Glaciolacustrine", and "Glacial Till" soil deposits. Refer to Figure 3, Interpretive Subsurface Profile for a graphical representation of the depths, thicknesses, and sequencing of these deposits.

	Individual	Date
Prepared By:	T. Jones	8/6/2024
Checked By:	N. Sherwood	10/16/2024
Reviewed By:	B. Steinert	1/28/2025

TABLE III
Subsurface Exploration Bedrock Core Data
Merrills Corner Bridge No. 0714 over Shepards River
MaineDOT WIN 27530.00
Old County Road, Brownfield, Maine

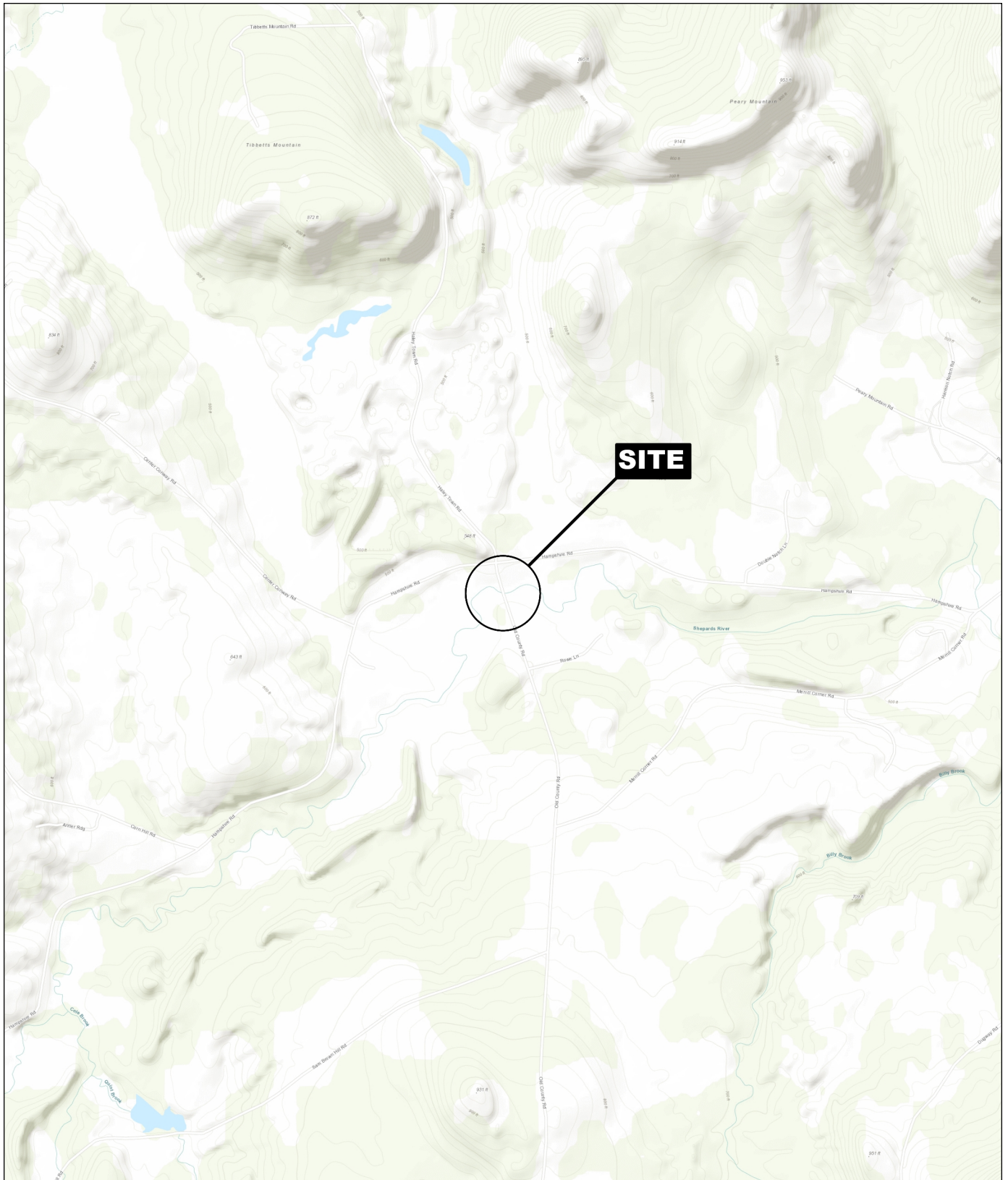
Haley & Aldrich, Inc. File No.: 0210010-000

Boring No. ¹	Ground Surface Elevation ² (ft)	Bedrock Core Diameter (in.)	Run					Total Core Recovery ³		Rock Quality Designation ^{4,5}			Physical Rock Parameters		Lithologic, Bedrock Mass, and Discontinuity Description ⁶
			No.	Depth Below Ground Surface (ft)			Total Length (in.)	Recovered Length (in.)	%	Length (in.)	%	Rock Quality	Weathering	Estimated Field Strength	
				Top	Bottom	Midpoint									
BB-BSR-101A	473.9	NQ (2")	R1	61.0	66.0	63.5	60	60	100%	56	93%	Excellent	Fresh	Very Hard	White to grey, fine to coarse grained biotite GRANODIORITE, joints horizontal to low angle, very close to moderately close, tight to open, rough, planar, fresh.
			R2	66.0	71.0	68.5	60	55	92%	52	87%	Good	Fresh	Very Hard	
BB-BSR-102	473.0	NQ (2")	R1	58.0	60.5	59.3	30	17	57%	0	0%	Very Poor	Slightly Weathered	Very Hard	White, black, and grey biotite GRANITE, joints horizontal, very close to close, rough, planar, slightly weathered, open.
			R2	60.5	65.5	63.0	60	60	100%	45	75%	Fair	Fresh to Slightly Weathered	Very Hard	White, black, and grey biotite GRANODIORITE, joints horizontal to low angle, very close to close, rough, planar, slightly weathered, open. At 62.2 ft transitions to greyish black aphanitic BASALT, joints horizontal to low angle, very close to close, rough, planar, slightly weathered, open.
			R3	65.5	70.5	68.0	60	52	87%	40	67%	Fair	Fresh	Hard to Very Hard	Greyish black aphanitic BASALT, joints horizontal, very close to close, rough, planar, slightly weathered, open. At 66.4 ft transitions to white to tan, fine to coarse grained, pegmatite GRANITE, joints horizontal to low angle, very close to close, rough, planar, slightly weathered, open.

Notes:

- ¹ As-drilled boring locations are shown on Figure 2, Boring Location Plan.
- ² Ground surface elevations at as-drilled boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet, and reference the North American Vertical Datum of 1988 (NAVD 88).
- ³ Total core recovery (TCR) is the length of core recovered divided by the length of the run.
- ⁴ Rock Quality Designation (RQD) is the total length of intact, full-diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of at least 4 in.) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.
- ⁵ Designation based on RQD in accordance with MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms" Field Identification Information, dated May 2024.
- ⁶ Refer to the boring logs and bedrock core photographs in Appendix A for additional information.

	Individual	Date
Prepared By:	T. Jones	8/6/2024
Checked By:	N. Sherwood	10/16/2024
Reviewed By:	B. Steinert	1/28/2025



0210010_000_LOCUS_HALEYALDRICH10J000ES



SITE COORDINATES: 43°56'06"N, 70°56'55"W



MAP SOURCE: USGS

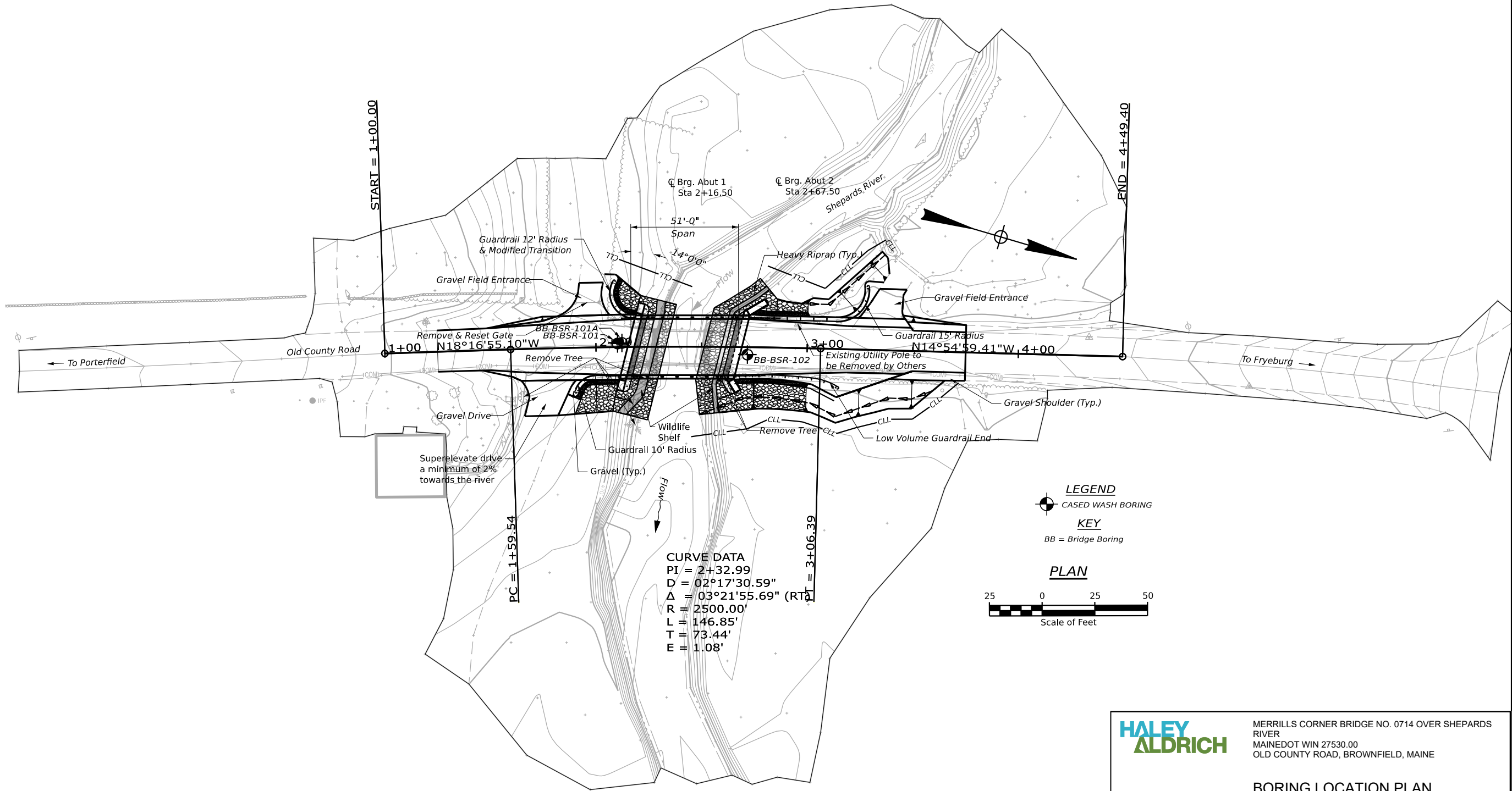
**HALEY
ALDRICH**

MERRILLS CORNER BRIDGE NO. 0714 OVER SHEPARDS RIVER
MAINEDOT WIN 27530.00
OLD COUNTY ROAD, BROWNFIELD, MAINE

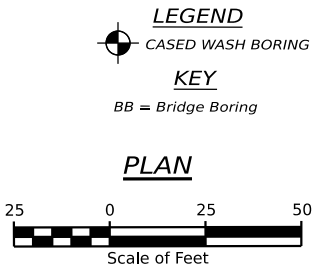
PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 2,000 FEET
OCTOBER 2025

FIGURE 1



CURVE DATA
PI = 2+32.99
D = 02°17'30.59"
Δ = 03°21'55.69" (RT)
R = 2500.00'
L = 146.85'
T = 73.44'
E = 1.08'



HALEY
ALDRICH

MERRILLS CORNER BRIDGE NO. 0714 OVER SHEPARDS RIVER
MAINEDOT WIN 27530.00
OLD COUNTY ROAD, BROWNFIELD, MAINE

BORING LOCATION PLAN

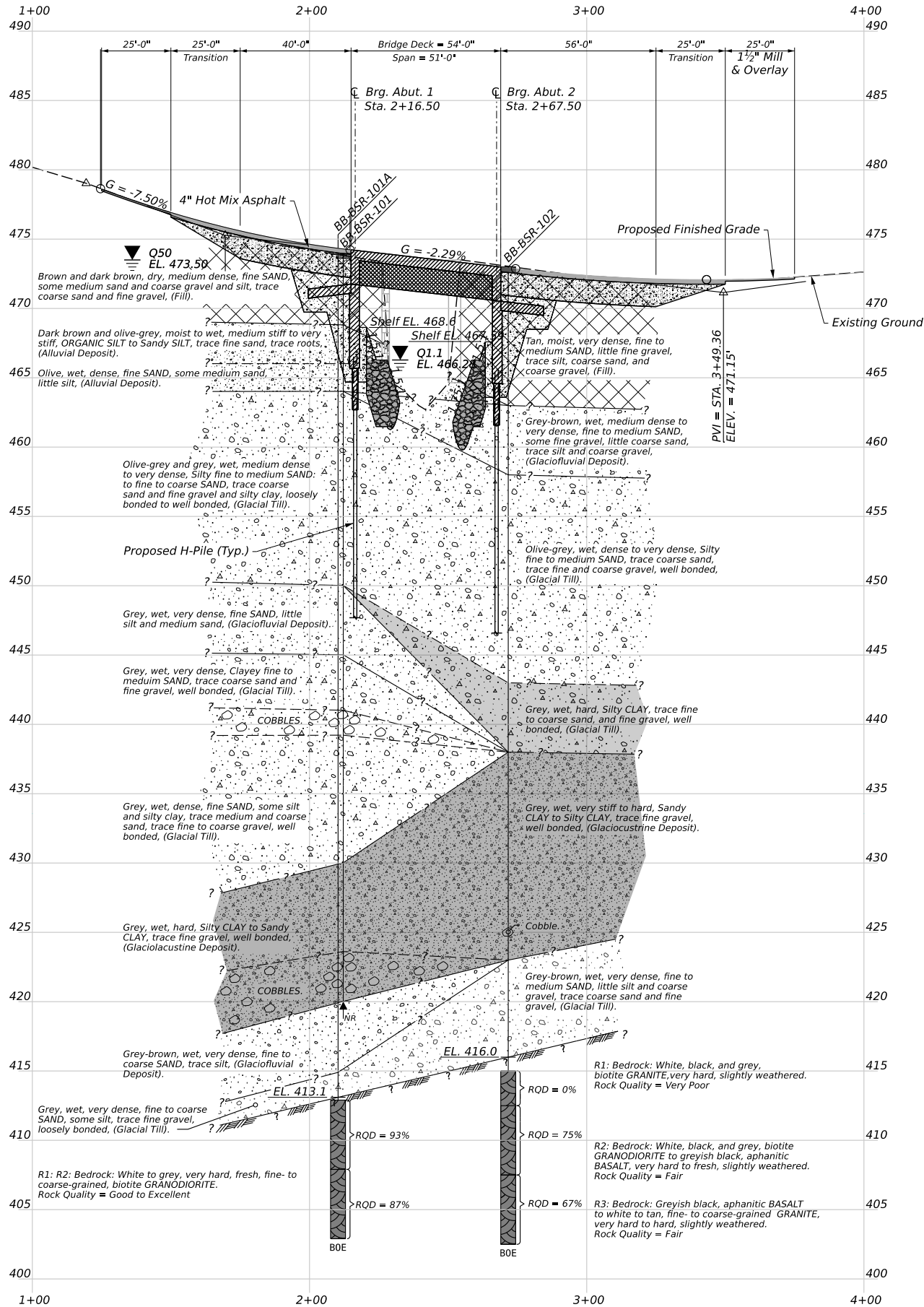
SCALE: AS SHOWN
OCTOBER 2025

FIGURE 2

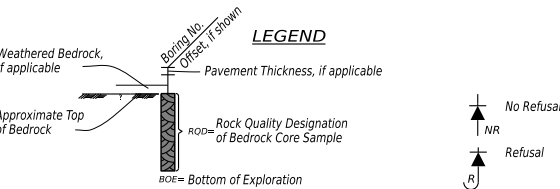
MERRILLS CORNER BRIDGE NO. 0714
CROSSING SHEPARDS RIVER
BROWNFIELD

PROJ. MANAGER	JULIE BRASK	BY	DATE
DESIGN-DETAILED	EDB	EDB	10/14/2024
CHECKED-REVIEWED	EDB		
DESIGN-DETAILED	T. JONES	T. WHITE	OCT 2025
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			
SIGNATURE			
P.E. NUMBER			
DATE			

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
Federal No. 2753000
WIN 27530.00



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



PROFILE

HALEY
ALDRICH

MERRILLS CORNER BRIDGE NO. 0714 OVER SHEPARD'S RIVER
MAINEDOT WIN 27530.00
OLD COUNTY ROAD, BROWNFIELD, MAINE

INTERPRETIVE SUBSURFACE
PROFILE

SCALE: AS SHOWN
OCTOBER 2025

FIGURE 3

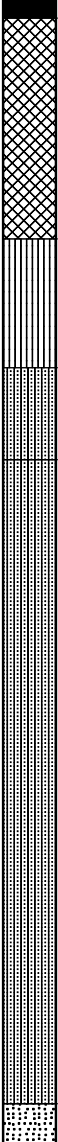
MERRILLS CORNER BRIDGE NO. 0714
CROSSING SHEPARD'S RIVER
BROWNFIELD

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
Federal No. 2753000
WIN. 27530.00

PROJ. MANAGER	JULIE BRASK	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
DESIGNED-DETAILED	EDB	EDB				
CHECKED-REVIEWED	EDB	EDB				
DESIGNED-DETAILED	T. JONES	T. WHITE	JAN 2025			
REVISIONS 1						
REVISIONS 2						
REVISIONS 3						
REVISIONS 4						
FIELD CHANGES						

APPENDIX A
Boring Logs and
Bedrock Core Photographs

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.	<u>Descriptive Term</u>		<u>Portion of Total (%)</u>																																									
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace		0 - 10																																									
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	little		11 - 20																																									
			GC	Clayey gravels, gravel-sand-clay mixtures.	some		21 - 35																																									
	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	adjective (e.g. Sandy, Clayey)		36 - 50																																									
		(little or no fines)	SP	Poorly-graded sands, Gravelly sand, little or no fines.																																												
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.	TERMS DESCRIBING DENSITY/CONSISTENCY <u>Coarse-grained soils</u> (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value). <table><tr><th><u>Density of Cohesionless Soils</u></th><th><u>Standard Penetration Resistance</u> N₆₀-Value (blows per foot)</th></tr><tr><td>Very loose</td><td>0 - 4</td></tr><tr><td>Loose</td><td>5 - 10</td></tr><tr><td>Medium Dense</td><td>11 - 30</td></tr><tr><td>Dense</td><td>31 - 50</td></tr><tr><td>Very Dense</td><td>> 50</td></tr></table> <u>Fine-grained soils</u> (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated. <table><tr><th><u>Consistency of Cohesive soils</u></th><th><u>SPT N₆₀-Value (blows per foot)</u></th><th><u>Approximate Undrained Shear Strength (psf)</u></th><th><u>Field Guidelines</u></th></tr><tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily penetrates</td></tr><tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr><tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr><tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr><tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr><tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr></table>				<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance</u> N ₆₀ -Value (blows per foot)	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50	<u>Consistency of Cohesive soils</u>	<u>SPT N₆₀-Value (blows per foot)</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>	Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates	Soft	2 - 4	250 - 500	Thumb easily penetrates	Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort	Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort	Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail	Hard	>30	over 4000	Indented by thumbnail with difficulty
			<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance</u> N ₆₀ -Value (blows per foot)																																												
			Very loose	0 - 4																																												
	Loose	5 - 10																																														
	Medium Dense	11 - 30																																														
	Dense	31 - 50																																														
Very Dense	> 50																																															
<u>Consistency of Cohesive soils</u>	<u>SPT N₆₀-Value (blows per foot)</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>																																													
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Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort																																													
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail																																													
Hard	>30	over 4000	Indented by thumbnail with difficulty																																													
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.	Rock Quality Designation (RQD): RQD (%) = <u>sum of the lengths of intact pieces of core* > 4 inches</u> length of core advance *Minimum NQ rock core (1.88 in. OD of core) Rock Quality Based on RQD <table><tr><th><u>Rock Quality</u></th><th><u>RQD (%)</u></th></tr><tr><td>Very Poor</td><td>≤25</td></tr><tr><td>Poor</td><td>26 - 50</td></tr><tr><td>Fair</td><td>51 - 75</td></tr><tr><td>Good</td><td>76 - 90</td></tr><tr><td>Excellent</td><td>91 - 100</td></tr></table>				<u>Rock Quality</u>	<u>RQD (%)</u>	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100																													
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Desired Soil Observations (in this order, if applicable): Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level					Desired Rock Observations (in this order, if applicable): Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock quality (very poor, poor, etc.) ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12 Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))																																											
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information					Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth																																											

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Merrills Corner Bridge No. 0714 over Shepards River Location: Old County Road, Brownfield, Maine		Boring No.: BB-BSR-101 WIN: 027530.00					
Driller: MaineDOT		Elevation (ft.): 473.9		Auger ID/OD: 5" Solid Stem							
Operator: T. Garrett		Datum: NAVD 88		Sampler: 24" Standard Split Spoon							
Logged By: H. Hollauer		Rig Type: CME 45C Truck		Hammer Wt./Fall: SS-140#/30";NW-300#/30"							
Date Start/Finish: 3/20/2024-3/21/2024		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2.0 in. ID							
Boring Location: Sta. 2+12.2, 1.7 ft Lt.		Casing ID/OD: NW-3.0 in. ID		Water Level*: Not Measured							
Hammer Efficiency Factor: 0.962		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (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				
0	1D	24/18	0.5 - 2.5	10/6/4/2	10	16	SSA	473.5		Bituminous Concrete	G#241493 A-2-4, SM WC=14.5%
								470.0		Brown, dry, medium dense, fine SAND, some medium sand, little silt, trace coarse sand and fine gravel (Fill). PID Reading = 0.2 ppm	
								468.7		Dark brown, dry, medium dense, fine SAND, some silt and coarse gravel, little medium sand, trace fine gravel and coarse sand (Fill).	
5	2D	24/16	4.0 - 6.0	5/9/6/5	15	24		468.7		Dark brown, moist, very stiff, ORGANIC SILT, trace fine sand, trace roots, (Alluvial Deposit). PID Reading = 0.1 ppm	
								465.9		Olive-grey, wet, medium stiff, Sandy SILT, trace fine sand, trace roots, (Alluvial Deposit). PID Reading = 0.2 ppm	
	3D	24/22	6.0 - 8.0	1/2/2/2	4	6	18	465.9		Olive, wet, dense, fine SAND, some medium sand, little silt (Alluvial Deposit). PID Reading = 0.2 ppm	
								463.9		Olive-grey, wet, medium dense, Silty fine SAND, some medium sand, trace coarse sand and fine gravel, loosely bonded (Glacial Till).	
	4D	24/12	8.0 - 10.0	6/14/14/13	28	45	36	463.9		Olive-grey, wet, very dense, Silty fine to medium SAND, trace coarse sand and silty clay, well bonded (Glacial Till).	
								449.9		Grey, wet, very dense, fine to coarse SAND, little silt and gravel, trace lean clay, well bonded (Glacial Till). Note: Washed ahead of casing from 14 to 51.1 ft. Open borehole from 14 to 24 ft while sampling.	
10	5D	24/18	10.0 - 12.0	9/11/7/10	18	29	20	449.9		Grey, wet, very dense, fine to medium SAND, some silt, little fine gravel, trace coarse sand, well bonded (Glacial Till).	
	6D	24/16	12.0 - 14.0	16/33/45/42	78	125	45				
	7D	24/14	14.0 - 16.0	29/45/43/57	88	141	OPEN				
20	8D	24/16	19.0 - 21.0	11/26/32/19	58	93					
25	9D	24/12	24.0 - 26.0	15/39/29/28	68	109					
Remarks: 1. Photoionization detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm). 2. River water level approximately 8.4 ft below bridge deck level. 3. OC = Organic Content											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 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-BSR-101	

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Merrills Corner Bridge No. 0714 over Shepards River Location: Old County Road, Brownfield, Maine				Boring No.: BB-BSR-101A WIN: 027530.00			
Driller: MaineDOT				Elevation (ft.): 473.9				Auger ID/OD: 5" Solid Stem			
Operator: T. Garrett				Datum: NAVD 88				Sampler: Standard Split Spoon			
Logged By: H. Hollauer/T. Ramey				Rig Type: CME 45C Truck				Hammer Wt./Fall: SS-140#/30";NW-300#/30"			
Date Start/Finish: 3/21/2024-3/22/2024				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2.0 in. ID			
Boring Location: Sta. 2+10.3, 1.7 ft Lt.				Casing ID/OD: NW-3.0 in.ID/HW-4.0 in.ID				Water Level*: Not Measured			
Hammer Efficiency Factor: 0.962				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
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0							HW		Notes: Refer to test boring BB-BSR-101 for subsurface conditions encountered from 0 to 49 ft. HW casing advanced from 0 to 35 ft with no sampling. NW casing advanced from 35 to 49 ft with no sampling.		
5											
10											
15											
20											
25											
Remarks: 1. River water level approximately 8.4 ft below bridge deck level.											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.									Page 1 of 3		
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							65				
							65				
							44				
	2D	24/20	54.0 - 56.0	24/20/18/41	38	61	26	414.9		Grey-brown, wet, very dense, fine to coarse SAND, trace silt (Glaciofluvial Deposit).	
55							37				
							36				
							38				
							67	413.1		Grey, wet, very dense, fine to coarse SAND, some silt, trace fine gravel, loosely bonded (Glacial Till).	
60	3D	9.6/3	59.0 - 59.8	50/50(3")			53				
							96				
	R1	60/60	61.0 - 66.0	RQD = 93%			NQ				
								402.9		Top of Bedrock at Elev. 413.1 ft. Note: Roller bit through bedrock from 60.8 to 61.0 ft. White to grey, fine-to coarse-grained biotite GRANODIORITE, very hard, fresh, joints horizontal to low angle, very close to moderately close, tight to open, rough, planar, fresh. Rock Mass Quality = Excellent Recovery = 100% R1: Core Times (min:sec) 61-62 ft (1:48) 62-63 ft (2:15) 63-64 ft (3:02) 64-65 ft (3:15) 65-66 ft (3:20) Similar to R1 Rock Mass Quality = Good Recovery = 92% R2 Core Times (min:sec) 66-67 ft (2:35) 67-68 ft (2:38) 68-69 ft (2:45) 69-70 ft (2:37) 70-71 ft (3:28)	
65	R2	60/55	66.0 - 71.0	RQD = 87%							
70								71.0		Bottom of Exploration at 71.0 feet below ground surface.	
75											
Remarks: 1. River water level approximately 8.4 ft below bridge deck level.											
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-BSR-101A	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Merrills Corner Bridge No. 0714 over Shepards River Location: Old County Road, Brownfield, Maine				Boring No.: BB-BSR-102 WIN: 027530.00			
Driller: Seaboard Drilling, LLC				Elevation (ft.): 473.0				Auger ID/OD: N/A			
Operator: W. Williams				Datum: NAVD 88				Sampler: Standard Split Spoon			
Logged By: T. Ramey/T. Jones				Rig Type: Acker AD2 Truck				Hammer Wt./Fall: SS-140#/30";NW-300#/30"			
Date Start/Finish: 3/20/2024-3/22/2024				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2.0 in. ID			
Boring Location: Sta. 2+71.7, 3.5 ft Rt.				Casing ID/OD: NW-3.0 in.ID/HW-4.0 in.ID				Water Level*: Not Measured			
Hammer Efficiency Factor: 0.966				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected			
T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Sample Information											
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/11	0.0 - 2.0	20/22/20/22	42	68	10	472.6		Bituminous Concrete	G#270317 A-1-b, SW-SM WC=2.8%
							55			Tan, moist, very dense, fine to medium SAND, little fine gravel, trace silt, coarse sand, and coarse gravel (Fill). PID Reading = 0.0 ppm	
	2D	24/12	2.0 - 4.0	18/22/29/11	51	82	93			Tan, moist, very dense, fine to medium SAND, little fine gravel, trace silt, coarse sand, and coarse gravel (Fill). PID Reading = 0.0 ppm	
							115				
							33				
5							11				
	MD	24/0	6.0 - 8.0	1/1/1/1	2	3	18				
							21				
	MD	24/0	8.0 - 10.0	27/13/11/13	24	39	281			Note: Recovered 6" of wood/tree root in 8 to 10 ft sample; drillers switched from HW to NW casing.	
							80				
10	3D	24/8	10.0 - 12.0	27/21/23/27	44	71	115	463.0	Grey-brown, wet, very dense, fine to medium SAND, some fine gravel, little coarse sand, trace silt and coarse gravel (Glaciofluvial Deposit). PID Reading = 0.0 ppm	G#270318 A-1-b, SW-SM WC=9.3%	
							339		Grey-brown, wet, medium dense, fine to medium SAND, some fine gravel, little coarse sand, trace silt and coarse gravel (Glaciofluvial Deposit).		
	4D	24/6	12.0 - 14.0	7/6/6/8	12	19	18				
							43				
	MD	21.6/0	14.0 - 15.8	9/11/50/50(3")	61	98	58	458.0			
15							26		Note: Washed ahead of casing from 14 to 42 ft.	G#270319 A-4, SM WC=10.8%	
	5D	24/24	16.0 - 18.0	2/17/25/26	42	68	47		Olive-grey, wet, very dense, Silty fine to medium SAND, trace coarse sand, trace fine and coarse gravel, well bonded (Glacial Till).		
							37				
							33				
							36				
20	6D	24/24	20.0 - 22.0	19/25/29/30	54	87	32		Olive-grey, wet, very dense, Silty fine to medium SAND, trace coarse sand, trace lean clay, trace fine gravel, well bonded.		
							38				
							32				
25							43				
Remarks: 1. Hammer serial number 0877506 2. Photoionization detector (PID) model Honeywell MiniRAE 3000 readings in parts per million (ppm). 3. River water level approximately 8.4 ft below bridge deck level.											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 4	
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Merrills Corner Bridge No. 0714 over Shepards River Location: Old County Road, Brownfield, Maine				Boring No.: BB-BSR-102 WIN: 027530.00																																																																																																																																																																																																																																				
Driller: Seaboard Drilling, LLC				Elevation (ft.): 473.0				Auger ID/OD: N/A																																																																																																																																																																																																																																				
Operator: W. Williams				Datum: NAVD 88				Sampler: Standard Split Spoon																																																																																																																																																																																																																																				
Logged By: T. Ramey/T. Jones				Rig Type: Acker AD2 Truck				Hammer Wt./Fall: SS-140#/30";NW-300#/30"																																																																																																																																																																																																																																				
Date Start/Finish: 3/20/2024-3/22/2024				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2.0 in. ID																																																																																																																																																																																																																																				
Boring Location: Sta. 2+71.7, 3.5 ft Rt.				Casing ID/OD: NW-3.0 in.ID/HW-4.0 in.ID				Water Level*: Not Measured																																																																																																																																																																																																																																				
Hammer Efficiency Factor: 0.966				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																								
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**BEDROCK CORE PHOTOGRAPHS
MERRILLS CORNER BRIDGE NO. 0714 OVER SHEPARDS RIVER
MAINEDOT WIN 27530.00
OLD COUNTY ROAD, BROWNFIELD, MAINE**



Top Row: BB-BSR-101A, Run No. R1: 61.0 ft (left) to 66.0 ft (right)
Middle Row: BB-BSR-101A, Run No. R2: 66.0 ft (left) to 71.0 ft (right)
Bottom Row: EMPTY

**BEDROCK CORE PHOTOGRAPHS
MERRILLS CORNER BRIDGE NO. 0714 OVER SHEPARDS RIVER
MAINEDOT WIN 27530.00
OLD COUNTY ROAD, BROWNFIELD, MAINE**



Top Row: BB-BSR-102, Run No. R1: 58.0 ft (left) to 60.5 ft (middle-left); BB-BSR-102, Run No. R2: 60.5 ft (middle-left) to 63.6 ft (right)
Middle Row: BB-BSR-102, Run No. R2: 63.6 ft (left) to 65.5 ft (middle); BB-BSR-102, Run No. R3: 65.5 ft (middle) to 67.9 ft (right)
Bottom Row: BB-BSR-102, Run No. R3: 67.9 ft (left) to 70.5 ft (middle)

APPENDIX B

Geotechnical Laboratory Test Results

Work Number: 27530.00

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



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **241493** Boring No./Sample No. **BB-BSR-101/1D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **0.5-2.5**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	100.0
⅜ in. [9.5 mm]	98.7
¼ in. [6.3 mm]	97.6
No. 4 [4.75 mm]	96.8
No. 10 [2.00 mm]	92.3
No. 20 [0.850 mm]	81.1
No. 40 [0.425 mm]	62.3
No. 60 [0.250 mm]	44.0
No. 100 [0.150 mm]	29.9
No. 200 [0.075 mm]	17.4

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	14.5

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **6/5/2024**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **241494** Boring No./Sample No. **BB-BSR-101/2D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **4.0-6.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	77.4
¾ in. [19.0 mm]	77.4
½ in. [12.5 mm]	73.0
⅜ in. [9.5 mm]	70.7
¼ in. [6.3 mm]	68.9
No. 4 [4.75 mm]	68.1
No. 10 [2.00 mm]	64.6
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	49.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	24.5
[0.0338 mm]	17.6
[0.0218 mm]	14.9
[0.0128 mm]	12.2
[0.0094 mm]	8.1
[0.0067 mm]	6.8
[0.0034 mm]	4.1
[0.0014 mm]	2.7

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.55
Loss on Ignition, % (T 267)	7.4
Water Content (T 265), %	21.0

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **6/7/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **241495** Boring No./Sample No. **BB-BSR-101/3D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **6.0-8.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	61.9
[0.0305 mm]	41.8
[0.0207 mm]	29.8
[0.0126 mm]	18.3
[0.0090 mm]	16.0
[0.0066 mm]	9.2
[0.0033 mm]	6.9
[0.0014 mm]	4.6

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.64
Loss on Ignition, % (T 267)	2.9
Water Content (T 265), %	44.2

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **6/7/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.

Boring No./Sample No.

Sample Description

Sampled

Received

270312

BB-BSR-101/4D

GEOTECHNICAL (DISTURBED)

3/20/2024

5/28/2024

Sample Type: GEOTECHNICAL

Location:

Station: TBD

Offset, ft:

Dbfg, ft: 8.0-10.0

WIN/Town 027530.00 - BROWNFIELD

Sampler: BRUCE WILDER

TEST RESULTS

Sieve Analysis (T 27, T 11)	
Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	100.0
No. 4 [4.75 mm]	99.9
No. 10 [2.00 mm]	99.8
No. 20 [0.850 mm]	93.6
No. 40 [0.425 mm]	67.5
No. 60 [0.250 mm]	43.8
No. 100 [0.150 mm]	28.7
No. 200 [0.075 mm]	14.7

Miscellaneous Tests	
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	14.6

Consolidation (T 216)					
Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)						
Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270313** Boring No./Sample No. **BB-BSR-101/5D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **10.0-12.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	98.8
⅜ in. [9.5 mm]	98.8
¼ in. [6.3 mm]	97.7
No. 4 [4.75 mm]	97.2
No. 10 [2.00 mm]	94.2
No. 20 [0.850 mm]	86.2
No. 40 [0.425 mm]	73.3
No. 60 [0.250 mm]	62.3
No. 100 [0.150 mm]	51.6
No. 200 [0.075 mm]	35.4

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	14.9

Consolidation (T 216)

Trimming, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **6/5/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270314** Boring No./Sample No. **BB-BSR-101/8D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **19.0-21.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	91.4
⅜ in. [9.5 mm]	88.4
¼ in. [6.3 mm]	84.3
No. 4 [4.75 mm]	81.9
No. 10 [2.00 mm]	74.5
No. 20 [0.850 mm]	62.7
No. 40 [0.425 mm]	52.5
No. 60 [0.250 mm]	45.9
No. 100 [0.150 mm]	40.6
No. 200 [0.075 mm]	32.6

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	10.2

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **5/31/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270315** Boring No./Sample No. **BB-BSR-101/9D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **24.0-26.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	99.9
No. 20 [0.850 mm]	99.4
No. 40 [0.425 mm]	86.5
No. 60 [0.250 mm]	55.2
No. 100 [0.150 mm]	32.5
No. 200 [0.075 mm]	18.2

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	14.4

Consolidation (T 216)

Trimming, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **6/5/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270316** Boring No./Sample No. **BB-BSR-101/10D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **29.0-31.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	100.0
⅜ in. [9.5 mm]	97.7
¼ in. [6.3 mm]	96.7
No. 4 [4.75 mm]	96.0
No. 10 [2.00 mm]	91.6
No. 20 [0.850 mm]	81.9
No. 40 [0.425 mm]	68.3
No. 60 [0.250 mm]	57.6
No. 100 [0.150 mm]	49.3
No. 200 [0.075 mm]	35.5

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	6.4

Consolidation (T 216)

Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **6/7/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.

Boring No./Sample No.

Sample Description

Sampled

Received

241496

BB-BSR-101/14D

GEOTECHNICAL (DISTURBED)

3/20/2024

5/31/2024

Sample Type: **GEOTECHNICAL** Location:

Station: **TBD**

Offset, ft:

Dbfg, ft: **49.0-50.8**

WIN/Town 027530.00 - BROWNFIELD

Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	100.0
⅜ in. [9.5 mm]	99.2
¼ in. [6.3 mm]	98.3
No. 4 [4.75 mm]	97.7
No. 10 [2.00 mm]	95.7
No. 20 [0.850 mm]	93.1
No. 40 [0.425 mm]	85.9
No. 60 [0.250 mm]	78.3
No. 100 [0.150 mm]	71.3
No. 200 [0.075 mm]	59.6

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	23.8

Consolidation (T 216)

Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

[illegible]

Comments:

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Reported by: **GREGORY LIDSTONE**

Date Reported: **6/7/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.

Boring No./Sample No.

Sample Description

Sampled

Received

270317

BB-BSR-102/1D&2D

GEOTECHNICAL (DISTURBED)

3/20/2024

5/28/2024

Sample Type: GEOTECHNICAL

Location:

Station: TBD

Offset, ft:

Dbfg, ft: 0.0-4.0

WIN/Town 027530.00 - BROWNFIELD

Sampler: BRUCE WILDER

TEST RESULTS

Sieve Analysis (T 27, T 11)	
Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	94.1
½ in. [12.5 mm]	90.1
⅜ in. [9.5 mm]	87.1
¼ in. [6.3 mm]	81.9
No. 4 [4.75 mm]	79.0
No. 10 [2.00 mm]	70.3
No. 20 [0.850 mm]	53.6
No. 40 [0.425 mm]	35.0
No. 60 [0.250 mm]	23.5
No. 100 [0.150 mm]	16.1
No. 200 [0.075 mm]	9.2

Miscellaneous Tests	
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	2.8

Consolidation (T 216)					
Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)						
Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270318** Boring No./Sample No. **BB-BSR-102/3D&4D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **10.0-14.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	91.7
½ in. [12.5 mm]	86.5
⅜ in. [9.5 mm]	76.2
¼ in. [6.3 mm]	69.4
No. 4 [4.75 mm]	65.1
No. 10 [2.00 mm]	53.3
No. 20 [0.850 mm]	38.6
No. 40 [0.425 mm]	26.2
No. 60 [0.250 mm]	18.4
No. 100 [0.150 mm]	13.1
No. 200 [0.075 mm]	8.0

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	9.3

Consolidation (T 216)

Trimming, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **6/5/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270319** Boring No./Sample No. **BB-BSR-102/5D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **16.0-18.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	95.4
½ in. [12.5 mm]	95.4
⅜ in. [9.5 mm]	94.7
¼ in. [6.3 mm]	93.3
No. 4 [4.75 mm]	91.7
No. 10 [2.00 mm]	88.1
No. 20 [0.850 mm]	78.0
No. 40 [0.425 mm]	68.4
No. 60 [0.250 mm]	61.2
No. 100 [0.150 mm]	53.6
No. 200 [0.075 mm]	42.1

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	10.8

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **6/6/2024**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270320** Boring No./Sample No. **BB-BSR-102/7D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **25.0-27.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	98.4
⅜ in. [9.5 mm]	94.9
¼ in. [6.3 mm]	94.1
No. 4 [4.75 mm]	93.1
No. 10 [2.00 mm]	88.1
No. 20 [0.850 mm]	77.3
No. 40 [0.425 mm]	65.8
No. 60 [0.250 mm]	57.0
No. 100 [0.150 mm]	50.0
No. 200 [0.075 mm]	37.2

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	10.2

Consolidation (T 216)

Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **5/31/2024**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **270321** Boring No./Sample No. **BB-BSR-102/9D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **3/20/2024** Received **5/28/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **TBD** Offset, ft: Dbfg, ft: **35.0-37.0**

WIN/Town **027530.00 - BROWNFIELD** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	96.6
⅜ in. [9.5 mm]	95.8
¼ in. [6.3 mm]	95.6
No. 4 [4.75 mm]	95.3
No. 10 [2.00 mm]	92.5
No. 20 [0.850 mm]	86.2
No. 40 [0.425 mm]	79.8
No. 60 [0.250 mm]	73.7
No. 100 [0.150 mm]	67.3
No. 200 [0.075 mm]	55.2

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	11.4

Consolidation (T 216)

Trimming, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

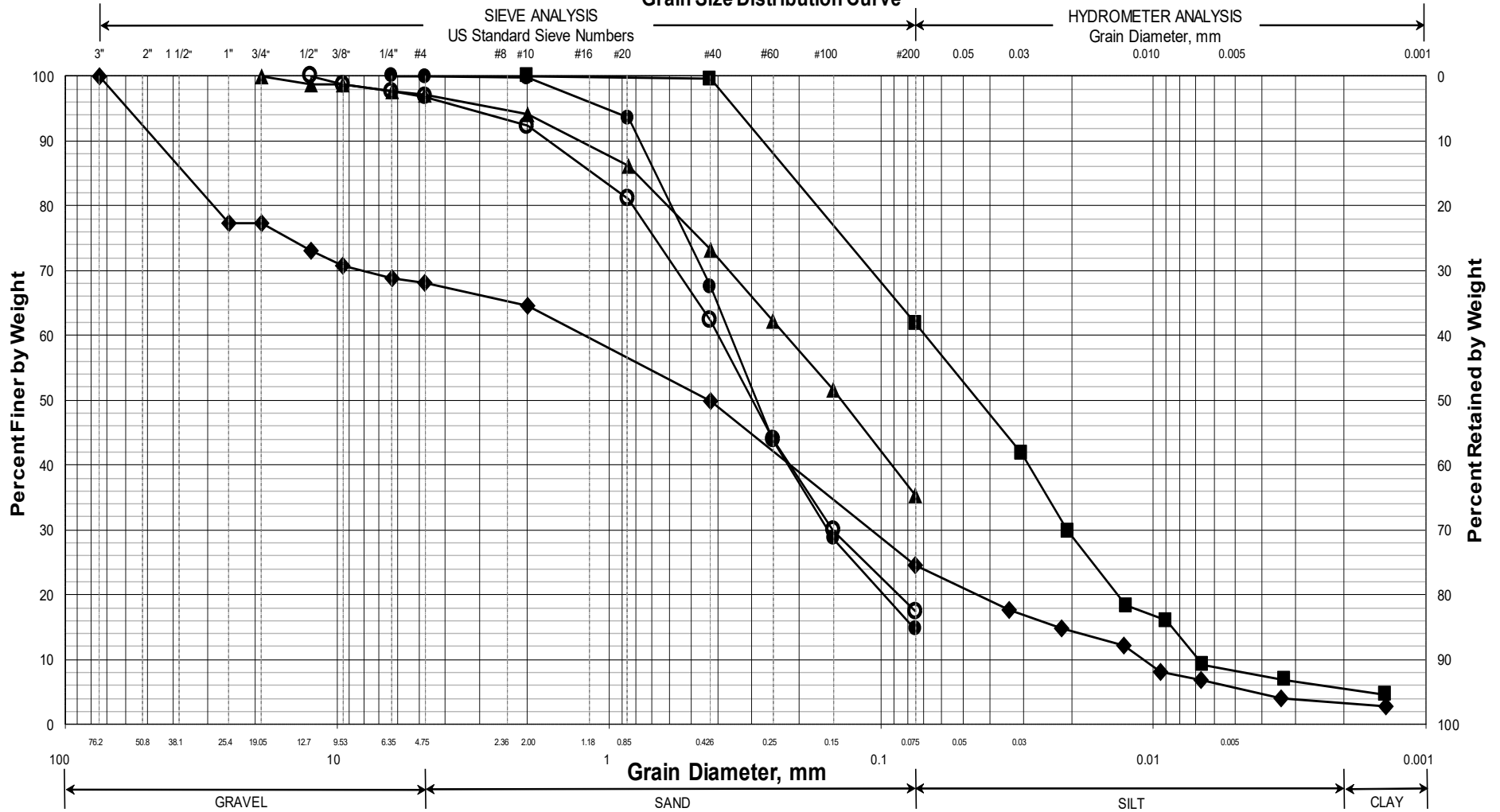
Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **5/31/2024**

Paper Copy: Lab File; Project File; Geotech File

Maine Department of Transportation Grain Size Distribution Curve

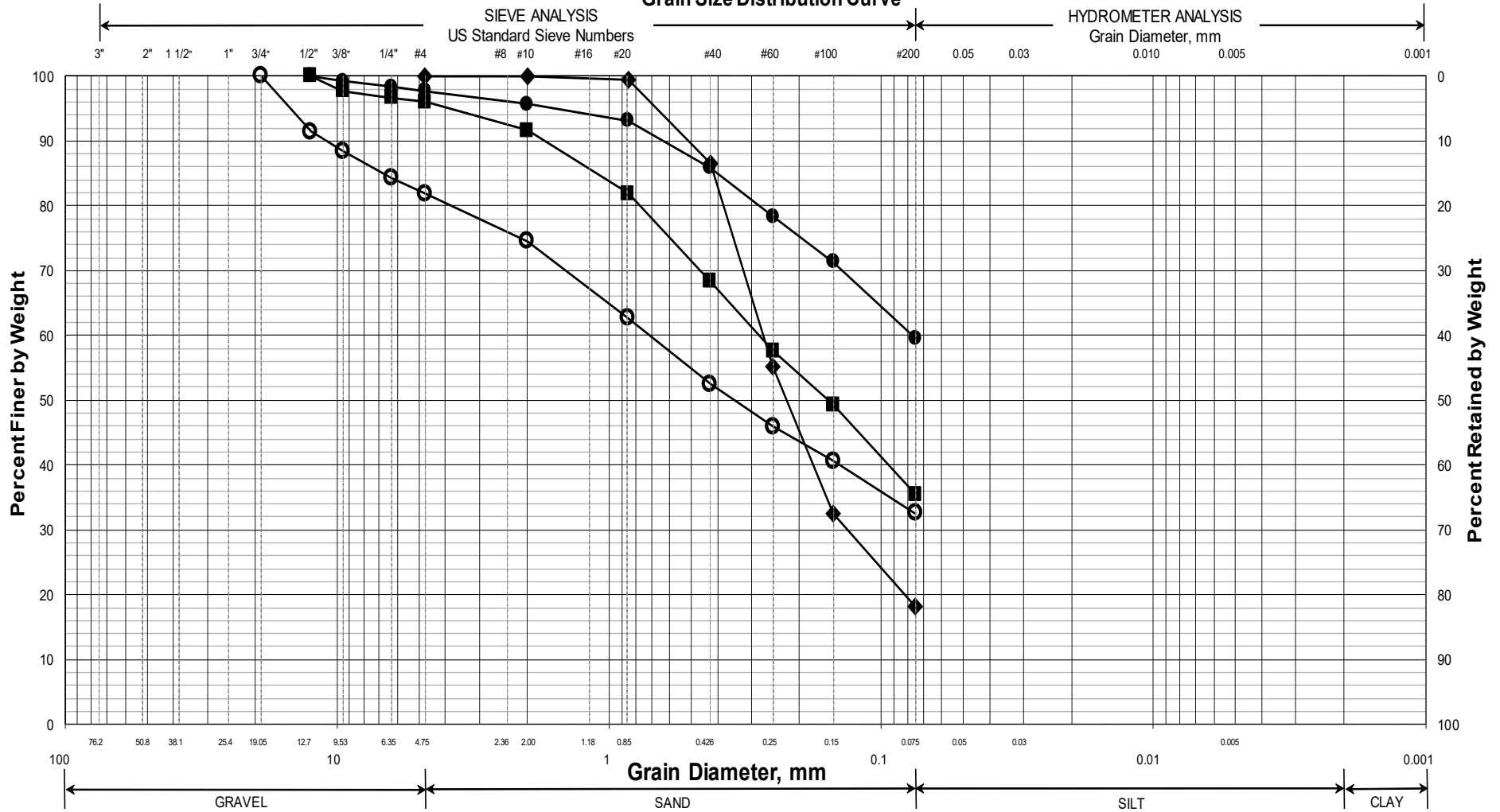


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
O	BB-BSR-101/1D	TBD		0.5-2.5	SAND, little silt, trace gravel.	14.5			
◆	BB-BSR-101/2D	TBD		4.0-6.0	SAND, some gravel, some silt, trace clay.	21			NP
■	BB-BSR-101/3D	TBD		6.0-8.0	Sandy SILT, trace clay.	44.2			NP
●	BB-BSR-101/4D	TBD		8.0-10.0	SAND, little silt, trace gravel.	14.6			
▲	BB-BSR-101/5D	TBD		10.0-12.0	SAND, some silt, trace gravel.	14.9			
X									

WIN
027530.00
Town
Brownfield
Reported by/Date
WHITE, TERRY A 6/10/2024

Maine Department of Transportation Grain Size Distribution Curve

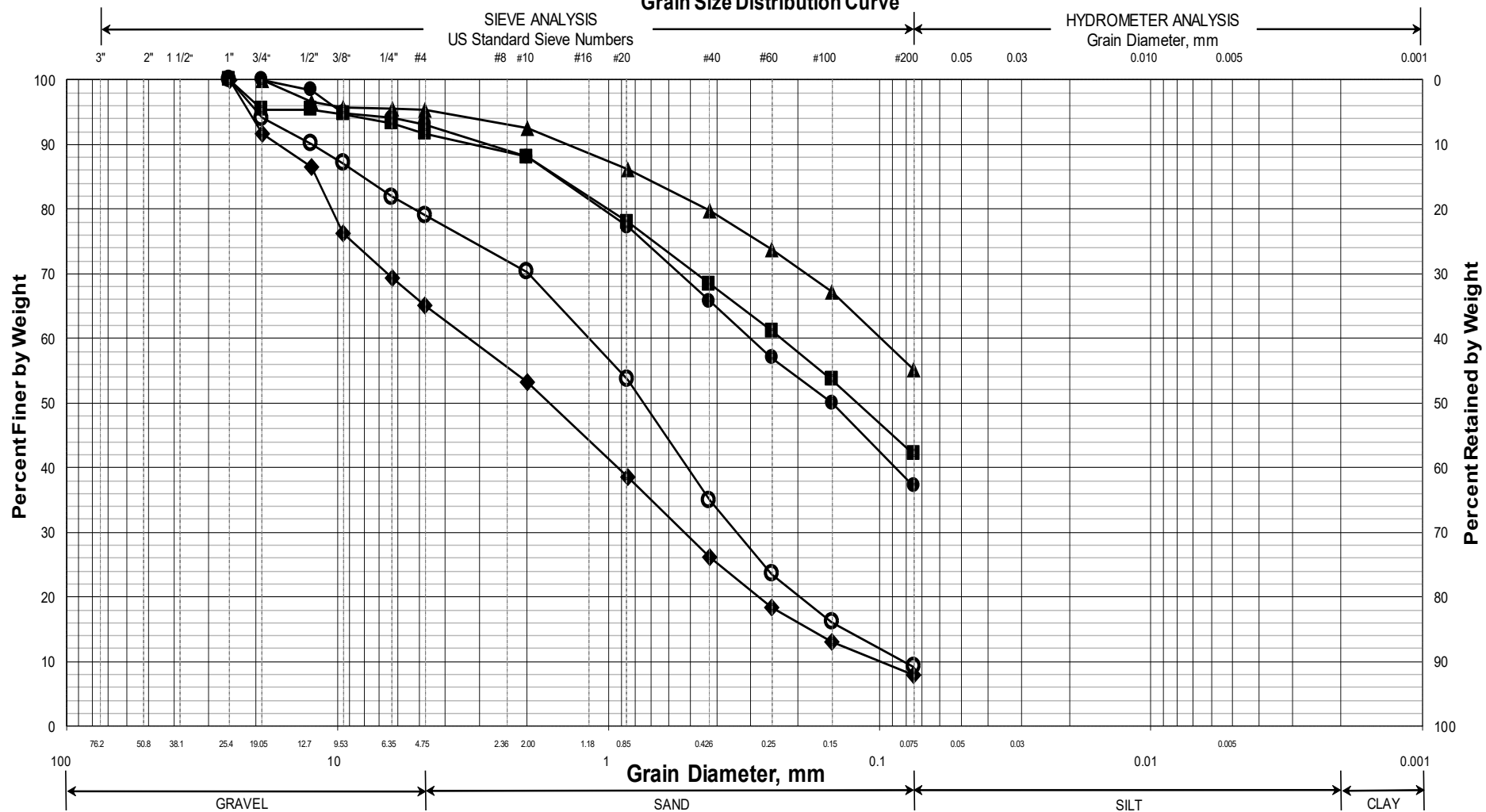


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-BSR-101/8D	TBD		19.0-21.0	SAND, some silt, little gravel.	10.2			
◆	BB-BSR-101/9D	TBD		24.0-26.0	SAND, little silt.	14.4			
■	BB-BSR-101/10D	TBD		29.0-31.0	SAND, some silt, trace gravel.	6.4			
●	BB-BSR-101/14D	TBD		49.0-50.8	Sandy SILT, trace gravel.	23.8			
▲									
X									

WIN
027530.00
Town
Brownfield
Reported by/Date
WHITE, TERRY A 6/10/2024

Grain Size Distribution Curve



UNIFIED CLASSIFICATION

[illegible]

WIN
027530.00
Town
Brownfield
Reported by/Date
WHITE, TERRY A 6/10/2024

APPENDIX C

Geotechnical Calculations

Design Soil Properties

		CALCULATIONS		File No.: 0210010-000
				Sheet: 1 of 5
Client:	Maine Department of Transportation			Date: 16-Sep-2024
Project:	Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00			Computed by: TPJ
Subject:	Design Soil Parameters			Checked by: EMH

PROBLEM STATEMENT & OBJECTIVE
Develop design soil properties.

EXECUTIVE SUMMARY
Design soil properties are presented below for foundation design.

REFERENCES
 1. Performance and Use of the Standard Pen. Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998)
 2. Soil property correlations are provided on following sheets.

AVAILABLE INFORMATION
 1. Boring logs BB-BSR-101,-101A, and -102.

ASSUMPTIONS
 1. Elevations are based on the NAVD 88 datum.
 2. Unit weights were developed based on design experience and typical densities of deposits encountered.

DESIGN SOIL PROPERTIES

Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
Existing Fill	125	28	0
Alluvial Deposit (Cohesive)	120	0	400
Alluvial Deposit (Cohesionless)	125	34	0
Glacial Deposits (Cohesive)	130	0	4,000
Glacial Deposits (Cohesionless)	130	40	0

Boring ID	Depth (ft)	Elevation (ft)	Sample Number	Geological Description	Cohesive or Cohesionless or Rock	SPT N (bl/ft)	f ₁ (deg.)	f ₂ (deg.)	f ₃ (deg.)	f ₄ (deg.)	f ₅ (deg.)	f ₆ (deg.)	f ₇ (deg.)	f ₈ (deg.)	f _{LOW} (deg.)	f _{HIGH} (deg.)	f _{AVE} (deg.)	f _{SD} (deg.)	f _{AVE-1SD} (deg.)	f _{AVE+1SD} (deg.)	SU ₁ (psf)	SU ₂ (psf)	SU _{AVE} (psf)
BB-BSR-101	1.5	472.4	1D	Fill	Cohesionless	10	37.0	32.0	27.0	30.5	32.1	28.4	40.2	30.5	27.0	40.2	32.2	4.4	27.9	36.6	--	--	--
BB-BSR-101	5.0	468.9	2D	Fill	Cohesionless	15	39.7	34.7	29.7	34.0	34.9	31.4	44.8	32.5	29.7	44.8	35.2	4.9	30.4	40.1	--	--	--
BB-BSR-101	7.0	466.9	3D	Alluvial Deposit	Cohesive	4	--	--	--	--	--	--	--	--	--	--	--	--	--	--	609	730	670
BB-BSR-101	9.0	464.9	4D	Alluvial Deposit	Cohesionless	28	45.1	40.1	35.1	40.9	40.3	37.5	50.6	37.0	35.1	50.6	40.8	5.0	35.9	45.8	--	--	--
BB-BSR-101	11.0	462.9	5D	Glacial Till	Cohesionless	18	41.6	36.6	31.6	36.5	36.8	33.6	44.1	34.0	31.6	44.1	36.9	4.2	32.7	41.0	--	--	--
BB-BSR-101	13.0	460.9	6D	Glacial Till	Cohesionless	78	60.7	55.7	50.7	61.1	56.1	45.0	69.6	47.8	45.0	69.6	55.8	8.0	47.8	63.9	--	--	--
BB-BSR-101	15.0	458.9	7D	Glacial Till	Cohesionless	88	62.9	57.9	52.9	64.0	58.3	45.0	70.8	48.0	45.0	70.8	57.5	8.6	48.9	66.1	--	--	--
BB-BSR-101	20.0	453.9	8D	Glacial Till	Cohesionless	58	55.8	50.8	45.8	54.8	51.1	45.0	58.4	45.7	45.0	58.4	50.9	5.1	45.8	56.0	--	--	--
BB-BSR-101	25.0	448.9	9D	Glaciofluvial Deposit	Cohesionless	68	60.3	55.3	50.3	60.5	55.6	45.0	61.6	47.7	45.0	61.6	54.5	6.3	48.3	60.8	--	--	--
BB-BSR-101	30.0	443.9	10D	Glacial Till	Cohesionless	34	49.9	44.9	39.9	47.2	45.2	42.9	48.1	41.3	39.9	49.9	44.9	3.4	41.5	48.4	--	--	--
BB-BSR-101	35.0	438.9	11D	Glacial Till	Cohesionless	50	56.0	51.0	46.0	55.0	51.3	45.0	53.6	45.9	45.0	56.0	50.5	4.4	46.1	54.9	--	--	--
BB-BSR-101	40.0	433.9	12D	Glacial Till	Cohesionless	28	48.2	43.2	38.2	45.0	43.5	40.9	44.3	39.8	38.2	48.2	42.9	3.2	39.7	46.1	--	--	--
BB-BSR-101	45.0	428.9	13D	Glaciolacustrine Deposit	Cohesive	64	--	--	--	--	--	--	--	--	--	--	--	--	--	--	13778	16552	15165
BB-BSR-101	50.0	423.9	14D	Glaciolacustrine Deposit	Cohesive	37	--	--	--	--	--	--	--	--	--	--	--	--	--	--	7949	9549	8749
BB-BSR-101A	50.0	423.9	1D	Glaciolacustrine Deposit	Cohesive	26	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5574	6696	6135
BB-BSR-101A	55.0	418.9	2D	Glaciofluvial Deposit	Cohesionless	38	52.0	47.0	42.0	49.9	47.3	45.0	46.2	43.0	42.0	52.0	46.6	3.3	43.2	49.9	--	--	--
BB-BSR-101A	60.0	413.9	3D	Glacial Till	Cohesionless	50	56.0	51.0	46.0	55.0	51.3	45.0	49.4	45.9	45.0	56.0	50.0	4.2	45.8	54.1	--	--	--

Friction Angle Correlation References:

- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Angular and well-grained soil particles, (12*(N60)^0.5+25, Dunham 1954
- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Round and well-grained or angular and uniform grained soil particles, (12*(N60)^0.5+20, Dunham 1954
- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Round and uniform-grained soil particles, (12*(N60)^0.5+15, Dunham 1954
- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Sandy, (20*(N60)^0.5+15, Ohsaki et al. 1959
- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Granular, 3.5*(N60)^0.5+20, Muromachi et al. 1974
- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Sandy, (15*(N60)^0.5+15<=45 and N60>5, Japan Road Association 1990
- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Sandy, [20*(N1)60]^0.5+20, Hatanaka and Uchida 1996
- Peck, Hansen and Thornburn 1974, -0.0016*(N60)^2 + 0.3726*N60+26.286

Undrained Shear Strength Correlation References

- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Terzaghi and Peck 1967
- Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Tschebotarioff 1973

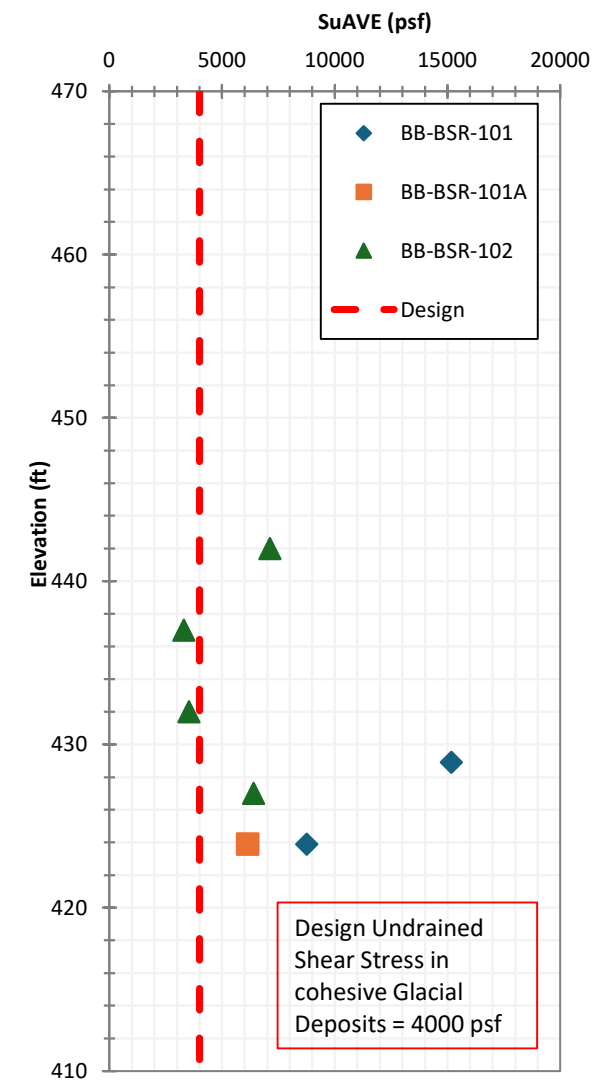
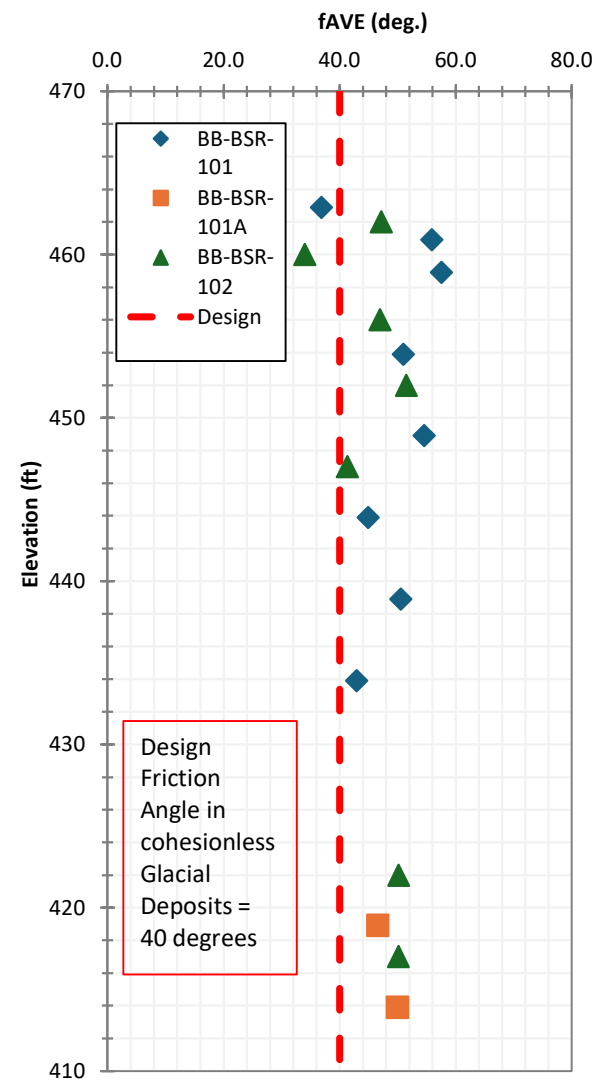
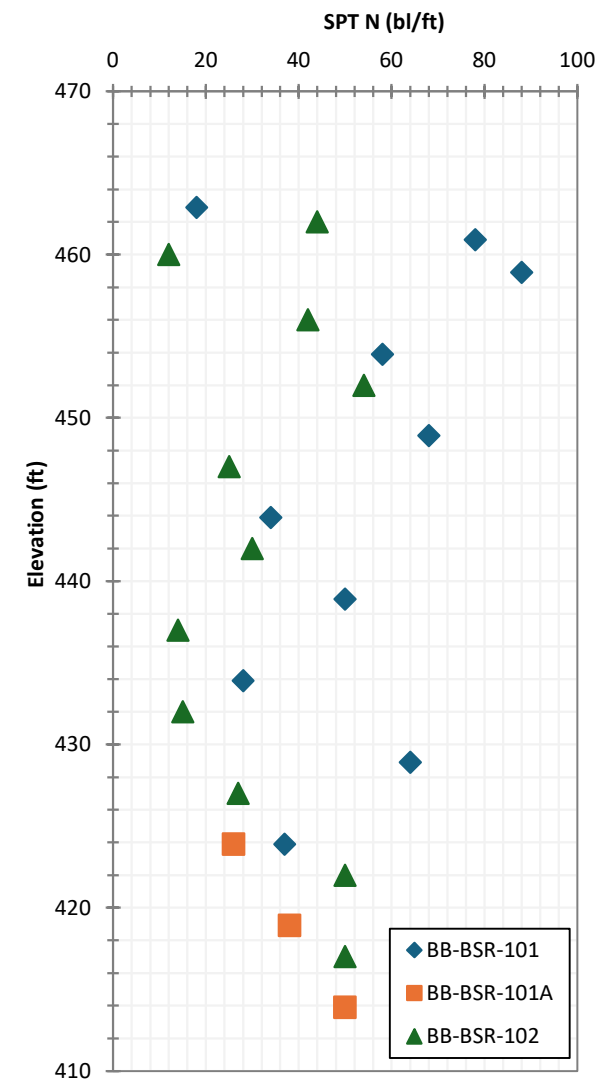


File No.: 0210010-000
Date: 9/16/2024
Project: Merrills Corner Bridge
Subject: BB-BSR-101/101A Props
Client: MaineDOT
Computed by: TPJ
Checked by: EMH
Sheet: 2 of 6

Boring ID	Depth (ft)	Elevation (ft)	Sample Number	Geological Description	Cohesive or Cohesionless or Rock	SPT N (bl/ft)	f ₁ (deg.)	f ₂ (deg.)	f ₃ (deg.)	f ₄ (deg.)	f ₅ (deg.)	f ₆ (deg.)	f ₇ (deg.)	f ₈ (deg.)	f _{LOW} (deg.)	f _{HIGH} (deg.)	f _{AVE} (deg.)	f _{SD} (deg.)	f _{AVE-1SD} (deg.)	f _{AVE+1SD} (deg.)	SU ₁ (psf)	SU ₂ (psf)	SU _{AVE} (psf)
BB-BSR-102	1.0	472.0	1D	Fill	Cohesionless	42	49.7	44.7	39.7	46.8	44.9	42.6	61.5	41.1	39.7	61.5	46.4	6.9	39.5	53.3	--	--	--
BB-BSR-102	3.0	470.0	2D	Fill	Cohesionless	51	52.2	47.2	42.2	50.1	47.5	45.0	65.8	43.2	42.2	65.8	49.1	7.5	41.6	56.6	--	--	--
BB-BSR-102	11.0	462.0	3D	Glaciofluvial Deposit	Cohesionless	44	51.1	46.1	41.1	48.7	46.3	44.2	57.8	42.3	41.1	57.8	47.2	5.4	41.8	52.5	--	--	--
BB-BSR-102	13.0	460.0	4D	Glaciofluvial Deposit	Cohesionless	12	39.0	34.0	29.0	33.1	34.2	30.7	39.5	32.0	29.0	39.5	33.9	3.7	30.2	37.7	--	--	--
BB-BSR-102	17.0	456.0	5D	Glacial Till	Cohesionless	42	51.3	46.3	41.3	48.9	46.5	44.4	54.1	42.4	41.3	54.1	46.9	4.4	42.5	51.3	--	--	--
BB-BSR-102	21.0	452.0	6D	Glacial Till	Cohesionless	54	56.5	51.5	46.5	55.6	51.8	45.0	58.8	46.1	45.0	58.8	51.5	5.2	46.3	56.7	--	--	--
BB-BSR-102	26.0	447.0	7D	Glacial Till	Cohesionless	25	46.4	41.4	36.4	42.7	41.6	38.9	45.0	38.2	36.4	46.4	41.3	3.4	37.9	44.7	--	--	--
BB-BSR-102	31.0	442.0	8D	Glacial Till	Cohesive	30	--	--	--	--	--	--	--	--	--	--	--	--	--	--	6465	7766	7115
BB-BSR-102	36.0	437.0	9D	Glaciolacustrine Deposit	Cohesive	14	--	--	--	--	--	--	--	--	--	--	--	--	--	--	2996	3598	3297
BB-BSR-102	41.0	432.0	10D	Glaciolacustrine Deposit	Cohesive	15	--	--	--	--	--	--	--	--	--	--	--	--	--	--	3213	3859	3536
BB-BSR-102	46.0	427.0	11D	Glaciolacustrine Deposit	Cohesive	27	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5814	6984	6399
BB-BSR-102	51.0	422.0	12D	Glacial Till	Cohesionless	50	56.1	51.1	46.1	55.1	51.4	45.0	50.7	45.9	45.0	56.1	50.2	4.2	46.0	54.4	--	--	--
BB-BSR-102	56.0	417.0	13D	Glacial Till	Cohesionless	50	56.1	51.1	46.1	55.1	51.4	45.0	50.0	45.9	45.0	56.1	50.1	4.2	45.9	54.3	--	--	--
<u>Friction Angle Correlation References:</u> 1. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Angular and well-grained soil particles, (12*(N60)^0.5+25, Dunham 1954 2. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Round and well-grained or angular and uniform grained soil particles, (12*(N60)^0.5+20, Dunham 1954 3. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Round and uniform-grained soil particles, (12*(N60)^0.5+15, Dunham 1954 4. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Sandy, (20*(N60)^0.5+15, Ohsaki et al. 1959 5. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Granular, 3.5*(N60)^0.5+20, Muromachi et al. 1974 6. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Sandy, (15*(N60)^0.5+15<=45 and N60>5, Japan Road Association 1990 7. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Sandy, [20*(N1)60]^0.5+20, Hatanaka and Uchida 1996 8. Peck, Hansen and Thornburn 1974, -0.0016*(N60)^2 + 0.3726*N60+26.286 <u>Undrained Shear Strength Correlation References</u> 1. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Terzaghi and Peck 1967 2. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice (McGregor and Duncan, 1998) - Tschebotarioff 1973																							



File No.: 0210010-000
 Date: 9/16/2024
 Project: Merrills Corner Bridge
 Subject: BB-BSR-102 Props
 Client: MaineDOT
 Computed by: TPJ
 Checked by: EMH
 Sheet: 3 of 6



Client: Maine Department of Transportation
Computed by: TPJ Checked by: EMH
Sheet: 6 of 6

HALEY ALDRICH
File No.: 0210010-000
Date: 9/20/2024
Project: Brownfield - Merrills Corner Bridge No. 0714
Subject: Design Soil Parameters [Glacial Deposits]

Seismic Site Class and Seismic Design Parameters

Client: Maine Department of Transportation

Date: 9/21/2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Seismic Site Class and Seismic Design Parameters

Checked by: EMH

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class for the proposed bridge structure at Merrills Corner Bridge using available subsurface and SPT N information. Determine the seismic design parameters (see Sheet 7).

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the borings conducted near the proposed bridge structure, we recommend a Seismic Site Class D. See seismic design parameters on Sheet 7.

REFERENCES

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

AVAILABLE INFORMATION

1. Boring logs dated 3-20-2024 to 3-22-2024 by MaineDOT and Seaboard Drilling, LLC. (Borings BB-BSR-101/101A, and BB-BSR-102).
2. Elevations reference the NAVD 88 datum.

ASSUMPTIONS

1. 1. Where SPT N, Vs or su data was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N, Vs or su for the extended profile was then assumed based on the available information.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.4.2.1-1), requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. Categorize the site using one of the following three methods (Method A, B, or C).

Method A

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

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

where

V_{si} = shear wave velocity of i th soil (ft/s).

d_i = thickness of i th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

Client: Maine Department of Transportation

Date: 9/21/2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Seismic Site Class and Seismic Design Parameters

Checked by: EMH

PROCEDURE**Method B**

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

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th soil layer not to exceed 100 ft (blows/ft).

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

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

Method C

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

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

Boring logs dated 3-20-2024 to 3-22-2024 by MaineDOT and Seaboard Drilling, LLC. (Borings BB-BSR-101/101A, and BB-BSR-102).

Elevations reference the NAVD 88 datum.

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th cohesionless soil layer (blows/ft).

d_i = thickness of i th cohesionless soil layer (ft).

m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and m .

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

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

Check the site against the three categories of Site Class F (see attached Table 3.4.2.1-1), requiring site-specific ground motion

where

Categorize the site using one of the following three methods (Method A, B, or C).

s_{ui} = undrained shear strength of i th cohesive soil layer (psf), not to exceed 5000 psf

d_i = thickness of i th cohesive soil layer (ft).

k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and k .

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

Client: Maine Department of Transportation

Date: 9/21/2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Seismic Site Class and Seismic Design Parameters

Checked by: EMH

SITE CLASS DEFINITIONS

AASHTO Table 3.10.3.1-1 from LRFD 2020

Table 3.10.3.1-1—Site Class Definitions

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

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

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile
 $\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile
 $\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile
 PI = plasticity index (ASTM D4318)
 w = moisture content (ASTM D2216)

Client:	Maine Department of Transportation
---------	------------------------------------

Date: 9/21/2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Seismic Site Class and Seismic Design Parameters

Checked by: EMH

CALCULATIONS - METHOD B

Boring ID: BB-BSR-101/101A

Ground Surface El.:	473.9
---------------------	-------

Boring ID	Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
101	1D	1.5	472.4	Fill	3.0	10	0.30
101	2D	5.0	468.9	Fill	3.0	15	0.20
101	3D	7.0	466.9	Alluvial	2.0	4	0.50
101	4D	9.0	464.9	Alluvial	2.0	28	0.07
101	5D	11.0	462.9	Glacial	2.0	18	0.11
101	6D	13.0	460.9	Glacial	2.0	78	0.03
101	7D	15.0	458.9	Glacial	5.0	88	0.06
101	8D	20.0	453.9	Glacial	5.0	58	0.09
101	9D	25.0	448.9	Glacial	5.0	68	0.07
101	10D	30.0	443.9	Glacial	5.0	34	0.15
101	11D	35.0	438.9	Glacial	5.0	100	0.05
101	12D	40.0	433.9	Glacial	5.0	28	0.18
101	13D	45.0	428.9	Glacial	5.0	64	0.08
101	14D	50.0	423.9	Glacial	5.0	37	0.14
101A	2D	55.0	418.9	Glacial	5.0	38	0.13
101A	3D	60.0	413.9	Glacial	1.8	100	0.02
101A	R1	60.8	413.1	Bedrock	39.2	100	0.39
				Totals =	100.0		2.555

N-bar (blows/ft) = 39.1

Site Class = D

Client: Maine Department of Transportation

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Subject: Seismic Site Class and Seismic Design Parameters

CALCULATIONS - METHOD B

Boring ID:	BB-BSR-102
Ground Surface El.:	473.0

Boring ID	Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
102	1D	1.0	472.0	Fill	2.0	42	0.05
102	2D	3.0	470.0	Fill	4.0	51	0.08
102	MD	7.0	466.0	No Recovery	2.0	2	1.00
102	MD	9.0	464.0	No Recovery	2.0	24	0.08
102	3D	11.0	462.0	Glacial	2.0	44	0.05
102	4D	13.0	460.0	Glacial	2.0	12	0.17
102	MD	15.0	458.0	No Recovery	2.0	61	0.03
102	5D	17.0	456.0	Glacial	3.0	42	0.07
102	6D	21.0	452.0	Glacial	4.5	54	0.08
102	7D	26.0	447.0	Glacial	5.0	25	0.20
102	8D	31.0	442.0	Glacial	5.0	30	0.17
102	9D	36.0	437.0	Glacial	5.0	14	0.36
102	10D	41.0	432.0	Glacial	5.0	15	0.33
102	11D	46.0	427.0	Glacial	5.0	27	0.19
102	12D	51.0	422.0	Glacial	5.0	100	0.05
102	13D	56.0	417.0	Glacial	3.5	100	0.04
102	R1	57.0	416.0	Bedrock	43.0	100	0.43
				Totals =	100.0		3.366

N-bar (blows/ft) = 29.7

Client: Maine Department of Transportation

Date: 9/21/2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Seismic Site Class and Seismic Design Parameters

Checked by: EMH

RESULTS SUMMARY

Boring ID	Parameter	Average Value	Site Class
BB-BSR-101/101A	N-bar (blows/ft	39.1	D
BB-BSR-102	N-bar (blows/ft	29.7	D

CONCLUSIONS & RECOMMENDATIONS

Based on the subsurface conditions encountered at the borings, we recommend a Seismic Site Class D.

Client: Maine Department of Transportation

Date: 9/21/2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Seismic Site Class and Seismic Design Parameters

Checked by: EMH

Name: Merrills Bridge

Lat.: 43.935315 deg.

Long.: -70.948728 deg.

Code: aashto-2009

Risk Cat.: III (not used for AASHTO and ASCE41 options)

Site Class: D

Link Text: <https://earthquake.usgs.gov/ws/designmaps/aashto-2009.json?latitude=43.935315&longitude=-70.948728&siteClass=D&title=Merrills Bridge>
(copy this link to a browser and hit enter; the browser will retrieve the seismic parameters from USGS)

Please carefully read the USGS documentation that explains the output.

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    "ss": 0.199,  
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    "s1": 0.049,  
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    "sd1": 0.118,  
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  }  
}
```

Design Parameter	Design Value
Site factor for short-period range of acceleration response spectrum, F_a =	1.6
Site factor for long-period range of acceleration response spectrum, F_v =	2.4
Site factor at zero-period on acceleration response spectrum, F_{pga} =	1.595
Horizontal response spectral acceleration coefficient at 0.2-s period on rock, S_s (g) =	0.199
Horizontal response spectral acceleration coefficient at 1.0-s period on rock, S_1 (g) =	0.049
Peak seismic ground acceleration coefficient on rock, PGA (g) =	0.102
Horizontal response spectral acceleration coefficient at 0.2-s period modified by F_a , S_{DS} (g) =	0.318
Horizontal response spectral acceleration coefficient at 1.0-s period modified by F_v , S_{D1} (g) =	0.118
Peak seismic ground acceleration coefficient modified by F_{pga} , A_s (g) =	0.163
Seismic Site Class =	D
Seismic Zone =	1

Global Stability



CALCULATIONS

File No.: 0210010-000

Sheet: 1 of 1

Client: Maine Department of Transportation

Date: 18-Sep-2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Global Stability

Checked by: EMH

PROBLEM STATEMENT AND OBJECTIVE

Calculate the static global stability minimum factor of safety for the proposed bridge structures in the longitudinal direction.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. Slide version 9.0 by RocScience.
3. MaineDOT Bridge Design Guide, 2003 with interim revisions through June 2018.

AVAILABLE INFORMATION

1. Interpretive Subsurface Profile prepared by MaineDOT dated 9/4/24.
2. Boring logs BB-BSR-101, BB-BSR-101A, and BB-BSR-102.

ASSUMPTIONS

1. Water level will be modeled at El. 465 based on measurements of the river level during drilling.
2. Transverse stability will be assessed during final design.
3. A 250 psf traffic surcharge will be modeled along the roadway.
4. Seismic cases will have a seismic force of $A_s/2$ ($0.163g/2$) = 0.08g based on the seismic site class calculations.
5. Circular failure surfaces were assessed using Spencer's and Bishop's methods. Noncircular failure surfaces were assessed using Spencer's method.
6. Glacial Deposits includes Glacial Till, Glaciofluvial Deposits, and Glaciolacustrine Deposits as shown on the boring logs.

SOIL PROPERTIES

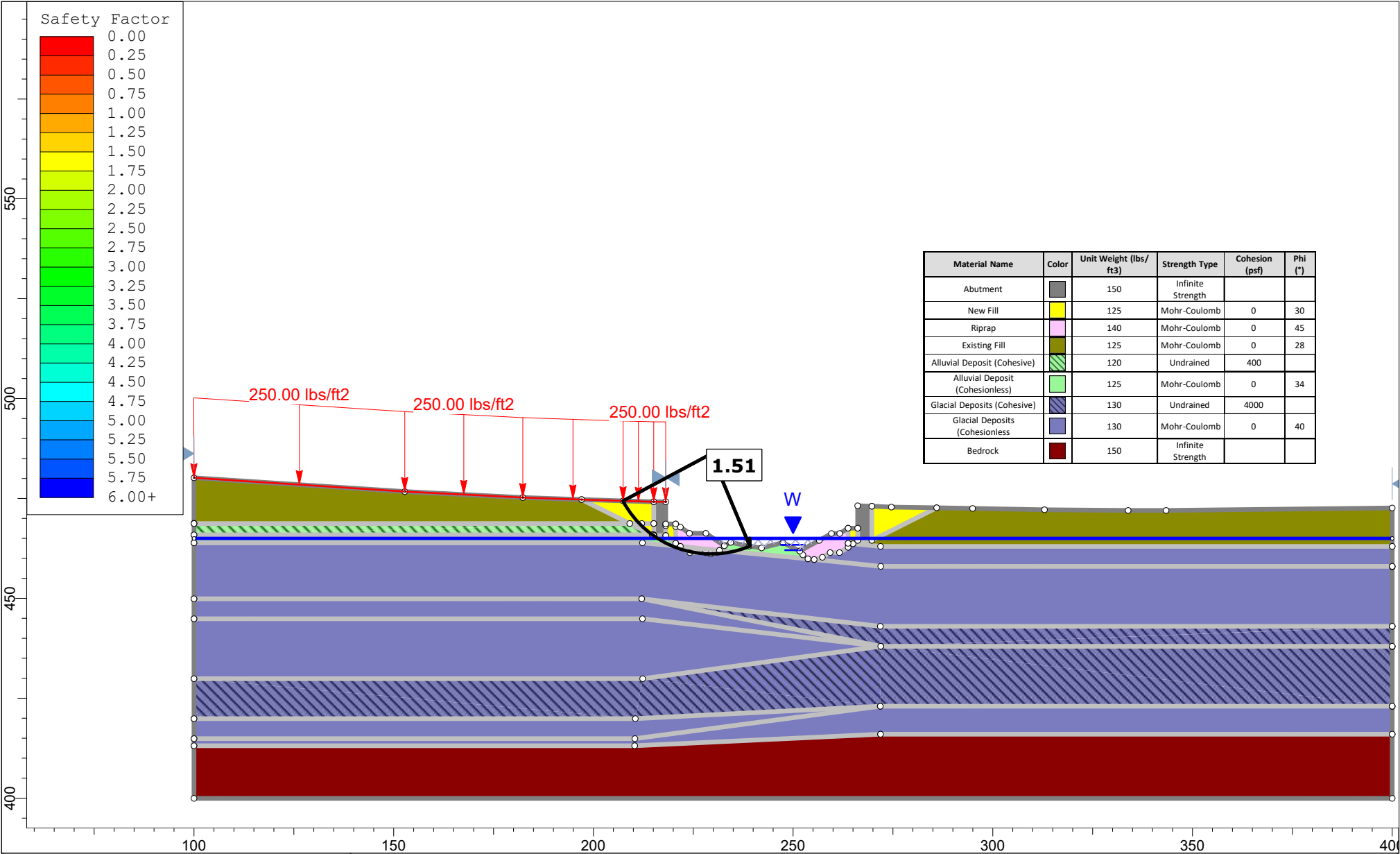
Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
New Fill (Granular Borrow)	125	30	0
Riprap	140	45	0
Existing Fill	125	28	0
Alluvial Deposit (Cohesive)	120	0	400
Alluvial Deposit (Cohesionless)	125	34	0
Glacial Deposits (Cohesive)	130	0	4,000
Glacial Deposits (Cohesionless)	130	40	0
Bedrock	Infinite Strength		

RESULTS AND CONCLUSIONS

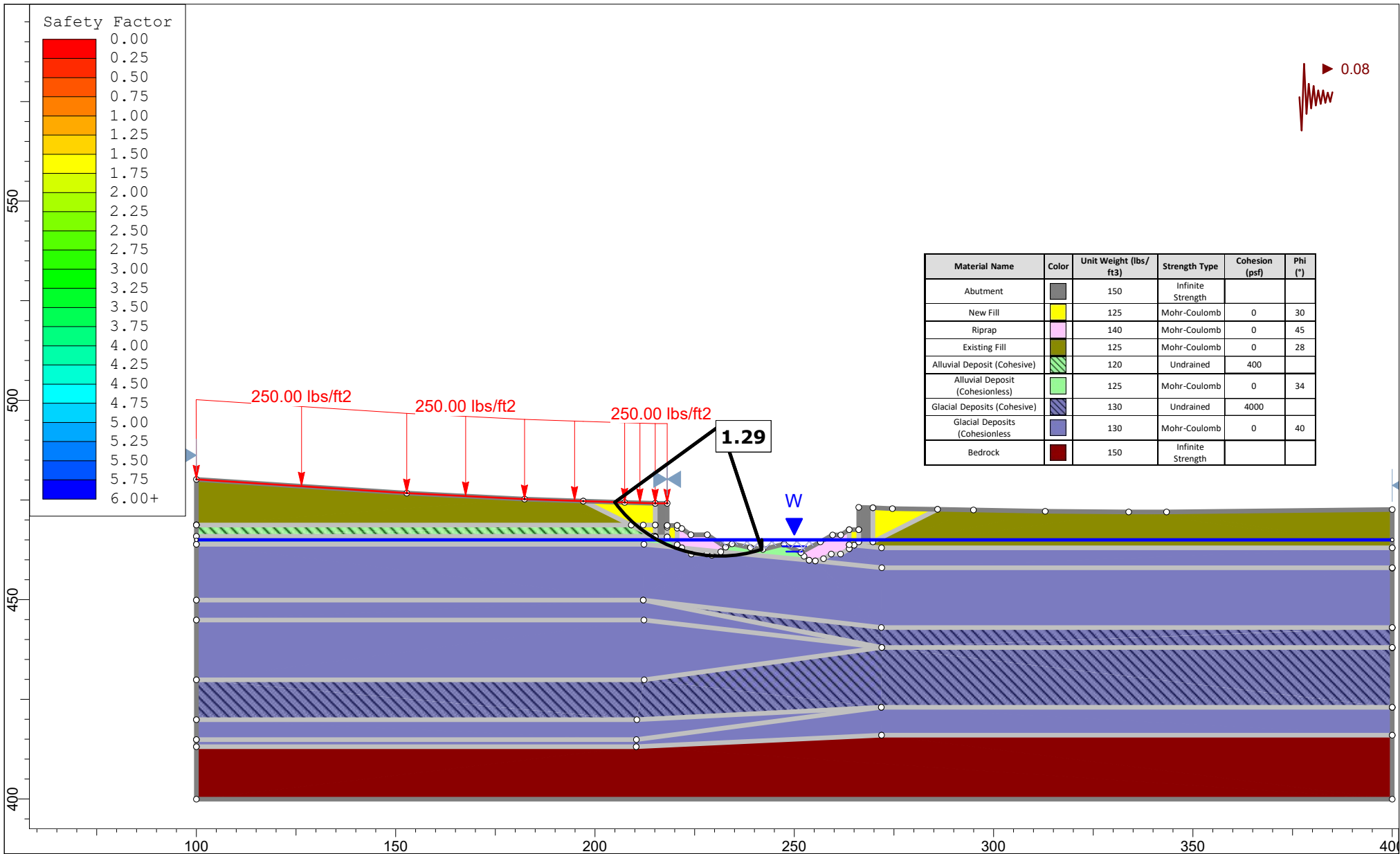
Structure	F.S.	F.S.
	Static	Seismic
Abutment No. 1 Longitudinal	1.51	1.29
Abutment No. 2 Longitudinal	1.50	1.29

Based on AASHTO LRFD Section 11.6.2.3, an acceptable resistance factor for where the geotechnical parameters and subsurface stratigraphy are well defined is 0.75 ($F.S. = 1/0.75 = 1.3$).

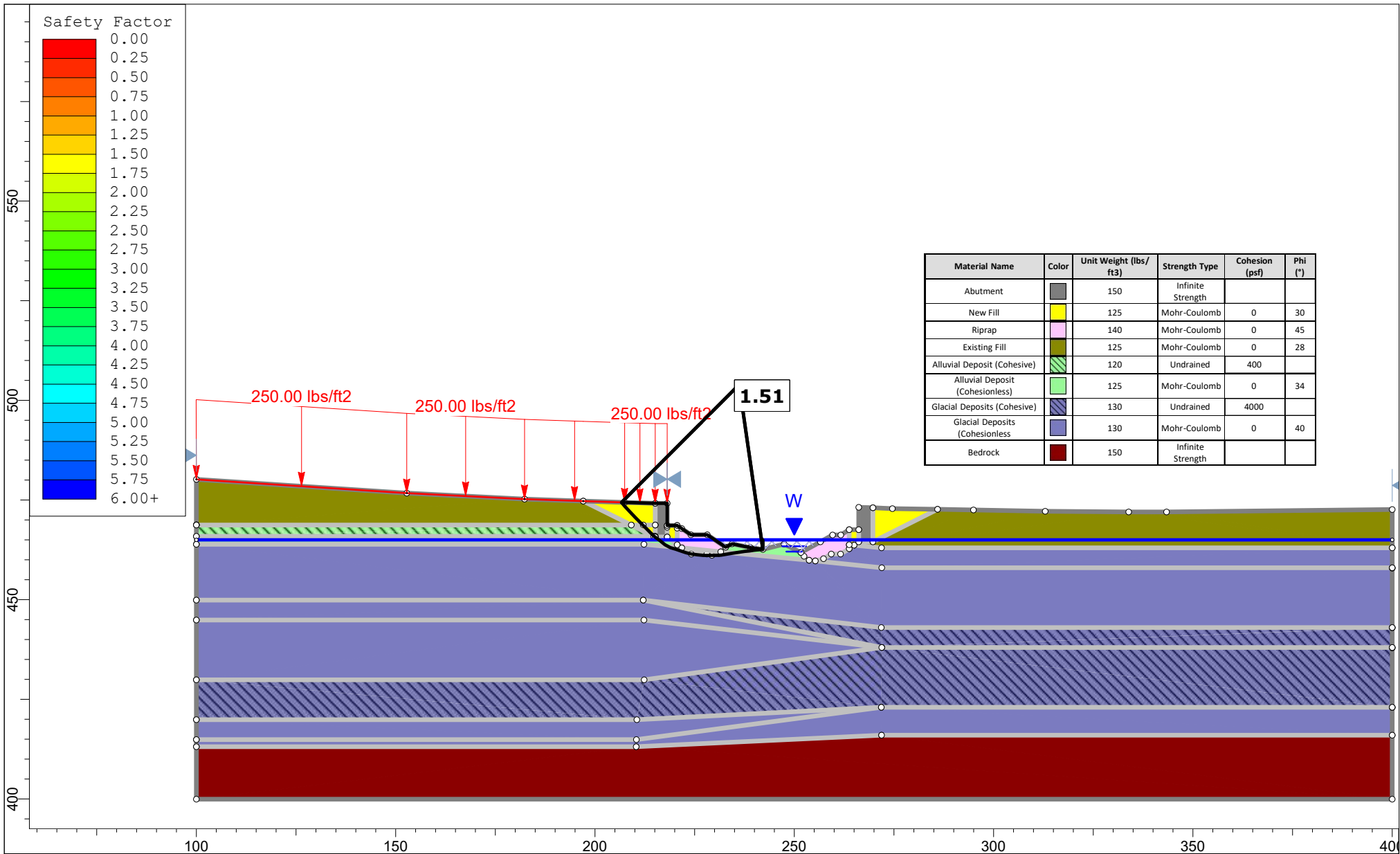
Based on recent MaineDOT guidance (L. Krusinski), a minimum pseudo-static factor of safety of 1.1 is acceptable.



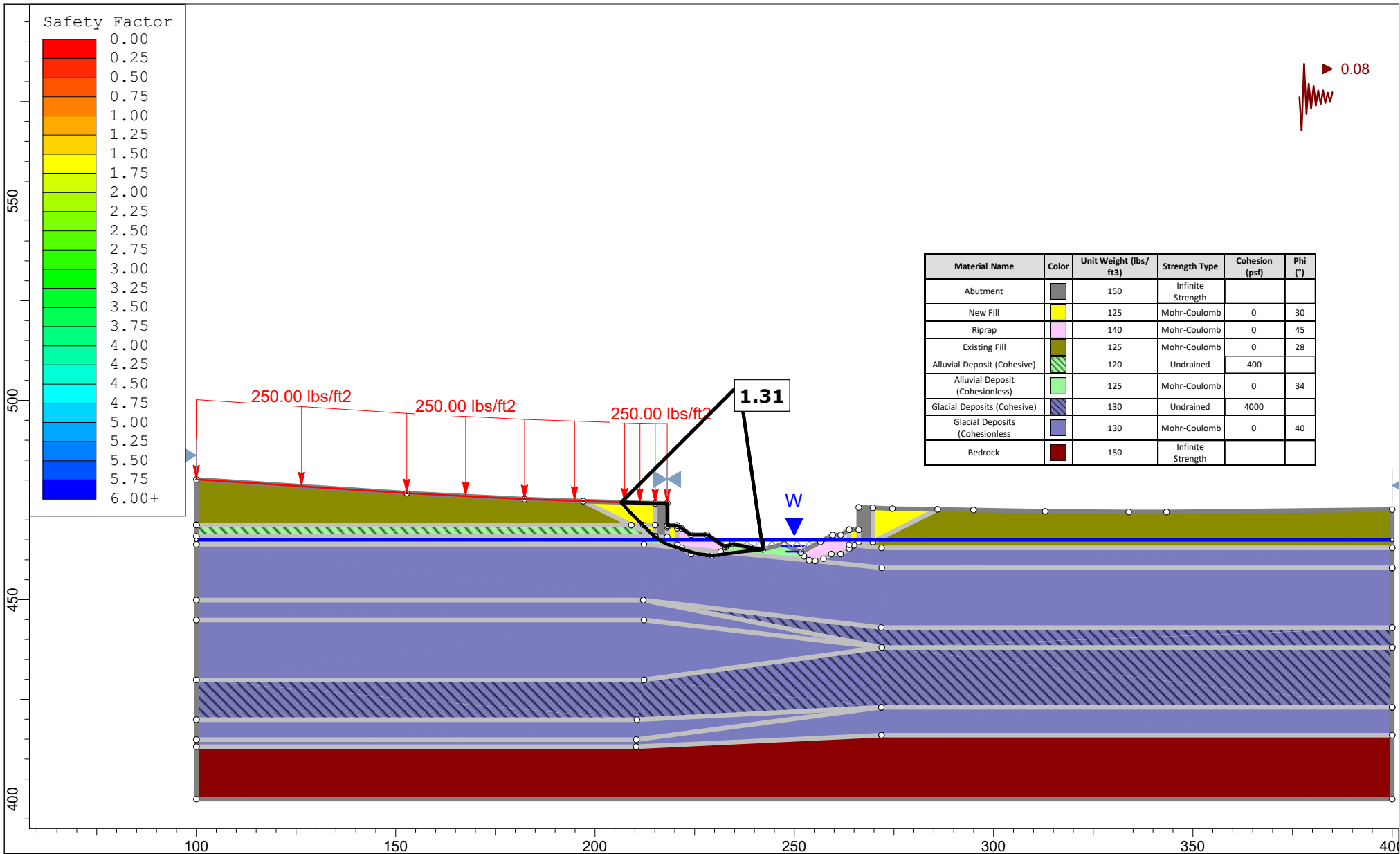
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	Group	Longitudinal Stability	Scenario Abutment 1 Circular - Static
	Drawn By	Tucker Jones	Company Haley & Aldrich, Inc.
	Date	9/18/2024, 8:21:26 PM	File Name 2024-0918-HAI-Merrills Bridge-Long Stability-d1.slmd



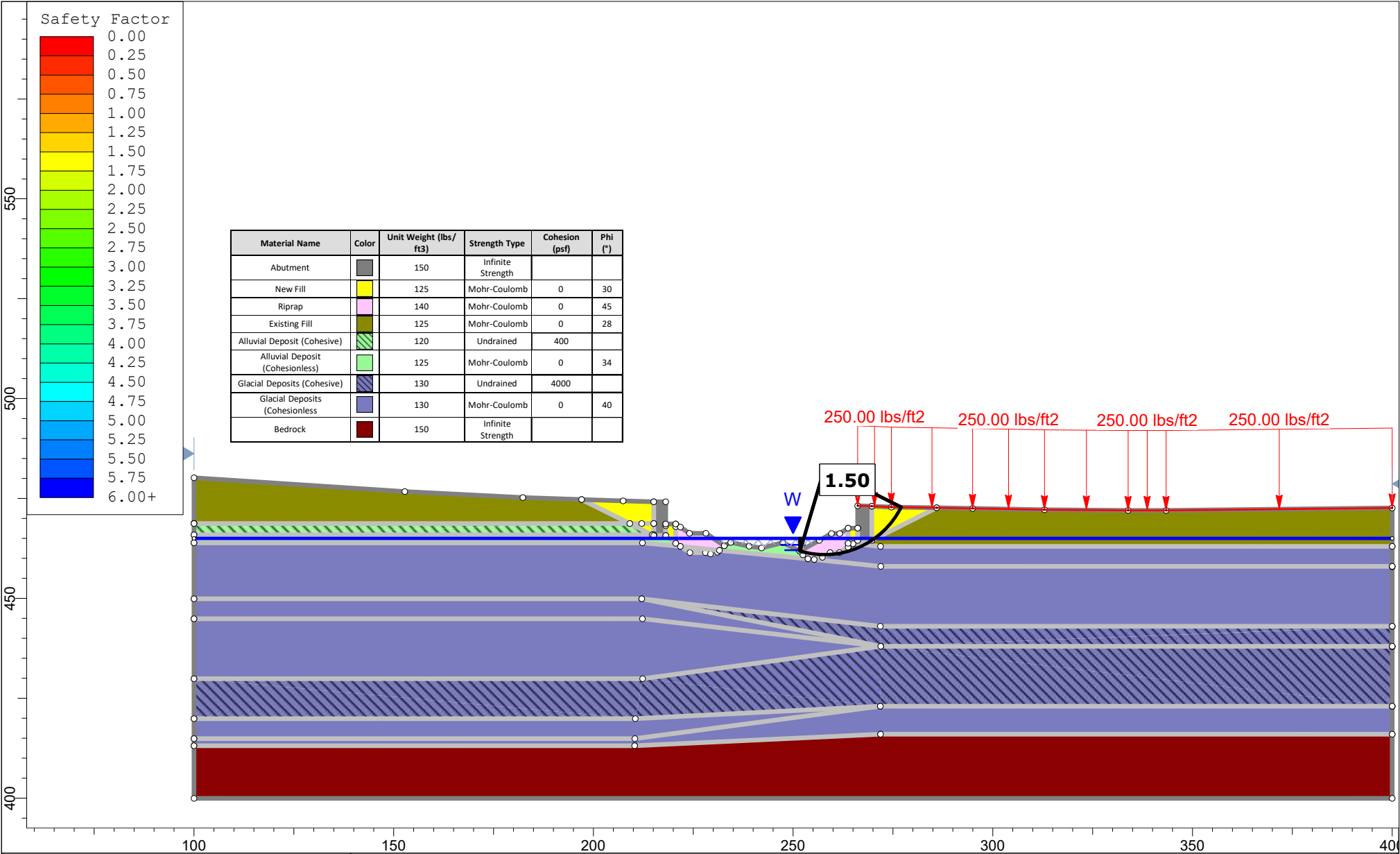
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	Date	9/18/2024, 8:21:26 PM	File Name	2024-0918-HAI-Merrills Bridge-Long Stability-d1.slmd
	SLIDEINTERPRET 9.029			



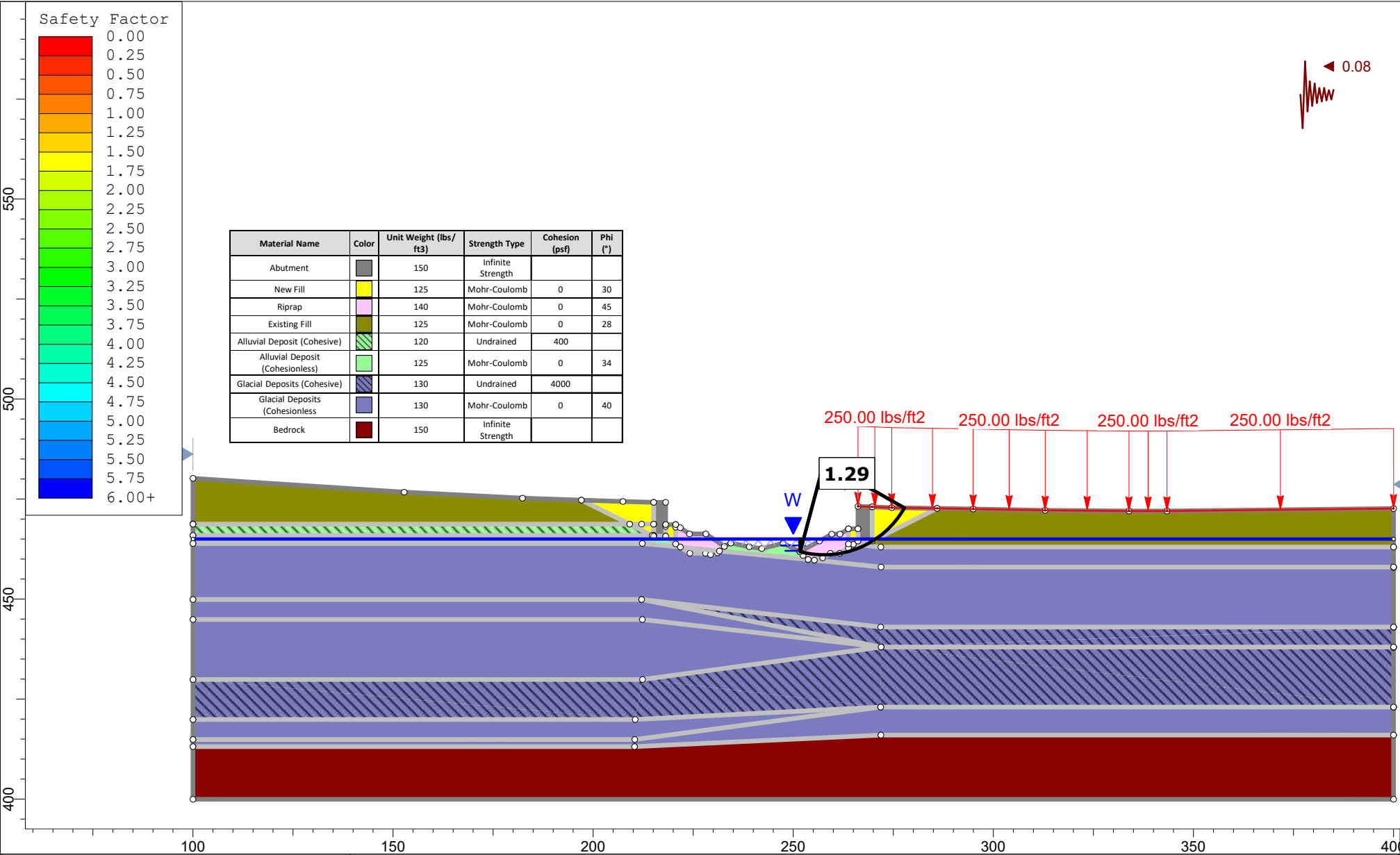
Project	Merrills Corner Bridge	
Group	Longitudinal Stability	Scenario Abutment 1 Noncircular - Static
Drawn By	Tucker Jones	Company Haley & Aldrich, Inc.
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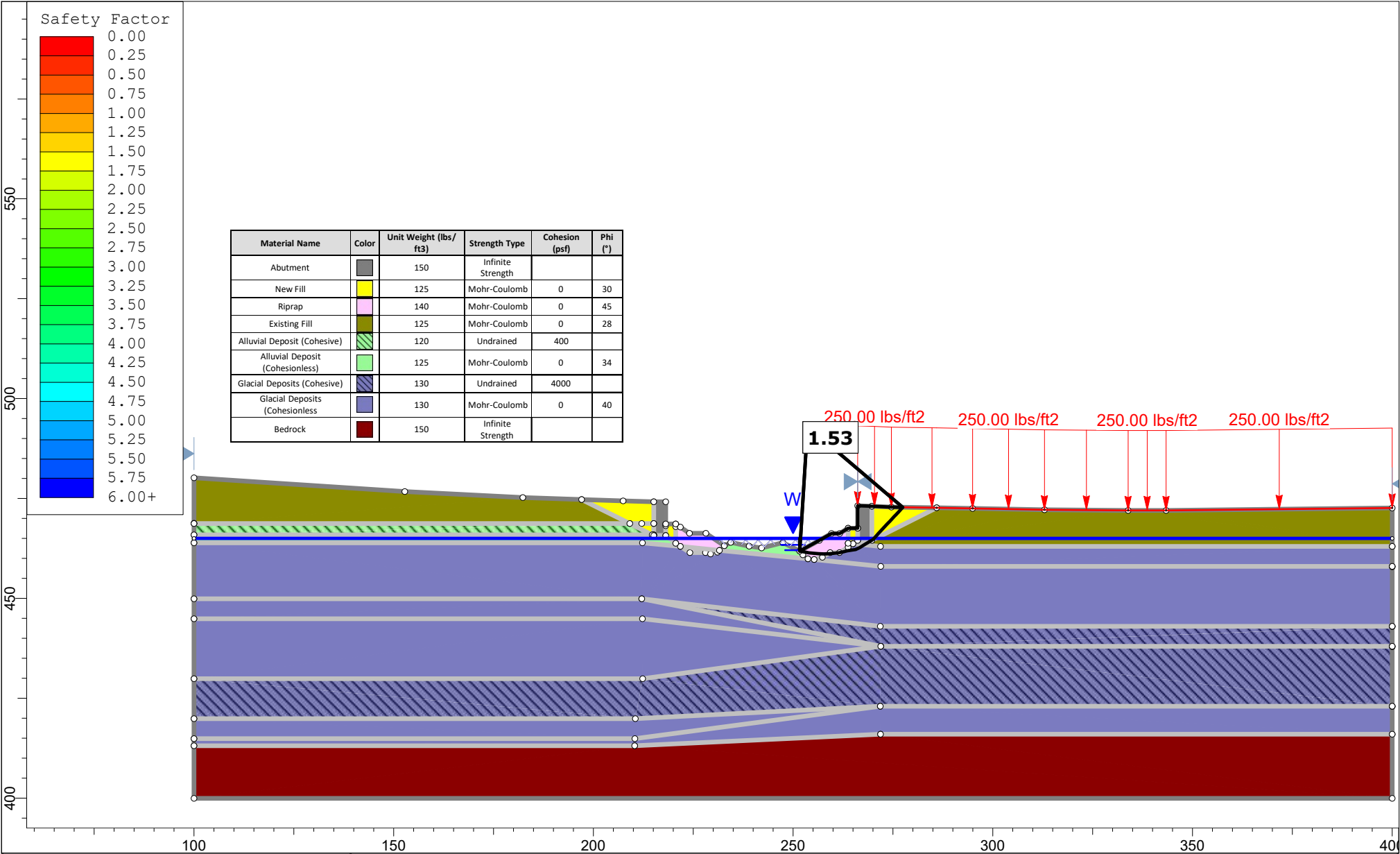
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	SLIDEINTERPRET 9.029			



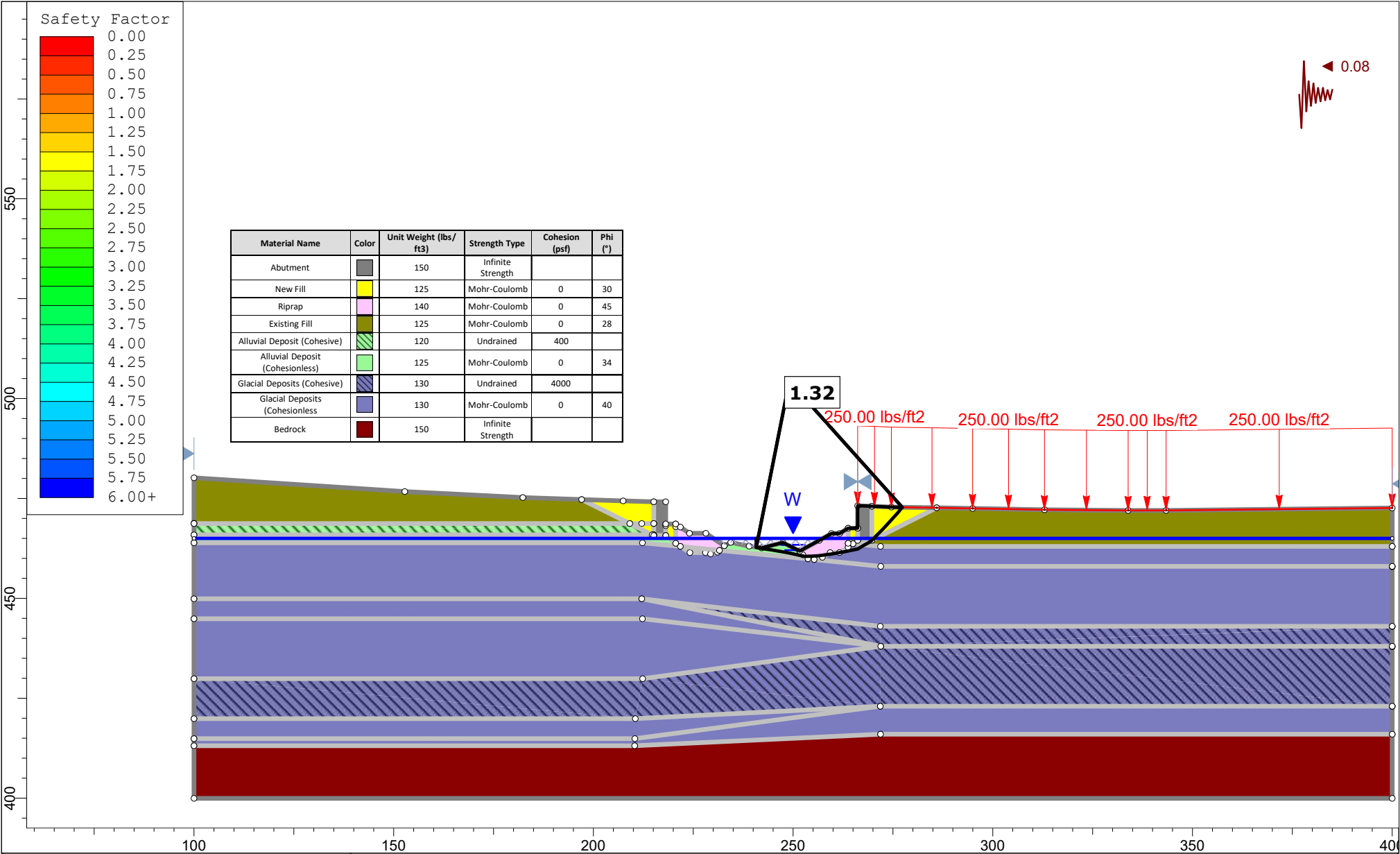
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	SLIDEINTERPRET 9.029			



 rocscience	Project		Merrills Corner Bridge	
	Group	Longitudinal Stability	Scenario	Abutment 2 Circular - Psuedostatic
	Drawn By	Tucker Jones	Company	Haley & Aldrich, Inc.
	Date	9/18/2024, 8:21:26 PM	File Name	2024-0918-HAI-Merrills Bridge-Long Stability-d1.slmd
	SLIDEINTERPRET 9.029			



 rocscience	Project		Merrills Corner Bridge	
	Group	Longitudinal Stability	Scenario	Abutment 2 Noncircular - Static
	Drawn By	Tucker Jones	Company	Haley & Aldrich, Inc.
	Date	9/18/2024, 8:21:26 PM	File Name	2024-0918-HAI-Merrills Bridge-Long Stability-d1.slmd
	SLIDEINTERPRET 9.029			



 rocscience	Project		Merrills Corner Bridge	
	Group	Longitudinal Stability	Scenario	Abutment 2 Noncircular - Psuedostatic
	Drawn By	Tucker Jones	Company	Haley & Aldrich, Inc.
	Date	9/18/2024, 8:21:26 PM	File Name	2024-0918-HAI-Merrills Bridge-Long Stability-d1.slmd
	SLIDEINTERPRET 9.029			

Pile Structural Resistance

PROBLEM STATEMENT AND OBJECTIVE

Calculate the structural axial pile resistances.

REFERENCES

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

ASSUMPTIONS

- 1. Per MaineDOT request, assume steel HP14x89 for the structural evaluations.
- 2. A pile yield stress of 50 ksi was considered for the structural resistance evaluations.
- 3. The pile axial structural resistance was evaluated assuming no corrosion.
- 4. The pile unbraced length is 0 ft (fully embedded pile) for the structural axial resistance calculations.
- 5. Piles will be driven to the top of bedrock. The structural resistance of the pile is assumed to govern the design based on AASHTO LRFD Section 10.7.3.2.
- 6. A resistance factor of 0.5 for the pile axial compression at the Strength Limit State was selected based on AASHTO LRFD Section 6.5.4.2 (H-Piles subjected to severe driving conditions where use of a pile tip is necessary).
- 7. A resistance factor of 1.0 for the pile compression axial resistance at the Extreme Event Limit State was selected based on AASHTO LRFD Section 10.5.5.3.2.

SUMMARY

Steel H-pile Section	Pile Nominal Structural Resistance in Axial Compression (kips)	Factored Structural Resistance (kips)		
		Service Limit State ($\phi=1.0$)	Strength Limit State ($\phi=0.5$)	Extreme Limit State ($\phi=1.0$)
HP14x89	1,305	1,305	653	1,305

Notes:

- 1. Piles are driven to hard rock. Therefore piles are governed by the structural resistance based on Section 10.7.3.2.3.
- 2. The structural resistance was calculated using Section 6.9.4.1 as referenced from Section 10.7.3.13.
- 3. The calculated resistance presented above only addresses axial resistance.
- 4. If the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated by others.

STRUCTURAL RESISTANCE - AASHTO LRFD

Note: This calculation only addresses axial resistance, if the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated by others. Refer to Section 6.15.

10.7.3.2 - Point Bearing Piles on Rock

10.7.3.2.3 Piles Driven to Hard Rock

The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal resistance shall not exceed the values obtained from Section 6.9.4.1 with the resustance factors specified in Section 6.5.4.2 and 6.15 for severe driving conditions.

10.7.3.13 - Pile Structural Resistance

10.7.3.12.1 - Steel Piles

The nomial axial compression resistance in the structural limit state for piles loaded in compression shall be as specified in Article 6.9.4.1 for non composite piles. The resistance factors for the compression limit state are specified in Article 6.5.4.2.

6.9.4 - Noncomposite Members

6.9.4.1 - Nominal Compressive Resistance

If $\frac{P_0}{P_e} \leq 2.25$, then $P_n = P_0 * (0.658^{\frac{P_0}{P_e}})$ 6.9.4.1.1-1

Otherise: $P_n = 0.877P_e$ 6.9.4.1.1-2

where

A_g = gross cross-sectional area of the member

F_y = specified minimum yield strength

P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional buckling or flexural-torisonal buckling as applicable

P_0 = nominal yield resistance = $F_y A_g$

6.9.4.1.2 - Elastic Flexural Buckling Resistance

The elastic critical buckling resistance, P_e , based on flexural buckling shall be taken as

$$P_e = \frac{\pi^2 E}{(\frac{Kl}{r_s})^2} A_g$$
 6.9.4.1.2-1

where

A_g = gross cross-sectional area of the member

K = effective length factor in the plane of buckling determined as specified in Article 4.6.2.5

l = unbraced length in the plane of buckling

r_s = radius of gyration about the axis normal to the plane of buckling

STRUCTURAL AXIAL RESISTANCE CALCULATIONS

Pile	Gross Section Area A_g (in ²)	Corrosion (in)	Effective Gross Area A_{eff} (in ²)	Effective Length Factor K	Unbraced l (ft)	Yield Stress fy (ksi)	Elastic Modulus E (ksi)	Radius of Gyration r_{weak} (in)	P_0 (kips)	P_e (kips)	P_0/P_e	P_n (kips)	ϕ (Strength Limit State)	ϕP_n (kips)
HP14x89	26.1	0	26.1	2	0	50	29,000	3.53	1,305	1.6E+15	8.1E-13	1,305	0.5	653

Note: The resistance factor of 0.5 for the Strength Limit State was selected based on Section 6.5.4.2 for H-piles subjected to severe driving conditions where the use of a pile tip is necessary.

Pile Drivability Resistance

Client: Maine Department of Transportation

Date: 2-Oct-2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Pile Drivability Analysis

Checked by: EMH

PROBLEM STATEMENT & OBJECTIVE

Determine nominal and factored drivability resistances for HP14x89 steel H-piles being considered to support the proposed abutments.

EXECUTIVE SUMMARY

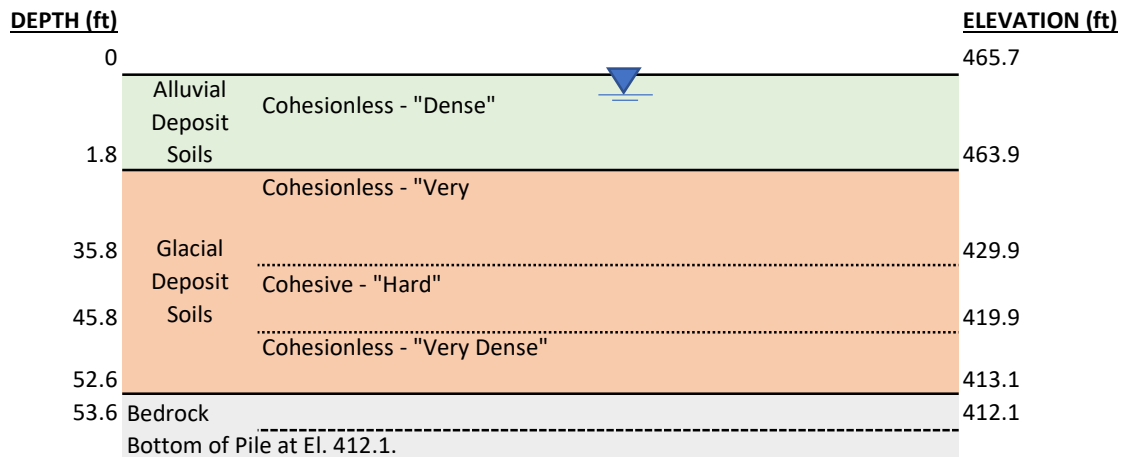
HP14x89 steel H-piles can be installed to a nominal resistance equal to 800 kips (factored resist. = 520 kips) with a Delmag D 25-32 OED hammer operating on the max fuel setting at a penetration resistance equal to 9 bpi and with a maximum compressive stress during driving less than 45 ksi.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. Maine Department of Transportation Standard Specifications, March 2020, with interim revisions through 9/7/22.
3. GRLWEAP 2014 v14.1.20.1 technical manual.

AVAILABLE INFORMATION

1. Boring logs BB-BSR-101 and BB-BSR-101A. Note we are assuming stratigraphy at both abutments are similar.
2. Soil profile below shows average thicknesses encountered below the bottom of Abutment No. 1 (El. 465.7).
Note that soil stratigraphy is similar to soils at Abutment No. 2 for driving.



Groundwater = El. 465.0 (depth = 0.7 ft relative to bottom of abutment)

ASSUMPTIONS

1. Elevations are in feet (ft) and reference the North American Vertical Datum of 1988 (NAVD88).
2. Soil and rock conditions based on referenced borings. Assume similar soil stratigraphy at Abutment No. 2.
3. Piles will be driven approximately 1 ft into rock based on conditions encountered in the referenced test borings.
4. One steel H-pile section will be analyzed (HP14x89). Piles are assumed to be plumb (vertical).
5. Piles are approx. 58.6-ft long, which includes 1 ft of pen. into rock and 5 ft of stickup above bottom of abutment.
6. Factored resist. is taken as the nominal resist. multiplied by 0.65 (resistance factor for CAPWAP dynamic testing).
7. Shaft and toe quake and damping factors selected based on experience and default values suggested in WEAP.
8. Penetration resistance is limited to a maximum of 15 bpi in accordance with the MaineDOT Std. Specifications.
9. Limit pile compressive stress during driving to $0.9F_y$ or 45 ksi for $F_y = 50$ ksi steel. It is noted that in our experience, WEAP tends to overpredict compressive driving stresses (i.e., actual compressive driving stresses are less than those predicted/calculated by WEAP).
10. Select pile hammer(s) based on those readily available to local pile driving contractors.



CALCULATIONS

File No.: 0210010-000

Sheet: 2 of 2

Client: Maine Department of Transportation

Date: 2-Oct-2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Pile Drivability Analysis

Checked by: EMH

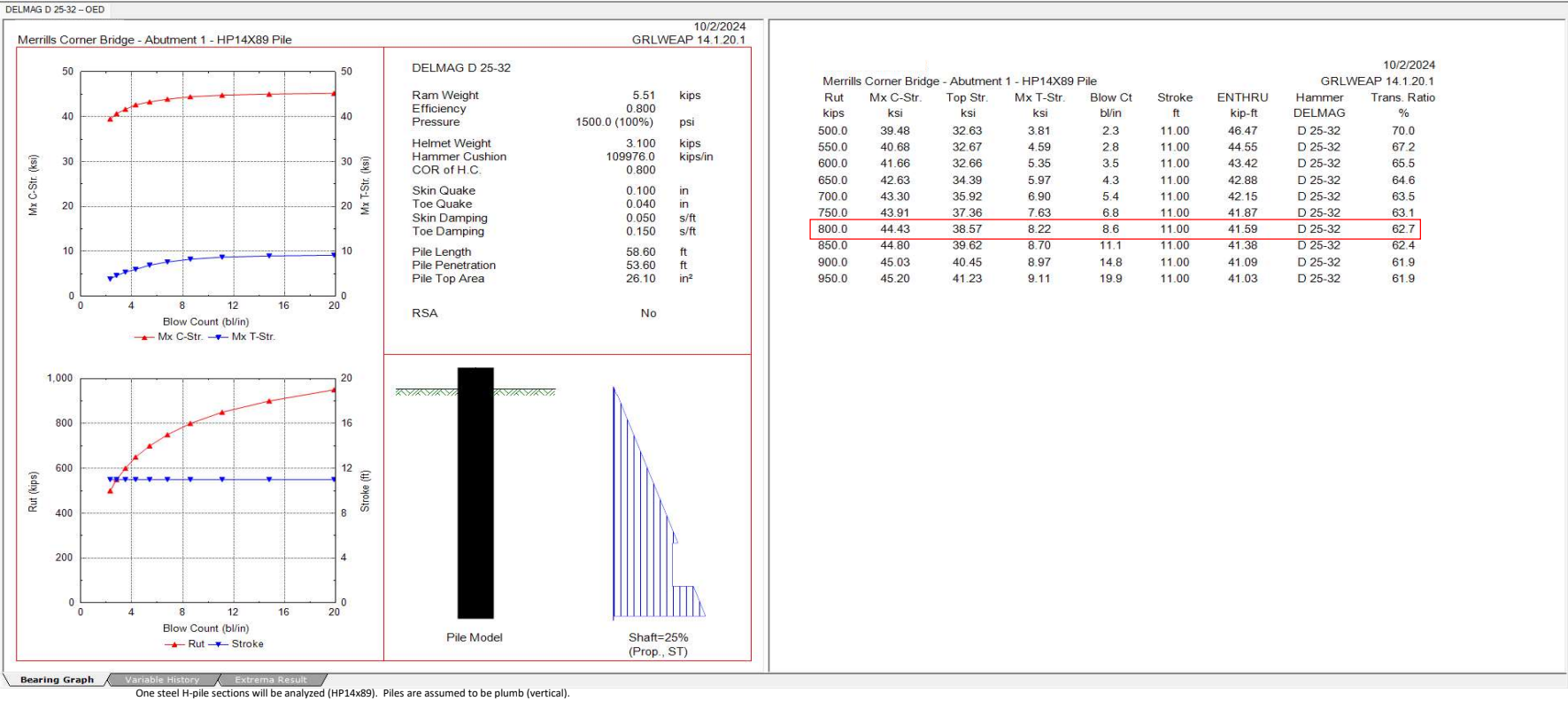
CALCULATIONS AND RESULTS (SEE ATTACHED FOR WEAP OUTPUT)

Determine nominal and factored drivability resistances for HP14x89 steel H-piles

HP14x89 Steel H-pile:

Pile Hammer Make and Model =	Delmag D 25-32	
Maximum Rated Energy =	66,340	ft-lbs.
Fuel Setting =	max	
Ram Stroke =	10.8	ft
Nominal Resistance =	800	kips
Transferred Energy at Nominal Resistance =	41,590	ft-lbs.
Maximum Compressive Stress =	44	ksi
Penetration Resistance =	9	bpi
Factored Compressive Resistance =	520	kips ($f_{dyn} = 0.65$)

HP14x89 WEAP Output



Lateral Pile Analyses

Client: Maine Department of Transportation

Date: 7/31/2025

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: CFE

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

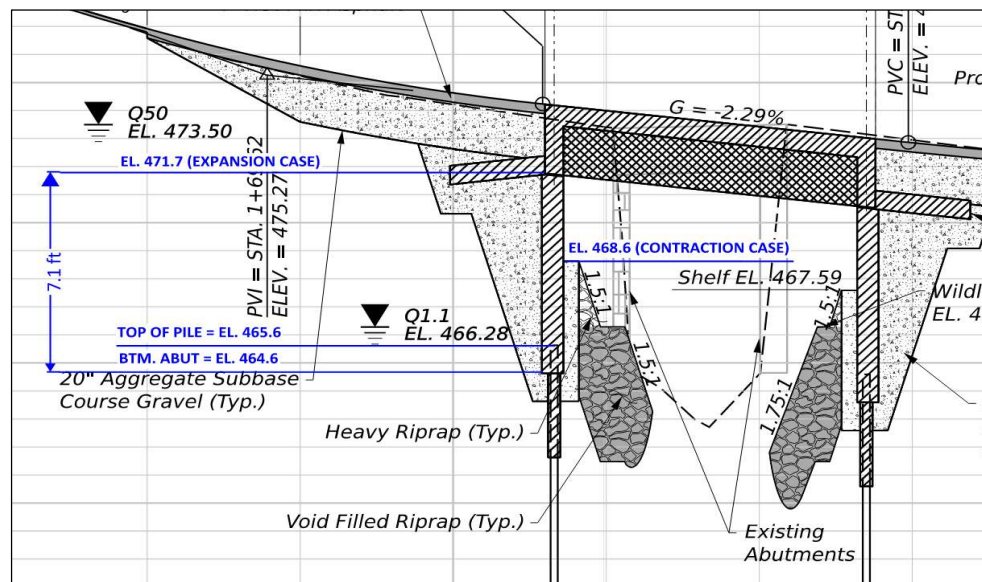
Checked by: EMH

PROBLEM STATEMENT & OBJECTIVE

Determine design soil profile and soil properties for Abutment 1 integral abutment and lateral pile analysis.
Conduct lateral pile analysis for HP14x89 steel H-piles to determine whether combined stresses are above or below the elastic range.

AVAILABLE INFORMATION, REFERENCES AND ASSUMPTIONS

1. Abutment 1 test boring logs BB-BSR-101, BB-BSR-101A, and BB-BSR-102.
2. Ensoft, Inc. LPILE v2022 Technical Manual, March 2024.
3. Bottom of abutment and top of pile elevations provided by MaineDOT.
4. Maximum factored (STR) axial compressive pile load (262.9 kips) and thermal displacement (0.19 in.) provided by MaineDOT.
5. Evaluate HP14x89 steel H-pile, weak axis orientation.
6. Assume no reduction in cross-sectional area due to corrosion.
7. Lateral pile evaluations for a single pile only, group effects not considered.
8. Separate evaluations completed for each pile section and for expansion and contraction conditions because of the differences in proposed finish grade behind (El. 471.7; expansion case) and in front (El. 468.6; contraction case) of the abutment. Expan. case ground elevation assumed at bottom of approach slab (per MaineDOT request).
9. Concrete pile collar extending below the bottom of the abutment, if required/present, is not considered.
10. Pile head condition is assumed to be "fixed" (pile head rotation = 0 deg.).

PROFILE

Parameter	Expansion Case ¹	Contraction Case
GS Elev.	El. 471.7	El. 468.6
Pile Top Elev.	El. 464.6	El. 464.6
Pile Top Depth	7.1 ft	5.7 ft
Top of Alluvial Dep.	El. 464.6	
Top of GT/GF Elev.	El. 463.9	
Top of Glac. Lac. Dep. El.	El. 429.9	
Top of Bedrock Elev.	El. 413.1	
Groundwater Elev.	El. 466.3	

¹ (assumed to be at the bottom elev. of the approach slab)
(pile top elev. assumed to be bottom of abutment elev.)
(difference between gs elev. and pile top elev.)
(assumed to be coincident with bottom of abutment elev.)
(0.7 ft below bottom of abutment)
(34.7 ft below bottom of abutment)
(51.5 ft below bottom of abutment)
(1.7 ft above bottom of abutment)

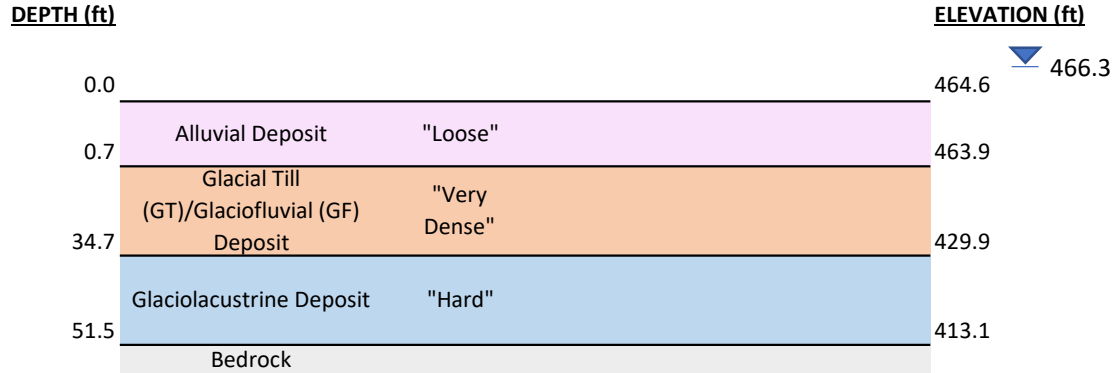
Client: Maine Department of Transportation

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

SOIL PROFILE (CONT.)

Soil profile below and summarized on the previous sheet shows approximate average thicknesses encountered below the bottom of Abut. 1 (El. 464.6)



Summary of LPILE Soil Property Input Parameters (per Haley & Aldrich PGDR dated 2/28/25)

Stratum	Moisture Condition	LPile Soil Model	Unit Weight (pcf)		Friction Angle (ϕ , deg.)	Undrained Shear Strength (S_u , psf)	Soil Modulus (k, pci)	Strain Factor (e_{50} , -)
			Total	Buoyant				
Alluv. Dep.	Wet	Sand Reese	125	62.6	34	-	160	-
GT/GF	Wet	Sand Reese	130	67.6	40	-	160	-
Glac. Dep.	Wet	Stiff Clay w/o Free Water (Reese)	130	67.6	-	4000	-	0.005

PILE PROPERTIES

Parameter	Steel H-pile Section
	HP14x89
Yield Strength (F_y , ksi)	50
Modulus of Elasticity (E , ksi)	29,000
Elastic Section Modulus (S_y , in ³)	44.3
Flange Width (b , in.)	14.7
Section Depth (d , in.)	13.8
Flange Thickness (t_f , in.)	0.615
Web Thickness (t_w , in.)	0.615
Cross Sectional Area (A_g , in ²)	26.1

Client: Maine Department of Transportation

Date: 7/31/2025

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: CFE

Subject: Abut. 1 Soil Profile and Properties for Integral Abut. Pile Design + Lateral Pile Analysis

Checked by: EMH

SUMMARY OF RESULTS

Pile Section	Case	Maximum Factored Axial Comp. Pile Load ¹ (kip)	Shear Force Resulting in 0.19 in. of Lateral Pile Head Disp. ² (kips)	Maximum Moment at Pile Head ² (kip-ft)	Depth Below Pile Head to Fixity ³ (ft)	Total Stress at Pile Head ² (ksi)	Top Unbraced Length Segment ⁴ (L1, ft)	Bottom Unbraced Length Segment ⁴ (L2, ft)
HP14x89	Expansion	262.9	74.7	162.5	16.1	54.2	2.8	9.1
	Contraction		59.3	141.6	16.7	48.5	3.0	9.4

Notes:

¹ Provided by MaineDOT.

² Taken directly from LPILE Summary Reports (see attached output).

³ Taken as the depth along the pile to the second point of zero lateral deflection.

⁴ Calculated by linear interpolation (see attached output).

Abutment 1 - HP14x89 Expansion

=====

LPILE for Version 2022-12.012

License ID : 235486821
License Type : (Network License)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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LPILE Sm Global, Global License

=====

This model was prepared by:
CFE

Haley & Aldrich, Inc.
Portland, ME

Files Used for Analysis

Path to file locations:
\Users\cfernandezescobar\haleyaldrich.com\Maine Department of Transportation - 0210010.MaineDOT-Merrills
Bridge No. 0714\Calculations\Lateral Pile Analyses\F2\

Name of input data file:
2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Expansion Case-F.lp12d

Name of output report file:
2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Expansion Case-F.lp12o

Name of plot output file:

2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Expansion Case-F.lp12p

Name of runtime message file:

2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Expansion Case-F.lp12r

Date and Time of Analysis

Date: July 31, 2025

Time: 15:41:24

Problem Title

Project Name: Brownfield Merrills Corner Bridge No 0714

Job Number: 0210010-000

Client: Maine Department of Transportation

Engineer: Camilo Fernandez Escobar

Description: Abutment 1 Expansion Case F

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 40.000 ft
Depth of ground surface below top of pile = -7.1000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.7000
2	40.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 40.000000 ft
Pile width = 13.800000 in

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	-7.10000 ft
Distance from top of pile to bottom of layer	=	-1.70000 ft
Effective unit weight at top of layer	=	125.000000 pcf
Effective unit weight at bottom of layer	=	125.000000 pcf
Friction angle at top of layer	=	34.000000 deg.
Friction angle at bottom of layer	=	34.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	-1.70000 ft
Distance from top of pile to bottom of layer	=	0.700000 ft
Effective unit weight at top of layer	=	62.600000 pcf
Effective unit weight at bottom of layer	=	62.600000 pcf
Friction angle at top of layer	=	34.000000 deg.
Friction angle at bottom of layer	=	34.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 3 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	0.700000 ft
Distance from top of pile to bottom of layer	=	34.700000 ft
Effective unit weight at top of layer	=	67.600000 pcf
Effective unit weight at bottom of layer	=	67.600000 pcf
Friction angle at top of layer	=	40.000000 deg.
Friction angle at bottom of layer	=	40.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 4 is stiff clay without free water

Distance from top of pile to top of layer	=	34.700000 ft
Distance from top of pile to bottom of layer	=	51.500000 ft
Effective unit weight at top of layer	=	67.600000 pcf
Effective unit weight at bottom of layer	=	67.600000 pcf
Undrained cohesion at top of layer	=	4000. psf
Undrained cohesion at bottom of layer	=	4000. psf
Epsilon-50 at top of layer	=	0.005000
Epsilon-50 at bottom of layer	=	0.005000

(Depth of the lowest soil layer extends 11.500 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand (Reese, et al.)	-7.100 -1.700	125.0000 125.0000	-- --	34.0000 34.0000	-- --	160.0000 160.0000
2	Sand (Reese, et al.)	-1.700 0.7000	62.6000 62.6000	-- --	34.0000 34.0000	-- --	160.0000 160.0000
3	Sand (Reese, et al.)	0.7000 34.7000	67.6000 67.6000	-- --	40.0000 40.0000	-- --	160.0000 160.0000
4	Stiff Clay w/o Free Water	34.7000 51.5000	67.6000 67.6000	4000. 4000.	-- --	0.00500 0.00500	-- --

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	5	y = 0.190000 in	S = 0.0000 in/in	262890.	N.A.	Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	40.000000 ft
Flange Width	=	14.700000 in
Section Depth	=	13.800000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.811550 sq. in.
Moment of Inertia	=	325.837265 in^4
Elastic Bending Stiffness	=	9449281. kip-in^2
Plastic Modulus, Z	=	67.636247in^3
Plastic Moment Capacity = Fy Z	=	3382.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1290.578 kips
Nominal Axial Tensile Capacity	=	-1290.578 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
-----	-----
1	262.890

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 262.890 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000467	44.1580136	9448389.	82.4967557	11.1711919	
0.00000935	88.3160273	9448389.	44.9233779	12.1574084	
0.00001402	132.4740409	9448389.	32.3989186	13.1436248	
0.00001869	176.6320546	9448389.	26.1366889	14.1298416	
0.00002337	220.7900682	9448389.	22.3793511	15.1160581	
0.00002804	264.9480819	9448389.	19.8744593	16.1022745	
0.00003272	309.1060955	9448389.	18.0852508	17.0884911	
0.00003739	353.2641091	9448389.	16.7433445	18.0747077	
0.00004206	397.4221228	9448389.	15.6996395	19.0609242	
0.00004674	441.5801364	9448389.	14.8646756	20.0471408	
0.00005141	485.7381501	9448389.	14.1815232	21.0333573	
0.00005608	529.8961637	9448389.	13.6122296	22.0195739	
0.00006076	574.0541774	9448389.	13.1305197	23.0057904	
0.00006543	618.2121910	9448389.	12.7176254	23.9920069	
0.00007010	662.3702046	9448389.	12.3597837	24.9782235	
0.00007478	706.5282183	9448389.	12.0466722	25.9644401	
0.00007945	750.6862319	9448389.	11.7703974	26.9506567	
0.00008412	794.8442456	9448389.	11.5248198	27.9368732	
0.00008880	839.0022592	9448389.	11.3050924	28.9230897	
0.00009347	883.1602729	9448389.	11.1073378	29.9093063	
0.00009815	927.3182865	9448389.	10.9284169	30.8955228	
0.0001028	971.4763001	9448389.	10.7657616	31.8817393	
0.0001075	1016.	9448389.	10.6172502	32.8679559	
0.0001122	1060.	9448389.	10.4811148	33.8541725	
0.0001168	1104.	9448389.	10.3558702	34.8403890	
0.0001215	1148.	9448389.	10.2402598	35.8266055	
0.0001262	1192.	9448389.	10.1332132	36.8128221	
0.0001309	1236.	9448389.	10.0338127	37.7990386	
0.0001355	1281.	9448389.	9.9412674	38.7852552	
0.0001402	1325.	9448389.	9.8548919	39.7714717	
0.0001449	1369.	9448389.	9.7740889	40.7576883	

0.0001496	1413.	9448389.	9.6983361	41.7439048	
0.0001542	1457.	9448389.	9.6271744	42.7301214	
0.0001589	1501.	9448389.	9.5601987	43.7163379	
0.0001636	1546.	9448389.	9.4970502	44.7025545	
0.0001682	1590.	9448389.	9.4374099	45.6887710	
0.0001729	1634.	9448389.	9.3809934	46.6749876	
0.0001776	1678.	9448389.	9.3275462	47.6612041	
0.0001823	1722.	9448389.	9.2768399	48.6474207	
0.0001916	1810.	9444103.	9.1836346	50.0000000	Y
0.0002010	1892.	9413671.	9.1040650	50.0000000	Y
0.0002103	1969.	9359973.	9.0366409	50.0000000	Y
0.0002197	2041.	9289863.	8.9792465	50.0000000	Y
0.0002290	2109.	9207853.	8.9302973	50.0000000	Y
0.0002384	2173.	9116470.	8.8887068	50.0000000	Y
0.0002477	2234.	9019283.	8.8532041	50.0000000	Y
0.0002570	2292.	8918506.	8.8228475	50.0000000	Y
0.0002664	2348.	8814690.	8.7971493	50.0000000	Y
0.0002757	2402.	8709873.	8.7752772	50.0000000	Y
0.0002851	2453.	8605091.	8.7566709	50.0000000	Y
0.0002944	2502.	8499271.	8.7408580	50.0000000	Y
0.0003038	2548.	8386360.	8.7255994	50.0000000	Y
0.0003131	2589.	8268413.	8.7107431	50.0000000	Y
0.0003225	2627.	8146926.	8.6962287	50.0000000	Y
0.0003318	2662.	8022764.	8.6818801	50.0000000	Y
0.0003412	2694.	7897563.	8.6678004	50.0000000	Y
0.0003505	2724.	7772029.	8.6540976	50.0000000	Y
0.0003599	2752.	7646799.	8.6407892	50.0000000	Y
0.0003692	2778.	7522824.	8.6277163	50.0000000	Y
0.0003786	2801.	7400303.	8.6147148	50.0000000	Y
0.0003879	2824.	7279070.	8.6020602	50.0000000	Y
0.0003973	2844.	7159937.	8.5896959	50.0000000	Y
0.0004066	2864.	7043603.	8.5774181	50.0000000	Y
0.0004160	2882.	6929145.	8.5658335	50.0000000	Y
0.0004253	2899.	6817003.	8.5539351	50.0000000	Y
0.0004346	2915.	6707161.	8.5425572	50.0000000	Y
0.0004440	2931.	6600570.	8.5313795	50.0000000	Y
0.0004533	2945.	6495495.	8.5203278	50.0000000	Y
0.0004627	2958.	6393702.	8.5093716	50.0000000	Y

0.0004720	2971.	6293907.	8.4989667	50.0000000	Y
0.0004814	2983.	6196739.	8.4883460	50.0000000	Y
0.0004907	2994.	6102125.	8.4783181	50.0000000	Y
0.0005001	3005.	6009417.	8.4681420	50.0000000	Y
0.0005094	3016.	5919847.	8.4582437	50.0000000	Y
0.0005188	3025.	5831511.	8.4485653	50.0000000	Y
0.0005281	3035.	5746119.	8.4391279	50.0000000	Y
0.0005375	3043.	5662682.	8.4294944	50.0000000	Y
0.0005468	3052.	5581167.	8.4204915	50.0000000	Y
0.0005562	3060.	5501860.	8.4114652	50.0000000	Y
0.0005935	3089.	5203738.	8.3766669	50.0000000	Y
0.0006309	3113.	4933791.	8.3441988	50.0000000	Y
0.0006683	3133.	4688429.	8.3135805	50.0000000	Y
0.0007057	3151.	4465145.	8.2847838	50.0000000	Y
0.0007431	3166.	4261178.	8.2576413	50.0000000	Y
0.0007805	3180.	4074116.	8.2322547	50.0000000	Y
0.0008179	3192.	3902464.	8.2078951	50.0000000	Y
0.0008553	3202.	3744047.	8.1854265	50.0000000	Y
0.0008927	3211.	3597676.	8.1635425	50.0000000	Y
0.0009300	3220.	3461847.	8.1429727	50.0000000	Y
0.0009674	3227.	3335914.	8.1236089	50.0000000	Y
0.0010048	3234.	3218521.	8.1046560	50.0000000	Y
0.0010422	3240.	3108919.	8.0871266	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	262.8900000000	3240.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-7.100	0.00	N.A.	No	0.00	27527.
2	-1.700	5.3999	Yes	No	27527.	34973.
3	0.7000	6.7435	Yes	No	62500.	6364263.
4	34.7000	150.4171	No	No	6426763.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection

for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)

Displacement of pile head	=	0.190000 inches
Rotation of pile head	=	0.000E+00 radians
Axial load on pile head	=	262890.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
0.00	0.1900	-1949755.	74746.	0.00	54166.	9.37E+09	-650.014	8211.	
0.4000	0.1876	-1598486.	71377.	-9.09E-04	46242.	9.37E+09	-696.773	17828.	
0.8000	0.1813	-1262239.	66803.	-0.00164	38658.	9.45E+09	-1209.	32022.	
1.2000	0.1719	-953045.	60871.	-0.00220	31683.	9.45E+09	-1262.	35251.	
1.6000	0.1601	-672322.	54722.	-0.00261	25351.	9.45E+09	-1300.	38962.	
2.0000	0.1468	-421119.	48429.	-0.00289	19684.	9.45E+09	-1322.	43232.	
2.4000	0.1324	-200105.	42069.	-0.00305	14699.	9.45E+09	-1328.	48158.	
2.8000	0.1175	-9563.	35717.	-0.00310	10401.	9.45E+09	-1319.	53862.	
3.2000	0.1026	150606.	29448.	-0.00307	13582.	9.45E+09	-1293.	60503.	
3.6000	0.08806	280883.	23338.	-0.00296	16521.	L1=[9,563/((9,563+150,606)/(3.2-2.8))]+2.8 = 2.8 FT			
4.0000	0.07420	382116.	17456.	-0.00279	18804.	9.45E+09	-1198.	77490.	

4.4000	0.06128	455504.	11870.	-0.00258	20460.	9.45E+09	-1130.	88475.
0.00								
4.8000	0.04947	502574.	6642.	-0.00233	21522.	9.45E+09	-1049.	101757.
0.00								
5.2000	0.03889	525159.	1921.	-0.00207	22031.	9.45E+09	-918.337	113357.
0.00								
5.6000	0.02958	526248.	-2014.	-0.00180	22056.	9.45E+09	-721.314	117043.
0.00								
6.0000	0.02156	510381.	-5046.	-0.00154	21698.	9.45E+09	-542.277	120730.
0.00								
6.4000	0.01478	481693.	-7267.	-0.00129	21051.	9.45E+09	-383.177	124416.
0.00								
6.8000	0.00918	443868.	-8775.	-0.00105	20197.	9.45E+09	-245.016	128102.
0.00								
7.2000	0.00466	400113.	-9670.	-8.40E-04	19210.	9.45E+09	-127.968	131789.
0.00								
7.6000	0.00112	353153.	-10053.	-6.49E-04	18151.	9.45E+09	-31.515	135475.
0.00								
8.0000	-0.00157	305240.	-10020.	-4.81E-04	17070.	9.45E+09	45.4157	139162.
0.00								
8.4000	-0.00351	258179.	-9660.	-3.38E-04	16009.	9.45E+09	104.3159	142848.
0.00								
8.8000	-0.00481	213355.	-9057.	-2.19E-04	14998.	9.45E+09	146.9744	146534.
0.00								
9.2000	-0.00560	171781.	-8284.	-1.21E-04	14060.	9.45E+09	175.3616	150221.
0.00								
9.6000	-0.00597	134137.	-7403.	-4.30E-05	13211.	9.45E+09	191.5292	153907.
0.00								
10.0000	-0.00602	100819.	-6469.	1.67E-05	12459.	9.45E+09	197.5261	157594.
0.00								
10.4000	-0.00581	71988.	-5527.	6.06E-05	11809.	9.45E+09	195.3283	161280.
0.00								
10.8000	-0.00543	47612.	-4609.	9.09E-05	11259.	9.45E+09	186.7858	164966.
0.00								
11.2000	-0.00494	27508.	-3745.	1.10E-04	10805.	9.45E+09	173.5826	168653.
0.00								
11.6000	-0.00438	11386.	-2951.	1.20E-04	10442.	9.45E+09	157.2113	172339.
0.00								

$$L2=[11,386/((11,386+1,121)/(12.0-11.6))]+11.6-2.82 = 9.1 \text{ FT}$$

0.00	12.0000	-0.00379	-1121.	-2240.	1.23E-04	10210.	9.45E+09	138.9591	176026.
0.00	12.4000	-0.00320	-10426.	-1619.	1.20E-04	10420.	9.45E+09	119.9039	179712.
0.00	12.8000	-0.00264	-16961.	-1089.	1.13E-04	10568.	9.45E+09	100.9189	183398.
0.00	13.2000	-0.00212	-21160.	-647.963	1.03E-04	10662.	9.45E+09	82.6839	187085.
0.00	13.6000	-0.00165	-23441.	-291.839	9.16E-05	10714.	9.45E+09	65.7011	190771.
0.00	14.0000	-0.00124	-24193.	-13.401	7.95E-05	10731.	9.45E+09	50.3147	194458.
0.00	14.4000	-8.90E-04	-23771.	195.5118	6.73E-05	10721.	9.45E+09	36.7322	198144.
0.00	14.8000	-5.96E-04	-22486.	343.7795	5.56E-05	10692.	9.45E+09	25.0460	201830.
0.00	15.2000	-3.56E-04	-20611.	440.5039	4.46E-05	10650.	9.45E+09	15.2558	205517.
0.00	15.6000	-1.67E-04	-18370.	494.6100	3.47E-05	10599.	9.45E+09	7.2884	209203.
0.00	16.0000	-2.29E-05	-15950.	514.5438	2.60E-05	10545.	9.45E+09	1.0174	212890.
0.00	16.4000	8.25E-05	-13496.	508.0565	1.85E-05	10489.	9.45E+09	-3.720	216576.
0.00	16.8000	1.55E-04	-11119.	482.0637	1.23E-05	DEPTH TO FIXITY =[2.29E-05/((2.29E-05+8.25E-05)/(16.4-16))]+16.0=16.1 FT			
0.00	17.2000	2.00E-04	-8899.	442.5704	7.19E-06	10386.	9.45E+09	-9.346	223949.
0.00	17.6000	2.24E-04	-6889.	394.6483	3.18E-06	10340.	9.45E+09	-10.622	227635.
0.00	18.0000	2.31E-04	-5119.	342.4556	1.31E-07	10300.	9.45E+09	-11.125	231322.
0.00	18.4000	2.25E-04	-3602.	289.2895	-2.08E-06	10266.	9.45E+09	-11.028	235008.
0.00	18.8000	2.11E-04	-2336.	237.6602	-3.59E-06	10238.	9.45E+09	-10.485	238694.
0.00	19.2000	1.91E-04	-1311.	189.3799	-4.52E-06	10215.	9.45E+09	-9.632	242381.

19.6000 0.00	1.67E-04	-506.788	145.6595	-4.98E-06	10196.	9.45E+09	-8.585	246067.
20.0000 0.00	1.43E-04	99.9290	107.2066	-5.08E-06	10187.	9.45E+09	-7.437	249754.
20.4000 0.00	1.19E-04	535.2260	74.3209	-4.92E-06	10197.	9.45E+09	-6.265	253440.
20.8000 0.00	9.57E-05	825.8327	46.9836	-4.58E-06	10204.	9.45E+09	-5.125	257126.
21.2000 0.00	7.47E-05	997.8194	24.9386	-4.11E-06	10207.	9.45E+09	-4.060	260813.
21.6000 0.00	5.62E-05	1076.	7.7633	-3.59E-06	10209.	9.45E+09	-3.096	264499.
22.0000 0.00	4.03E-05	1081.	-5.070	-3.04E-06	10209.	9.45E+09	-2.251	268186.
22.4000 0.00	2.70E-05	1035.	-14.145	-2.50E-06	10208.	9.45E+09	-1.530	271872.
22.8000 0.00	1.63E-05	951.9215	-20.060	-2.00E-06	10206.	9.45E+09	-0.934	275558.
23.2000 0.00	7.85E-06	847.0885	-23.397	-1.54E-06	10204.	9.45E+09	-0.457	279245.
23.6000 0.00	1.49E-06	731.1929	-24.704	-1.14E-06	10201.	9.45E+09	-0.08781	282931.
24.0000 0.00	-3.09E-06	612.8054	-24.472	-7.98E-07	10199.	9.45E+09	0.1843	286618.
24.4000 0.00	-6.17E-06	498.2704	-23.135	-5.15E-07	10196.	9.45E+09	0.3730	290304.
24.8000 0.00	-8.03E-06	392.0094	-21.059	-2.89E-07	10194.	9.45E+09	0.4920	293990.
25.2000 0.00	-8.94E-06	296.8330	-18.547	-1.14E-07	10192.	9.45E+09	0.5546	297677.
25.6000 0.00	-9.13E-06	214.2450	-15.840	1.56E-08	10190.	9.45E+09	0.5732	301363.
26.0000 0.00	-8.79E-06	144.7261	-13.123	1.07E-07	10188.	9.45E+09	0.5589	305050.
26.4000 0.00	-8.10E-06	87.9904	-10.531	1.66E-07	10187.	9.45E+09	0.5213	308736.
26.8000 0.00	-7.20E-06	43.2090	-8.155	1.99E-07	10186.	9.45E+09	0.4687	312422.

27.2000 0.00	-6.19E-06	9.1988	-6.051	2.13E-07	10185.	9.45E+09	0.4078	316109.
27.6000 0.00	-5.16E-06	-15.422	-4.248	2.11E-07	10185.	9.45E+09	0.3438	319795.
28.0000 0.00	-4.17E-06	-32.110	-2.748	1.99E-07	10186.	9.45E+09	0.2808	323482.
28.4000 0.00	-3.25E-06	-42.307	-1.542	1.80E-07	10186.	9.45E+09	0.2216	327168.
28.8000 0.00	-2.44E-06	-47.371	-0.607	1.57E-07	10186.	9.45E+09	0.1682	330854.
29.2000 0.00	-1.74E-06	-48.530	0.08819	1.33E-07	10186.	9.45E+09	0.1215	334541.
29.6000 0.00	-1.16E-06	-46.860	0.5766	1.09E-07	10186.	9.45E+09	0.08205	338227.
30.0000 0.00	-7.00E-07	-43.269	0.8932	8.57E-08	10186.	9.45E+09	0.04988	341914.
30.4000 0.00	-3.42E-07	-38.501	1.0720	6.49E-08	10186.	9.45E+09	0.02460	345600.
30.8000 0.00	-7.69E-08	-33.142	1.1444	4.67E-08	10186.	9.45E+09	0.00559	349286.
31.2000 0.00	1.07E-07	-27.632	1.1390	3.13E-08	10186.	9.45E+09	-0.00787	352973.
31.6000 0.00	2.24E-07	-22.287	1.0802	1.86E-08	10185.	9.45E+09	-0.01661	356659.
32.0000 0.00	2.86E-07	-17.309	0.9888	8.56E-09	10185.	9.45E+09	-0.02145	360346.
32.4000 0.00	3.06E-07	-12.815	0.8817	9.08E-10	10185.	9.45E+09	-0.02319	364032.
32.8000 0.00	2.94E-07	-8.848	0.7719	-4.59E-09	10185.	9.45E+09	-0.02256	367718.
33.2000 0.00	2.62E-07	-5.394	0.6692	-8.21E-09	10185.	9.45E+09	-0.02025	371405.
33.6000 0.00	2.16E-07	-2.403	0.5801	-1.02E-08	10185.	9.45E+09	-0.01685	375091.
34.0000 0.00	1.64E-07	0.2011	0.5086	-1.08E-08	10185.	9.45E+09	-0.01293	378778.
34.4000 0.00	1.12E-07	2.5072	0.4561	-1.01E-08	10185.	9.45E+09	-0.00896	382464.

34.8000	6.72E-08	4.6053	0.2330	-8.26E-09	10185.	9.45E+09	-0.08399	6000000.
0.00								
35.2000	3.32E-08	4.7653	-0.06803	-5.88E-09	10185.	9.45E+09	-0.04146	6000000.
0.00								
35.6000	1.08E-08	3.9670	-0.200	-3.66E-09	10185.	9.45E+09	-0.01346	6000000.
0.00								
36.0000	-1.96E-09	2.8559	-0.226	-1.93E-09	10185.	9.45E+09	0.00244	6000000.
0.00								
36.4000	-7.72E-09	1.7993	-0.197	-7.43E-10	10185.	9.45E+09	0.00965	6000000.
0.00								
36.8000	-9.09E-09	0.9638	-0.147	-4.15E-11	10185.	9.45E+09	0.01136	6000000.
0.00								
37.2000	-8.12E-09	0.3895	-0.09524	3.02E-10	10185.	9.45E+09	0.01014	6000000.
0.00								
37.6000	-6.19E-09	0.04871	-0.05233	4.14E-10	10185.	9.45E+09	0.00774	6000000.
0.00								
38.0000	-4.15E-09	-0.114	-0.02132	3.97E-10	10185.	9.45E+09	0.00518	6000000.
0.00								
38.4000	-2.38E-09	-0.157	-0.00175	3.28E-10	10185.	9.45E+09	0.00297	6000000.
0.00								
38.8000	-9.94E-10	-0.132	0.00836	2.55E-10	10185.	9.45E+09	0.00124	6000000.
0.00								
39.2000	6.90E-11	-0.07734	0.01114	2.02E-10	10185.	9.45E+09	-8.63E-05	6000000.
0.00								
39.6000	9.44E-10	-0.02509	0.00810	1.76E-10	10185.	9.45E+09	-0.00118	6000000.
0.00								
40.0000	1.76E-09	0.00	0.00	1.69E-10	10185.	9.45E+09	-0.00220	3000000.
0.00								

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = 0.19000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -1949755. inch-lbs
 Maximum shear force = 74746. lbs
 Depth of maximum bending moment = 0.000000 feet below pile head
 Depth of maximum shear force = 0.000000 feet below pile head
 Number of iterations = 6
 Number of zero deflection points = 6

Summary of Pile-head Responses for Conventional Analyses

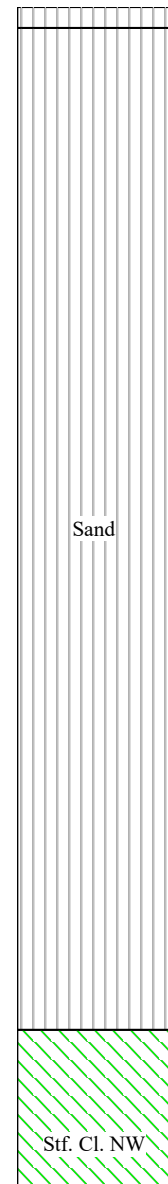
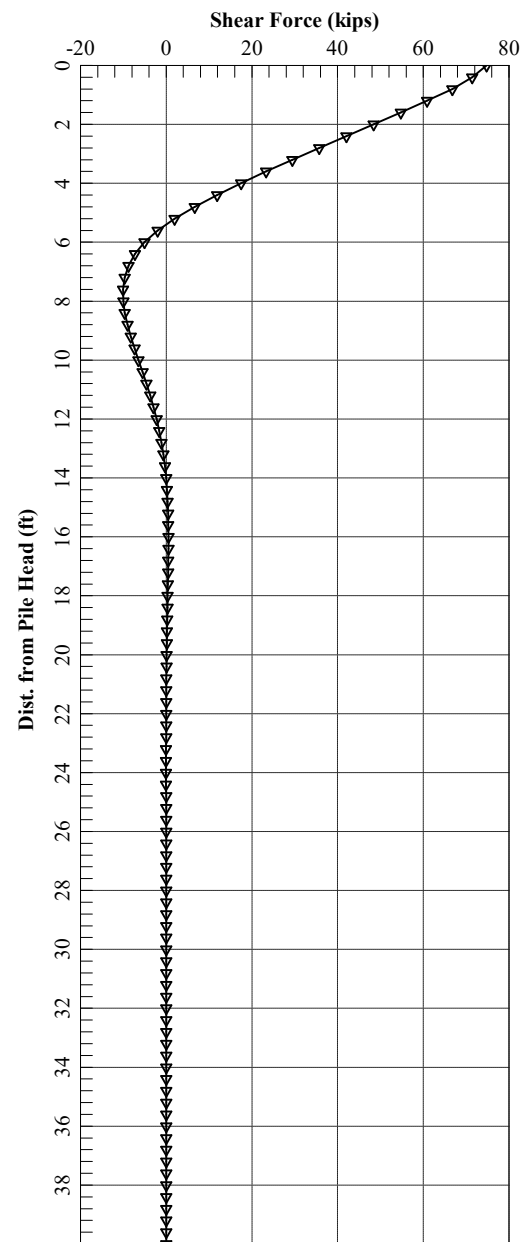
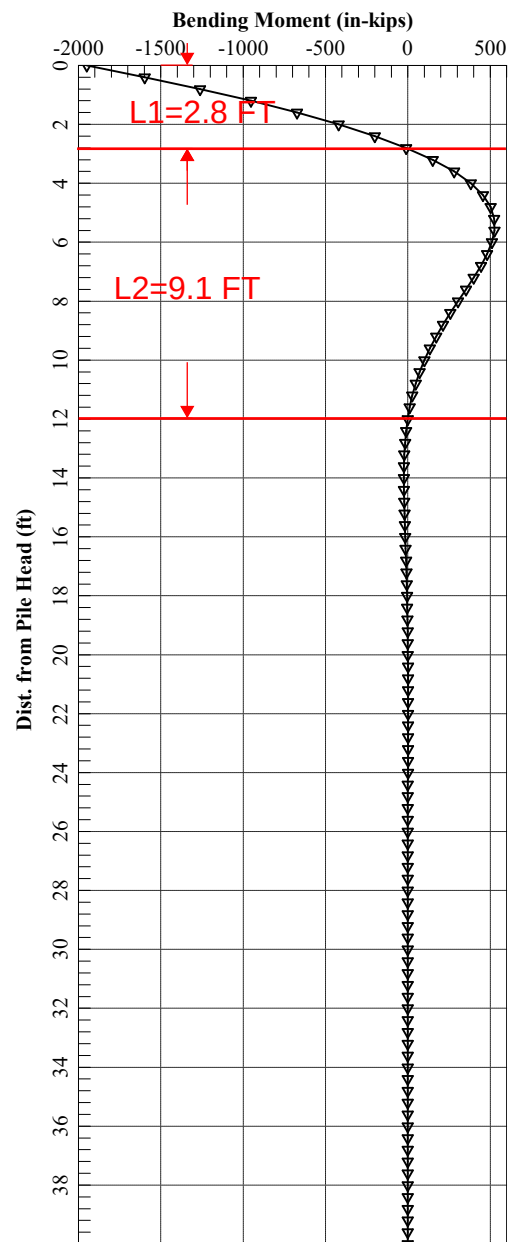
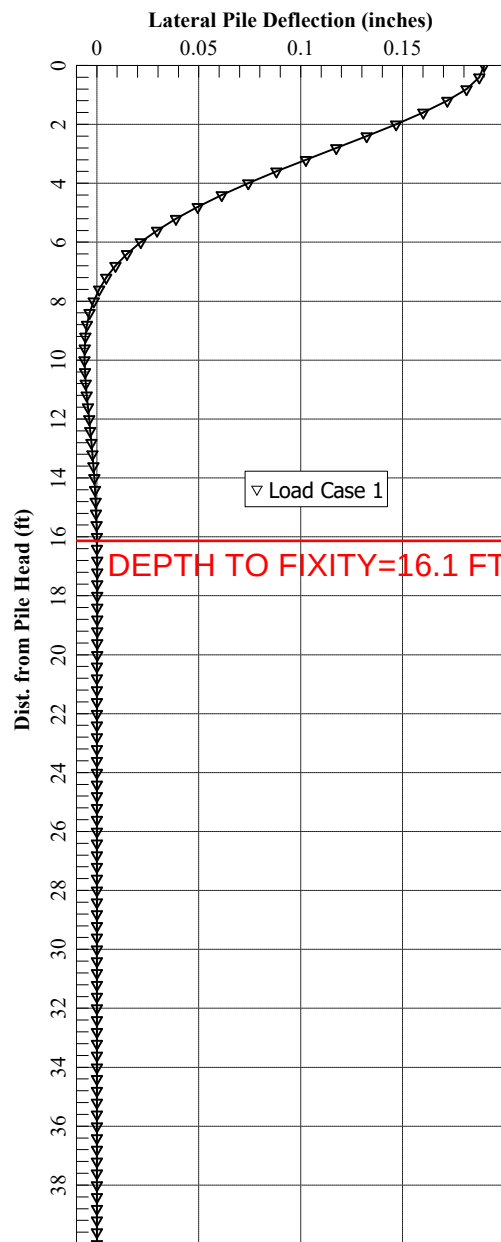
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	0.1900	S, rad	0.00	262890.	0.1900	0.00	74746.	-1949755.

Maximum pile-head deflection = 0.1900000000 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.



Abutment 1 - HP14x89 Contraction

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LFile for Version 2022-12.012

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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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This model was prepared by:
CFE

Haley & Aldrich, Inc.
Portland, ME

Files Used for Analysis

Path to file locations:
\Users\cfernandezescobar\haleyaldrich.com\Maine Department of Transportation - 0210010.MaineDOT-Merrills
Bridge No. 0714\Calculations\Lateral Pile Analyses\F2\

Name of input data file:
2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Contraction Case-F.lp12d

Name of output report file:
2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Contraction Case-F.lp12o

Name of plot output file:

2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Contraction Case-F.lp12p

Name of runtime message file:

2025-0731-HAI-27530 Brownfield Merrills Corner Bridge No 0714 Contraction Case-F.lp12r

Date and Time of Analysis

Date: July 31, 2025

Time: 15:42:52

Problem Title

Project Name: Brownfield Merrills Corner Bridge No 0714

Job Number: 0210010-000

Client: Maine Department of Transportation

Engineer: Camilo Fernandez Escobar

Description: Abutment 1 Contraction Case F

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 40.000 ft
Depth of ground surface below top of pile = -5.7000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.7000
2	40.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 40.000000 ft
Pile width = 13.800000 in

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	-5.70000 ft
Distance from top of pile to bottom of layer	=	-1.70000 ft
Effective unit weight at top of layer	=	125.000000 pcf
Effective unit weight at bottom of layer	=	125.000000 pcf
Friction angle at top of layer	=	34.000000 deg.
Friction angle at bottom of layer	=	34.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	-1.70000 ft
Distance from top of pile to bottom of layer	=	0.700000 ft
Effective unit weight at top of layer	=	62.600000 pcf
Effective unit weight at bottom of layer	=	62.600000 pcf
Friction angle at top of layer	=	34.000000 deg.
Friction angle at bottom of layer	=	34.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 3 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	0.700000 ft
Distance from top of pile to bottom of layer	=	34.700000 ft
Effective unit weight at top of layer	=	67.600000 pcf
Effective unit weight at bottom of layer	=	67.600000 pcf
Friction angle at top of layer	=	40.000000 deg.
Friction angle at bottom of layer	=	40.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 4 is stiff clay without free water

Distance from top of pile to top of layer	=	34.700000 ft
Distance from top of pile to bottom of layer	=	51.500000 ft
Effective unit weight at top of layer	=	67.600000 pcf
Effective unit weight at bottom of layer	=	67.600000 pcf
Undrained cohesion at top of layer	=	4000. psf
Undrained cohesion at bottom of layer	=	4000. psf
Epsilon-50 at top of layer	=	0.005000
Epsilon-50 at bottom of layer	=	0.005000

(Depth of the lowest soil layer extends 11.500 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand (Reese, et al.)	-5.700 -1.700	125.0000 125.0000	-- --	34.0000 34.0000	-- --	160.0000 160.0000
2	Sand (Reese, et al.)	-1.700 0.7000	62.6000 62.6000	-- --	34.0000 34.0000	-- --	160.0000 160.0000
3	Sand (Reese, et al.)	0.7000 34.7000	67.6000 67.6000	-- --	40.0000 40.0000	-- --	160.0000 160.0000
4	Stiff Clay w/o Free Water	34.7000 51.5000	67.6000 67.6000	4000. 4000.	-- --	0.00500 0.00500	-- --

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	5	y = 0.190000 in	S = 0.0000 in/in	262890.	N.A.	

Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	40.000000 ft
Flange Width	=	14.700000 in
Section Depth	=	13.800000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.811550 sq. in.
Moment of Inertia	=	325.837265 in^4
Elastic Bending Stiffness	=	9449281. kip-in^2
Plastic Modulus, Z	=	67.636247in^3
Plastic Moment Capacity = Fy Z	=	3382.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1290.578 kips
Nominal Axial Tensile Capacity	=	-1290.578 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
-----	-----
1	262.890

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 262.890 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000467	44.1580136	9448389.	82.4967557	11.1711919	
0.00000935	88.3160273	9448389.	44.9233779	12.1574084	
0.00001402	132.4740409	9448389.	32.3989186	13.1436248	
0.00001869	176.6320546	9448389.	26.1366889	14.1298416	
0.00002337	220.7900682	9448389.	22.3793511	15.1160581	
0.00002804	264.9480819	9448389.	19.8744593	16.1022745	
0.00003272	309.1060955	9448389.	18.0852508	17.0884911	
0.00003739	353.2641091	9448389.	16.7433445	18.0747077	
0.00004206	397.4221228	9448389.	15.6996395	19.0609242	
0.00004674	441.5801364	9448389.	14.8646756	20.0471408	
0.00005141	485.7381501	9448389.	14.1815232	21.0333573	
0.00005608	529.8961637	9448389.	13.6122296	22.0195739	
0.00006076	574.0541774	9448389.	13.1305197	23.0057904	
0.00006543	618.2121910	9448389.	12.7176254	23.9920069	
0.00007010	662.3702046	9448389.	12.3597837	24.9782235	
0.00007478	706.5282183	9448389.	12.0466722	25.9644401	
0.00007945	750.6862319	9448389.	11.7703974	26.9506567	
0.00008412	794.8442456	9448389.	11.5248198	27.9368732	
0.00008880	839.0022592	9448389.	11.3050924	28.9230897	
0.00009347	883.1602729	9448389.	11.1073378	29.9093063	
0.00009815	927.3182865	9448389.	10.9284169	30.8955228	
0.0001028	971.4763001	9448389.	10.7657616	31.8817393	
0.0001075	1016.	9448389.	10.6172502	32.8679559	
0.0001122	1060.	9448389.	10.4811148	33.8541725	
0.0001168	1104.	9448389.	10.3558702	34.8403890	
0.0001215	1148.	9448389.	10.2402598	35.8266055	
0.0001262	1192.	9448389.	10.1332132	36.8128221	
0.0001309	1236.	9448389.	10.0338127	37.7990386	
0.0001355	1281.	9448389.	9.9412674	38.7852552	
0.0001402	1325.	9448389.	9.8548919	39.7714717	
0.0001449	1369.	9448389.	9.7740889	40.7576883	

0.0001496	1413.	9448389.	9.6983361	41.7439048	
0.0001542	1457.	9448389.	9.6271744	42.7301214	
0.0001589	1501.	9448389.	9.5601987	43.7163379	
0.0001636	1546.	9448389.	9.4970502	44.7025545	
0.0001682	1590.	9448389.	9.4374099	45.6887710	
0.0001729	1634.	9448389.	9.3809934	46.6749876	
0.0001776	1678.	9448389.	9.3275462	47.6612041	
0.0001823	1722.	9448389.	9.2768399	48.6474207	
0.0001916	1810.	9444103.	9.1836346	50.0000000	Y
0.0002010	1892.	9413671.	9.1040650	50.0000000	Y
0.0002103	1969.	9359973.	9.0366409	50.0000000	Y
0.0002197	2041.	9289863.	8.9792465	50.0000000	Y
0.0002290	2109.	9207853.	8.9302973	50.0000000	Y
0.0002384	2173.	9116470.	8.8887068	50.0000000	Y
0.0002477	2234.	9019283.	8.8532041	50.0000000	Y
0.0002570	2292.	8918506.	8.8228475	50.0000000	Y
0.0002664	2348.	8814690.	8.7971493	50.0000000	Y
0.0002757	2402.	8709873.	8.7752772	50.0000000	Y
0.0002851	2453.	8605091.	8.7566709	50.0000000	Y
0.0002944	2502.	8499271.	8.7408580	50.0000000	Y
0.0003038	2548.	8386360.	8.7255994	50.0000000	Y
0.0003131	2589.	8268413.	8.7107431	50.0000000	Y
0.0003225	2627.	8146926.	8.6962287	50.0000000	Y
0.0003318	2662.	8022764.	8.6818801	50.0000000	Y
0.0003412	2694.	7897563.	8.6678004	50.0000000	Y
0.0003505	2724.	7772029.	8.6540976	50.0000000	Y
0.0003599	2752.	7646799.	8.6407892	50.0000000	Y
0.0003692	2778.	7522824.	8.6277163	50.0000000	Y
0.0003786	2801.	7400303.	8.6147148	50.0000000	Y
0.0003879	2824.	7279070.	8.6020602	50.0000000	Y
0.0003973	2844.	7159937.	8.5896959	50.0000000	Y
0.0004066	2864.	7043603.	8.5774181	50.0000000	Y
0.0004160	2882.	6929145.	8.5658335	50.0000000	Y
0.0004253	2899.	6817003.	8.5539351	50.0000000	Y
0.0004346	2915.	6707161.	8.5425572	50.0000000	Y
0.0004440	2931.	6600570.	8.5313795	50.0000000	Y
0.0004533	2945.	6495495.	8.5203278	50.0000000	Y
0.0004627	2958.	6393702.	8.5093716	50.0000000	Y

0.0004720	2971.	6293907.	8.4989667	50.0000000	Y
0.0004814	2983.	6196739.	8.4883460	50.0000000	Y
0.0004907	2994.	6102125.	8.4783181	50.0000000	Y
0.0005001	3005.	6009417.	8.4681420	50.0000000	Y
0.0005094	3016.	5919847.	8.4582437	50.0000000	Y
0.0005188	3025.	5831511.	8.4485653	50.0000000	Y
0.0005281	3035.	5746119.	8.4391279	50.0000000	Y
0.0005375	3043.	5662682.	8.4294944	50.0000000	Y
0.0005468	3052.	5581167.	8.4204915	50.0000000	Y
0.0005562	3060.	5501860.	8.4114652	50.0000000	Y
0.0005935	3089.	5203738.	8.3766669	50.0000000	Y
0.0006309	3113.	4933791.	8.3441988	50.0000000	Y
0.0006683	3133.	4688429.	8.3135805	50.0000000	Y
0.0007057	3151.	4465145.	8.2847838	50.0000000	Y
0.0007431	3166.	4261178.	8.2576413	50.0000000	Y
0.0007805	3180.	4074116.	8.2322547	50.0000000	Y
0.0008179	3192.	3902464.	8.2078951	50.0000000	Y
0.0008553	3202.	3744047.	8.1854265	50.0000000	Y
0.0008927	3211.	3597676.	8.1635425	50.0000000	Y
0.0009300	3220.	3461847.	8.1429727	50.0000000	Y
0.0009674	3227.	3335914.	8.1236089	50.0000000	Y
0.0010048	3234.	3218521.	8.1046560	50.0000000	Y
0.0010422	3240.	3108919.	8.0871266	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	262.8900000000	3240.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	-5.700	0.00	N.A.	No	0.00	15087.
2	-1.700	4.0000	Yes	No	15087.	22454.
3	0.7000	5.5333	Yes	No	37541.	5729859.
4	34.7000	135.4704	No	No	5767400.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection

for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)		
Displacement of pile head	=	0.190000 inches
Rotation of pile head	=	0.000E+00 radians
Axial load on pile head	=	262890.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
0.00	0.1900	-1699368.	59322.	0.00	48518.	9.45E+09	-432.735	5466.	
0.4000	0.1879	-1419484.	57088.	-7.92E-04	42205.	9.45E+09	-461.560	11789.	
0.8000	0.1824	-1149325.	54035.	-0.00144	36111.	9.45E+09	-810.443	21328.	
1.2000	0.1741	-897102.	50019.	-0.00196	30421.	9.45E+09	-862.959	23798.	
1.6000	0.1635	-664186.	45772.	-0.00236	25167.	9.45E+09	-906.553	26609.	
2.0000	0.1514	-451732.	41339.	-0.00264	20375.	9.45E+09	-940.352	29815.	
2.4000	0.1381	-260653.	36770.	-0.00283	16065.	9.45E+09	-963.646	33482.	
2.8000	0.1243	-91610.	32115.	-0.00292	12251.	9.45E+09	-975.900	37696.	
3.2000	0.1102	55007.	27429.	-0.00292	11426.	9.45E+09	-976.768	42560.	
3.6000	0.09619	179084.	22766.	-0.00286	14225.	L1=[91,610/((91,610+55007)/(3.2-2.8))]+2.8 = 3.0 FT			
4.0000	0.08266	280788.	18182.	-0.00275	16519.	9.45E+09	-943.938	54814.	

0.00	4.4000	0.06981	360563.	13731.	-0.00259	18318.	9.45E+09	-910.517	62604.
0.00	4.8000	0.05784	419128.	9467.	-0.00239	19639.	9.45E+09	-866.240	71884.
0.00	5.2000	0.04690	457467.	5440.	-0.00216	20504.	9.45E+09	-811.654	83077.
0.00	5.6000	0.03706	476812.	1698.	-0.00193	20941.	9.45E+09	-747.398	96791.
0.00	6.0000	0.02840	478631.	-1627.	-0.00168	20982.	9.45E+09	-637.880	107827.
0.00	6.4000	0.02089	465447.	-4323.	-0.00144	20684.	9.45E+09	-485.411	111514.
0.00	6.8000	0.01453	440780.	-6324.	-0.00121	20128.	9.45E+09	-348.659	115200.
0.00	7.2000	0.00924	407798.	-7710.	-9.99E-04	19384.	9.45E+09	-228.751	118886.
0.00	7.6000	0.00494	369284.	-8562.	-8.01E-04	18515.	9.45E+09	-126.107	122573.
0.00	8.0000	0.00154	327628.	-8962.	-6.24E-04	17575.	9.45E+09	-40.550	126259.
0.00	8.4000	-0.00106	284827.	-8990.	-4.69E-04	16610.	9.45E+09	28.5958	129946.
0.00	8.8000	-0.00296	242503.	-8724.	-3.35E-04	15655.	9.45E+09	82.3959	133632.
0.00	9.2000	-0.00427	201922.	-8233.	-2.22E-04	14740.	9.45E+09	122.2022	137318.
0.00	9.6000	-0.00509	164026.	-7581.	-1.29E-04	13885.	9.45E+09	149.5594	141005.
0.00	10.0000	-0.00551	129472.	-6823.	-5.45E-05	13106.	9.45E+09	166.1184	144691.
0.00	10.4000	-0.00561	98661.	-6008.	3.41E-06	12411.	9.45E+09	173.5626	148378.
0.00	10.8000	-0.00548	71787.	-5175.	4.67E-05	11804.	9.45E+09	173.5445	152064.
0.00	11.2000	-0.00517	48865.	-4356.	7.74E-05	11287.	9.45E+09	167.6364	155750.
0.00	11.6000	-0.00474	29773.	-3576.	9.73E-05	10857.	9.45E+09	157.2915	159437.

0.00	12.0000	-0.00423	14287.	-2854.	1.09E-04	10507.	9.45E+09	143.8174	163123.
0.00	12.4000	-0.00369	2105.	-2200.	1.13E-04	10232.	9.45E+09	128.3592	166810.
0.00	12.8000	-0.00315	-7120.	-1624.	1.11E-04	10346.	9.45E+09	111.8918	170496.
0.00	13.2000	-0.00262	-13764.	-1127.	1.06E-04	10495	L2=[2,105/((2,105+7,120)/(12.8-12.4))]+12.4-3.0 = 9.4 FT		
0.00	13.6000	-0.00213	-18204.	-708.582	9.80E-05	10596.	9.45E+09	78.9831	177869.
0.00	14.0000	-0.00168	-20813.	-366.216	8.81E-05	10654.	9.45E+09	63.6690	181555.
0.00	14.4000	-0.00129	-21942.	-94.310	7.72E-05	10680.	9.45E+09	49.6253	185242.
0.00	14.8000	-9.42E-04	-21914.	113.7761	6.61E-05	10679.	9.45E+09	37.0772	188928.
0.00	15.2000	-6.52E-04	-21017.	265.5096	5.52E-05	10659.	9.45E+09	26.1451	192614.
0.00	15.6000	-4.12E-04	-19504.	368.7286	4.49E-05	10625.	9.45E+09	16.8628	196301.
0.00	16.0000	-2.21E-04	-17590.	431.2665	3.55E-05	10582.	9.45E+09	9.1946	199987.
0.00	16.4000	-7.19E-05	-15453.	460.6588	2.71E-05	10534.	9.45E+09	3.0522	203674.
0.00	16.8000	3.91E-05	-13236.	463.9259	1.98E-05	10484.	9.45E+09	-1.691	207360.
0.00	17.2000	1.18E-04	-11049.	447.4226	1.36E-05	104	DEPTH TO FIXITY =[7.19E-05/((7.19E-05+3.91E-05)/(16.8-16.4))]+16.4=16.7 FT		
0.00	17.6000	1.70E-04	-8975.	416.7480	8.52E-06	10387.	9.45E+09	-7.596	214733.
0.00	18.0000	2.00E-04	-7070.	376.7036	4.45E-06	10344.	9.45E+09	-9.090	218419.
0.00	18.4000	2.12E-04	-5370.	331.2924	1.29E-06	10306.	9.45E+09	-9.832	222106.
0.00	18.8000	2.12E-04	-3893.	283.7502	-1.07E-06	10273.	9.45E+09	-9.977	225792.
0.00	19.2000	2.02E-04	-2643.	236.5990	-2.73E-06	10245.	9.45E+09	-9.669	229478.

19.6000 0.00	1.86E-04	-1615.	191.7173	-3.81E-06	10221.	9.45E+09	-9.032	233165.
20.0000 0.00	1.66E-04	-793.291	150.4194	-4.42E-06	10203.	9.45E+09	-8.176	236851.
20.4000 0.00	1.44E-04	-159.629	113.5391	-4.66E-06	10189.	9.45E+09	-7.191	240538.
20.8000 0.00	1.21E-04	308.4492	81.5128	-4.62E-06	10192.	9.45E+09	-6.153	244224.
21.2000 0.00	9.91E-05	634.5639	54.4600	-4.38E-06	10199.	9.45E+09	-5.119	247910.
21.6000 0.00	7.88E-05	842.3303	32.2563	-4.01E-06	10204.	9.45E+09	-4.133	251597.
22.0000 0.00	6.06E-05	954.3429	14.6001	-3.55E-06	10207.	9.45E+09	-3.224	255283.
22.4000 0.00	4.47E-05	991.4581	1.0698	-3.06E-06	10207.	9.45E+09	-2.413	258970.
22.8000 0.00	3.13E-05	972.3320	-8.828	-2.56E-06	10207.	9.45E+09	-1.711	262656.
23.2000 0.00	2.02E-05	913.1714	-15.618	-2.08E-06	10206.	9.45E+09	-1.119	266342.
23.6000 0.00	1.13E-05	827.6539	-19.826	-1.64E-06	10204.	9.45E+09	-0.635	270029.
24.0000 0.00	4.43E-06	726.9811	-21.955	-1.24E-06	10201.	9.45E+09	-0.252	273715.
24.4000 0.00	-6.57E-07	620.0263	-22.470	-9.02E-07	10199.	9.45E+09	0.03799	277402.
24.8000 0.00	-4.23E-06	513.5494	-21.784	-6.14E-07	10197.	9.45E+09	0.2477	281088.
25.2000 0.00	-6.55E-06	412.4497	-20.257	-3.79E-07	10194.	9.45E+09	0.3886	284774.
25.6000 0.00	-7.86E-06	320.0379	-18.190	-1.92E-07	10192.	9.45E+09	0.4726	288461.
26.0000 0.00	-8.40E-06	238.3085	-15.830	-5.06E-08	10190.	9.45E+09	0.5111	292147.
26.4000 0.00	-8.35E-06	168.2014	-13.368	5.26E-08	10189.	9.45E+09	0.5146	295834.
26.8000 0.00	-7.89E-06	109.8425	-10.951	1.23E-07	10187.	9.45E+09	0.4924	299520.

27.2000 0.00	-7.17E-06	62.7590	-8.683	1.67E-07	10186.	9.45E+09	0.4527	303206.
27.6000 0.00	-6.29E-06	26.0649	-6.632	1.90E-07	10186.	9.45E+09	0.4020	306893.
28.0000 0.00	-5.35E-06	-1.384	-4.837	1.96E-07	10185.	9.45E+09	0.3459	310579.
28.4000 0.00	-4.41E-06	-20.863	-3.314	1.90E-07	10185.	9.45E+09	0.2885	314266.
28.8000 0.00	-3.52E-06	-33.681	-2.062	1.76E-07	10186.	9.45E+09	0.2331	317952.
29.2000 0.00	-2.71E-06	-41.107	-1.067	1.57E-07	10186.	9.45E+09	0.1818	321638.
29.6000 0.00	-2.01E-06	-44.318	-0.304	1.36E-07	10186.	9.45E+09	0.1361	325325.
30.0000 0.00	-1.41E-06	-44.365	0.2548	1.13E-07	10186.	9.45E+09	0.09666	329011.
30.4000 0.00	-9.21E-07	-42.158	0.6399	9.12E-08	10186.	9.45E+09	0.06383	332698.
30.8000 0.00	-5.34E-07	-38.452	0.8830	7.07E-08	10186.	9.45E+09	0.03745	336384.
31.2000 0.00	-2.42E-07	-33.859	1.0140	5.24E-08	10186.	9.45E+09	0.01713	340070.
31.6000 0.00	-3.17E-08	-28.850	1.0606	3.64E-08	10186.	9.45E+09	0.00227	343757.
32.0000 0.00	1.08E-07	-23.770	1.0472	2.31E-08	10186.	9.45E+09	-0.00783	347443.
32.4000 0.00	1.90E-07	-18.855	0.9951	1.23E-08	10185.	9.45E+09	-0.01389	351130.
32.8000 0.00	2.26E-07	-14.247	0.9217	3.84E-09	10185.	9.45E+09	-0.01669	354816.
33.2000 0.00	2.27E-07	-10.016	0.8410	-2.32E-09	10185.	9.45E+09	-0.01694	358502.
33.6000 0.00	2.03E-07	-6.168	0.7635	-6.43E-09	10185.	9.45E+09	-0.01535	362189.
34.0000 0.00	1.65E-07	-2.670	0.6965	-8.67E-09	10185.	9.45E+09	-0.01259	365875.
34.4000 0.00	1.20E-07	0.5399	0.6440	-9.21E-09	10185.	9.45E+09	-0.00926	369562.

34.8000	7.66E-08	3.5362	0.3919	-8.18E-09	10185.	9.45E+09	-0.09580	6000000.
0.00								
35.2000	4.17E-08	4.3228	0.03689	-6.18E-09	10185.	9.45E+09	-0.05211	6000000.
0.00								
35.6000	1.73E-08	3.9059	-0.140	-4.09E-09	10185.	9.45E+09	-0.02160	6000000.
0.00								
36.0000	2.40E-09	2.9888	-0.199	-2.34E-09	10185.	9.45E+09	-0.00300	6000000.
0.00								
36.4000	-5.20E-09	2.0007	-0.191	-1.07E-09	10185.	9.45E+09	0.00650	6000000.
0.00								
36.8000	-7.92E-09	1.1611	-0.151	-2.71E-10	10185.	9.45E+09	0.00989	6000000.
0.00								
37.2000	-7.80E-09	0.5486	-0.104	1.63E-10	10185.	9.45E+09	0.00975	6000000.
0.00								
37.6000	-6.35E-09	0.1606	-0.06172	3.43E-10	10185.	9.45E+09	0.00794	6000000.
0.00								
38.0000	-4.51E-09	-0.04470	-0.02914	3.73E-10	10185.	9.45E+09	0.00563	6000000.
0.00								
38.4000	-2.77E-09	-0.120	-0.00730	3.31E-10	10185.	9.45E+09	0.00347	6000000.
0.00								
38.8000	-1.33E-09	-0.116	0.00502	2.71E-10	10185.	9.45E+09	0.00167	6000000.
0.00								
39.2000	-1.73E-10	-0.07266	0.00953	2.23E-10	10185.	9.45E+09	2.16E-04	6000000.
0.00								
39.6000	8.09E-10	-0.02469	0.00762	1.98E-10	10185.	9.45E+09	-0.00101	6000000.
0.00								
40.0000	1.73E-09	0.00	0.00	1.92E-10	10185.	9.45E+09	-0.00216	3000000.
0.00								

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = 0.19000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -1699368. inch-lbs
 Maximum shear force = 59322. lbs
 Depth of maximum bending moment = 0.000000 feet below pile head
 Depth of maximum shear force = 0.000000 feet below pile head
 Number of iterations = 6
 Number of zero deflection points = 6

Summary of Pile-head Responses for Conventional Analyses

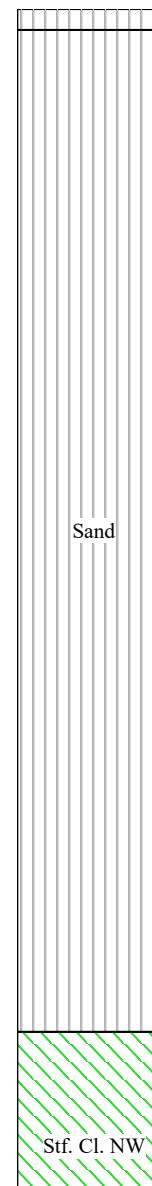
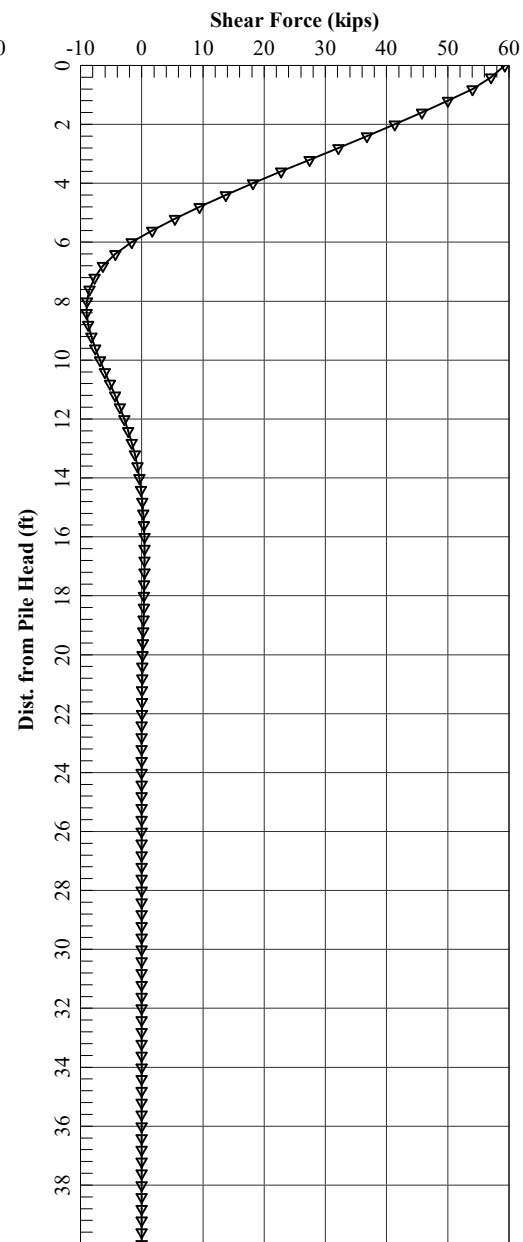
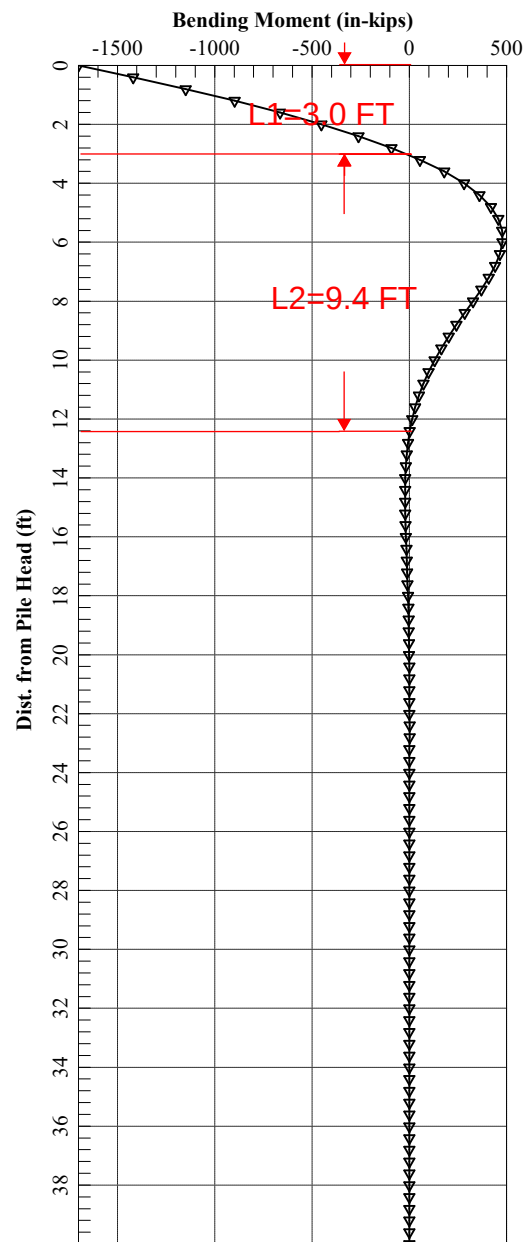
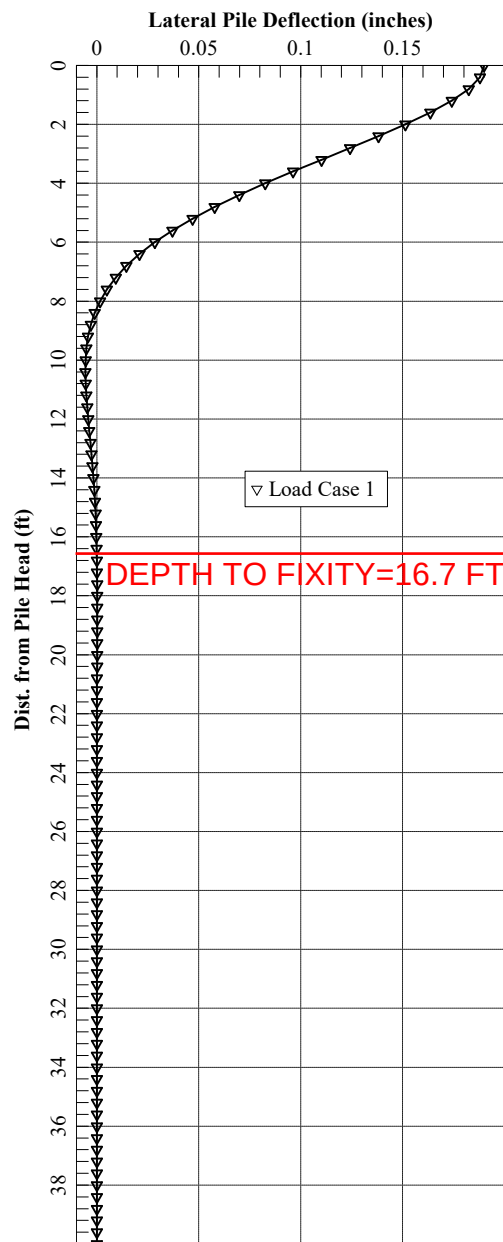
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	0.1900	S, rad	0.00	262890.	0.1900	0.00	59322.	-1699368.

Maximum pile-head deflection = 0.1900000000 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.



Lateral Earth Pressures

Client: Maine Department of Transportation

Project: Merrills Corner Bridge No. 0714, MaineDOT WIN 027530.00, Brownfield, Maine

Subject: Active Lateral Earth Pressure Coefficients for Abutments and Wingwalls

PROBLEM STATEMENT & OBJECTIVE

Calculate static active earth pressure coefficients for abutment breastwalls and wingwalls.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.
2. MaineDOT Bridge Design Guide, August 2003, with interim revisions through June 2018.
3. Drawings prepared by MaineDOT.

ASSUMPTIONS

1. Abutment breastwalls and wingwalls are backfilled with Granular Borrow.
2. Free draining material used to backfill walls and drains are provided (i.e., no hydrostatic pressures develop).

SOIL PROPERTIES AND WALL GEOMETRY

designates input cell

Total Unit Weight, γ (pcf) =	125	pcf	Soil Type 4, BDG Table 3-3	
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, BDG Table 3-3	
Backslope Angle, β =	0	26.6	29.7	degrees (horiz., 2H:1V, 1.75H:1V)
Backface of Wall Angle to Horizontal, α =	90	degrees	per MaineDOT drawings	
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, BDG Table 3-3	

RANKINE STATIC ACTIVE LATERAL EARTH PRESSURE COEFFICIENT (K_a)

$$K_a = \cos \beta \cdot \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

MaineDOT BDG Section 3.6.5.2

$$K_{a, \text{Rankine}} = \begin{matrix} 0.31 & 0.46 & 0.56 \end{matrix}$$

COULOMB STATIC ACTIVE LATERAL EARTH PRESSURE COEFFICIENT (K_a)

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \cdot \sin(\alpha - \delta) \cdot \left(1 + \frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)} \right)^2}$$

MaineDOT BDG Section 3.6.6

$$K_{a, \text{Coulomb}} = \begin{matrix} 0.27 & 0.46 & 0.54 \end{matrix}$$

Frost Penetration

Client: Maine Department of Transportation

Date: 16-Sep-2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Frost Penetration Depth Evaluation

Checked by: EMH

OBJECTIVE:

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

REFERENCES:

1. MaineDOT Bridge Design Guide, August 2003, with June 2018 updates.
2. Boring logs BB-BSR-101, BB-BSR-101A, and BB-BSR-102.
3. Laboratory testing performed by the MaineDOT Laboratory.
4. Abutment elevations from interpretive subsurface profile provided by MaineDOT dated 9/4/2024.

EVALUATION:

1. Gather relevant information from soil samples taken near proposed bridge abutment locations:

Substructure (Wingwall El.)	Boring No.	Sample No.	Depth (ft)	Sample Elevation	USCS	Moisture Content (%)
Abutment No. 1 (El. 465.7)	BB-BSR-101	1D	0.5 to 2.5	472.4	SM	14.5
	BB-BSR-101	2D	4 to 6	468.9	SC-SM	21.0
	BB-BSR-101	3D	6 to 8	466.9	ML	44.2
	<i>BB-BSR-101</i>	<i>4D</i>	<i>8 to 10</i>	<i>464.9</i>	<i>SM</i>	<i>14.6</i>
	<i>BB-BSR-101</i>	<i>5D</i>	<i>10 to 12</i>	<i>462.9</i>	<i>SM</i>	<i>14.9</i>
	<i>BB-BSR-101</i>	<i>8D</i>	<i>19 to 21</i>	<i>453.9</i>	<i>SM</i>	<i>10.2</i>
Abutment No. 2 (El. 464.6)	BB-BSR-102	1D & 2D	0 to 4	471.0	SW-SM	2.8
	<i>BB-BSR-102</i>	<i>3D & 4D</i>	<i>10 to 14</i>	<i>461.0</i>	<i>SW-SM</i>	<i>9.3</i>
	<i>BB-BSR-102</i>	<i>5D</i>	<i>16 to 18</i>	<i>456.0</i>	<i>SM</i>	<i>10.8</i>

Note: Samples BB-BSR-101 (1D, 2D, 3D) and BB-BSR-102 (1D, 2D) are above the wingwall bearing elevations and were not considered. Samples considered are italicized above.

2. Per MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1380°F - days.

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

4. For coarse-grained soil with water content of approx. 12 percent (average of five moisture content results from above), per Table 5-1, maximum depth of frost is approximately 6.5 ft (78 in.).

Recommend abutment breastwall and wingwall footings be founded at least 6.5 ft below the lowest adjacent ground surface exposed to freezing.

Client: Maine Department of Transportation

Date: 16-Sep-2024

Project: Merrills Corner Bridge No. 0714 Over Shepards River - MaineDOT WIN 027530.00

Computed by: TPJ

Subject: Frost Penetration Depth Evaluation

Checked by: EMH

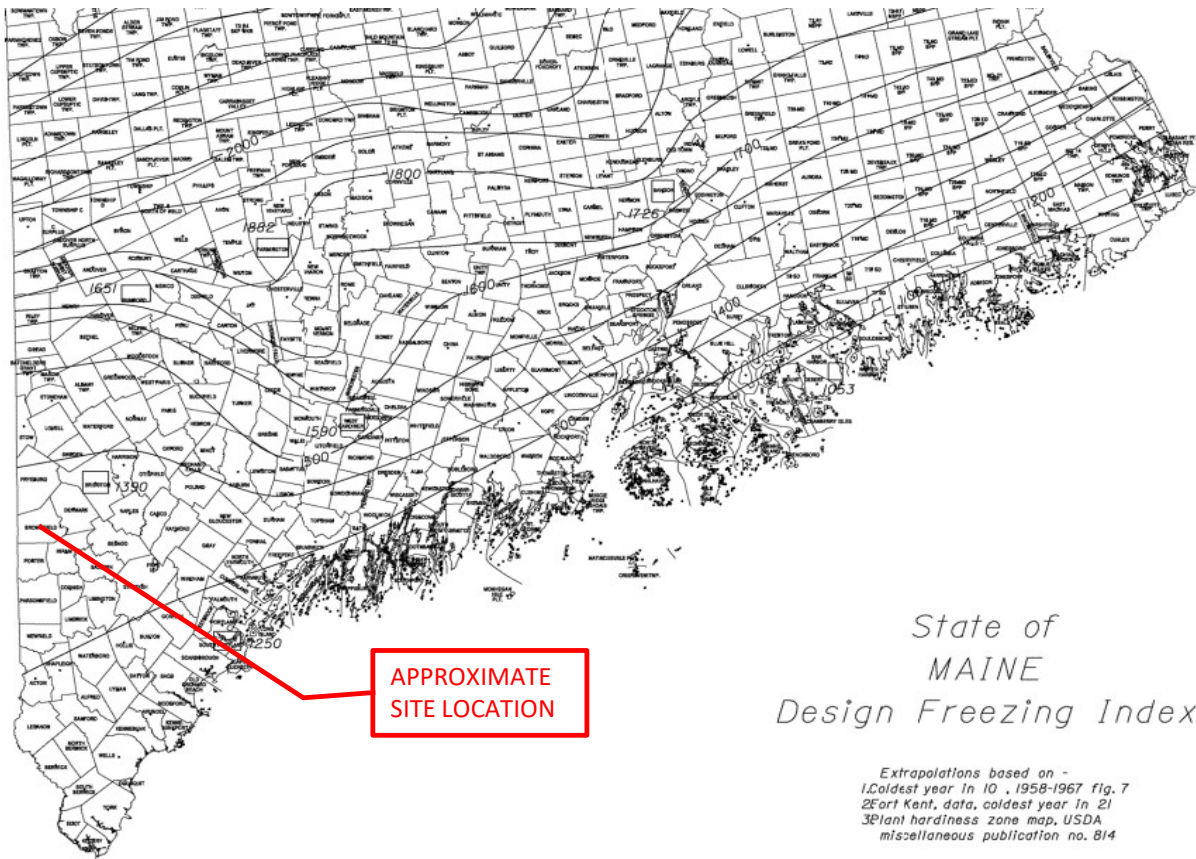


Table 5-1 Depth of Frost Penetration

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