REPORT

Geotechnical Design Report (GDR)

Smith Brook Bridge No. 3189 Crossing Smith Brook, Bancroft Township, Maine (WIN 028262.00)

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1.0 INTRODUCTION AND PROJECT BACKGROUND

This Geotechnical Design Report (GDR) summarizes the results of WSP's geotechnical investigation and recommendations for the replacement of Smith Brook Bridge No. 3189 Crossing Smith Brook in Bancroft Township, Maine (WIN 028262.00). The location of the existing culvert is shown in Figure 1 (attached). The purpose of this GDR is to present soil and bedrock information for the proposed bridge replacement site based on the results of subsurface investigations and laboratory tests; present recommended geotechnical parameters for the proposed replacement bridge; and provide preliminary geotechnical design recommendations for construction of the replacement bridge.

Bridge #3189 carries Bancroft Road over Smith Brook. The existing structure consists of a single corrugated steel sectional plate pipe culvert constructed in 1963. The proposed replacement design provided in the Draft PS&E Plans[1] is a 90-foot long, single-span, integral abutment bridge supported by H-piles driven to bedrock.

2.0 GEOLOGIC SETTING

2.1 Regional Surficial Geology

The site is located within the Mattawamkeag River Valley in the Wytopitlock Quadrangle. Surficial materials are characterized by glacial stream deposits overlying glacial till.[2] The glacial-stream deposits consist of well sorted, loose coarse sand and cobbly gravel that may include minor glacial till and is commonly overlapped or entirely buried by glacial marine deposits in the coastal lowland with moderate to high permeability. The glacial till consists of a heterogeneous mixture of sand, silt, clay, and gravel to cobble clasts of granite derived from north-northwest plutons. Two glacial till varieties are mapped in the area: basal till, which is fine grained and very compact, with low permeability and poor drainage; and ablation till, which is loose, sandy, and stony, with moderate permeability and fair to good drainage. Stratification is rare in the glacial till. The unit generally overlies bedrock but may overlie or include sand and gravel.[3]

2.2 Regional Bedrock Geology

Bedrock at the site is mapped as metamorphic unit including unnamed metalimestone, shale, conglomerate metasiltstone and slate[4]. The bedrock was stratified as deriving from the Lower Silurian to middle Ordovician age sequence.[5,6] To the southeast of the site location, the bedrock is mapped as Belle Lake Slate which is a metamorphic rock derived from a pelitic protolith. This Upper Ordovician (post-Arenigian) aged rock is described as having a primary mineralogical composition of granite. It is also marked in the Greenschist facies zone of Second Devonian Metamorphism as well as an Ordovician mafic and minor felsic volcanic rock.[4]

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1 Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Bridge 3189. Draft PS&E Plans dated April 22, 2026, 36 pp.

2 Newman, William A., 1979, Preliminary report on the surficial geology of the Sherman, Mattawamkeag Lake, and the northern half of the Mattawamkeag and Wytopitlock [15-minute] quadrangles, Maine: Maine Geological Survey, Open-File Report 79-21, 3 p., scale 1:62,500. *Maine Geological Survey Publications*. 150. http://digitalmaine.com/mgs_publications/150

3 Newman, William A. and Holland, William R., 1986, Reconnaissance surficial geology of the Wytopitlock [15-minute] quadrangle, Maine: Maine Geological Survey, Open-File Map 86-27, map, scale 1:62,500. *Maine Geological Survey Maps*. 755. http://digitalmaine.com/mgs_maps/755

4 Larrabee, D.M., Spencer, C.W., and Swift, D.J.P., 1965, Bedrock geology of the Grand Lake area, Aroostook, Hancock, Penobscot, and Washington Counties, Maine, Contributions to general geology, 1964, Bulletin 1201-E, U.S. Geological Survey.

5 Osberg, P.H., Hussey, A.M., and Boone, G.M., 1985, Bedrock geologic map of Maine. Maine Geological Survey, Geologic Map Series BGMM.

6 Warner, J., Doyle, R.G, Hussey II, A.M., 1967, Generalized map of regional metamorphic zones, Maine Geological survey.

3.0 SUBSURFACE INVESTIGATIONS

WSP completed two (2) borings (BB-BSB-101 and BB-BSB-102) between March 20 and March 24, 2025, to collect subsurface geotechnical information. WSP surveyed the as-drilled boring locations following completion of the drilling program. Boring location coordinates and ground surface elevations are summarized in Table 1 (attached) and boring locations with respect to existing site features are illustrated on the Boring Location Plan (BLP) shown in Figure 2 (attached). A WSP field engineer monitored drilling activities, selected sampling intervals, logged subsurface conditions encountered, and obtained soil samples for use in visual descriptions and subsequent laboratory testing and classification.

WSP subcontracted Seaboard Drilling, LLC (Seaboard) of Bangor, Maine to complete the borings. Seaboard used a Diedrich D-50 Turbo drill rig and solid stem auger (SSA) and cased wash boring methods. Borings BB-BSB-101 and BB-BSB-102 were advanced using 4-inch solid stem augers (SSA) to 10 feet below ground surface (bgs), after which they were advanced using a 4-inch diameter casing in 5-foot lengths installed with an automatic safety hammer. Boring BB-BSB-101 was advanced beyond 35 feet bgs by telescoping 3-inch diameter casing through the 4-inch diameter casing to bedrock. Soil in the casing was washed out using a roller bit and water to the depth where samples were subsequently collected. Borings BB-BSB-101 and BB-BSB-102 were advanced to refusal at depths of 56 feet and 50 feet bgs, respectively, after which 10 feet and 12 feet of rock core was collected, respectively.

Seaboard performed Standard Penetration Testing (SPT) using a calibrated automatic hammer system and standard 2-inch split spoon sampler in general accordance with American Society for Testing and Materials (ASTM) D1586 for both borings. Sampling of overburden materials was conducted at approximately 2-foot intervals in the upper 9 feet of the ground surface in an open, uncased hole and at 5-foot intervals thereafter. Seaboard drove the split spoon sampler 24 inches with a 140-pound hammer dropped 30 inches and WSP recorded the number of hammer blows required to advance the split spoon sampler through each 6-inch increment. Measured uncorrected N-values, calculated as the sum of the hammer blows to advance the sampler during the 6-inch to 12-inch and 12-inch to 18-inch intervals are provided in the boring logs in Appendix A. WSP used the average calibrated hammer energy transfer ratio of 100% provided by Seaboard[7] to convert the measured N-values to N_{60} values for further calculations. Soil samples were collected and stored in sealed glass jars for later evaluation and laboratory testing.

Seaboard collected rock core using NQ2 size (1.98-inch diameter) diamond tipped core barrels following refusal of either the casing, split spoon sampler, or roller bit to additional advance within the borehole. Rock core was placed in wooden boxes and transported to the WSP office. WSP recorded the lithology, Total Core Recovery (TCR), Rock Quality Designation (RQD), and coring rates for each core run in the boring logs provided in Appendix A. A detailed summary of the rock core quality parameters for the recovered rock core is presented in Table 2 (attached) and photographs of the rock core are presented in Appendix B.

The boring logs provided in Appendix A present details of the sampling methods used, field data obtained, and soil and rock conditions encountered during the investigation. Soils were field categorized in general accordance with ASTM D2488. WSP field engineer characterized the bedrock lithology, and the descriptions were revised in the office. A description of the boring log symbols and terms used for the soil and rock descriptions precedes the boring logs in Appendix A.

[7] 2024PA00175 - Seaboard - SPT Energy Calibration, prepared by GRL Engineers Inc., dated 10/23/2024.

4.0 LABORATORY TESTING PROGRAM

GeoTesting Express, LLC tested the collected soil samples in accordance with applicable American Association of State Highway and Transportation Officials (AASHTO) and American Society for Testing and Materials (ASTM) testing procedures. WSP selected representative SPT split spoon soil samples from each of the borings for geotechnical laboratory testing to assist in soil classification. The types and numbers of each of the laboratory tests conducted on soil samples are presented in Table 4-1. Measured index and classification test results from soil samples are summarized in Table 2 (attached) and include AASHTO and ASTM Unified Soil Classification System (USCS) classifications. Soil testing results are included on the boring logs in Appendix A. Complete laboratory testing results are provided in Appendix C.

Table 4-1: Type and Number of Laboratory Tests Performed

Soil Laboratory Test	Test Standard	No. Tests Completed
Grain Size Analysis (sieve only)	ASTM D6913 AASHTO T 88	9
Grain Size Analysis (sieve and hydrometer)	ASTM D6913, ASTM D7929 AASHTO T 88	2
Moisture Content	ASTM D2216 AASHTO T 265	10

5.0 SUBSURFACE CONDITIONS

The soils encountered in the borings generally include fill materials placed during construction of the existing culvert and roadway and naturally occurring Alluvial Sand and Gravel over Alluvial Silt, which is underlain by fine to coarse gravel with medium to coarse sand interpreted as Glacial Till that contains intermittent cobbles. The boring logs in Appendix A provide detailed descriptions of the soil conditions encountered in the borings. WSP's Interpretive Subsurface Profile (ISP) along the centerline of the roadway is presented in Figure 3 (attached). The following descriptions summarize the major stratigraphic units from the existing ground surface to depth. Table 5-1 summarizes the major stratigraphic units from the existing ground surface to depth.

Table 5-1: Summary of Encountered Subsurface Soils and Fill Materials

Material	Approximate Thickness (feet)	Generalized Description	Classification[1]	N_{60}[2]
Asphalt Pavement	0.5	N/A	N/A	N/A
Fill	14.5 to 14.6	SILT, trace to little sand, trace to little gravel	A-4 SM, ML	17 to 77 (medium dense to very dense)
Alluvial Sand and Gravel	5.0 to 13.4	SAND, some gravel, some silt, trace clay (3.2 foot boulder in BB-BSB-101)	A-1-a GW	13 to 25 (medium dense)

Material	Approximate Thickness (feet)	Generalized Description	Classification[1]	N_{60}[2]
Alluvial Silt	12.0 to 20.1	SILT, trace to little sand, trace gravel	A-4 ML	3 to 25 (soft to very stiff)
Glacial Till	7.5 to 18.0	GRAVEL, little to some sand, trace silt, intermittent cobbles	A-1-a GW	25 to 173 (medium dense to very dense)
Weathered Bedrock (BB-BSA-101)	1	N/A	N/A	N/A

Notes: [1]Classification based on AASHTO M 145 (A-1 through A-7) or ASTM D2487 for Unified Soil Classification System (USCS) or ASTM D2488 for Visual-Manual Classification. [2]SPT N-value corrected for hammer efficiency.

Bedrock: Bedrock was encountered in borings between 50 feet bgs (Elevation (EL) 300.3 feet) and 56 feet bgs (EL 295.3 feet). The bedrock observed consists of medium strong (R3) to strong (R4), fresh (W1) to moderately weathered (W3) metalimestone from the Lower Silurian to middle Ordovician age sequence. Rock Quality Designation (RQD) ranged from very poor to fair (0% to 18%) and estimated Rock Mass Rating (RMR) ranged from 30 to 53. Table 2 (attached) provides detailed information about recovery, RQD, RMR, and descriptions of lithology, rock mass, and discontinuities.

Groundwater: Groundwater levels were measured the borings upon their completion and prior to removal of the casing. Measured groundwater levels ranged from 19.3 feet bgs (EL 332.0 feet) and 24.3 feet bgs (EL 325.9 feet). Smith Brook water levels at the time the borings were performed (March 2025) were estimated to be 15 feet to 20 feet below the roadway surface. Per the Draft PS&E plans[1], the river level associated with ordinary high water ($Q_{1.1}$) at the existing bridge is EL 332.9 feet and the river level associated with design Q_{50} is EL 340.7 feet. The interpreted groundwater levels shown on the ISP (Figure 3) are based on these water level measurements and moisture contents (e.g., dry, moist, wet) observed in the soil samples collected from the borings. In general, groundwater measurements made during drilling are influenced by the addition of water used during the drilling process and the measurements may not be representative of a stabilized water level. Groundwater levels also fluctuate due to other natural variations, such as seasonal/climatic influences, precipitation, barometric pressure, and temperature, and due to other variations, such as construction and groundwater pumping.

6.0 GEOTECHNICAL ANALYSIS AND RECOMMENDATIONS

Based on the Draft PS&E plans[1], the proposed abutments will be supported by H-piles driven to bedrock. Abutment 1 will be located at Station (Sta.) 2+70.00 with the base at EL 340.2 feet and a distance between the base of the abutment and bottom of the approach slab of approximately 6.8 feet. Bedrock elevation is approximately at EL 295.3 feet at Abutment 1; thus H-piles will be driven to bedrock with anticipated pile lengths of approximately 44.9 feet. Abutment 2 will be located at Sta. 3+60.03 with the base at EL 339.7 feet and a distance between the base of the abutment and bottom of the approach slab of approximately 6.8 feet. Bedrock elevation is approximately at EL 300.3 feet at Abutment 2; thus H-piles will be driven to bedrock with anticipated pile lengths of approximately 39.9 feet. The proposed abutment locations are shown on the BLP (Figure 2, attached) and the ISP (Figure 3, attached).

WSP's design is based on both the 60% PS&E plans[8] and Draft PS&E plans[1], as noted in the subsequent discussions. Our analyses and recommendations are described in the subsections below. Calculations, references, and assumptions are provided in Appendix D.

6.1 Design Parameters

WSP evaluated the subsurface geotechnical data collected and developed geotechnical material design parameters to serve as a basis for calculations, analyses, and evaluations discussed herein. Table 6-1 provides a summary of design parameters for each soil layer encountered (refer to Figure 3, attached) and material properties for fills as identified in the MaineDOT Standard Specifications.[9] These design parameters served as a basis for calculations, analyses, and evaluations discussed herein.

Table 6-1: Engineering Design Parameters for In Situ Soils and Fills

Soil Layer		γ (pcf)	ϕ (°)	$^1K_0$
In Situ Materials	Asphalt	140	N/A	N/A
	Existing Fill	125	36	0.41
	Alluvial Sand and Gravel	125	31	0.48
	Alluvial Silt	115	31	0.48
	Glacial Till	130	36	0.41
Construction Materials	Asphalt	140	N/A	N/A
	Granular Borrow (Section 703.19)	125	32	0.47
	Aggregate Subbase (Section 703.06)	135	36	0.41
	Heavy Riprap (Section 703.28)	140	42	N/A

Note: [1]Lateral earth pressure coefficient at rest was determined using the equation $K_0 = 1\text{-}\sin\phi'$

6.2 Frost Penetration Depth

WSP estimated an average frost penetration depth for roadway and bridge abutment fills using the MaineDOT Bridge Design Guide (BDG)[10]. The site has an air design freezing index of approximately 2,000 °F-degree days. For roadway and bridge abutment coarse-grained fills with an approximate average water content of 11.4%, we estimate a frost penetration depth of 7.7 feet at the site. Refer to calculation in Appendix D.

MaineDOT BDG[10] Section 5.2.1 indicates that pile-supported integral abutments should be embedded a minimum depth of 4.0 feet for frost protection. Additionally, Section 5.3.4.3 indicates that footings shall be placed below the depth of frost penetration, and that rip rap is not considered frost protection. The base of the proposed bridge

8 Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Bridge 3189. 60% PS&E Plans dated February 4, 2026, 31pp.

9 State of Maine, Department of Transportation. March 2026. Standard Specifications. https://www.maine.gov/dot/sites/maine.gov.dot/files/inline-files/MaineDOT_Standard_Specifications_March_2026.pdf

10 Maine Department of Transportation, Bridge Design Guide, 2003 updated 2018. https://www.maine.gov/dot/sites/maine.gov.dot/files/inline-files/Complete2003BDGwithUpdatesto2018_0_0.pdf

abutments is approximately 10 feet laterally from the face of the riprap, resulting in a foundation base depth greater than the frost penetration depth. However, we note that the proposed 3-foot thick riprap layer on the abutment foreslope will have large voids and allow cold air and frost to penetrate more deeply toward the base of the abutment.

6.3 Seismic Site Classification

WSP analyzed the seismic site classification following the criteria in Article C3.4.2.2 of AASHTO[11] for the soils near proposed Abutment 1 and Abutment 2. The results indicate the site is Site Class DE at Abutment 1 and Site Class D at Abutment 2. WSP determined Seismic Zone per Article 3.5 of AASHTO.[11] The one-second period response acceleration at the site, S_{D1}, is less than 0.15g; therefore, the site is in Seismic Zone 1. The slope adjusted peak ground acceleration, k_{av}, is determined using methodology described in FHWA GEC 3[12] that equates it to the product of the slope height reduction factor, α, and the site adjusted peak ground acceleration (PGA), k_{max}. The site adjusted PGA, k_{max}, is derived by multiplying the USGS mapped acceleration coefficient and the AASHTO PGA factor. The slope reduction factor, α, is derived using an empirical relationship described in FHWA GEC 3[12] that uses the slope height and a function of its shape labelled as β. Table 6-2 provides additional seismic parameters determined using AASHTO[11] for the site. Refer to the full methodology of the analysis, calculations, and locations in Appendix D.

Table 6-2: Seismic Site Parameters

Abutment	A_s	k_{av}	S_s	S_1	S_{D1}	Site Class	SDC
Abutment 1	0.093	0.082	0.218	0.090	0.090	DE	A
Abutment 2	0.095	0.084	0.233	0.093	0.093	D	A

Notes: A_s is the zero period response seismic coefficient; S_S is the short period response seismic coefficient (0.2 seconds); S_1 is the long period response seismic coefficient (1.0 seconds); S_{D1} is the equivalent response spectral acceleration coefficient at 1.0 seconds.

6.4 Global Stability Analyses

6.4.1 Global Stability Modeling – Longitudinal at Abutments

WSP evaluated global stability longitudinal to the centerline of Bancroft Road between Sta. 1+00 and Sta. 6+00 that includes the proposed abutments, abutment foreslopes, and Smith Brook. The embankment geometry is from the existing and proposed grades shown in the 60% PS&E plans.[8] These analyses incorporated material design parameters estimated from SPT N_{60} values and laboratory test data. For the longitudinal sections at Abutment 1 and Abutment 2, we evaluated the factors of safety of potential failure surfaces for two water level scenarios extending across the entire slope including below the proposed abutments and approach slabs: 1) headwater elevation $Q_{1.1}$ (1.1 year return period) at EL 332.9 feet, and 2) headwater elevation Q_{50} (50 year return period) at EL 340.7 feet. We assume the Smith Brook water levels will extend into the embankment because the subsurface

[11] American Association of State Highway and Transportation Officials (2023). "AASHTO Guide Specifications for LRFD Seismic Bridge Design," 3rd Ed, October 2023, Washington DC.

[12] FHWA. 2011. Geotechnical Engineering Circular No. 3 - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations Reference Manual, Publication No. FHWA-NHI-11-032.

materials are predominantly coarse-grained with a permeability that will likely allow water to flow freely into and out of the voids when flood water levels increase and recede.

Section 11.6.3.7 of AASHTO LRFD[13] indicates that global stability of earth slopes with and without foundations should use an appropriate resistance factor, where ϕ = 0.75 (FS = 1.3) may be used where geotechnical parameters (e.g., stratigraphy and strength) are well defined and ϕ = 0.65 (FS = 1.5) should be used where geotechnical parameters are based on limited information or the subsurface conditions are highly variable. WSP's subsurface investigation consisted of one boring behind each proposed bridge abutment, where soil strength in the existing embankment fill, and alluvial sand and gravel and alluvial silts was determined using correlations to limited numbers of SPT N-values corrected for hammer efficiency and rod depth. Based on this limited subsurface information behind the proposed abutments, we recommend a project-specific minimum FS of 1.5 for the embankments with abutments.

For the foreslopes between the abutments and Smith Brook, we recommend using a minimum FS of 1.3 as a minimum thickness of 4 feet of Riprap is planned that will provide surface stability to the slopes and the water levels within the streambed will provide additional resistance.

FHWA GEC 3[12] recommends a minimum FS of 1.1 for pseudo-static seismic conditions, and our analyses use FS = 1.1 for all scenarios. The pseudo-static seismic analyses are based on FHWA GEC 3[12], which recommends using 50% of the peak average acceleration of the potential failure mass, k_{max}, discussed in Section 6.3.

We performed the global stability analyses using the two-dimensional limit equilibrium modeling software *Slide2* by Rocscience[14] for the proposed static and pseudo-static seismic conditions. We analyzed global stability using the Spencer method with auto-refined search for circular failure surfaces and the Spencer Method with a Cuckoo search plus surface altering optimization for non-circular failure surfaces. The search algorithm that produced a lower factor of safety was selected and the analyses show that non-circular surfaces govern potential slope failures. Refer to the full methodology of the analysis, material properties, and calculations in Appendix D.

6.4.2 Global Stability Results – Longitudinal at Abutments

Results are summarized in Table 6-3 for both the static conditions and pseudo-static seismic conditions for the proposed geometry longitudinal to the roadway centerline for the $Q_{1.1}$ and Q_{50} Smith Brook water levels.

Table 6-3: Global Stability Factors of Safety – Longitudinal at Proposed Abutments and Foreslopes

Condition	Location	Factor of Safety Water Table at $Q_{1.1}$ (EL 332.9 feet)	Factor of Safety Water Table at Q_{50} (EL 340.7 feet)
Static	Abutment 1 (Non-Circular)	1.32	1.28
	Abutment 2 (Non-Circular)	1.40	1.37
	Abutment 1 Foreslope (Non-Circular)	1.45	1.50
	Abutment 2 Foreslope (Non-Circular)	1.45	1.50

13 American Association of State Highway and Transportation Officials (2024). "LRFD Bridge Design Specifications," 10th Ed.

14 Rocscience Inc. Slide2 software package, Version 9.031, build date November 8, 2023.

Condition	Location	Factor of Safety Water Table at $Q_{1.1}$ (EL 332.9 feet)	Factor of Safety Water Table at Q_{50} (EL 340.7 feet)
Pseudo-static seismic	Abutment 1 (Non-Circular)	1.19	1.16
	Abutment 2 (Non-Circular)	1.28	1.25
	Abutment 1 Foreslope (Non-Circular)	1.29	1.33
	Abutment 2 Foreslope (Non-Circular)	1.30	1.35

Note: Minimum factor of safety (FS) under static loading conditions is FS = 1.5 for embankments with potential failure surfaces that extend below structures and a minimum FS = 1.3 for potential failure surfaces for embankments alone. The minimum FS under pseudo-static seismic loading conditions is 1.1 for all scenarios. Scenarios in red provided minimum factors of safety below the proposed minimum value and need to be improved.

Results show that the lower bound factors of safety do not meet the minimum FS of 1.5 under static conditions for potential failure surfaces that extend below both Abutment 1 and Abutment 2 and their approach slabs under static conditions when Granular Borrow is used as abutment backfill material for both design water levels evaluated. These low factors of safety are due to the geometry and low strength soils below the existing embankment fills and need to be improved.

Results show the lower bound factors of safety exceed the minimum FS of 1.1 under pseudo-static conditions for potential failure surfaces that extend below Abutment 1 and Abutment 2. Results show that lower bound factors of safety for the abutment foreslopes as designed exceed the minimum FS of 1.5 under static and minimum FS of 1.1 under pseudo-static conditions.

To improve the factors of safety, we elected to include the proposed H-piles, HP14x89, into the global stability modeling to augment the resistance of soils below the abutments to potential slope failures. H-Pile resistance was incorporated using the Slide2 support type “Pile/Micro Pile” applied to the center of the of the base of each abutment footings at the proposed bottom of abutment elevation. The force application was defined as “Active” alongside an out-of-plane spacing of 7.715 ft taken from the pile calculations (discussed in Section 6.6). A pile shear strength of 24,600 lbs was used. This represents 10% of the calculated factored shear resistance of 246,000 lbs, as use of the full pile shear strength created stiffness discontinuities in the stability models. Updated factors of safety for global stability that incorporate the H-piles are summarized in Table 6-4.

Results show that the lower bound factors of safety exceed the recommended FS of 1.5 under static conditions for potential failure surfaces that extend below Abutment 1 and Abutment 2 and their approach slabs under static conditions when Granular Borrow and pile shear are used as abutment backfill material for both design water levels evaluated.

Table 6-4: Global Stability Factors of Safety – Longitudinal at Proposed Abutments Inclusive of Piles

Condition	Location	Factor of Safety Water Table at $Q_{1.1}$ (EL 332.9 feet)	Factor of Safety Water Table at Q_{50} (EL 340.7 feet)
Static	Abutment 1 (Non-Circular)	1.55	1.53
	Abutment 2 (Non-Circular)	1.57	1.58

Condition	Location	Factor of Safety Water Table at $Q_{1.1}$ (EL 332.9 feet)	Factor of Safety Water Table at Q_{50} (EL 340.7 feet)
Pseudo-static seismic	Abutment 1 (Non-Circular)	1.40	1.36
	Abutment 2 (Non-Circular)	1.42	1.40

Note: Minimum factor of safety (FS) under static loading conditions is FS = 1.5 for embankments with potential failure surfaces that extend below structures and a minimum FS of 1.3 for potential failure surfaces for embankments alone. The minimum FS under pseudo-static seismic loading conditions is 1.1 for all scenarios.

6.4.3 Global Stability for Approach Embankments

The profile for the embankment fills below the roadway in the 60% PS&E plans[8] shows the following:

- No substantial grade change at the roadway surface that would increase embankment loading within in situ soils at and near the bridge location.
- No substantial embankment widening or slope changes along the project extents, where slopes appear to be the same maximum 2H:1V grade on the embankments.

Our proposed minimum factor of safety (FS) of 1.3 for global stability for approach embankments correlates to the AASHTO LRFD[13] recommended earth slope global stability resistance factor of ϕ = 0.75 for slopes with and without foundations when geotechnical parameters (e.g., stratigraphy and strength) are well defined, as these embankments have been in place since their construction and slope failures are not evident.

The proposed 2H:1V slopes are comprised of embankment fills underlain by alluvial sands and gravels and alluvial silts, and bedrock. We assume the strengths of the existing materials are adequate to support minor changes to the existing roadway slopes described above. We do not anticipate minimum global stability failure surfaces that extend into the in situ soils to have FS < 1.3.

Shallow portions of the proposed 2H:1V slopes will be grubbed and infilled to the PDR design slopes with granular borrow that has a design unit weight of 125 pcf and a design friction angle of $\phi = 32^{o}$ provided in the MaineDOT Bridge Design Guide[10]. The 2H:1V embankment slope angle of 26.5° is less than the angle of internal friction of the embankment fill material (32°); however, this combination of friction angle and slope angle typically results in calculated surficial FS < 1.3. Based on discussions with MaineDOT, we understand that slopes constructed with granular borrow using methods commensurate with MaineDOT Standard Specifications[9] have historically performed adequately.

6.5 Lateral Earth Pressure at Bridge Abutments

Per the Draft PS&E Plans[1], proposed Abutment 1 is approximately 56.5 feet wide, 3.8 feet thick, and founded at EL 340.2 feet. The bottom of the approach slab is 6.8 feet above the base of the abutment. Proposed Abutment 2 is approximately 53.8 feet wide, 3.8 feet thick, and founded at EL 339.7 feet. The bottom of the approach slab is 6.8 feet above the base of the abutment. WSP used the bridge design team’s preliminary estimate of abutment movement of 0.14 inches from thermal expansion to analyze lateral earth pressure on the abutments between the bottom of the approach slab and base of the abutment.

Under longitudinal expansion, the abutments will be subject to passive earth pressure. Under longitudinal contraction, the abutments will be subject to active earth pressure. MaineDOT allows MassDOT[15] guidance to be used to determine the passive earth pressure coefficient when it is less than the maximum passive earth pressure coefficient identified in AASHTO LRFD[13] for full passive pressures. Per AASHTO[13] Table C3.11.1-1, MassDOT LRFD Bridge Manual[15] Figure 3.10.8-1, the abutment height provided, and the anticipated thermal expansion, the maximum estimated wall rotation is 0.0019, which is less than 0.02 that MassDOT[15] specifies is required to surpass to develop full passive earth pressure. When the MassDOT[15] guidance yields a K_p value less than K_p determined by Rankine theory, MaineDOT prefers the use of K_p determined using Rankine theory. Thus, we evaluated K_p using both methods. For this case, the Rankine earth pressure coefficient was greater than the MassDOT coefficient, which was used to determine active lateral earth pressure (assuming level backfill and no frictional interaction between the abutment wall and the backfill).

Engineering parameters and calculated earth pressure coefficients for the backfill material are presented in Table 6-5. We estimate that the resultant force generated from passive earth pressure using the Rankine method, the governing scenario, at Abutment 1 is an unfactored load of 9,406 pounds per foot of abutment length and acts at EL 342.5, at Abutment 2 the resultant force is an unfactored load of 9,406 pounds per foot of abutment length and acts at EL 342.0feet. These values assume the fill is free draining (i.e., no water pressure is allowed to build up behind the abutments).

Per the PS&E plans[1] , wingwalls are proposed to be in line, and cast monolithically, with the abutments. In accordance with the VTrans Integral Abutment Bridge Design Guidelines[16], we recommend the wingwalls be designed using the K_p passive lateral earth pressure coefficient provided in Table 6-5 as the wingwalls will move in concert with the abutments. Refer to the full methodology of the analysis, material properties, and calculations in Appendix D.

Table 6-5: Recommended Lateral Earth Pressure Coefficients at the Proposed Abutments

Abutment Fill	γ (pcf)	ϕ (°)	^1K_a	^2K_p	$^3K_0$
Granular Borrow (Section 703.19)	125	32	0.31	3.25	0.47

Notes: [1]Active earth pressure, K_a, was determined using Rankine Method.; [2]Passive earth pressure, K_p, was determined using the Rankine Method, [3]At-rest lateral earth pressure coefficient, K_0, was determined as $K_0 = 1\text{-sin } \phi$.

6.6 Proposed Bridge Abutment Pile Foundations

The bridge design team provided preliminary factored Strength I loads for the piles at the abutments, assuming five piles per abutment. We performed the pile design using these loads, which consisted of a Strength I factored axial load of 363 kips per pile for both Abutment 1 and Abutment 2. The bridge design team also provided expected lateral movement and rotation at the integral abutments, consisting of a total thermal expansion of 0.14 inches with 0.010 radians of rotation, and total thermal contraction of 0.39 inches with 0.0 radians of rotation. We evaluated two loading scenarios in the lateral analyses consisting of the proposed factored axial load and contraction displacement, and the proposed factored axial load, thermal displacement, and rotation. We assumed

[15] MassDOT LRFD Bridge Manual - Part 1, January 2020 Revision (https://www.mass.gov/doc/chapter-3-lrfd-bridge-design-guidelines/download)

[16] Vermont Agency of Transportation (VTrans) 2008 Integral Abutment Bridge Design Guidelines, 2nd Ed. https://vtrans.vermont.gov/sites/aot/files/highway/documents/structures/SEI-08-004-1.pdf

steel HP 14x89 piles driven to bedrock. Abutment 1 is expected to be supported by piles with a minimum length of 44.9 feet based on BB-BSB-101, Abutment 2 is expected to be supported by piles with a minimum length of 39.9 feet based on BB-BSA-102.

WSP evaluated the pile foundations under two conditions: 1) the "normal" condition, represented by fully intact embankments and underlying construction fills and in situ soils, and 2) the scour condition, represented by unbraced pile lengths to account for potential scour that may occur under extreme conditions. The Preliminary Design Report (PDR)[17] provided maximum scour depths below the thalweg of Smith Brook of 29.0 feet for Q_{100} and 33.6 feet at Q_{500}, both of which would have caused scour of the stream to below the interpreted bedrock elevation and are impractical for pile design for scour. We assumed a more realistic scour elevation of EL 318.0 feet based on the base elevation of the existing scour pool at the outlet of the existing culvert near the inlet to the Mattawamkeag River. For the scoured condition, the piles were modeled with an unbraced length of pile from the bottom of the proposed abutments (EL. 340.2 feet, per 60% PS&E plans[8]) to EL 318.0 feet for an unbraced pile length of 22.2 feet.

The following describes WSP's axial resistance analyses and lateral response of the abutment piles due to abutment deflection. WSP's pile evaluation did not include the following:

- Downdrag loading: Per AASHTO LRFD[13] Article 3.11.8, downdrag loads on piles can be assumed to be fully developed when settlement in the soils surrounding the piles is 0.4 inches or greater. We anticipate the grade change of approximately 1-foot or less and the replacement of embankment fills with granular borrow behind the proposed abutments will result in minimal settlements in the existing predominantly coarse-grained soils at the proposed abutments. Thus, we did not evaluate potential downdrag loading for these piles.
- Section loss from corrosion: We did not consider section less from corrosion associated with road salts.
- Change in loading due to scour: We evaluated the piles in the scour condition using the loading provided by the bridge design team for the proposed conditions and did not use an alternate loading where soil was removed from behind the abutments. In the scour condition, it is likely that the lateral earth pressure would be reduced by some unknown amount, however high water levels and velocities and debris may contribute to other loading. Thus, for conservatism, we did not alter the proposed loading at the abutments and piles for the scoured condition.

6.6.1 Axial Pile Resistance

WSP analyzed the nominal structural and geotechnical pile resistance of HP 14x89 piles at the abutments following the design procedures outlined in AASHTO LRFD[13] and MaineDOT Bridge Design Guide.[10] Since the piles will be driven to hard rock, the nominal resistance of the piles will be controlled by the structural limit state in accordance with AASHTO LRFD[13] Article 10.7.3.2.3 or the drivability resistance, whichever is less.

The factored pile structural resistance, P_r, was calculated for the piles using the results of the LPILE analysis outlined in Section 6.6.2 and resistance factors of ϕ_c = 0.70 for combined axial and bending loading and ϕ_c = 0.50 for axial compression in the lower segment of the pile based on the potential for hard driving conditions.

[17] Preliminary Design Report, Smith Brook Bridge #3189 over Smith Brook, Bancroft TWP, Maine, WIN 28262.00, prepared by Hoyle Tanner & Associates, Dated 8/15/2025.

As outlined in AASHTO LRFD[13] Article 10.7.3.2.3, the nominal axial geotechnical resistance of piles driven to point bearing on hard rock should not exceed the nominal structural resistance values obtained from AASHTO LRFD[13] Article 6.9.4.1 with a resistance factor of $\phi_c = 0.50$, for severe driving conditions applied. As such, the controlling geotechnical pile resistance is equal to the structural pile resistance.

We performed drivability analyses using GRLWEAP[18] software to determine that the required pile resistance can be achieved at the abutments considering locally available diesel hammers. Per the MaineDOT Standard Specification[9] Section 501.042, the driving stresses were limited to 45 ksi (90% of the pile steel yield stress) and the number of hammer blows at the required resistance was limited to between 3 and 15 blows per inch. The nominal pile driving resistance was calculated using a resistance factor $\phi_{dyn} = 0.65$ for driving criteria that will be established by dynamic testing of at least two piles per site condition or 2% of the production piles. The maximum applied factored axial pile loads should not exceed the governing axial pile resistance shown in Table 6-6.

Table 6-6: Strength Limit State Factored Axial Pile Resistance

Abutment	Factored Structural Resistance[1] $\phi_c = 0.50$ (kips)	Factored Geotechnical Resistance[2] $\phi_c = 0.50$ (kips)	Nominal Pile Driving Resistance[3] $\phi_{dyn} = 0.65$ (kips)	Governing Axial Pile Resistance (kips)
Abutment 1	653	653	565	565
Abutment 2	653	653	565	565

Notes: [1]The displayed structural resistance is the minimum of the factored resistance in the top segment of the pile ($P_{r.top}$), the factored resistance in the second segment of the pile ($P_{r.2nd}$), and the factored resistance in the bottom segment of the pile ($P_{r.bottom}$). [2]Per AASHTO LRFD Article 10.7.3.2.3, the displayed controlling geotechnical resistance is equal to the factored structural resistance in the bottom segment of the pile ($P_{r.bottom}$). [3]The nominal pile driving resistance is based on design axial load divided by the dynamic resistance factor $\Phi_{dyn} = 0.65$.

6.6.2 Lateral Pile Resistance

Lateral response of the abutment piles was evaluated using LPile[19] analysis software. The input parameters were developed based on layer response models defined by the LPile software, laboratory test results from site soils, correlations to soil properties determined from the field investigations, correlations to soil properties identified in the FB-MultiPier user manual[20], and standard properties provided in the MaineDOT Bridge Design Guide[10]. The input parameters are summarized in Appendix D.

The piles were modeled for lateral response in the weak axis assuming factored pile loads for two load cases and an HP 14x89 pile size: Load case 1 consists of 0.39 inches of lateral deflection from bridge contraction; and Load case 2 consists of 0.14 inches of lateral deflection from bridge expansion and 0.01 radians of rotation. An axial load of 367 kips was applied at Abutment 1 and Abutment 2 inclusive of the estimated load of 363 kips in Strength

[18] GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.

[19] Ensoft Inc. (2022). LPile, version 2022.12.07.

[20] Bridge Software Institute. "FB-MultiPier Soil Parameter Table (US Customary Units)." Accessed on 4/10/2020. <https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf>

I provided by the bridge designers and the pile weight. The pile-head-to-abutment connection was assumed to be fixed, where maximum deflection is at the pile cap and there is no lateral deflection at the pile tip. Results of the Abutment 1 and Abutment 2 L-Pile first iteration analyses are summarized in Table 6-7.

Table 6-7: LPile Lateral Analysis Results – First Iteration

Abutment	Pile Length (feet)	Axial Load Analyzed (kips)	Lateral Deflection (inch)	L-Pile Moment at Pile Head (in-kips)	Plastic Hinge Forms	Plastic Hinge Moment (in-kips)
Abutment 1 Load Case 2 Normal	44.9	367	0.14	2392	Yes	1631
Abutment 1 Load Case 2 Scour	44.9	367	0.14	781	No	-
Abutment 2 Load Case 2 Normal	39.9	367	0.14	2411	Yes	1633
Abutment 2 Load Case 2 Scour	39.9	367	0.14	835	No	-

For Abutment 1, WSP analyzed a pile length of 44.9 feet. The piles achieve fixity with no lateral deflection at the pile tip for both load cases. Load case 2 produces the maximum moment and was used for the subsequent analysis.

- For load case 1 (Strength I load case and a maximum lateral deflection of 0.39 inches in the pile cap), a maximum moment of 139 kip-feet occurs in the piles evaluated for the normal condition and a maximum moment of 13.7 kip-feet occurs in the piles evaluated for the scour condition.
- For load case 2 (Strength I load case and a maximum lateral deflection of 0.14 inches in the pile cap), a maximum moment of 199 kip-feet occurs in the piles for the normal condition and a maximum moment of 65.1 kip-feet occurs in the piles for the scour condition.

For Abutment 2, WSP analyzed a pile length of 39.9 feet. The piles achieve fixity with no lateral deflection at the pile tip for both load cases. Load case 2 produces the maximum moment and was used for the subsequent analysis.

- For load case 1 (Strength I load case and a maximum lateral deflection of 0.39 inches in the pile cap), a maximum moment of 141 kip-feet occurs in the piles for the normal condition and a maximum moment of 14.9 kip-feet occurs in the piles for the scour condition.
- For load case 2 (Strength I load case and a maximum lateral deflection of 0.14 inches in the pile cap), a maximum moment of 201 kip-feet occurs in the piles for the normal condition, and a maximum moment of 69.5 kip-feet occurs in the piles for scour condition.

For the normal condition, a plastic hinge forms at the top of the pile at both abutments, and thus we assume the pile head will enter plastic deformation. We performed a second LPile iteration using the same displacement and axial load conditions as the first iteration but set the pile head moment equal to the plastic hinge moment predicted in the first iteration. For the scour condition, moments are lower at the top of the pile and a plastic hinge did not form.

Results of the LPile second iteration analyses are summarized in Table 6-8 and indicate that the demand ratio for combined bending is less than 1.0 in pile segment 2. Table 6-9 shows the structural demand ratios for the first and second segments from the first iteration (with scour) and indicates the combined ratio for bending is less than 1.0 in the second segment of the piles. Results show that, with the exception of the plastic hinge location, the piles remain within the elastic range over the remainder of their length and are stable against buckling. The results indicate that the nominal structural resistance in pile segment 1 is sufficient to support the loads analyzed.

Table 6-8: LPile Lateral Analysis Results – Second Iteration for Normal Condition

Location	Axial Load Analyzed (kips)	Lateral Deflection (inch)	L-Pile Plastic Hinge Moment at Pile Head (in-kips)	Structural Resistance Demand Ratio Segment 1[1]	Structural Resistance Demand Ratio Segment 2[1]	Combined Axial & Bending Demand Ratio Segment 2[2]
Abutment 1 Load Case 2 Normal	367	0.14	1631	0.53	0.45	0.48
Abutment 2 Load Case 2 Normal	367	0.14	1633	0.53	0.45	0.48

Notes: [1]The displayed structural resistance demand ratio is the maximum of the ratio in the top segment of the pile ($P_u/P_{r.top}$), the ratio in the second segment of the pile ($P_u/P_{r.2nd}$), and the ratio in the bottom segment of the pile ($P_u/P_{r.bottom}$). The ratio should be less than 1.0 to avoid pile failure and greater than 0.20 to avoid using an unnecessarily large pile, as recommended in the MaineDOT BDG (page 5-43). [2]The combined axial and bending ratio should be less than 1.0, as per AASHTO LRFD Article 6.9.2.2.1

Table 6-9: LPile Lateral Analysis Results – First Iteration for Scour Condition

Location	Axial Load Analyzed (kips)	Lateral Deflection (inch)	Structural Resistance Demand Ratio Segment 1[1]	Structural Resistance Demand Ratio Segment 2[1]	Combined Axial & Bending Demand Ratio Segment 2[2]
Abutment 1 Load Case 2 Scour	367	0.14	0.68	0.56	0.60
Abutment 2 Load Case 2 Scour	367	0.14	0.64	0.56	0.61

Notes: [1]The displayed structural resistance demand ratio is the maximum of the ratio in the top segment of the pile ($P_u/P_{r.top}$), the ratio in the second segment of the pile ($P_u/P_{r.2nd}$), and the ratio in the bottom segment of the pile ($P_u/P_{r.bottom}$). The ratio should be less than 1.0 to avoid pile failure and greater than 0.20 to avoid using an unnecessarily large pile, as recommended in the MaineDOT BDG (page 5-43). [2]The combined axial and bending ratio should be less than 1.0, as per AASHTO LRFD Article 6.9.2.2.1

6.6.3 Summary of Pile Analysis and Recommendations

We recommend the abutment piles be driven to, and seated on, bedrock and oriented with the weak axis parallel to the centerline of bearing. The recommended maximum factored loads for design of HP 14x89 piles at Abutment 1 and Abutment 2 including the pile weight, the recommended pile lengths, and the recommended pile orientation are summarized in Table 6-10.

Table 6-10: Summary of HP 14x89 Pile Evaluation

Location	Recommended Nominal Pile Driving Resistance[1] (kips)	Recommended Maximum Factored Axial Load Per Pile (kips)	Estimated Pile Length (feet)	Pile Axis Orientation
Abutment 1	565	363	44.9	Weak
Abutment 2	565	363	39.9	Weak

Note: [1]Using a resistance factor of 0.65 for driving criteria that will be established by dynamic testing of at least two piles per site condition, but no less than 2% of the production piles.

6.7 Scour and Riprap

As discussed above, the scour analysis presented in the PDR[17] provided scour depths of 29 feet below the thalweg for Q_{100} flow conditions and 33.6 feet below the thalweg for Q_{500} flow conditions, which would result in scour to bedrock. We understand that a design minimum thickness of 4.0 feet of heavy Riprap will prevent scour along the abutment foreslopes and is meant to protect the pile foundations and abutments. However, the materials in the streambed remain subject to scour.

The proposed abutments will be armored with Riprap in accordance with MaineDOT Standard Details[21] and Standard Specifications[9]. The embankments and foreslopes of the bridge abutments at the water level are designed to include 4-foot thick minimum layer of heavy riprap overlying a 12-inch minimum layer of Granular Borrow (Section 703.19) protective aggregate cushion (Detail 610(03)). The riprap slope with the underlaying protective aggregate cushion will be underlain by a Class 1 non-woven erosion control geotextile meeting the requirements of Section 722.03.

7.0 GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

7.1 Embankment Construction

All areas proposed for embankment fill placement and abutment construction should be cleared, grubbed, and stripped of existing vegetation, pavement, and topsoil. During the grubbing and stripping process, unsuitable materials exposed at the subgrade level, such as wood, logs, tree stumps, forest mat materials, peat, debris fill, or other materials that may decay or collapse should be removed. Prior to placing new fill materials, the existing slopes should be benched to provide a stable surface for compaction and ensure new fills are keyed into the

[21] Maine DOT Standard Details, March 2026. https://www.maine.gov/dot/sites/maine.gov.dot/files/inline-files/2026%20Standard%20Details%203_2026.pdf

existing embankment. Bench widths should be at least 3 feet, and each bench should be excavated as the layers of the new embankment are being constructed.

Subgrade surfaces for embankments should be prepared in accordance with MaineDOT Specification[9] Section 203.09 for preparation of the embankment area. which requires a firm, unyielding surface. Embankment fill should meet the requirements of MaineDOT Standard Specification[9] Section 703.19 for Granular Borrow. Embankment excavation and fill placement should follow MaineDOT Specification[9] Section 203.13 for embankment construction and should meet the requirements described in this report and at least 90 percent of the maximum dry density as determined by AASHTO T 180. Structural fill materials and placement methods for abutment construction should meet the requirements of MaineDOT Standard Specifications[9], specifically Section 203.10 through Section 203.12, and 203.17.

If wet subgrade conditions are encountered in abutment and embankment fill areas, wet and disturbed subgrade material should be excavated and replaced with an appropriate size stone and covered with a geotextile (both per MaineDOT Standard Specifications[9]) to allow proper compaction of overlying fills. If seepages persist over a broad subgrade area, provisions should be made to allow for positive drainage beneath and within the new abutments and embankment fills. Positive drainage could be provided by a column of crushed stone wrapped in filter fabric that daylights beyond the new toe of slope.

7.2 Excavation Support

At the writing of this report, the sequence of the work is unknown. Construction activities will require soil excavation to below both the stream water level and water table encountered during the geotechnical site investigation. Construction will require cofferdams and earth support systems to support culvert and stream bed excavations and to control water flow. Design of all temporary earth support systems should comply with Section 511 Cofferdams of the MaineDOT Standard Specifications[9] and OSHA requirements. Temporary dewatering of the stream and excavation may be required and should be performed to limit disturbance to the culvert bearing surfaces. Dewatering activities should comply with Section 656 Temporary Soil Erosion and Water Pollution Control of the MaineDOT Standard Specifications. The contractor should additionally control groundwater and surface water infiltration so that construction can be completed under dry conditions.

7.3 Bridge Abutments and Piles

Adequate drainage should be installed behind the integral abutments. We recommend the use of French drains with weap holes per MaineDOT Standard Specification[9] Section 512, rather than the use of geocomposite drainage board, to assure efficacy of abutment drainage throughout the design life of the bridge.

The estimated pile lengths provided in this report were based on an average interpreted depth to bedrock at the abutment locations. However, borings show that bedrock elevation varies at the site. Over the width of the abutments, there may be bedrock elevation differences that should be anticipated. We recommend a prefabricated pile tip be used in accordance with MaineDOT Standard Specification[9] 501.048 to account for the sloping bedrock and that the number of pile driving blow counts be limited to between 3 and 15 blows per inch.

To establish the actual pile lengths needed to develop the required axial resistance of the driven piles, we recommend implementing a field-verification program consisting of a wave equation analysis and dynamic testing with signal matching including the following:

- Because of the weak and fractured bedrock encountered in the boring logs, we anticipate the piles will penetrate at least 6 inches into the bedrock. Pile lengths should account for this penetration into rock.

- Prior to the beginning of pile driving, a wave equation analysis should be conducted on the contractor's proposed driving system to ensure the hammer is capable of driving the piles to the required capacities without overstressing the piles to the required penetration depths and within a reasonable number of hammer blow counts, typically 3 to 15 blows per inch at end of driving (EOD). If more than one pile section is used at the site, each different pile section should have its own separate wave equation analysis completed.
- Dynamic testing in accordance with MaineDOT specified procedures should be used to establish the driving criteria at the beginning of production pile driving. Two percent of the production piles or a minimum of one pile per substructure should be subjected to dynamic testing. MaineDOT's recent guidance is to perform one dynamic pile test per substructure per construction stage. Because there is a temporary detour planned, we assume the abutments will be constructed in a single stage and therefore one dynamic pile test is recommended per abutment. We recommend the first production pile for each structure be tested. We also recommend a 24-hour restrike test be performed for each tested pile because of anticipated pile penetration into the weak and fractured bedrock encountered on site. Dynamic testing and field inspection should include verification of hammer stroke or bounce chamber pressures and hammer blows throughout the pile driving operations. Signal matching analysis of the dynamic test data using methods described by Rausche et al. (1972)[22] should be conducted to determine pile bearing resistance.
- We recommend that MaineDOT's typical refusal criteria of 10 blows per 0.5 inches be implemented to reduce structural damage to the piles. The Engineer should review all dynamic pile testing results before the piles are accepted.

7.4 Temporary Detour

The Draft PS&E plans[1] recommend an on-site temporary detour bridge. We assume the selected Contractor will be responsible for temporary embankment and/or temporary culvert geotechnical stability or bearing resistance during temporary operations.

8.0 REPORT AND EXPLORATION LIMITATIONS

This Geotechnical Design Report (GDR) was prepared for the exclusive use of MaineDOT for the replacement of Smith Brook Bridge No. 3189 of Bancroft Road over Smith Brook in Bancroft TWP, Maine. We conducted our evaluations and compiled our recommendations in accordance with generally accepted soil and foundation engineering practices in this geographical area and under similar time and financial constraints. WSP makes no other warranty, either express or implied. If changes in the nature, design, or location of the proposed project are planned, WSP should be notified to review the appropriateness of our conclusions and recommendations, and to modify the recommendations as appropriate to reflect the changes in design. In addition, WSP should review the final plans and specifications to evaluate compliance with these recommendations.

Our analyses and recommendations are based, in part, on information obtained from the referenced subsurface explorations completed at the discrete locations described in the report. Variations in the nature and extent of subsurface conditions between explorations should be expected. WSP should be notified if conditions encountered during construction vary from those described in this report so that we may re-evaluate, and if necessary, revise the recommendations made in this report.

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[22] Rausche, F., F. Moses, and G. G. Goble. 1972. "Soil Resistance Predictions from Pile Dynamics," *Journal of the Soil Mechanics and Foundation Division*. American Society of Civil Engineers, Reston, VA, Vol. 98, No. SM9, pp. 917–937.

The professional services provided by WSP for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report and have not been investigated or addressed.

Signature Page

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https://wsponlinenam.sharepoint.com/sites/us-mainedot-northern/shared documents/3189-smith brook bridge - 28262_00/technical/design/geotechnical/09 reports/02 gdr/win 028262.00 bridge 3189 signed gdr wsp 20260721.docx

Tables

Table 1: Subsurface Exploration Locations
Preliminary Geotechnical Design Report
Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft Township, ME
MaineDOT WIN 028262.00

Boring Designation[1,2,3]	As-Drilled Locations[4]		Existing Ground Surface Elevation[4]	Bedrock Depth	Maximum Depth of Investigation
	Northing	Easting	(feet)	(feet bgs)	(feet bgs[5])
BB-BSB-101	672592.06	2257916.48	351.3	55.0	66.0
BB-BSB-102	672612.95	2257964.67	350.3	50.0	62.0

Notes:

1. Boring locations are shown in Figure 2 - Boring Location Plan of the Preliminary Geotechnical Design Report.
2. Borings were performed by Seaboard Drilling LLC in March 2025.
3. Boring and rock probe logs are presented in Appendix A of the Preliminary Geotechnical Design Report.
4. As-drilled boring locations and ground surface elevations were surveyed by WSP in April 2025 are are located in the electronic file "Borings.dgn"
5. bgs = below ground surface.

Prepared By: RJN
Checked By: LDN
Reviewed By: MEL

Table 2: Summary of Rock Core Quality
Preliminary Geotechnical Design Report
Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft Township, ME
MaineDOT WIN 028262.00

Test Boring Designation	Core Size (in)	Run No.	Run: Midpoint Depth Below Bedrock Surface (ft)	Run: Depth Below Ground Surface (ft) Start	End	Midpoint	Run: Length (ft)	TCR[1] Length (ft)	TCR[1] %	RQD[2] Length (ft)	RQD[2] %	RQD[2] Designation	Physical Rock Parameters: Weathering[3]	Estimated Field Strength[3]	Rock Mass Rating [RMR][4]	Lithologic, Rock Mass and Discontinuity Description[5,6]
BB-BSB-101	NQ2 (2)	R1	1.6	56.0	57.2	56.6	1.2	1.2	100%	0.00	0%	Very Poor	Slightly Weathered (W2)	Medium Strong (R3)	36	Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities smooth, highly fractured, broken rock core.
		R2	2.9	57.2	58.6	57.9	1.4	1.1	79%	0.00	0%	Very Poor	Moderately Weathered (W3)	Medium Strong (R3)	30	Grey, fine grained, METALIMESTONE, medium strong, moderately weathered; discontinuities smooth, highly fractured, broken rock core.
		R3	5.0	58.6	61.4	60.0	2.8	1.6	57%	0.00	0%	Very Poor	Slightly Weathered (W2)	Strong (R4)	40	Grey, fine grained, METALIMESTONE, strong, slightly weathered; discontinuities vertical to moderately dipping, close to very closely spaced, tight to open, no infilling, smooth, planar; average 5.6 fractures per foot.
		R4	8.7	61.4	66.0	63.7	4.6	4.2	91%	0.85	18%	Very Poor	Fresh (W1)	Strong (R4)	45	Grey, fine grained, METALIMESTONE, strong, fresh; discontinuities vertical to low angle dipping, close to very closely spaced, tight to open, quartz infilling between 61.4 and 62.4 ft bgs, quartz seems over entire run, silt seems between fractures at 65.0 ft bgs, joints with fine sand infilling, syncline and monocline folding, smooth to rough, planar to stepped; average 1.9 fractures per foot, reacts with acid.
BB-BSB-102	NQ2 (2)	R1	0.0	50.0	50.3	50.2	0.3	0.3	100%	0.00	0%	Very Poor	Slightly Weathered (W2)	Strong (R4)	35	Grey, fine grained, METALIMESTONE, strong, slightly weathered; discontinuities moderately dipping, very closely spaced, open, silty sand infilling at the top, smooth, planar; average 3.3 fractures per foot.
		R2	2.5	51.0	54.0	52.5	3.0	2.5	83%	0.00	0%	Very Poor	Slightly Weathered (W2)	Medium Strong (R3)	36	Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertically dipping, close to very closely spaced, open, tan silty infilling and minor quartz seems smooth, planar, partially broken rock core, more than 10 fractures per foot.
		R3	4.8	54.0	55.5	54.8	1.5	1.5	100%	0.00	0%	Very Poor	Slightly Weathered (W2)	Medium Strong (R3)	33	Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertical to horizontally dipping, closely spaced, open, tan silty infilling, smooth, planar; average 4.6 fractures per foot.
		R4	6.3	55.5	57.0	56.3	1.5	0.9	60%	0.00	0%	Very Poor	Slightly Weathered (W2)	Medium Strong (R3)	33	Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertical to horizontally dipping, very closely spaced, open, tan silty infilling, smooth, planar; average 9.0 fractures per foot in first 0.6 feet of run, highly fractured bottom 0.3 feet of run.
		R5	8.3	57.0	59.6	58.3	2.6	1.4	54%	0.00	0%	Very Poor	Slightly Weathered (W2)	Medium Strong (R3)	33	Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertical to horizontally dipping, very closely spaced, open, tan silty infilling, quartz in bottom 0.2 ft, smooth, planar; average 5.7 fractures per foot.
		R6	10.8	59.6	62.0	60.8	2.4	1.8	75%	0.35	15%	Very Poor	Fresh (W1)	Strong (R4)	53	Grey, fine grained, METALIMESTONE, strong, fresh; discontinuities moderately dipping, closely spaced, tight, quartz infilling, monocline folds, rough, stepped; average 0.55 fractures per foot.

Notes:

1. TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.

2. RQD = rock quality designation. RQD is the total length of intact, full diameter core pieces recovered with a length greater than or equal to 4 inches measured along the core axis. The percent RQD is the total length of RQD measured divided by the run length. Note that vertical discontinuities are not included in determination of RQD.

3. Weathering and Estimated Field Strength based on Tables II.4 and II.3 (respectively) in Wyllie and Mah, 2004, Rock Slope Engineering: Civil and Mining, 4th Edition (based on ISRM, 1981).

4. Rock Mass Rating (RMR) System (Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, groundwater conditions, and orientation of discontinuities. These ratings are summed to provide the RMR value. The rating adjustment for joint orientation was assigned a value of 0; correlation of geologic field mapping data of exposed rock outcrops with the rock core samples and proposed foundation type may allow for a different rating adjustment for joint orientation, and thus a modification to the RMR value shown on this table.

5. Bedrock formation name from: Osberg, Philip H., Hussey, Arthur M., II, and Boone, Gary M. (editors), 1985, Bedrock geologic map of Maine: Maine Geological Survey, 1 plate, correlation chart, tectonic inset map, metamorphic inset map, color geologic map, cross sections, scale 1:500,000. Maine Geological Survey Maps. 23. http://digitalmaine.com/mgs_maps/23

6. ft = feet, in = inches

Prepared by: LMP/LDN
Checked by: RJN
Reviewed by: MEL

Table 3: Summary of Soil Index and Classification Laboratory Testing Results
Preliminary Geotechnical Design Report
Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft Township, ME
MaineDOT WIN 028262.00

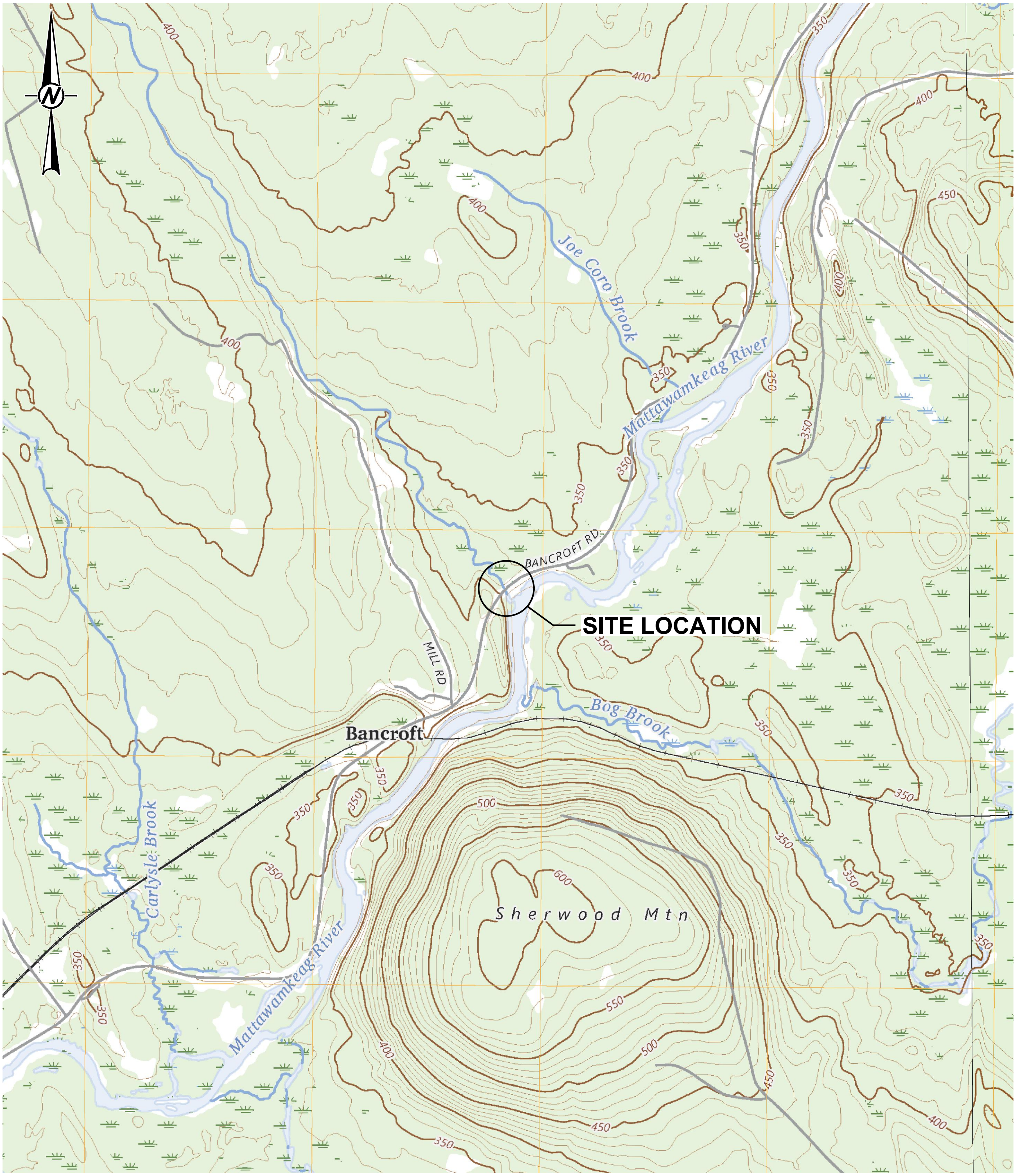
Test Boring Designation[1]	Ground Surface Elevation [2] (feet)	Sample Number	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet) [2]	Laboratory Testing[3]		Soil Classification	
					Sieve Minus No. 200[4] (%)	Moisture Content [5] (%)	AASHTO[6]	USCS[7]
BB-BSB-101	351.3	3D	5.0 - 7.0	346.3 - 344.3	56.7	11.1	A-4 (0)	*ML*
		5D	10.0 - 12.0	341.3 - 339.3	51.6	11.5	A-4 (0)	*ML*
		9D	30.0 - 32.0	321.3 - 319.3	90.3	46.8	A-4 (0)	ML
		11D	40.0 - 42.0	311.3 - 309.3	97.6	40.5	A-4 (0)	*ML*
		13D	50.0 - 52.0	301.3 - 299.3	2.3	-	A-1-a (1)	GW
BB-BSB-102	350.3	3D	5.0 - 7.0	345.3 - 343.3	40.7	11.2	A-4 (0)	*SM*
		4D	7.0 - 9.0	343.3 - 341.3	47.3	11.7	A-4 (0)	SM
		6D	15.0 - 17.0	335.3 - 333.3	4.6	11.9	A-1-a (1)	GW
		7D	20.0 - 22.0	330.3 - 328.3	82.8	70.3	A-4 (0)	*ML*
		8D	25.0 - 27.0	325.3 - 323.3	68.6	22.8	A-4 (0)	*ML*
		9D	30.0 - 32.0	320.3 - 318.3	97.8	35.7	A-4 (0)	*ML*

Notes:

1. Boring locations are illustrated in Figure 2 - Boring Location Plan of the Preliminary Geotechnical Design Report.
2. As-drilled boring locations and ground surface elevations were surveyed by WSP in April 2025 are are located in the electronic file "Borings.dgn"
3. Laboratory testing was performed by Geotesting Express of Acton, MA.
4. Particle size testing was performed in accordance with ASTM D6913, Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.
5. Moisture content testing was performed in accordance with ASTM D2216, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.
6. AASHTO classification is based on AASHTO M145-91 and includes both Group and Subgroup classification and Group Index (in parentheses) based on the percent passing the #200 sieve and Atterberg Limits. Classification provided by GeoTesting Express.
7. USCS classification was performed by WSP based on interpreted laboratory test results. Items in italics use assumed fines type based on field characterization.
8. Complete laboratory test results for soil testing are provided in Appendix C of the Preliminary Geotechnical Design Report.

Prepared By: RJN
Checked By: LDN
Reviewed By: MEL

Figures



REFERENCE(S)

1. BASE MAP TAKEN FROM U.S.G.S. 7.5-MINUTE WYTOPITLOCK AND JIMMEY MOUNTAIN QUADRANGLES OF MAINE.

0 1000 2000

1" = 2000' FEET

CLIENT

MAINE DEPARTMENT OF TRANSPORTATION

PROJECT

NORTH BRIDGE BUNDLE (NBB), BRIDGE #3189

CONSULTANT

wsp

YYYY-MM-DD	2026-07-21
DESIGNED	LDN
PREPARED	RG
REVIEWED	MEL
APPROVED	

TITLE

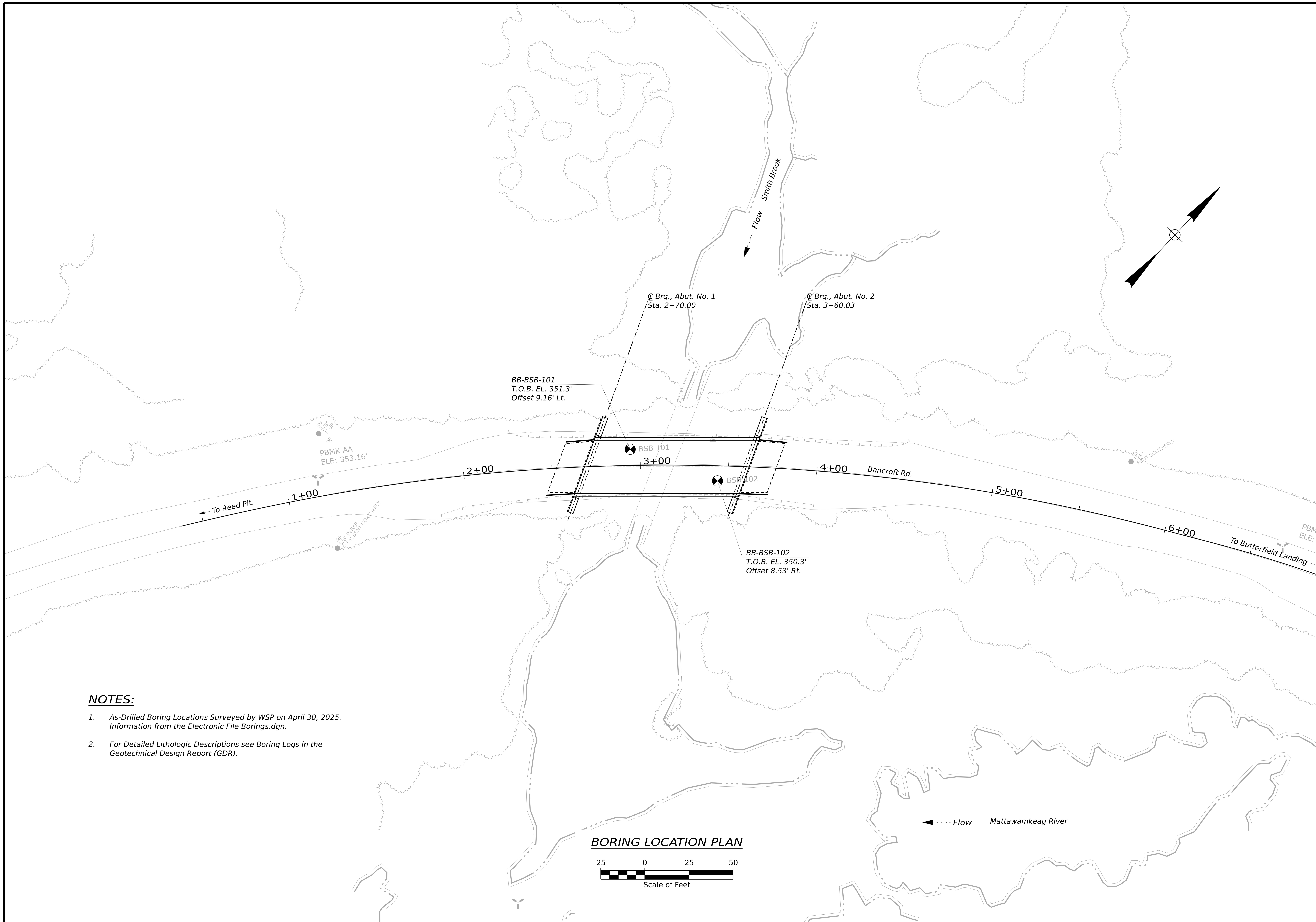
SITE LOCATION MAP
BANCROFT TOWNSHIP SMITH BROOK BRIDGE #3189 REPLACEMENT
MAINEDOT WIN: 028262.00

PROJECT NO.	CONTROL	REV.	FIGURE
US0040947.9162	0001-001	0	1

Last Edited By: usrg700666 Date: 2025-06-17 Time:3:10:23 PM | Printed By: USRG700666 Date: 2025-06-17 Time:3:10:35 PM
Path: \\corp.pbwan.net\US\CentralData\USMJT100\CADD\Projects\MaineDOT\NBB_Bridge\99_PROJECTS\US0040947.9162\0001_GeotechDesign_3189\02_PRODUCTION\DWG\ | File Name: US0040947.9162-0001-001.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

0 1 in



NOTES:

1. As-Drilled Boring Locations Surveyed by WSP on April 30, 2025. Information from the Electronic File Borings.dgn.
2. For Detailed Lithologic Descriptions see Boring Logs in the Geotechnical Design Report (GDR).

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

Federal Project No. 2826200

WIN 28262.00

	BY	DATE
PROJ. MANAGER	R. MYERS	
DESIGN-DETAILED	D. BURGESS	
CHECKED-REVIEWED	M. LANDON	
DESIGN2-DETAILED2	D. HENDERSON	7/15/2026
DESIGN3-DETAILED3	D. HENDERSON	7/15/2026
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

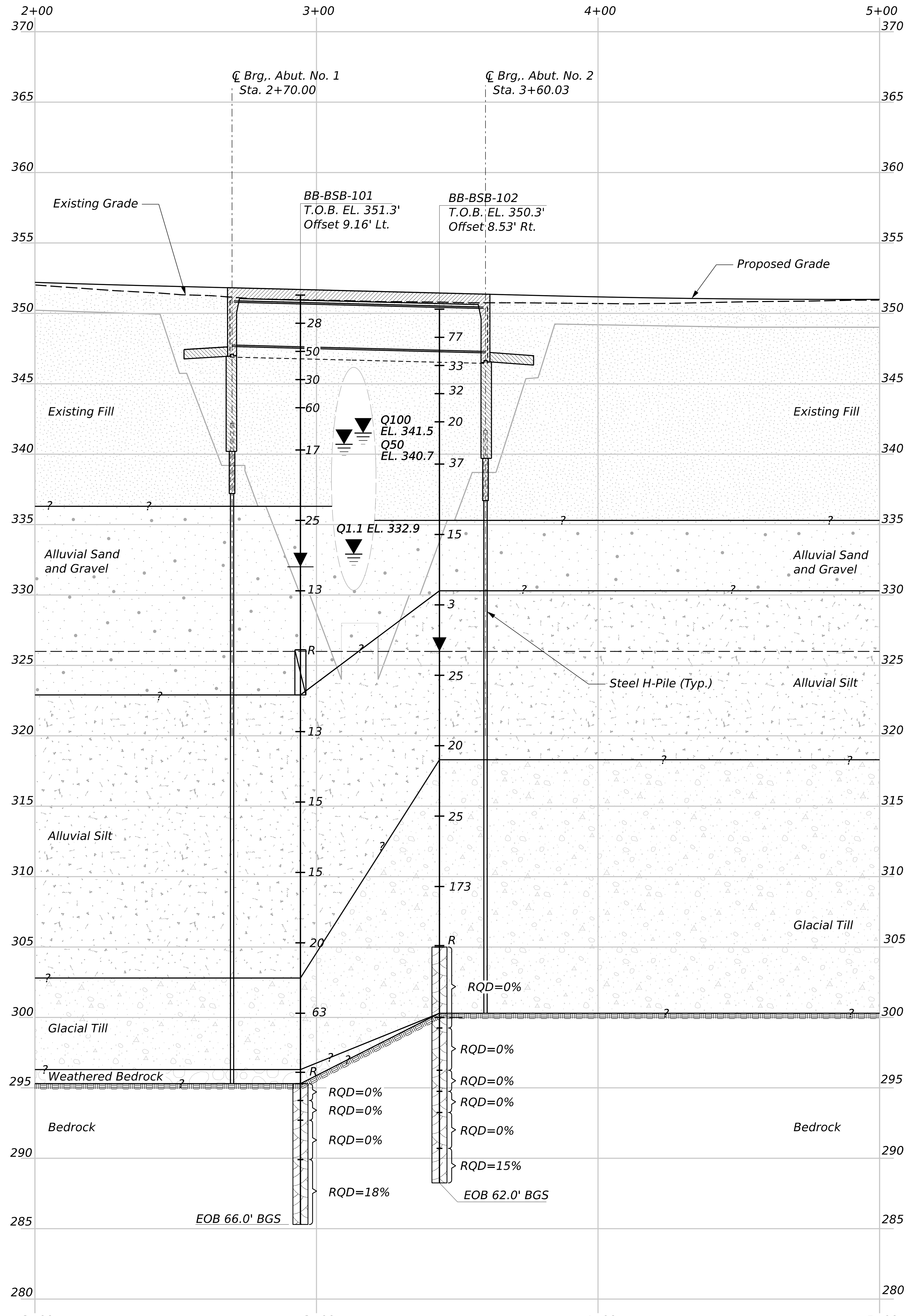
SMITH BROOK BRIDGE BRIDGE NO. 3189
CROSSING SMITH BROOK
BANCROFT TWP

BORING LOCATION PLAN

SHEET NUMBER

2

OF 3



INTERPRETIVE SUBSURFACE PROFILE

Horiz. 25 0 25 50

Vert. 5 0 5 10

Scale of Feet

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line

Top of Boring Probe

30 SPT: N60 - Value (Corrected for Hammer Efficiency)

Boulder

R Refusal

—? Strata Interface

Rock Quality Designation (RQD) of Bedrock Core Sample

EOB End of Boring Depth Below Ground Surface (BGS)

LEGEND

Brown, Medium Dense to Very Dense, Silt and Trace to Little Coarse to Fine Sand, Trace to Little Coarse to Fine Gravel (Fill)

Grey, Medium Dense to Dense, Fine to Medium Sand, Some Fine to Coarse Gravel, Some Silt, Trace Clay (Alluvial Sand and Gravel)

Grey, Medium Stiff to Stiff, Silt, Trace to Little Fine to Coarse Sand, Trace Fine to Coarse Gravel, Slightly Plastic (Alluvial Silt)

Grey, Medium Dense to Very Dense, Fine to Coarse Gravel and Sand, Little to Trace Silt (Glacial Till)

Weathered Bedrock

Grey, Fine Grained, Medium Strong to Strong, Moderately Weathered to Fresh, Metalimestone (Bedrock)

Existing Ground Surface

Existing Ground Water Elevations

Existing Ground Water Surface

Design Water Level

Existing Bridge Structure

Proposed Bridge Substructure and Superstructure

NOTES:

1. As-drilled boring locations surveyed by WSP on April 30, 2025. Information from the electronic file Borings.dgn.
2. For detailed lithologic descriptions see boring logs in the Geotechnical Design Report (GDR).
3. For complete laboratory data, see laboratory results in the Geotechnical Report (GDR).
4. Groundwater surface is interpreted from localized water levels and measurements taken during the subsurface exploration program, see the Geotechnical Design Report (GDR) and Boring Logs.
5. This generalized subsurface profile is intended to convey trends in the subsurface conditions. The boundaries between strata are approximate and idealized and have been developed based on interpretations of widely spaced explorations. Actual soil and rock transitions may vary and may be erratic.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

Federal Project No. 2826200

WIN 28262.00

	BY:	DATE
PROJ. MANAGER	R. MYERS	
DESIGN-DETAILED	D. BURGESS	D. HENDERSON 7/15/2026
CHECKED-REVIEWED	M. LANDON	D. HENDERSON 7/15/2026
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

SMITH BROOK BRIDGE BRIDGE NO. 3189
CROSSING SMITH BROOK
BANCROFT TWP

INTERPRETIVE SUBSURFCE PROFILE

SHEET NUMBER

3

OF 3

Date: 7/17/2026

Username: sallard

Filename: c:\bms\wsp-pb-us-pw-02\wsp__william.gerhold\d1012386\GeoProfile_Bancroft Road.dgn

APPENDIX A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

Desired Rock Observations (in this order, if applicable):
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
-tightness (tight, open, or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

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

Sample Container Labeling Requirements:

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

Table A-2

Classification of Rock Material Strengths[1]

Grade	Description	Field Identification	Approx. Range of Uniaxial Compressive Strength	
			MPa	psi
S1	Very soft clay	Easily penetrated several inches by fist	<0.025	<4
S2	Soft clay	Easily penetrated several inches by thumb	0.025-0.05	4-7
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05-0.10	7-15
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort	0.10-0.25	15-35
S5	Very stiff clay	Readily indented by thumbnail	0.25-0.50	35-70
S6	Hard clay	Indented with difficulty by thumbnail	>0.50	>70
R0	Extremely weak rock	Indented by thumbnail	0.25-1.0	35-150
R1	Very weak rock	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife	1-5	150-725
R2	Weak rock	Can be peeled by a pocket knife with difficulty; shallow indentations made by firm blow with point of geological hammer	5-25	725-3,500
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer	25-50	3,500-7,000
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100	7,000-15,000
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250	15,000-36,000
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250	>36,000

Note: Grades S1 to S6 apply to cohesive soils, for example clays, silty clays, and combinations of silts and clays with sand, generally slow draining. Discontinuity wall strength will generally be characterized by grades R0-R6 (rock) while S1-S6 (clay) will generally apply to filled discontinuities. Rock material strength descriptions are included in the rock core descriptions in the boring logs. Rock material strength grades (R0-R6) are not included in the rock core descriptions to avoid confusion with the numbering of the rock core runs.

[1] International Society for Rock Mechanics (ISRM), Commission on standardization of laboratory and field tests (1978): Suggested methods for the quantitative description of discontinuities in rock masses. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., Vol. 15, No. 6, pp. 319-368.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	North Bridge Bundle (NBB), Bridge #3189	**Boring No.:**	BB-BSB-101		
Location:	Bancroft, ME	**WIN:**	028262.00		
Driller:	Seaboard Drilling, LLC	**Elevation (ft.)**	351.32	**Auger ID/OD:**	4 in
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	2 in Standard Split Spoon
Logged By:	Ryan Nagle	**Rig Type:**	Diedrich D-50 (SN: 367)	**Hammer Wt./Fall:**	140 lbs /30 in
Date Start/Finish:	7:41 3/24/25-2:47 3/24/25	**Drilling Method:**	SSA / Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N:672592.06 E:2257916.48	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	19.3 ft at 12:49 on 3/24/25
Hammer Efficiency Factor:	1.0	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	350.9		5.5" of Asphalt Pavement ——0.5	
	1D	24/18	1.00 - 3.00	19-9-8-14	17	28				Advanced with augers from 0.46 to 1 foot bgs. Brown, moist, medium dense, SILT, trace to little coarse to fine sand, little fine gravel (FILL).	
	2D	24/18	3.00 - 5.00	14-15-15-14	30	50				Brown, moist, very dense, SILT, trace to little coarse to fine sand, little fine gravel (FILL).	
5	3D	24/17	5.00 - 7.00	6-11-7-12	18	30				Brown, moist, dense, SILT, trace to little coarse to fine sand, little fine gravel (FILL). 7.0 to 7.6 SAA	WC = 11.1% Fines = 56.7% A-4 (0), ML
	4D	24/12	7.00 - 9.00	9-22-14-12	36	60				Brown, moist, very dense, SILT, trace to little coarse to fine sand, little fine gravel (FILL). Tv = 1 tsf, pp = 1.5 tsf	
10	5D	24/15.6	10.00 - 12.00	9-6-4-7	10	17	PUSH			Olive to brown, moist, medium dense, SILT, trace to little fine to coarse sand, trace to little coarse to fine gravel (FILL).	WC = 11.5% Fines = 51.6% A-4 (0), ML
							67				
							77	336.3		——15.0	
15	6D	24/14.4	15.00 - 17.00	10-8-7-8	15	25	81			Brown, moist, medium dense, fine to medium SAND, some fine gravel, some silt (ALLUVIAL SAND AND GRAVEL).	
							103				
							95				
							65			Driller Note: Casing driving and washout became easier at approximately 18 ft bgs.	
							40				
20	7D	24/4	20.00 - 22.00	5-4-4-6	8	13	28			Grey, wet, medium dense, fine to medium SAND, some fine to coarse gravel, some silt, trace clay, low plasticity (ALLUVIAL SAND AND GRAVEL). Driller Note: Wash water turns grey at approximately 20 ft bgs.	
							47				
							103				
							105				
							90	326.1			
25	8D	2/2	25.00 - 25.17	75/2"	R		OPEN			Grey, wet, very dense, fine to medium SAND, some fine to coarse gravel, some silt, trace clay, low plasticity (ALLUVIAL SAND AND GRAVEL). ——25.2	
										Granite Boulder from 25.2 to 28.4 ft bgs	
		4.8/4	28.00 - 28.40	Boulder				322.9		Grey with black and white mottling, fine grained, GRANITE, no discontinuities, no infilling, rough, planar [BOULDER]. ——28.4	

Remarks:

1. Hammer Efficiency factor provided by Seaboard and taken from "2024PA00175-Seaboard-SPT Energy Calibration" by GRL Engineers Inc., dated 10/23/2024.
2. As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" on 5/6/25.
3. Water level reading taken on 3/24/25 at 12:49 was made before the start of rock coring with bottom of casing at 55.7 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: North Bridge Bundle (NBB), Bridge #3189
Location: Bancroft, ME

Boring No.: BB-BSB-101
WIN: 028262.00

Driller:	Seaboard Drilling, LLC	**Elevation (ft.)**	351.32	**Auger ID/OD:**	4 in
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	2 in Standard Split Spoon
Logged By:	Ryan Nagle	**Rig Type:**	Diedrich D-50 (SN: 367)	**Hammer Wt./Fall:**	140 lbs /30 in
Date Start/Finish:	7:41 3/24/25-2:47 3/24/25	**Drilling Method:**	SSA / Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N:672592.06 E:2257916.48	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	19.3 ft at 12:49 on 3/24/25
Hammer Efficiency Factor:	1.0	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	9D	24/12	30.00 - 32.00	9-4-4-5	8	13	50			Grey, wet, medium stiff, SILT, trace fine sand (ALLUVIAL SILT).	WC = 46.8% Fines = 90.3% A-4 (0), ML
							42				
							40				
							55				
							41				
35	10D	24/17	35.00 - 37.00	5-4-5-4	9	15	49			Grey, wet, stiff, SILT, trace sand, slightly plastic (ALLUVIAL SILT).	
							43				
							48				
							54			Driller Note: Clay layers noticed in wash water.	
							60				
40	11D	24/10	40.00 - 42.00	4-4-5-6	9	15	52			Grey, wet, stiff, SILT, trace fine sand, slightly plastic (ALLUVIAL SILT).	WC = 40.5% Fines = 97.6% A-4 (0), ML
							55				
							78				
							81				
							93				
45	12D	24/13	45.00 - 47.00	6-6-6-6	12	20	84			Grey, wet, stiff, SILT, trace fine sand, slightly plastic (ALLUVIAL SILT).	
							96				
							116				
							117	302.8		48.5	
							113			Driller Note: Material became more gravelly at 48.5 ft bgs.	
50	13D	24/7	50.00 - 52.00	15-17-21-42	38	63	52			Grey, wet, very dense, fine to coarse GRAVEL, little medium to coarse sand, trace silt, well graded (GLACIAL TILL).	Fines = 2.3% A-1-a (1), GW
							60				
							83				
							78				
							194	296.3		55.0	
55	14D	5/4	55.00 - 55.42	100/5"	R		56	295.3		Weathered bedrock 56.0	
	R1	14.4/14.4	56.00 - 57.20	RQD = 0%						R1 (56.0' - 57.2'): Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities smooth, extremely fractured, broken rock core. Rock Mass Quality = very poor 100% Recovery Rock Core Rate (min:sec)	
	R2	16.8/13.2	57.20 - 58.60	RQD = 0%							
	R3	33.6/19.2	58.60 - 61.40	RQD = 0%							

Remarks:

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3. Water level reading taken on 3/24/25 at 12:49 was made before the start of rock coring with bottom of casing at 55.7 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: North Bridge Bundle (NBB), Bridge #3189
Location: Bancroft, ME

Boring No.: BB-BSB-101
WIN: 028262.00

Driller:	Seaboard Drilling, LLC	**Elevation (ft.)**	351.32	**Auger ID/OD:**	4 in
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	2 in Standard Split Spoon
Logged By:	Ryan Nagle	**Rig Type:**	Diedrich D-50 (SN: 367)	**Hammer Wt./Fall:**	140 lbs /30 in
Date Start/Finish:	7:41 3/24/25-2:47 3/24/25	**Drilling Method:**	SSA / Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N:672592.06 E:2257916.48	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	19.3 ft at 12:49 on 3/24/25
Hammer Efficiency Factor: 1.0		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
60										56.0-57.0 ft (2:46) 57.0-57.2 ft (1:36) R2 (57.2' - 58.6'): Grey, fine grained, METALIMESTONE, medium strong, moderately weathered; discontinuities smooth, highly fractured, broken rock core. Rock Mass Quality = very poor 79% Recovery Rock Core Rate (min:sec) 57.2-58.0 ft (2:58) 58.0-58.6 ft (6:19)	
	R4	55.2/50.4	61.40 - 66.00	RQD = 18%							
65								285.3		R3 (58.6' - 61.4'): Grey, fine grained, METALIMESTONE, strong, slightly weathered; discontinuities vertical to moderately dipping, close to very closely spaced, tight to open, no infilling, smooth, planar, average 5.6 fractures per foot. Rock Mass Quality = very poor 91% Recovery Rock Core Rate (min:sec) 58.6-59.0 ft (1:07) 59.0-60.0 ft (2:35) 60.0-61.0 ft (2:36) 61.0-61.4 ft (1:15)	
70										R4 (61.4' - 66.0'): Grey, fine grained, METALIMESTONE, strong, fresh; discontinuities vertical to low angle dipping, close to very closely spaced, tight to open, quartz infilling between 61.4 and 62.4 ft bgs, quartz seems over entire run, silt seems between fractures at 65.0 ft bgs, joints with fine sand infilling, syncline and monocline folding, smooth to rough, planar to stepped, average 1.9 fractures per foot, reacts with acid. Rock Mass Quality = very poor 91% Recovery Rock Core Rate (min:sec) 61.4-62.0 ft (1:11) 62.0-63.0 ft (5:20) 63.0-64.0 ft (7:22) 64.0-65.0 ft (5:24) 65.0-66.0 ft (6:19)	
75										66.0	
80										**Bottom of Exploration at 66.0 feet below ground surface.** Boring backfilled with bentonite chips in the rock core socket, gravel to bottom of pavement, and patched with cold patch asphalt.	
85											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	North Bridge Bundle (NBB), Bridge #3189	**Boring No.:**	BB-BSB-102		
Location:	Bancroft, ME	**WIN:**	028262.00		
Driller:	Seaboard Drilling, LLC	**Elevation (ft.)**	350.25	**Auger ID/OD:**	4 in
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	2 in Standard Split Spoon
Logged By:	Ryan Nagle	**Rig Type:**	Diedrich D-50 (SN: 367)	**Hammer Wt./Fall:**	140 lbs /30 in
Date Start/Finish:	2:12 3/20/25-3:20 3/21/25	**Drilling Method:**	SSA / Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N:672612.95 E:2257964.67	**Casing ID/OD:**	4.0 in /4.5 in	**Water Level*:**	24.3 at 8:16 on 3/21/25
Hammer Efficiency Factor:	1.0	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	349.9		5" of Asphalt Pavement 0.4	
	1D	24/21.6	1.00 - 3.00	10-22-24-13	46	77				Brown, dry, very dense, trace to little coarse to fine SAND, trace to little coarse to fine gravel, some silt (FILL).	
	2D	24/10.8	3.00 - 5.00	10-11-9-10	20	33				Brown, moist, dense, trace to little coarse to fine SAND, trace to little coarse to fine gravel, some silt (FILL).	
5	3D	24/13.2	5.00 - 7.00	6-9-10-7	19	32				Brown, moist, dense, trace to little coarse to fine SAND, trace to little coarse to fine gravel, some silt (FILL).	WC = 11.2% Fines = 40.7% A-4 (0), SM
	4D	24/13.2	7.00 - 9.00	5-5-7-5	12	20				Brown, moist, medium dense, trace to little coarse to fine SAND, trace to little coarse to fine gravel, some silt (FILL).	WC = 11.7% Fines = 47.3% A-4 (0), SM
10	5D	24/18	10.00 - 12.00	9-11-11-8	22	37	97			Brown to olive, moist, dense, trace to little coarse to fine SAND, trace to little coarse to fine gravel, some silt (FILL).	
							89				
							86				
							73				
							63				
15	6D	24/9.6	15.00 - 17.00	5-4-5-4	9	15	21	335.3		15.0 (ALLUVIAL SAND AND GRAVEL). Driller Note: Gravelly layer noted at 15 ft bgs due to lack of wash water return.	WC = 11.9% Fines = 4.6% A-1-a (1), GW
							40				
							51				
							43				
							46				
20	7D	24/20.4	20.00 - 22.00	1-1-1-2	2	3	41	330.3		20.0 Olive, wet, soft, SILT, little fine sand (ALLUVIAL SILT).	WC = 70.3% Fines = 82.8% A-4 (0), ML
							36				
							46			Driller Note: Driller observes another increase of gravel content around 22 or 23 ft bgs during casing wash.	
							73				
							93				
25	8D	24/10.8	25.00 - 27.00	9-8-7-7	15	25	59			Grey, wet, very stiff, SILT, trace fine to coarse gravel, trace fine to coarse sand (ALLUVIAL SILT).	WC = 22.8% Fines = 68.6% A-4 (0), ML
							67				
							61				
							73				
							83				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	North Bridge Bundle (NBB), Bridge #3189
Location:	Bancroft, ME
Boring No.:	BB-BSB-102
WIN:	028262.00

Driller:	Seaboard Drilling, LLC	**Elevation (ft.)**	350.25	**Auger ID/OD:**	4 in
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	2 in Standard Split Spoon
Logged By:	Ryan Nagle	**Rig Type:**	Diedrich D-50 (SN: 367)	**Hammer Wt./Fall:**	140 lbs /30 in
Date Start/Finish:	2:12 3/20/25-3:20 3/21/25	**Drilling Method:**	SSA / Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N:672612.95 E:2257964.67	**Casing ID/OD:**	4.0 in /4.5 in	**Water Level*:**	24.3 at 8:16 on 3/21/25
Hammer Efficiency Factor:	1.0	**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
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$S_{u(lab)}$ = 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
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	9D	24/14.4	30.00 - 32.00	5-5-7-8	12	20	49			Grey, wet, very stiff, SILT, trace fine sand (ALLUVIAL SILT).	WC = 35.7% Fines = 97.8% A-4 (0), ML
							87				
							137	318.3		32.0	
							146				
							139				
35	10D	24/9.6	35.00 - 37.00	9-7-8-6	15	25	44			Grey, wet, medium dense, fine to medium SAND, little fine to coarse gravel, little silt (GLACIAL TILL).	
							64				
							89				
							117			Driller Note: Thick layer of gravel starting at 38.5 ft bgs.	
							158				
40	11D	24/14.4	40.00 - 42.00	21-64-40-40	104	173	OPEN			Grey, wet, very dense, fine to coarse GRAVEL, little medium to coarse sand, trace silt, well graded (GLACIAL TILL). Cobble fragments in bottom 2 inches.	
										Driller Notes: Cobbles at 42.3 ft bgs.	
	12D	4/0	45.00 - 45.33	50/3"	R						
45	R1	60/4	45.30 - 50.30	RQD = 0%						Grey, wet, very dense, cobble fragments (GLACIAL TILL AND COBBLES).	
50								300.3		50.0	
										R1 (50.0' - 50.3'): 45.3 to 50.0 ft bgs: Glacial Till and cobbles cored with cobbles recovered. 50.0 to 50.3 ft bgs: Grey, fine grained, METALIMESTONE, strong, slightly weathered; discontinuities moderately dipping, very closely spaced, open, silty sand infilling at the top, smooth, planar. 49.3-50.3 ft (3:27)	
	R2	36/30	51.00 - 54.00	RQD = 0%						R2 (51.0' - 54.0'): Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertically dipping, close to very closely spaced, open, tan silty infilling and minor quartz seems, smooth, planar, partially broken rock core, average 10+ fractures per foot.	
										Rock Mass Quality = very poor	
										83% Recovery	
	R3	18/18	54.00 - 55.50	RQD = 0%						Rock Core Rate (min:sec)	
55	R4	18/10.8	55.50 - 57.00	RQD = 0%						51.0-52.0 ft (2:15)	
										52.0-53.0 ft (3:12)	
	R5	31.2/16.8	57.00 - 59.60	RQD = 0%						53.0-54.0 ft (3:13)	
	R6	28.8/21.6	59.60 - 62.00	RQD = 15%							

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: North Bridge Bundle (NBB), Bridge #3189
Location: Bancroft, ME

Boring No.: BB-BSB-102
WIN: 028262.00

Driller:	Seaboard Drilling, LLC	**Elevation (ft.)**	350.25	**Auger ID/OD:**	4 in
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	2 in Standard Split Spoon
Logged By:	Ryan Nagle	**Rig Type:**	Diedrich D-50 (SN: 367)	**Hammer Wt./Fall:**	140 lbs /30 in
Date Start/Finish:	2:12 3/20/25-3:20 3/21/25	**Drilling Method:**	SSA / Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N:672612.95 E:2257964.67	**Casing ID/OD:**	4.0 in /4.5 in	**Water Level*:**	24.3 at 8:16 on 3/21/25
Hammer Efficiency Factor:	1.0	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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.
60								288.3		R3 (54.0' - 55.5'): Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertical to horizontally dipping, closely spaced, open, tan silty infilling, smooth, planar, average 4.6 fractures per foot. Rock Mass Quality = very poor 100% Recovery Rock Core Rate (min:sec) 54.0-55.0 ft (4:00) 55.0-55.5 ft (1:50)	
65										R4 (55.5' - 57.0'): Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertical to horizontally dipping, very closely spaced, open, tan silty infilling, smooth, planar, average 9.0 fractures per foot in first 0.6 feet of run, highly fractured bottom 0.3 feet of run. Rock Mass Quality = very poor 60% Recovery Rock Core Rate (min:sec) 55.5-56.0 ft (2:25) 56.0-57.0 ft (5:16)	
70										R5 (57.0' - 59.6'): Grey, fine grained, METALIMESTONE, medium strong, slightly weathered; discontinuities vertical to horizontally dipping, very closely spaced, open, tan silty infilling, quartz in bottom 0.2 ft, smooth, planar, average 5.7 fractures per foot. Rock Mass Quality = very poor 54% Recovery Rock Core Rate (min:sec) 57.0-58.0 ft (4:25) 58.0-59.0 ft (4:32) 59.0-59.6 ft (2:16)	
75										R6 (59.6' - 62.0'): Grey, fine grained, METALIMESTONE, strong, fresh; discontinuities moderately dipping, closely spaced, tight, quartz infilling, monocline folds, rough, stepped, average 0.55 fractures per foot. Rock Mass Quality = very poor 75% Recovery Rock Core Rate (min:sec) 59.6-60.0 ft (2:01) 60.0-61.0 ft (3:38) 61.0-62.0 ft (3:52)	
80										62.0 **Bottom of Exploration at 62.0 feet below ground surface.** Boring backfilled with bentonite chips in the rock core socket, gravel to bottom of pavement and patched with cold patch asphalt.	
85											

Remarks:

1. Hammer Efficiency factor provided by Seaboard and taken from "2024PA00175-Seaboard-SPT Energy Calibration" by GRL Engineers Inc., dated 10/23/2024.
2. As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" on 5/6/25.
3. Water level reading taken on 3/21/25 at 8:16 was taken after settling overnight with the bottom of casing at 30 ft bgs.

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

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

APPENDIX B

Rock Core Photographs

APPENDIX B
ROCK CORE PHOTOGRAPHS
BANCROFT ROAD BRIDGE NO. 3189 CROSSING SMITH BROOK
BANCROFT TOWNSHIP, MAINE
MAINEDOT WIN 028262.00

Boring	Run	Depth Below Surface			Recovery				RQD				Rock Type	Box Row	Date Cored
		Feet			Feet			%	Feet			%			
BB-BSB-102	R4	55.5	-	57.0	0.9	-	1.5	60	0.0	-	1.5	0	METALIMESTONE	Row 1	3/21/2025
	R5	57.0	-	59.6	1.4	-	2.6	54	0.0	-	2.6	0	METALIMESTONE	Row 1	3/21/2025
	R6	59.6	-	62.0	1.8	-	2.4	75	0.35	-	2.4	15	METALIMESTONE	Row 1	3/21/2025
BB-BSB-101	-	28.0	-	28.4	0.2	-	0.4	50	0.0	-	0.4	0	COBBLE	Row 1	3/24/2025
	R1	56.0	-	57.2	1.2	-	1.2	100	0.0	-	1.2	0	METALIMESTONE	Row 2	3/24/2025
	R2	57.2	-	58.6	1.1	-	1.4	79	0.0	-	1.4	0	METALIMESTONE	Row 2	3/24/2025
	R3	58.6	-	61.4	1.6	-	2.8	57	0.0	-	2.8	0	METALIMESTONE	Row 2	3/24/2025
	R4	61.4	-	66.0	4.2	-	4.6	91	0.85	-	4.6	18	METALIMESTONE	Row 3	3/24/2025

Notes:
1. "Box row" indicates the section of the box where the core is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.

Prepared By: RJN/LDN
Checked By: LMP
Reviewed By: MEL

APPENDIX B
ROCK CORE PHOTOGRAPHS
BANCROFT ROAD BRIDGE NO. 3189 CROSSING SMITH BROOK
BANCROFT TOWNSHIP, MAINE
MAINEDOT WIN 028262.00

Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
BB-BSB-102	R1	45.3 - 50.3	0.3 - 5.0	6	0.0 - 5.0	0	Glacial Till & Cobbles	Row 3 (Right)	3/21/2025
	R2	51.0 - 54.0	2.5 - 3.0	83	0.0 - 3.0	0	METALIMESTONE	Row 4	3/21/2025
	R3	54.0 - 55.5	1.5 - 1.5	100	0.0 - 1.5	0	METALIMESTONE	Row 4	3/21/2025

Notes:

1. "Box row" indicates the section of the box where the core is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.
3. Data for Rows 1, 2, and part of Row 3 is not described as they are samples from WIN 027270.00.

Prepared By: RJN/LDN
Checked By: LMP
Reviewed By: MEL

APPENDIX C

Laboratory Testing Results

Client:	WSP USA, Inc.			
Project:	MEDOT N. Bridge Bundle			
Location:	ME		Project No:	GTX-320898
Boring ID:	---	Sample Type: ---	Tested By:	ajl
Sample ID:	---	Test Date: 04/25/25	Checked By:	ank
Depth :	---	Test Id: 812367		

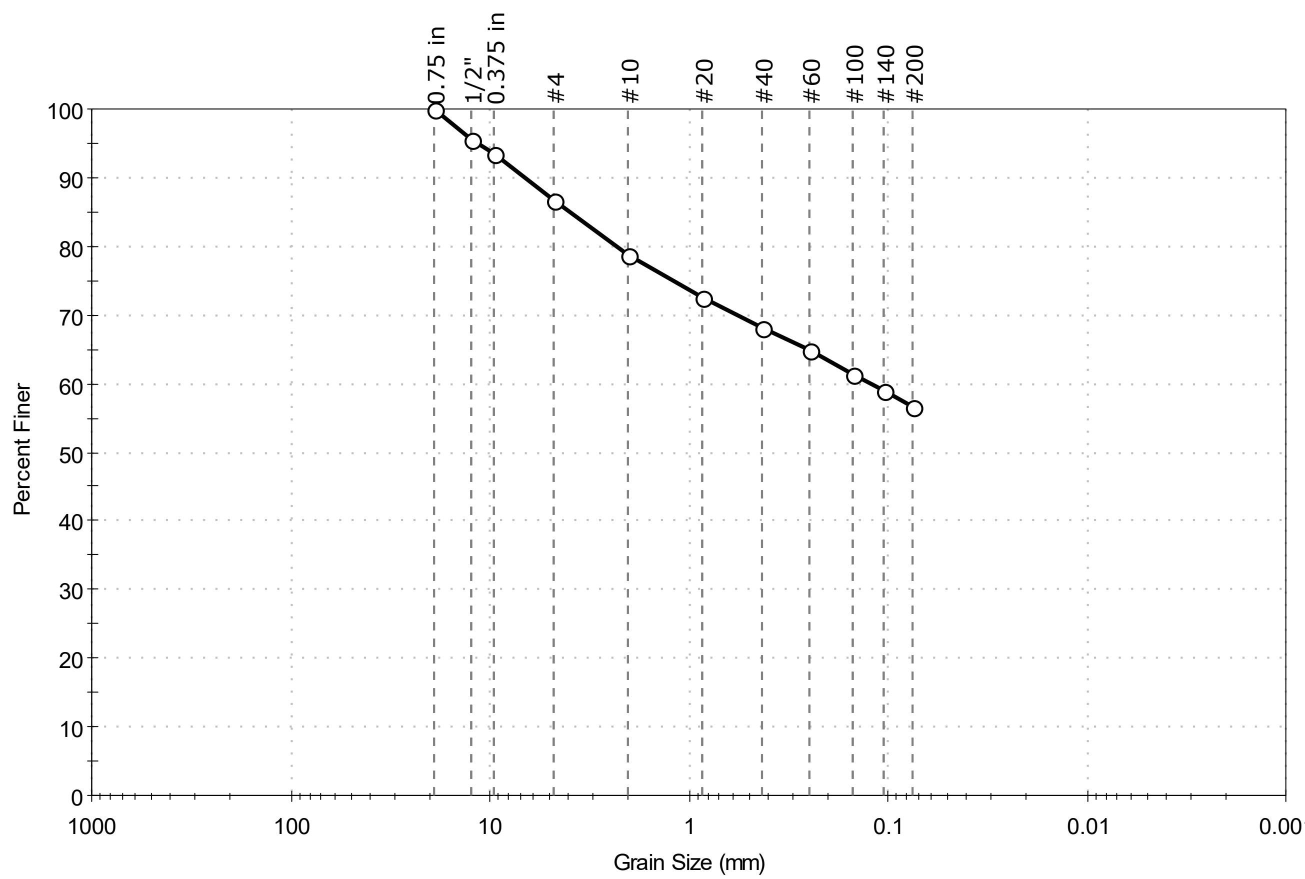
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-BSB-101	S- 3 (3D)	5.0-7.0 ft	Moist, brown sandy silt	11.1
BB-BSB-101	S- 5 (5D)	10.0-12.0 ft	Moist, olive brown sandy silt with gravel	11.5
BB-BSB-101	S- 9 (9D)	30.0-32.0 ft	Moist, grayish brown silt	46.8
BB-BSB-101	S- 11 (11D)	40.0-42.0 ft	Moist, brownish gray silt	40.5
BB-BSB-102	S- 3 (3D)	5.0-7.0 ft	Moist, olive brown silty sand with gravel	11.2
BB-BSB-102	S- 4 (4D)	7.0-9.0 ft	Moist, olive brown silty sand with gravel	11.7
BB-BSB-102	S- 6 (6D)	15.0-17.0 ft	Moist, light brown gravel with sand	11.9
BB-BSB-102	S- 7 (7D)	20.0-22.0 ft	Moist, olive brown silt with sand	70.3
BB-BSB-102	S- 8 (8D)	25.0-27.0 ft	Moist, gray silt with gravel	22.8
BB-BSB-102	S- 9 (9D)	30.0-32.0 ft	Moist, grayish brown silt	35.7

Notes: Temperature of Drying : 110º Celsius

Client: WSP USA, Inc.			
Project: MEDOT N. Bridge Bundle			
Location: ME		Project No:	GTX-320898
Boring ID: BB-BSB-101	Sample Type: Jar	Tested By:	ajl
Sample ID: S-3 (3D)	Test Date: 04/25/25	Checked By:	ank
Depth : 5.0-7.0 ft	Test Id: 812371		
Test Comment: ---			
Visual Description: Moist, brown sandy silt			
Sample Comment: ---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	13.3	30.0	56.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
1/2"	12.50	96		
0.375 in	9.50	94		
#4	4.75	87		
#10	2.00	79		
#20	0.85	73		
#40	0.42	68		
#60	0.25	65		
#100	0.15	61		
#140	0.11	59		
#200	0.075	57		

Coefficients

D_{85} =3.9630 mm	D_{30} =N/A
D_{60} =0.1230 mm	D_{15} =N/A
D_{50} =N/A	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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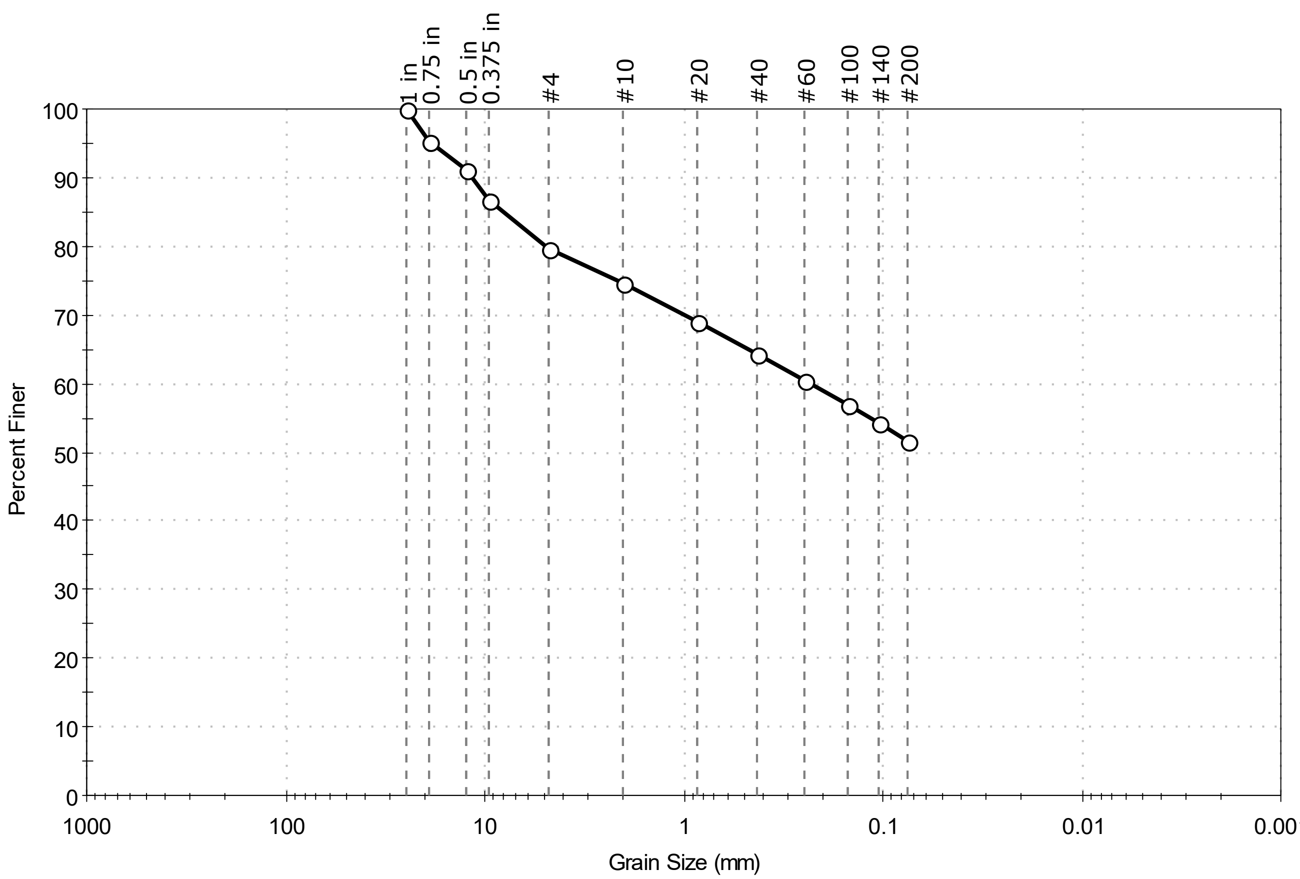
GeoTesting EXPRESS
A Sercel Business

Client:	WSP USA, Inc.		
Project:	MEDOT N. Bridge Bundle		
Location:	ME	Project No:	GTX-320898
Boring ID:	BB-BSB-101	Sample Type: Jar	Tested By: ajl
Sample ID:	S-5 (5D)	Test Date: 04/24/25	Checked By: ank
Depth :	10.0-12.0 ft	Test Id: 812372	
Test Comment:	---		
Visual Description:	Moist, olive brown sandy silt with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.3	28.1	51.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	95		
0.5 in	12.50	91		
0.375 in	9.50	87		
#4	4.75	80		
#10	2.00	75		
#20	0.85	69		
#40	0.42	64		
#60	0.25	61		
#100	0.15	57		
#140	0.11	54		
#200	0.075	52		

Coefficients

D_{85} =8.0554 mm D_{30} =N/A
D_{60} =0.2302 mm D_{15} =N/A
D_{50} =N/A D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM   N/A

AASHTO   Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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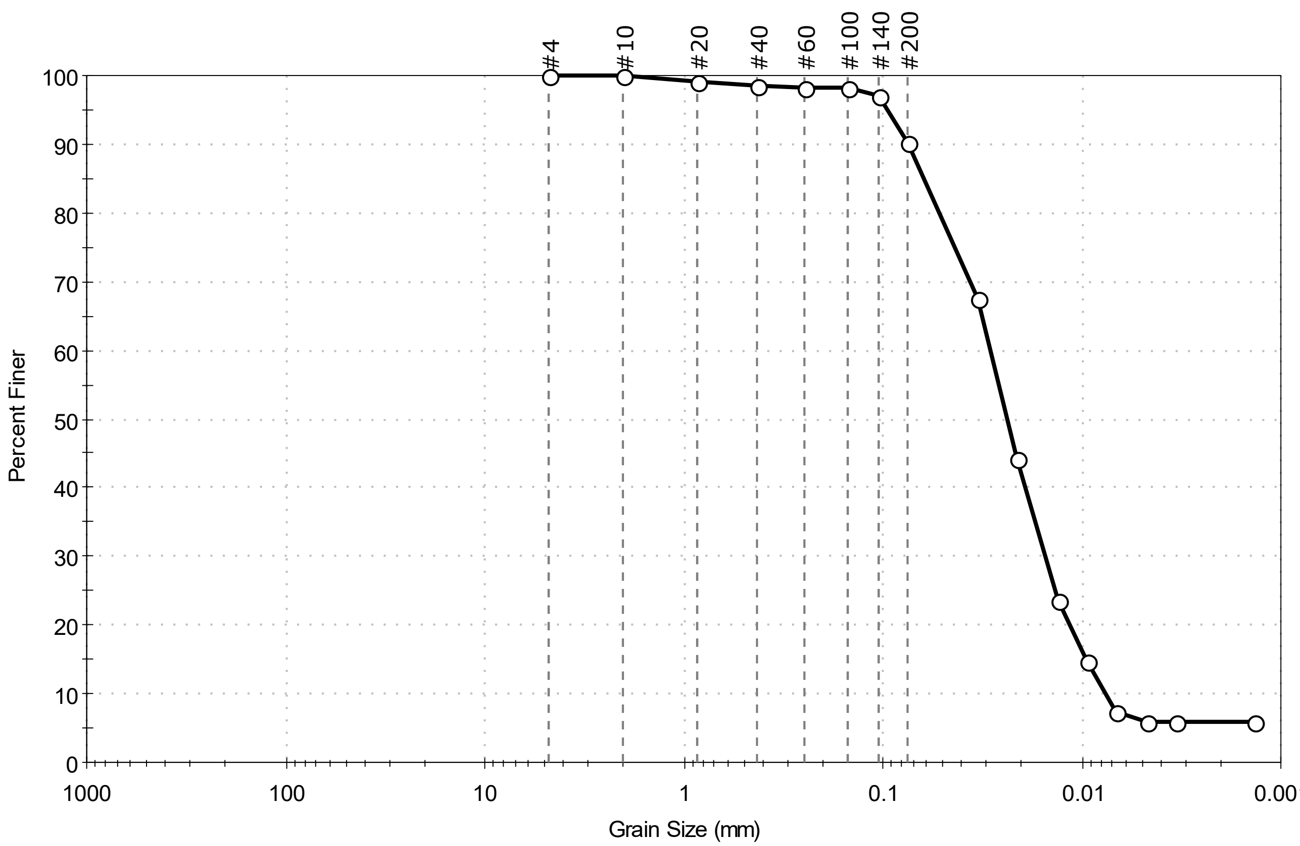
GeoTesting EXPRESS
A Sercel Business

Client: WSP USA, Inc.			
Project: MEDOT N. Bridge Bundle			
Location: ME		Project No:	GTX-320898
Boring ID: BB-BSB-101	Sample Type: Jar	Tested By:	ajl
Sample ID: S-9 (9D)	Test Date: 04/25/25	Checked By:	ank
Depth : 30.0-32.0 ft	Test Id: 812376		
Test Comment: ---			
Visual Description: Moist, grayish brown silt			
Sample Comment: ---			

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	9.7	90.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	99		
#60	0.25	98		
#100	0.15	98		
#140	0.11	97		
#200	0.075	90		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0336	68		
---	0.0210	44		
---	0.0131	24		
---	0.0094	15		
---	0.0067	7		
---	0.0047	6		
---	0.0033	6		
---	0.0014	6		

Coefficients

D_{85} =0.0621 mm	D_{30} =0.0152 mm
D_{60} =0.0288 mm	D_{15} =0.0095 mm
D_{50} =0.0236 mm	D_{10} =0.0075 mm
C_u =3.840	C_c =1.070

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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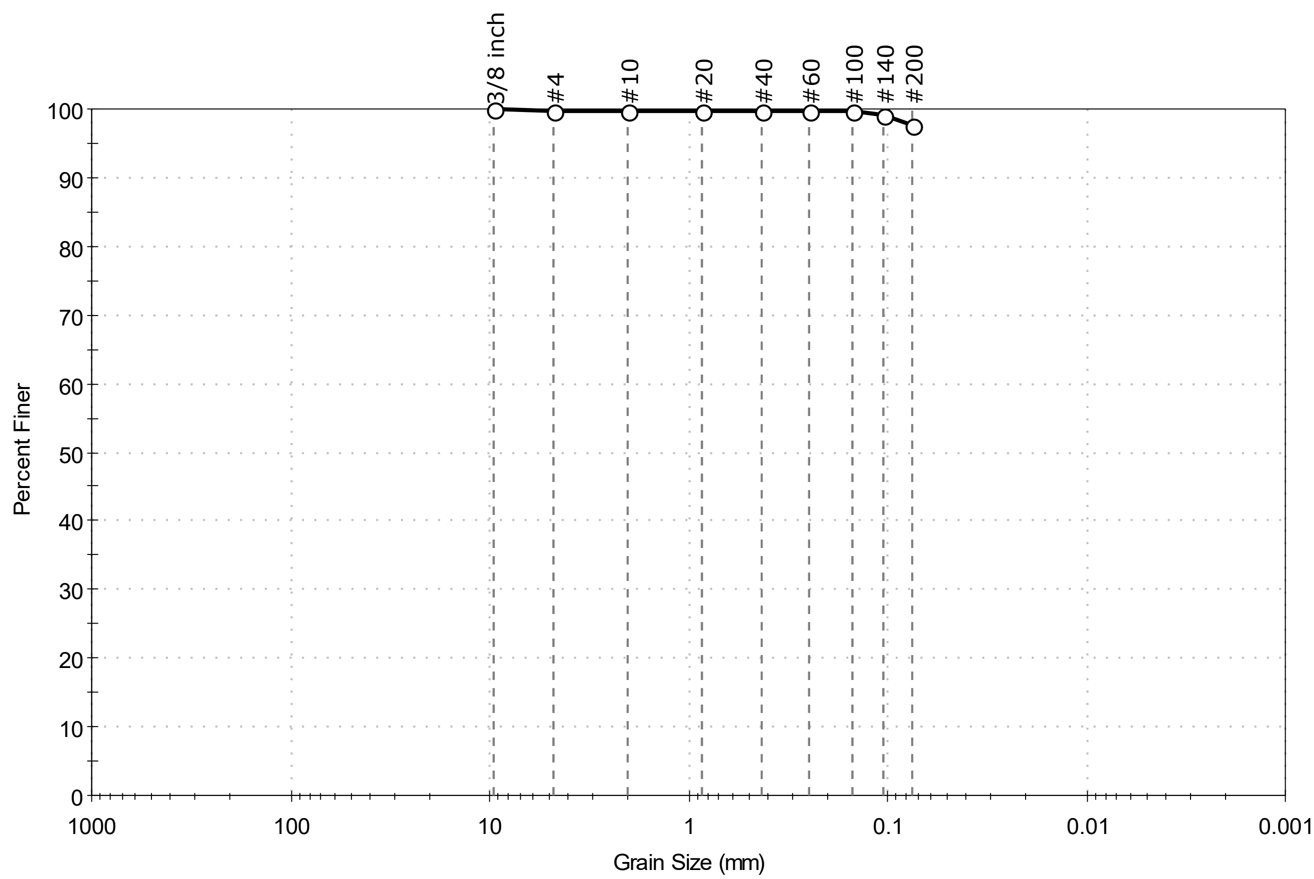
Client:	WSP USA, Inc.				
Project:	MEDOT N. Bridge Bundle				
Location:	ME			Project No:	GTX-320898
Boring ID:	BB-BSB-101	Sample Type:	Jar	Tested By:	ajl
Sample ID:	S-11 (11D)	Test Date:	04/24/25	Checked By:	ank
Depth :	40.0-42.0 ft	Test Id:	812373		
Test Comment:	---				
Visual Description:	Moist, brownish gray silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.2	2.2	97.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/8 inch	9.50	100		
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#140	0.11	99		
#200	0.075	98		

Coefficients

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

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

D_{50} =N/A D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

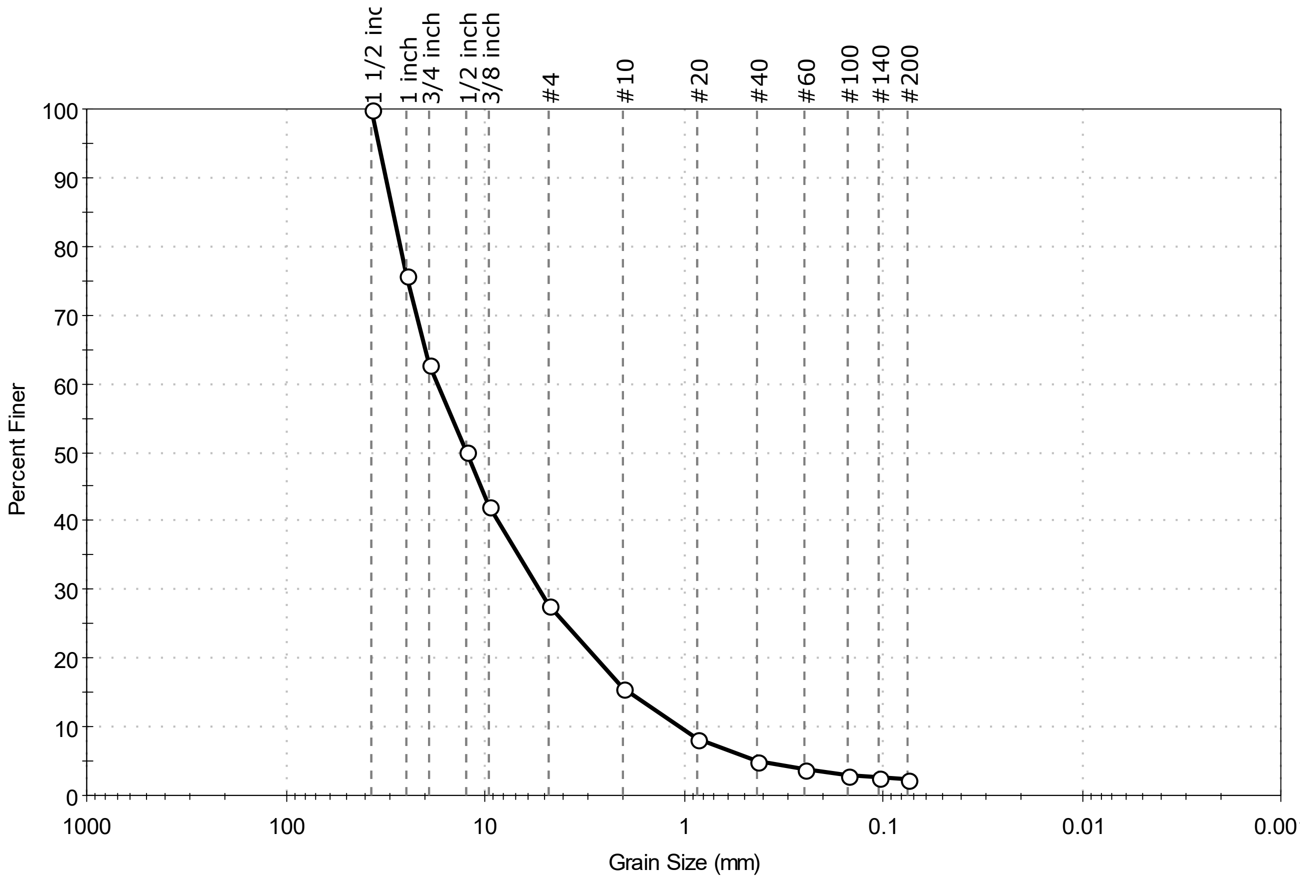
Sand/Gravel Hardness : ---

Client: WSP USA, Inc.
Project: MEDOT N. Bridge Bundle
Location: ME
Project No: GTX-320898

Boring ID: BB-BSB-101
Sample Type: Jar
Tested By: ajl
Sample ID: S-13 (13D)
Test Date: 04/25/25
Checked By: ank
Depth : 50.0-52.0 ft
Test Id: 812374

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	72.1	25.6	2.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	76		
3/4 inch	19.00	63		
1/2 inch	12.50	50		
3/8 inch	9.50	42		
#4	4.75	28		
#10	2.00	16		
#20	0.85	8		
#40	0.42	5		
#60	0.25	4		
#100	0.15	3		
#140	0.11	3		
#200	0.075	2.3		

Coefficients

D_{85} =29.1480 mm
D_{30} =5.2723 mm
D_{60} =17.2835 mm
D_{15} =1.8305 mm
D_{50} =12.4911 mm
D_{10} =1.0249 mm
C_u =16.864
C_c =1.569

Classification

ASTM Well-graded GRAVEL with Sand (GW)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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GeoTesting EXPRESS
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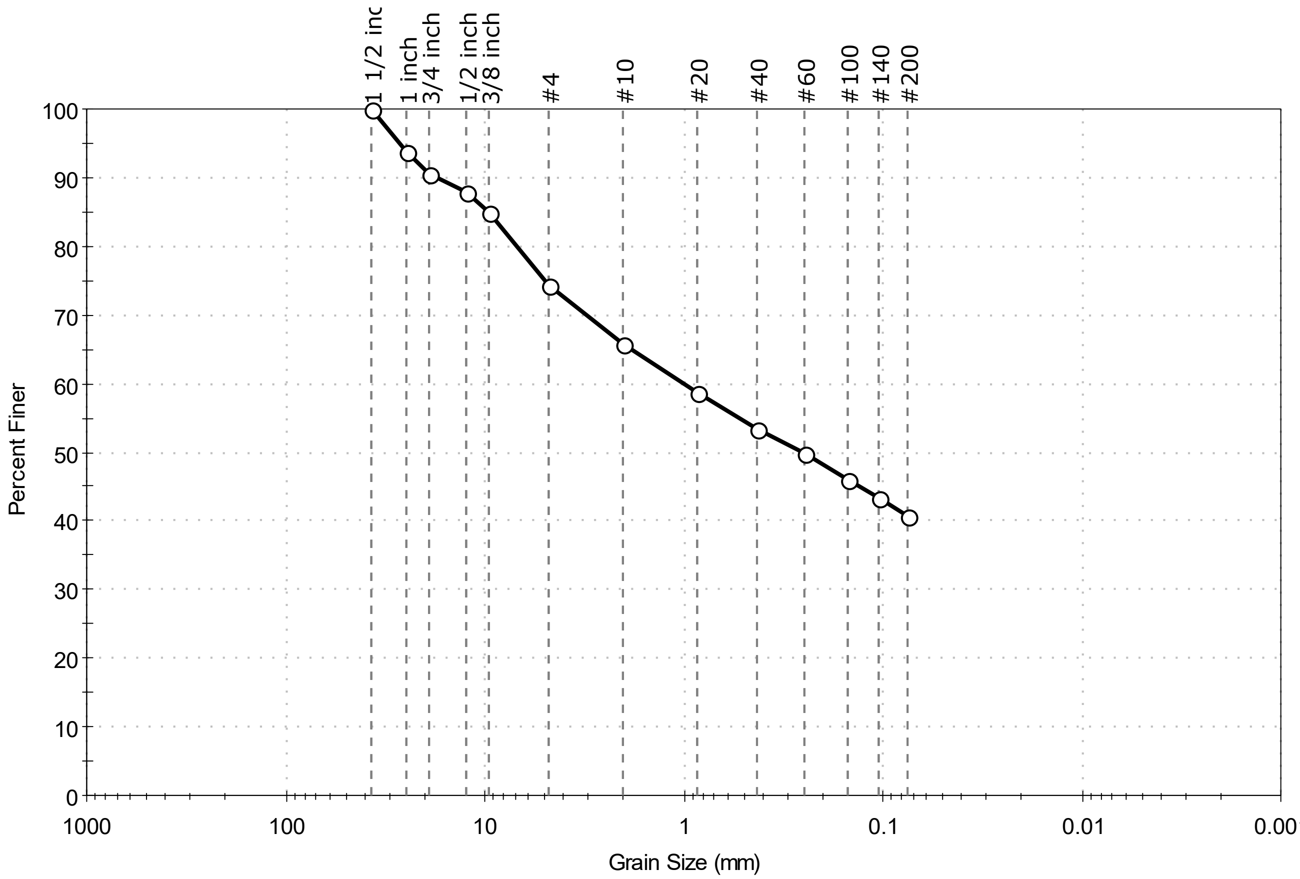
Client: WSP USA, Inc.			
Project: MEDOT N. Bridge Bundle			
Location: ME		Project No:	GTX-320898
Boring ID: BB-BSB-102	Sample Type: Jar	Tested By:	ajl
Sample ID: S-3 (3D)	Test Date: 04/25/25	Checked By:	ank
Depth : 5.0-7.0 ft	Test Id: 812358		
Test Comment: ---			
Visual Description: Moist, olive brown silty sand with gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.6	33.7	40.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	94		
3/4 inch	19.00	91		
1/2 inch	12.50	88		
3/8 inch	9.50	85		
#4	4.75	74		
#10	2.00	66		
#20	0.85	59		
#40	0.42	53		
#60	0.25	50		
#100	0.15	46		
#140	0.11	43		
#200	0.075	41		

Coefficients

D_{85} =9.5289 mm	D_{30} =N/A
D_{60} =0.9995 mm	D_{15} =N/A
D_{50} =0.2568 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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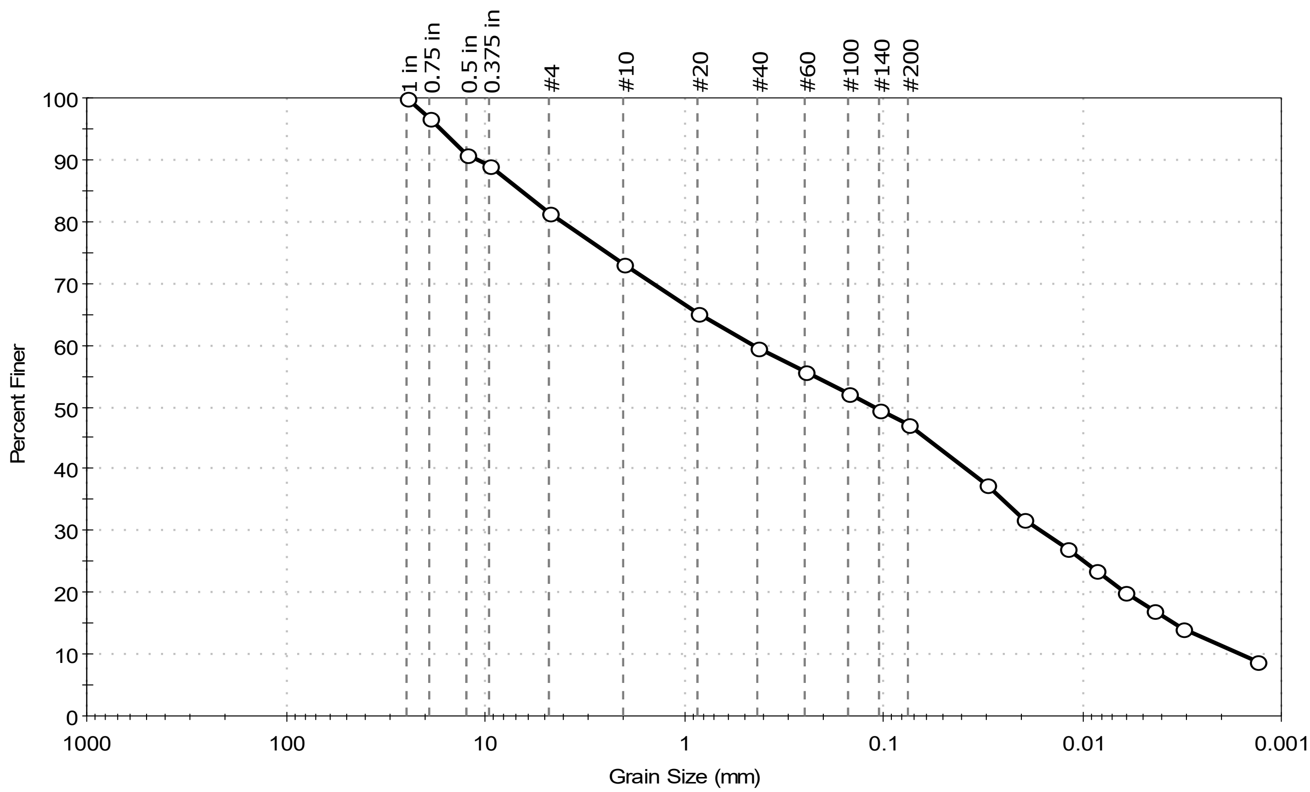
Client:	WSP USA, Inc.		
Project:	MEDOT N. Bridge Bundle		
Location:	ME	Project No:	GTX-320898
Boring ID: BB-BSB-102	Sample Type: Jar	Tested By:	ajl
Sample ID: S-4 (4D)	Test Date: 04/25/25	Checked By:	ank
Depth : 7.0-9.0 ft	Test Id: 812369		

Test Comment: ---

Visual Description: Moist, olive brown silty sand with gravel

Sample Comment: ---

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	18.5	34.2	47.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	97		
0.5 in	12.50	91		
0.375 in	9.50	89		
#4	4.75	82		
#10	2.00	73		
#20	0.85	65		
#40	0.42	60		
#60	0.25	56		
#100	0.15	52		
#140	0.11	50		
#200	0.075	47		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0300	38		
---	0.0199	32		
---	0.0119	27		
---	0.0086	23		
---	0.0062	20		
---	0.0044	17		
---	0.0032	14		
---	0.0013	9		

Coefficients

D_{85} =6.5190 mm	D_{30} =0.0165 mm
D_{60} =0.4432 mm	D_{15} =0.0035 mm
D_{50} =0.1116 mm	D_{10} =0.0016 mm
C_u =277.000	C_c =0.384

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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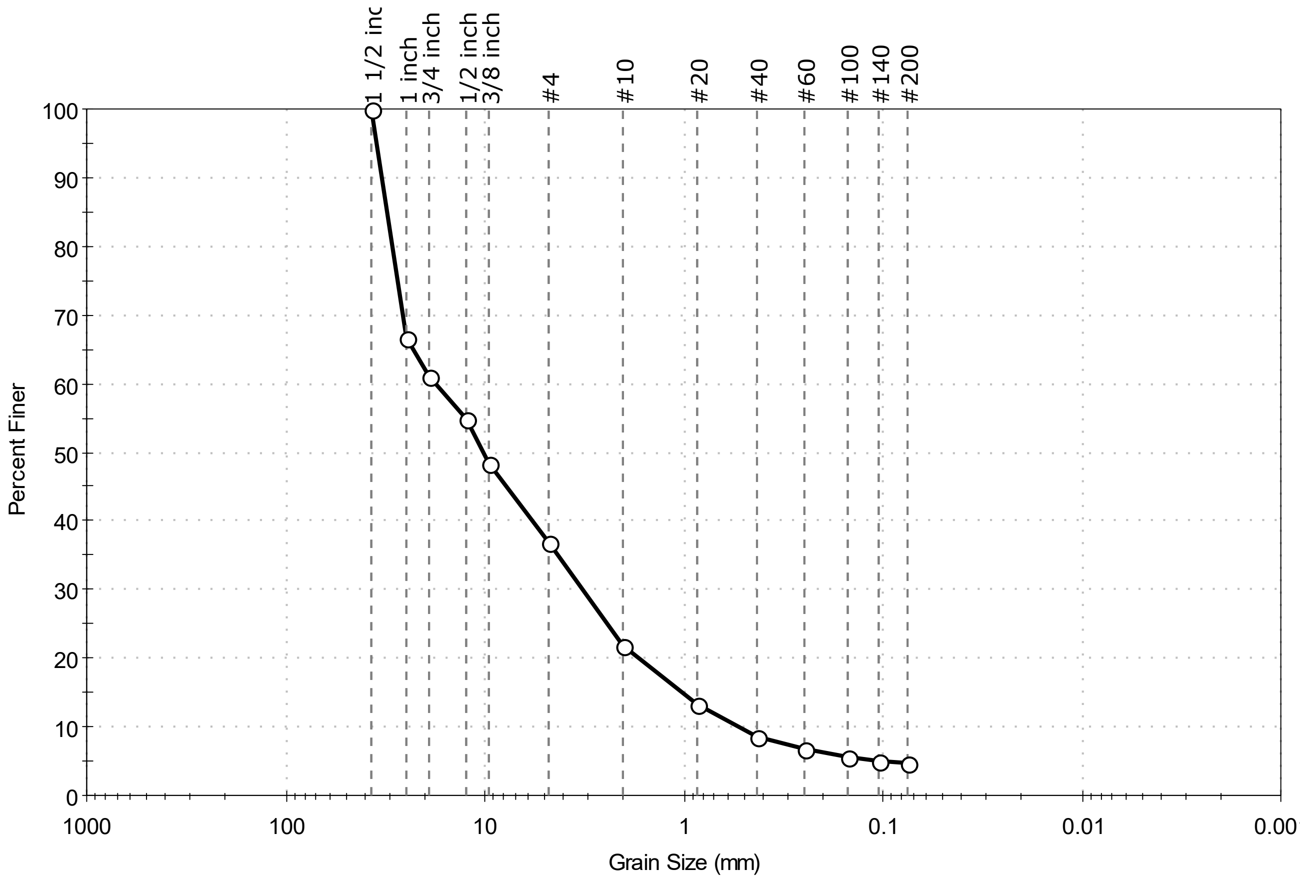
Client:	WSP USA, Inc.				
Project:	MEDOT N. Bridge Bundle				
Location:	ME			Project No:	GTX-320898
Boring ID:	BB-BSB-102	Sample Type:	Jar	Tested By:	ajl
Sample ID:	S-6 (6D)	Test Date:	04/25/25	Checked By:	ank
Depth :	15.0-17.0 ft	Test Id:	812359		

Test Comment: ---

Visual Description: Moist, light brown gravel with sand

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	63.0	32.4	4.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	67		
3/4 inch	19.00	61		
1/2 inch	12.50	55		
3/8 inch	9.50	48		
#4	4.75	37		
#10	2.00	22		
#20	0.85	13		
#40	0.42	9		
#60	0.25	7		
#100	0.15	6		
#140	0.11	5		
#200	0.075	4.6		

Coefficients

D_{85} =31.2562 mm D_{30} =3.1729 mm

D_{60} =17.6510 mm D_{15} =0.9967 mm

D_{50} =10.1717 mm D_{10} =0.5147 mm

C_u =34.294 C_c =1.108

Classification

ASTM Well-graded GRAVEL with Sand (GW)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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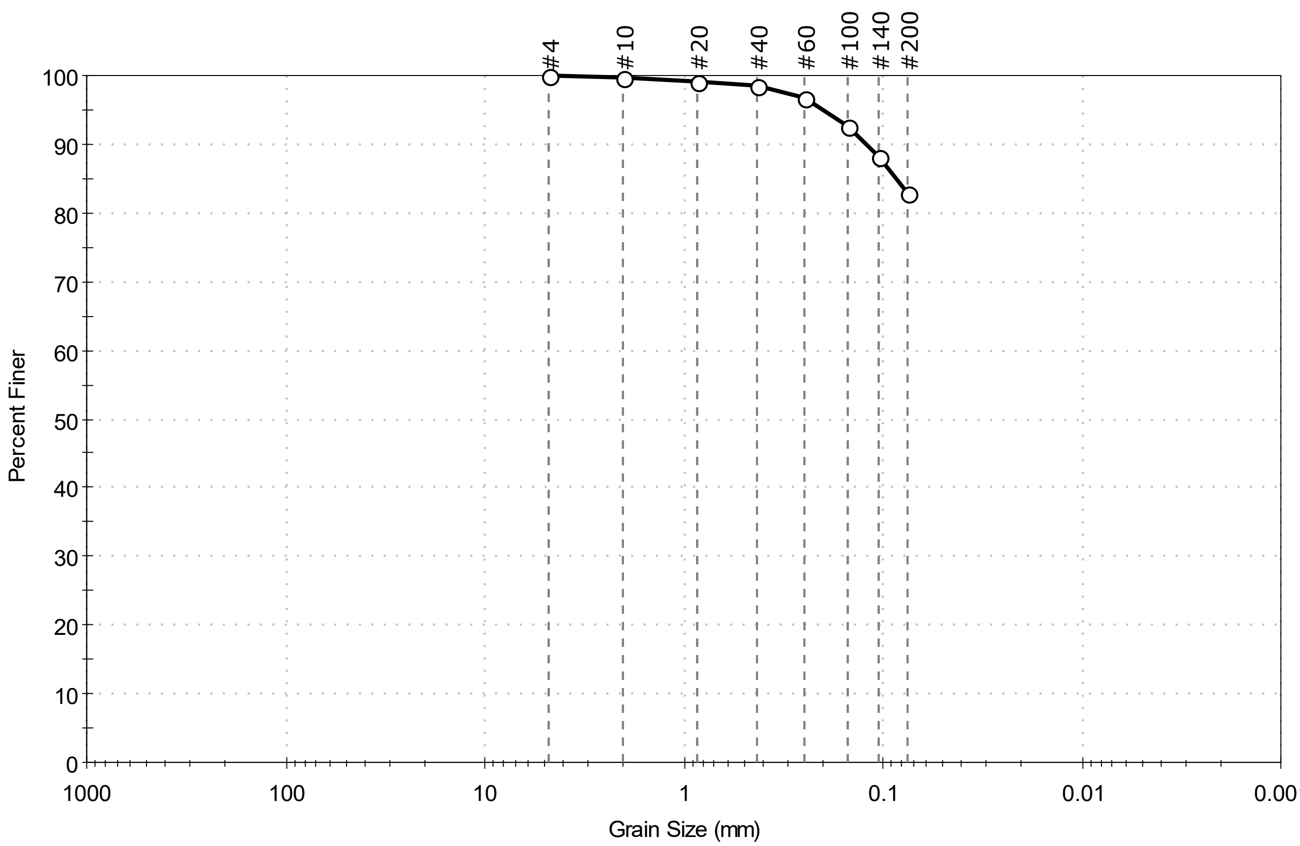
Client:	WSP USA, Inc.		
Project:	MEDOT N. Bridge Bundle		
Location:	ME	Project No:	GTX-320898
Boring ID: BB-BSB-102	Sample Type: Jar	Tested By:	ajl
Sample ID: S-7 (7D)	Test Date: 04/24/25	Checked By:	ank
Depth : 20.0-22.0 ft	Test Id: 812360		
Test Comment:	---		
Visual Description:	Moist, olive brown silt with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	17.2	82.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	98		
#60	0.25	97		
#100	0.15	93		
#140	0.11	88		
#200	0.075	83		

Coefficients

D_{85} =0.0867 mm D_{30} =N/A

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

D_{50} =N/A D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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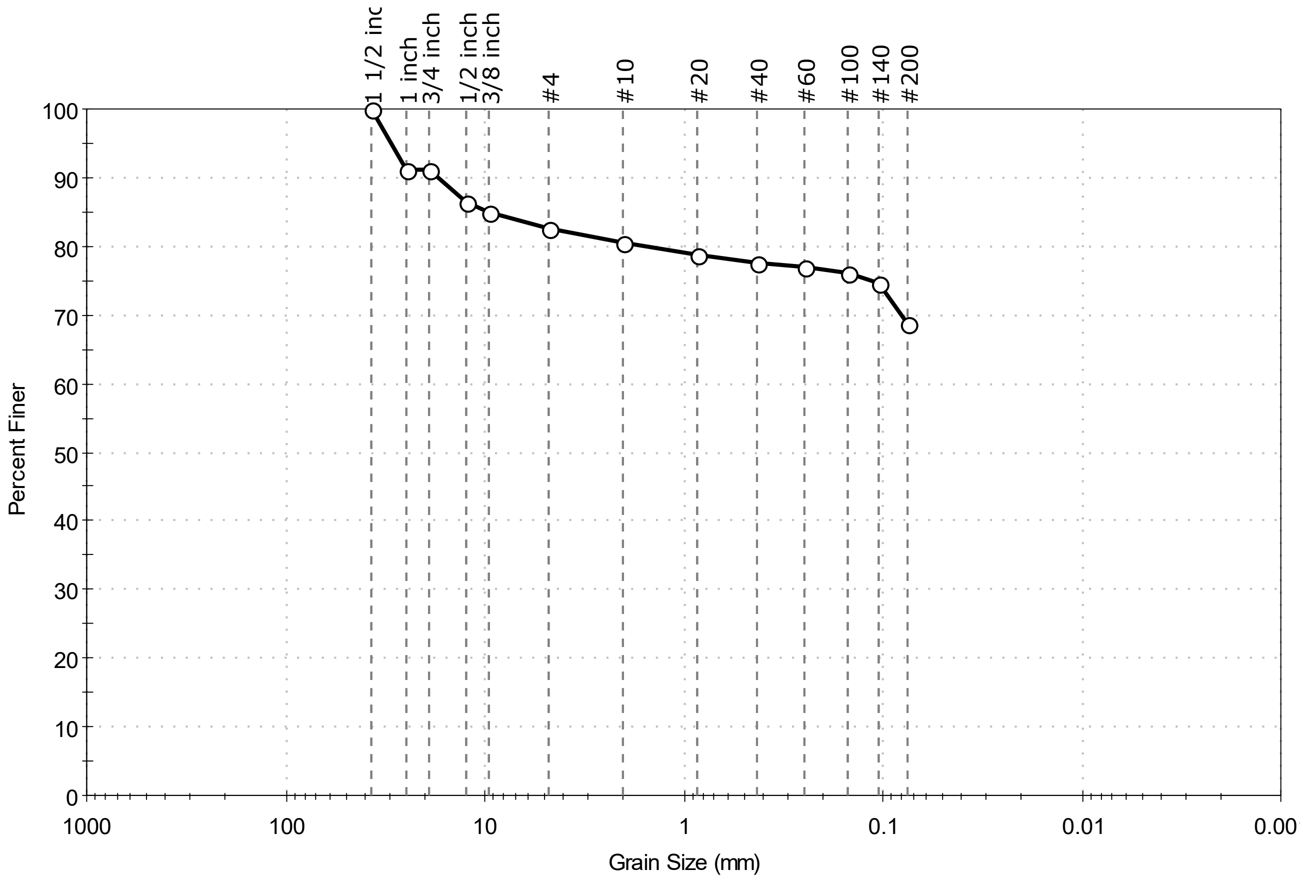
GeoTesting EXPRESS
A Sercel Business

Client:	WSP USA, Inc.				
Project:	MEDOT N. Bridge Bundle				
Location:	ME			Project No:	GTX-320898
Boring ID:	BB-BSB-102	Sample Type:	Jar	Tested By:	ajl
Sample ID:	S-8 (8D)	Test Date:	04/25/25	Checked By:	ank
Depth :	25.0-27.0 ft	Test Id:	812361		

Test Comment: ---
Visual Description: Moist, gray silt with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	17.3	14.1	68.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	91		
3/4 inch	19.00	91		
1/2 inch	12.50	86		
3/8 inch	9.50	85		
#4	4.75	83		
#10	2.00	81		
#20	0.85	79		
#40	0.42	78		
#60	0.25	77		
#100	0.15	76		
#140	0.11	75		
#200	0.075	69		

Coefficients

D_{85} =9.5788 mm
D_{30} =N/A
D_{60} =N/A
D_{15} =N/A
D_{50} =N/A
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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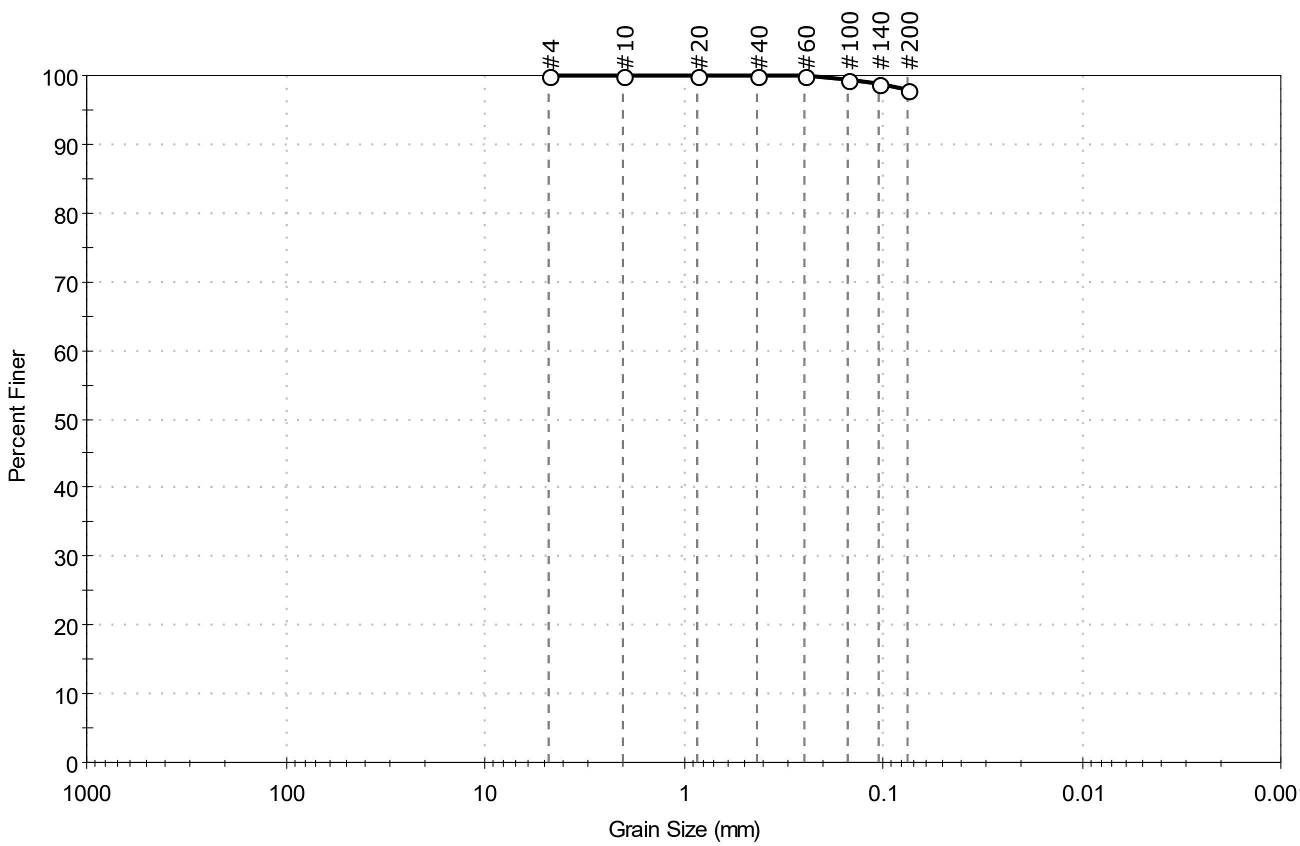
GeoTesting EXPRESS
A Sercel Business

Client:	WSP USA, Inc.		
Project:	MEDOT N. Bridge Bundle		
Location:	ME	Project No:	GTX-320898
Boring ID: BB-BSB-102	Sample Type: Jar	Tested By:	ajl
Sample ID: S-9 (9D)	Test Date: 04/24/25	Checked By:	ank
Depth : 30.0-32.0 ft	Test Id: 812362		
Test Comment:	---		
Visual Description:	Moist, grayish brown silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	2.2	97.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	99		
#140	0.11	99		
#200	0.075	98		

Coefficients

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

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

D_{50} =N/A D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM   N/A

AASHTO   Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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APPENDIX D

Calculations

CALCULATIONS

Date:	7/8/2025	**Made by:**	LDN
Project No.:	US0040947.9162	**Checked by:**	JKW
Subject:	Frost Penetration Depth - Bridge Structures in Fill	**Reviewed by:**	MEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

OBJECTIVE
Determine the depth of frost penetration at the proposed bridge site.

REFERENCES
1. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
2. WSP interpreted subsurface Profile (Figure 3, Preliminary Geotechnical Design Report).
3. WSP geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report).
4. GeoTesting Express laboratory testing results (Appendix C, Preliminary Geotechnical Design Report).

ASSUMPTIONS
1. Based on the available moisture contents in Existing Fill and Silt materials around the existing bridge, it is assumed that the new backfill and existing soils will have a similar behavior for the purpose of frost penetration depth calculations. Thus, average moisture content in Existing Granular Fill and Silt will be used for this evaluation.
2. New fill is assumed to consist of Granular Borrow.

CALCULATION
Follow the procedure in the MaineDOT Bridge Design Guide Section 5.2.1 to determine the depth of frost penetration.

The site location is Bancroft, Maine. Using the Maine Design Freezing Index Map (Reference 1, Figure 5-1), the maximum design freezing index at the site location is 2,000 degree-days.

Based on Reference 2, the proposed bridge will be founded within the proposed fill (Granular and Gravel Borrow) and existing fill which are coarse-grained soils; and silt which is a fine-grained soil.

Based on References 3 and 4, water contents in the upper coarse-grained granular fill are 11.1%, 11.5%, 11.2%, and 11.7%, and in the lower fine-grained silt are 46.8%, 40.5%, 11.9%, 70.3%, 22.8% and 35.7%. Average water content is 11.4% in the coarse-grained fill and 38.0% in the fine-grained silt.

From Reference 1, Table 5-1, with design freezing index = 2,000, coarse-grained soil, and w = 11.4%.

Frost Penetration = 92.8 inches 7.7 feet

CONCLUSIONS
The depth of frost penetration at the proposed bridge site is estimated to be 7.7 feet in areas where new fill associated with bridge foundations will be placed.

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

OBJECTIVE

Determine the seismic site class and acceleration coefficient at the proposed bridge No. 3189 carrying Bancroft Rd over the Smith Brook in Bancroft, Maine.

METHOD

Use the procedure outlined in AASHTO Guide Specifications for LRFD Seismic Bridge Design (Ref. 1) to determine site class and seismic coefficients.

REFERENCES

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 3rd Ed, October 2023.

2. WSP boring logs, included as Appendix A of the Preliminary Geotechnical Design Report.

3. WSP Material properties Analysis titled "NBB Bridge 3189 Geotechnical Material Properties.xlsx"

4. State of Maine Department of Transportation, Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road (Sheets 1 - 7), Dated 7/18/2025 (Draft PIC Plans).

5. Wair, B.R., DeJong, J.T., and Shantz, T. December 2012. Guidelines for Estimation of Shear Wave Velocity Profiles. Pacific Earthquake Engineering Research Center.

6. Braja M. Das, Principles of Foundation Engineering 7th Ed., 2011

7. AASHTO LRFD Bridge Design Specifications, 10th Ed, 2024

8. FHWA. August 2011. Geotechnical Engineering Circular No. 3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual, Rev. 1. Publication No. FHWA-NHI-11-032.

ASSUMPTIONS

1. Shear wave velocities are correlated using average N_{60}-values (corrected only for hammer efficiencies and rod lengths) in the analysis. Average N_{60} values were taken as averages from the specific borings and layers and are based on values presented in Ref. 2.

2. The strata and SPT blow counts encountered at boring BB-BSB-101 was used to evaluate seismic site class for the west abutment. Information from boring BB-BSB-102 was used to evaluate seismic site class for the east abutment.

3. The bottom strata encountered during drilling at each boring was extended to a depth of 100 feet below ground surface.

ATTACHMENTS

1. Results from the AASHTO-USGS Seismic Design Ground Motion Database

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

CALCULATIONS

1. Determine site class per AASHTO LRFD Seismic Bridge Design (Reference 1)

Ref. 1, Table 3.4.2.1-1

Site Class*	
C	Very dense soil or hard clay with 1450 ft/s < $\bar{v}_s \leq$ 2100 ft/s
CD	Dense sand or very stiff clay with 1000 ft/s < $\bar{v}_s \leq$ 1450 ft/s
D	Medium dense sand or stiff clay with 700 ft/s < $\bar{v}_s \leq$ 1000 ft/s
DE	Loose sand or medium stiff clay with 500 ft/s < $\bar{v}_s \leq$ 700 ft/s
E	Very loose sand or soft clay with $\bar{v}_s \leq$ 500 ft/s
F	Peats, organic clays, high plasticity clay, thick soft/medium stiff clays

* Site class A & B pertain to rock and are not included in this table.

The seismic site class will be determined for each abutment using shear wave velocities correlated from average N_{60} values for each soil layer. The AASHTO Guide Specifications (Ref. 1) Article C3.4.2.2 recommend the procedures given in Wair et al. (Ref. 5) to estimate shear wave velocity based on empirical correlations.

$$V_S = 30N_{60}{}^{0.215}\sigma_v'^{\,0.275}$$ (Ref. 5, Table 4.11, All Soils, Eqn. 4.17)

where:

V_S = correlated shear wave velocity in m/s

N_{60} = corrected SPT blow count, not to exceed 100 blows per foot (Ref. 5, page 40)

σ'_v = vertical effective stress in kPa

The soil layering profiles over the top 100 feet below the approach slab of Abutment 1 at Station 2+70.00 and Abutment 2 at Station 3+60.03 are determined using WSP's interpreted subsurface profile (Ref. 3).

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Abutment 1, from 100 feet below the approach slab:

Soil Layer	Elevation (ft)		Unit Weight	σ'_v layer midpoint	σ'_v layer midpoint	Average N_{60}	Correlated V_S	Correlated V_S
	Top	Bottom	(pcf)	(psf)	(kPa)		(m/s)	(ft/s)
Fill	351.3	336.3	125	938	45	28	175	574
Alluvial Sand and Gravel	336.3	322.9	125	1162	56	45	205	673
Alluvial Silt	322.9	302.8	115	2335	112	16	199	651
Glacial Till	302.8	296.3	130	4189	201	63	314	1031
Weathered Bedrock	296.3	295.3	160	6713	321	100	395	1296
Bedrock	295.3	251.3	165	12543	601	100	469	1539

Abutment 2, 100 feet below the approach slab:

Soil Layer	Elevation (ft)		Unit Weight	σ'_v layer midpoint	σ'_v layer midpoint	Average N_{60}	Correlated V_S	Correlated V_S
	Top	Bottom	(pcf)	(psf)	(kPa)		(m/s)	(ft/s)
Fill	350.3	335.3	125	938	45	30	177	582
Alluvial Sand and Gravel	335.3	330.3	125	2188	105	13	187	614
Alluvial Silt	330.3	318.3	115	2684	129	16	206	676
Glacial Till	318.3	300.3	130	4113	197	94	341	1118
Bedrock	300.3	250.3	165	8884	425	100	427	1400

The average $\bar{v}_s$ for the top 100 feet of each soil profile shall be determined as:

$$\overline{v_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$ (Ref. 1, Eq. 3.4.2.2-1)

where:
v_{si} = shear wave velocity of the soil layer (ft/sec)
d_i = thickness of the soil layer (ft)

Per Ref. 1 Article 3.4.2.2, if shear wave velocity is not measured but instead is determined via correlations from SPT blow counts, the resulting uncertainty should be evaluated by estimating $\bar{v}_s$ using each of $\bar{v}_s$, $\bar{v}_s$/1.3, and 1.3 $\bar{v}_s$. If the different average shear wave velocities result in different site classes the most critical seismic site class will be used for the ground motion analysis, per Ref. 1 Section 3.4.2.2.

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Abutment 1:

Soil Layer	d_i (feet)	v_{si} (ft/s)	d_i/v_{si}	$v_{si}/1.3$ (ft/s)	$d_i/(v_{si}/1.3)$	$1.3v_{si}$ (ft/s)	$d_i/(1.3v_{si})$
Fill	15.0	574	0.026	441	0.034	746	0.020
Alluvial Sand and Gravel	13.4	673	0.020	517	0.026	875	0.015
Alluvial Silt	20.1	651	0.031	501	0.040	847	0.024
Glacial Till	6.5	1031	0.006	793	0.008	1340	0.005
Weathered Bedrock	1.0	1296	0.001	997	0.001	1685	0.001
Bedrock	44.0	1539	0.029	1184	0.037	2001	0.022
Sum:	100.0		0.11		0.15		0.09
$\bar{v}_s$ =			888		683		1155
Seismic Site Class =			D		**DE**		CD

Abutment 2:

Soil Layer	d_i (feet)	v_{si} (ft/s)	d_i/v_{si}	$v_{si}/1.3$ (ft/s)	$d_i/(v_{si}/1.3)$	$1.3v_{si}$ (ft/s)	$d_i/(1.3v_{si})$
Fill	15.0	582	0.026	448	0.033	757	0.020
Alluvial Sand and Gravel	5.0	614	0.008	472	0.011	798	0.006
Alluvial Silt	12.0	676	0.018	520	0.023	879	0.014
Glacial Till	18.0	1118	0.016	860	0.021	1454	0.012
Bedrock	50.0	1400	0.036	1077	0.046	1820	0.027
Sum:	100.0		0.10		0.13		0.08
$\bar{v}_s$ =			966		743		1256
Seismic Site Class =			D		**D**		CD

2. Determine acceleration coefficients.

As per Ref. 1 Section 3.4.1, the AASTO-USGS Seismic Design Ground Motion database shall be used to determine the design response spectral, A_s, S_s, and S_1. A_s is taken as the spectral ground motion acceleration at a response period of 0 seconds. S_s is taken as the spectral ground motion acceleration at a response period of 0.2 seconds, and S_1 is taken as the spectral ground motion acceleration at a response period of 1 second.

wsp

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Abutment 1:

A_s =	0.093	g	Horizontal peak ground acceleration coefficient with 1.5% probability of exceedance in 75 years (Ref. 1 Section 3.4)
S_S $S_{0.2}$=	0.218	g	Horizontal response spectral acceleration coefficient at 0.2-second period with 1.5% probability of exceedance in 75 years and 5% critical damping (Ref. 1 Section 3.4)
S_1 =	0.090	g	Horizontal response spectral acceleration coefficient at 1.0-second period with 1.5% probability of exceedance in 75 years and 5% critical damping (Ref. 1 Section 3.4)

Abutment 2:

A_s =	0.095	g	Horizontal peak ground acceleration coefficient with 1.5% probability of exceedance in 75 years (Ref. 1 Section 3.4)
S_S $S_{0.2}$=	0.233	g	Horizontal response spectral acceleration coefficient at 0.2-second period with 1.5% probability of exceedance in 75 years and 5% critical damping (Ref. 1 Section 3.4)
S_1 =	0.093	g	Horizontal response spectral acceleration coefficient at 1.0-second period with 1.5% probability of exceedance in 75 years and 5% critical damping (Ref. 1 Section 3.4)

3. Determine the Seismic Design category

1 Table 3.5-1 using the seismic design parameter S_{D1}.

Per AASHTO (Ref. 1) Section 3.5, SD1 shall be taken as the maximum of:

$0.9 * TS_a$ and S_1 Ref. 1 Section 3.5

Where: T = The spectral response period (seconds)

S_a = Spectral ground motion acceleration for spectral response period, a

Abutment 1, S_1 = 0.090 g, From Part 2

Abutment 2, S_1 = 0.093 g, From Part 2

For sites where: $\grave{v}_s > 1450$ ft/s Calculate S_{D1} for response periods 1 through 2 seconds

$\grave{v}_s \leq 1450$ ft/s Calculate S_{D1} for response periods 1 through 5 seconds and not less than 100 percent of S_a at 1 second

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Abutment 1:	$\bar{v}_s$ =	888	ft/s	From Part 1
Abutment 2:	$\bar{v}_s$ =	966	ft/s	From Part 1

Use the results form the AASHTO-USGS Seismic Ground Motion Database (Attachment 1) to calculate S_{D1}

Abutment 1:

Response Period, T (sec)	Ground Motion Acceleration, S_a (g)	Design Spectral Response Acceleration, S_{D1} (g)
1	0.0904	0.08136
1.5	0.0566	0.07641
2	0.0389	0.07002
3	0.0214	0.05778
4	0.0137	0.04932
5	0.0097	0.04365

S_{D1} = 0.090

Abutment 2:

Response Period, T (sec)	Ground Motion Acceleration, S_a (g)	Design Spectral Response Acceleration, S_{D1} (g)
1	0.0927	0.08343
1.5	0.0579	0.078165
2	0.0397	0.07146
3	0.0218	0.05886
4	0.0142	0.05112
5	0.0101	0.04545

S_{D1} = 0.093

Ref. 1 Table 3.5-1

Value of S_{D1}	SDC
$S_{D1} \leq 0.15$	A
$0.15 \leq S_{D1} \leq 0.30$	B
$0.30 \leq S_{D1} \leq 0.50$	C
$0.50 \leq S_{D1}$	D

Abutment 1 Seismic Design Category (SDC) = A

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Abutment 2 Seismic Design Category (SDC) = A

4. Determine the slope height adjusted peak ground acceleration using the method described in FHWA GEC No.3 Section 6.2.2 (Ref. 8)

$k_{av} = \alpha * k_{max}$ Ref. 8, Eq. 6-2

Where:

$k_{max} = PGA * F_{PGA}$ Ref. 8, Eq. 6-1

$\alpha = 1 + 0.01 * H * (0.5 * \beta - 1)$ Ref. 8, Eq. 6-3

Where:

H = height of the slope in the direction of sliding

$\beta = F_v * S_1 / k_{max}$ Ref. 8, Eq. 6-4

$A_s = PGA * F_{PGA}$ Ref. 8, Page 3-62

$k_{max} = A_s$ Combining Ref. 8 Eq. 6-1 and Page 3-62

$S_{D1} = F_v * S_1$ Ref. 8 Table 6-1

$\beta = S_{D1} / A_s$ Combining Ref. 8 Eq. 6-4 and Table 6-1

$$k_{av} = [1 + 0.01 * H * (0.5 * (S_{D1}/A_s) - 1)] * A_s$$

	Abut. 1	Abut. 2	
H (feet) =	22.7	22.5	
β =	0.97	0.97	
α =	0.88	0.88	Ref. 8, Eq. 6-3
k_{av} =	0.082	0.084	Ref. 8, Eq. 6-2

CONCLUSIONS

Using WSP's 2024 boring data, the seismic site class was determined to be Site Class DE for Abutment 1 and D for Abutment 2, and the seismic design category was determined to be A for both abutments. Seismic parameters are summarized in the table below.

Seismic Parameters							
Abutment	A_s	k_{av}	S_s	S_1	S_{D1}	Site Class	SDC
Abutment 1	0.093	0.082	0.218	0.090	0.090	DE	A

CALCULATIONS

Date:	8/6/2025	**Made by:**	JKW
Project No.:	US0040947.9162	**Checked by:**	RJN
Subject:	Seismic Site Class Analysis	**Reviewed by:**	JEL
Project Short Title:	Bancroft Road Bridge No. 3169 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Abutment 2	0.095	0.084	0.233	0.093	0.093	D	A

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

OBJECTIVE

Determine the global stability factor of safety for proposed conditions at the proposed bridge No. 3189 carrying Bancroft Road over Smith Brook in Bancroft TWP, Maine. The analysis considers two water table scenarios: Q_{50} and $Q_{1.1}$ flooding.

REFERENCES

1. WSP Soil and Rock Exploration Logs, included as Appendix A of the Preliminary Geotechnical Design Report.
2. WSP Corrected N_{60}-value to friction angle correlation titled "NBB Bridge 3189 Geotechnical Material Properties.xlsx"
3. WSP Interpretive subsurface profile, included as Figure 3 of the of the Preliminary Geotechnical Design Report.
4. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
5. Rocscience Slide2 Software Package Version 9.038 64bit, build date 2/28/2025.
6. American Association of State Highway and Transportation Officials (AASHTO). 2024. LRFD Bridge Design Specifications, 10th Edition.
7. WSP Seismic Site Class Analysis titled "Seismic Site Class US0040947.9162 Smith Brook Bridge 3189.pdf", Dated 8/6/2025.
8. FHWA. August 2011. Geotechnical Engineering Circular No. 3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual, Rev. 1. Publication No. FHWA-NHI-11-032.
9. State of Maine Department of Transportation, Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Dated February 5, 2026 (60% Plans).
10. Hoyle Tanner, Abutment Loading Calculations, provided in email titled "Bridge 3189 updated pile loads", received by WSP via email on March 13th, 2026.
11. WSP Abutment Pile Analysis for North Bridge Bundle Bridge #3189 titled "NBB Bridge 3189 Smith Brook Bridge Abutment Pile Design.pdf", Dated 3/19/2026.

ATTACHMENTS

1. Slide2 material properties.
2. Slide2 Abutment 1 and Abutment 2 Q_{50} Scenario output figures
3. Slide2 Abutment 1 and Abutment 2 $Q_{1.1}$ Scenario output figures
4. Slide 2 Abutment 1 and Abutment 2 Pile detail output figures

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

ASSUMPTIONS

1. Circular surface searches were analyzed using the Spencer method with an auto refine search. Non-circular surface searches were analyzed using the Spencer method and cuckoo search with surface altering optimization. Non-circular surface searches for the foreslope analysis were analyzed using the Spencer method and Simulated Annealing search with surface altering optimization.

2. As per AASTHTO LRFD (Ref. 6, Section 11.6.3.7): a minimum allowable factor of safety (FS) of 1.3 is used where the geotechnical parameters and subsurface stratigraphy are well defined, corresponding to the recommended earth slope global stability resistance factor of ϕ = 0.75; and a minimum allowable FS of 1.5 is used where the geotechnical parameters and subsurface stratigraphy are highly variable or based on limited information, corresponding to the recommended earth slope global stability resistance factor of ϕ = 0.65. Based on the limited number of borings and samples characterized, as well as the presence of a structural element, FS ≥ 1.5 for static loading is recommended for potential failure surfaces on the abutment slopes parallel to the roadway, while FS ≥ 1.3 for static loading is recommended for potential failure surfaces on the abutment foreslopes.

3. Live load surcharge loading is taken as 2 feet of soil (H_{eq} = 2) regardless of abutment height for the case of an abutment with a structural approach slab per Section 5.4.1.7C and Section 3.6.8 of Ref 4: 2 x 125 pcf (granular borrow) = 250 pcf.

4. For pseudo-static models, a horizontal seismic load of 0.5 x k_{av} was applied in the direction of the slope failure as per FHWA GEC-3 2011 (Ref. 8, Section 6.2.2). The value of k_{av} (height adjusted horizontal peak ground acceleration coefficient scaled by site class factor) was estimated in Ref. 7.

5. As per FHWA Design Guide (Ref. 8) a minimum allowable factor of safety (FS) of 1.1 is recommended for pseudo-static conditions. For preliminary evaluations, a FS ≥ 1.1 for pseudo-static loading is recommended for potential failure surfaces on the abutment slopes parallel to the roadway.

6. Two water table scenarios were analyzed: water table corresponding to the Q_{50} and $Q_{1.1}$ flooding reported in Ref.9.

7. Ground surface elevations for the proposed final conditions were obtained from the 60% Plans (Ref. 9).

8. Subsurface stratigraphy was assumed by interpolating subsurface information from the nearest boring locations onto the cross-section and profile (Ref. 1 and 3).

9. The abutment H-piles, their material properties, and their spacing were calculated (Reference 11) and used as the "Pile/Micro Pile" support type in the Slide2 model.

10. The bottom of Abutment 1 and Abutment 2 are at EL 340.4 feet with 1 feet of Granular Borrow below to EL 339.4 feet.

11. It is assumed the failures will not occur through the approach slabs. Thus, the cohesion value for the Approach Slab was set to prevent failure surfaces from passing through the approach slab.

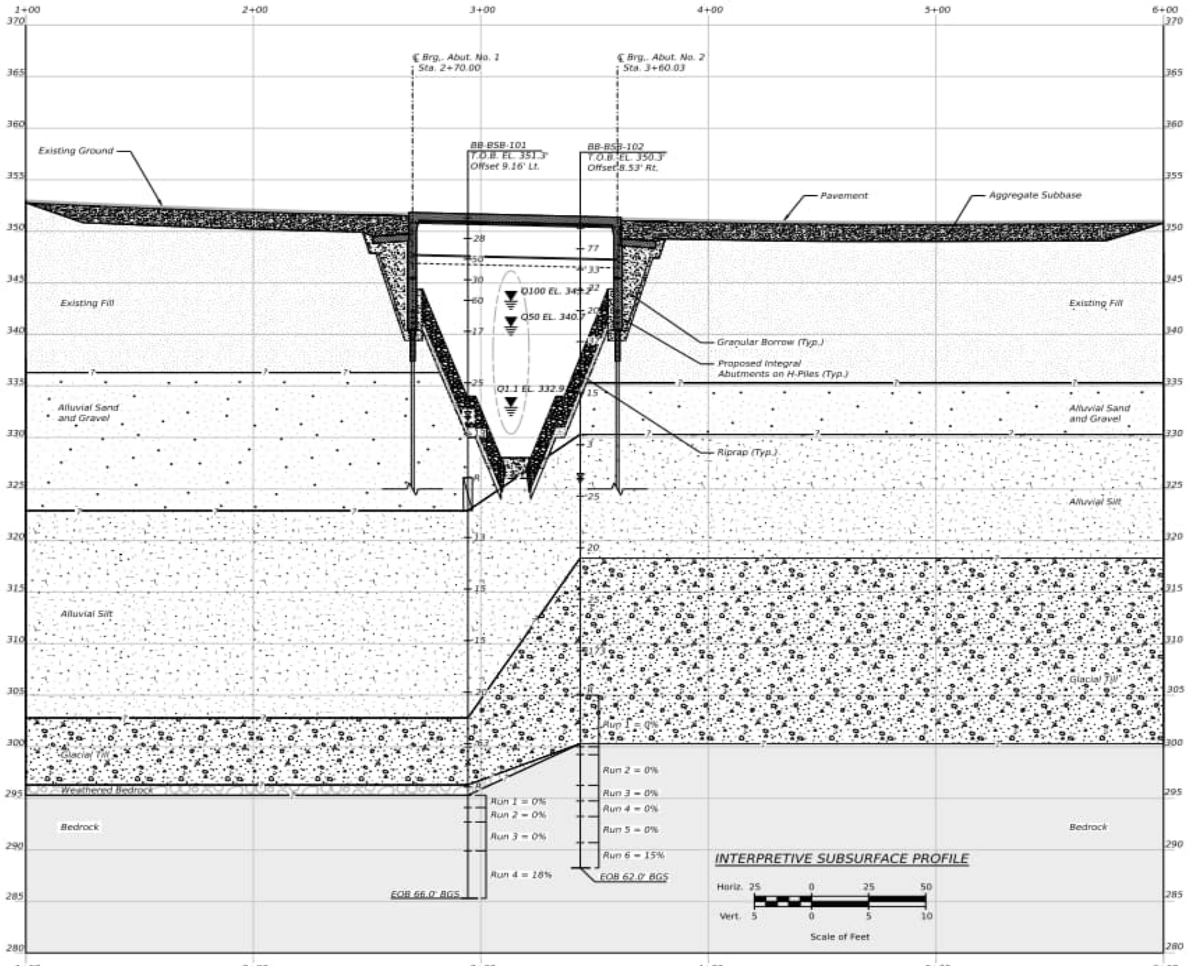
12. 10% of the normal shear strength of the HP 14x89 piles is used in the Slide2 model. Full nominal shear strength of the piles adds an extensive amount of shear resistance resulting in irregular non-circular failure surfaces.

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

CALCULATION

Analyze slope stability based on the proposed and existing conditions shown in the 60% Plans (Ref. 9) along the Bancroft Road centerline profile from STA. 1+00 to 6+00.



Bancroft Road Interpretive Subsurface Profile (Ref. 3)

A. The material properties selected for use in the Slide2 models are shown in the table below and in Attachment 1 figures (Figure 1).

B. The design unit weight for the Fill (Granular Borrow and Aggregate Subbase) material was taken from Ref. 4 Table 3-3. The Gravel Borrow unit weight was used for aggregate subbase. The design unit weights for the other soil layers were determined using typical values, engineering judgement, and knowledge gained from previous experience.

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

C. The friction angles for the in-situ soils were correlated from SPT N60 values using Peck, Hanson, & Thornburn (1974), with correction factors C_N from Liao and Whitman (1986), Skempton (1986), and Peck et al. (1974) using in-situ vertical effective stress. The friction angle for the Fill (Gravel Borrow, Granular Borrow, and Aggregate Subbase) material was taken from Ref. 4, Table 3-3. The gravel borrow friction angle was used for aggregate subbase. The friction angles for the Heavy Riprap and Pavement layers were determined from engineering judgement and past experience.

D. The soil layer properties shown below were used to analyze circular surfaces and non-circular surfaces using the Spencer method.

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)
Pavement	140	Mohr-Coulomb	0	45
Fill (Granular Borrow)[1]	125	Mohr-Coulomb	0	32
Fill (Aggregate subbase)[1]	135	Mohr-Coulomb	0	36
Heavy Riprap[1]	140	Mohr-Coulomb	0	42
Existing Fill	125	Mohr-Coulomb	0	36
Alluvial Sand and Gravel	125	Mohr-Coulomb	0	31
Alluvial Silt	115	Mohr-Coulomb	0	31
Glacial Till	130	Mohr-Coulomb	0	36
Bedrock	165	Infinite Strength		
Abutment	0.1	Infinite Strength		
Approach Slab	150	Mohr-Coulomb	10,000	0

Note: [1]Construction material proposed

Seismic Loading

Location	K_{av}	Seismic Loading
Abutment 1	0.084	0.042
Abutment 2	0.082	0.041

(Assump. 4)

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

Results - Granular Borrow Fill Behind Abutments - Longitudinal to Centerline - Inclusive of Abutments

The results of the Slide2 global stability analyses completed along the Bancroft Road profile between STA. 1+00 and STA. 6+00 for the proposed conditions and headwater elevations based on predicted flood events (Q_{50} and $Q_{1.1}$) are summarized in the following table. The circular stability runs were excluded from the table since the factor of safety results were either equal to or greater than the non-circular failure factors of safety.

$Q_{1.1}$ Elevation	332.9	feet NAVD88	Ref. 9
Q_{50} Elevation	340.7	feet NAVD88	Ref. 9

Location	Surface Type	Global Factor of Safety: Water Level Q_{50}	Global Factor of Safety: Water Level $Q_{1.1}$	Figure #	Condition
Longitudinal at Abutments - Inclusive of the Abutments	Non-Circular Static - Abutment 1[1]	1.28	1.37	2A/3A	Proposed profile geometry, gravel borrow, **Static loading**
	Non-Circular Static - Abutment 2[1]	1.32	1.40	2B/3B	
	Non-Circular Pseudo-static - Abutment 1[2]	1.16	1.25	2C/3C	Proposed profile geometry, gravel borrow, **Pseudo-Static loading**
	Non-Circular Pseudo-static - Abutment 2[2]	1.19	1.28	2D/3D	

Notes: [1]Static failure surfaces in red do not meet the minimum requirements of a FS of 1.5 (Assumption 2). [2]Pseudo-static failure surfaces meet the minimum requirements of a FS of 1.1 (Assumption 2).

Improving global stability factors of safety - including Abutment H-piles

Some of the results of the Slide2 global stability analyses for the proposed conditions and flood event headwater elevations yielded results that do not meet the minimum requirements of a static FS greater than 1.5 (Assumption 2) under static conditions and 1.1 under pseudo-static conditions.

To improve the factors of safety, we elected to include the proposed H-piles (HP14x89) into the global stability modeling to augment the resistance of soils below the abutments to potential slope failures. H-Pile resistance was incorporated using the Slide2 support type "Pile/Micro Pile" applied to the center of the base of each abutment footings at the proposed top of pile elevation.

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

Pile Shear

HP 14 x 89 piles are evaluated to support abutments No.1 and No. 2 in the longitudinal static and Pseudo-static seismic stability models. The spacing used in the stability evaluation is based on the proposed pile spacing provided in Ref. 9. The pile shear strength was determined using AASHTO LRFD (Ref. 6).

$$V_r = \phi_v V_n$$ (Ref. 6, Eqn. 6.12.1.2.3a-2)

where:

V_r = factored shear capacity, kip

ϕ_v = resistance factor for shear

V_n = nominal shear resistance, kip

$$V_n = CV_p$$ (Ref. 6, Eqn. 6.10.9.2-1)

C = ratio of shear-buckling resistance to shear yield strength as per Ref. 6, Eqns. 6.10.9.3.2-4, 6.10.9.3.2-5, or 6.10.9.3.2-6 and k = 5.0

V_p = plastic shear force, kip

$$V_p = 0.58F_y D t_w$$ (Ref. 6, Eqn. 6.10.9.2-2)

F_y =	specified minimum yield strength of steel, ksi	F_y =	50	ksi
D =	depth of the pile section, inches	D =	13.8	in
t_w =	thickness of the web, inches	t_w =	0.615	in

Determine C:

If: $$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$$ (Ref. 6, Eqn. 6.10.9.3.2-4)

Then: C = 1.0

where:

E =	Elastic modulus of steel, ksi	E =	29,000	ksi
k =	shear bucking coefficient	k =	5.0	

$$1.12\sqrt{\frac{Ek}{F_y}} = 60.3$$

$$\frac{D}{t_w} = 22.4$$

Hence:

C = 1.0

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

Determine nominal shear capacity, V_n:

$$V_n = CV_p$$ (Ref. 6, Eqn. 6.10.9.2-1)

C = 1.0 (From Above)

V_p = 246 kips (Ref. 6, Eq. 6.10.9.2-2)

V_n = 246 kips

Determine factored shear capacity, V_r:

$$V_r = \phi_v V_n$$ (Ref. 6, Eqn. 6.12.1.2.3a-2)

f_v = 1.0 (Ref. 6, Article 6.5.4.2, "For Shear")

V_n = 246 (From Above)

V_r = 246 kips

Support	Force Application	Out-Of-Plance Spacing (ft)	Shear Strength (lbs) (Assump. 12)	Force Orientation
Pile/Micro-Pile	Active (Method A)	7.72	24,600	Parallel to Surface

(Assumption 9)

The results of the Slide2 global stability analyses completed along the Bancroft Road profile between STA. 1+00 and STA. 6+00 for the proposed conditions, including the H-piles, and headwater elevations based on predicted flood events (Q_{50} and $Q_{1.1}$) are summarized in the following table. The circular stability runs were excluded from the table since the results were either equal to the non-circular failures, or resulted in a higher factor of safety than the non-circular failures.

Location	Surface Type	Global Factor of Safety: Water Level Q_{50}	Global Factor of Safety: Water Level $Q_{1.1}$	Figure #	Condition
Longitudinal at Abutments - Inclusive of Abutments and H-Piles	Non-Circular Static - Abutment 1[1]	1.53	1.55	2E/3E	Proposed profile geometry - Static loading
	Non-Circular Static - Abutment 2[1]	1.58	1.57	2F/3F	
	Non-Circular Pseudo-static - Abutment 1[2]	1.36	1.40	2G/3G	Proposed profile geometry - Pseudo-static loading
	Non-Circular Pseudo-static - Abutment 2[2]	1.40	1.42	2H/3H	

Notes: [1]Static failure surfaces meet the minimum requirements of a FS of 1.5 (Assumption 2). [2]Pseudo-static failure surfaces meet the minimum requirements of a FS of 1.1 (Assumption 2).

CALCULATIONS

Date:	3/31/2026	**Made by:**	LDN
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM/DEB
Subject:	Global Stability Analysis - Longitudinal at Proposed Abutments	**Reviewed by:**	MEL/JDL
Project Short	Bancroft Road Bridge No. 3189, Crossing Smith Brook, Bancroft TWP, ME		

Results - Longitudinal to Centerline - Abutment Foreslopes

In addition to evaluating global stability inclusive of the abutments, the abutment foreslopes were considered separately. The results of the Slide2 stability analyses for the foreslope proposed conditions and water table elevations discussed above are summarized in the following table. The circular stability runs were excluded from the table since the results were either equal to the non-circular failures, or resulted in a higher factor of safety than the non-circular failures. Both scenarios with proposed conditions with or without piles resulted with identical results.

Location	Surface Type	Foreslope Factor of Safety		Figure #	Condition
		Water Level Q_{50}	Water Level $Q_{1.1}$		
Longitudinal at Abutment Foreslopes	Non-Circular Static - Abutment 1[1]	1.50	1.45	2I/3I	Proposed Foreslope geometry - Static loading
	Non-Circular Static - Abutment 2[1]	1.50	1.45	2J/3J	
	Non-Circular Pseudo-static - Abutment 1[2]	1.33	1.29	2K/3K	Proposed Foreslope geometry - Pseudo-static loading
	Non-Circular Pseudo-static - Abutment 2[2]	1.35	1.30	2L/3L	

Notes: [1]Static failure surfaces meet the minimum requirements of a FS of 1.3 (Assumption 2). [2]Pseudo-static failure surfaces meet the minimum requirements of a FS of 1.1 (Assumption 2).

CONCLUSIONS

1. The analyzed global stability assuming granular borrow proposed as fill behind the abutments show the combination of the in situ soils, standard integral abutment geometry, and design water levels yield inadequate factors of safety (FS < 1.5) for potential failure surfaces that extend below the abutments. This is the result of weak in situ alluvial sands and silts that are influenced by the river water levels. These weak in situ soils are at and below the river level. Replacement or improvement of these soils would require costly excavation or other methods.

2. Therefore, we propose to incorporate the additional resistance from H-piles at the abutment-pile interface elevation in the global stability evaluations. With the addition of the pile resistance, the computed factors of safety are above the recommended FS of 1.5 and the proposed conditions meet the minimum FS of 1.1 or greater for pseudo-static conditions for potential failure surfaces that extend below the abutments.

3. The analyzed proposed foreslope conditions at the abutments meet the recommended FS of 1.3 or greater for static conditions and FS of 1.1 or greater for pseudo-static conditions.

500 480 460 440 420 400

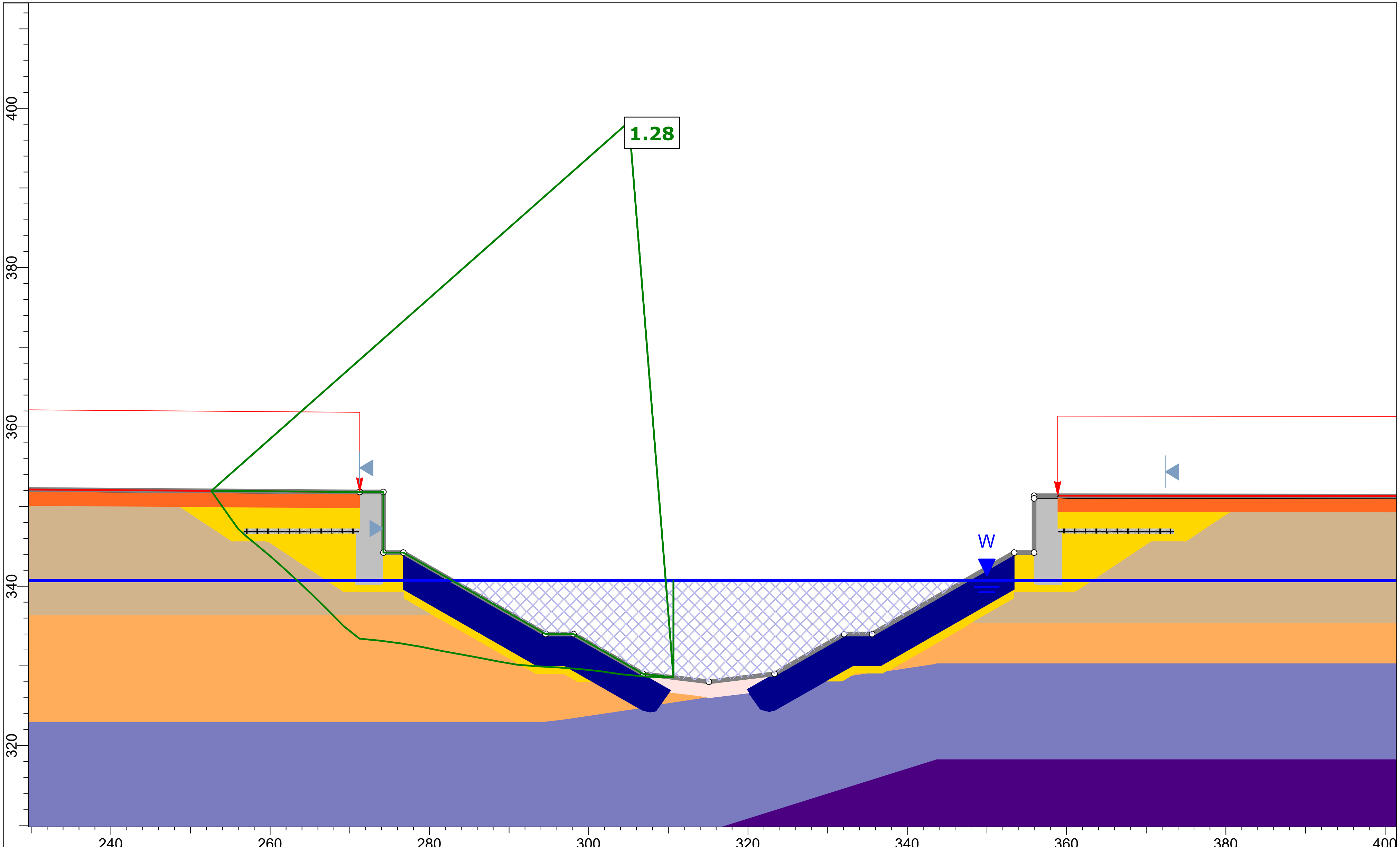
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary	Water Surface	Hu Type	Hu
Pavement		140	Mohr-Coulomb	0	45		Water Table	Custom	1
Granular Borrow		125	Mohr-Coulomb	0	32		Water Table	Custom	1
Aggregate Subbase		135	Mohr-Coulomb	0	36		Water Table	Custom	1
Heavy Riprap		140	Mohr-Coulomb	0	42		Water Table	Custom	1
Special Fill		135	Mohr-Coulomb	0	36		Water Table	Custom	1
Existing Fill		125	Mohr-Coulomb	0	36		Water Table	Custom	1
Alluvial Sand and Gravel		125	Mohr-Coulomb	0	31		Water Table	Custom	1
Alluvial Silt		115	Mohr-Coulomb	0	31		Water Table	Custom	1
Glacial Till		130	Mohr-Coulomb	0	36		Water Table	Custom	1
Bedrock		165	Infinite Strength			Yes	Water Table	Custom	0
Abutment		0.1	Infinite Strength			Yes	Water Table	Custom	0
Approach Slab		150	Mohr-Coulomb	10000	0		Water Table	Custom	0

240 260 280 300 320 340 360 380 400

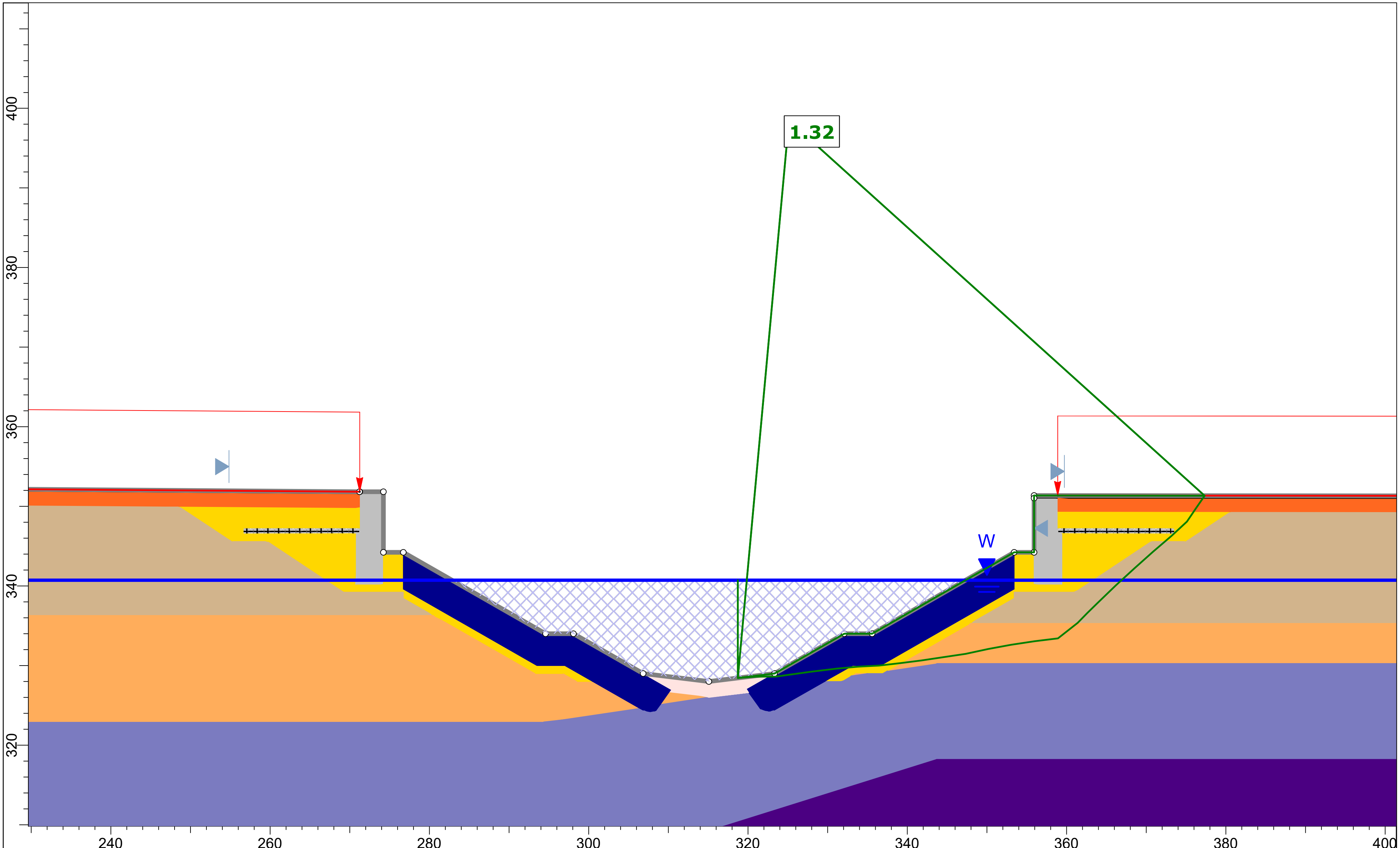
Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME		
Analysis Description	Material Properties		
Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:206
Date 3/25/2026	*File Name* Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd		

Figure 1

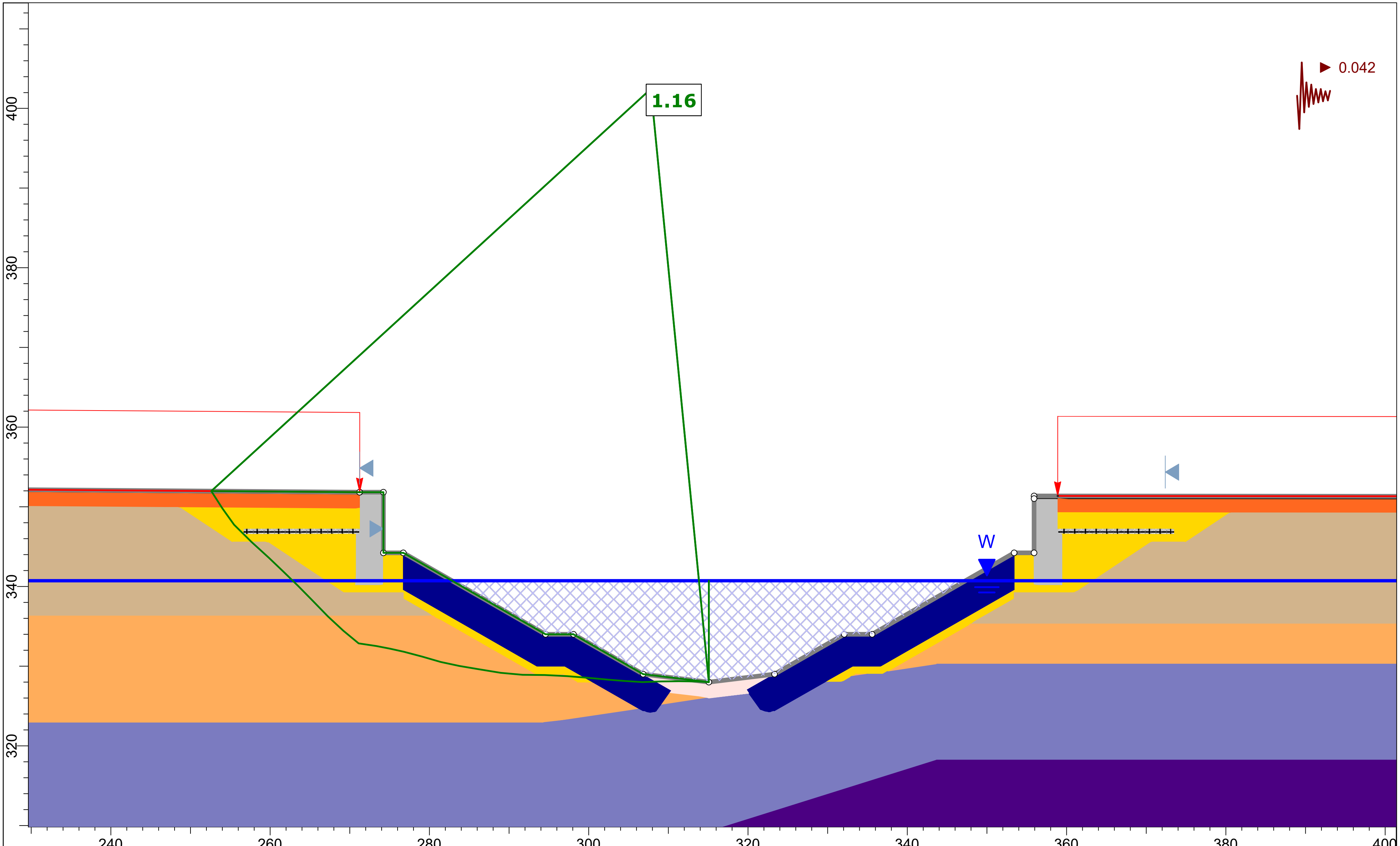
SLIDE 9.040



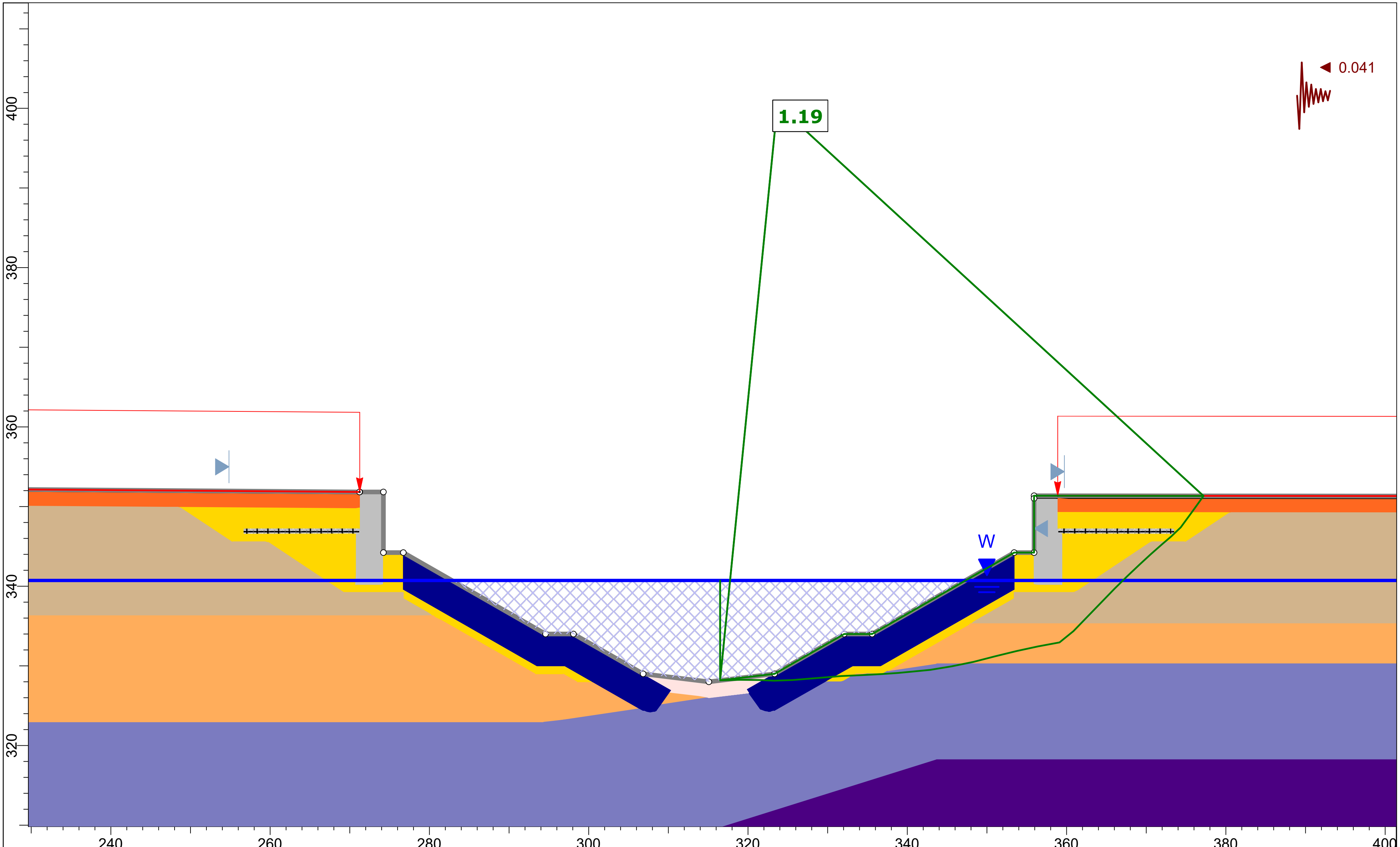
wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Static), Q50					
	Drawn By LDN	*Checked By* DEB		*Reviewed By* MEL	*Scale* 1:200		**Figure 2A**
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd					



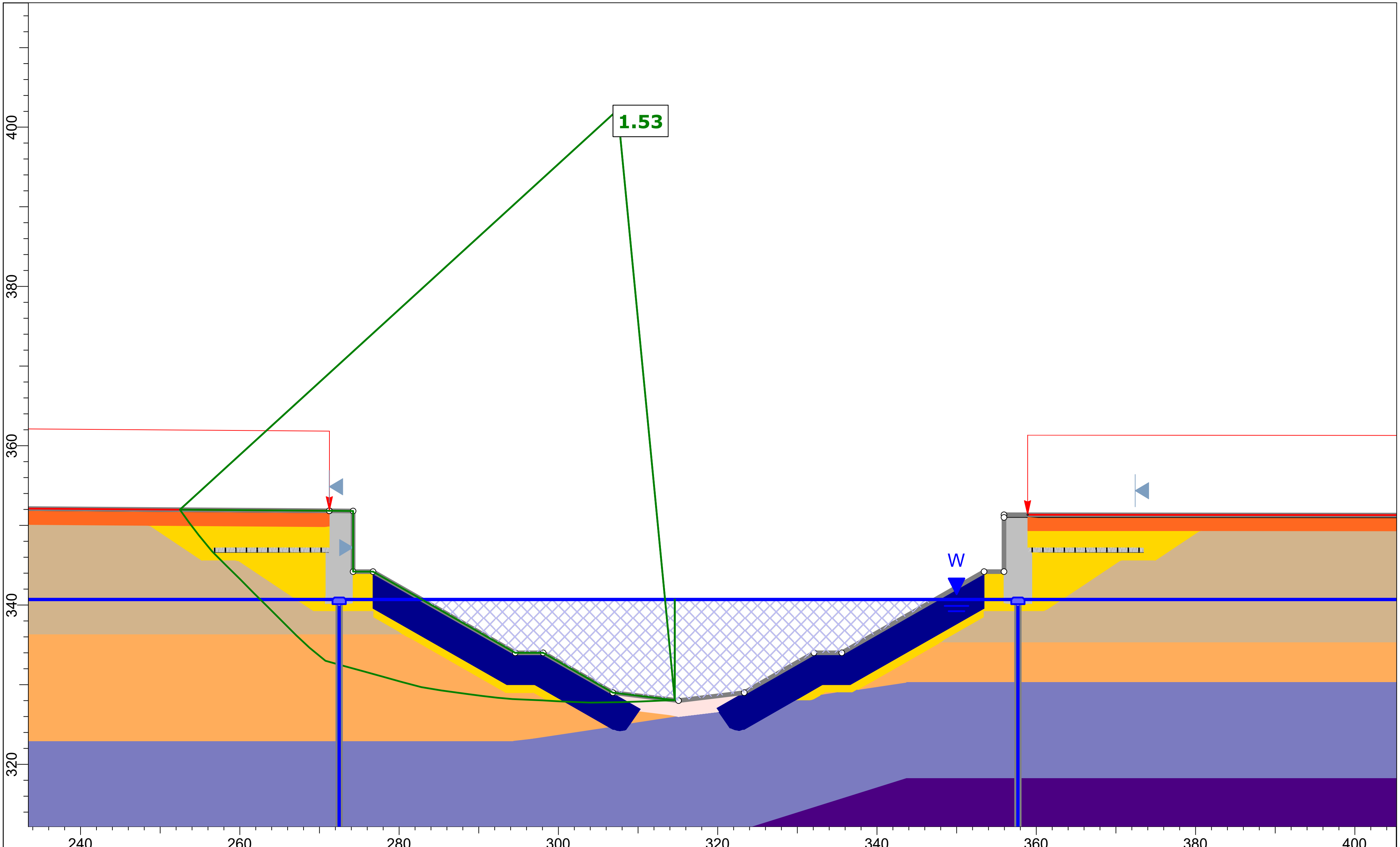
wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Static), Q50					
	Drawn By LDN	*Checked By* DEB		*Reviewed By* MEL		*Scale* 1:200	**Figure 2B**
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd					



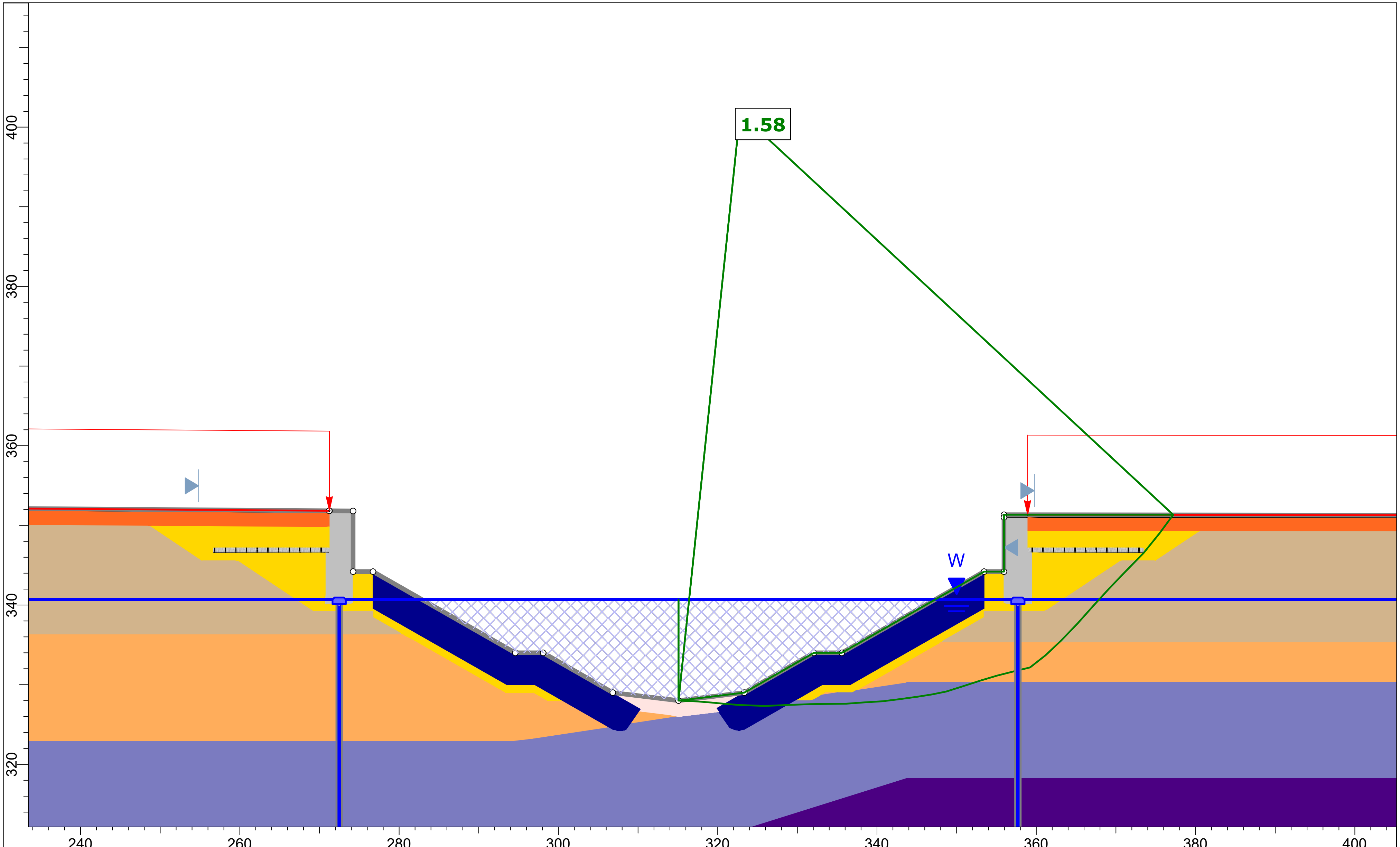
wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Pseudostatic), Q50					
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:200	**Figure 2C**		
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd					



wsp	Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Pseudostatic), Q50					
	Drawn By	LDN	Checked By DEB	Reviewed By MEL	Scale	1:200	**Figure 2D**
SLIDEINTERPRET 9.040	Date	3/24/2026	File Name	Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd			



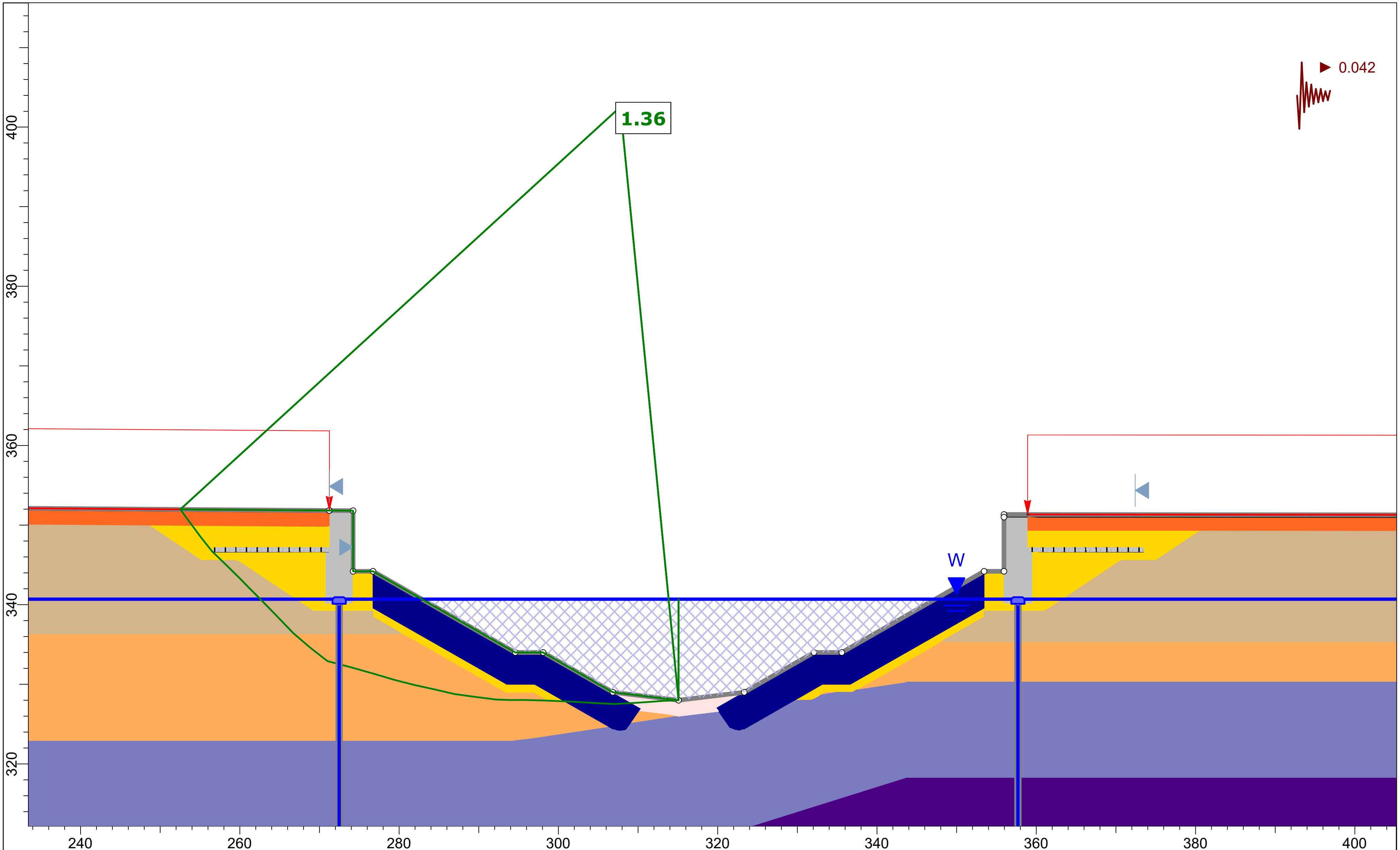
wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Static), Q50 and Piles					
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:200			Figure 2E
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd					



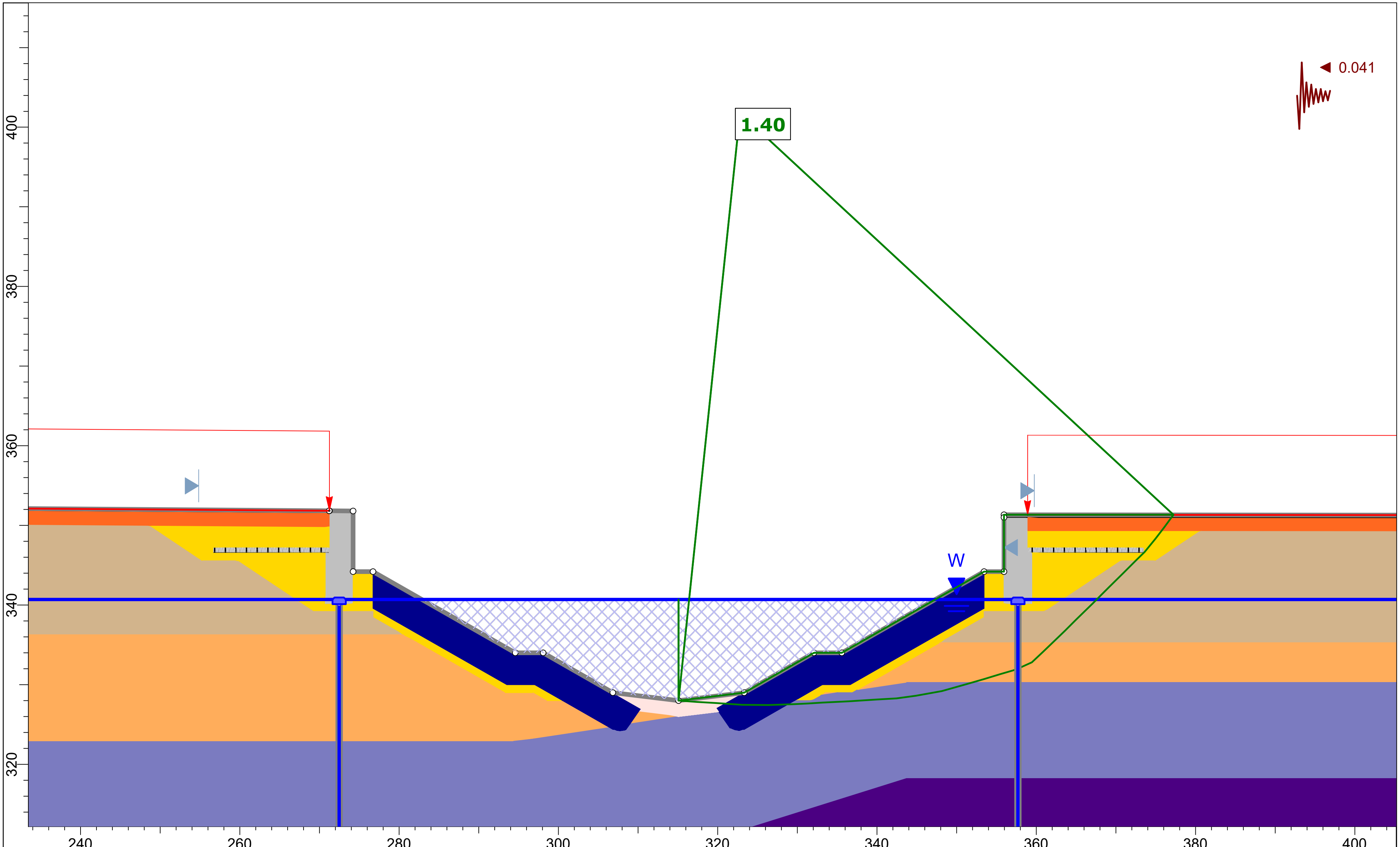
SLIDEINTERPRET 9.040

Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME
Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Static), Q50 and Piles
Drawn By	LDN
Checked By	DEB
Reviewed By	MEL
Scale	1:200
Date	3/24/2026
File Name	Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd

Figure 2F



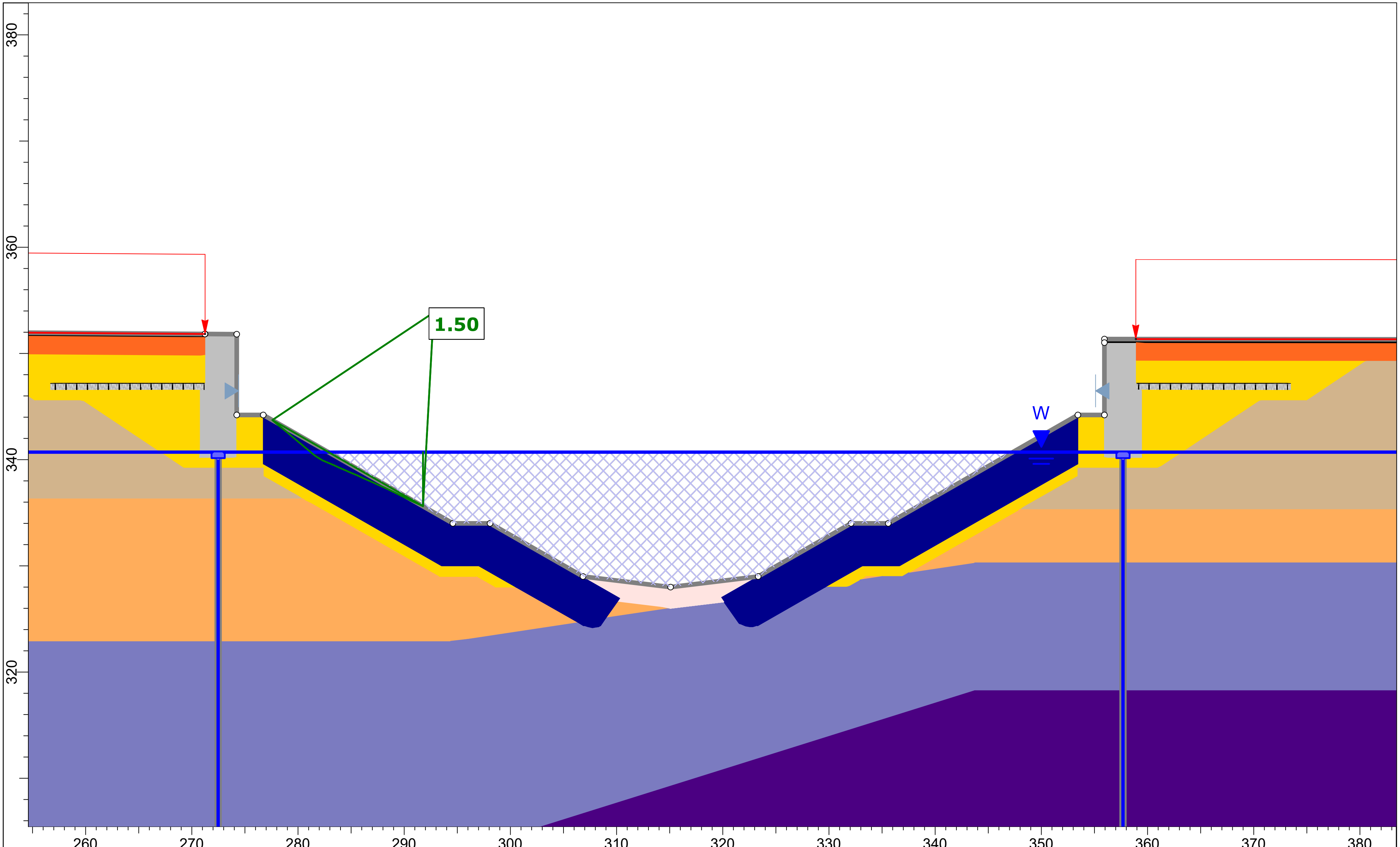
wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Pseudostatic), Q50 and Piles					
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:200	**Figure 2G**		
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd					



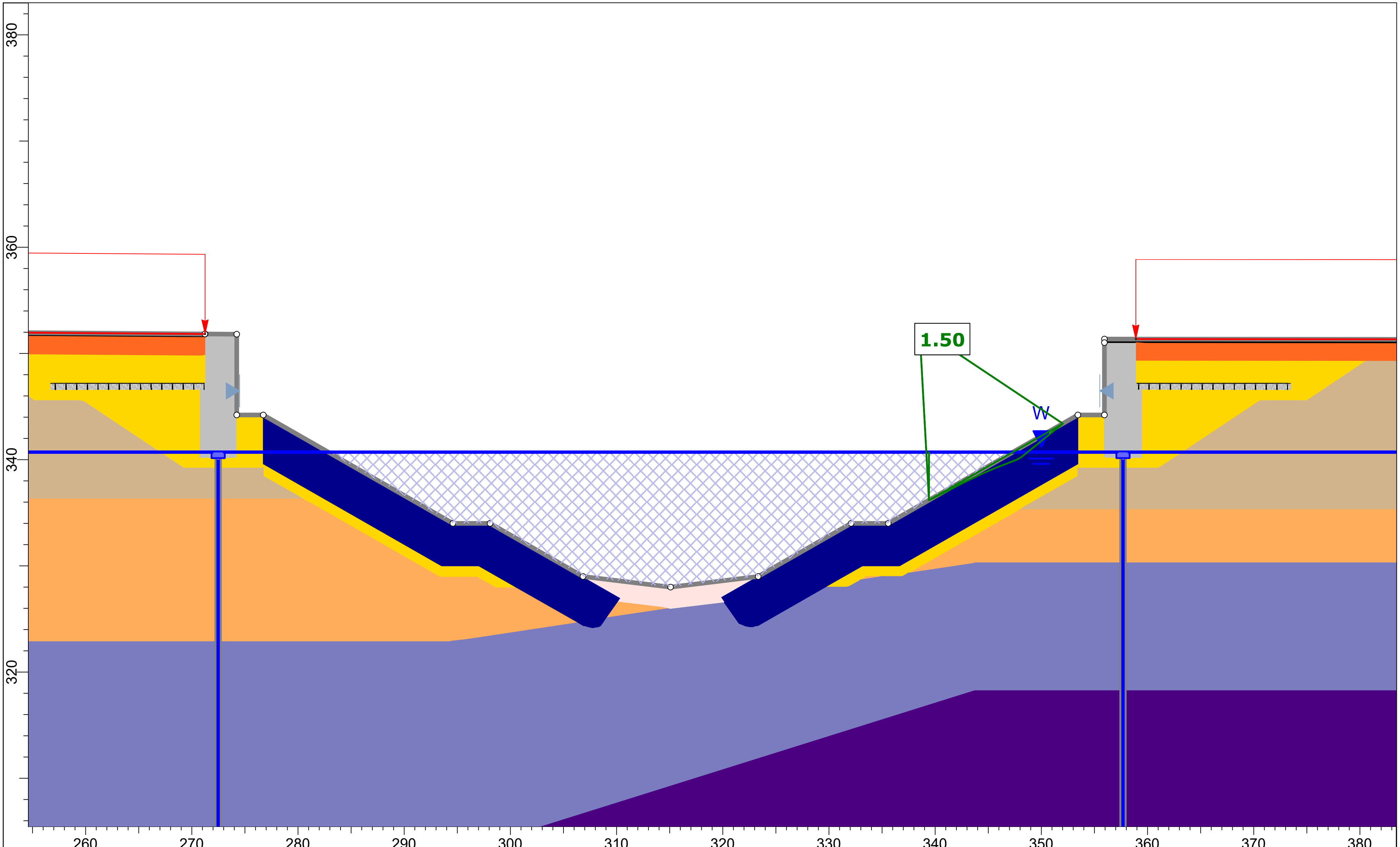
Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME						
Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Pseudostatic), Q50 and Piles						
Drawn By	LDN	*Checked By*	DEB	*Reviewed By*	MEL	*Scale*	1:200
Date	3/24/2026	*File Name*	Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd				

Figure 2H

SLIDEINTERPRET 9.040



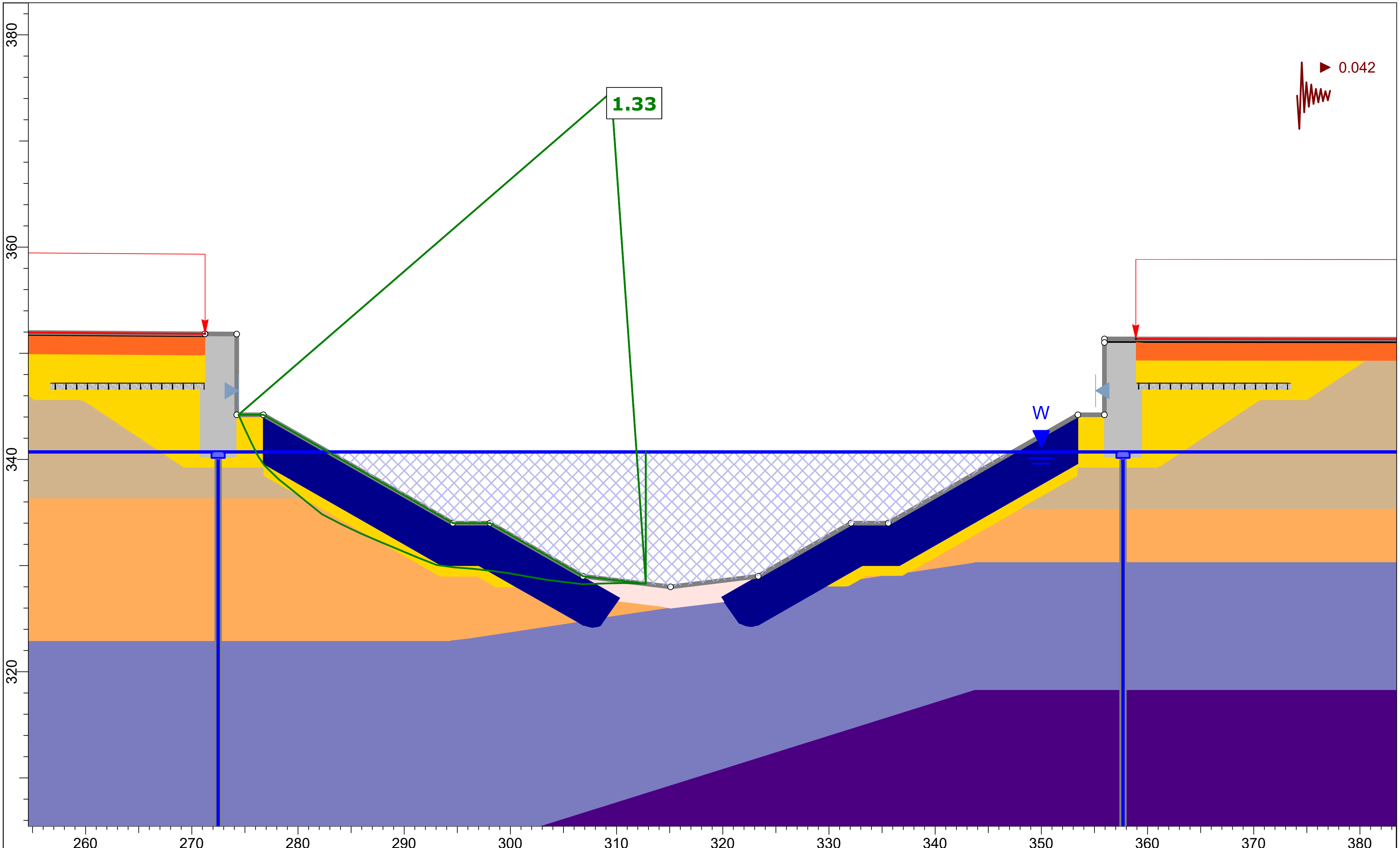
wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Static), Q50 and Piles					
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:150			**Figure 2I**
SLIDEINTERPRET 9.040	*Date* 3/25/2026	*File Name* Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd					



Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME						
Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Static), Q50 and Piles						
Drawn By	LDN	*Checked By*	DEB	*Reviewed By*	MEL	*Scale*	1:150
Date	3/25/2026	*File Name*	Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd				

Figure 2J

SLIDEINTERPRET 9.040

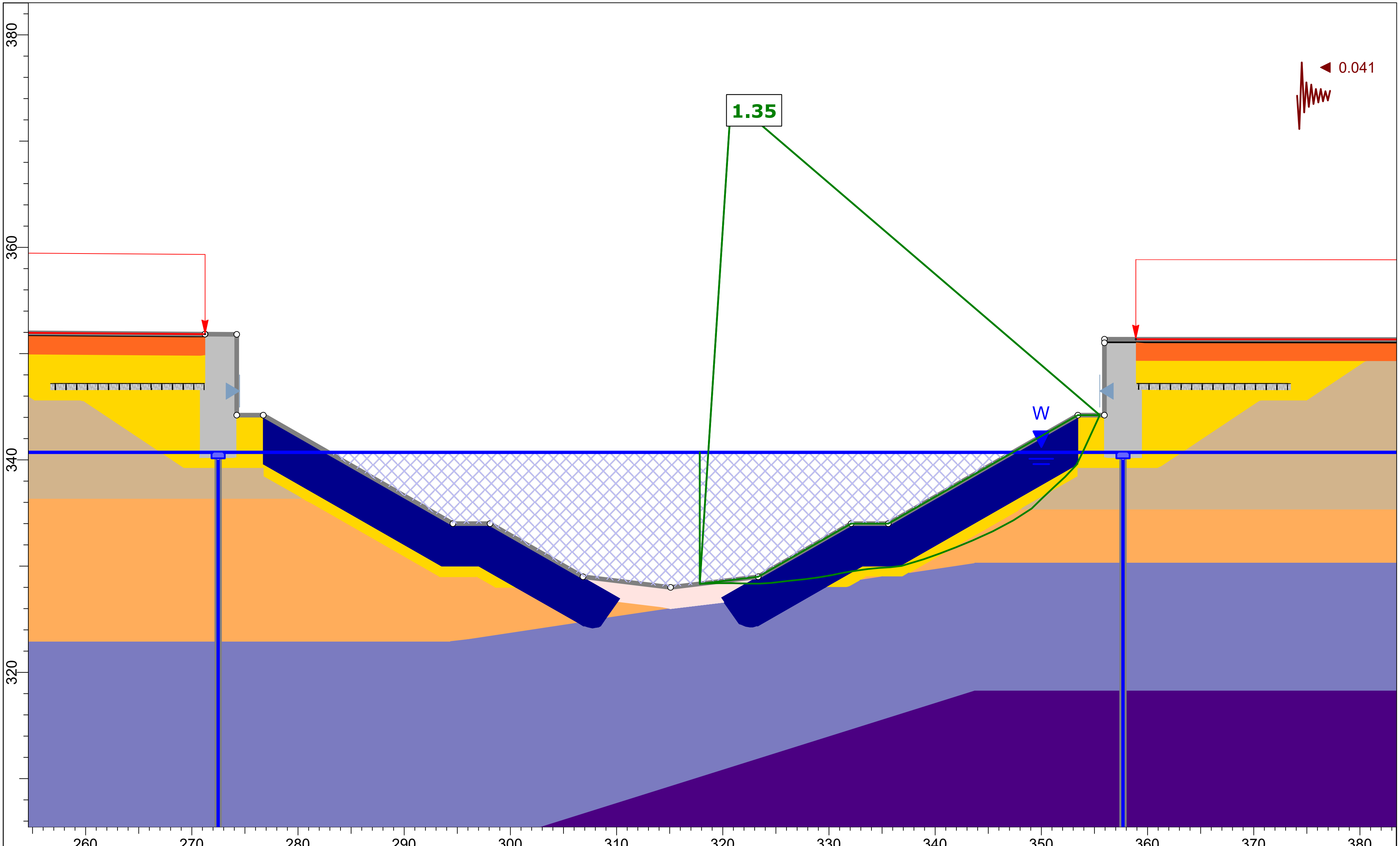


SLIDEINTERPRET 9.040

Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME
Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Pseudostatic), Q50 and Piles

Drawn By	LDN	*Checked By*	DEB	*Reviewed By*	MEL	*Scale*	1:150
Date	3/25/2026	*File Name*	Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd				

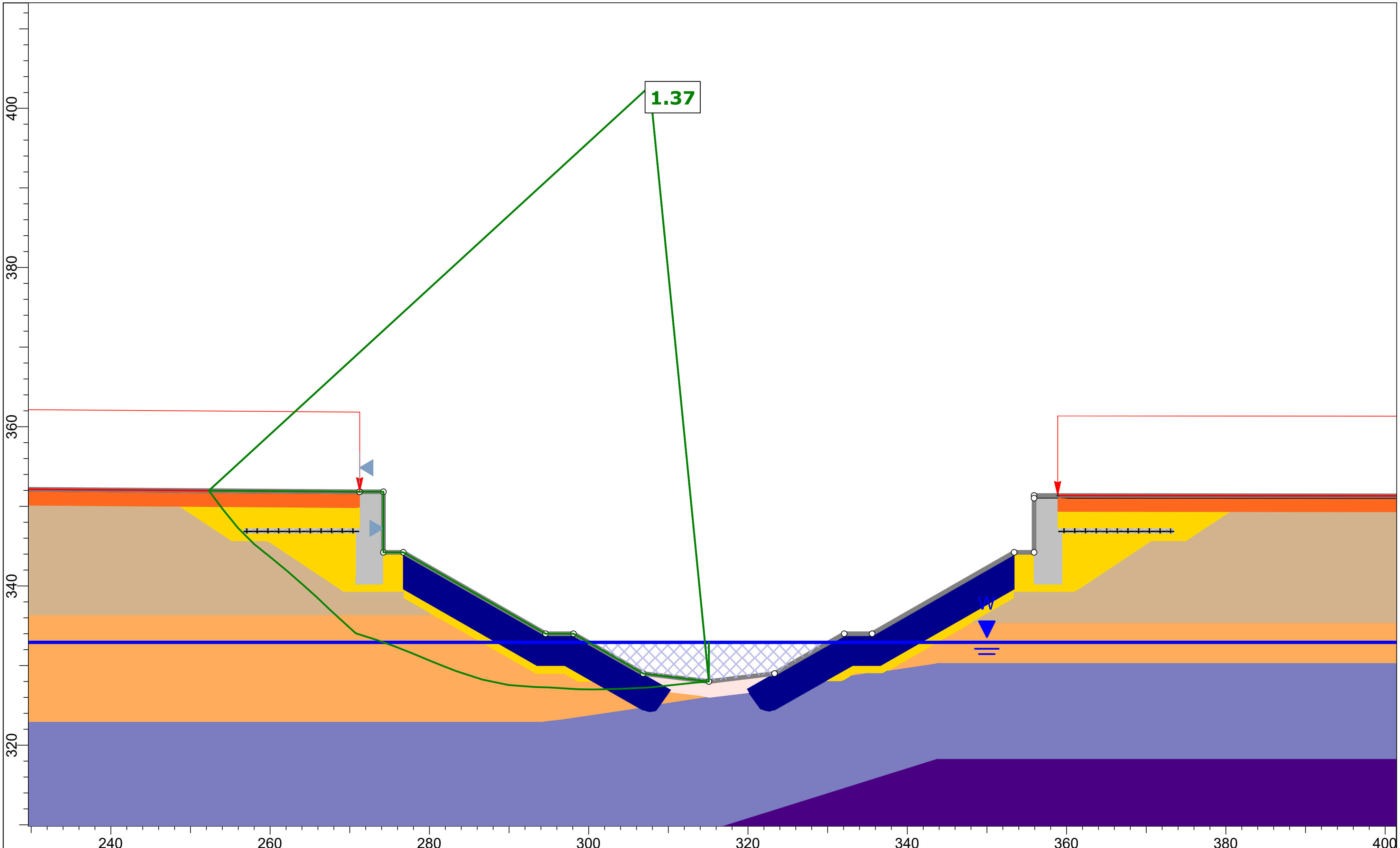
Figure 2K



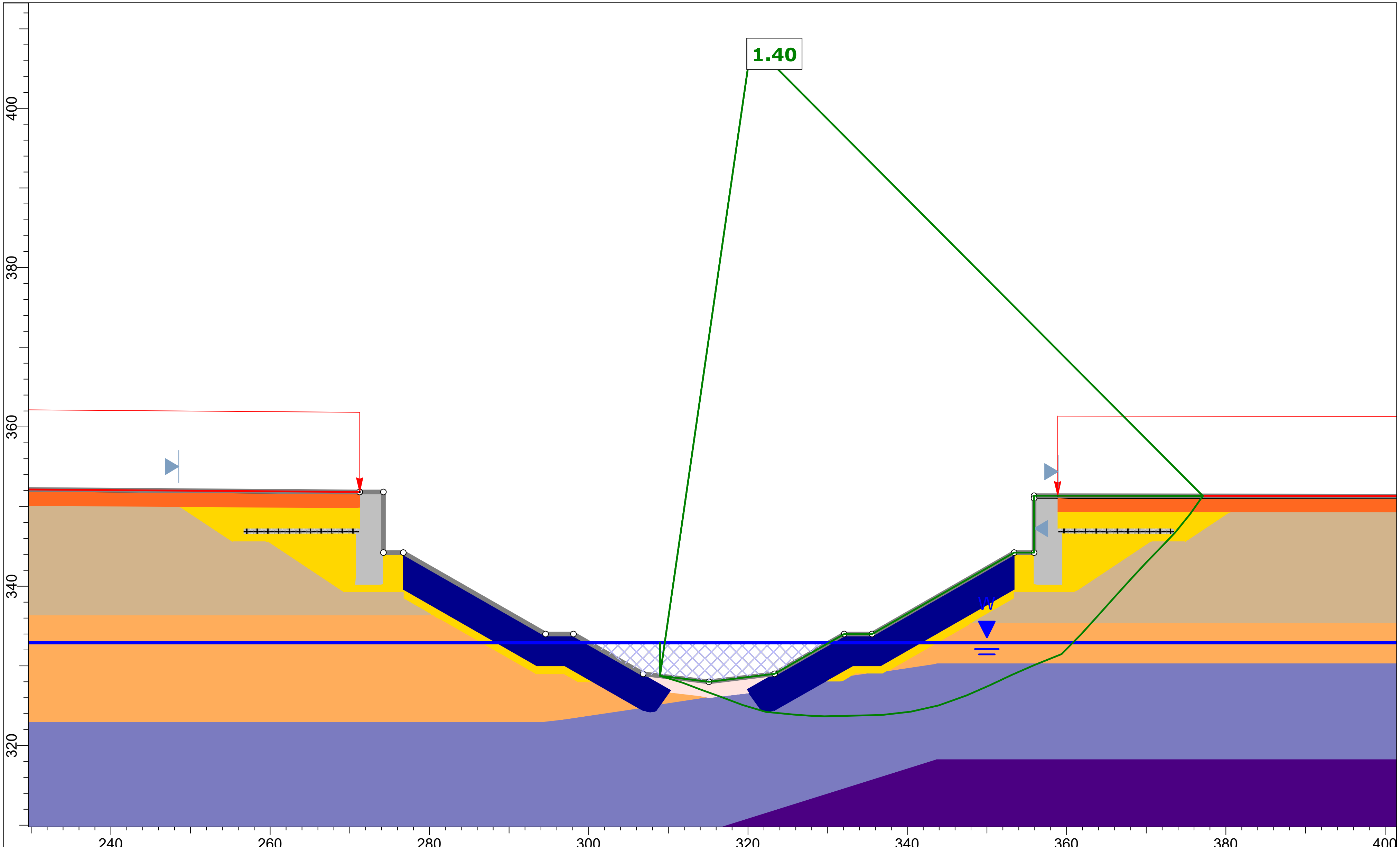
Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME		
Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Pseudostatic), Q50 and Piles		
Drawn By: LDN	Checked By: DEB	Reviewed By: MEL	Scale: 1:150
Date: 3/25/2026	File Name: Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd		

Figure 2L

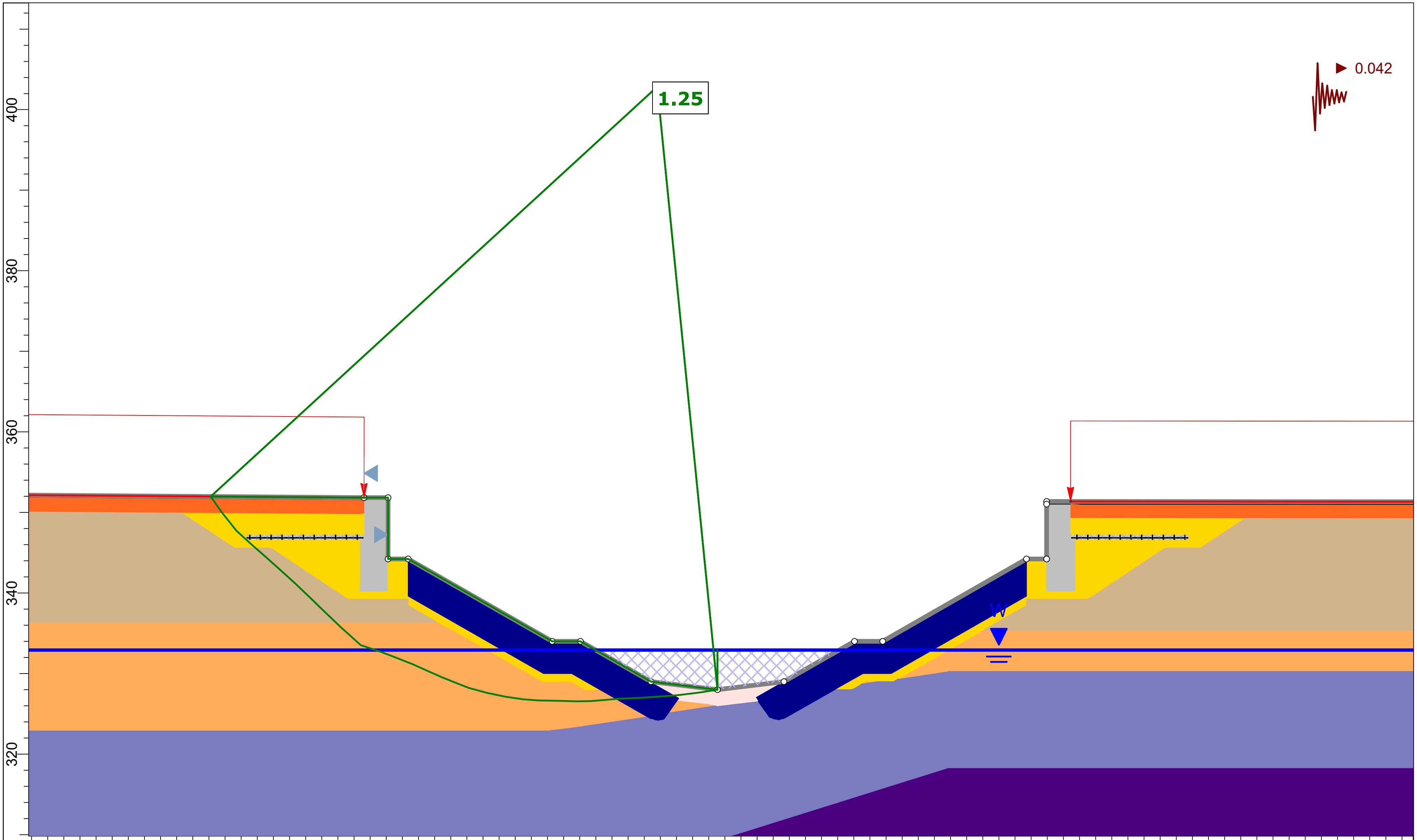
SLIDEINTERPRET 9.040



wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Static), Q1.1					
	Drawn By LDN	*Checked By* DEB		*Reviewed By* MEL	*Scale* 1:200	**Figure 3A**	
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd					

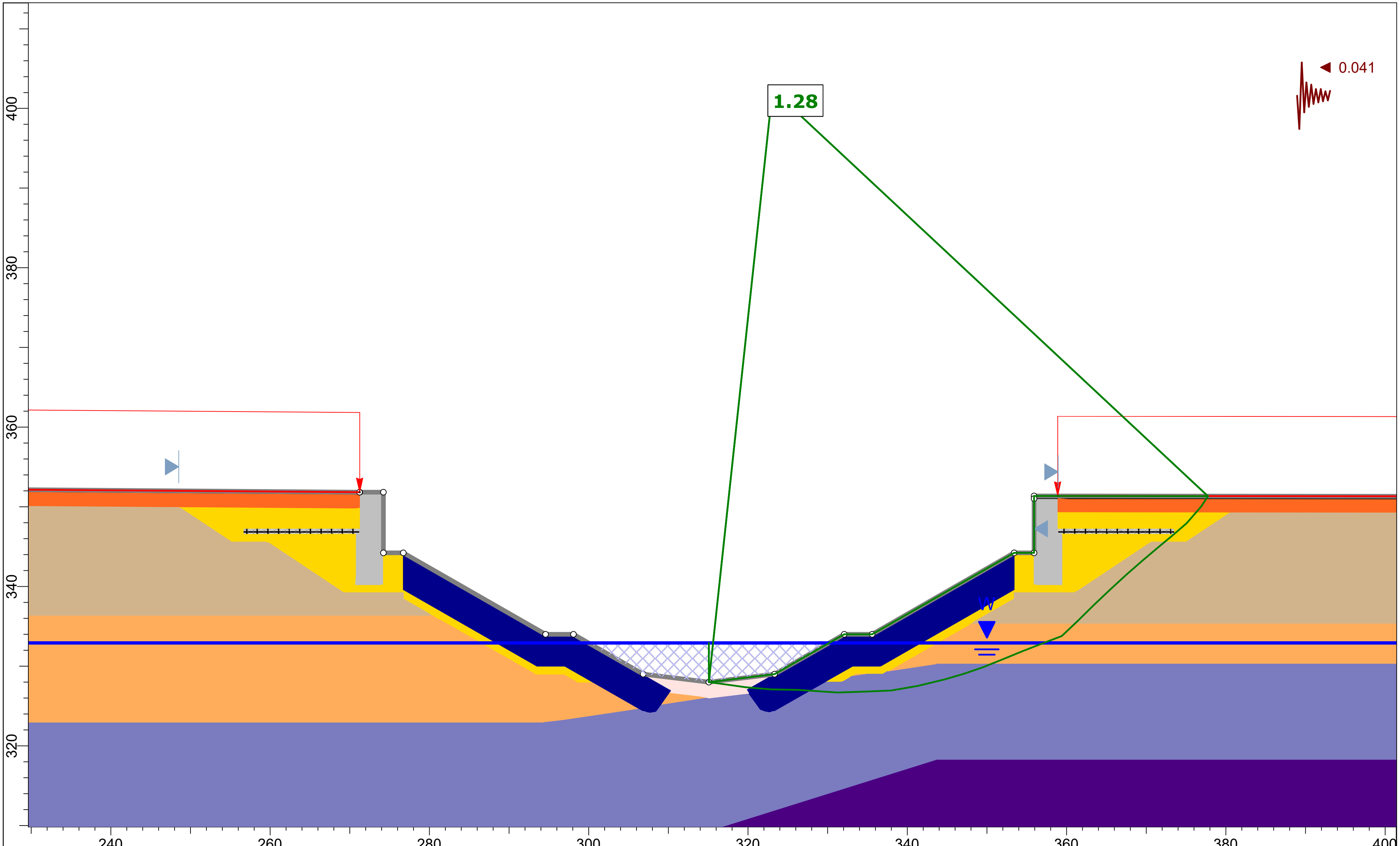


wsp	Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Static), Q1.1					
	Drawn By	LDN	Checked By DEB	Reviewed By MEL	Scale	1:200	Figure 3B
SLIDEINTERPRET 9.040	Date	3/24/2026	File Name	Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd			

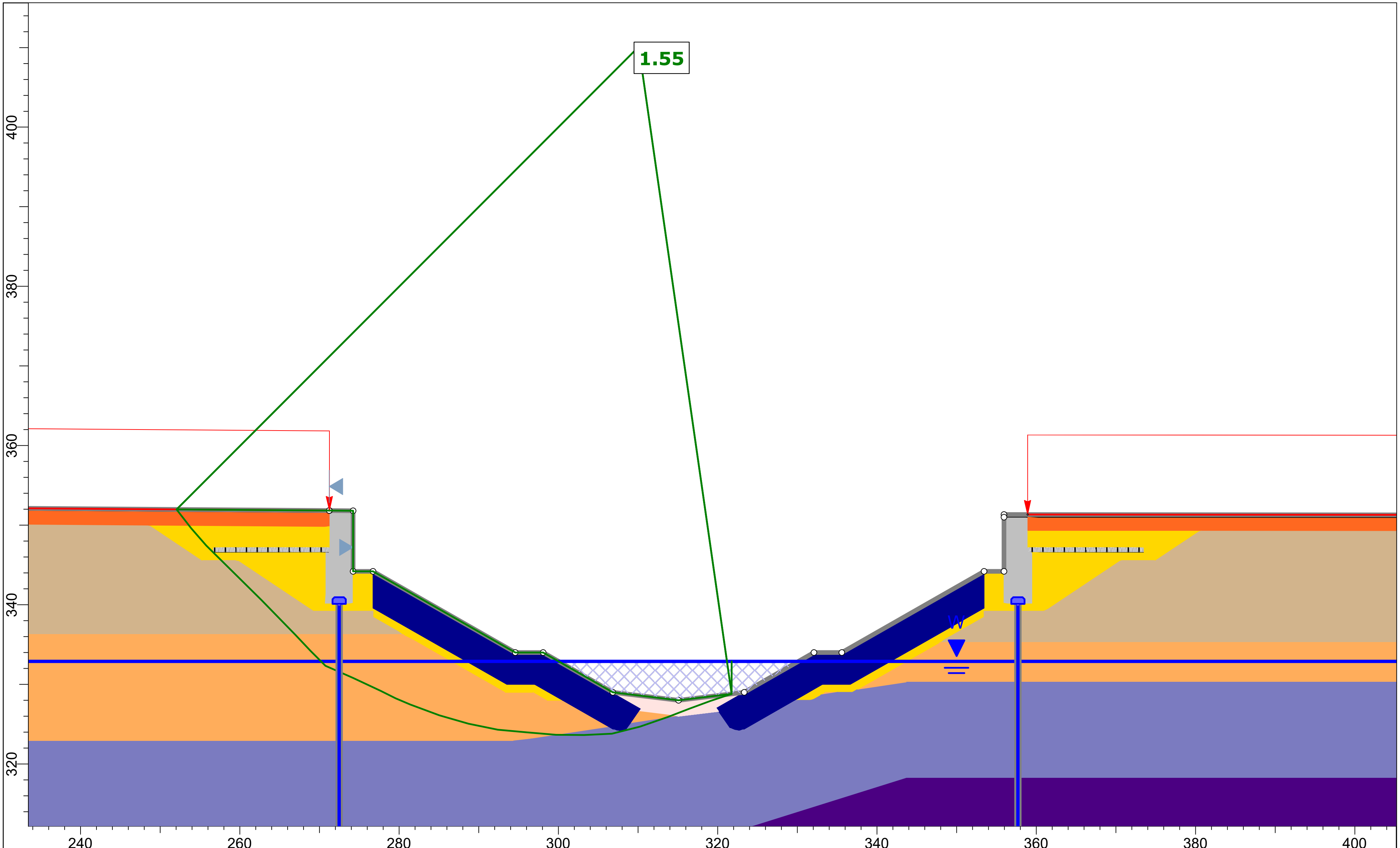


Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME				
Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Pseudostatic), Q1.1				
Drawn By: LDN	Checked By: DEB	Reviewed By: MEL	Scale: 1:200		**Figure 3C**
Date: 3/24/2026	File Name: Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd				

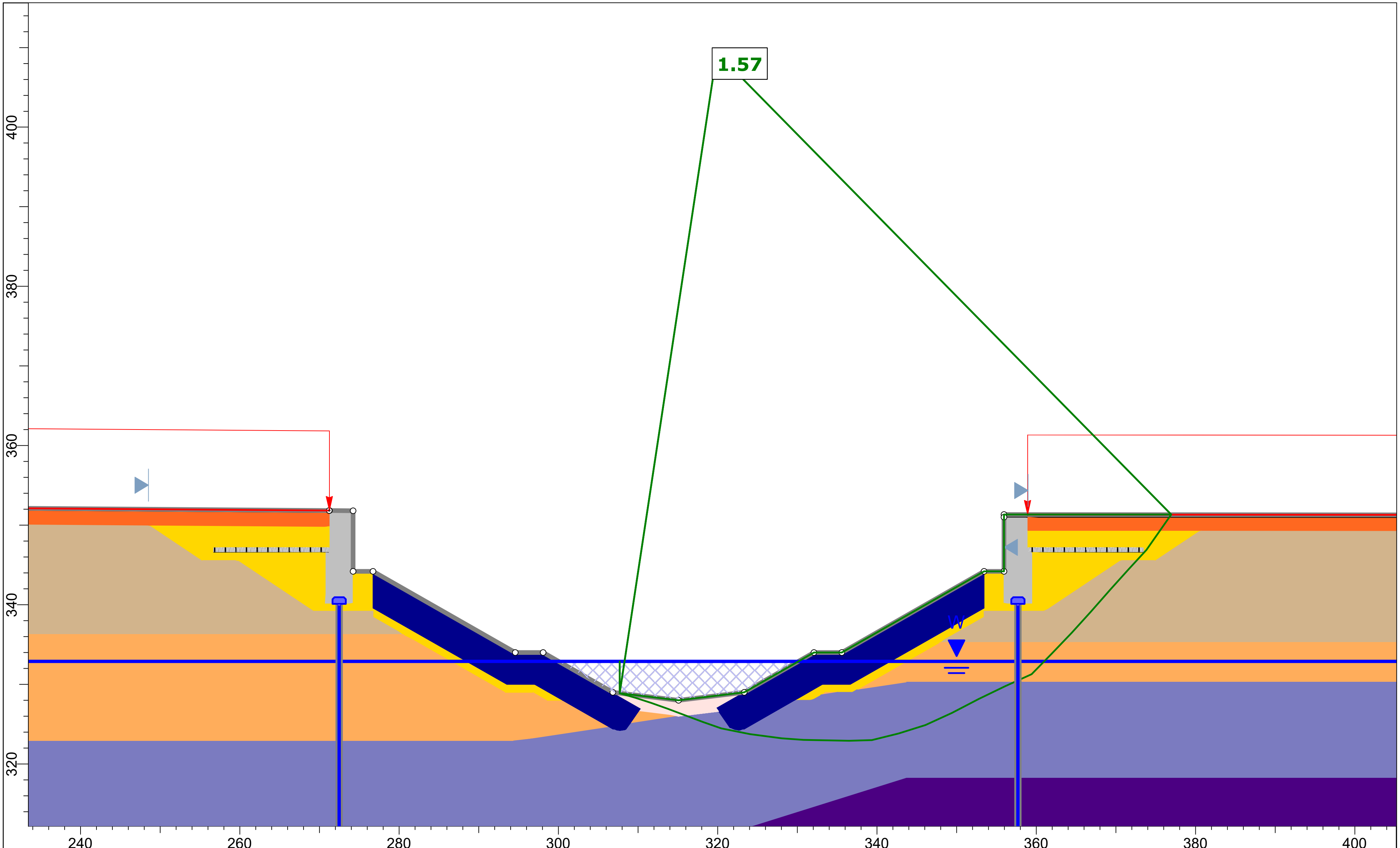
SLIDEINTERPRET 9.040



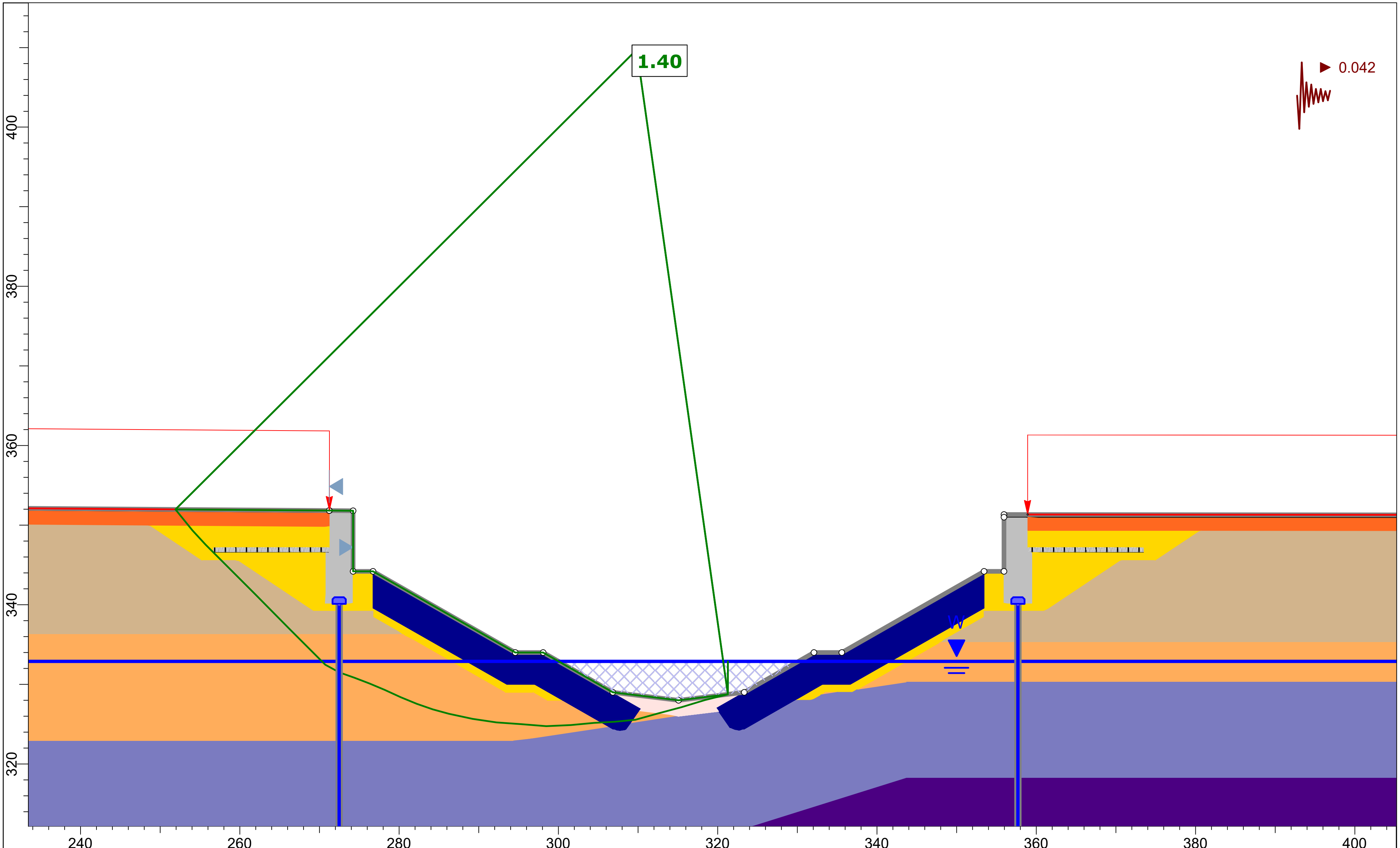
wsp	Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Pseudostatic), Q1.1					
	Drawn By LDN	Checked By DEB		Reviewed By MEL		Scale 1:200	**Figure 3D**
SLIDEINTERPRET 9.040	Date 3/24/2026	File Name Bridge 3189 Longitudinal Global Stability NO PILES 3.24.26.slmd					



	Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME				
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Static), Q1.1 and Piles				
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:200	**Figure 3E**	
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd				



wsp	Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Static), Q1.1 and Piles					
	Drawn By	LDN	Checked By	DEB	Reviewed By	MEL	Scale 1:200
SLIDEINTERPRET 9.040	Date	3/24/2026	File Name	Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd			Figure 3F

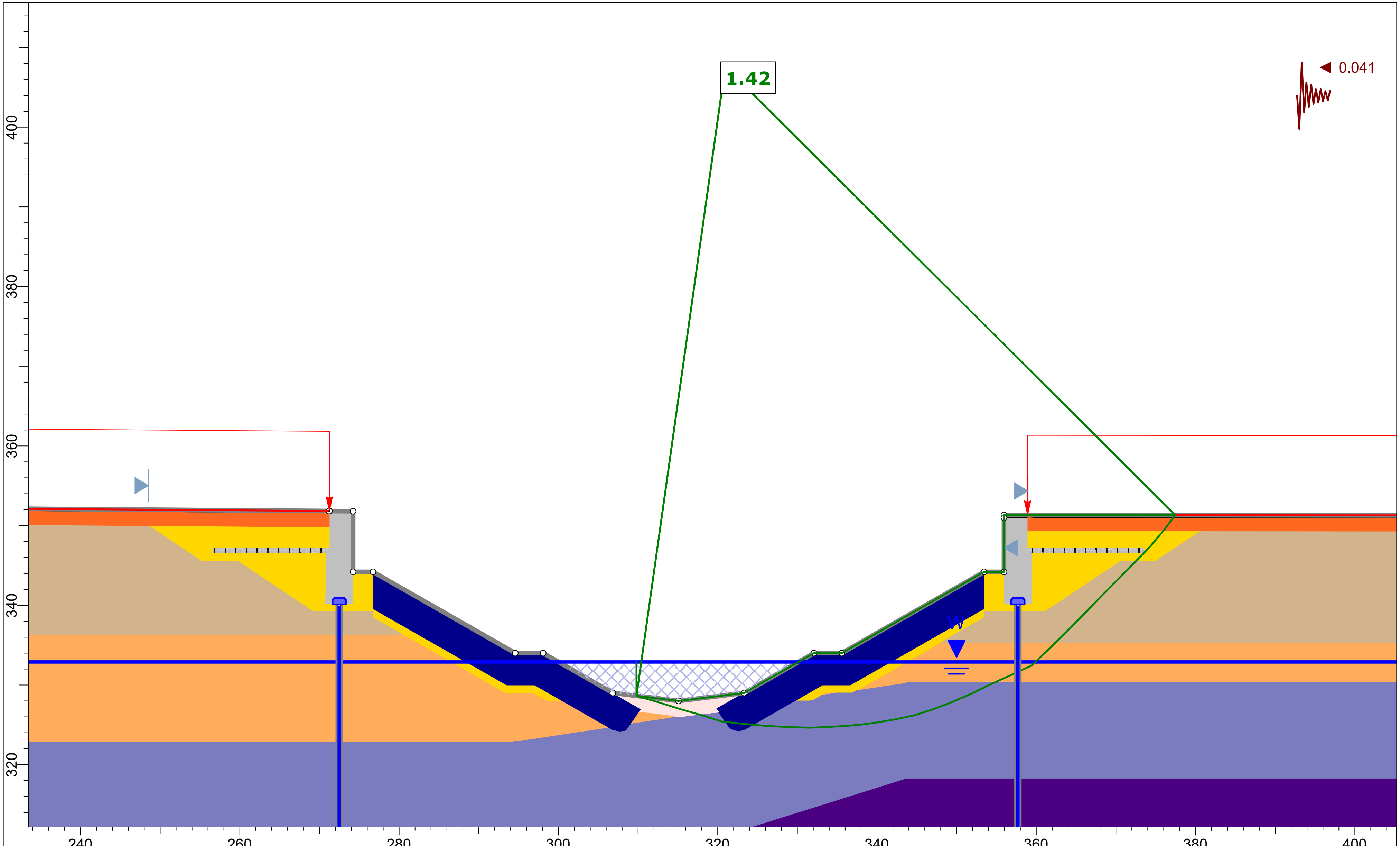


SLIDEINTERPRET 9.040

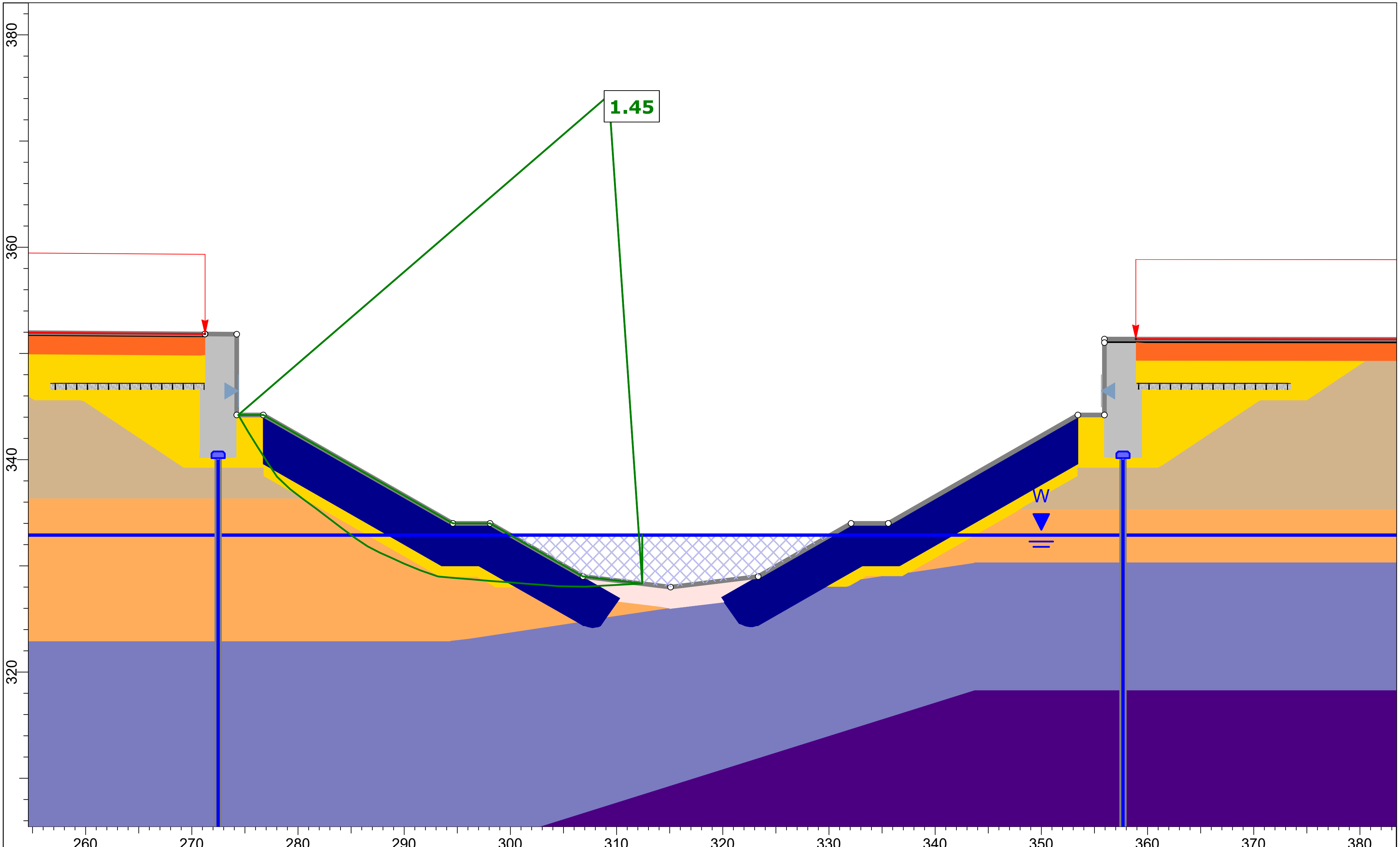
Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME
Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Pseudostatic), Q1.1 and Piles

Drawn By	LDN	*Checked By*	DEB	*Reviewed By*	MEL	*Scale*	1:200
Date	3/24/2026	*File Name*	Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd				

Figure 3G



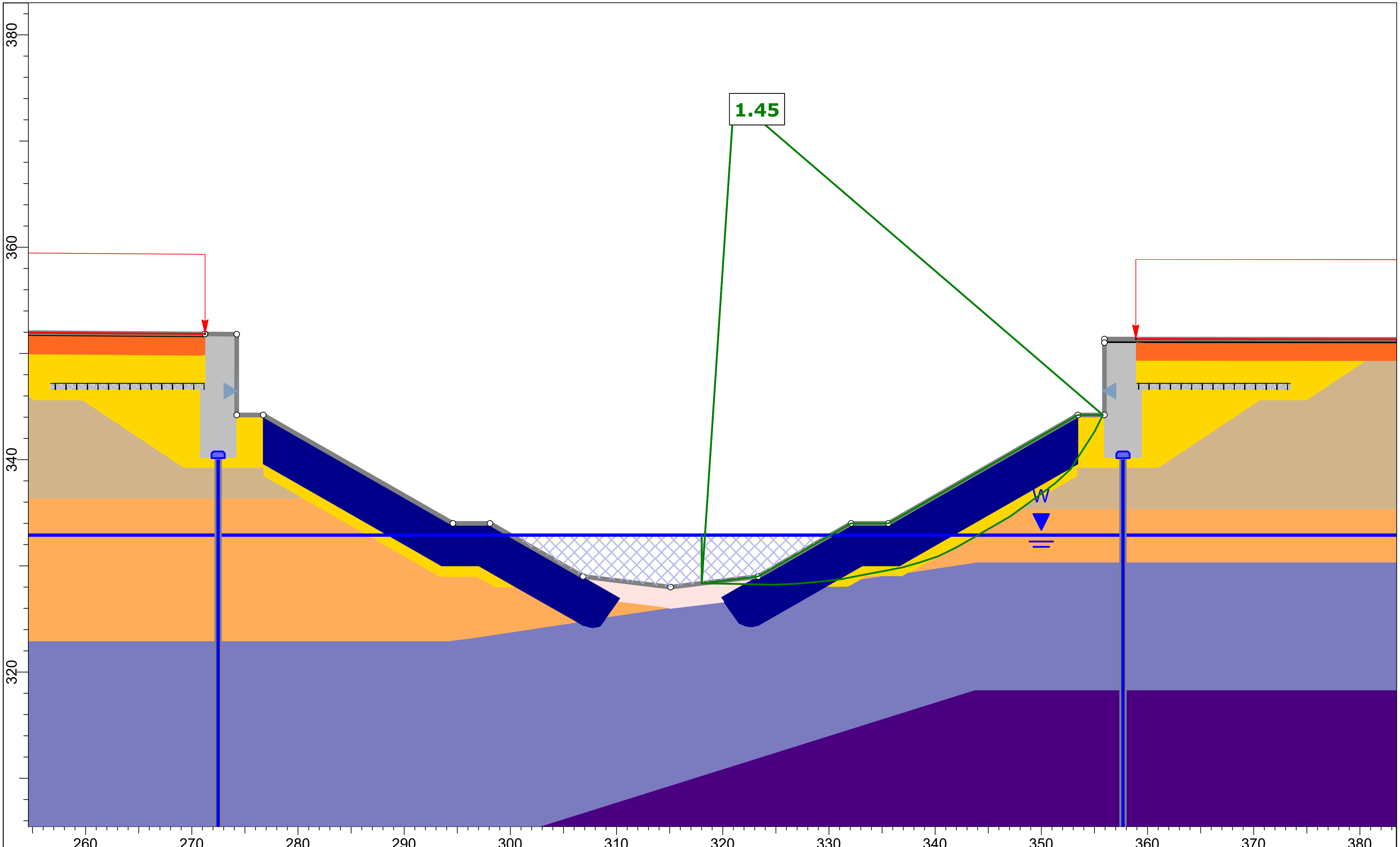
wsp	*Project* MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME				
	Analysis Description Abutment 2 - Proposed conditions (Non-Circular, Pseudostatic), Q1.1 and Piles				
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:200	**Figure 3H**
SLIDEINTERPRET 9.040	*Date* 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd			



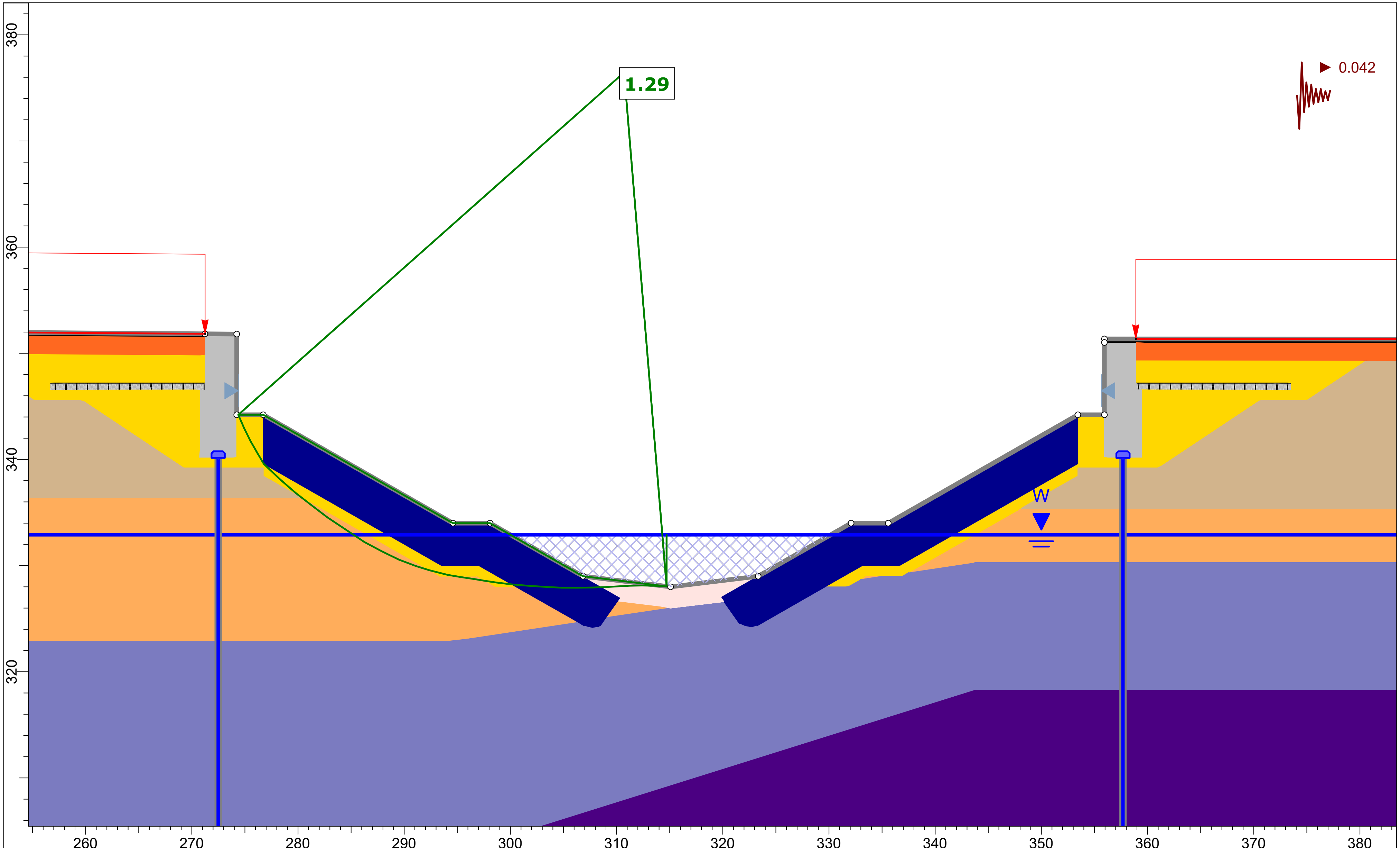
Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME		
Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Static), Q1.1 and Piles		
Drawn By: LDN	Checked By: DEB	Reviewed By: MEL	Scale: 1:150
Date: 3/25/2026	File Name: Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd		

Figure 3I

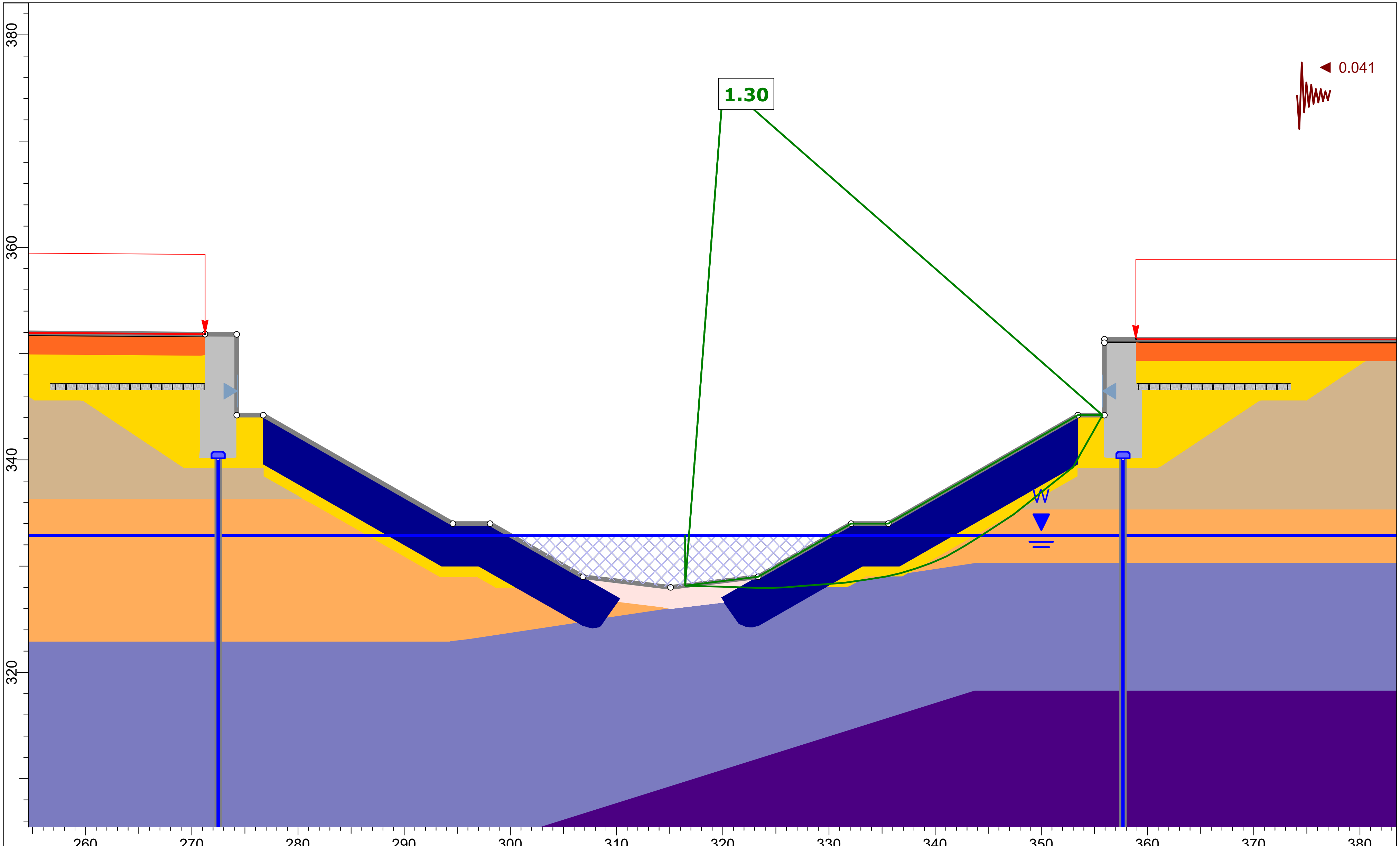
SLIDEINTERPRET 9.040



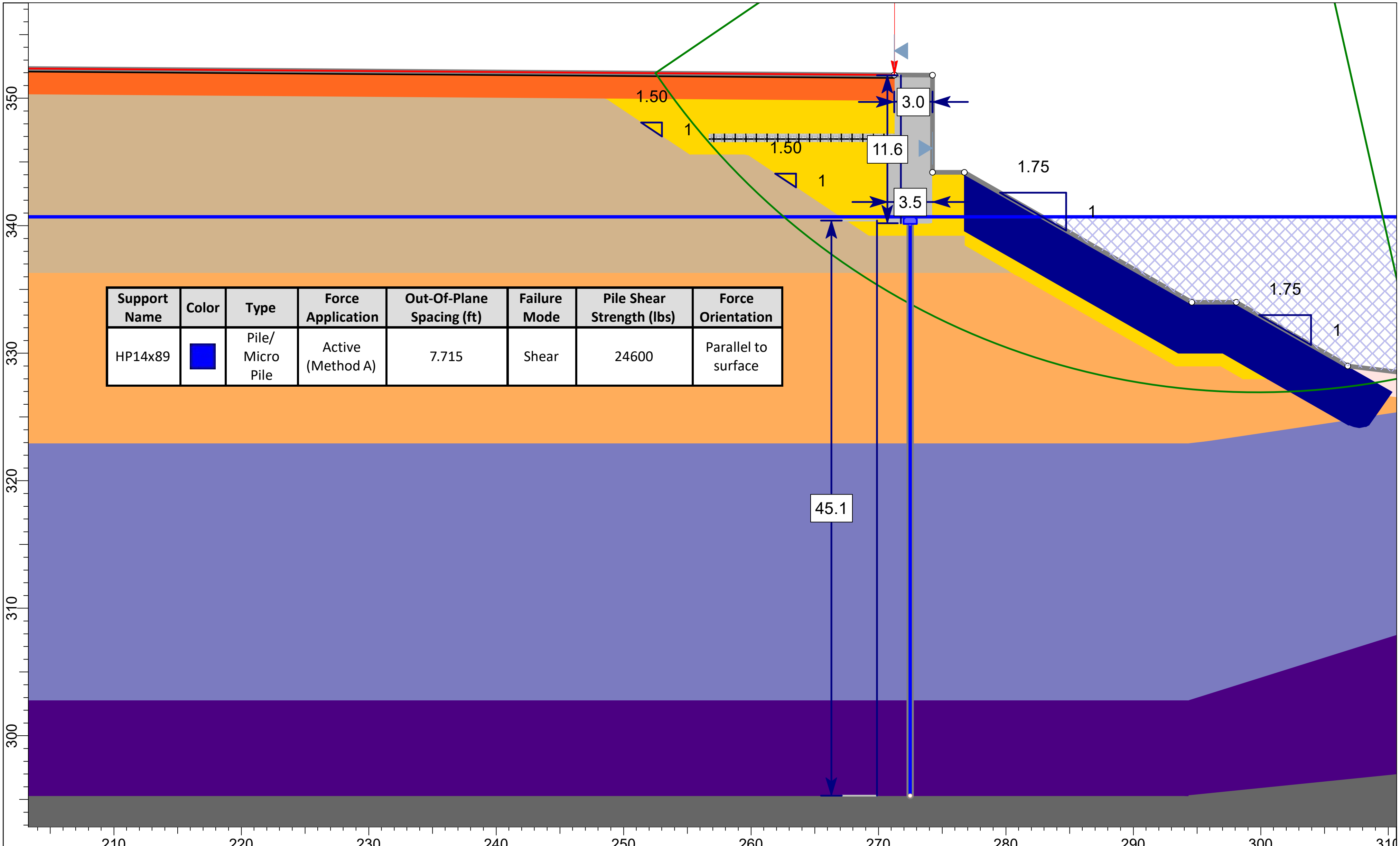
wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Static), Q1.1 and Piles					
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:150	**Figure 3J**		
SLIDEINTERPRET 9.040	*Date* 3/25/2026	*File Name* Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd					



wsp	*Project*	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME					
	Analysis Description	Abutment 1 - Proposed conditions (Non-Circular, Pseudostatic), Q1.1 and Piles					
	Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:150	**Figure 3K**		
SLIDEINTERPRET 9.040	*Date* 3/25/2026	*File Name* Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd					



wsp	Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME						
	Analysis Description	Abutment 2 - Proposed conditions (Non-Circular, Pseudostatic), Q1.1 and Piles						
	Drawn By	LDN	Checked By	DEB	Reviewed By	MEL	Scale	1:150
SLIDEINTERPRET 9.040	Date	3/25/2026	File Name	Bridge 3189 Longitudinal Global Stability Foreslopes 3.24.26.slmd				Figure 3L



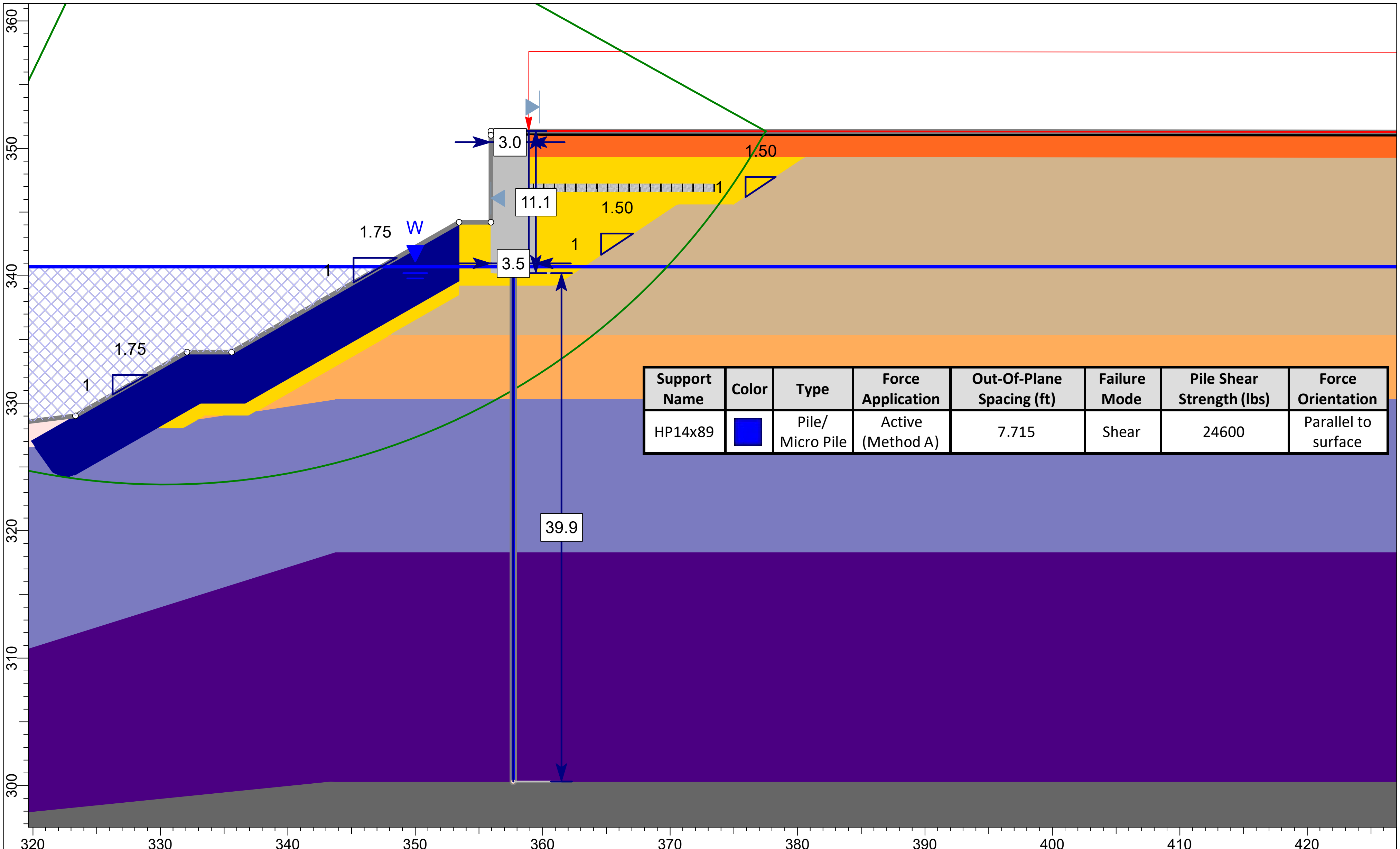
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
HP14x89		Pile/ Micro Pile	Active (Method A)	7.715	Shear	24600	Parallel to surface

wsp

SLIDEINTERPRET 9.040

Project	MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME
Analysis Description	Abutment 1 Pile Details

Drawn By	LDN	*Checked By*	DEB	*Reviewed By*	MEL	*Scale*	1:125	**Figure 4A**
Date	3/24/2026	*File Name*	Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd					



Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
HP14x89		Pile/ Micro Pile	Active (Method A)	7.715	Shear	24600	Parallel to surface

Project MaineDOT North Bridge Bundle Bridge #3189, Bancroft TWP, ME

Analysis Description East Abutment - Proposed conditions (Pseudostatic)

Drawn By LDN	*Checked By* DEB	*Reviewed By* MEL	*Scale* 1:125	**Figure 4B**
Date 3/24/2026	*File Name* Bridge 3189 Longitudinal Global Stability PILES 3.24.26.slmd			

SLIDEINTERPRET 9.040

CALCULATIONS

Date:	3/26/2026	**Made by:**	RJN/ATM
Project No.:	US0040947.9162, WIN 028262.00	**Checked by:**	LDN
Subject:	Lateral Earth Pressure at Abutments 1 and 2	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

OBJECTIVE

Determine lateral earth pressures acting on the proposed Abutments 1 and 2.

REFERENCES

1. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
2. AASHTO LRFD Bridge Design Specifications, 10th Ed. 2024.
3. Pile Load Calculations provided by Hoyle Tanner, Received on Thursday July 24, 2025.
4. Hoyle Tanner, Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Bridge 3189. Draft PS&E Plans dated April 22, 2026.
5. Das, Braja M. 2011. Principles of Foundation Engineering, 7th ed. Cengage Learning, Stamford, CT.
6. WSP boring logs, included as Appendix A of the Geotechnical Design Report.
7. WSP Material properties Analysis titled "NBB Bridge 3189 Geotechnical Material Properties.xlsx"
8. MassDOT Highway Division. LRFD Bridge Manual - Part I, January 2020 Revision. Chapter 3: Bridge Design Guidelines. https://www.mass.gov/doc/chapter-3-lrfd-bridge-design-guidelines/download

ASSUMPTIONS

1. The backfill surface behind each abutment is assumed to be horizontal.
2. The backfill is assumed to be free draining (i.e., no water pressure is allowed to build up behind the abutment wall).

CALCULATION

1. Calculate expected wall rotation for the integral abutment to select earth pressure case.

Per the MaineDOT Bridge Design Guide Section 5.4.2.11 (Ref. 1), integral abutments should be designed for the passive earth pressure (P_p) which results on the back face of the wall when the bridge expands.

Per AASHTO LRFD Table C3.11.1-1 (Ref. 2), in order to develop full passive earth pressure, wall rotation (the ratio of lateral abutment movement to abutment height) must exceed 0.02 for a medium dense cohesionless backfill such as the proposed Granular Borrow abutment backfill.

Table C3.11.1-1—Approximate Values of Relative Movements Required to Reach Active or Passive Earth Pressure Conditions (Clough and Duncan, 1991)

Type of Backfill	Values of Δ/H Active	Values of Δ/H Passive
Dense sand	0.001	0.01
Medium dense sand	0.002	0.02
Loose sand	0.004	0.04
Compacted silt	0.002	0.02
Compacted lean clay	0.010	0.05
Compacted fat clay	0.010	0.05

CALCULATIONS

Date:	3/26/2026	**Made by:**	RJN/ATM
Project No.:	US0040947.9162, WIN 028262.00	**Checked by:**	LDN
Subject:	Lateral Earth Pressure at Abutments 1 and 2	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

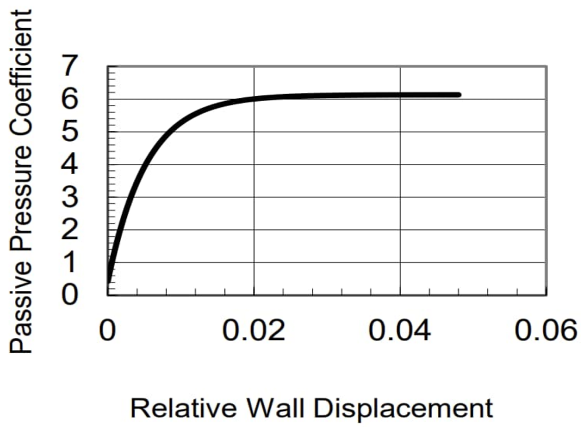
Maximum thermal expansion movement (from increasing temperature) =	0.143	inches	=	0.01	feet	(Ref. 3)
Abutment 1 height =	6.8	feet	(Ref. 4, Sheet 4, measured from the bottom of the approach slab to the bottom of the abutment as per Ref. 1 Section 5.4.2.11)			
Abutment 2 height =	6.8	feet				
Maximum wall rotation, Abutment 1 =	0.0018					
Maximum wall rotation, Abutment 2 =	0.0018					

Since the anticipated wall rotation does not exceed 0.02, full passive earth pressure is not expected to develop. The magnitude of partial earth pressure will be calculated using the procedure in the MassDOT LRFD Bridge Manual Section 3.10.8 (Ref. 8).

2. Calculate the movement-dependent passive earth pressure coefficient for conditions between full passive and at-rest earth pressure.

Ref. 8, Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$



Abutment 1:

δ_T/H =	0.0018	(from Step 1)
K =	2.04	(Ref. 8, Fig. 3.10.8-1)

Abutment 2:

δ_T/H =	0.0018	(from Step 1)
K =	2.04	(Ref. 8, Fig. 3.10.8-1)

3. Calculate the full passive earth pressure coefficient using Rankine theory for comparison with MassDOT K_p.

$$K_p = \tan^2\left(45° + \frac{\phi}{2}\right)$$ (Ref. 5, Eqn. 13.22)

where:

ϕ internal friction angle of backfill =	32	degrees	(MaineDOT Granular Borrow Ref. 1 Table 3-3)
K_p =	3.25		

Since the MassDOT procedure produces a passive earth pressure less than Rankine theory for passive earth pressure, the Rankine theory K_p will be used in design per MaineDOT.

CALCULATIONS

Date:	3/26/2026	**Made by:**	RJN/ATM
Project No.:	US0040947.9162, WIN 028262.00	**Checked by:**	LDN
Subject:	Lateral Earth Pressure at Abutments 1 and 2	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

4. Determine the unfactored passive pressure P_p acting on the abutments.

$$P_p = \frac{1}{2} \cdot \gamma_{soil} \cdot H_{abut}^{\ 2} \cdot K_p$$ (Ref. 1, page 5-51)

where:

γ_{soil} = unit weight of backfill =	125	pcf	(MaineDOT Granular Borrow Ref.1 Table 3-3)
H_{abut} = height of the Abutment 1 backwall =	6.8	feet	(Ref. 4)
H_{abut} = height of the Abutment 2 backwall =	6.8	feet	(Ref. 4)
K_p = passive earth pressure coefficient =	3.25	(from Step 3)	
Abutment 1 P_p =	9,406	lbs per foot of abutment width	
Abutment 2 P_p =	9,406	lbs per foot of abutment width	

The resultant lateral earth load acts at a height of H/3 above the base of the wall.
Based on Ref. 4, the base of Abutment 1 is EL 340.2 ft and Abutment 2 is EL 339.7 ft

For Abutment 1, the load acts at:	342.5	feet elevation
For Abutment 2, the load acts at:	342.0	feet elevation

5. Calculate Rankine active earth pressure coefficient.

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$ (Ref. 1, page 3-7)

for horizontal backfill surface, where:

ϕ internal friction angle of backfill =	32	degrees	(MaineDOT Granular Borrow Ref. 1 Table 3-3)
K_a =	0.31		

6. Calculate the at-rest lateral earth pressure coefficient.

$$K_0 = 1 - \sin\phi$$ (Ref. 5, Eqn. 13.5) For coarse-grained soils

where:

ϕ internal friction angle of backfill =	32	degrees	(MaineDOT Granular Borrow Ref. 1 Table 3-3)
K_0 =	0.47		

CALCULATIONS

Date:	3/26/2026	**Made by:**	RJN/ATM
Project No.:	US0040947.9162, WIN 028262.00	**Checked by:**	LDN
Subject:	Lateral Earth Pressure at Abutments 1 and 2	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

CONCLUSIONS

With the wall geometry and thermal lateral movement provided by Hoyle Tanner, the calculated rotation is less than that required to develop full passive earth pressure. Thus the Rankine method was used to determine the full passive earth pressure as prefered by MaineDOT. For the recommended soil parameters and given wall geometry, the full passive earth pressure coefficient is K_p = 3.25, which corresponds to an unfactored passive earth force of P_p = 9,406 pounds per linear foot for Abutment 1 acting at elevation 342.5 feet and an unfactored passive earth force of P_p = 9,406 pounds per linear foot for Abutment 2 acting at elevations 342.0 feet. The Rankine method was used to determine the active earth pressure coefficient of K_a = 0.31 and the at-rest lateral earth pressure coefficient K_0 = 0.47 for the granular borrow backfill.

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

OBJECTIVE

Determine if the proposed HP14x89 piles will provide adequate support for Abutment No. 1 of the proposed Smith Brook Bridge (Bridge No. 3189) based on the anticipated thermal movement, rotation and axial loads for the Strength I Load Combination.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Ed, 2024

2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

3. Hoyle Tanner, Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Bridge 3189. 60% Plans dated February 5. 2026.

4. WSP Interpretive Subsurface Profile, included as Sheet 3 of the Geotechnical Design Report (GDR).

5. FHWA. July 2016. GEC-012: Design and Construction of Driven Pile Foundations, Volume 1. Publication No. FHWA-NHI-16-009.

6. Email from Hoyle Tanner Structural team received on March 13, 2026 with subject line "RE: [External] NBB-Geotech-Ext: Bridge 3189 updated pile loads".

7. Hoyle Tanner, Abutment Loading Calculations, provided in "Integral Abutment Pile Loads_2025_07_24.pdf", received by WSP via email on July 24th, 2025.

8. WSP Laboratory Testing Results, included as Appendix C of the GDR.

9. WSP, Material properties table named "NBB Bridge 3189 Geotechnical Materials Properties.xlsx"

10. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2025. https://bsi.ce.ufl.edu/documentation/fbmp/manual

11. WSP USA Inc. Table 4: Summary of Rock Laboratory Testing Results, included in the GDR.

12. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

13. AISC Steel Construction Manual, 13th Ed. 2005.

14. WSP Boring Logs, included as Appendix A of the Geotechnical Design Report.

15. VTrans Structures Section, Integral Abutment Committee. 2008. Integral Abutment Bridge Design Guidelines, 2nd Ed.

16. FHWA, September 2018, GEC-010: Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024.

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on five HP14x89 Grade 50 piles with a center-to-center pile spacing of 7.715 feet, equal to the proposed bridge girder spacing adjusted for the abutment skew from Ref. 3, Sheet 8 and Ref. 7 Page 10. The piles are assumed to be driven to bedrock and thus end-bearing.

2. The selected pile orientation is weak axis bending (Ref. 2, page 5-42).

3. The vertical load is assumed to be evenly distributed.

4. The soil profile used in the analysis is based on Ref. 4 at the centerline of the proposed Abutment No. 1.

5. An UCS for rock of 7,000 psi is assumed based on a field characterization of R3 (Medium Strong Rock) to R4 (Strong Rock) from Ref. 14.

6. Based on previous experience on nearby projects and the bridge being located within a no salt zone, section loss due to corrosion is not considered for this anlaysis.

7. The piles are modeled in L-Pile as a fixed connection at the top of the pile.

8. Depth to fixity for deflection is taken as the depth along the pile to the point of zero lateral deflection; zero deflection is assumed to be deflection values less than 0.001 inches (Ref. 2 Section 5.4.2.5). Depth to fixity for bending moment is taken as the depth along the pile where the bending moment is less than 5% of the maximum bending moment (Ref. 16 page 12-19).

ATTACHMENTS

1. LPile analysis output for Strength I

CALCULATION

1. Select the pile section parameters

	Intact Section (Ref. 12)
Pile depth, d =	13.8
Pile width, b_f =	14.7
Web thickness, t_w =	0.615
Flange thickness, t_f =	0.615
Fillet area =	0.3
Section area =	26.1
Moment of inertia about the x-axis, I_x =	888
Moment of inertia about the y-axis, I_y =	326
Radius of gyration about the x-axis, r_x =	5.83
Radius of gyration about the y-axis, r_y =	3.53

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Elastic section modulus about the x-axis, S_x =	129
Elastic section modulus about the y-axis, S_y =	44.3
Plastic section modulus about the x-axis, Z_x =	143
Plastic section modulus about the y-axis, Z_y =	67.6

2. Determine pile loads and select the preliminary pile size.

Determine the factored substructure vertical dead and live load (P_u) distributed to each pile.

Strength I factored vertical load per pile = 363.0 kips (Ref. 6)

Pile weight = 0.089 kip/ft x 44.9 ft = 4.0 kips

P_u = 367 kips (Total factored axial load including pile weight.)

Select the steel pile strength.

F_y = 50 ksi (Assumption 1)
E = 29,000 ksi (typical value for steel)

Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} = 0.50 for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)

ϕ_{cu} = 0.70 for axial resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)

ϕ_f = 1.00 for flexural resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 524 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 734 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

$R_n =$ 734 kips

Use the required nominal axial pile resistance to estimate an approximate initial pile area.

$$A_{s.req} = \frac{R_n}{0.80\ Fy}$$ (Ref. 2, section 5-42)

$A_{s.req} =$ 18.3 in^2

Select a pile size with an area of approximately $A_{s.req}$.

HP14x89 section as per Assumption 1 and Step 1:

$A_s =$ 26.1 in^2 (Step 1)

3. Use LPile analysis to determine the pile unbraced length and maximum moment at the top of the pile.

The following input parameters were used in the LPile analysis:

Pile Properties

Section type:	Steel H Section Weak Axis		(Assumption 2)	
Length of section:	44.9	ft	(piles driven to bedrock)	
Flange width, b_f:	14.7	in	(Step 1)	
Section depth, d:	13.8	in	(Step 1)	
Flange thickness, t_f:	0.615	in	(Step 1)	
Web thickness, t_w:	0.615	in	(Step 1)	
Moment of inertia, I_y:	326	in^4	(Step 1)	about the weak axis, since weak-axis bending
Plastic section modulus, Z_y:	67.6	in^3	(Step 1)	about the weak axis, since weak-axis bending
Elastic section modulus, S_y:	44.3	in^3	(Step 1)	about the weak axis, since weak-axis bending
Radius of gyration, r_y:	3.53	in	(Step 1)	about the weak axis, since weak-axis bending
Pile batter:	Vertical		(pile battering not required)	

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Pile Loading

Case	ΔT (in) [1]	Rotation (rad)[2]	Axial Load (lbs)[3]
1 (Thermal contraction)	-0.39	0.000	367,000
2 (Thermal expansion)	0.14	-0.010	367,000

1) Strength I Lateral deflection due to abutment thermal expansion or contraction per Ref. 6
2) Assumed Rotations per Ref. 6.
3). From Step 2

Soil Layers

Layer	Depth below base of abutment[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Undrained Shear Strength (psf)	Friction Angle (°)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Existing Fill	0.0 - 3.9 ft	Sand (Reese)	125	-	36	165.0	-	-
Alluvial Sand & Gravel (AWT)	3.9 - 14.2 ft	Sand (Reese)	125	-	31	69	-	-
Alluvial Sand & Gravel (BWT)	14.2 - 17.2 ft	Sand (Reese)	62.6	-	31	42		
Alluvial Silt	17.2 - 37.4 ft	Sand (Reese)	52.6	-	31	42	-	-
Glacial Till	37.4 - 44.9 ft	Sand (Reese)	67.6	-	36	100.0	-	-
Bedrock	> 44.9 ft	Strong Rock (Vuggy Limestone)	102.6	-	-	-	-	7,000

1) Per Ref. 4, 3 and 14.
2) Ref. 9
3) Ref. 10 - interpolation based on friction angle for subgrade modulus
4) Assumption 5

The full LPile output is provided in Attachment 1.

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Obtain the maximum moment in the top segment of the pile using the loading case that produces the highest moment.

The thermal expansion load case (Load case 2), resulted in the highest moment in the pile.

$M_{u.Top}$ = 2392 in-kips (from LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 8.2 ft (from LPile)
$l_{b.top}$ = 98.4 in

$l_{b.2nd}$ = 11.6 ft (from LPile)
$l_{b.2nd}$ = 139.5 in

4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the interaction of combined axial and flexural load effects on a single pile.

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each segment.

For the top segment (rotation fixed with no plastic hinge):

$$\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{top} = 1.2 (Ref. 1, Table C4.6.2.5-1)
r_y = 3.53 in (Step 1)

λ_{top} = 33.5 OK

For the second segment (pinned at top and bottom):

$$\lambda_{2nd} = \frac{K_{2nd} l_{b.2nd}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{2nd} = 39.5 OK

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Calculate the critical elastic buckling resistance, P_e, and the nominal yield resistance, P_o.

Use Ref. 1 Table 6.9.4.1.1-1 to select equation for P_e based on cross-section shape and potential buckling mode.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\, l_b}{r_y}\right)^2} A_s \qquad \text{(Ref. 1, Eqn 6.9.4.1.2-1)}$$

$P_{e.top}$ = 6673 kips
$P_{e.2nd}$ = 4784 kips

$$P_o = F_y A_s \qquad \text{(Ref. 1, Article 6.9.4.1)}$$

P_o = 1305 kips

Calculate the nominal structural pile resistance, P_n, for both segments of the upper zone of the pile as well as the lower zone of the pile.

Determine P_o/P_e to select equation for P_n as per Ref. 1 Article 6.9.4.1.

$P_o/P_{e.top}$ = 0.20 ≤ 2.25
$P_o/P_{e.2nd}$ = 0.27 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 1202 kips
$P_{n.2nd}$ = 1164 kips

$$P_{n.bottom} = \left(0.658^{(0)}\right) P_o \qquad \text{(0 for a fully braced pile - Ref. 15, Appendix B, Eqn 6-9)}$$

$P_{n.bottom}$ = 1305 kips

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

$$P_{r.top} = \phi_{cu} P_{n.top}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.top} =$ 842 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.2nd} =$ 815 kips

$$P_{r.bottom} = \phi_{cl} P_{n.bottom}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.bottom} =$ 653 kips

Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20 (Ref. 2, page 5-43).

$$\frac{P_u}{P_{r.top}} = 0.44$$ OK

$$\frac{P_u}{P_{r.2nd}} = 0.45$$ OK

Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads (Ref. 15, Appendix B Eqn 6-17). Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check} \left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$$\frac{P_u}{P_{r.bottom}} = 0.56$$ OK

Determine the nominal and factored flexural resistance about the H-pile weak axis.

Slenderness ratio for the flange:

$$\lambda_f = \frac{b_f}{2t_f}$$ (Ref. 1, Eqn 6.12.2.2.1c-3)

$\lambda_f =$ 12.0

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
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Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1c-4)

$\lambda_{pf} =$ 9.2

Limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1c-5)

$\lambda r_f =$ 24.1

Since λpf < λf ≤ λrf, pile is noncompact per Ref. 1, Article 6.12.2.2.2c-12.

Elastic and plastic section moduli about the weak axis:

$S_y =$ 44.3 in^3 (Step 1)

$Z_y =$ 67.6 in^3 (Step 1)

Per Ref.1 section 6.12.2.2.1 the nominal flexural resistance about the weak axis shall be taken as the smaller value based on yielding or FLB (Flange Local Buckling).

Nominal flexural resistance based on yielding:

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad \text{, if } \lambda_f \leq \lambda_{pf}$$ (Ref. 1, Eqn 6.12.2.2.1b-1)

Nominal flexural resistance based on FLB:

Since λpf < λf ≤ λrf,

$$M_n = M_p - \left(M_p - 0.7\, F_{yf}\, S_y\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \text{ if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-1)

$$M_n = \frac{0.69 E S_y}{\lambda_f^2} \qquad \text{if } \lambda_f > \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-2)

For welded sections Fyf shall be taken as 0.5Fyc, where Fyc is the minimum yield stress of the compression flange.

$M_p =$ 3380 in-kips

$M_n =$ 2892 in-kips

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Factored flexural resistance:

$\phi_f =$ 1.00 (Ref. 2, page 5-42)

$$M_r = \phi_f M_n$$

$M_r =$ 2892 in-kips

For all members (including noncompact):

$$M_p' = \left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$

(Ref. 1, Eqn 6.9.2.2.1-3 and Ref. 15 Eq 4.5.2-3)

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'.

$M_p' =$ 1631 in-kips = 1630834 inch-lb

If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation, and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top} =$ 2392 in-kips (From Step 3)

$M_{u.Top}$ > M_p' Plastic Hinge Forms

Check the second segment of the upper zone of the pile to ensure that this segment of the pile does not form a plastic hinge.

$M_{u.2nd} =$ 171 in-kips (LPile)

Check: $$\frac{P_u}{P_{r.2nd}} + \left(\frac{M_{u.2nd}}{M_r}\right) < 1$$

(Ref. 1, Eqn 6.9.2.2.1-3 for noncompact memeber, Ref. 15, Eqn 4.5.2-3)

Check: 0.51 < 1 OK

5. Run another LPile analysis with displacement, plastic moment (M_p'), and P_u as load conditions, and calculate new unbraced lengths from the moment vs. depth curve. Then repeat Step 4 with the new unbraced lengths.

$l_{b.top} =$ 8.8 ft (LPile)

$l_{b.top} =$ 105.2 in

$l_{b.2nd} =$ 11.5 ft (LPile)

$l_{b.2nd} =$ 138.2 in

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

$M_{u.2nd}$ = 96 in-kips (LPile)

Since a plastic hinge developed at the pile head, the value of K for the top segment becomes 2.1 for free rotation condition (Ref. 2, page 5-43).

K_{top} = 2.1 (Ref. 1, Table C4.6.2.5-1)
K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{top} = 62.58 < 120 OK
λ_{2nd} = 39.14 < 120 OK

$P_{e.top}$ = 1907 kips
$P_{e.2nd}$ = 4876 kips

$P_o/P_{e.top}$ = 0.68 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd}$ = 0.27 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 980 kips
$P_{n.2nd}$ = 1167 kips

$P_{r.top}$ = 686 kips
$P_{r.2nd}$ = 817 kips

$\frac{P_u}{P_{r.top}}$ = 0.53 > 0.20 OK

$\frac{P_u}{P_{r.2nd}}$ = 0.45 > 0.20 OK

Check the second segment of the upper zone of the pile to ensure that this segment of the pile does not form a plastic hinge.

$M_{u.2nd}$ = 96 in-kips (LPile)

Check: $$\frac{P_u}{P_{r.2nd}} + \left(\frac{M_{u.2nd}}{M_r}\right) < 1$$ (Ref. 1, Eqn 6.9.2.2.1-3 for noncompact memeber, Ref. 15, Eqn 4.5.2-3)

Check: 0.48 < 1 OK

CALCULATIONS

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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

6. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPile output against the structural shear resistance per AISC G7.

$V_u =$ 21.1 kips (from LPile)

AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition, Specifications G2 and G7 (Ref. 13) to ensure the pile will not shear under the longitudinal load.

Per Ref. 13 Section G7: for singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength V_n for each shear resisting element shall be determined using Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ and $k_v = 1.2$.

$$V_n = 0.6 F_y A_w C_v$$ (Ref. 13, Eqn G2-1)

where:

$C_v =$ 1.0 for HP shapes with $F_y \le 50$ ksi (Ref. 13, Section G7)

$A_w = b_f t_f =$ 9.0 in^2 (Ref. 13, Section G7)

Since there are two shear resisting elements (i.e., two flanges):

$2 \times A_w =$ 18.1 in^2

$V_n =$ 542 kips

$$V_r = \Phi_v V_n$$

$\phi_v =$ 1.00 (Ref. 1, Article 6.5.4.2)

$V_r =$ 542 kips

Check that the shear resistance is sufficient:

$V_u < V_r$ OK

7. Check that the maximum factored applied pile load does not exceed the factored pile drivability resistance.

For steel piles in compression, driving stresses should not exceed 90% of the yield strength of the pile material (Ref. 2, page 5-104).

$\sigma_{dr} =$ 0.9 × 50 ksi = 45 ksi

This translates into an ultimate maximum driving force that can be applied to the pile of:

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

$P_{dr} = \sigma_{dr} A_s$ (Ref. 15, Appendix B, Eqn 7-23)

$P_{dr} =$ 1175 kips

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. In this design, the pile will be bearing on rock. The driving criteria will be established by dynamic testing.

$\phi_{dyn} =$ 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{dyn}}$$ (Ref. 2, Appendix B, Eqn 7-25)

$R_{ndr} =$ 565 kips

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_{dr}) calculated above.

$P_{n.top} =$ 1202 kips (From Step 4)

$P_{n.2nd} =$ 1164 kips (From Step 4)

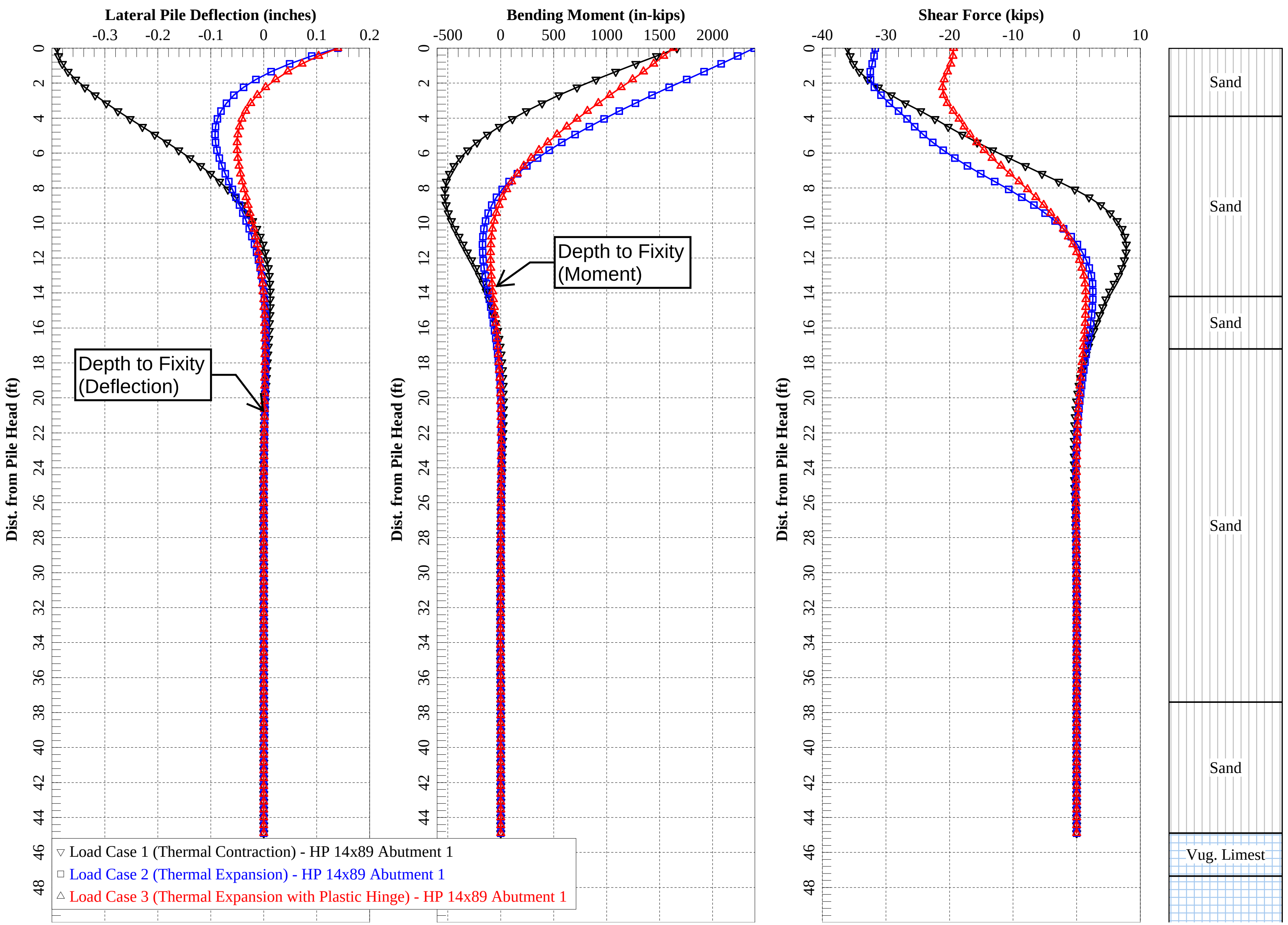
Check $R_{ndr} < P_n$: OK

Check $R_{ndr} < P_{dr}$: OK

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 2392 in-kips (199 ft-kips) and a plastic hinge moment of 1631 in-kips (136 ft-kips) occurs at the top of the pile under the Strength I load case with a maximum bridge expansion of 0.14 inches and a rotation of 0.010 radians. The results indicate that the depth to bedrock is sufficient for driven piles to achieve fixity. Based on the intact section used in the analysis, HP14x89 piles are adequately sized to accommodate the anticipated design loads and thermal movement at Abutment No.1 of the Smith Brook Bridge (Bridge No. 3189).

Attachment 1



===

LPile for Windows, Version 2022-12.007

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

===

This copy of LPile is being used by:

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Files Used for Analysis

Path to file locations:
\Users\USLP710461\Downloads\OneDrive_2026-03-19\LPile Models\

Name of input data file:
Abutment 1 HP14x89_ 3.20.26.lp12d

Name of output report file:
Abutment 1 HP14x89_ 3.20.26.lp12o

Name of plot output file:
Abutment 1 HP14x89_ 3.20.26.lp12p

Name of runtime message file:
Abutment 1 HP14x89_ 3.20.26.lp12r

Date and Time of Analysis

Date: March 20, 2026 Time: 13:34:33

Problem Title

Project Name: WIN 028262.00 Northern Bridge Bundle - Bridge No. 3189 -

Job Number: US0040947.9162

Client: MaineDOT

Engineer: WSP USA Inc.

Description: HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1

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                                      =       44.900 ft
Depth of ground surface below top of pile                 =       0.0000 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.

            Depth Below          Pile
Point        Pile Head         Diameter
 No.           feet             inches
-----      -------------     -------------
  1            0.000           14.7000
  2           44.900           14.7000


Input Structural Properties for Pile Sections:
----------------------------------------------

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                      =    44.900000 ft
```

```
   Pile width                                            =    13.800000 in



---------------------------------------------------------------------------------
                       Soil and Rock Layering Information
---------------------------------------------------------------------------------

The soil profile is modelled using 6 layers

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

   Distance from top of pile to top of layer             =       0.0000 ft
   Distance from top of pile to bottom of layer          =     3.900000 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                        =    36.000000 deg.
   Friction angle at bottom of layer                     =    36.000000 deg.
   Subgrade k at top of layer                            =   165.000000 pci
   Subgrade k at bottom of layer                         =   165.000000 pci


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

   Distance from top of pile to top of layer             =     3.900000 ft
   Distance from top of pile to bottom of layer          =    14.200000 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                        =    31.000000 deg.
   Friction angle at bottom of layer                     =    31.000000 deg.
   Subgrade k at top of layer                            =    69.000000 pci
   Subgrade k at bottom of layer                         =    69.000000 pci


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

   Distance from top of pile to top of layer             =    14.200000 ft
   Distance from top of pile to bottom of layer          =    17.200000 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                        =    31.000000 deg.
   Friction angle at bottom of layer                     =    31.000000 deg.
   Subgrade k at top of layer                            =    42.000000 pci
   Subgrade k at bottom of layer                         =    42.000000 pci


Layer 4 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer             =    17.200000 ft
   Distance from top of pile to bottom of layer          =    37.400000 ft
```

```
   Effective unit weight at top of layer               =    52.600000 pcf
   Effective unit weight at bottom of layer            =    52.600000 pcf
   Friction angle at top of layer                      =    31.000000 deg.
   Friction angle at bottom of layer                   =    31.000000 deg.
   Subgrade k at top of layer                          =    42.000000 pci
   Subgrade k at bottom of layer                       =    42.000000 pci


Layer 5 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer           =    37.400000 ft
   Distance from top of pile to bottom of layer        =    44.900000 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                      =    36.000000 deg.
   Friction angle at bottom of layer                   =    36.000000 deg.
   Subgrade k at top of layer                          =   100.000000 pci
   Subgrade k at bottom of layer                       =   100.000000 pci


Layer 6 is strong rock (vuggy limestone)

   Distance from top of pile to top of layer           =    44.900000 ft
   Distance from top of pile to bottom of layer        =    55.000000 ft
   Effective unit weight at top of layer               =   102.600000 pcf
   Effective unit weight at bottom of layer            =   102.600000 pcf
   Uniaxial compressive strength at top of layer       =        7000. psi
   Uniaxial compressive strength at bottom of layer    =        7000. psi


 (Depth of the lowest soil layer extends 10.100 ft below the pile tip)



--------------------------------------------------------------------------------
                        Summary of Input Soil Properties
--------------------------------------------------------------------------------

Layer         Soil Type          Layer      Effective    Angle of     Uniaxial

 Num.           Name             Depth       Unit Wt.    Friction        qu
 kpy
          (p-y Curve Type)        ft           pcf         deg.         psi
 pci
-----   -------------------   ----------   ----------   ----------   ----------
----------
  1           Sand                  0.00     125.0000      36.0000       --
165.0000
          (Reese, et al.)         3.9000     125.0000      36.0000       --
165.0000
```

2	Sand	3.9000	125.0000	31.0000	--	69.0000
	(Reese, et al.)	14.2000	125.0000	31.0000	--	69.0000
3	Sand	14.2000	62.6000	31.0000	--	42.0000
	(Reese, et al.)	17.2000	62.6000	31.0000	--	42.0000
4	Sand	17.2000	52.6000	31.0000	--	42.0000
	(Reese, et al.)	37.4000	52.6000	31.0000	--	42.0000
5	Sand	37.4000	67.6000	36.0000	--	100.0000
	(Reese, et al.)	44.9000	67.6000	36.0000	--	100.0000
6	Strong Rock	44.9000	102.6000	--	7000.	--
	(Vuggy Limestone)	55.0000	102.6000	--	7000.	--

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 = 3

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.39000 in	S = 0.0000 in/in	367000.	N.A.	Yes
2	5	y = 0.140000 in	S = -0.01000 in/in	367000.	N.A.	Yes
3	4	y = 0.140000 in	M = 1630834. in-lbs	367000.	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                                          =    44.900000 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:
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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
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1	367.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 367.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000442	41.7773444	9448389.	118.2345231	15.1514881	
0.00000884	83.5546887	9448389.	62.7922615	16.0845352	
0.00001326	125.3320331	9448389.	44.3115077	17.0175823	
0.00001769	167.1093775	9448389.	35.0711308	17.9506295	
0.00002211	208.8867218	9448389.	29.5269046	18.8836766	
0.00002653	250.6640662	9448389.	25.8307538	19.8167238	
0.00003095	292.4414106	9448389.	23.1906462	20.7497709	
0.00003537	334.2187549	9448389.	21.2105654	21.6828180	
0.00003979	375.9960993	9448389.	19.6705026	22.6158653	
0.00004422	417.7734437	9448389.	18.4384523	23.5489123	
0.00004864	459.5507880	9448389.	17.4304112	24.4819595	
0.00005306	501.3281324	9448389.	16.5903769	25.4150066	
0.00005748	543.1054768	9448389.	15.8795787	26.3480537	
0.00006190	584.8828211	9448389.	15.2703231	27.2811009	
0.00006632	626.6601655	9448389.	14.7423015	28.2141480	
0.00007075	668.4375098	9448389.	14.2802827	29.1471952	
0.00007517	710.2148542	9448389.	13.8726190	30.0802423	
0.00007959	751.9921986	9448389.	13.5102513	31.0132894	
0.00008401	793.7695429	9448389.	13.1860275	31.9463366	
0.00008843	835.5468873	9448389.	12.8942262	32.8793838	
0.00009285	877.3242317	9448389.	12.6302154	33.8124309	
0.00009728	919.1015760	9448389.	12.3902056	34.7454780	
0.0001017	960.8789204	9448389.	12.1710662	35.6785251	
0.0001061	1003.	9448389.	11.9701885	36.6115723	
0.0001105	1044.	9448389.	11.7853809	37.5446194	
0.0001150	1086.	9448389.	11.6147893	38.4776666	
0.0001194	1128.	9448389.	11.4568342	39.4107137	
0.0001238	1170.	9448389.	11.3101615	40.3437608	
0.0001282	1212.	9448389.	11.1736042	41.2768080	
0.0001326	1253.	9448389.	11.0461508	42.2098551	
0.0001371	1295.	9448389.	10.9269201	43.1429023	
0.0001415	1337.	9448389.	10.8151413	44.0759494	
0.0001459	1379.	9448389.	10.7101371	45.0089965	
0.0001503	1420.	9448389.	10.6113095	45.9420437	
0.0001548	1462.	9448389.	10.5181292	46.8750908	
0.0001592	1504.	9448389.	10.4301256	47.8081380	

0.0001636	1546.	9448389.	10.3468790	48.7411851	
0.0001680	1588.	9448389.	10.2680138	49.6742323	
0.0001724	1629.	9443723.	10.1940496	50.0000000	Y
0.0001813	1706.	9409764.	10.0617064	50.0000000	Y
0.0001901	1778.	9351217.	9.9471160	50.0000000	Y
0.0001990	1845.	9275072.	9.8474020	50.0000000	Y
0.0002078	1909.	9186226.	9.7603030	50.0000000	Y
0.0002167	1969.	9088653.	9.6839052	50.0000000	Y
0.0002255	2026.	8985149.	9.6166785	50.0000000	Y
0.0002343	2080.	8877224.	9.5575126	50.0000000	Y
0.0002432	2132.	8767936.	9.5049842	50.0000000	Y
0.0002520	2182.	8657023.	9.4585881	50.0000000	Y
0.0002609	2230.	8546441.	9.4172999	50.0000000	Y
0.0002697	2276.	8436902.	9.3804708	50.0000000	Y
0.0002786	2320.	8328745.	9.3476031	50.0000000	Y
0.0002874	2363.	8221937.	9.3183563	50.0000000	Y
0.0002962	2405.	8117206.	9.2921946	50.0000000	Y
0.0003051	2445.	8014781.	9.2687565	50.0000000	Y
0.0003139	2485.	7914802.	9.2477355	50.0000000	Y
0.0003228	2523.	7815381.	9.2284423	50.0000000	Y
0.0003316	2558.	7712825.	9.2096343	50.0000000	Y
0.0003405	2590.	7608368.	9.1911858	50.0000000	Y
0.0003493	2621.	7502911.	9.1730365	50.0000000	Y
0.0003582	2649.	7397217.	9.1551297	50.0000000	Y
0.0003670	2676.	7291027.	9.1378246	50.0000000	Y
0.0003758	2701.	7185666.	9.1207678	50.0000000	Y
0.0003847	2724.	7081536.	9.1039508	50.0000000	Y
0.0003935	2746.	6977697.	9.0876016	50.0000000	Y
0.0004024	2767.	6875589.	9.0712177	50.0000000	Y
0.0004112	2786.	6774695.	9.0554922	50.0000000	Y
0.0004201	2804.	6676230.	9.0399111	50.0000000	Y
0.0004289	2822.	6578534.	9.0246785	50.0000000	Y
0.0004377	2838.	6483272.	9.0094336	50.0000000	Y
0.0004466	2854.	6389730.	8.9950233	50.0000000	Y
0.0004554	2868.	6297812.	8.9803119	50.0000000	Y
0.0004643	2882.	6208019.	8.9661427	50.0000000	Y
0.0004731	2895.	6120073.	8.9522270	50.0000000	Y
0.0004820	2908.	6034149.	8.9382517	50.0000000	Y
0.0004908	2920.	5950014.	8.9249942	50.0000000	Y
0.0004996	2932.	5867483.	8.9116029	50.0000000	Y
0.0005085	2943.	5787469.	8.8985011	50.0000000	Y
0.0005173	2953.	5708419.	8.8856728	50.0000000	Y
0.0005262	2963.	5631656.	8.8731656	50.0000000	Y
0.0005615	2999.	5340961.	8.8245281	50.0000000	Y
0.0005969	3030.	5075908.	8.7791277	50.0000000	Y
0.0006323	3056.	4833303.	8.7361931	50.0000000	Y
0.0006677	3079.	4611513.	8.6958050	50.0000000	Y
0.0007030	3099.	4407785.	8.6579820	50.0000000	Y
0.0007384	3116.	4220367.	8.6220712	50.0000000	Y
0.0007738	3132.	4047417.	8.5879985	50.0000000	Y

0.0008092	3146.	3887458.	8.5556478	50.0000000	Y
0.0008445	3158.	3739153.	8.5250335	50.0000000	Y
0.0008799	3169.	3601594.	8.4957228	50.0000000	Y
0.0009153	3179.	3473493.	8.4683918	50.0000000	Y
0.0009507	3188.	3353733.	8.4419138	50.0000000	Y
0.0009860	3197.	3241904.	8.4165778	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	367.0000000000	3197.

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	0.00	0.00	N.A.	No	0.00	16622.
2	3.9000	4.7124	Yes	No	16622.	289497.
3	14.2000	15.0132	Yes	No	306119.	192959.
4	17.2000	18.4400	Yes	No	499079.	1768491.
5	37.4000	31.2838	Yes	No	2267570.	1621018.
6	44.9000	44.9000	No	Yes	N.A.	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 = -0.390000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 367000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.390	1664884.	-35987.	0.00	51774.	9.43E+09	0.00	0.00	0.00
0.4490	-0.387	1471039.	-35639.	8.96E-04	47401.	9.43E+09	60.8141	845.7286	0.00
0.8980	-0.380	1277297.	-35113.	0.00168	43031.	9.45E+09	134.4682	1905.	0.00
1.3470	-0.369	1086018.	-34183.	0.00235	38716.	9.45E+09	210.5603	3072.	0.00
1.7960	-0.355	899628.	-32860.	0.00292	34512.	9.45E+09	280.5084	4258.	0.00
2.2450	-0.338	720366.	-31177.	0.00338	30468.	9.45E+09	344.2665	5490.	0.00
2.6940	-0.319	550286.	-29185.	0.00374	26631.	9.45E+09	395.1956	6685.	0.00
3.1430	-0.297	391058.	-26957.	0.00401	23040.	9.45E+09	431.6319	7817.	0.00
3.5920	-0.275	243920.	-24557.	0.00419	19721.	9.45E+09	459.5919	8996.	0.00
4.0410	-0.252	109849.	-22310.	0.00430	16696.	9.45E+09	374.3973	7996.	0.00
4.4900	-0.229	-13477.	-20221.	0.00432	14522.	9.45E+09	400.9926	9435.	0.00
4.9390	-0.206	-125147.	-17996.	0.00428	17041.	9.45E+09			

424.8743	11128.	0.00				
5.3880	-0.183	-224341.	-15658.	0.00418	19279.	9.45E+09
442.8881	13052.	0.00				
5.8370	-0.161	-310425.	-13215.	0.00403	21221.	9.45E+09
464.1447	15568.	0.00				
6.2860	-0.139	-382685.	-10673.	0.00383	22851.	9.45E+09
479.1552	18521.	0.00				
6.7350	-0.119	-440602.	-8070.	0.00360	24157.	9.45E+09
487.3278	22005.	0.00				
7.1840	-0.101	-483876.	-5442.	0.00334	25133.	9.45E+09
488.1924	26144.	0.00				
7.6330	-0.08339	-512432.	-2830.	0.00305	25778.	9.45E+09
481.3943	31106.	0.00				
8.0820	-0.06773	-526434.	-311.641	0.00275	26093.	9.45E+09
453.2655	36056.	0.00				
8.5310	-0.05370	-526685.	1931.	0.00245	26099.	9.45E+09
379.3134	38059.	0.00				
8.9800	-0.04128	-515330.	3780.	0.00216	25843.	9.45E+09
306.9587	40062.	0.00				
9.4290	-0.03045	-494482.	5248.	0.00187	25373.	9.45E+09
237.7338	42065.	0.00				
9.8780	-0.02114	-466176.	6354.	0.00160	24734.	9.45E+09
172.8808	44068.	0.00				
10.3270	-0.01326	-432325.	7125.	0.00134	23971.	9.45E+09
113.3505	46071.	0.00				
10.7760	-0.00670	-394695.	7591.	0.00110	23122.	9.45E+09
59.8125	48075.	0.00				
11.2250	-0.00136	-354885.	7787.	8.90E-04	22224.	9.45E+09
12.6736	50078.	0.00				
11.6740	0.00289	-314306.	7746.	6.99E-04	21308.	9.45E+09
-27.896	52081.	0.00				
12.1230	0.00617	-274183.	7504.	5.31E-04	20403.	9.45E+09
-61.931	54084.	0.00				
12.5720	0.00861	-235548.	7095.	3.86E-04	19532.	9.45E+09
-89.639	56087.	0.00				
13.0210	0.01033	-199250.	6554.	2.62E-04	18713.	9.45E+09
-111.359	58090.	0.00				
13.4700	0.01143	-165961.	5910.	1.58E-04	17962.	9.45E+09
-127.529	60093.	0.00				
13.9190	0.01203	-136186.	5193.	7.17E-05	17290.	9.45E+09
-138.643	62096.	0.00				
14.3680	0.01221	-110282.	4582.	1.43E-06	16706.	9.45E+09
-88.396	39017.	0.00				
14.8170	0.01205	-86821.	4101.	-5.48E-05	16177.	9.45E+09
-89.951	40236.	0.00				
15.2660	0.01162	-65873.	3618.	-9.83E-05	15704.	9.45E+09
-89.380	41456.	0.00				
15.7150	0.01099	-47445.	3143.	-1.31E-04	15289.	9.45E+09
-87.011	42675.	0.00				
16.1640	0.01021	-31490.	2684.	-1.53E-04	14929.	9.45E+09

-83.170	43894.	0.00				
16.6130	0.00934	-17914.	2250.	-1.67E-04	14623.	9.45E+09
-78.167	45113.	0.00				
17.0620	0.00841	-6587.	1844.	-1.74E-04	14367.	9.45E+09
-72.296	46333.	0.00				
17.5110	0.00746	2649.	1472.	-1.75E-04	14278.	9.45E+09
-65.825	47552.	0.00				
17.9600	0.00652	9971.	1136.	-1.72E-04	14443.	9.45E+09
-58.999	48771.	0.00				
18.4090	0.00561	15569.	836.7946	-1.64E-04	14570.	9.45E+09
-52.032	49991.	0.00				
18.8580	0.00475	19639.	575.1035	-1.54E-04	14661.	9.45E+09
-45.107	51210.	0.00				
19.3070	0.00394	22377.	350.1921	-1.42E-04	14723.	9.45E+09
-38.379	52429.	0.00				
19.7560	0.00321	23976.	160.6632	-1.29E-04	14759.	9.45E+09
-31.973	53648.	0.00				
20.2050	0.00255	24619.	4.5228	-1.15E-04	14774.	9.45E+09
-25.985	54868.	0.00				
20.6540	0.00197	24480.	-120.674	-1.01E-04	14771.	9.45E+09
-20.487	56087.	0.00				
21.1030	0.00146	23720.	-217.687	-8.76E-05	14753.	9.45E+09
-15.524	57306.	0.00				
21.5520	0.00102	22481.	-289.473	-7.44E-05	14726.	9.45E+09
-11.123	58526.	0.00				
22.0010	6.57E-04	20895.	-339.078	-6.21E-05	14690.	9.45E+09
-7.290	59745.	0.00				
22.4500	3.55E-04	19073.	-369.543	-5.07E-05	14649.	9.45E+09
-4.018	60964.	0.00				
22.8990	1.11E-04	17113.	-383.832	-4.04E-05	14604.	9.45E+09
-1.286	62183.	0.00				
23.3480	-7.97E-05	15096.	-384.767	-3.12E-05	14559.	9.45E+09
0.9384	63403.	0.00				
23.7970	-2.25E-04	13090.	-374.985	-2.31E-05	14514.	9.45E+09
2.6927	64622.	0.00				
24.2460	-3.29E-04	11147.	-356.898	-1.62E-05	14470.	9.45E+09
4.0211	65841.	0.00				
24.6950	-3.99E-04	9308.	-332.675	-1.04E-05	14428.	9.45E+09
4.9705	67061.	0.00				
25.1440	-4.41E-04	7603.	-304.227	-5.57E-06	14390.	9.45E+09
5.5892	68280.	0.00				
25.5930	-4.59E-04	6052.	-273.206	-1.68E-06	14355.	9.45E+09
5.9256	69499.	0.00				
26.0420	-4.59E-04	4666.	-241.008	1.38E-06	14324.	9.45E+09
6.0261	70718.	0.00				
26.4910	-4.45E-04	3449.	-208.784	3.69E-06	14296.	9.45E+09
5.9352	71938.	0.00				
26.9400	-4.19E-04	2401.	-177.456	5.36E-06	14273.	9.45E+09
5.6938	73157.	0.00				
27.3890	-3.87E-04	1516.	-147.734	6.48E-06	14253.	9.45E+09

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5.3391     74376.       0.00
   27.8380  -3.50E-04   783.5287   -120.138   7.13E-06     14236.   9.45E+09
4.9043     75596.       0.00
   28.2870  -3.10E-04   192.8692    -95.022   7.41E-06     14223.   9.45E+09
4.4184     76815.       0.00
   28.7360  -2.70E-04   -269.739    -72.596   7.39E-06     14225.   9.45E+09
3.9060     78034.       0.00
   29.1850  -2.30E-04   -618.650    -52.948   7.14E-06     14232.   9.45E+09
3.3875     79253.       0.00
   29.6340  -1.93E-04   -868.523    -36.064   6.71E-06     14238.   9.45E+09
2.8796     80473.       0.00
   30.0830  -1.58E-04     -1034.    -21.854   6.17E-06     14242.   9.45E+09
2.3952     81692.       0.00
   30.5320  -1.26E-04     -1128.    -10.164   5.55E-06     14244.   9.45E+09
1.9439     82911.       0.00
   30.9810  -9.81E-05     -1165.     -0.799   4.90E-06     14245.   9.45E+09
1.5324     84131.       0.00
   31.4300  -7.35E-05     -1156.     6.4678   4.24E-06     14245.   9.45E+09
1.1649     85350.       0.00
   31.8790  -5.25E-05     -1112.    11.8781   3.59E-06     14244.   9.45E+09
0.8434     86569.       0.00
   32.3280  -3.49E-05     -1043.    15.6801   2.98E-06     14242.   9.45E+09
0.5679     87788.       0.00
   32.7770  -2.04E-05   -955.163    18.1194   2.41E-06     14240.   9.45E+09
0.3375     89008.       0.00
   33.2260  -8.94E-06   -856.855    19.4316   1.89E-06     14238.   9.45E+09
0.1496     90227.       0.00
   33.6750  -7.44E-08   -753.237    19.8382   1.43E-06     14235.   9.45E+09
0.00126     91446.       0.00
   34.1240   6.47E-06   -648.734    19.5417   1.03E-06     14233.   9.45E+09
-0.111     92665.       0.00
   34.5730   1.10E-05   -546.730    18.7242   6.89E-07     14231.   9.45E+09
-0.192     93885.       0.00
   35.0220   1.39E-05   -449.687    17.5457   4.05E-07     14229.   9.45E+09
-0.245     95104.       0.00
   35.4710   1.54E-05   -359.260    16.1435   1.74E-07     14227.   9.45E+09
-0.275     96323.       0.00
   35.9200   1.58E-05   -276.415    14.6327  -6.79E-09     14225.   9.45E+09
-0.286     97543.       0.00
   36.3690   1.53E-05   -201.551    13.1067  -1.43E-07     14223.   9.45E+09
-0.281     98762.       0.00
   36.8180   1.42E-05   -134.612    11.6385  -2.39E-07     14221.   9.45E+09
-0.264     99981.       0.00
   37.2670   1.27E-05    -75.190    10.2819  -2.99E-07     14220.   9.45E+09
-0.239    101200.       0.00
   37.7160   1.10E-05    -22.632     8.2937  -3.27E-07     14219.   9.45E+09
-0.499    243857.       0.00
   38.1650   9.22E-06    15.4742     5.8122  -3.29E-07     14219.   9.45E+09
-0.422    246760.       0.00
   38.6140   7.48E-06    41.2998     3.7409  -3.12E-07     14219.   9.45E+09
```

							-0.346	249663.	0.00
39.0630	5.86E-06	57.0216	2.0680	-2.84E-07	14220.	9.45E+09	-0.275	252566.	0.00
39.5120	4.41E-06	64.7096	0.7650	-2.50E-07	14220.	9.45E+09	-0.209	255469.	0.00
39.9610	3.17E-06	66.2526	-0.207	-2.12E-07	14220.	9.45E+09	-0.152	258372.	0.00
40.4100	2.12E-06	63.3148	-0.894	-1.75E-07	14220.	9.45E+09	-0.103	261275.	0.00
40.8590	1.27E-06	57.3176	-1.339	-1.41E-07	14220.	9.45E+09	-0.06248	264178.	0.00
41.3080	6.02E-07	49.4418	-1.588	-1.11E-07	14220.	9.45E+09	-0.02986	267081.	0.00
41.7570	8.23E-08	40.6435	-1.679	-8.49E-08	14219.	9.45E+09	-0.00412	269984.	0.00
42.2060	-3.13E-07	31.6795	-1.648	-6.43E-08	14219.	9.45E+09	0.01585	272887.	0.00
42.6550	-6.11E-07	23.1399	-1.521	-4.87E-08	14219.	9.45E+09	0.03126	275790.	0.00
43.1040	-8.38E-07	15.4817	-1.320	-3.77E-08	14219.	9.45E+09	0.04332	278693.	0.00
43.5530	-1.02E-06	9.0636	-1.060	-3.07E-08	14219.	9.45E+09	0.05314	281596.	0.00
44.0020	-1.17E-06	4.1779	-0.751	-2.69E-08	14219.	9.45E+09	0.06168	284499.	0.00
44.4510	-1.31E-06	1.0779	-0.397	-2.54E-08	14218.	9.45E+09	0.06969	287402.	0.00
44.9000	-1.44E-06	0.00	0.00	-2.51E-08	14218.	9.45E+09	0.07768	145153.	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            =     -0.3900000 inches
Computed slope at pile head     =       0.000000 radians
Maximum bending moment          =       1664884. inch-lbs
Maximum shear force             =        -35987. 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 =             4
```

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 0.140000 inches
Rotation of pile head = -1.000E-02 radians
Axial load on pile head = 367000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.1400	2392317.	-31657.	-0.01000	68183.	8.15E+09	0.00	0.00	0.00
0.4490	0.09038	2239480.	-31853.	-0.00847	64735.	8.15E+09	-39.777	2371.	0.00
0.8980	0.04874	2082561.	-32156.	-0.00710	61195.	8.87E+09	-72.686	8035.	0.00
1.3470	0.01391	1921031.	-32452.	-0.00590	57552.	9.17E+09	-37.110	14370.	0.00
1.7960	-0.01483	1756191.	-32410.	-0.00483	53833.	9.37E+09	52.7270	19160.	0.00
2.2450	-0.03813	1590884.	-31812.	-0.00387	50104.	9.45E+09	169.1050	23898.	0.00
2.6940	-0.05654	1428692.	-30763.	-0.00301	46446.	9.45E+09	220.2957	20994.	0.00
3.1430	-0.07056	1271285.	-29470.	-0.00224	42895.	9.45E+09	259.7810	19837.	0.00
3.5920	-0.08067	1119986.	-28001.	-0.00156	39482.	9.45E+09	285.5591	19072.	0.00
4.0410	-0.08735	975713.	-26669.	-9.61E-04	36228.	9.45E+09	208.8312	12881.	0.00
4.4900	-0.09103	836403.	-25470.	-4.44E-04	33085.	9.45E+09	236.0981	13975.	0.00
4.9390	-0.09213	703004.	-24123.	-4.90E-06	30076.	9.45E+09	264.0068	15439.	0.00
5.3880	-0.09108	576476.	-22630.	3.60E-04	27222.	9.45E+09	289.9144	17151.	0.00
5.8370	-0.08825	457715.	-20981.	6.55E-04	24543.	9.45E+09	322.4668	19687.	0.00
6.2860	-0.08402	347799.	-19163.	8.84E-04	22064.	9.45E+09	352.1981	22585.	0.00
6.7350	-0.07872	247715.	-17195.	0.00105	19806.	9.45E+09			

378.4165	25900.	0.00				
7.1840	-0.07266	158338.	-15096.	0.00117	17790.	9.45E+09
400.5207	29699.	0.00				
7.6330	-0.06611	80409.	-12892.	0.00124	16032.	9.45E+09
417.8490	34053.	0.00				
8.0820	-0.05932	14520.	-10697.	0.00127	14546.	9.45E+09
396.9605	36056.	0.00				
8.5310	-0.05248	-39861.	-8629.	0.00126	15118.	9.45E+09
370.7048	38059.	0.00				
8.9800	-0.04576	-83436.	-6713.	0.00122	16101.	9.45E+09
340.2746	40062.	0.00				
9.4290	-0.03930	-117038.	-4970.	0.00117	16859.	9.45E+09
306.8517	42065.	0.00				
9.8780	-0.03320	-141600.	-3412.	0.00109	17413.	9.45E+09
271.5666	44068.	0.00				
10.3270	-0.02754	-158119.	-2046.	0.00101	17785.	9.45E+09
235.4656	46071.	0.00				
10.7760	-0.02236	-167624.	-873.830	9.14E-04	18000.	9.45E+09
199.4868	48075.	0.00				
11.2250	-0.01769	-171149.	106.5984	8.17E-04	18079.	9.45E+09
164.4435	50078.	0.00				
11.6740	-0.01355	-169706.	902.5631	7.20E-04	18047.	9.45E+09
131.0148	52081.	0.00				
12.1230	-0.00994	-164269.	1524.	6.25E-04	17924.	9.45E+09
99.7426	54084.	0.00				
12.5720	-0.00682	-155751.	1984.	5.33E-04	17732.	9.45E+09
71.0348	56087.	0.00				
13.0210	-0.00419	-144995.	2297.	4.48E-04	17489.	9.45E+09
45.1719	58090.	0.00				
13.4700	-0.00200	-132765.	2479.	3.68E-04	17213.	9.45E+09
22.3193	60093.	0.00				
13.9190	-2.20E-04	-119737.	2546.	2.96E-04	16919.	9.45E+09
2.5405	62096.	0.00				
14.3680	0.00119	-106499.	2530.	2.32E-04	16621.	9.45E+09
-8.635	39017.	0.00				
14.8170	0.00228	-93393.	2461.	1.75E-04	16325.	9.45E+09
-17.012	40236.	0.00				
15.2660	0.00308	-80675.	2351.	1.25E-04	16038.	9.45E+09
-23.672	41456.	0.00				
15.7150	0.00363	-68553.	2210.	8.27E-05	15765.	9.45E+09
-28.730	42675.	0.00				
16.1640	0.00397	-57188.	2045.	4.68E-05	15508.	9.45E+09
-32.322	43894.	0.00				
16.6130	0.00413	-46697.	1865.	1.72E-05	15272.	9.45E+09
-34.597	45113.	0.00				
17.0620	0.00415	-37158.	1676.	-6.71E-06	15057.	9.45E+09
-35.712	46333.	0.00				
17.5110	0.00406	-28614.	1483.	-2.55E-05	14864.	9.45E+09
-35.829	47552.	0.00				
17.9600	0.00388	-21077.	1292.	-3.96E-05	14694.	9.45E+09

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-35.108     48771.       0.00
   18.4090   0.00363    -14536.      1106.  -4.98E-05     14546.   9.45E+09
-33.704     49991.       0.00
   18.8580   0.00334     -8957.   930.1104  -5.65E-05     14420.   9.45E+09
-31.765     51210.       0.00
   19.3070   0.00302     -4289.   765.2625  -6.03E-05     14315.   9.45E+09
-29.426     52429.       0.00
   19.7560   0.00269   -471.734   613.7576  -6.16E-05     14229.   9.45E+09
-26.812     53648.       0.00
   20.2050   0.00236      2568.   476.7807  -6.10E-05     14276.   9.45E+09
-24.033     54868.       0.00
   20.6540   0.00204      4907.   354.9600  -5.89E-05     14329.   9.45E+09
-21.186     56087.       0.00
   21.1030   0.00173      6626.   248.4437  -5.56E-05     14368.   9.45E+09
-18.352     57306.       0.00
   21.5520   0.00144      7804.   156.9779  -5.15E-05     14394.   9.45E+09
-15.599     58526.       0.00
   22.0010   0.00117      8521.    79.9808  -4.68E-05     14411.   9.45E+09
-12.982     59745.       0.00
   22.4500  9.31E-04      8852.    16.6147  -4.19E-05     14418.   9.45E+09
-10.540     60964.       0.00
   22.8990  7.19E-04      8866.    -34.149  -3.68E-05     14418.   9.45E+09
-8.304     62183.       0.00
   23.3480  5.35E-04      8629.    -73.469  -3.18E-05     14413.   9.45E+09
-6.292     63403.       0.00
   23.7970  3.76E-04      8200.   -102.582  -2.70E-05     14403.   9.45E+09
-4.515     64622.       0.00
   24.2460  2.43E-04      7631.   -122.756  -2.25E-05     14391.   9.45E+09
-2.974     65841.       0.00
   24.6950  1.34E-04      6966.   -135.251  -1.84E-05     14376.   9.45E+09
-1.664     67061.       0.00
   25.1440  4.55E-05      6246.   -141.288  -1.46E-05     14359.   9.45E+09
-0.577     68280.       0.00
   25.5930 -2.35E-05      5502.   -142.024  -1.12E-05     14343.   9.45E+09
0.3036     69499.       0.00
   26.0420 -7.57E-05      4760.   -138.531  -8.32E-06     14326.   9.45E+09
0.9931     70718.       0.00
   26.4910 -1.13E-04      4042.   -131.785  -5.81E-06     14310.   9.45E+09
1.5110     71938.       0.00
   26.9400 -1.38E-04      3363.   -122.657  -3.70E-06     14294.   9.45E+09
1.8772     73157.       0.00
   27.3890 -1.53E-04      2734.   -111.910  -1.96E-06     14280.   9.45E+09
2.1122     74376.       0.00
   27.8380 -1.59E-04      2165.   -100.196  -5.62E-07     14267.   9.45E+09
2.2359     75596.       0.00
   28.2870 -1.59E-04      1657.    -88.063   5.28E-07     14256.   9.45E+09
2.2677     76815.       0.00
   28.7360 -1.54E-04      1213.    -75.958   1.35E-06     14246.   9.45E+09
2.2257     78034.       0.00
   29.1850 -1.45E-04   833.1566    -64.233   1.93E-06     14237.   9.45E+09
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2.1263	79253.	0.00				
29.6340	-1.33E-04	513.6463	-53.158	2.31E-06	14230.	9.45E+09
1.9847	80473.	0.00				
30.0830	-1.20E-04	251.1724	-42.925	2.53E-06	14224.	9.45E+09
1.8138	81692.	0.00				
30.5320	-1.06E-04	41.0696	-33.661	2.62E-06	14219.	9.45E+09
1.6250	82911.	0.00				
30.9810	-9.14E-05	-121.906	-25.437	2.59E-06	14221.	9.45E+09
1.4279	84131.	0.00				
31.4300	-7.77E-05	-243.292	-18.276	2.49E-06	14224.	9.45E+09
1.2303	85350.	0.00				
31.8790	-6.46E-05	-328.687	-12.164	2.32E-06	14226.	9.45E+09
1.0385	86569.	0.00				
32.3280	-5.26E-05	-383.564	-7.057	2.12E-06	14227.	9.45E+09
0.8573	87788.	0.00				
32.7770	-4.18E-05	-413.122	-2.888	1.89E-06	14228.	9.45E+09
0.6900	89008.	0.00				
33.2260	-3.22E-05	-422.181	0.4233	1.66E-06	14228.	9.45E+09
0.5392	90227.	0.00				
33.6750	-2.39E-05	-415.111	2.9696	1.42E-06	14228.	9.45E+09
0.4060	91446.	0.00				
34.1240	-1.69E-05	-395.787	4.8474	1.19E-06	14227.	9.45E+09
0.2910	92665.	0.00				
34.5730	-1.11E-05	-367.568	6.1541	9.69E-07	14227.	9.45E+09
0.1940	93885.	0.00				
35.0220	-6.48E-06	-333.302	6.9850	7.69E-07	14226.	9.45E+09
0.1144	95104.	0.00				
35.4710	-2.85E-06	-295.339	7.4303	5.90E-07	14225.	9.45E+09
0.05093	96323.	0.00				
35.9200	-1.25E-07	-255.565	7.5737	4.33E-07	14224.	9.45E+09
0.00226	97543.	0.00				
36.3690	1.81E-06	-215.437	7.4902	2.98E-07	14223.	9.45E+09
-0.03324	98762.	0.00				
36.8180	3.09E-06	-176.031	7.2461	1.87E-07	14222.	9.45E+09
-0.05735	99981.	0.00				
37.2670	3.83E-06	-138.091	6.8980	9.72E-08	14222.	9.45E+09
-0.07187	101200.	0.00				
37.7160	4.14E-06	-102.082	6.1999	2.87E-08	14221.	9.45E+09
-0.187	243857.	0.00				
38.1650	4.14E-06	-71.395	5.1851	-2.07E-08	14220.	9.45E+09
-0.189	246760.	0.00				
38.6140	3.91E-06	-46.126	4.1861	-5.42E-08	14219.	9.45E+09
-0.181	249663.	0.00				
39.0630	3.55E-06	-26.071	3.2490	-7.48E-08	14219.	9.45E+09
-0.166	252566.	0.00				
39.5120	3.11E-06	-10.819	2.4035	-8.53E-08	14219.	9.45E+09
-0.147	255469.	0.00				
39.9610	2.63E-06	0.1664	1.6665	-8.84E-08	14218.	9.45E+09
-0.126	258372.	0.00				
40.4100	2.16E-06	7.4881	1.0448	-8.62E-08	14219.	9.45E+09

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-0.105    261275.       0.00
   40.8590   1.70E-06    11.7666     0.5383  -8.07E-08     14219.   9.45E+09
-0.08349    264178.       0.00
   41.3080   1.29E-06    13.6080     0.1416  -7.35E-08     14219.   9.45E+09
-0.06375    267081.       0.00
   41.7570   9.11E-07    13.5834     -0.153  -6.57E-08     14219.   9.45E+09
-0.04565    269984.       0.00
   42.2060   5.78E-07    12.2183     -0.355  -5.84E-08     14219.   9.45E+09
-0.02926    272887.       0.00
   42.6550   2.82E-07     9.9898     -0.473  -5.20E-08     14219.   9.45E+09
-0.01444    275790.       0.00
   43.1040   1.71E-08     7.3309     -0.514  -4.71E-08     14219.   9.45E+09
-8.83E-04    278693.       0.00
   43.5530  -2.25E-07     4.6381     -0.485  -4.37E-08     14219.   9.45E+09
0.01178    281596.       0.00
   44.0020  -4.54E-07     2.2821     -0.388  -4.17E-08     14218.   9.45E+09
0.02395    284499.       0.00
   44.4510  -6.75E-07     0.6188     -0.227  -4.09E-08     14218.   9.45E+09
0.03600    287402.       0.00
   44.9000  -8.94E-07       0.00       0.00  -4.07E-08     14218.   9.45E+09
0.04818    145153.       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. 2:

```
Pile-head deflection              =     0.14000000 inches
Computed slope at pile head       =     -0.0100000 radians
Maximum bending moment            =       2392317. inch-lbs
Maximum shear force               =        -32452. lbs
Depth of maximum bending moment   =       0.000000 feet below pile head
Depth of maximum shear force      =     1.34700000 feet below pile head
Number of iterations              =              7
Number of zero deflection points =              5
```

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 3

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head   =   0.140000 inches

Moment at pile head = 1630834.0 in-lbs
Axial load at pile head = 367000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.1400	1630834.	-19367.	-0.00718	51006.	9.44E+09	0.00	0.00	0.00
0.4490	0.1038	1539767.	-19478.	-0.00628	48951.	9.44E+09	-41.329	2145.	0.00
0.8980	0.07235	1445762.	-19809.	-0.00543	46831.	9.45E+09	-81.478	6068.	0.00
1.3470	0.04533	1347763.	-20327.	-0.00463	44620.	9.45E+09	-110.568	13142.	0.00
1.7960	0.02246	1245033.	-20840.	-0.00389	42303.	9.45E+09	-79.857	19160.	0.00
2.2450	0.00341	1138581.	-21096.	-0.00321	39902.	9.45E+09	-15.141	23950.	0.00
2.6940	-0.01215	1030406.	-20962.	-0.00259	37462.	9.45E+09	64.7855	28740.	0.00
3.1430	-0.02453	922950.	-20376.	-0.00204	35038.	9.45E+09	152.6619	33530.	0.00
3.5920	-0.03408	818884.	-19413.	-0.00154	32690.	9.45E+09	204.7104	32363.	0.00
4.0410	-0.04112	719838.	-18491.	-0.00110	30456.	9.45E+09	137.5699	18028.	0.00
4.4900	-0.04594	623975.	-17691.	-7.17E-04	28294.	9.45E+09	159.4403	18701.	0.00
4.9390	-0.04884	532036.	-16773.	-3.87E-04	26220.	9.45E+09	181.2975	20000.	0.00
5.3880	-0.05011	444760.	-15741.	-1.09E-04	24251.	9.45E+09	201.6185	21677.	0.00
5.8370	-0.05002	362836.	-14583.	1.21E-04	22403.	9.45E+09	228.3270	24596.	0.00
6.2860	-0.04881	287132.	-13286.	3.07E-04	20695.	9.45E+09	253.1397	27946.	0.00
6.7350	-0.04671	218452.	-11902.	4.51E-04	19146.	9.45E+09	260.4968	30047.	0.00
7.1840	-0.04395	157089.	-10496.	5.58E-04	17762.	9.45E+09	261.4195	32050.	0.00
7.6330	-0.04070	103137.	-9099.	6.32E-04	16545.	9.45E+09	257.2362	34053.	0.00
8.0820	-0.03714	56537.	-7737.	6.78E-04	15494.	9.45E+09	248.5179	36056.	0.00
8.5310	-0.03340	17088.	-6432.	6.99E-04	14604.	9.45E+09			

235.9226	38059.	0.00				
8.9800	-0.02961	-15532.	-5203.	6.99E-04	14569.	9.45E+09
220.1578	40062.	0.00				
9.4290	-0.02587	-41743.	-4066.	6.83E-04	15160.	9.45E+09
201.9474	42065.	0.00				
9.8780	-0.02225	-62044.	-3031.	6.53E-04	15618.	9.45E+09
182.0034	44068.	0.00				
10.3270	-0.01883	-76991.	-2107.	6.13E-04	15955.	9.45E+09
161.0020	46071.	0.00				
10.7760	-0.01564	-87178.	-1298.	5.67E-04	16185.	9.45E+09
139.5658	48075.	0.00				
11.2250	-0.01272	-93215.	-602.997	5.15E-04	16321.	9.45E+09
118.2494	50078.	0.00				
11.6740	-0.01009	-95714.	-21.685	4.61E-04	16377.	9.45E+09
97.5309	52081.	0.00				
12.1230	-0.00775	-95273.	450.6749	4.07E-04	16368.	9.45E+09
77.8068	54084.	0.00				
12.5720	-0.00571	-92466.	820.2856	3.53E-04	16304.	9.45E+09
59.3910	56087.	0.00				
13.0210	-0.00394	-87831.	1095.	3.02E-04	16200.	9.45E+09
42.5170	58090.	0.00				
13.4700	-0.00245	-81863.	1283.	2.54E-04	16065.	9.45E+09
27.3428	60093.	0.00				
13.9190	-0.00121	-75008.	1394.	2.09E-04	15910.	9.45E+09
13.9581	62096.	0.00				
14.3680	-2.01E-04	-67664.	1436.	1.68E-04	15745.	9.45E+09
1.4565	39017.	0.00				
14.8170	6.01E-04	-60201.	1428.	1.32E-04	15576.	9.45E+09
-4.488	40236.	0.00				
15.2660	0.00122	-52800.	1390.	9.95E-05	15409.	9.45E+09
-9.372	41456.	0.00				
15.7150	0.00167	-45612.	1329.	7.14E-05	15247.	9.45E+09
-13.250	42675.	0.00				
16.1640	0.00199	-38757.	1250.	4.74E-05	15093.	9.45E+09
-16.193	43894.	0.00				
16.6130	0.00218	-32329.	1157.	2.71E-05	14948.	9.45E+09
-18.281	45113.	0.00				
17.0620	0.00228	-26395.	1055.	1.04E-05	14814.	9.45E+09
-19.604	46333.	0.00				
17.5110	0.00229	-21000.	947.7380	-3.16E-06	14692.	9.45E+09
-20.254	47552.	0.00				
17.9600	0.00225	-16169.	838.4125	-1.38E-05	14583.	9.45E+09
-20.327	48771.	0.00				
18.4090	0.00215	-11911.	729.9947	-2.18E-05	14487.	9.45E+09
-19.917	49991.	0.00				
18.8580	0.00201	-8217.	624.8442	-2.75E-05	14404.	9.45E+09
-19.114	51210.	0.00				
19.3070	0.00185	-5069.	524.8459	-3.13E-05	14333.	9.45E+09
-18.005	52429.	0.00				
19.7560	0.00167	-2437.	431.4407	-3.34E-05	14273.	9.45E+09

-16.667	53648.	0.00				
20.2050	0.00149	-287.136	345.6628	-3.42E-05	14225.	9.45E+09
-15.173	54868.	0.00				
20.6540	0.00131	1423.	268.1819	-3.39E-05	14251.	9.45E+09
-13.587	56087.	0.00				
21.1030	0.00112	2737.	199.3467	-3.27E-05	14280.	9.45E+09
-11.964	57306.	0.00				
21.5520	9.53E-04	3700.	139.2312	-3.09E-05	14302.	9.45E+09
-10.351	58526.	0.00				
22.0010	7.92E-04	4359.	87.6793	-2.86E-05	14317.	9.45E+09
-8.785	59745.	0.00				
22.4500	6.45E-04	4758.	44.3487	-2.60E-05	14326.	9.45E+09
-7.299	60964.	0.00				
22.8990	5.12E-04	4940.	8.7518	-2.32E-05	14330.	9.45E+09
-5.915	62183.	0.00				
23.3480	3.95E-04	4944.	-19.706	-2.04E-05	14330.	9.45E+09
-4.649	63403.	0.00				
23.7970	2.93E-04	4808.	-41.692	-1.76E-05	14327.	9.45E+09
-3.512	64622.	0.00				
24.2460	2.05E-04	4565.	-57.915	-1.49E-05	14321.	9.45E+09
-2.510	65841.	0.00				
24.6950	1.32E-04	4243.	-69.100	-1.24E-05	14314.	9.45E+09
-1.642	67061.	0.00				
25.1440	7.16E-05	3869.	-75.968	-1.01E-05	14306.	9.45E+09
-0.907	68280.	0.00				
25.5930	2.31E-05	3464.	-79.212	-8.01E-06	14297.	9.45E+09
-0.297	69499.	0.00				
26.0420	-1.48E-05	3047.	-79.490	-6.16E-06	14287.	9.45E+09
0.1943	70718.	0.00				
26.4910	-4.33E-05	2632.	-77.409	-4.54E-06	14278.	9.45E+09
0.5781	71938.	0.00				
26.9400	-6.37E-05	2231.	-73.522	-3.15E-06	14269.	9.45E+09
0.8650	73157.	0.00				
27.3890	-7.73E-05	1852.	-68.318	-1.99E-06	14260.	9.45E+09
1.0665	74376.	0.00				
27.8380	-8.51E-05	1503.	-62.227	-1.03E-06	14252.	9.45E+09
1.1944	75596.	0.00				
28.2870	-8.84E-05	1186.	-55.615	-2.64E-07	14245.	9.45E+09
1.2599	76815.	0.00				
28.7360	-8.80E-05	904.3225	-48.789	3.31E-07	14239.	9.45E+09
1.2742	78034.	0.00				
29.1850	-8.48E-05	658.7791	-41.996	7.77E-07	14233.	9.45E+09
1.2474	79253.	0.00				
29.6340	-7.96E-05	448.7044	-35.432	1.09E-06	14229.	9.45E+09
1.1889	80473.	0.00				
30.0830	-7.30E-05	272.6381	-29.247	1.30E-06	14225.	9.45E+09
1.1072	81692.	0.00				
30.5320	-6.56E-05	128.4063	-23.544	1.41E-06	14221.	9.45E+09
1.0096	82911.	0.00				
30.9810	-5.78E-05	13.3385	-18.393	1.45E-06	14219.	9.45E+09

0.9025	84131.	0.00					
31.4300	-4.99E-05	-75.545	-13.830	1.44E-06	14220.	9.45E+09	
0.7912	85350.	0.00					
31.8790	-4.23E-05	-141.374	-9.867	1.37E-06	14222.	9.45E+09	
0.6801	86569.	0.00					
32.3280	-3.51E-05	-187.301	-6.492	1.28E-06	14223.	9.45E+09	
0.5726	87788.	0.00					
32.7770	-2.85E-05	-216.394	-3.680	1.16E-06	14223.	9.45E+09	
0.4714	89008.	0.00					
33.2260	-2.26E-05	-231.559	-1.391	1.04E-06	14224.	9.45E+09	
0.3783	90227.	0.00					
33.6750	-1.74E-05	-235.481	0.4220	9.04E-07	14224.	9.45E+09	
0.2946	91446.	0.00					
34.1240	-1.28E-05	-230.587	1.8108	7.71E-07	14224.	9.45E+09	
0.2209	92665.	0.00					
34.5730	-9.05E-06	-219.018	2.8307	6.43E-07	14223.	9.45E+09	
0.1576	93885.	0.00					
35.0220	-5.92E-06	-202.626	3.5368	5.23E-07	14223.	9.45E+09	
0.1045	95104.	0.00					
35.4710	-3.41E-06	-182.973	3.9825	4.13E-07	14223.	9.45E+09	
0.06101	96323.	0.00					
35.9200	-1.47E-06	-161.342	4.2186	3.15E-07	14222.	9.45E+09	
0.02660	97543.	0.00					
36.3690	-2.20E-08	-138.758	4.2913	2.29E-07	14222.	9.45E+09	
4.04E-04	98762.	0.00					
36.8180	9.99E-07	-116.005	4.2425	1.56E-07	14221.	9.45E+09	
-0.01854	99981.	0.00					
37.2670	1.66E-06	-93.660	4.1083	9.67E-08	14221.	9.45E+09	
-0.03125	101200.	0.00					
37.7160	2.04E-06	-72.116	3.7753	4.94E-08	14220.	9.45E+09	
-0.09236	243857.	0.00					
38.1650	2.20E-06	-53.172	3.2556	1.37E-08	14220.	9.45E+09	
-0.101	246760.	0.00					
38.6140	2.19E-06	-37.088	2.7115	-1.21E-08	14219.	9.45E+09	
-0.101	249663.	0.00					
39.0630	2.07E-06	-23.905	2.1775	-2.95E-08	14219.	9.45E+09	
-0.09684	252566.	0.00					
39.5120	1.87E-06	-13.507	1.6776	-4.01E-08	14219.	9.45E+09	
-0.08869	255469.	0.00					
39.9610	1.63E-06	-5.668	1.2277	-4.56E-08	14219.	9.45E+09	
-0.07834	258372.	0.00					
40.4100	1.38E-06	-0.09719	0.8364	-4.72E-08	14218.	9.45E+09	
-0.06688	261275.	0.00					
40.8590	1.12E-06	3.5322	0.5077	-4.63E-08	14219.	9.45E+09	
-0.05514	264178.	0.00					
41.3080	8.81E-07	5.5569	0.2416	-4.37E-08	14219.	9.45E+09	
-0.04366	267081.	0.00					
41.7570	6.54E-07	6.3079	0.03564	-4.03E-08	14219.	9.45E+09	
-0.03277	269984.	0.00					
42.2060	4.47E-07	6.1003	-0.114	-3.67E-08	14219.	9.45E+09	

```
-0.02262    272887.       0.00
   42.6550   2.58E-07     5.2289     -0.210  -3.35E-08     14219.   9.45E+09
-0.01321    275790.       0.00
   43.1040   8.56E-08     3.9682     -0.258  -3.09E-08     14219.   9.45E+09
-0.00443    278693.       0.00
   43.5530  -7.48E-08     2.5745     -0.259  -2.90E-08     14218.   9.45E+09
0.00391    281596.       0.00
   44.0020  -2.27E-07     1.2913     -0.216  -2.79E-08     14218.   9.45E+09
0.01200    284499.       0.00
   44.4510  -3.76E-07     0.3550     -0.130  -2.75E-08     14218.   9.45E+09
0.02004    287402.       0.00
   44.9000  -5.23E-07       0.00       0.00  -2.74E-08     14218.   9.45E+09
0.02818    145153.       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. 3:

```
Pile-head deflection             =     0.14000000 inches
Computed slope at pile head      =     -0.0071825 radians
Maximum bending moment           =       1630834. inch-lbs
Maximum shear force              =        -21096. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =     2.24500000 feet below pile head
Number of iterations             =              6
Number of zero deflection points =              5
```

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 Load                Load                  Axial    Pile-head  Pile-head  Max
Shear Max Moment
Case Type   Pile-head    Type     Pile-head   Loading  Deflection  Rotation    in
```

```
Pile    in Pile
 No.  1      Load 1       2        Load 2       lbs      inches     radians      lbs
     in-lbs
---- ----- ---------- ---------- ---------- ---------- ---------- ----------
---------- ----------
  1  y, in     -0.390   S, rad         0.00    367000.     -0.390       0.00
-35987.   1664884.
  2  y, in     0.1400   S, rad     -0.01000    367000.     0.1400   -0.01000
-32452.   2392317.
  3  y, in     0.1400  M, in-lb    1630834.    367000.     0.1400   -0.00718
-21096.   1630834.


Maximum pile-head deflection = -0.3900000000 inches
Maximum pile-head rotation   = -0.0100000000 radians = -0.572958 deg.

The analysis ended normally.
```

CALCULATIONS

Date:	4/13/2026	**Made by:**	DEB
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

OBJECTIVE

Determine if the proposed HP14x89 piles will provide adequate support for Abutment No. 1 of the proposed Smith Brook Bridge (Bridge No. 3189) based on the anticipated thermal movement, rotation, scour conditions, and axial loads for the Strength I Load Combination.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Ed, 2024

2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

3. Hoyle Tanner, Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Bridge 3189. 60% Plans dated February 5. 2026.

4. WSP Interpretive Subsurface Profile, included as Sheet 3 of the Geotechnical Design Report (GDR).

5. FHWA. July 2016. GEC-012: Design and Construction of Driven Pile Foundations, Volume 1. Publication No. FHWA-NHI-16-009.

6. Email from Hoyle Tanner Structural team received on March 13, 2026 with subject line "RE: [External] NBB-Geotech-Ext: Bridge 3189 updated pile loads".

7. Hoyle Tanner, Abutment Loading Calculations, provided in "Integral Abutment Pile Loads_2025_07_24.pdf", received by WSP via email on July 24th, 2025.

8. WSP Laboratory Testing Results, included as Appendix C of the GDR.

9. WSP, Material properties table named "NBB Bridge 3189 Geotechnical Materials Properties.xlsx"

10. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2025. https://bsi.ce.ufl.edu/documentation/fbmp/manual

11. WSP USA Inc. Table 4: Summary of Rock Laboratory Testing Results, included in the GDR.

12. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

13. AISC Steel Construction Manual, 13th Ed. 2005.

14. WSP Boring Logs, included as Appendix A of the Geotechnical Design Report.

15. VTrans Structures Section, Integral Abutment Committee. 2008. Integral Abutment Bridge Design Guidelines, 2nd Ed.

16. FHWA, September 2018, GEC-010: Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024.

17. Hoyle, Tanner Northern Bridge Bundle-Bancroft TWP Bridge No. 3189 Scour Calculations - Proposed Conditions dated 08/2025.

18. Preliminary Design Report, Smith Brook Bridge # 3189 over Smith Brook Bancroft TWP, Maine WIN 28262.00.

CALCULATIONS

Date:	4/13/2026	**Made by:**	DEB
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on five HP14x89 Grade 50 piles with a center-to-center pile spacing of 7.715 feet, equal to the proposed bridge girder spacing adjusted for the abutment skew from Ref. 3, Sheet 27 and Ref. 7 Page 10. The piles are assumed to be driven to bedrock and thus end-bearing.

2. The selected pile orientation is weak axis bending (Ref. 2, page 5-42).

3. The vertical load is assumed to be evenly distributed.

4. The soil profile used in the analysis is based on Ref. 4 at the centerline of the proposed Abutment No. 1.

5. An UCS for rock of 7,000 psi is assumed based on a field characterization of R3 (Medium Strong Rock) to R4 (Strong Rock) from Ref. 14.

6. Based on previous experience on nearby projects and the bridge being located within a no salt zone, section loss due to corrosion is not considered for this analysis.

7. The piles are modeled in L-Pile as a fixed connection at the top of the pile.

8. Depth to fixity for deflection is taken as the depth along the pile to the point of zero lateral deflection; zero deflection is assumed to be deflection values less than 0.001 inches (Ref. 2 Section 5.4.2.5). Depth to fixity for bending moment is taken as the depth along the pile where the bending moment is less than 5% of the maximum bending moment (Ref. 16 page 12-19).

9. It is assumed the piles will have an unbraced length to the bottom depth of the scour pool located down stream of the proposed abutments at EL. 318.0 feet to account for scour at the bridge abutments (Ref. 17, sheet 17 of 40). The maximum calculated scour depths identified in the project PDR (Ref. 18) are 29.0' for Q100 and 33.6' for Q500 for the streambed, as determined from stream bed samples alone. These depths were determined to be impractical for evaluation of the potential scour depth, and therefore the depth of the scour hole measured during the project survey was instead used for this evaluation.

10. In the fully scoured condition, we expect that the piles would experience axial loading with possible moments and rotations due to thermal expansion and contraction. In this condition, it is likely that the soil behnd the abutment would be scoured away to some degree or entirely. However, the loading provided for the abutment/piles for the fully intact soil and embankment condition is used as an upper bound for the pile design in the scoured condition, where only the piles have an unsupported length.

CALCULATIONS

Date:	4/13/2026	**Made by:**	DEB
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

ATTACHMENTS

1. LPile analysis output for Strength I

CALCULATION

1. Select the pile section parameters

	Intact Section (Ref. 12)
Pile depth, d =	13.8
Pile width, b_f =	14.7
Web thickness, t_w =	0.615
Flange thickness, t_f =	0.615
Fillet area =	0.3
Section area =	26.1
Moment of inertia about the x-axis, I_x =	888
Moment of inertia about the y-axis, I_y =	326
Radius of gyration about the x-axis, r_x =	5.83
Radius of gyration about the y-axis, r_y =	3.53
Elastic section modulus about the x-axis, S_x =	129
Elastic section modulus about the y-axis, S_y =	44.3
Plastic section modulus about the x-axis, Z_x =	143
Plastic section modulus about the y-axis, Z_y =	67.6

2. Determine pile loads and select the preliminary pile size.

Determine the factored substructure vertical dead and live load (P_u) distributed to each pile.

Strength I factored vertical load per pile = 363.0 kips (Ref. 6)

Pile weight = 0.089 kip/ft x 44.9 ft = 4.0 kips

P_u = 367 kips (Total factored axial load including pile weight.)

Select the steel pile strength.

F_y = 50 ksi (Assumption 1)
E = 29,000 ksi (typical value for steel)

CALCULATIONS

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Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} =	0.50	for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)
ϕ_{cu} =	0.70	for axial resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)
ϕ_f =	1.00	for flexural resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 524 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 734 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 734 kips

Use the required nominal axial pile resistance to estimate an approximate initial pile area.

$$A_{s.req} = \frac{R_n}{0.80\ Fy}$$ (Ref. 2, section 5-42)

$A_{s.req}$ = 18.3 in^2

Select a pile size with an area of approximately $A_{s.req}$.

HP14x89 section as per Assumption 1 and Step 1:

A_s = 26.1 in^2 (Step 1)

CALCULATIONS

Date:	4/13/2026	**Made by:**	DEB
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

3. Use LPile analysis to determine the pile unbraced length and maximum moment at the top of the pile.

The following input parameters were used in the LPile analysis:

Pile Properties

Section type:	Steel H Section Weak Axis		(Assumption 2)	
Length of section:	44.9	ft	(piles driven to bedrock)	
Flange width, b_f:	14.7	in	(Step 1)	
Section depth, d:	13.8	in	(Step 1)	
Flange thickness, t_f:	0.615	in	(Step 1)	
Web thickness, t_w:	0.615	in	(Step 1)	
Moment of inertia, I_y:	326	in^4	(Step 1)	about the weak axis, since weak-axis bending
Plastic section modulus, Z_y:	67.6	in^3	(Step 1)	about the weak axis, since weak-axis bending
Elastic section modulus, S_y:	44.3	in^3	(Step 1)	about the weak axis, since weak-axis bending
Radius of gyration, r_y:	3.53	in	(Step 1)	about the weak axis, since weak-axis bending
Pile batter:	Vertical		(pile battering not required)	

Pile Loading

Case	ΔT (in) [1]	Rotation (rad)[2]	Axial Load (lbs)[3]
1 (Thermal contraction)	-0.39	0.000	367,000
2 (Thermal expansion)	0.14	-0.010	367,000

1) Strength I Lateral deflection due to abutment thermal expansion or contraction per Ref. 6
2) Assumed Rotations per Ref. 6.
3). From Step 2

CALCULATIONS

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Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1 with Scour	**Reviewed by:**	MEL
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Soil Layers

Layer	**Depth below base of abutment[1]**	**Lateral Model**	**Effective Unit Weight (pcf)[2]**	**Undrained Shear Strength (psf)**	**Friction Angle (°)[2]**	**Subgrade Modulus (pci)[3]**	**Major Principal Strain at 50%[3]**	**UCS (psi)[4]**
Unsupported length - Scour[5]	0.0 - 22.2 ft	Sand (Reese)	-	-	-	-	-	-
Alluvial Silt	22.2 - 37.4 ft	Sand (Reese)	52.6	-	31	42	-	-
Glacial Till	37.4 - 44.9 ft	Sand (Reese)	67.6	-	36	100.0	-	-
Bedrock	> 44.9 ft	Strong Rock (Vuggy Limestone)	102.6	-	-	-	-	7,000

1) Per Ref. 3, 4, 14, and 17.
2) Ref. 9
3) Ref. 10 - interpolation based on friction angle for subgrade modulus
4) Assumption 5
5) Assumption 9 & 10.

The full LPile output is provided in Attachment 1.

Obtain the maximum moment in the top segment of the pile using the loading case that produces the highest moment.

The thermal expansion load case (Load case 2), resulted in the highest moment in the pile.

$M_{u.Top} =$ 781 in-kips (from LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top} =$ 20.9 ft (from LPile)
$l_{b.top} =$ 250.7 in

$l_{b.2nd} =$ 16.8 ft (from LPile)
$l_{b.2nd} =$ 201.4 in

CALCULATIONS

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4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the interaction of combined axial and flexural load effects on a single pile.

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each segment.

For the top segment (rotation fixed with no plastic hinge):

$$\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{top} = 1.2 (Ref. 1, Table C4.6.2.5-1)

r_y = 3.53 in (Step 1)

λ_{top} = 85.2 OK

For the second segment (pinned at top and bottom):

$$\lambda_{2nd} = \frac{K_{2nd} l_{b.2nd}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{2nd} = 57.0 OK

Calculate the critical elastic buckling resistance, P_e, and the nominal yield resistance, P_o.

Use Ref. 1 Table 6.9.4.1.1-1 to select equation for P_e based on cross-section shape and potential buckling mode.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\, l_b}{r_y}\right)^2} A_s$$ (Ref. 1, Eqn 6.9.4.1.2-1)

$P_{e.top}$ = 1029 kips

$P_{e.2nd}$ = 2295 kips

$$P_o = F_y A_s$$ (Ref. 1, Article 6.9.4.1)

P_o = 1305 kips

CALCULATIONS

Date:	4/13/2026	**Made by:**	DEB
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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Calculate the nominal structural pile resistance, P_n, for both segments of the upper zone of the pile as well as the lower zone of the pile.

Determine P_o/P_e to select equation for P_n as per Ref. 1 Article 6.9.4.1.

$P_o/P_{e.top}$ = 1.27 ≤ 2.25

$P_o/P_{e.2nd}$ = 0.57 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 767 kips

$P_{n.2nd}$ = 1029 kips

$$P_{n.bottom} = \left(0.658^{(0)}\right) P_o$$

(0 for a fully braced pile - Ref. 15, Appendix B, Eqn 6-9)

$P_{n.bottom}$ = 1305 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

$$P_{r.top} = \phi_{cu} P_{n.top}$$

(Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.top}$ = 537 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$

(Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.2nd}$ = 720 kips

$$P_{r.bottom} = \phi_{cl} P_{n.bottom}$$

(Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.bottom}$ = 653 kips

Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20 (Ref. 2, page 5-43).

$$\frac{P_u}{P_{r.top}} = 0.68 \quad \text{OK}$$

$$\frac{P_u}{P_{r.2nd}} = 0.51 \quad \text{OK}$$

CALCULATIONS

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Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads (Ref. 15, Appendix B Eqn 6-17). Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check}\left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$$\frac{P_u}{P_{r.bottom}} = \quad 0.56 \qquad \text{OK}$$

Determine the nominal and factored flexural resistance about the H-pile weak axis.

Slenderness ratio for the flange:

$$\lambda_f = \frac{b_f}{2t_f} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1c-3)}$$

$\lambda_f =$ 12.0

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1c-4)}$$

$\lambda_{pf} =$ 9.2

Limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1c-5)}$$

$\lambda r_f =$ 24.1

Since λpf < λf ≤ λrf, pile is noncompact per Ref. 1, Article 6.12.2.2.2c-12.

Elastic and plastic section moduli about the weak axis:

$S_y =$ 44.3 in^3 (Step 1)

$Z_y =$ 67.6 in^3 (Step 1)

CALCULATIONS

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Per Ref.1 section 6.12.2.2.1 the nominal flexural resistance about the weak axis shall be taken as the smaller value based on yielding or FLB (Flange Local Buckling).

Nominal flexural resistance based on yielding:

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad \text{, if } \lambda_f \leq \lambda_{pf}$$ (Ref. 1, Eqn 6.12.2.2.1b-1)

Nominal flexural resistance based on FLB:
Since λpf < λf ≤ λrf,

$$M_n = M_p - \left(M_p - 0.7\, F_{yf}\, S_y\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \text{ if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-1)

$$M_n = \frac{0.69 E S_y}{\lambda_f^2} \qquad \text{if } \lambda_f > \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-2)

For welded sections Fyf shall be taken as 0.5Fyc, where Fyc is the minimum yield stress of the compression flange.

$M_p =$ 3380 in-kips
$M_n =$ 2892 in-kips

Factored flexural resistance:

$\phi_f =$ 1.00 (Ref. 2, page 5-42)

$$M_r = \phi_f M_n$$

$M_r =$ 2892 in-kips

For all members (including noncompact):

$$M'_p = \left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$ (Ref. 1, Eqn 6.9.2.2.1-3 and Ref. 15 Eq 4.5.2-3)

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'.

$M_p' =$ 916 in-kips = 916296 inch-lb

If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation, and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top} =$ 781 in-kips (From Step 3)
$M_{u.Top}$ < M_p' No Plastic Hinge Forms

CALCULATIONS

Date:	4/13/2026	**Made by:**	DEB
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Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Check the second segment of the upper zone of the pile to ensure that this segment of the pile does not form a plastic hinge.

$M_{u.2nd}$ = 266 in-kips (LPile)

Check: $$\frac{P_u}{P_{r.2nd}} + \left(\frac{M_{u.2nd}}{M_r}\right) < 1$$ (Ref. 1, Eqn 6.9.2.2.1-3 for noncompact member, Ref. 15, Eqn 4.5.2-3)

Check: 0.60 < 1 OK

5. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPile output against the structural shear resistance per AISC G7.

V_u = 3.7 kips (from LPile)

AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition, Specifications G2 and G7 (Ref. 13) to ensure the pile will not shear under the longitudinal load.

Per Ref. 13 Section G7: for singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength V_n for each shear resisting element shall be determined using Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ and $k_v = 1.2$.

$$V_n = 0.6 F_y A_w C_v$$ (Ref. 13, Eqn G2-1)

where:

C_v = 1.0 for HP shapes with $F_y \le 50$ ksi (Ref. 13, Section G7)

$A_w = b_f t_f$ = 9.0 in^2 (Ref. 13, Section G7)

Since there are two shear resisting elements (i.e., two flanges):

$2 \times A_w$ = 18.1 in^2

V_n = 542 kips

$$V_r = \Phi_v V_n$$

ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)

V_r = 542 kips

Check that the shear resistance is sufficient:

V_u < V_r OK

CALCULATIONS

Date:	4/13/2026	**Made by:**	DEB
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Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

6. Check that the maximum factored applied pile load does not exceed the factored pile drivability resistance.

For steel piles in compression, driving stresses should not exceed 90% of the yield strength of the pile material (Ref. 2, page 5-104).

$\sigma_{dr} =$ 0.9 × 50 ksi = 45 ksi

This translates into an ultimate maximum driving force that can be applied to the pile of:

$$P_{dr} = \sigma_{dr} A_s$$ (Ref. 15, Appendix B, Eqn 7-23)

$P_{dr} =$ 1175 kips

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. In this design, the pile will be bearing on rock. The driving criteria will be established by dynamic testing.

$\phi_{dyn} =$ 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{dyn}}$$ (Ref. 2, Appendix B, Eqn 7-25)

$R_{ndr} =$ 565 kips

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_{dr}) calculated above.

$P_{n.top} =$ 767 kips (From Step 4)

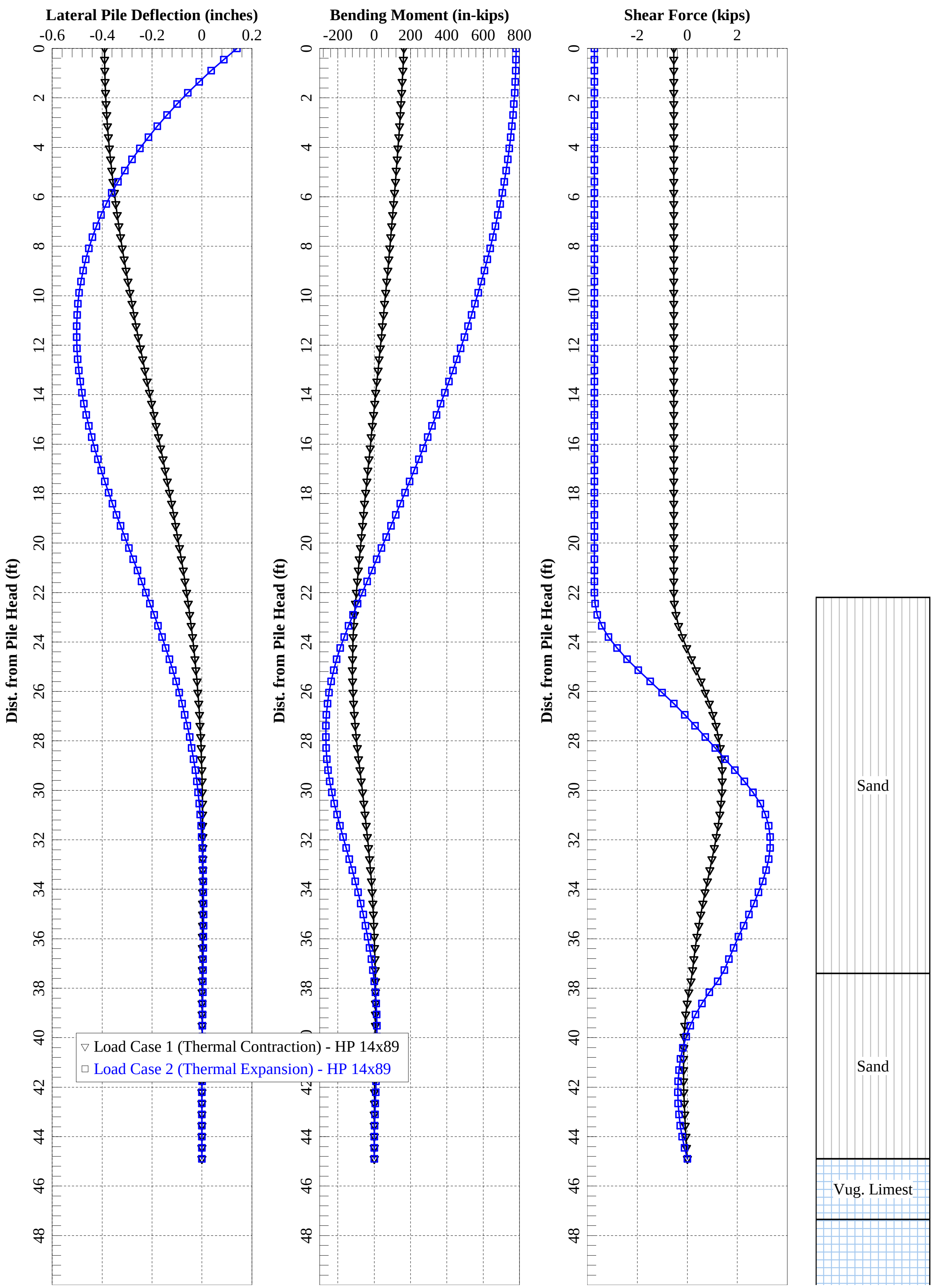
$P_{n.2nd} =$ 1029 kips (From Step 4)

Check $R_{ndr} < P_n$: OK

Check $R_{ndr} < P_{dr}$: OK

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 781 in-kips (65 ft-kips) occurs at the top of the pile under the Strength I load case with a maximum bridge expansion of -0.14 inches and a rotation of 0.010 radians. The results indicate when evaluating an assumed scour depth of 22.2 feet below the bottom of footing for a pile unsupported length that the depth to bedrock is sufficient for driven piles to achieve fixity. Based on the intact section used in the analysis, HP14x89 piles are adequately sized to accommodate the anticipated design loads, thermal movement, and scour conditions at Abutment No.1 of the Smith Brook Bridge (Bridge No. 3189).



Lateral Pile Deflection (inches)
-0.6
-0.4
-0.2
0
0.2
Bending Moment (in-kips)
-200
0
200
400
600
800
Shear Force (kips)
-2
0
2
Dist. from Pile Head (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
▽ Load Case 1 (Thermal Contraction) - HP 14x89
□ Load Case 2 (Thermal Expansion) - HP 14x89
Sand
Sand
Vug. Limest

==

LPile for Windows, Version 2022-12.007

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

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Files Used for Analysis

Path to file locations:
\Users\USDB712838\Downloads\

Name of input data file:
Abutment 1 HP14x89_Scour_4.13.26.lp12d

Name of output report file:
Abutment 1 HP14x89_Scour_4.13.26.lp12o

Name of plot output file:
Abutment 1 HP14x89_Scour_4.13.26.lp12p

Name of runtime message file:
Abutment 1 HP14x89_Scour_4.13.26.lp12r

Date and Time of Analysis

Date: April 14, 2026 Time: 8:41:03

Problem Title

Project Name: WIN 028262.00 Northern Bridge Bundle - Bridge No. 3189 -

Job Number: US0040947.9162

Client: MaineDOT

Engineer: WSP USA Inc.

Description: HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 1

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                                         =      44.900 ft
Depth of ground surface below top of pile                    =     22.2000 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.

            Depth Below          Pile
Point        Pile Head         Diameter
 No.           feet             inches
-----      -------------     -------------
  1            0.000           14.7000
  2           44.900           14.7000


Input Structural Properties for Pile Sections:
----------------------------------------------

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                         =    44.900000 ft
```

```
   Pile width                                          =    13.800000 in



--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 3 layers

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

   Distance from top of pile to top of layer           =    22.200000 ft
   Distance from top of pile to bottom of layer        =    37.400000 ft
   Effective unit weight at top of layer               =    52.600000 pcf
   Effective unit weight at bottom of layer            =    52.600000 pcf
   Friction angle at top of layer                      =    31.000000 deg.
   Friction angle at bottom of layer                   =    31.000000 deg.
   Subgrade k at top of layer                          =    42.000000 pci
   Subgrade k at bottom of layer                       =    42.000000 pci


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

   Distance from top of pile to top of layer           =    37.400000 ft
   Distance from top of pile to bottom of layer        =    44.900000 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                      =    36.000000 deg.
   Friction angle at bottom of layer                   =    36.000000 deg.
   Subgrade k at top of layer                          =   100.000000 pci
   Subgrade k at bottom of layer                       =   100.000000 pci


Layer 3 is strong rock (vuggy limestone)

   Distance from top of pile to top of layer           =    44.900000 ft
   Distance from top of pile to bottom of layer        =    55.000000 ft
   Effective unit weight at top of layer               =   102.600000 pcf
   Effective unit weight at bottom of layer            =   102.600000 pcf
   Uniaxial compressive strength at top of layer       =        7000. psi
   Uniaxial compressive strength at bottom of layer    =        7000. psi


 (Depth of the lowest soil layer extends 10.100 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	Angle of Friction deg.	Uniaxial qu psi	kpy pci
1	Sand	22.2000	52.6000	31.0000	--	42.0000
	(Reese, et al.)	37.4000	52.6000	31.0000	--	42.0000
2	Sand	37.4000	67.6000	36.0000	--	100.0000
	(Reese, et al.)	44.9000	67.6000	36.0000	--	100.0000
3	Strong Rock	44.9000	102.6000	--	7000.	--
	(Vuggy Limestone)	55.0000	102.6000	--	7000.	--

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 = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.39000 in	S = 0.0000 in/in	367000.	N.A.	Yes
2	5	y = 0.140000 in	S = -0.01000 in/in	367000.	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                                      =    44.900000 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	367.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 367.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000442	41.7773444	9448389.	118.2345231	15.1514881	
0.00000884	83.5546887	9448389.	62.7922615	16.0845352	
0.00001326	125.3320331	9448389.	44.3115077	17.0175823	
0.00001769	167.1093775	9448389.	35.0711308	17.9506295	
0.00002211	208.8867218	9448389.	29.5269046	18.8836766	
0.00002653	250.6640662	9448389.	25.8307538	19.8167238	
0.00003095	292.4414106	9448389.	23.1906462	20.7497709	
0.00003537	334.2187549	9448389.	21.2105654	21.6828180	
0.00003979	375.9960993	9448389.	19.6705026	22.6158653	
0.00004422	417.7734437	9448389.	18.4384523	23.5489123	
0.00004864	459.5507880	9448389.	17.4304112	24.4819595	
0.00005306	501.3281324	9448389.	16.5903769	25.4150066	
0.00005748	543.1054768	9448389.	15.8795787	26.3480537	
0.00006190	584.8828211	9448389.	15.2703231	27.2811009	
0.00006632	626.6601655	9448389.	14.7423015	28.2141480	
0.00007075	668.4375098	9448389.	14.2802827	29.1471952	
0.00007517	710.2148542	9448389.	13.8726190	30.0802423	
0.00007959	751.9921986	9448389.	13.5102513	31.0132894	
0.00008401	793.7695429	9448389.	13.1860275	31.9463366	
0.00008843	835.5468873	9448389.	12.8942262	32.8793838	
0.00009285	877.3242317	9448389.	12.6302154	33.8124309	
0.00009728	919.1015760	9448389.	12.3902056	34.7454780	
0.0001017	960.8789204	9448389.	12.1710662	35.6785251	
0.0001061	1003.	9448389.	11.9701885	36.6115723	
0.0001105	1044.	9448389.	11.7853809	37.5446194	
0.0001150	1086.	9448389.	11.6147893	38.4776666	
0.0001194	1128.	9448389.	11.4568342	39.4107137	
0.0001238	1170.	9448389.	11.3101615	40.3437608	
0.0001282	1212.	9448389.	11.1736042	41.2768080	
0.0001326	1253.	9448389.	11.0461508	42.2098551	
0.0001371	1295.	9448389.	10.9269201	43.1429023	
0.0001415	1337.	9448389.	10.8151413	44.0759494	
0.0001459	1379.	9448389.	10.7101371	45.0089965	
0.0001503	1420.	9448389.	10.6113095	45.9420437	
0.0001548	1462.	9448389.	10.5181292	46.8750908	
0.0001592	1504.	9448389.	10.4301256	47.8081380	

0.0001636	1546.	9448389.	10.3468790	48.7411851	
0.0001680	1588.	9448389.	10.2680138	49.6742323	
0.0001724	1629.	9443723.	10.1940496	50.0000000	Y
0.0001813	1706.	9409764.	10.0617064	50.0000000	Y
0.0001901	1778.	9351217.	9.9471160	50.0000000	Y
0.0001990	1845.	9275072.	9.8474020	50.0000000	Y
0.0002078	1909.	9186226.	9.7603030	50.0000000	Y
0.0002167	1969.	9088653.	9.6839052	50.0000000	Y
0.0002255	2026.	8985149.	9.6166785	50.0000000	Y
0.0002343	2080.	8877224.	9.5575126	50.0000000	Y
0.0002432	2132.	8767936.	9.5049842	50.0000000	Y
0.0002520	2182.	8657023.	9.4585881	50.0000000	Y
0.0002609	2230.	8546441.	9.4172999	50.0000000	Y
0.0002697	2276.	8436902.	9.3804708	50.0000000	Y
0.0002786	2320.	8328745.	9.3476031	50.0000000	Y
0.0002874	2363.	8221937.	9.3183563	50.0000000	Y
0.0002962	2405.	8117206.	9.2921946	50.0000000	Y
0.0003051	2445.	8014781.	9.2687565	50.0000000	Y
0.0003139	2485.	7914802.	9.2477355	50.0000000	Y
0.0003228	2523.	7815381.	9.2284423	50.0000000	Y
0.0003316	2558.	7712825.	9.2096343	50.0000000	Y
0.0003405	2590.	7608368.	9.1911858	50.0000000	Y
0.0003493	2621.	7502911.	9.1730365	50.0000000	Y
0.0003582	2649.	7397217.	9.1551297	50.0000000	Y
0.0003670	2676.	7291027.	9.1378246	50.0000000	Y
0.0003758	2701.	7185666.	9.1207678	50.0000000	Y
0.0003847	2724.	7081536.	9.1039508	50.0000000	Y
0.0003935	2746.	6977697.	9.0876016	50.0000000	Y
0.0004024	2767.	6875589.	9.0712177	50.0000000	Y
0.0004112	2786.	6774695.	9.0554922	50.0000000	Y
0.0004201	2804.	6676230.	9.0399111	50.0000000	Y
0.0004289	2822.	6578534.	9.0246785	50.0000000	Y
0.0004377	2838.	6483272.	9.0094336	50.0000000	Y
0.0004466	2854.	6389730.	8.9950233	50.0000000	Y
0.0004554	2868.	6297812.	8.9803119	50.0000000	Y
0.0004643	2882.	6208019.	8.9661427	50.0000000	Y
0.0004731	2895.	6120073.	8.9522270	50.0000000	Y
0.0004820	2908.	6034149.	8.9382517	50.0000000	Y
0.0004908	2920.	5950014.	8.9249942	50.0000000	Y
0.0004996	2932.	5867483.	8.9116029	50.0000000	Y
0.0005085	2943.	5787469.	8.8985011	50.0000000	Y
0.0005173	2953.	5708419.	8.8856728	50.0000000	Y
0.0005262	2963.	5631656.	8.8731656	50.0000000	Y
0.0005615	2999.	5340961.	8.8245281	50.0000000	Y
0.0005969	3030.	5075908.	8.7791277	50.0000000	Y
0.0006323	3056.	4833303.	8.7361931	50.0000000	Y
0.0006677	3079.	4611513.	8.6958050	50.0000000	Y
0.0007030	3099.	4407785.	8.6579820	50.0000000	Y
0.0007384	3116.	4220367.	8.6220712	50.0000000	Y
0.0007738	3132.	4047417.	8.5879985	50.0000000	Y

0.0008092	3146.	3887458.	8.5556478	50.0000000	Y
0.0008445	3158.	3739153.	8.5250335	50.0000000	Y
0.0008799	3169.	3601594.	8.4957228	50.0000000	Y
0.0009153	3179.	3473493.	8.4683918	50.0000000	Y
0.0009507	3188.	3353733.	8.4419138	50.0000000	Y
0.0009860	3197.	3241904.	8.4165778	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	367.0000000000	3197.

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	22.2000	0.00	N.A.	No	0.00	133678.
2	37.4000	13.1367	Yes	No	133678.	373138.
3	44.9000	22.7000	No	Yes	N.A.	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 = -0.390000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 367000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.390	163939.	-534.835	0.00	17916.	9.45E+09	0.00	0.00	0.00
0.4490	-0.390	160966.	-534.524	9.26E-05	17849.	9.45E+09	0.00	0.00	0.00
0.8980	-0.389	157812.	-534.524	1.84E-04	17778.	9.45E+09	0.00	0.00	0.00
1.3470	-0.388	154480.	-534.524	2.73E-04	17703.	9.45E+09	0.00	0.00	0.00
1.7960	-0.386	150974.	-534.524	3.60E-04	17624.	9.45E+09	0.00	0.00	0.00
2.2450	-0.384	147298.	-534.524	4.45E-04	17541.	9.45E+09	0.00	0.00	0.00
2.6940	-0.381	143455.	-534.524	5.28E-04	17454.	9.45E+09	0.00	0.00	0.00
3.1430	-0.378	139451.	-534.524	6.08E-04	17364.	9.45E+09	0.00	0.00	0.00
3.5920	-0.375	135290.	-534.524	6.87E-04	17270.	9.45E+09	0.00	0.00	0.00
4.0410	-0.371	130976.	-534.524	7.63E-04	17173.	9.45E+09	0.00	0.00	0.00
4.4900	-0.367	126514.	-534.524	8.36E-04	17072.	9.45E+09	0.00	0.00	0.00
4.9390	-0.362	121910.	-534.524	9.07E-04	16968.	9.45E+09	0.00	0.00	0.00
5.3880	-0.357	117168.	-534.524	9.75E-04	16861.	9.45E+09	0.00	0.00	0.00

5.8370	-0.351	112294.	-534.524	0.00104	16751.	9.45E+09	0.00	0.00	0.00
6.2860	-0.346	107293.	-534.524	0.00110	16639.	9.45E+09	0.00	0.00	0.00
6.7350	-0.339	102172.	-534.524	0.00116	16523.	9.45E+09	0.00	0.00	0.00
7.1840	-0.333	96935.	-534.524	0.00122	16405.	9.45E+09	0.00	0.00	0.00
7.6330	-0.326	91589.	-534.524	0.00127	16284.	9.45E+09	0.00	0.00	0.00
8.0820	-0.319	86140.	-534.524	0.00132	16162.	9.45E+09	0.00	0.00	0.00
8.5310	-0.312	80593.	-534.524	0.00137	16036.	9.45E+09	0.00	0.00	0.00
8.9800	-0.304	74956.	-534.524	0.00142	15909.	9.45E+09	0.00	0.00	0.00
9.4290	-0.297	69234.	-534.524	0.00146	15780.	9.45E+09	0.00	0.00	0.00
9.8780	-0.289	63434.	-534.524	0.00149	15649.	9.45E+09	0.00	0.00	0.00
10.3270	-0.281	57562.	-534.524	0.00153	15517.	9.45E+09	0.00	0.00	0.00
10.7760	-0.272	51626.	-534.524	0.00156	15383.	9.45E+09	0.00	0.00	0.00
11.2250	-0.264	45631.	-534.524	0.00159	15248.	9.45E+09	0.00	0.00	0.00
11.6740	-0.255	39585.	-534.524	0.00161	15111.	9.45E+09	0.00	0.00	0.00
12.1230	-0.246	33495.	-534.524	0.00163	14974.	9.45E+09	0.00	0.00	0.00
12.5720	-0.238	27366.	-534.524	0.00165	14836.	9.45E+09	0.00	0.00	0.00
13.0210	-0.229	21207.	-534.524	0.00166	14697.	9.45E+09	0.00	0.00	0.00
13.4700	-0.220	15024.	-534.524	0.00167	14557.	9.45E+09	0.00	0.00	0.00
13.9190	-0.211	8823.	-534.524	0.00168	14417.	9.45E+09	0.00	0.00	0.00
14.3680	-0.202	2613.	-534.524	0.00168	14277.	9.45E+09	0.00	0.00	0.00
14.8170	-0.192	-3600.	-534.524	0.00168	14300.	9.45E+09	0.00	0.00	0.00
15.2660	-0.183	-9809.	-534.524	0.00168	14440.	9.45E+09	0.00	0.00	0.00
15.7150	-0.174	-16007.	-534.524	0.00167	14580.	9.45E+09	0.00	0.00	0.00
16.1640	-0.165	-22187.	-534.524	0.00166	14719.	9.45E+09	0.00	0.00	0.00
16.6130	-0.156	-28342.	-534.524	0.00165	14858.	9.45E+09	0.00	0.00	0.00

17.0620	-0.148	-34465.	-534.524	0.00163	14996.	9.45E+09	0.00	0.00	0.00
17.5110	-0.139	-40549.	-534.524	0.00161	15133.	9.45E+09	0.00	0.00	0.00
17.9600	-0.130	-46587.	-534.524	0.00158	15269.	9.45E+09	0.00	0.00	0.00
18.4090	-0.122	-52573.	-534.524	0.00156	15404.	9.45E+09	0.00	0.00	0.00
18.8580	-0.113	-58499.	-534.524	0.00152	15538.	9.45E+09	0.00	0.00	0.00
19.3070	-0.105	-64360.	-534.524	0.00149	15670.	9.45E+09	0.00	0.00	0.00
19.7560	-0.09745	-70148.	-534.524	0.00145	15801.	9.45E+09	0.00	0.00	0.00
20.2050	-0.08974	-75857.	-534.524	0.00141	15930.	9.45E+09	0.00	0.00	0.00
20.6540	-0.08227	-81480.	-534.524	0.00136	16056.	9.45E+09	0.00	0.00	0.00
21.1030	-0.07504	-87012.	-534.524	0.00132	16181.	9.45E+09	0.00	0.00	0.00
21.5520	-0.06808	-92445.	-534.524	0.00126	16304.	9.45E+09	0.00	0.00	0.00
22.0010	-0.06141	-97774.	-534.524	0.00121	16424.	9.45E+09	0.00	0.00	0.00
22.4500	-0.05504	-102993.	-518.753	0.00115	16542.	9.45E+09	5.8543	573.1033	0.00
22.8990	-0.04898	-107926.	-457.340	0.00109	16653.	9.45E+09	16.9419	1864.	0.00
23.3480	-0.04326	-112245.	-344.272	0.00103	16750.	9.45E+09	25.0283	3117.	0.00
23.7970	-0.03788	-115711.	-194.714	9.66E-04	16829.	9.45E+09	30.4870	4337.	0.00
24.2460	-0.03285	-118162.	-21.316	8.99E-04	16884.	9.45E+09	33.8774	5556.	0.00
24.6950	-0.02819	-119496.	165.4526	8.31E-04	16914.	9.45E+09	35.4503	6775.	0.00
25.1440	-0.02390	-119665.	356.4803	7.63E-04	16918.	9.45E+09	35.4583	7995.	0.00
25.5930	-0.01997	-118671.	544.0089	6.95E-04	16895.	9.45E+09	34.1515	9214.	0.00
26.0420	-0.01641	-116552.	721.6110	6.28E-04	16848.	9.45E+09	31.7736	10433.	0.00
26.4910	-0.01321	-113378.	884.1444	5.62E-04	16776.	9.45E+09	28.5581	11652.	0.00
26.9400	-0.01035	-109248.	1028.	4.99E-04	16683.	9.45E+09	24.7247	12872.	0.00
27.3890	-0.00783	-104276.	1149.	4.38E-04	16571.	9.45E+09	20.4769	14091.	0.00
27.8380	-0.00563	-98593.	1248.	3.80E-04	16442.	9.45E+09	15.9991	15310.	0.00

28.2870	-0.00373	-92334.	1322.	3.26E-04	16301.	9.45E+09	11.4552	16530.	0.00
28.7360	-0.00212	-85638.	1371.	2.75E-04	16150.	9.45E+09	6.9875	17749.	0.00
29.1850	-7.72E-04	-78643.	1398.	2.28E-04	15992.	9.45E+09	2.7162	18968.	0.00
29.6340	3.36E-04	-71481.	1401.	1.85E-04	15831.	9.45E+09	-1.261	20187.	0.00
30.0830	0.00122	-64274.	1385.	1.47E-04	15668.	9.45E+09	-4.866	21407.	0.00
30.5320	0.00192	-57136.	1350.	1.12E-04	15507.	9.45E+09	-8.045	22626.	0.00
30.9810	0.00243	-50168.	1299.	8.13E-05	15350.	9.45E+09	-10.759	23845.	0.00
31.4300	0.00279	-43455.	1236.	5.46E-05	15199.	9.45E+09	-12.989	25065.	0.00
31.8790	0.00302	-37070.	1161.	3.17E-05	15055.	9.45E+09	-14.732	26284.	0.00
32.3280	0.00313	-31071.	1078.	1.23E-05	14919.	9.45E+09	-15.996	27503.	0.00
32.7770	0.00315	-25501.	989.6922	-3.87E-06	14794.	9.45E+09	-16.803	28722.	0.00
33.2260	0.00309	-20391.	898.1360	-1.70E-05	14678.	9.45E+09	-17.182	29942.	0.00
33.6750	0.00297	-15756.	805.5837	-2.73E-05	14574.	9.45E+09	-17.173	31161.	0.00
34.1240	0.00280	-11602.	714.0180	-3.51E-05	14480.	9.45E+09	-16.816	32380.	0.00
34.5730	0.00259	-7923.	625.1797	-4.06E-05	14397.	9.45E+09	-16.160	33600.	0.00
35.0220	0.00236	-4704.	540.5523	-4.42E-05	14325.	9.45E+09	-15.253	34819.	0.00
35.4710	0.00211	-1923.	461.3538	-4.61E-05	14262.	9.45E+09	-14.145	36038.	0.00
35.9200	0.00186	449.7541	388.5358	-4.65E-05	14229.	9.45E+09	-12.885	37257.	0.00
36.3690	0.00161	2448.	322.7879	-4.57E-05	14274.	9.45E+09	-11.521	38477.	0.00
36.8180	0.00137	4109.	264.5457	-4.38E-05	14311.	9.45E+09	-10.099	39696.	0.00
37.2670	0.00114	5472.	214.0023	-4.11E-05	14342.	9.45E+09	-8.663	40915.	0.00
37.7160	9.28E-04	6578.	144.1328	-3.77E-05	14367.	9.45E+09	-17.272	100320.	0.00
38.1650	7.35E-04	7174.	59.6786	-3.38E-05	14380.	9.45E+09	-14.077	103223.	0.00
38.6140	5.64E-04	7354.	-8.166	-2.96E-05	14384.	9.45E+09	-11.107	106126.	0.00
39.0630	4.16E-04	7203.	-60.747	-2.55E-05	14381.	9.45E+09	-8.411	109029.	0.00

39.5120	2.90E-04	6800.	-99.607	-2.15E-05	14372.	9.45E+09	-6.014	111932.	0.00
39.9610	1.84E-04	6215.	-126.390	-1.78E-05	14359.	9.45E+09	-3.927	114836.	0.00
40.4100	9.81E-05	5509.	-142.746	-1.44E-05	14343.	9.45E+09	-2.144	117739.	0.00
40.8590	2.89E-05	4734.	-150.266	-1.15E-05	14325.	9.45E+09	-0.647	120642.	0.00
41.3080	-2.58E-05	3935.	-150.418	-9.03E-06	14307.	9.45E+09	0.5908	123545.	0.00
41.7570	-6.84E-05	3148.	-144.505	-7.01E-06	14289.	9.45E+09	1.6041	126448.	0.00
42.2060	-1.01E-04	2405.	-133.635	-5.42E-06	14273.	9.45E+09	2.4310	129351.	0.00
42.6550	-1.27E-04	1730.	-118.702	-4.24E-06	14257.	9.45E+09	3.1119	132254.	0.00
43.1040	-1.47E-04	1143.	-100.386	-3.42E-06	14244.	9.45E+09	3.6870	135157.	0.00
43.5530	-1.64E-04	661.6751	-79.154	-2.91E-06	14233.	9.45E+09	4.1939	138060.	0.00
44.0020	-1.78E-04	301.4472	-55.287	-2.63E-06	14225.	9.45E+09	4.6657	140963.	0.00
44.4510	-1.92E-04	76.3260	-28.901	-2.53E-06	14220.	9.45E+09	5.1284	143866.	0.00
44.9000	-2.06E-04	0.00	0.00	-2.51E-06	14218.	9.45E+09	5.5996	73385.	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	=	-0.3900000 inches
Computed slope at pile head	=	0.000000 radians
Maximum bending moment	=	163939. inch-lbs
Maximum shear force	=	1401. lbs
Depth of maximum bending moment	=	0.000000 feet below pile head
Depth of maximum shear force	=	29.63400000 feet below pile head
Number of iterations	=	6
Number of zero deflection points	=	2
Pile deflection at ground	=	-0.0585870 inches

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 0.140000 inches
Rotation of pile head = -1.000E-02 radians
Axial load on pile head = 367000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.1400	781101.	-3712.	-0.01000	31838.	9.45E+09	0.00	0.00	0.00
0.4490	0.08732	780435.	-3712.	-0.00955	31823.	9.45E+09	0.00	0.00	0.00
0.8980	0.03704	778890.	-3712.	-0.00911	31788.	9.45E+09	0.00	0.00	0.00
1.3470	-0.01085	776467.	-3712.	-0.00867	31733.	9.45E+09	0.00	0.00	0.00
1.7960	-0.05635	773168.	-3712.	-0.00822	31659.	9.45E+09	0.00	0.00	0.00
2.2450	-0.09948	768997.	-3712.	-0.00779	31565.	9.45E+09	0.00	0.00	0.00
2.6940	-0.140	763958.	-3712.	-0.00735	31451.	9.45E+09	0.00	0.00	0.00
3.1430	-0.179	758059.	-3712.	-0.00691	31318.	9.45E+09	0.00	0.00	0.00
3.5920	-0.215	751305.	-3712.	-0.00648	31166.	9.45E+09	0.00	0.00	0.00
4.0410	-0.249	743703.	-3712.	-0.00606	30994.	9.45E+09	0.00	0.00	0.00
4.4900	-0.280	735263.	-3712.	-0.00564	30804.	9.45E+09	0.00	0.00	0.00
4.9390	-0.309	725994.	-3712.	-0.00522	30595.	9.45E+09	0.00	0.00	0.00
5.3880	-0.336	715906.	-3712.	-0.00481	30367.	9.45E+09	0.00	0.00	0.00
5.8370	-0.361	705010.	-3712.	-0.00440	30122.	9.45E+09	0.00	0.00	0.00
6.2860	-0.384	693320.	-3712.	-0.00400	29858.	9.45E+09	0.00	0.00	0.00
6.7350	-0.404	680848.	-3712.	-0.00361	29577.	9.45E+09	0.00	0.00	0.00
7.1840	-0.423	667609.	-3712.	-0.00323	29278.	9.45E+09			

0.00	0.00	0.00				
7.6330	-0.439	653616.	-3712.	-0.00285	28962.	9.45E+09
0.00	0.00	0.00				
8.0820	-0.453	638886.	-3712.	-0.00248	28630.	9.45E+09
0.00	0.00	0.00				
8.5310	-0.466	623436.	-3712.	-0.00212	28281.	9.45E+09
0.00	0.00	0.00				
8.9800	-0.476	607284.	-3712.	-0.00177	27917.	9.45E+09
0.00	0.00	0.00				
9.4290	-0.485	590446.	-3712.	-0.00143	27537.	9.45E+09
0.00	0.00	0.00				
9.8780	-0.492	572942.	-3712.	-0.00110	27142.	9.45E+09
0.00	0.00	0.00				
10.3270	-0.497	554793.	-3712.	-7.77E-04	26733.	9.45E+09
0.00	0.00	0.00				
10.7760	-0.500	536017.	-3712.	-4.66E-04	26310.	9.45E+09
0.00	0.00	0.00				
11.2250	-0.502	516638.	-3712.	-1.66E-04	25872.	9.45E+09
0.00	0.00	0.00				
11.6740	-0.502	496675.	-3712.	1.23E-04	25422.	9.45E+09
0.00	0.00	0.00				
12.1230	-0.500	476153.	-3712.	4.00E-04	24959.	9.45E+09
0.00	0.00	0.00				
12.5720	-0.497	455094.	-3712.	6.66E-04	24484.	9.45E+09
0.00	0.00	0.00				
13.0210	-0.493	433522.	-3712.	9.19E-04	23998.	9.45E+09
0.00	0.00	0.00				
13.4700	-0.488	411460.	-3712.	0.00116	23500.	9.45E+09
0.00	0.00	0.00				
13.9190	-0.481	388935.	-3712.	0.00139	22992.	9.45E+09
0.00	0.00	0.00				
14.3680	-0.473	365971.	-3712.	0.00160	22474.	9.45E+09
0.00	0.00	0.00				
14.8170	-0.463	342595.	-3712.	0.00181	21946.	9.45E+09
0.00	0.00	0.00				
15.2660	-0.453	318832.	-3712.	0.00199	21410.	9.45E+09
0.00	0.00	0.00				
15.7150	-0.442	294710.	-3712.	0.00217	20866.	9.45E+09
0.00	0.00	0.00				
16.1640	-0.430	270255.	-3712.	0.00233	20315.	9.45E+09
0.00	0.00	0.00				
16.6130	-0.417	245496.	-3712.	0.00248	19756.	9.45E+09
0.00	0.00	0.00				
17.0620	-0.403	220460.	-3712.	0.00261	19191.	9.45E+09
0.00	0.00	0.00				
17.5110	-0.389	195175.	-3712.	0.00273	18621.	9.45E+09
0.00	0.00	0.00				
17.9600	-0.374	169670.	-3712.	0.00283	18046.	9.45E+09
0.00	0.00	0.00				
18.4090	-0.358	143974.	-3712.	0.00292	17466.	9.45E+09

```
0.00        0.00        0.00
   18.8580     -0.342    118115.     -3712.    0.00300     16883.   9.45E+09
0.00        0.00        0.00
   19.3070     -0.326     92124.     -3712.    0.00306     16297.   9.45E+09
0.00        0.00        0.00
   19.7560     -0.309     66028.     -3712.    0.00310     15708.   9.45E+09
0.00        0.00        0.00
   20.2050     -0.292     39858.     -3712.    0.00313     15118.   9.45E+09
0.00        0.00        0.00
   20.6540     -0.275     13643.     -3712.    0.00315     14526.   9.45E+09
0.00        0.00        0.00
   21.1030     -0.259    -12587.     -3712.    0.00315     14502.   9.45E+09
0.00        0.00        0.00
   21.5520     -0.242    -38803.     -3712.    0.00313     15094.   9.45E+09
0.00        0.00        0.00
   22.0010     -0.225    -64976.     -3712.    0.00310     15684.   9.45E+09
0.00        0.00        0.00
   22.4500     -0.208    -91075.     -3689.    0.00306     16273.   9.45E+09
8.3581   216.3696      0.00
   22.8990     -0.192   -116829.     -3599.    0.00300     16854.   9.45E+09
25.0195   702.8786      0.00
   23.3480     -0.176   -141725.     -3419.    0.00293     17415.   9.45E+09
41.9641      1286.      0.00
   23.7970     -0.160   -165242.     -3151.    0.00284     17946.   9.45E+09
57.3428      1928.      0.00
   24.2460     -0.145   -186909.     -2808.    0.00274     18435.   9.45E+09
70.0951      2601.      0.00
   24.6950     -0.131   -206330.     -2404.    0.00263     18873.   9.45E+09
79.7005      3284.      0.00
   25.1440     -0.117   -223205.     -1957.    0.00250     19253.   9.45E+09
86.2185      3973.      0.00
   25.5930     -0.104   -237325.     -1486.    0.00237     19572.   9.45E+09
88.8067      4611.      0.00
   26.0420   -0.09136   -248599.     -1009.    0.00223     19826.   9.45E+09
88.1369      5198.      0.00
   26.4910   -0.07970   -257034.   -540.939    0.00209     20016.   9.45E+09
85.6986      5793.      0.00
   26.9400   -0.06884   -262692.    -99.510    0.00194     20144.   9.45E+09
78.1575      6117.      0.00
   27.3890   -0.05878   -265784.   313.8676    0.00179     20214.   9.45E+09
75.2865      6901.      0.00
   27.8380   -0.04954   -266391.   719.3590    0.00164     20227.   9.45E+09
75.2300      8181.      0.00
   28.2870   -0.04112   -264514.      1119.    0.00149     20185.   9.45E+09
73.1384      9583.      0.00
   28.7360   -0.03351   -260215.      1513.    0.00134     20088.   9.45E+09
73.0270     11740.      0.00
   29.1850   -0.02671   -253503.      1903.    0.00119     19937.   9.45E+09
71.7097     14468.      0.00
   29.6340   -0.02068   -244423.      2281.    0.00105     19732.   9.45E+09
```

68.7211	17909.	0.00				
30.0830	-0.01540	-233072.	2631.	9.13E-04	19476.	9.45E+09
61.1706	21407.	0.00				
30.5320	-0.01083	-219683.	2918.	7.84E-04	19174.	9.45E+09
45.4935	22626.	0.00				
30.9810	-0.00695	-204725.	3124.	6.63E-04	18836.	9.45E+09
30.7385	23845.	0.00				
31.4300	-0.00369	-188645.	3253.	5.51E-04	18474.	9.45E+09
17.1500	25065.	0.00				
31.8790	-0.00101	-171853.	3312.	4.48E-04	18095.	9.45E+09
4.9141	26284.	0.00				
32.3280	0.00114	-154726.	3310.	3.55E-04	17709.	9.45E+09
-5.839	27503.	0.00				
32.7770	0.00282	-137593.	3253.	2.72E-04	17322.	9.45E+09
-15.032	28722.	0.00				
33.2260	0.00407	-120741.	3152.	1.98E-04	16942.	9.45E+09
-22.634	29942.	0.00				
33.6750	0.00496	-104411.	3014.	1.34E-04	16574.	9.45E+09
-28.657	31161.	0.00				
34.1240	0.00552	-88794.	2847.	7.89E-05	16221.	9.45E+09
-33.152	32380.	0.00				
34.5730	0.00580	-74040.	2660.	3.24E-05	15889.	9.45E+09
-36.199	33600.	0.00				
35.0220	0.00587	-60253.	2461.	-5.86E-06	15578.	9.45E+09
-37.907	34819.	0.00				
35.4710	0.00574	-47499.	2255.	-3.66E-05	15290.	9.45E+09
-38.404	36038.	0.00				
35.9200	0.00547	-35806.	2050.	-6.03E-05	15026.	9.45E+09
-37.836	37257.	0.00				
36.3690	0.00509	-25171.	1850.	-7.77E-05	14786.	9.45E+09
-36.360	38477.	0.00				
36.8180	0.00463	-15564.	1660.	-8.93E-05	14570.	9.45E+09
-34.143	39696.	0.00				
37.2670	0.00413	-6929.	1484.	-9.57E-05	14375.	9.45E+09
-31.355	40915.	0.00				
37.7160	0.00360	802.1999	1218.	-9.75E-05	14237.	9.45E+09
-67.075	100320.	0.00				
38.1650	0.00308	6586.	878.8274	-9.54E-05	14367.	9.45E+09
-58.976	103223.	0.00				
38.6140	0.00257	10650.	583.3298	-9.05E-05	14459.	9.45E+09
-50.711	106126.	0.00				
39.0630	0.00210	13229.	332.0433	-8.37E-05	14517.	9.45E+09
-42.565	109029.	0.00				
39.5120	0.00167	14559.	123.7395	-7.57E-05	14547.	9.45E+09
-34.756	111932.	0.00				
39.9610	0.00129	14862.	-43.808	-6.74E-05	14554.	9.45E+09
-27.437	114836.	0.00				
40.4100	9.47E-04	14353.	-173.486	-5.90E-05	14542.	9.45E+09
-20.699	117739.	0.00				
40.8590	6.51E-04	13226.	-268.536	-5.12E-05	14517.	9.45E+09

```
-14.583    120642.       0.00
   41.3080   3.96E-04     11662.   -332.282  -4.41E-05     14481.   9.45E+09
-9.079    123545.       0.00
   41.7570   1.76E-04      9820.   -367.900  -3.79E-05     14440.   9.45E+09
-4.142    126448.       0.00
   42.2060  -1.28E-05      7847.   -378.228  -3.29E-05     14395.   9.45E+09
0.3081    129351.       0.00
   42.6550  -1.78E-04      5874.   -365.624  -2.90E-05     14351.   9.45E+09
4.3702    132254.       0.00
   43.1040  -3.25E-04      4022.   -331.874  -2.62E-05     14309.   9.45E+09
8.1575    135157.       0.00
   43.5530  -4.60E-04      2401.   -278.144  -2.43E-05     14273.   9.45E+09
11.7868    138060.       0.00
   44.0020  -5.87E-04      1121.   -204.988  -2.33E-05     14244.   9.45E+09
15.3683    140963.       0.00
   44.4510  -7.11E-04   284.7802   -112.413  -2.29E-05     14225.   9.45E+09
18.9952    143866.       0.00
   44.9000  -8.35E-04       0.00       0.00  -2.28E-05     14218.   9.45E+09
22.7319     73385.       0.00
```

```
* This analysis computed pile response using nonlinear moment-curvature rela-
  tionships. 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 inter-
  polated from the output for nonlinear bending properties relative to the
  magnitude of bending moment developed in the pile.


Output Summary for Load Case No. 2:

Pile-head deflection             =     0.14000000 inches
Computed slope at pile head      =     -0.0100000 radians
Maximum bending moment           =        781101. inch-lbs
Maximum shear force              =         -3712. 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             =              9
Number of zero deflection points =              3
Pile deflection at ground        =     -0.2173874 inches


--------------------------------------------------------------------------------
            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	-0.390	S, rad	0.00	367000.	-0.390	0.00	1401.	163939.
2	y, in	0.1400	S, rad	-0.01000	367000.	0.1400	-0.01000	-3712.	781101.

Maximum pile-head deflection = -0.3900000000 inches
Maximum pile-head rotation   = -0.0100000000 radians = -0.572958 deg.

The analysis ended normally.

CALCULATIONS

Date:	4/15/2026	**Made by:**	DEB
Project No.:	US0040947.9162	**Checked by:**	ATM
Subject:	Pile Drivability at Abutment No. 1 - HP14x89 - Driving	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine if the proposed H-piles at Abutment No. 1 can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/aisc/publications/steel-construction-manual/15th-ed-steel-construction-manual/shapes-database-v150/
3. Smith Brook Bridge No. 3189 over Smith Brook, Bancroft TWP 60% Plans, Maine, prepared for MaineDOT, dated February 5, 2026.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled NBB Bridge 3189 Smith Brook Abutment Final Pile Design, dated 3/19/2026.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 10th Ed. 2024.
9. Email from Hoyle Tanner Structural team received on March 13, 2026 with subject line "RE: [External] NBB-Geotech-Ext: Bridge 3189 updated pile loads".

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on five (5) HP14x89 Grade 50 piles with a center-to-center pile spacing of 7.715 feet, Ref. 3, Sheet 27 and Ref. 9 Abutment No. 1. The piles are to be driven to bedrock.
2. The soil profile used in the analysis is based on Ref. 10 at the centerline of the proposed Abutment 1.
3. As per the MaineDOT Standard Specifications Section 501.042 (Ref. 4), the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material.
4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).
5. This analysis looks at resistance during driving and upon reaching the tip elevation in bedrock. A resistance distribution of 90% shaft resistance and 10% toe resistance is assumed during driving, and 10% shaft resistance and 90% toe resistance is assumed upon reaching the tip elevation.
6. Assume a pile cap embedment of 2-feet for the piles (Ref. 3 sheet 27).

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	4/15/2026	**Made by:**	DEB
Project No.:	US0040947.9162	**Checked by:**	ATM
Subject:	Pile Drivability at Abutment No. 1 - HP14x89 - Driving	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 7)
Elastic modulus =	285.0	ksi	(Ref. 7)
Cushion thickness =	3.5	in	(Ref. 7)
Coefficient of restitution =	0.80		(Ref. 7)
Helmet weight =	2.7	kips	(Ref. 7)

Pile Information:

For an HP14x89 pile section:

	Intact Section					
Section depth, d =	13.8	in	(Ref. 2)			
Section width, b_f =	14.7	in	(Ref. 2)			
Web thickness, t_w =	0.615	in	(Ref. 2)			
Flange thickness, t_f =	0.615	in	(Ref. 2)			
Cross-sectional area, A_s =	26.1	in^2	=	0.18	ft^2	(Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)			
90% of steel yield stress =	45	ksi				
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)			
Steel unit weight =	491	pcf				
Box perimeter = (d × 2) + (b_f × 2) =	57.0	in	=	4.8	ft	
Pile head elevation =	342.2	ft	(Ref. 3, Sheet 27)			
Pile tip elevation =	295.3	ft	(Ref. 6)			
ɩutment embedment length =	2.0	ft	(Ref. 3, sheet 27)			
Pile length below abutment=	44.9	ft	(Ref. 6) (excluding pile cap embedment)			

Ultimate Capacities:

Axial load on pile, P_u =	367	kips	(Ref. 6, includes pile weight)
φ_{dyn} =	0.65		(Ref. 8, Table 10.5.5.2.3-1)
Pile driving resistance, R_{ndr} =	565	kips	

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
ɔe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.05	s/ft	for non-cohesive soils (Ref. 1, included as Attachment 1)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

CALCULATIONS

Date:	4/15/2026	**Made by:**	DEB
Project No.:	US0040947.9162	**Checked by:**	ATM
Subject:	Pile Drivability at Abutment No. 1 - HP14x89 - Driving	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Soil Profile Input, to calculate Variable Resistance Distribution:
(Ref. 10)

Pile head elevation = 340.2 ft Does not include embedment in pile cap
Groundwater elevation = 326.0 ft = 14.2 ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Existing Fill	340.2	336.3	0.0	3.9	125	36	-	-	-	Sand
Alluvial Sand & Gravel	336.3	323.0	3.9	17.2	125	31	-	-	-	Sand
Alluvial Silt	323.0	302.8	17.2	37.4	115	31	-	-	-	Sand
Glacial Till	302.8	295.3	37.4	44.9	130	36	-	-	-	Sand
Bedrock	295.3	-	44.9	-	165	-	-	24.0	1008.0	Rock Other

2. Analyze an open ended diesel (OED) hammer to determine the R_{ndr} for the abutment based on the stress and blow counts outlined in Ref. 4.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
During Driving	90%	10%	APE D25-42	55,814	Fuel Setting 3 (90%)	565	5.5	28.23	56%
Reaching tip elevation	10%	90%	APE D25-42	55,814	Fuel Setting 3 (90%)	565	6.5	39.64	79%

CONCLUSIONS

The analysis indicates the APE D25-42 hammer operated at fuel setting 3 (90% of the combustion pressure) should be able to overcome the soil resistance in order to drive the plumb piles at Abutment No. 1 to bedrock and a nominal resistance of 565 kips while staying at a blow count of between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT Standard Specifications.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

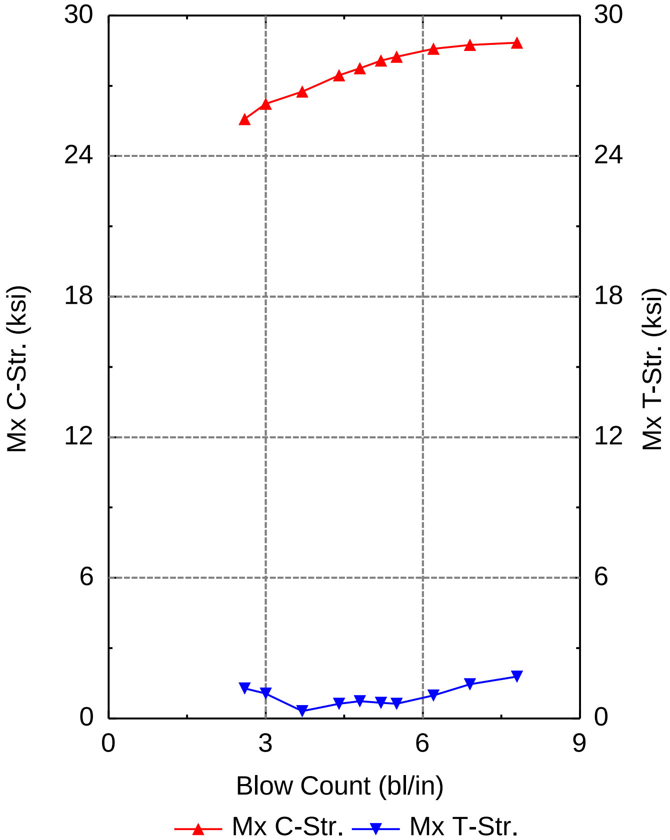
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

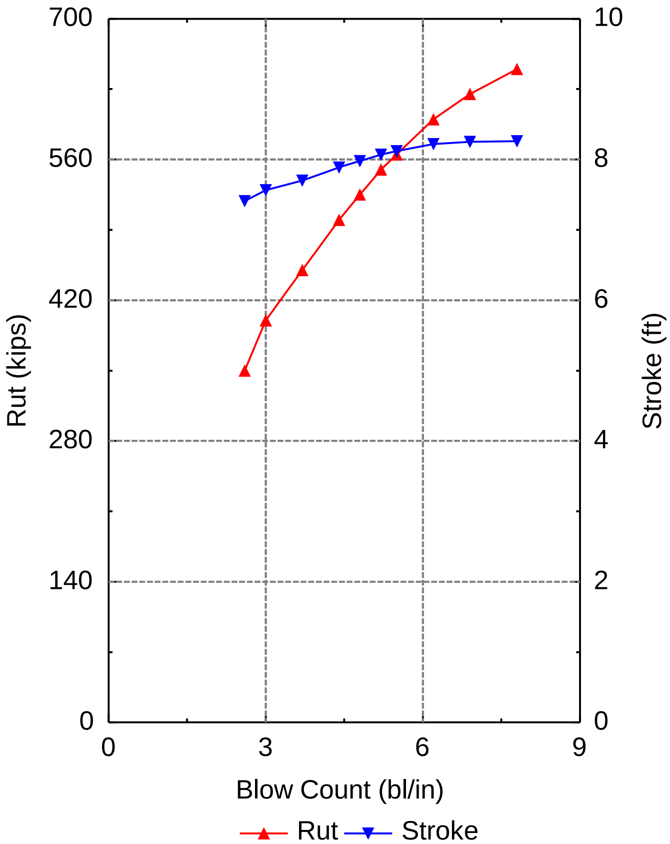
* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

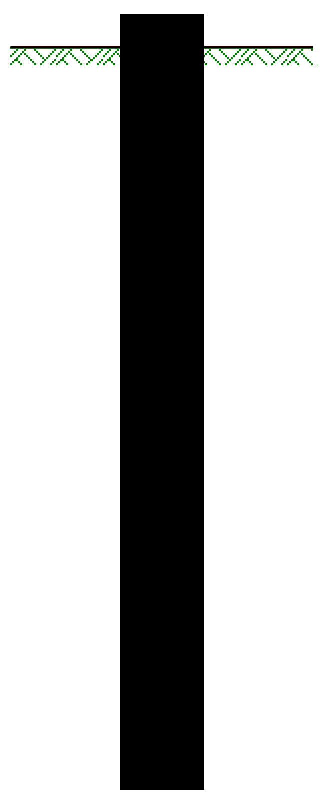
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



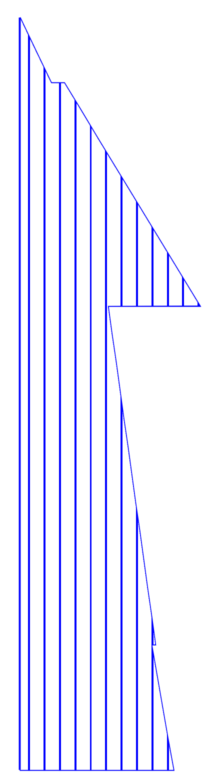


APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1282.5 (90%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.123	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	46.90	ft
Pile Penetration	44.90	ft
Pile Top Area	26.10	in²
RSA	No	



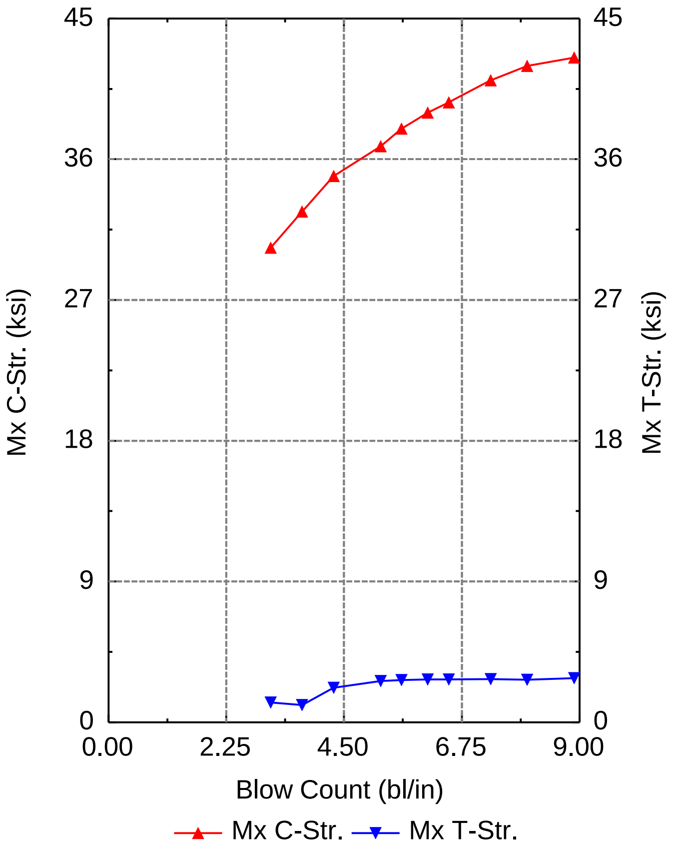
Pile Model



Shaft=90%
(Prop., SA)

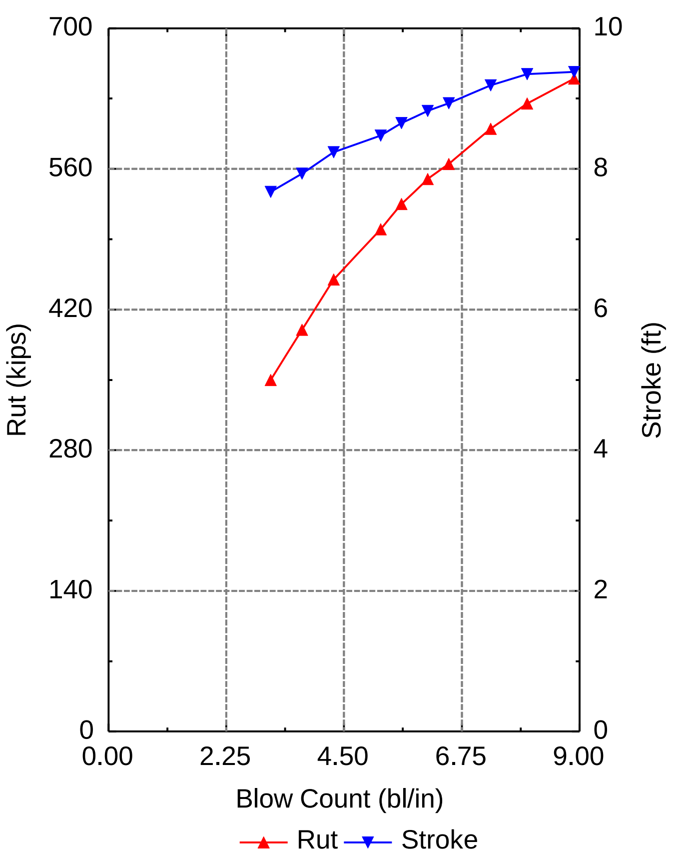
Bearing Graph Summary — APE D 25-42

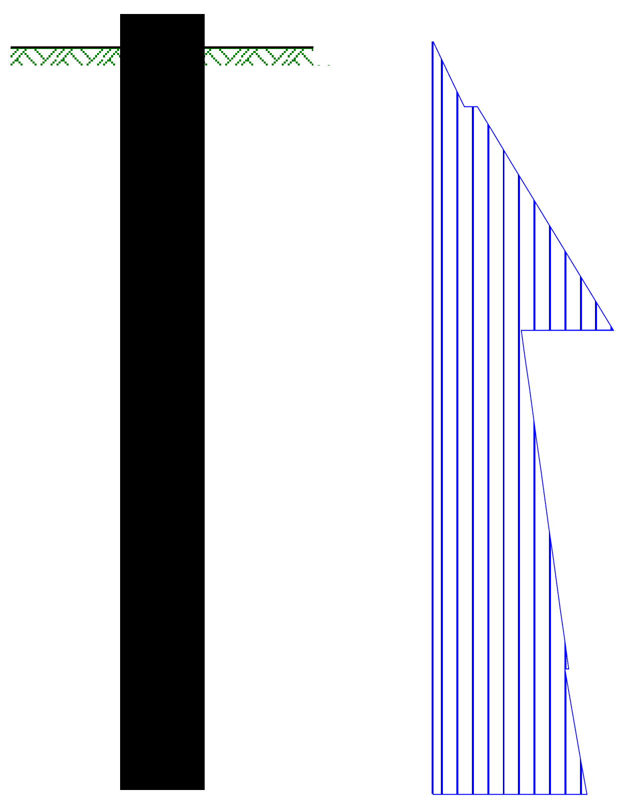
Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham.	Pow. Trfr. R.
kips	ksi	ksi	ksi	bl/in	ft	kip-ft	APE	%
350.0	25.58	25.12	1.27	2.6	7.41	19.43	D 25-42	31.3
400.0	26.23	25.71	1.05	3.0	7.56	18.96	D 25-42	30.6
450.0	26.75	26.23	0.31	3.6	7.70	18.93	D 25-42	30.5
500.0	27.45	26.87	0.63	4.4	7.89	19.21	D 25-42	31.0
525.0	27.74	27.20	0.73	4.8	7.98	19.32	D 25-42	31.2
550.0	28.08	27.49	0.67	5.2	8.07	19.38	D 25-42	31.2
565.0	28.23	27.66	0.63	5.5	8.12	19.42	D 25-42	31.3
600.0	28.58	27.98	0.98	6.2	8.22	19.45	D 25-42	31.4
625.0	28.75	28.12	1.46	6.9	8.25	19.37	D 25-42	31.2
650.0	28.83	28.20	1.78	7.8	8.26	19.22	D 25-42	31.0



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1282.5 (90%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	46.90	ft
Pile Penetration	44.90	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model

Shaft=10%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham.	Pow. Trfr. R.
kips	ksi	ksi	ksi	bl/in	ft	kip-ft	APE	%
350.0	30.34	25.38	1.26	3.1	7.68	20.06	D 25-42	32.4
400.0	32.65	26.18	1.10	3.7	7.93	20.56	D 25-42	33.2
450.0	34.92	27.06	2.21	4.3	8.24	21.17	D 25-42	34.1
500.0	36.83	27.76	2.64	5.2	8.47	21.49	D 25-42	34.6
525.0	37.96	28.26	2.69	5.6	8.65	21.99	D 25-42	35.5
550.0	38.98	28.86	2.73	6.1	8.82	22.51	D 25-42	36.3
565.0	39.64	29.59	2.74	6.5	8.93	22.87	D 25-42	36.9
600.0	41.04	31.13	2.75	7.3	9.19	23.66	D 25-42	38.2
625.0	41.97	32.13	2.72	8.0	9.35	24.12	D 25-42	38.9
650.0	42.50	32.95	2.82	8.9	9.38	24.20	D 25-42	39.0

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
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Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 2	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

OBJECTIVE

Determine if the proposed HP14x89 piles will provide adequate support for Abutment No. 2 of the proposed Smith Brook Bridge (Bridge No. 3189) based on the anticipated thermal movement, rotation and axial loads for the Strength I Load Combination.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Ed, 2024

2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

3. Hoyle Tanner, Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Bridge 3189. 60% Plans dated February 5. 2026.

4. WSP Interpretive Subsurface Profile, included as Sheet 3 of the Geotechnical Design Report (GDR).

5. FHWA. July 2016. GEC-012: Design and Construction of Driven Pile Foundations, Volume 1. Publication No. FHWA-NHI-16-009.

6. Email from Hoyle Tanner Structural team received on March 13, 2026 with subject line "RE: [External] NBB-Geotech-Ext: Bridge 3189 updated pile loads".

7. Hoyle Tanner, Abutment Loading Calculations, provided in "Integral Abutment Pile Loads_2025_07_24.pdf", received by WSP via email on July 24th, 2025.

8. WSP Laboratory Testing Results, included as Appendix C of the GDR.

9. WSP, Material properties table named "NBB Bridge 3189 Geotechnical Materials Properties.xlsx"

10. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2025. https://bsi.ce.ufl.edu/documentation/fbmp/manual

11. WSP USA Inc. Table 4: Summary of Rock Laboratory Testing Results, included in the GDR.

12. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

13. AISC Steel Construction Manual, 13th Ed. 2005.

14. WSP Boring Logs, included as Appendix A of the Geotechnical Design Report.

15. VTrans Structures Section, Integral Abutment Committee. 2008. Integral Abutment Bridge Design Guidelines, 2nd Ed.

16. FHWA, September 2018, GEC-010: Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024.

CALCULATIONS

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ASSUMPTIONS

1. The proposed abutment is assumed to be supported on five HP14x89 Grade 50 piles with a center-to-center pile spacing of 7.715 feet, equal to the proposed bridge girder spacing adjusted for the abutment skew from Ref. 3, Sheet 8 and Ref. 7 Page 10. The piles are assumed to be driven to bedrock and thus end-bearing.
2. The selected pile orientation is weak axis bending (Ref. 2, page 5-42).
3. The vertical load is assumed to be evenly distributed.
4. The soil profile used in the analysis is based on Ref. 4 at the centerline of the proposed Abutment No. 1.
5. An UCS for rock of 7,000 psi is assumed based on a field characterization of R3 (Medium Strong Rock) to R4 (Strong Rock) from Ref. 14.
6. Based on previous experience on nearby projects and the bridge being located within a no salt zone, section loss due to corrosion is not considered for this anlaysis.
7. Depth to fixity for deflection is taken as the depth along the pile to the point of zero lateral deflection; zero deflection is assumed to be deflection values less than 0.001 inches (Ref. 2 Section 5.4.2.5). Depth to fixity for bending moment is taken as the depth along the pile where the bending moment is less than 5% of the maximum bending moment (Ref. 16 page 12-19).
8. The piles are modeled in L-Pile as a fixed connection at the top of the pile.

ATTACHMENTS

1. LPile analysis output for Strength I

CALCULATION

1. Select the pile section parameters

	Intact Section (Ref. 12)
Pile depth, d =	13.8
Pile width, b_f =	14.7
Web thickness, t_w =	0.615
Flange thickness, t_f =	0.615
Fillet area =	0.3
Section area =	26.1
Moment of inertia about the x-axis, I_x =	888
Moment of inertia about the y-axis, I_y =	326
Radius of gyration about the x-axis, r_x =	5.83
Radius of gyration about the y-axis, r_y =	3.53
Elastic section modulus about the x-axis, S_x =	129

CALCULATIONS

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Elastic section modulus about the y-axis, S_y = 44.3

Plastic section modulus about the x-axis, Z_x = 143

Plastic section modulus about the y-axis, Z_y = 67.6

2. Determine pile loads and select the preliminary pile size.

Determine the factored substructure vertical dead and live load (P_u) distributed to each pile.

Strength I factored vertical load per pile = 363.0 kips (Ref. 6)

Pile weight = 0.089 kip/ft x 39.9 ft = 3.6 kips

P_u = 367 kips (Total factored axial load including pile weight.)

Select the steel pile strength.

F_y = 50 ksi (Assumption 1)

E = 29,000 ksi (typical value for steel)

Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} =	0.50	for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)
ϕ_{cu} =	0.70	for axial resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)
ϕ_f =	1.00	for flexural resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 524 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 733 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

CALCULATIONS

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$R_n =$ 733 kips

Use the required nominal axial pile resistance to estimate an approximate initial pile area.

$$A_{s.req} = \frac{R_n}{0.80\,Fy}$$ (Ref. 2, section 5-42)

$A_{s.req} =$ 18.3 in^2

Select a pile size with an area of approximately $A_{s.req}$.

HP14x89 section as per Assumption 1 and Step 1:

$A_s =$ 26.1 in^2 (Step 1)

3. Use LPile analysis to determine the pile unbraced length and maximum moment at the top of the pile.

The following input parameters were used in the LPile analysis:

Pile Properties

Section type:	Steel H Section Weak Axis		(Assumption 2)	
Length of section:	39.9	ft	(piles driven to bedrock)	
Flange width, b_f:	14.7	in	(Step 1)	
Section depth, d:	13.8	in	(Step 1)	
Flange thickness, t_f:	0.615	in	(Step 1)	
Web thickness, t_w:	0.615	in	(Step 1)	
Moment of inertia, I_y:	326	in^4	(Step 1)	about the weak axis, since weak-axis bending
Plastic section modulus, Z_y:	67.6	in^3	(Step 1)	about the weak axis, since weak-axis bending
Elastic section modulus, S_y:	44.3	in^3	(Step 1)	about the weak axis, since weak-axis bending
Radius of gyration, r_y:	3.53	in	(Step 1)	about the weak axis, since weak-axis bending
Pile batter:	Vertical		(pile battering not required)	

CALCULATIONS

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Pile Loading

Case	ΔT (in) [1]	Rotation (rad)[2]	Axial Load (lbs)[3]
1 (Thermal contraction)	0.39	0.000	367,000
2 (Thermal expansion)	-0.14	0.010	367,000

1) Strength I Lateral deflection due to abutment thermal expansion or contraction per Ref. 6
2) Assumed Rotations per Ref. 6.
3). From Step 2

Soil Layers

Layer	Depth below base of abutment[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Undrained Shear Strength (psf)	Friction Angle (°)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Existing Fill	0.0 - 4.5 ft	Sand (Reese)	125	-	36	165.0	-	-
Alluvial Sand & Gravel	4.5 - 9.9 ft	Sand (Reese)	125	-	31	69	-	-
Alluvial Silt (AWT)	9.9 - 14.2 ft	Sand (Reese)	115	-	31	69		
Alluvial Silt (BWT)	14.2 - 21.8 ft	Sand (Reese)	52.6	-	31	42	-	-
Glacial Till	21.8 - 39.9 ft	Sand (Reese)	67.6	-	36	100.0	-	-
Bedrock	> 39.9 ft	Strong Rock (Vuggy Limestone)	102.6	-	-	-	-	7,000

1) Per Ref. 4, 3 and 14.
2) Ref. 9
3) Ref. 10 - interpolation based on friction angle for subgrade modulus
4) Assumption 5

The full LPile output is provided in Attachment 1.

CALCULATIONS

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Obtain the maximum moment in the top segment of the pile using the loading case that produces the highest moment.

The thermal expansion load case (Load case 2), resulted in the highest moment in the pile.

$M_{u.Top}$ = 2411 in-kips (from LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 8.2 ft (from LPile)
$l_{b.top}$ = 98.2 in

$l_{b.2nd}$ = 11.6 ft (from LPile)
$l_{b.2nd}$ = 139.1 in

4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the interaction of combined axial and flexural load effects on a single pile.

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each segment.

For the top segment (rotation fixed with no plastic hinge):

$$\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{top} = 1.2 (Ref. 1, Table C4.6.2.5-1)
r_y = 3.53 in (Step 1)

λ_{top} = 33.4 OK

For the second segment (pinned at top and bottom):

$$\lambda_{2nd} = \frac{K_{2nd} l_{b.2nd}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{2nd} = 39.4 OK

Calculate the critical elastic buckling resistance, P_e, and the nominal yield resistance, P_o.

CALCULATIONS

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Use Ref. 1 Table 6.9.4.1.1-1 to select equation for P_e based on cross-section shape and potential buckling mode.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\,l_b}{r_y}\right)^2} A_s$$ (Ref. 1, Eqn 6.9.4.1.2-1)

$P_{e.top}$ = 6699 kips
$P_{e.2nd}$ = 4812 kips

$$P_o = F_y A_s$$ (Ref. 1, Article 6.9.4.1)

P_o = 1305 kips

Calculate the nominal structural pile resistance, P_n, for both segments of the upper zone of the pile as well as the lower zone of the pile.

Determine P_o/P_e to select equation for P_n as per Ref. 1 Article 6.9.4.1.

$P_o/P_{e.top}$ = 0.19 ≤ 2.25
$P_o/P_{e.2nd}$ = 0.27 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 1203 kips
$P_{n.2nd}$ = 1165 kips

$$P_{n.bottom} = \left(0.658^{(0)}\right) P_o$$ (0 for a fully braced pile - Ref. 15, Appendix B, Eqn 6-9)

$P_{n.bottom}$ = 1305 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

$$P_{r.top} = \phi_{cu} P_{n.top}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.top}$ = 842 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.2nd}$ = 815 kips

CALCULATIONS

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$$P_{r.bottom} = \phi_{cl} P_{n.bottom}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.bottom} =$ 653 kips

Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20 (Ref. 2, page 5-43).

$\frac{P_u}{P_{r.top}} =$ 0.44 OK

$\frac{P_u}{P_{r.2nd}} =$ 0.45 OK

Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads (Ref. 15, Appendix B Eqn 6-17). Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check} \left(\frac{P_u}{P_{r.bottom}} < 1 \right)$$

$\frac{P_u}{P_{r.bottom}} =$ 0.56 OK

Determine the nominal and factored flexural resistance about the H-pile weak axis.

Slenderness ratio for the flange:

$$\lambda_f = \frac{b_f}{2t_f}$$ (Ref. 1, Eqn 6.12.2.2.1c-3)

$\lambda_f =$ 12.0

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1c-4)

$\lambda_{pf} =$ 9.2

Limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1c-5)

$\lambda r_f =$ 24.1

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	DEB/ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 2	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Since λpf < λf ≤ λrf, pile is noncompact per Ref. 1, Article 6.12.2.2.2c-12.

Elastic and plastic section moduli about the weak axis:

S_y = 44.3 in^3 (Step 1)

Z_y = 67.6 in^3 (Step 1)

Per Ref.1 section 6.12.2.2.1 the nominal flexural resistance about the weak axis shall be taken as the smaller value based on yielding or FLB (Flange Local Buckling).

Nominal flexural resistance based on yielding:

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad \text{, if } \lambda_f \leq \lambda_{pf}$$ (Ref. 1, Eqn 6.12.2.2.1b-1)

Nominal flexural resistance based on FLB:

Since λpf < λf ≤ λrf,

$$M_n = M_p - \left(M_p - 0.7\, F_{yf}\, S_y\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \text{ if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-1)

$$M_n = \frac{0.69 E S_y}{\lambda_f^2} \qquad \text{if } \lambda_f > \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-2)

For welded sections Fyf shall be taken as 0.5Fyc, where Fyc is the minimum yield stress of the compression flange.

M_p = 3380 in-kips

M_n = 2892 in-kips

Factored flexural resistance:

ϕ_f = 1.00 (Ref. 2, page 5-42)

$$M_r = \phi_f M_n$$

M_r = 2892 in-kips

For all members (including noncompact):

$$M_p' = \left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$ (Ref. 1, Eqn 6.9.2.2.1-3 and Ref. 15 Eq 4.5.2-3)

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'.

M_p' = 1633 in-kips = 1632757 inch-lb

CALCULATIONS

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If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation, and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top}$ =	2411	in-kips	(From Step 3)
$M_{u.Top}$	>	M_p'	Plastic Hinge Forms

Check the second segment of the upper zone of the pile to ensure that this segment of the pile does not form a plastic hinge.

$M_{u.2nd}$ =	168	in-kips	(LPile)

Check: $$\frac{P_u}{P_{r.2nd}} + \left(\frac{M_{u.2nd}}{M_r}\right) < 1$$ (Ref. 1, Eqn 6.9.2.2.1-3 for noncompact memeber, Ref. 15, Eqn 4.5.2-3)

Check: 0.51 < 1 OK

5. Run another LPile analysis with displacement, plastic moment (M_p'), and P_u as load conditions, and calculate new unbraced lengths from the moment vs. depth curve. Then repeat Step 4 with the new unbraced lengths.

$l_{b.top}$ =	8.8	ft	(LPile)
$l_{b.top}$ =	105.3	in	
$l_{b.2nd}$ =	11.4	ft	(LPile)
$l_{b.2nd}$ =	137.3	in	
$M_{u.2nd}$ =	93	in-kips	(LPile)

Since a plastic hinge developed at the pile head, the value of K for the top segment becomes 2.1 for free rotation condition (Ref. 2, page 5-43).

K_{top} =	2.1			(Ref. 1, Table C4.6.2.5-1)
K_{2nd} =	1.0			(Ref. 1, Table C4.6.2.5-1)
λ_{top} =	62.62	<	120	OK
λ_{2nd} =	38.91	<	120	OK
$P_{e.top}$ =	1905	kips		
$P_{e.2nd}$ =	4934	kips		

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

$P_o/P_{e.top} =$ 0.68 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd} =$ 0.26 ≤ 2.25 (to select P_n equation)

$P_{n.top} =$ 980 kips
$P_{n.2nd} =$ 1168 kips

$P_{r.top} =$ 686 kips
$P_{r.2nd} =$ 818 kips

$\frac{P_u}{P_{r.top}} =$ 0.53 > 0.20 OK

$\frac{P_u}{P_{r.2nd}} =$ 0.45 > 0.20 OK

Check the second segment of the upper zone of the pile to ensure that this segment of the pile does not form a plastic hinge.

$M_{u.2nd} =$ 93 in-kips (LPile)

Check: $$\frac{P_u}{P_{r.2nd}} + \left(\frac{M_{u.2nd}}{M_r}\right) < 1$$ (Ref. 1, Eqn 6.9.2.2.1-3 for noncompact memeber, Ref. 15, Eqn 4.5.2-3)

Check: 0.48 < 1 OK

6. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPile output against the structural shear resistance per AISC G7.

$V_u =$ 21.4 kips (from LPile)

AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition, Specifications G2 and G7 (Ref. 13) to ensure the pile will not shear under the longitudinal load.

Per Ref. 13 Section G7: for singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength V_n for each shear resisting element shall be determined using Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ and $k_v = 1.2$.

CALCULATIONS

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$$V_n = 0.6F_yA_wC_v$$ (Ref. 13, Eqn G2-1)

where:

$C_v =$ 1.0 for HP shapes with $F_y \leq 50$ ksi (Ref. 13, Section G7)

$A_w = b_ft_f =$ 9.0 in^2 (Ref. 13, Section G7)

Since there are two shear resisting elements (i.e., two flanges):

$2 \times A_w =$ 18.1 in^2

$V_n =$ 542 kips

$$V_r = \Phi_vV_n$$

$\phi_v =$ 1.00 (Ref. 1, Article 6.5.4.2)

$V_r =$ 542 kips

Check that the shear resistance is sufficient:

V_u < V_r OK

7. Check that the maximum factored applied pile load does not exceed the factored pile drivability resistance.

For steel piles in compression, driving stresses should not exceed 90% of the yield strength of the pile material (Ref. 2, page 5-104).

$\sigma_{dr} =$ 0.9 × 50 ksi = 45 ksi

This translates into an ultimate maximum driving force that can be applied to the pile of:

$$P_{dr} = \sigma_{dr}A_s$$ (Ref. 15, Appendix B, Eqn 7-23)

$P_{dr} =$ 1175 kips

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. In this design, the pile will be bearing on rock. The driving criteria will be established by dynamic testing.

$\phi_{dyn} =$ 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{dyn}}$$ (Ref. 2, Appendix B, Eqn 7-25)

$R_{ndr} =$ 564 kips

CALCULATIONS

Date:	3/19/2026	**Made by:**	ATM/LMP
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Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 2	**Reviewed by:**	CCB
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_{dr}) calculated above.

$P_{n.top}$ =	1203	kips	(From Step 4)
$P_{n.2nd}$ =	1165	kips	(From Step 4)

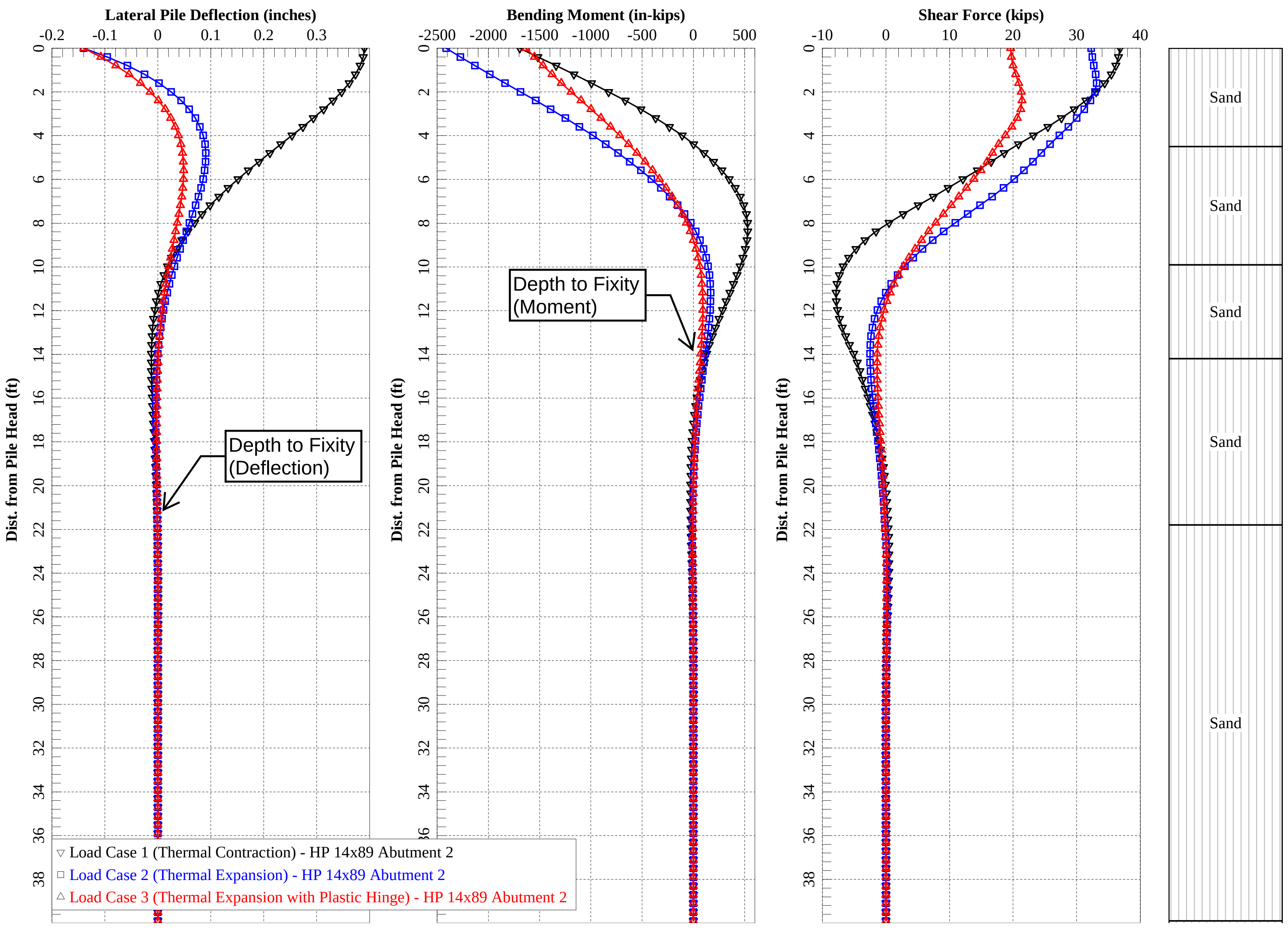
Check $R_{ndr} < P_n$: OK

Check $R_{ndr} < P_{dr}$: OK

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 2411 in-kips (201 ft-kips) and a plastic hinge moment of 1633 in-kips (136 ft-kips) occurs at the top of the pile under the Strength I load case with a maximum bridge expansion of -0.14 inches and a rotation of 0.010 radians. The results indicate that the depth to bedrock is sufficient for driven piles to achieve fixity. Based on the intact section used in the analysis, HP14x89 piles are adequately sized to accommodate the anticipated design loads and thermal movement at Abutment No.2 of the Smith Brook Bridge (Bridge No. 3189).

Attachment 1



```
==============================================================================

                 LPile for Windows, Version 2022-12.007

              Analysis of Individual Piles and Drilled Shafts
             Subjected to Lateral Loading Using the p-y Method
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==============================================================================

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------------------------------------------------------------------------------
                         Files Used for Analysis
------------------------------------------------------------------------------

Path to file locations:
\Users\USLP710461\Downloads\OneDrive_2026-03-19\LPile Models\

Name of input data file:
Abutment 2 HP14x89_ 3.20.26.lp12d

Name of output report file:
Abutment 2 HP14x89_ 3.20.26.lp12o

Name of plot output file:
Abutment 2 HP14x89_ 3.20.26.lp12p

Name of runtime message file:
Abutment 2 HP14x89_ 3.20.26.lp12r


------------------------------------------------------------------------------
                        Date and Time of Analysis
```

Date: March 20, 2026 Time: 14:19:53

Problem Title

Project Name: WIN 028262.00 Northern Bridge Bundle - Bridge No. 3189 -

Job Number: US0040947.9162

Client: MaineDOT

Engineer: WSP USA Inc.

Description: HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 2

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                                     =      39.900 ft
Depth of ground surface below top of pile                =      0.0000 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	39.900	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

```
   Section 1 is a H weak axis steel pile
   Length of section                                     =    39.900000 ft
```

```
   Pile width                                          =    13.800000 in



--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 6 layers

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

   Distance from top of pile to top of layer           =       0.0000 ft
   Distance from top of pile to bottom of layer        =     4.500000 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                      =    36.000000 deg.
   Friction angle at bottom of layer                   =    36.000000 deg.
   Subgrade k at top of layer                          =   165.000000 pci
   Subgrade k at bottom of layer                       =   165.000000 pci


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

   Distance from top of pile to top of layer           =     4.500000 ft
   Distance from top of pile to bottom of layer        =     9.900000 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                      =    31.000000 deg.
   Friction angle at bottom of layer                   =    31.000000 deg.
   Subgrade k at top of layer                          =    69.000000 pci
   Subgrade k at bottom of layer                       =    69.000000 pci


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

   Distance from top of pile to top of layer           =     9.900000 ft
   Distance from top of pile to bottom of layer        =    14.200000 ft
   Effective unit weight at top of layer               =   115.000000 pcf
   Effective unit weight at bottom of layer            =   115.000000 pcf
   Friction angle at top of layer                      =    31.000000 deg.
   Friction angle at bottom of layer                   =    31.000000 deg.
   Subgrade k at top of layer                          =    69.000000 pci
   Subgrade k at bottom of layer                       =    69.000000 pci


Layer 4 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer           =    14.200000 ft
   Distance from top of pile to bottom of layer        =    21.800000 ft
```

```
   Effective unit weight at top of layer                =    52.600000 pcf
   Effective unit weight at bottom of layer             =    52.600000 pcf
   Friction angle at top of layer                       =    31.000000 deg.
   Friction angle at bottom of layer                    =    31.000000 deg.
   Subgrade k at top of layer                           =    42.000000 pci
   Subgrade k at bottom of layer                        =    42.000000 pci


Layer 5 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer            =    21.800000 ft
   Distance from top of pile to bottom of layer         =    39.900000 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                       =    36.000000 deg.
   Friction angle at bottom of layer                    =    36.000000 deg.
   Subgrade k at top of layer                           =   100.000000 pci
   Subgrade k at bottom of layer                        =   100.000000 pci


Layer 6 is strong rock (vuggy limestone)

   Distance from top of pile to top of layer            =    39.900000 ft
   Distance from top of pile to bottom of layer         =    50.000000 ft
   Effective unit weight at top of layer                =   102.600000 pcf
   Effective unit weight at bottom of layer             =   102.600000 pcf
   Uniaxial compressive strength at top of layer        =        7000. psi
   Uniaxial compressive strength at bottom of layer     =        7000. psi


 (Depth of the lowest soil layer extends 10.100 ft below the pile tip)



--------------------------------------------------------------------------------
                        Summary of Input Soil Properties
--------------------------------------------------------------------------------

Layer        Soil Type          Layer      Effective    Angle of     Uniaxial

 Num.          Name             Depth       Unit Wt.    Friction        qu
 kpy
          (p-y Curve Type)       ft           pcf          deg.         psi
 pci
-----   -------------------   ----------   ----------   ----------   ----------
----------
  1          Sand                 0.00     125.0000      36.0000       --
165.0000
          (Reese, et al.)       4.5000     125.0000      36.0000       --
165.0000
```

```
  2           Sand              4.5000     125.0000     31.0000       --
69.0000
          (Reese, et al.)       9.9000     125.0000     31.0000       --
69.0000
  3           Sand              9.9000     115.0000     31.0000       --
69.0000
          (Reese, et al.)      14.2000     115.0000     31.0000       --
69.0000
  4           Sand             14.2000      52.6000     31.0000       --
42.0000
          (Reese, et al.)      21.8000      52.6000     31.0000       --
42.0000
  5           Sand             21.8000      67.6000     36.0000       --
100.0000
          (Reese, et al.)      39.9000      67.6000     36.0000       --
100.0000
  6        Strong Rock         39.9000     102.6000      --            7000.
 --
         (Vuggy Limestone)     50.0000     102.6000      --            7000.
 --
```

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 = 3

```
Load    Load         Condition               Condition           Axial Thrust
Compute Top y     Run Analysis
 No.    Type           1                       2                  Force, lbs
vs. Pile Length
-----   ----   --------------------   ----------------------   ----------------
---------------    ------------
   1     5     y =     0.390000 in    S =       0.0000 in/in           367000.
    N.A.                Yes
   2     5     y =     -0.14000 in    S =     0.010000 in/in           367000.
    N.A.                Yes
   3     4     y =     -0.14000 in    M =    -1632757. in-lbs          367000.
    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	=	39.900000 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	367.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 367.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000442	41.7773444	9448389.	118.2345231	15.1514881	
0.00000884	83.5546887	9448389.	62.7922615	16.0845352	
0.00001326	125.3320331	9448389.	44.3115077	17.0175823	
0.00001769	167.1093775	9448389.	35.0711308	17.9506295	
0.00002211	208.8867218	9448389.	29.5269046	18.8836766	
0.00002653	250.6640662	9448389.	25.8307538	19.8167238	
0.00003095	292.4414106	9448389.	23.1906462	20.7497709	
0.00003537	334.2187549	9448389.	21.2105654	21.6828180	
0.00003979	375.9960993	9448389.	19.6705026	22.6158653	
0.00004422	417.7734437	9448389.	18.4384523	23.5489123	
0.00004864	459.5507880	9448389.	17.4304112	24.4819595	
0.00005306	501.3281324	9448389.	16.5903769	25.4150066	
0.00005748	543.1054768	9448389.	15.8795787	26.3480537	
0.00006190	584.8828211	9448389.	15.2703231	27.2811009	
0.00006632	626.6601655	9448389.	14.7423015	28.2141480	
0.00007075	668.4375098	9448389.	14.2802827	29.1471952	
0.00007517	710.2148542	9448389.	13.8726190	30.0802423	
0.00007959	751.9921986	9448389.	13.5102513	31.0132894	
0.00008401	793.7695429	9448389.	13.1860275	31.9463366	
0.00008843	835.5468873	9448389.	12.8942262	32.8793838	
0.00009285	877.3242317	9448389.	12.6302154	33.8124309	
0.00009728	919.1015760	9448389.	12.3902056	34.7454780	
0.0001017	960.8789204	9448389.	12.1710662	35.6785251	
0.0001061	1003.	9448389.	11.9701885	36.6115723	
0.0001105	1044.	9448389.	11.7853809	37.5446194	
0.0001150	1086.	9448389.	11.6147893	38.4776666	
0.0001194	1128.	9448389.	11.4568342	39.4107137	
0.0001238	1170.	9448389.	11.3101615	40.3437608	
0.0001282	1212.	9448389.	11.1736042	41.2768080	
0.0001326	1253.	9448389.	11.0461508	42.2098551	
0.0001371	1295.	9448389.	10.9269201	43.1429023	
0.0001415	1337.	9448389.	10.8151413	44.0759494	
0.0001459	1379.	9448389.	10.7101371	45.0089965	
0.0001503	1420.	9448389.	10.6113095	45.9420437	
0.0001548	1462.	9448389.	10.5181292	46.8750908	
0.0001592	1504.	9448389.	10.4301256	47.8081380	

0.0001636	1546.	9448389.	10.3468790	48.7411851	
0.0001680	1588.	9448389.	10.2680138	49.6742323	
0.0001724	1629.	9443723.	10.1940496	50.0000000	Y
0.0001813	1706.	9409764.	10.0617064	50.0000000	Y
0.0001901	1778.	9351217.	9.9471160	50.0000000	Y
0.0001990	1845.	9275072.	9.8474020	50.0000000	Y
0.0002078	1909.	9186226.	9.7603030	50.0000000	Y
0.0002167	1969.	9088653.	9.6839052	50.0000000	Y
0.0002255	2026.	8985149.	9.6166785	50.0000000	Y
0.0002343	2080.	8877224.	9.5575126	50.0000000	Y
0.0002432	2132.	8767936.	9.5049842	50.0000000	Y
0.0002520	2182.	8657023.	9.4585881	50.0000000	Y
0.0002609	2230.	8546441.	9.4172999	50.0000000	Y
0.0002697	2276.	8436902.	9.3804708	50.0000000	Y
0.0002786	2320.	8328745.	9.3476031	50.0000000	Y
0.0002874	2363.	8221937.	9.3183563	50.0000000	Y
0.0002962	2405.	8117206.	9.2921946	50.0000000	Y
0.0003051	2445.	8014781.	9.2687565	50.0000000	Y
0.0003139	2485.	7914802.	9.2477355	50.0000000	Y
0.0003228	2523.	7815381.	9.2284423	50.0000000	Y
0.0003316	2558.	7712825.	9.2096343	50.0000000	Y
0.0003405	2590.	7608368.	9.1911858	50.0000000	Y
0.0003493	2621.	7502911.	9.1730365	50.0000000	Y
0.0003582	2649.	7397217.	9.1551297	50.0000000	Y
0.0003670	2676.	7291027.	9.1378246	50.0000000	Y
0.0003758	2701.	7185666.	9.1207678	50.0000000	Y
0.0003847	2724.	7081536.	9.1039508	50.0000000	Y
0.0003935	2746.	6977697.	9.0876016	50.0000000	Y
0.0004024	2767.	6875589.	9.0712177	50.0000000	Y
0.0004112	2786.	6774695.	9.0554922	50.0000000	Y
0.0004201	2804.	6676230.	9.0399111	50.0000000	Y
0.0004289	2822.	6578534.	9.0246785	50.0000000	Y
0.0004377	2838.	6483272.	9.0094336	50.0000000	Y
0.0004466	2854.	6389730.	8.9950233	50.0000000	Y
0.0004554	2868.	6297812.	8.9803119	50.0000000	Y
0.0004643	2882.	6208019.	8.9661427	50.0000000	Y
0.0004731	2895.	6120073.	8.9522270	50.0000000	Y
0.0004820	2908.	6034149.	8.9382517	50.0000000	Y
0.0004908	2920.	5950014.	8.9249942	50.0000000	Y
0.0004996	2932.	5867483.	8.9116029	50.0000000	Y
0.0005085	2943.	5787469.	8.8985011	50.0000000	Y
0.0005173	2953.	5708419.	8.8856728	50.0000000	Y
0.0005262	2963.	5631656.	8.8731656	50.0000000	Y
0.0005615	2999.	5340961.	8.8245281	50.0000000	Y
0.0005969	3030.	5075908.	8.7791277	50.0000000	Y
0.0006323	3056.	4833303.	8.7361931	50.0000000	Y
0.0006677	3079.	4611513.	8.6958050	50.0000000	Y
0.0007030	3099.	4407785.	8.6579820	50.0000000	Y
0.0007384	3116.	4220367.	8.6220712	50.0000000	Y
0.0007738	3132.	4047417.	8.5879985	50.0000000	Y

0.0008092	3146.	3887458.	8.5556478	50.0000000	Y
0.0008445	3158.	3739153.	8.5250335	50.0000000	Y
0.0008799	3169.	3601594.	8.4957228	50.0000000	Y
0.0009153	3179.	3473493.	8.4683918	50.0000000	Y
0.0009507	3188.	3353733.	8.4419138	50.0000000	Y
0.0009860	3197.	3241904.	8.4165778	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	367.0000000000	3197.

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	0.00	0.00	N.A.	No	0.00	21997.
2	4.5000	5.4056	Yes	No	21997.	101099.
3	9.9000	10.8067	Yes	No	123096.	185634.
4	14.2000	15.1901	Yes	No	308730.	513487.
5	21.8000	19.9766	Yes	No	822218.	3234609.
6	39.9000	39.9000	No	Yes	N.A.	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 = 0.390000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 367000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3900	-1693395.	36837.	0.00	52417.	9.41E+09	0.00	0.00	0.00
0.3990	0.3879	-1516952.	36566.	-8.16E-04	48437.	9.41E+09	-53.206	656.6776	0.00
0.7980	0.3822	-1340374.	36157.	-0.00154	44454.	9.45E+09	-117.498	1472.	0.00
1.1970	0.3732	-1165296.	35432.	-0.00218	40504.	9.45E+09	-185.221	2376.	0.00
1.5960	0.3613	-993426.	34389.	-0.00272	36627.	9.45E+09	-250.715	3322.	0.00
1.9950	0.3471	-826419.	33046.	-0.00318	32860.	9.45E+09	-309.893	4275.	0.00
2.3940	0.3308	-665781.	31439.	-0.00356	29237.	9.45E+09	-361.698	5235.	0.00
2.7930	0.3130	-512842.	29606.	-0.00386	25787.	9.45E+09	-403.982	6180.	0.00
3.1920	0.2939	-368707.	27597.	-0.00408	22535.	9.45E+09	-434.983	7087.	0.00
3.5910	0.2739	-234216.	25458.	-0.00424	19502.	9.45E+09	-458.541	8017.	0.00
3.9900	0.2533	-110029.	23198.	-0.00432	16700.	9.45E+09	-485.352	9175.	0.00
4.3890	0.2325	3130.	20833.	-0.00435	14289.	9.45E+09			

-502.780	10356.	0.00				
4.7880	0.2116	104760.	18610.	-0.00432	16582.	9.45E+09
-425.691	9632.	0.00				
5.1870	0.1910	196538.	16534.	-0.00425	18652.	9.45E+09
-441.670	11070.	0.00				
5.5860	0.1709	278015.	14371.	-0.00413	20490.	9.45E+09
-461.626	12930.	0.00				
5.9850	0.1515	348662.	12122.	-0.00397	22083.	9.45E+09
-477.675	15095.	0.00				
6.3840	0.1329	408048.	9809.	-0.00378	23423.	9.45E+09
-488.512	17596.	0.00				
6.7830	0.1153	455871.	7458.	-0.00356	24502.	9.45E+09
-493.757	20497.	0.00				
7.1820	0.09886	491969.	5095.	-0.00332	25316.	9.45E+09
-493.108	23883.	0.00				
7.5810	0.08356	516325.	2750.	-0.00306	25865.	9.45E+09
-486.335	27866.	0.00				
7.9800	0.06953	529072.	486.3927	-0.00280	26153.	9.45E+09
-459.386	31636.	0.00				
8.3790	0.05677	530816.	-1556.	-0.00253	26192.	9.45E+09
-393.867	33218.	0.00				
8.7780	0.04530	523058.	-3288.	-0.00226	26017.	9.45E+09
-329.281	34800.	0.00				
9.1770	0.03511	507285.	-4714.	-0.00200	25661.	9.45E+09
-266.761	36382.	0.00				
9.5760	0.02614	484946.	-5849.	-0.00175	25157.	9.45E+09
-207.263	37964.	0.00				
9.9750	0.01835	457422.	-6708.	-0.00151	24537.	9.45E+09
-151.559	39546.	0.00				
10.3740	0.01167	426018.	-7311.	-0.00129	23828.	9.45E+09
-100.241	41127.	0.00				
10.7730	0.00602	391935.	-7680.	-0.00108	23059.	9.45E+09
-53.729	42709.	0.00				
11.1720	0.00133	356273.	-7838.	-8.90E-04	22255.	9.45E+09
-12.283	44291.	0.00				
11.5710	-0.00250	320011.	-7810.	-7.19E-04	21437.	9.45E+09
23.9840	45873.	0.00				
11.9700	-0.00556	284014.	-7620.	-5.66E-04	20625.	9.45E+09
55.0865	47455.	0.00				
12.3690	-0.00792	249027.	-7294.	-4.31E-04	19836.	9.45E+09
81.1498	49036.	0.00				
12.7680	-0.00968	215679.	-6855.	-3.13E-04	19084.	9.45E+09
102.3883	50618.	0.00				
13.1670	-0.01092	184486.	-6325.	-2.12E-04	18380.	9.45E+09
119.0854	52200.	0.00				
13.5660	-0.01171	155858.	-5725.	-1.26E-04	17734.	9.45E+09
131.5725	53782.	0.00				
13.9650	-0.01213	130108.	-5074.	-5.31E-05	17153.	9.45E+09
140.2091	55364.	0.00				
14.3640	-0.01222	107457.	-4526.	7.06E-06	16642.	9.45E+09

88.4822	34663.	0.00				
14.7630	-0.01206	86738.	-4100.	5.63E-05	16175.	9.45E+09
89.7186	35625.	0.00				
15.1620	-0.01168	67999.	-3671.	9.55E-05	15752.	9.45E+09
89.2807	36588.	0.00				
15.5610	-0.01114	51246.	-3248.	1.26E-04	15374.	9.45E+09
87.3982	37551.	0.00				
15.9600	-0.01048	36451.	-2837.	1.48E-04	15041.	9.45E+09
84.2985	38514.	0.00				
16.3590	-0.00973	23556.	-2443.	1.63E-04	14750.	9.45E+09
80.2025	39477.	0.00				
16.7580	-0.00892	12479.	-2071.	1.72E-04	14500.	9.45E+09
75.3212	40440.	0.00				
17.1570	-0.00808	3118.	-1724.	1.76E-04	14289.	9.45E+09
69.8525	41402.	0.00				
17.5560	-0.00723	-4645.	-1403.	1.76E-04	14323.	9.45E+09
63.9791	42365.	0.00				
17.9550	-0.00639	-10938.	-1112.	1.72E-04	14465.	9.45E+09
57.8669	43328.	0.00				
18.3540	-0.00559	-15893.	-849.310	1.65E-04	14577.	9.45E+09
51.6639	44291.	0.00				
18.7530	-0.00481	-19651.	-616.700	1.56E-04	14662.	9.45E+09
45.4997	45254.	0.00				
19.1520	-0.00409	-22347.	-413.245	1.45E-04	14723.	9.45E+09
39.4857	46217.	0.00				
19.5510	-0.00342	-24119.	-238.003	1.34E-04	14762.	9.45E+09
33.7150	47180.	0.00				
19.9500	-0.00281	-25096.	-89.625	1.21E-04	14785.	9.45E+09
28.2637	48142.	0.00				
20.3490	-0.00226	-25403.	33.5577	1.08E-04	14791.	9.45E+09
23.1912	49105.	0.00				
20.7480	-0.00177	-25156.	133.4676	9.56E-05	14786.	9.45E+09
18.5422	50068.	0.00				
21.1470	-0.00135	-24461.	212.2056	8.30E-05	14770.	9.45E+09
14.3475	51031.	0.00				
21.5460	-9.78E-04	-23415.	271.9907	7.09E-05	14747.	9.45E+09
10.6254	51994.	0.00				
21.9450	-6.68E-04	-22105.	339.5161	5.93E-05	14717.	9.45E+09
17.5807	126087.	0.00				
22.3440	-4.10E-04	-20372.	407.9458	4.86E-05	14678.	9.45E+09
11.0031	128380.	0.00				
22.7430	-2.03E-04	-18369.	447.5218	3.87E-05	14633.	9.45E+09
5.5282	130672.	0.00				
23.1420	-3.93E-05	-16223.	463.3710	3.00E-05	14584.	9.45E+09
1.0921	132965.	0.00				
23.5410	8.45E-05	-14037.	460.2677	2.23E-05	14535.	9.45E+09
-2.388	135257.	0.00				
23.9400	1.74E-04	-11894.	442.5584	1.57E-05	14487.	9.45E+09
-5.009	137550.	0.00				
24.3390	2.35E-04	-9855.	414.1134	1.02E-05	14441.	9.45E+09

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-6.873    139842.       0.00
   24.7380   2.72E-04     -7964.   378.3042   5.72E-06     14398.   9.45E+09
-8.085    142135.       0.00
   25.1370   2.90E-04     -6252.   338.0009   2.12E-06     14359.   9.45E+09
-8.750    144427.       0.00
   25.5360   2.93E-04     -4735.   295.5864  -6.67E-07     14325.   9.45E+09
-8.967    146720.       0.00
   25.9350   2.84E-04     -3419.   252.9830  -2.73E-06     14296.   9.45E+09
-8.829    149012.       0.00
   26.3340   2.66E-04     -2303.   211.6891  -4.18E-06     14270.   9.45E+09
-8.420    151305.       0.00
   26.7330   2.44E-04     -1378.   172.8216  -5.12E-06     14250.   9.45E+09
-7.815    153597.       0.00
   27.1320   2.17E-04   -630.080   137.1618  -5.62E-06     14233.   9.45E+09
-7.080    155890.       0.00
   27.5310   1.90E-04    -44.340   105.2036  -5.80E-06     14219.   9.45E+09
-6.269    158182.       0.00
   27.9300   1.62E-04   397.7178    77.1999  -5.71E-06     14227.   9.45E+09
-5.428    160475.       0.00
   28.3290   1.35E-04   714.9800    53.2082  -5.42E-06     14235.   9.45E+09
-4.593    162767.       0.00
   28.7280   1.10E-04   926.3021    33.1319  -5.01E-06     14239.   9.45E+09
-3.793    165060.       0.00
   29.1270   8.72E-05      1050.    16.7589  -4.51E-06     14242.   9.45E+09
-3.046    167352.       0.00
   29.5260   6.69E-05      1103.     3.7951  -3.96E-06     14243.   9.45E+09
-2.369    169645.       0.00
   29.9250   4.92E-05      1100.     -6.107  -3.40E-06     14243.   9.45E+09
-1.767    171937.       0.00
   30.3240   3.43E-05      1056.    -13.322  -2.86E-06     14242.   9.45E+09
-1.246    174230.       0.00
   30.7230   2.19E-05   982.5901    -18.235  -2.34E-06     14241.   9.45E+09
-0.806    176522.       0.00
   31.1220   1.18E-05   889.7229    -21.222  -1.87E-06     14239.   9.45E+09
-0.442    178815.       0.00
   31.5210   3.98E-06   785.9303    -22.640  -1.44E-06     14236.   9.45E+09
-0.150    181107.       0.00
   31.9200  -1.98E-06   677.9897    -22.819  -1.07E-06     14234.   9.45E+09
0.07566    183400.       0.00
   32.3190  -6.28E-06   571.1800    -22.055  -7.55E-07     14231.   9.45E+09
0.2436    185692.       0.00
   32.7180  -9.20E-06   469.4472    -20.606  -4.91E-07     14229.   9.45E+09
0.3613    187985.       0.00
   33.1170  -1.10E-05   375.5799    -18.696  -2.77E-07     14227.   9.45E+09
0.4366    190277.       0.00
   33.5160  -1.19E-05   291.3861    -16.510  -1.08E-07     14225.   9.45E+09
0.4768    192570.       0.00
   33.9150  -1.20E-05   217.8643    -14.197   2.10E-08     14223.   9.45E+09
0.4892    194862.       0.00
   34.3140  -1.17E-05   155.3626    -11.877   1.16E-07     14222.   9.45E+09
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0.4799    197155.       0.00
   34.7130  -1.09E-05   103.7243     -9.640   1.81E-07     14221.   9.45E+09
0.4546    199447.       0.00
   35.1120  -9.92E-06    62.4146     -7.551   2.23E-07     14220.   9.45E+09
0.4179    201740.       0.00
   35.5110  -8.77E-06    30.6306     -5.655   2.47E-07     14219.   9.45E+09
0.3739    204032.       0.00
   35.9100  -7.55E-06     7.3908     -3.981   2.57E-07     14219.   9.45E+09
0.3256    206324.       0.00
   36.3090  -6.32E-06     -8.392     -2.543   2.56E-07     14219.   9.45E+09
0.2753    208617.       0.00
   36.7080  -5.10E-06    -17.858     -1.346   2.50E-07     14219.   9.45E+09
0.2247    210909.       0.00
   37.1070  -3.93E-06    -22.156     -0.389   2.39E-07     14219.   9.45E+09
0.1749    213202.       0.00
   37.5060  -2.81E-06    -22.427     0.3319   2.28E-07     14219.   9.45E+09
0.1263    215494.       0.00
   37.9050  -1.74E-06    -19.780     0.8240   2.18E-07     14219.   9.45E+09
0.07923    217787.       0.00
   38.3040  -7.24E-07    -15.300     1.0934   2.09E-07     14219.   9.45E+09
0.03330    220079.       0.00
   38.7030   2.56E-07    -10.043     1.1447   2.02E-07     14219.   9.45E+09
-0.01188    222372.       0.00
   39.1020   1.21E-06     -5.049     0.9801   1.98E-07     14219.   9.45E+09
-0.05686    224664.       0.00
   39.5010   2.16E-06     -1.354     0.5994   1.97E-07     14218.   9.45E+09
-0.102    226957.       0.00
   39.9000   3.10E-06       0.00       0.00   1.96E-07     14218.   9.45E+09
-0.148    114625.       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.

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Output Summary for Load Case No. 1:

Pile-head deflection              =     0.39000000 inches
Computed slope at pile head       =       0.000000 radians
Maximum bending moment            =      -1693395. inch-lbs
Maximum shear force               =         36837. 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              =              7
Number of zero deflection points =              4
```

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.140000 inches
Rotation of pile head = 1.000E-02 radians
Axial load on pile head = 367000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.140	-2411462.	32286.	0.01000	68614.	8.10E+09	0.00	0.00	0.00
0.3990	-0.09553	-2272864.	32440.	0.00862	65488.	8.10E+09	35.4362	1776.	0.00
0.7980	-0.05750	-2131092.	32685.	0.00736	62290.	8.77E+09	66.7355	5557.	0.00
1.1970	-0.02503	-1985746.	32987.	0.00626	59011.	9.06E+09	59.3240	11348.	0.00
1.5960	0.00241	-1837196.	33111.	0.00526	55661.	9.28E+09	-7.611	15130.	0.00
1.9950	0.02531	-1687155.	32853.	0.00435	52276.	9.42E+09	-99.981	18913.	0.00
2.3940	0.04411	-1537899.	32169.	0.00354	48909.	9.45E+09	-185.762	20166.	0.00
2.7930	0.05917	-1391532.	31175.	0.00279	45608.	9.45E+09	-229.210	18547.	0.00
3.1920	0.07086	-1249180.	29999.	0.00212	42397.	9.45E+09	-262.140	17713.	0.00
3.5910	0.07951	-1111726.	28692.	0.00153	39296.	9.45E+09	-283.958	17099.	0.00
3.9900	0.08547	-979791.	27282.	9.96E-04	36320.	9.45E+09	-304.805	17074.	0.00
4.3890	0.08906	-853972.	25792.	5.32E-04	33482.	9.45E+09	-317.782	17085.	0.00
4.7880	0.09057	-734677.	24413.	1.29E-04	30791.	9.45E+09	-258.019	13641.	0.00
5.1870	0.09029	-620643.	23125.	-2.14E-04	28218.	9.45E+09	-280.320	14864.	0.00
5.5860	0.08852	-512483.	21713.	-5.01E-04	25779.	9.45E+09	-309.385	16735.	0.00
5.9850	0.08549	-410959.	20165.	-7.35E-04	23489.	9.45E+09			

-337.307	18891.	0.00				
6.3840	0.08147	-316802.	18489.	-9.20E-04	21365.	9.45E+09
-362.744	21317.	0.00				
6.7830	0.07669	-230679.	16698.	-0.00106	19422.	9.45E+09
-385.237	24052.	0.00				
7.1820	0.07134	-153182.	14808.	-0.00116	17674.	9.45E+09
-404.385	27141.	0.00				
7.5810	0.06562	-84819.	12854.	-0.00122	16132.	9.45E+09
-411.905	30055.	0.00				
7.9800	0.05970	-25823.	10923.	-0.00124	14801.	9.45E+09
-394.436	31636.	0.00				
8.3790	0.05371	24153.	9087.	-0.00124	14763.	9.45E+09
-372.618	33218.	0.00				
8.7780	0.04778	65566.	7363.	-0.00122	15697.	9.45E+09
-347.270	34800.	0.00				
9.1770	0.04201	98959.	5768.	-0.00118	16451.	9.45E+09
-319.213	36382.	0.00				
9.5760	0.03648	124945.	4311.	-0.00112	17037.	9.45E+09
-289.247	37964.	0.00				
9.9750	0.03125	144190.	3001.	-0.00106	17471.	9.45E+09
-258.131	39546.	0.00				
10.3740	0.02638	157389.	1840.	-9.79E-04	17769.	9.45E+09
-226.568	41127.	0.00				
10.7730	0.02188	165253.	830.7261	-8.97E-04	17946.	9.45E+09
-195.188	42709.	0.00				
11.1720	0.01779	168496.	-30.480	-8.12E-04	18019.	9.45E+09
-164.547	44291.	0.00				
11.5710	0.01410	167816.	-747.880	-7.27E-04	18004.	9.45E+09
-135.119	45873.	0.00				
11.9700	0.01083	163890.	-1328.	-6.43E-04	17915.	9.45E+09
-107.291	47455.	0.00				
12.3690	0.00795	157357.	-1780.	-5.62E-04	17768.	9.45E+09
-81.369	49036.	0.00				
12.7680	0.00545	148820.	-2113.	-4.84E-04	17575.	9.45E+09
-57.582	50618.	0.00				
13.1670	0.00331	138829.	-2337.	-4.11E-04	17350.	9.45E+09
-36.080	52200.	0.00				
13.5660	0.00151	127888.	-2464.	-3.44E-04	17103.	9.45E+09
-16.949	53782.	0.00				
13.9650	1.88E-05	116445.	-2505.	-2.82E-04	16845.	9.45E+09
-0.217	55364.	0.00				
14.3640	-0.00119	104892.	-2485.	-2.26E-04	16585.	9.45E+09
8.6067	34663.	0.00				
14.7630	-0.00214	93444.	-2426.	-1.75E-04	16326.	9.45E+09
15.9375	35625.	0.00				
15.1620	-0.00287	82278.	-2335.	-1.31E-04	16074.	9.45E+09
21.9191	36588.	0.00				
15.5610	-0.00340	71541.	-2219.	-9.19E-05	15832.	9.45E+09
26.6271	37551.	0.00				
15.9600	-0.00375	61351.	-2083.	-5.82E-05	15602.	9.45E+09

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30.1507     38514.       0.00
   16.3590   -0.00395     51797.     -1933.  -2.95E-05     15387.   9.45E+09
32.5891     39477.       0.00
   16.7580   -0.00403     42945.     -1773.  -5.54E-06     15187.   9.45E+09
34.0481     40440.       0.00
   17.1570   -0.00401     34835.     -1609.   1.42E-05     15004.   9.45E+09
34.6377     41402.       0.00
   17.5560   -0.00390     27487.     -1444.   3.00E-05     14838.   9.45E+09
34.4692     42365.       0.00
   17.9550   -0.00372     20906.     -1280.   4.22E-05     14690.   9.45E+09
33.6530     43328.       0.00
   18.3540   -0.00349     15077.     -1123.   5.13E-05     14559.   9.45E+09
32.2963     44291.       0.00
   18.7530   -0.00323      9975.   -972.249   5.77E-05     14443.   9.45E+09
30.5025     45254.       0.00
   19.1520   -0.00294      5564.   -831.311   6.16E-05     14344.   9.45E+09
28.3688     46217.       0.00
   19.5510   -0.00264      1798.   -701.186   6.35E-05     14259.   9.45E+09
25.9861     47180.       0.00
   19.9500   -0.00233     -1374.   -582.864   6.36E-05     14249.   9.45E+09
23.4382     48142.       0.00
   20.3490   -0.00203     -4007.   -476.955   6.22E-05     14309.   9.45E+09
20.8013     49105.       0.00
   20.7480   -0.00174     -6160.   -383.719   5.97E-05     14357.   9.45E+09
18.1443     50068.       0.00
   21.1470   -0.00146     -7891.   -303.106   5.61E-05     14396.   9.45E+09
15.5287     51031.       0.00
   21.5460   -0.00120     -9259.   -234.787   5.17E-05     14427.   9.45E+09
13.0091     51994.       0.00
   21.9450  -9.61E-04    -10321.   -143.029   4.68E-05     14451.   9.45E+09
25.3188    126087.       0.00
   22.3440  -7.50E-04    -10793.    -34.277   4.14E-05     14462.   9.45E+09
20.1084    128380.       0.00
   22.7430  -5.65E-04    -10795.    50.7549   3.60E-05     14462.   9.45E+09
15.4102    130672.       0.00
   23.1420  -4.06E-04    -10434.   114.6081   3.06E-05     14454.   9.45E+09
11.2619    132965.       0.00
   23.5410  -2.72E-04     -9805.   159.9466   2.55E-05     14440.   9.45E+09
7.6765    135257.       0.00
   23.9400  -1.62E-04     -8992.   189.4472   2.07E-05     14421.   9.45E+09
4.6463    137550.       0.00
   24.3390  -7.35E-05     -8064.   205.7126   1.64E-05     14400.   9.45E+09
2.1479    139842.       0.00
   24.7380  -4.92E-06     -7079.   211.2042   1.25E-05     14378.   9.45E+09
0.1460    142135.       0.00
   25.1370   4.65E-05     -6085.   208.1934   9.20E-06     14356.   9.45E+09
-1.404    144427.       0.00
   25.5360   8.32E-05     -5118.   198.7283   6.37E-06     14334.   9.45E+09
-2.550    146720.       0.00
   25.9350   1.07E-04     -4204.   184.6153   4.00E-06     14313.   9.45E+09
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-3.345    149012.       0.00
   26.3340   1.22E-04     -3364.   167.4116   2.09E-06     14294.   9.45E+09
-3.841    151305.       0.00
   26.7330   1.27E-04     -2609.   148.4279   5.72E-07     14277.   9.45E+09
-4.089    153597.       0.00
   27.1320   1.27E-04     -1945.   128.7386  -5.82E-07     14262.   9.45E+09
-4.136    155890.       0.00
   27.5310   1.22E-04     -1374.   109.1978  -1.42E-06     14249.   9.45E+09
-4.027    158182.       0.00
   27.9300   1.13E-04   -894.045    90.4588  -2.00E-06     14239.   9.45E+09
-3.801    160475.       0.00
   28.3290   1.03E-04   -500.588    72.9970  -2.35E-06     14230.   9.45E+09
-3.493    162767.       0.00
   28.7280   9.09E-05   -186.765    57.1328  -2.52E-06     14223.   9.45E+09
-3.133    165060.       0.00
   29.1270   7.86E-05    55.3894    43.0562  -2.56E-06     14220.   9.45E+09
-2.747    167352.       0.00
   29.5260   6.64E-05   234.5309    30.8490  -2.48E-06     14224.   9.45E+09
-2.353    169645.       0.00
   29.9250   5.48E-05   359.5313    20.5071  -2.33E-06     14227.   9.45E+09
-1.967    171937.       0.00
   30.3240   4.40E-05   439.1097    11.9601  -2.13E-06     14228.   9.45E+09
-1.603    174230.       0.00
   30.7230   3.44E-05   481.5528     5.0892  -1.90E-06     14229.   9.45E+09
-1.267    176522.       0.00
   31.1220   2.59E-05   494.5156     -0.257  -1.65E-06     14230.   9.45E+09
-0.966    178815.       0.00
   31.5210   1.86E-05   484.8917     -4.251  -1.40E-06     14229.   9.45E+09
-0.702    181107.       0.00
   31.9200   1.24E-05   458.7406     -7.072  -1.16E-06     14229.   9.45E+09
-0.476    183400.       0.00
   32.3190   7.42E-06   421.2642     -8.900  -9.41E-07     14228.   9.45E+09
-0.288    185692.       0.00
   32.7180   3.42E-06   376.8192     -9.910  -7.39E-07     14227.   9.45E+09
-0.134    187985.       0.00
   33.1170   3.42E-07   328.9588    -10.265  -5.60E-07     14226.   9.45E+09
-0.01360    190277.       0.00
   33.5160  -1.94E-06   280.4937    -10.110  -4.05E-07     14225.   9.45E+09
0.07800    192570.       0.00
   33.9150  -3.54E-06   233.5671     -9.579  -2.75E-07     14224.   9.45E+09
0.1441    194862.       0.00
   34.3140  -4.57E-06   189.7359     -8.783  -1.68E-07     14223.   9.45E+09
0.1884    197155.       0.00
   34.7130  -5.15E-06   150.0542     -7.818  -8.19E-08     14222.   9.45E+09
0.2145    199447.       0.00
   35.1120  -5.36E-06   115.1558     -6.764  -1.47E-08     14221.   9.45E+09
0.2258    201740.       0.00
   35.5110  -5.29E-06    85.3310     -5.684   3.61E-08     14220.   9.45E+09
0.2254    204032.       0.00
   35.9100  -5.01E-06    60.5972     -4.627   7.31E-08     14220.   9.45E+09
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0.2160    206324.       0.00
   36.3090  -4.59E-06    40.7612     -3.632   9.88E-08     14219.   9.45E+09
0.1999    208617.       0.00
   36.7080  -4.07E-06    25.4727     -2.724   1.16E-07     14219.   9.45E+09
0.1791    210909.       0.00
   37.1070  -3.48E-06    14.2680     -1.924   1.26E-07     14219.   9.45E+09
0.1551    213202.       0.00
   37.5060  -2.86E-06     6.6052     -1.244   1.31E-07     14219.   9.45E+09
0.1289    215494.       0.00
   37.9050  -2.23E-06     1.8908     -0.693   1.33E-07     14218.   9.45E+09
0.1014    217787.       0.00
   38.3040  -1.59E-06     -0.502     -0.276   1.33E-07     14218.   9.45E+09
0.07303    220079.       0.00
   38.7030  -9.50E-07     -1.220    0.00467   1.33E-07     14218.   9.45E+09
0.04414    222372.       0.00
   39.1020  -3.15E-07     -0.924     0.1457   1.32E-07     14218.   9.45E+09
0.01478    224664.       0.00
   39.5010   3.18E-07     -0.290     0.1450   1.32E-07     14218.   9.45E+09
-0.01508    226957.       0.00
   39.9000   9.50E-07       0.00       0.00   1.32E-07     14218.   9.45E+09
-0.04551    114625.       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. 2:

```
Pile-head deflection             =     -0.1400000 inches
Computed slope at pile head      =     0.01000000 radians
Maximum bending moment           =      -2411462. inch-lbs
Maximum shear force              =         33111. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =     1.59600000 feet below pile head
Number of iterations             =              7
Number of zero deflection points =              5
```

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 3

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head   =   -0.140000 inches

```
Moment at pile head                                        =   -1632757.0 in-lbs
Axial load at pile head                                    =     367000.0 lbs
```

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.140	-1632757.	19622.	0.00714	51049.	9.44E+09	0.00	0.00	0.00
0.3990	-0.108	-1550633.	19709.	0.00634	49196.	9.44E+09	36.6272	1627.	0.00
0.7980	-0.07933	-1466287.	19972.	0.00557	47294.	9.45E+09	73.1995	4418.	0.00
1.1970	-0.05443	-1378958.	20394.	0.00485	45324.	9.45E+09	102.8577	9048.	0.00
1.5960	-0.03288	-1288043.	20889.	0.00417	43273.	9.45E+09	103.9061	15130.	0.00
1.9950	-0.01446	-1193599.	21274.	0.00355	41143.	9.45E+09	57.1015	18913.	0.00
2.3940	0.00107	-1096783.	21399.	0.00297	38959.	9.45E+09	-5.088	22696.	0.00
2.7930	0.01394	-999107.	21202.	0.00243	36756.	9.45E+09	-77.097	26478.	0.00
3.1920	0.02438	-902308.	20648.	0.00195	34572.	9.45E+09	-154.117	30261.	0.00
3.5910	0.03264	-808239.	19797.	0.00152	32450.	9.45E+09	-201.330	29534.	0.00
3.9900	0.03893	-718067.	18794.	0.00113	30416.	9.45E+09	-217.696	26773.	0.00
4.3890	0.04348	-632245.	17733.	7.90E-04	28480.	9.45E+09	-225.569	24837.	0.00
4.7880	0.04650	-551031.	16776.	4.91E-04	26648.	9.45E+09	-174.135	17930.	0.00
5.1870	0.04818	-473319.	15901.	2.31E-04	24895.	9.45E+09	-191.510	19031.	0.00
5.5860	0.04871	-399576.	14927.	9.79E-06	23232.	9.45E+09	-215.184	21151.	0.00
5.9850	0.04827	-330410.	13842.	-1.75E-04	21672.	9.45E+09	-238.299	23635.	0.00
6.3840	0.04704	-266413.	12676.	-3.26E-04	20228.	9.45E+09	-248.625	25309.	0.00
6.7830	0.04515	-207878.	11474.	-4.47E-04	18908.	9.45E+09	-253.572	26891.	0.00
7.1820	0.04276	-154971.	10258.	-5.39E-04	17714.	9.45E+09	-254.274	28473.	0.00
7.5810	0.03999	-107756.	9048.	-6.05E-04	16649.	9.45E+09			

-251.035	30055.	0.00				
7.9800	0.03696	-66199.	7862.	-6.49E-04	15712.	9.45E+09
-244.242	31636.	0.00				
8.3790	0.03378	-30183.	6717.	-6.74E-04	14899.	9.45E+09
-234.333	33218.	0.00				
8.7780	0.03051	487.7447	5625.	-6.81E-04	14229.	9.45E+09
-221.786	34800.	0.00				
9.1770	0.02725	26074.	4598.	-6.74E-04	14807.	9.45E+09
-207.092	36382.	0.00				
9.5760	0.02406	46889.	3646.	-6.56E-04	15276.	9.45E+09
-190.745	37964.	0.00				
9.9750	0.02097	63290.	2774.	-6.28E-04	15646.	9.45E+09
-173.226	39546.	0.00				
10.3740	0.01804	75663.	1989.	-5.93E-04	15925.	9.45E+09
-154.988	41127.	0.00				
10.7730	0.01530	84416.	1291.	-5.52E-04	16123.	9.45E+09
-136.452	42709.	0.00				
11.1720	0.01276	89965.	681.7361	-5.08E-04	16248.	9.45E+09
-117.996	44291.	0.00				
11.5710	0.01043	92730.	159.9668	-4.62E-04	16310.	9.45E+09
-99.952	45873.	0.00				
11.9700	0.00833	93120.	-277.072	-4.15E-04	16319.	9.45E+09
-82.604	47455.	0.00				
12.3690	0.00646	91533.	-633.265	-3.68E-04	16283.	9.45E+09
-66.182	49036.	0.00				
12.7680	0.00481	88348.	-913.492	-3.22E-04	16211.	9.45E+09
-50.872	50618.	0.00				
13.1670	0.00338	83918.	-1123.	-2.79E-04	16111.	9.45E+09
-36.808	52200.	0.00				
13.5660	0.00214	78570.	-1269.	-2.37E-04	15991.	9.45E+09
-24.082	53782.	0.00				
13.9650	0.00110	72599.	-1357.	-1.99E-04	15856.	9.45E+09
-12.747	55364.	0.00				
14.3640	2.37E-04	66272.	-1392.	-1.64E-04	15713.	9.45E+09
-1.716	34663.	0.00				
14.7630	-4.68E-04	59846.	-1388.	-1.32E-04	15568.	9.45E+09
3.4792	35625.	0.00				
15.1620	-0.00103	53447.	-1361.	-1.03E-04	15424.	9.45E+09
7.8480	36588.	0.00				
15.5610	-0.00146	47180.	-1314.	-7.78E-05	15283.	9.45E+09
11.4247	37551.	0.00				
15.9600	-0.00177	41133.	-1253.	-5.54E-05	15146.	9.45E+09
14.2534	38514.	0.00				
16.3590	-0.00199	35376.	-1180.	-3.60E-05	15016.	9.45E+09
16.3861	39477.	0.00				
16.7580	-0.00212	29963.	-1098.	-1.95E-05	14894.	9.45E+09
17.8804	40440.	0.00				
17.1570	-0.00217	24933.	-1010.	-5.57E-06	14781.	9.45E+09
18.7983	41402.	0.00				
17.5560	-0.00217	20312.	-918.841	5.90E-06	14677.	9.45E+09

```
19.2037     42365.       0.00
   17.9550   -0.00212     16114.   -826.994   1.51E-05     14582.   9.45E+09
19.1618     43328.       0.00
   18.3540   -0.00203     12340.   -736.265   2.23E-05     14497.   9.45E+09
18.7369     44291.       0.00
   18.7530   -0.00190      8985.   -648.336   2.77E-05     14421.   9.45E+09
17.9920     45254.       0.00
   19.1520   -0.00176      6034.   -564.595   3.15E-05     14355.   9.45E+09
16.9876     46217.       0.00
   19.5510   -0.00160      3467.   -486.146   3.40E-05     14297.   9.45E+09
15.7812     47180.       0.00
   19.9500   -0.00143      1259.   -413.829   3.51E-05     14247.   9.45E+09
14.4266     48142.       0.00
   20.3490   -0.00126   -619.024   -348.233   3.53E-05     14232.   9.45E+09
12.9735     49105.       0.00
   20.7480   -0.00110     -2199.   -289.721   3.46E-05     14268.   9.45E+09
11.4678     50068.       0.00
   21.1470  -9.34E-04     -3515.   -238.443   3.31E-05     14298.   9.45E+09
9.9513     51031.       0.00
   21.5460  -7.79E-04     -4599.   -194.362   3.11E-05     14322.   9.45E+09
8.4619     51994.       0.00
   21.9450  -6.36E-04     -5485.   -134.012   2.85E-05     14342.   9.45E+09
16.7471    126087.       0.00
   22.3440  -5.06E-04     -5983.    -61.441   2.56E-05     14353.   9.45E+09
13.5664    128380.       0.00
   22.7430  -3.91E-04     -6164.     -3.449   2.26E-05     14357.   9.45E+09
10.6575    130672.       0.00
   23.1420  -2.90E-04     -6095.    41.3443   1.94E-05     14356.   9.45E+09
8.0533    132965.       0.00
   23.5410  -2.04E-04     -5836.    74.4390   1.64E-05     14350.   9.45E+09
5.7707    135257.       0.00
   23.9400  -1.33E-04     -5440.    97.3818   1.36E-05     14341.   9.45E+09
3.8128    137550.       0.00
   24.3390  -7.44E-05     -4951.   111.7089   1.09E-05     14330.   9.45E+09
2.1718    139842.       0.00
   24.7380  -2.80E-05     -4409.   118.8993   8.56E-06     14318.   9.45E+09
0.8317    142135.       0.00
   25.1370   7.63E-06     -3843.   120.3391   6.47E-06     14305.   9.45E+09
-0.230    144427.       0.00
   25.5360   3.40E-05     -3279.   117.2970   4.67E-06     14292.   9.45E+09
-1.041    146720.       0.00
   25.9350   5.23E-05     -2736.   110.9076   3.14E-06     14280.   9.45E+09
-1.628    149012.       0.00
   26.3340   6.41E-05     -2228.   102.1636   1.89E-06     14269.   9.45E+09
-2.024    151305.       0.00
   26.7330   7.04E-05     -1764.    91.9133   8.73E-07     14258.   9.45E+09
-2.258    153597.       0.00
   27.1320   7.24E-05     -1351.    80.8643   8.40E-08     14249.   9.45E+09
-2.358    155890.       0.00
   27.5310   7.12E-05   -990.404    69.5904  -5.09E-07     14241.   9.45E+09
```

```
-2.352    158182.       0.00
   27.9300   6.75E-05   -682.823    58.5418  -9.33E-07     14234.   9.45E+09
-2.264    160475.       0.00
   28.3290   6.22E-05   -426.528    48.0573  -1.21E-06     14228.   9.45E+09
-2.116    162767.       0.00
   28.7280   5.59E-05   -218.359    38.3776  -1.38E-06     14223.   9.45E+09
-1.927    165060.       0.00
   29.1270   4.90E-05    -54.182    29.6592  -1.45E-06     14220.   9.45E+09
-1.714    167352.       0.00
   29.5260   4.21E-05    70.7416    21.9877  -1.44E-06     14220.   9.45E+09
-1.490    169645.       0.00
   29.9250   3.52E-05   161.4421    15.3914  -1.38E-06     14222.   9.45E+09
-1.265    171937.       0.00
   30.3240   2.88E-05   222.9925     9.8530  -1.29E-06     14223.   9.45E+09
-1.048    174230.       0.00
   30.7230   2.29E-05   260.3148     5.3210  -1.16E-06     14224.   9.45E+09
-0.845    176522.       0.00
   31.1220   1.77E-05   278.0365     1.7194  -1.03E-06     14225.   9.45E+09
-0.660    178815.       0.00
   31.5210   1.31E-05   280.3908     -1.044  -8.86E-07     14225.   9.45E+09
-0.495    181107.       0.00
   31.9200   9.18E-06   271.1554     -3.069  -7.46E-07     14225.   9.45E+09
-0.351    183400.       0.00
   32.3190   5.93E-06   253.6211     -4.462  -6.13E-07     14224.   9.45E+09
-0.230    185692.       0.00
   32.7180   3.30E-06   230.5870     -5.323  -4.91E-07     14224.   9.45E+09
-0.130    187985.       0.00
   33.1170   1.23E-06   204.3746     -5.751  -3.80E-07     14223.   9.45E+09
-0.04904    190277.       0.00
   33.5160  -3.39E-07   176.8559     -5.835  -2.84E-07     14222.   9.45E+09
0.01364    192570.       0.00
   33.9150  -1.48E-06   149.4924     -5.658  -2.01E-07     14222.   9.45E+09
0.06037    194862.       0.00
   34.3140  -2.26E-06   123.3798     -5.290  -1.32E-07     14221.   9.45E+09
0.09325    197155.       0.00
   34.7130  -2.75E-06    99.2951     -4.793  -7.55E-08     14221.   9.45E+09
0.1144    199447.       0.00
   35.1120  -2.99E-06    77.7451     -4.218  -3.07E-08     14220.   9.45E+09
0.1259    201740.       0.00
   35.5110  -3.04E-06    59.0120     -3.606   3.99E-09     14220.   9.45E+09
0.1296    204032.       0.00
   35.9100  -2.95E-06    43.1965     -2.992   2.99E-08     14219.   9.45E+09
0.1271    206324.       0.00
   36.3090  -2.75E-06    30.2565     -2.400   4.85E-08     14219.   9.45E+09
0.1200    208617.       0.00
   36.7080  -2.49E-06    20.0407     -1.851   6.12E-08     14219.   9.45E+09
0.1095    210909.       0.00
   37.1070  -2.17E-06    12.3167     -1.358   6.94E-08     14219.   9.45E+09
0.09653    213202.       0.00
   37.5060  -1.82E-06     6.7947     -0.931   7.43E-08     14219.   9.45E+09
```

```
0.08193    215494.       0.00
   37.9050  -1.46E-06     3.1448     -0.576   7.68E-08     14219.   9.45E+09
0.06625    217787.       0.00
   38.3040  -1.08E-06     1.0108     -0.298   7.79E-08     14218.   9.45E+09
0.04987    220079.       0.00
   38.7030  -7.11E-07    0.01908   -0.09941   7.81E-08     14218.   9.45E+09
0.03302    222372.       0.00
   39.1020  -3.37E-07     -0.216    0.01747   7.81E-08     14218.   9.45E+09
0.01581    224664.       0.00
   39.5010   3.67E-08   -0.08799    0.05115   7.80E-08     14218.   9.45E+09
-0.00174    226957.       0.00
   39.9000   4.10E-07       0.00       0.00   7.80E-08     14218.   9.45E+09
-0.01963    114625.       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. 3:

Pile-head deflection               =     -0.1400000 inches
Computed slope at pile head        =     0.00714307 radians
Maximum bending moment             =      -1632757. inch-lbs
Maximum shear force                =         21399. lbs
Depth of maximum bending moment    =       0.000000 feet below pile head
Depth of maximum shear force       =     2.39400000 feet below pile head
Number of iterations               =              6
Number of zero deflection points =              5


--------------------------------------------------------------------------------
            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 Load                Load                  Axial    Pile-head  Pile-head  Max
Shear Max Moment
Case Type   Pile-head    Type     Pile-head   Loading  Deflection  Rotation    in
```

```
Pile     in Pile
 No.  1      Load 1       2        Load 2       lbs       inches     radians      lbs
     in-lbs
---- ----- ---------- ---------- ---------- ---------- ---------- ----------
---------- ----------
  1  y, in     0.3900   S, rad         0.00    367000.     0.3900       0.00
36837.  -1693395.
  2  y, in     -0.140   S, rad      0.01000    367000.     -0.140    0.01000
33111.  -2411462.
  3  y, in     -0.140  M, in-lb   -1632757.    367000.     -0.140    0.00714
21399.  -1632757.


Maximum pile-head deflection = 0.3900000000 inches
Maximum pile-head rotation   = 0.0100000000 radians = 0.572958 deg.

The analysis ended normally.
```

CALCULATIONS

Date:	4/10/2026	**Made by:**	DEB
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 2 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

OBJECTIVE

Determine if the proposed HP14x89 piles will provide adequate support for Abutment No. 2 of the proposed Smith Brook Bridge (Bridge No. 3189) based on the anticipated thermal movement, rotation, scour conditions, and axial loads for the Strength I Load Combination.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Ed, 2024

2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

3. Hoyle Tanner, Bancroft TWP, Aroostook County, Smith Brook Bridge over Smith Brook, Bancroft Road, Federal Aid Project No. 2826200, Bridge 3189. 60% Plans dated February 5. 2026.

4. WSP Interpretive Subsurface Profile, included as Sheet 3 of the Geotechnical Design Report (GDR).

5. FHWA. July 2016. GEC-012: Design and Construction of Driven Pile Foundations, Volume 1. Publication No. FHWA-NHI-16-009.

6. Email from Hoyle Tanner Structural team received on March 13, 2026 with subject line "RE: [External] NBB-Geotech-Ext: Bridge 3189 updated pile loads".

7. Hoyle Tanner, Abutment Loading Calculations, provided in "Integral Abutment Pile Loads_2025_07_24.pdf", received by WSP via email on July 24th, 2025.

8. WSP Laboratory Testing Results, included as Appendix C of the GDR.

9. WSP, Material properties table named "NBB Bridge 3189 Geotechnical Materials Properties.xlsx"

10. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2025. https://bsi.ce.ufl.edu/documentation/fbmp/manual

11. WSP USA Inc. Table 4: Summary of Rock Laboratory Testing Results, included in the GDR.

12. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

13. AISC Steel Construction Manual, 13th Ed. 2005.

14. WSP Boring Logs, included as Appendix A of the Geotechnical Design Report.

15. VTrans Structures Section, Integral Abutment Committee. 2008. Integral Abutment Bridge Design Guidelines, 2nd Ed.

16. FHWA, September 2018, GEC-010: Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024.

17. Hoyle, Tanner Northern Bridge Bundle-Bancroft TWP Bridge No. 3189 Scour Calculations - Proposed Conditions dated 08/2025.

18. Preliminary Design Report, Smith Brook Bridge # 3189 over Smith Brook Bancroft TWP, Maine WIN 28262.00.

CALCULATIONS

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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on five HP14x89 Grade 50 piles with a center-to-center pile spacing of 7.715 feet, equal to the proposed bridge girder spacing adjusted for the abutment skew from Ref. 3, Sheet 28 and Ref. 7 Page 10. The piles are assumed to be driven to bedrock and thus end-bearing.

2. The selected pile orientation is weak axis bending (Ref. 2, page 5-42).

3. The vertical load is assumed to be evenly distributed.

4. The soil profile used in the analysis is based on Ref. 4 at the centerline of the proposed Abutment No. 1.

5. An UCS for rock of 7,000 psi is assumed based on a field characterization of R3 (Medium Strong Rock) to R4 (Strong Rock) from Ref. 14.

6. Based on previous experience on nearby projects and the bridge being located within a no salt zone, section loss due to corrosion is not considered for this analysis.

7. Depth to fixity for deflection is taken as the depth along the pile to the point of zero lateral deflection; zero deflection is assumed to be deflection values less than 0.001 inches (Ref. 2 Section 5.4.2.5). Depth to fixity for bending moment is taken as the depth along the pile where the bending moment is less than 5% of the maximum bending moment (Ref. 16 page 12-19).

8. The piles are modeled in L-Pile as a fixed connection at the top of the pile.

9. It is assumed the piles will have an unbraced length to the bottom depth of the scour pool located down stream of the proposed abutments at EL. 318.0 feet to account for scour at the bridge abutments (Ref. 17, sheet 17 of 40). The maximum calculated scour depths identified in the project PDR (Ref. 18) are 29.0' for Q100 and 33.6' for Q500 for the streambed, as determined from stream bed samples alone. These depths were determined to be impractical for evaluation of the potential scour depth, and therefore the depth of the scour hole measured during the project survey was instead used for this evaluation.

10. In the fully scoured condition, we expect that the piles would experience axial loading with possible moments and rotations due to thermal expansion and contraction. In this condition, it is likely that the soil behnd the abutment would be scoured away to some degree or entirely. However, the loading provided for the abutment/piles for the fully intact soil and embankment condition is used as an upper bound for the pile design in the scoured condition, where only the piles have an unsupported length.

CALCULATIONS

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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

ATTACHMENTS

1. LPile analysis output for Strength I

CALCULATION

1. Select the pile section parameters

	Intact Section (Ref. 12)
Pile depth, d =	13.8
Pile width, b_f =	14.7
Web thickness, t_w =	0.615
Flange thickness, t_f =	0.615
Fillet area =	0.3
Section area =	26.1
Moment of inertia about the x-axis, I_x =	888
Moment of inertia about the y-axis, I_y =	326
Radius of gyration about the x-axis, r_x =	5.83
Radius of gyration about the y-axis, r_y =	3.53
Elastic section modulus about the x-axis, S_x =	129
Elastic section modulus about the y-axis, S_y =	44.3
Plastic section modulus about the x-axis, Z_x =	143
Plastic section modulus about the y-axis, Z_y =	67.6

2. Determine pile loads and select the preliminary pile size.

Determine the factored substructure vertical dead and live load (P_u) distributed to each pile.

Strength I factored vertical load per pile = 363.0 kips (Ref. 6)

Pile weight = 0.089 kip/ft x 39.9 ft = 3.6 kips

P_u = 367 kips (Total factored axial load including pile weight.)

Select the steel pile strength.

F_y = 50 ksi (Assumption 1)

E = 29,000 ksi (typical value for steel)

CALCULATIONS

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Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} =	0.50	for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)
ϕ_{cu} =	0.70	for axial resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)
ϕ_f =	1.00	for flexural resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 524 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 733 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 733 kips

Use the required nominal axial pile resistance to estimate an approximate initial pile area.

$$A_{s.req} = \frac{R_n}{0.80\ Fy}$$ (Ref. 2, section 5-42)

$A_{s.req}$ = 18.3 in^2

Select a pile size with an area of approximately $A_{s.req}$.

HP14x89 section as per Assumption 1 and Step 1:

A_s = 26.1 in^2 (Step 1)

CALCULATIONS

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3. Use LPile analysis to determine the pile unbraced length and maximum moment at the top of the pile.

The following input parameters were used in the LPile analysis:

Pile Properties

Section type:	Steel H Section Weak Axis		(Assumption 2)	
Length of section:	39.9	ft	(piles driven to bedrock)	
Flange width, b_f:	14.7	in	(Step 1)	
Section depth, d:	13.8	in	(Step 1)	
Flange thickness, t_f:	0.615	in	(Step 1)	
Web thickness, t_w:	0.615	in	(Step 1)	
Moment of inertia, I_y:	326	in^4	(Step 1)	about the weak axis, since weak-axis bending
Plastic section modulus, Z_y:	67.6	in^3	(Step 1)	about the weak axis, since weak-axis bending
Elastic section modulus, S_y:	44.3	in^3	(Step 1)	about the weak axis, since weak-axis bending
Radius of gyration, r_y:	3.53	in	(Step 1)	about the weak axis, since weak-axis bending
Pile batter:	Vertical		(pile battering not required)	

Pile Loading

Case	ΔT (in) [1]	Rotation (rad)[2]	Axial Load (lbs)[3]
1 (Thermal contraction)	0.39	0.000	367,000
2 (Thermal expansion)	-0.14	0.010	367,000

1) Strength I Lateral deflection due to abutment thermal expansion or contraction per Ref. 6
2) Assumed Rotations per Ref. 6.
3). From Step 2

CALCULATIONS

Date:	4/10/2026	**Made by:**	DEB
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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Soil Layers

Layer	Depth below base of abutment[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Undrained Shear Strength (psf)	Friction Angle (°)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Unsupported length - Scour[5]	0.0 - 22.2 ft	-	-	-	-	-	-	-
Glacial Till	22.2 - 39.9 ft	Sand (Reese)	67.6	-	36	100.0	-	-
Bedrock	> 39.9 ft	Strong Rock (Vuggy Limestone)	102.6	-	-	-	-	7,000

1) Per Ref. 3, 4, 14, and 17.
2) Ref. 9
3) Ref. 10 - interpolation based on friction angle for subgrade modulus
4) Assumption 5
5) Assumption 9 & 10.

The full LPile output is provided in Attachment 1.

Obtain the maximum moment in the top segment of the pile using the loading case that produces the highest moment.

The thermal expansion load case (Load case 2), resulted in the highest moment in the pile.

$M_{u.Top}$ = 835 in-kips (from LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 20.0 ft (from LPile)
$l_{b.top}$ = 239.6 in

$l_{b.2nd}$ = 16.2 ft (from LPile)
$l_{b.2nd}$ = 194.5 in

CALCULATIONS

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4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the interaction of combined axial and flexural load effects on a single pile.

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each segment.

For the top segment (rotation fixed with no plastic hinge):

$$\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120 \qquad \text{(Ref. 1, Article 6.9.3)}$$

where:

K_{top} = 1.2 (Ref. 1, Table C4.6.2.5-1)

r_y = 3.53 in (Step 1)

λ_{top} = 81.5 OK

For the second segment (pinned at top and bottom):

$$\lambda_{2nd} = \frac{K_{2nd} l_{b.2nd}}{r_y} \leq 120 \qquad \text{(Ref. 1, Article 6.9.3)}$$

where:

K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{2nd} = 55.1 OK

Calculate the critical elastic buckling resistance, P_e, and the nominal yield resistance, P_o.

Use Ref. 1 Table 6.9.4.1.1-1 to select equation for P_e based on cross-section shape and potential buckling mode.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\, l_b}{r_y}\right)^2} A_s \qquad \text{(Ref. 1, Eqn 6.9.4.1.2-1)}$$

$P_{e.top}$ = 1126 kips

$P_{e.2nd}$ = 2460 kips

$$P_o = F_y A_s \qquad \text{(Ref. 1, Article 6.9.4.1)}$$

P_o = 1305 kips

CALCULATIONS

Date:	4/10/2026	**Made by:**	DEB
Project No.:	US0040947.9162 / WIN 028262.00	**Checked by:**	ATM
Subject:	HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 2 with Scour	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Calculate the nominal structural pile resistance, P_n, for both segments of the upper zone of the pile as well as the lower zone of the pile.

Determine P_o/P_e to select equation for P_n as per Ref. 1 Article 6.9.4.1.

$P_o/P_{e.top}$ = 1.16 ≤ 2.25
$P_o/P_{e.2nd}$ = 0.53 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 803 kips
$P_{n.2nd}$ = 1045 kips

$$P_{n.bottom} = \left(0.658^{(0)}\right) P_o$$

(0 for a fully braced pile - Ref. 15, Appendix B, Eqn 6-9)

$P_{n.bottom}$ = 1305 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

$$P_{r.top} = \phi_{cu} P_{n.top}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.top}$ = 562 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.2nd}$ = 732 kips

$$P_{r.bottom} = \phi_{cl} P_{n.bottom}$$ (Ref. 1, Eqn. 6.9.2.1-1)

$P_{r.bottom}$ = 653 kips

Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20 (Ref. 2, page 5-43).

$$\frac{P_u}{P_{r.top}} = 0.65$$ OK

$$\frac{P_u}{P_{r.2nd}} = 0.50$$ OK

CALCULATIONS

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Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads (Ref. 15, Appendix B Eqn 6-17). Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check}\left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$$\frac{P_u}{P_{r.bottom}} = \quad 0.56 \qquad \text{OK}$$

Determine the nominal and factored flexural resistance about the H-pile weak axis.

Slenderness ratio for the flange:

$$\lambda_f = \frac{b_f}{2t_f} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1c-3)}$$

$\lambda_f =$ 12.0

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1c-4)}$$

$\lambda_{pf} =$ 9.2

Limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1c-5)}$$

$\lambda r_f =$ 24.1

Since $\lambda pf < \lambda f \leq \lambda rf$, pile is noncompact per Ref. 1, Article 6.12.2.2.2c-12.

Elastic and plastic section moduli about the weak axis:

$S_y =$	44.3	in^3	(Step 1)
$Z_y =$	67.6	in^3	(Step 1)

CALCULATIONS

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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

Per Ref.1 section 6.12.2.2.1 the nominal flexural resistance about the weak axis shall be taken as the smaller value based on yielding or FLB (Flange Local Buckling).

Nominal flexural resistance based on yielding:

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad \text{, if } \lambda_f \leq \lambda_{pf}$$ (Ref. 1, Eqn 6.12.2.2.1b-1)

Nominal flexural resistance based on FLB:

Since λpf < λf ≤ λrf,

$$M_n = M_p - \left(M_p - 0.7\, F_{yf}\, S_y\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \text{ if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-1)

$$M_n = \frac{0.69 E S_y}{\lambda_f^2} \qquad \text{if } \lambda_f > \lambda_{rf}$$ (Ref. 1, Eqn 6.12.2.2.1c-2)

For welded sections Fyf shall be taken as 0.5Fyc, where Fyc is the minimum yield stress of the compression flange.

M_p =	3380	in-kips
M_n =	2892	in-kips

Factored flexural resistance:

ϕ_f = 1.00 (Ref. 2, page 5-42)

$$M_r = \phi_f M_n$$

M_r = 2892 in-kips

For all members (including noncompact):

$$M_p' = \left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$ (Ref. 1, Eqn 6.9.2.2.1-3 and Ref. 15 Eq 4.5.2-3)

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'.

M_p' = 1007 in-kips = 1006639 inch-lb

If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation, and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top}$ =	835	in-kips	(From Step 3)
$M_{u.Top}$	<	M_p'	No Plastic Hinge Forms

CALCULATIONS

Date:	4/10/2026	**Made by:**	DEB
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Check the second segment of the upper zone of the pile to ensure that this segment of the pile does not form a plastic hinge.

$M_{u.2nd}$ = 295 in-kips (LPile)

Check: $$\frac{P_u}{P_{r.2nd}} + \left(\frac{M_{u.2nd}}{M_r}\right) < 1$$ (Ref. 1, Eqn 6.9.2.2.1-3 for noncompact member, Ref. 15, Eqn 4.5.2-3)

Check: 0.60 < 1 OK

5. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPile output against the structural shear resistance per AISC G7.

V_u = 4.1 kips (from LPile)

AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition, Specifications G2 and G7 (Ref. 13) to ensure the pile will not shear under the longitudinal load.

Per Ref. 13 Section G7: for singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength V_n for each shear resisting element shall be determined using Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ and $k_v = 1.2$.

$$V_n = 0.6F_y A_w C_v$$ (Ref. 13, Eqn G2-1)

where:

C_v = 1.0 for HP shapes with $F_y \leq 50$ ksi (Ref. 13, Section G7)

$A_w = b_f t_f$ = 9.0 in^2 (Ref. 13, Section G7)

Since there are two shear resisting elements (i.e., two flanges):

$2 \times A_w$ = 18.1 in^2

V_n = 542 kips

$$V_r = \Phi_v V_n$$

ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)

V_r = 542 kips

Check that the shear resistance is sufficient:

V_u < V_r OK

CALCULATIONS

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Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME		

6. Check that the maximum factored applied pile load does not exceed the factored pile drivability resistance.

For steel piles in compression, driving stresses should not exceed 90% of the yield strength of the pile material (Ref. 2, page 5-104).

σ_{dr} = 0.9 × 50 ksi = 45 ksi

This translates into an ultimate maximum driving force that can be applied to the pile of:

$$P_{dr} = \sigma_{dr} A_s$$ (Ref. 15, Appendix B, Eqn 7-23)

P_{dr} = 1175 kips

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. In this design, the pile will be bearing on rock. The driving criteria will be established by dynamic testing.

ϕ_{dyn} = 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{dyn}}$$ (Ref. 2, Appendix B, Eqn 7-25)

R_{ndr} = 564 kips

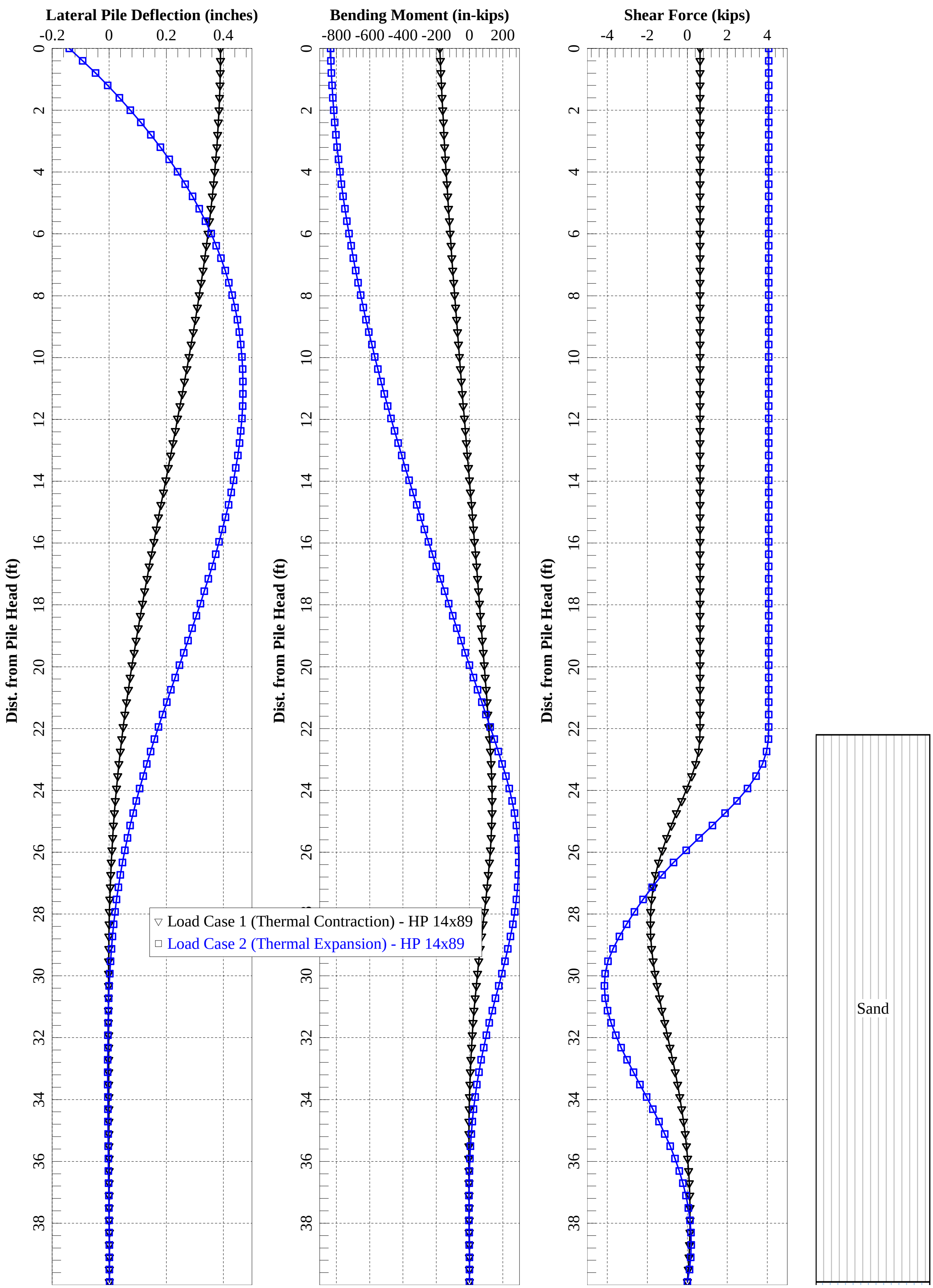
The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_{dr}) calculated above.

$P_{n.top}$ = 803 kips (From Step 4)
$P_{n.2nd}$ = 1045 kips (From Step 4)

Check $R_{ndr} < P_n$: OK
Check $R_{ndr} < P_{dr}$: OK

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 835 in-kips (70 ft-kips) occurs at the top of the pile under the Strength I load case with a maximum bridge expansion of -0.14 inches and a rotation of 0.010 radians. The results indicate when evaluating under an assumed scour depth of 22.2 feet below the bottom of footing for a pile unsupported length that the depth to bedrock is sufficient for driven piles to achieve fixity. Based on the intact section used in the analysis, HP14x89 piles are adequately sized to accommodate the anticipated design loads, thermal movement, and scour conditions at Abutment No.2 of the Smith Brook Bridge (Bridge No. 3189).



Lateral Pile Deflection (inches)
Bending Moment (in-kips)
Shear Force (kips)
Dist. from Pile Head (ft)
Load Case 1 (Thermal Contraction) - HP 14x89
Load Case 2 (Thermal Expansion) - HP 14x89
Sand

```
===============================================================================

                   LPile for Windows, Version 2022-12.007

               Analysis of Individual Piles and Drilled Shafts
              Subjected to Lateral Loading Using the p-y Method
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-------------------------------------------------------------------------------
                             Files Used for Analysis
-------------------------------------------------------------------------------

Path to file locations:
\Users\USDB712838\Downloads\

Name of input data file:
Abutment 2 HP14x89_ Scour_4.09.26.lp12d

Name of output report file:
Abutment 2 HP14x89_ Scour_4.09.26.lp12o

Name of plot output file:
Abutment 2 HP14x89_ Scour_4.09.26.lp12p

Name of runtime message file:
Abutment 2 HP14x89_ Scour_4.09.26.lp12r


-------------------------------------------------------------------------------
                            Date and Time of Analysis
```

```
--------------------------------------------------------------------------------

               Date:  April 14, 2026              Time:  16:18:36


--------------------------------------------------------------------------------
                                  Problem Title
--------------------------------------------------------------------------------

Project Name: WIN 028262.00 Northern Bridge Bundle - Bridge No. 3189 -


Job Number: US0040947.9162


Client: MaineDOT


Engineer: WSP USA Inc.


Description: HP14x89 Pile Design for Bridge 3189 Proposed Abutment No. 2




--------------------------------------------------------------------------------
                          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 = 39.900 ft
Depth of ground surface below top of pile = 22.2000 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	39.900	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 39.900000 ft

Pile width = 13.800000 in

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

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

Distance from top of pile to top of layer	=	22.200000 ft
Distance from top of pile to bottom of layer	=	39.900000 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	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	100.000000 pci
Subgrade k at bottom of layer	=	100.000000 pci

Layer 2 is strong rock (vuggy limestone)

Distance from top of pile to top of layer	=	39.900000 ft
Distance from top of pile to bottom of layer	=	50.000000 ft
Effective unit weight at top of layer	=	102.600000 pcf
Effective unit weight at bottom of layer	=	102.600000 pcf
Uniaxial compressive strength at top of layer	=	7000. psi
Uniaxial compressive strength at bottom of layer	=	7000. psi

(Depth of the lowest soil layer extends 10.100 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	Angle of Friction deg.	Uniaxial qu psi	kpy pci
1	Sand	22.2000	67.6000	36.0000	--	100.0000

```
          (Reese, et al.)        39.9000      67.6000      36.0000       --
100.0000
  2         Strong Rock          39.9000     102.6000       --           7000.
 --
         (Vuggy Limestone)       50.0000     102.6000       --           7000.
 --
```

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 = 2

```
Load    Load         Condition                Condition           Axial Thrust
Compute Top y     Run Analysis
 No.    Type            1                         2                Force, lbs
vs. Pile Length
-----   ----   --------------------   ----------------------   ----------------
---------------    ------------
   1     5     y =     0.390000 in    S =       0.0000 in/in           367000.
    N.A.               Yes
   2     5     y =     -0.14000 in    S =     0.010000 in/in           367000.
    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	=	39.900000 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	367.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 367.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000442	41.7773444	9448389.	118.2345231	15.1514881	
0.00000884	83.5546887	9448389.	62.7922615	16.0845352	

0.00001326	125.3320331	9448389.	44.3115077	17.0175823	
0.00001769	167.1093775	9448389.	35.0711308	17.9506295	
0.00002211	208.8867218	9448389.	29.5269046	18.8836766	
0.00002653	250.6640662	9448389.	25.8307538	19.8167238	
0.00003095	292.4414106	9448389.	23.1906462	20.7497709	
0.00003537	334.2187549	9448389.	21.2105654	21.6828180	
0.00003979	375.9960993	9448389.	19.6705026	22.6158653	
0.00004422	417.7734437	9448389.	18.4384523	23.5489123	
0.00004864	459.5507880	9448389.	17.4304112	24.4819595	
0.00005306	501.3281324	9448389.	16.5903769	25.4150066	
0.00005748	543.1054768	9448389.	15.8795787	26.3480537	
0.00006190	584.8828211	9448389.	15.2703231	27.2811009	
0.00006632	626.6601655	9448389.	14.7423015	28.2141480	
0.00007075	668.4375098	9448389.	14.2802827	29.1471952	
0.00007517	710.2148542	9448389.	13.8726190	30.0802423	
0.00007959	751.9921986	9448389.	13.5102513	31.0132894	
0.00008401	793.7695429	9448389.	13.1860275	31.9463366	
0.00008843	835.5468873	9448389.	12.8942262	32.8793838	
0.00009285	877.3242317	9448389.	12.6302154	33.8124309	
0.00009728	919.1015760	9448389.	12.3902056	34.7454780	
0.0001017	960.8789204	9448389.	12.1710662	35.6785251	
0.0001061	1003.	9448389.	11.9701885	36.6115723	
0.0001105	1044.	9448389.	11.7853809	37.5446194	
0.0001150	1086.	9448389.	11.6147893	38.4776666	
0.0001194	1128.	9448389.	11.4568342	39.4107137	
0.0001238	1170.	9448389.	11.3101615	40.3437608	
0.0001282	1212.	9448389.	11.1736042	41.2768080	
0.0001326	1253.	9448389.	11.0461508	42.2098551	
0.0001371	1295.	9448389.	10.9269201	43.1429023	
0.0001415	1337.	9448389.	10.8151413	44.0759494	
0.0001459	1379.	9448389.	10.7101371	45.0089965	
0.0001503	1420.	9448389.	10.6113095	45.9420437	
0.0001548	1462.	9448389.	10.5181292	46.8750908	
0.0001592	1504.	9448389.	10.4301256	47.8081380	
0.0001636	1546.	9448389.	10.3468790	48.7411851	
0.0001680	1588.	9448389.	10.2680138	49.6742323	
0.0001724	1629.	9443723.	10.1940496	50.0000000	Y
0.0001813	1706.	9409764.	10.0617064	50.0000000	Y
0.0001901	1778.	9351217.	9.9471160	50.0000000	Y
0.0001990	1845.	9275072.	9.8474020	50.0000000	Y
0.0002078	1909.	9186226.	9.7603030	50.0000000	Y
0.0002167	1969.	9088653.	9.6839052	50.0000000	Y
0.0002255	2026.	8985149.	9.6166785	50.0000000	Y
0.0002343	2080.	8877224.	9.5575126	50.0000000	Y
0.0002432	2132.	8767936.	9.5049842	50.0000000	Y
0.0002520	2182.	8657023.	9.4585881	50.0000000	Y
0.0002609	2230.	8546441.	9.4172999	50.0000000	Y
0.0002697	2276.	8436902.	9.3804708	50.0000000	Y
0.0002786	2320.	8328745.	9.3476031	50.0000000	Y
0.0002874	2363.	8221937.	9.3183563	50.0000000	Y

0.0002962	2405.	8117206.	9.2921946	50.0000000	Y
0.0003051	2445.	8014781.	9.2687565	50.0000000	Y
0.0003139	2485.	7914802.	9.2477355	50.0000000	Y
0.0003228	2523.	7815381.	9.2284423	50.0000000	Y
0.0003316	2558.	7712825.	9.2096343	50.0000000	Y
0.0003405	2590.	7608368.	9.1911858	50.0000000	Y
0.0003493	2621.	7502911.	9.1730365	50.0000000	Y
0.0003582	2649.	7397217.	9.1551297	50.0000000	Y
0.0003670	2676.	7291027.	9.1378246	50.0000000	Y
0.0003758	2701.	7185666.	9.1207678	50.0000000	Y
0.0003847	2724.	7081536.	9.1039508	50.0000000	Y
0.0003935	2746.	6977697.	9.0876016	50.0000000	Y
0.0004024	2767.	6875589.	9.0712177	50.0000000	Y
0.0004112	2786.	6774695.	9.0554922	50.0000000	Y
0.0004201	2804.	6676230.	9.0399111	50.0000000	Y
0.0004289	2822.	6578534.	9.0246785	50.0000000	Y
0.0004377	2838.	6483272.	9.0094336	50.0000000	Y
0.0004466	2854.	6389730.	8.9950233	50.0000000	Y
0.0004554	2868.	6297812.	8.9803119	50.0000000	Y
0.0004643	2882.	6208019.	8.9661427	50.0000000	Y
0.0004731	2895.	6120073.	8.9522270	50.0000000	Y
0.0004820	2908.	6034149.	8.9382517	50.0000000	Y
0.0004908	2920.	5950014.	8.9249942	50.0000000	Y
0.0004996	2932.	5867483.	8.9116029	50.0000000	Y
0.0005085	2943.	5787469.	8.8985011	50.0000000	Y
0.0005173	2953.	5708419.	8.8856728	50.0000000	Y
0.0005262	2963.	5631656.	8.8731656	50.0000000	Y
0.0005615	2999.	5340961.	8.8245281	50.0000000	Y
0.0005969	3030.	5075908.	8.7791277	50.0000000	Y
0.0006323	3056.	4833303.	8.7361931	50.0000000	Y
0.0006677	3079.	4611513.	8.6958050	50.0000000	Y
0.0007030	3099.	4407785.	8.6579820	50.0000000	Y
0.0007384	3116.	4220367.	8.6220712	50.0000000	Y
0.0007738	3132.	4047417.	8.5879985	50.0000000	Y
0.0008092	3146.	3887458.	8.5556478	50.0000000	Y
0.0008445	3158.	3739153.	8.5250335	50.0000000	Y
0.0008799	3169.	3601594.	8.4957228	50.0000000	Y
0.0009153	3179.	3473493.	8.4683918	50.0000000	Y
0.0009507	3188.	3353733.	8.4419138	50.0000000	Y
0.0009860	3197.	3241904.	8.4165778	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips

----	----------------	----------------
1	367.0000000000	3197.

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	22.2000	0.00	N.A.	No	0.00	401079.
2	39.9000	17.7000	No	Yes	N.A.	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 = 0.390000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 367000.0 lbs

Depth   Deflect.   Bending   Shear   Slope   Total   Bending   Soil

X feet	y inches	Moment in-lbs	Force lbs	S radians	Stress psi*	Stiffness lb-in^2	Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3900	-178670.	641.4386	0.00	18249.	9.45E+09	0.00	0.00	0.00
0.3990	0.3898	-175520.	641.1458	-8.97E-05	18178.	9.45E+09	0.00	0.00	0.00
0.7980	0.3891	-172215.	641.1458	-1.78E-04	18103.	9.45E+09	0.00	0.00	0.00
1.1970	0.3881	-168756.	641.1458	-2.64E-04	18025.	9.45E+09	0.00	0.00	0.00
1.5960	0.3866	-165146.	641.1458	-3.49E-04	17944.	9.45E+09	0.00	0.00	0.00
1.9950	0.3847	-161390.	641.1458	-4.32E-04	17859.	9.45E+09	0.00	0.00	0.00
2.3940	0.3825	-157490.	641.1458	-5.12E-04	17771.	9.45E+09	0.00	0.00	0.00
2.7930	0.3798	-153450.	641.1458	-5.91E-04	17680.	9.45E+09	0.00	0.00	0.00
3.1920	0.3768	-149273.	641.1458	-6.68E-04	17586.	9.45E+09	0.00	0.00	0.00
3.5910	0.3734	-144963.	641.1458	-7.42E-04	17488.	9.45E+09	0.00	0.00	0.00
3.9900	0.3697	-140524.	641.1458	-8.15E-04	17388.	9.45E+09	0.00	0.00	0.00
4.3890	0.3656	-135960.	641.1458	-8.85E-04	17285.	9.45E+09	0.00	0.00	0.00
4.7880	0.3612	-131275.	641.1458	-9.53E-04	17180.	9.45E+09	0.00	0.00	0.00
5.1870	0.3565	-126473.	641.1458	-0.00102	17071.	9.45E+09	0.00	0.00	0.00
5.5860	0.3515	-121558.	641.1458	-0.00108	16960.	9.45E+09	0.00	0.00	0.00
5.9850	0.3462	-116535.	641.1458	-0.00114	16847.	9.45E+09	0.00	0.00	0.00
6.3840	0.3406	-111409.	641.1458	-0.00120	16732.	9.45E+09	0.00	0.00	0.00
6.7830	0.3347	-106183.	641.1458	-0.00125	16614.	9.45E+09	0.00	0.00	0.00
7.1820	0.3286	-100862.	641.1458	-0.00131	16494.	9.45E+09	0.00	0.00	0.00
7.5810	0.3222	-95452.	641.1458	-0.00136	16372.	9.45E+09	0.00	0.00	0.00
7.9800	0.3156	-89957.	641.1458	-0.00140	16248.	9.45E+09	0.00	0.00	0.00
8.3790	0.3087	-84382.	641.1458	-0.00145	16122.	9.45E+09			

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0.00       0.00       0.00
    8.7780     0.3017    -78731.    641.1458   -0.00149     15994.   9.45E+09
0.00       0.00       0.00
    9.1770     0.2945    -73011.    641.1458   -0.00153     15865.   9.45E+09
0.00       0.00       0.00
    9.5760     0.2871    -67225.    641.1458   -0.00156     15735.   9.45E+09
0.00       0.00       0.00
    9.9750     0.2795    -61379.    641.1458   -0.00160     15603.   9.45E+09
0.00       0.00       0.00
   10.3740     0.2718    -55479.    641.1458   -0.00162     15470.   9.45E+09
0.00       0.00       0.00
   10.7730     0.2640    -49530.    641.1458   -0.00165     15336.   9.45E+09
0.00       0.00       0.00
   11.1720     0.2560    -43536.    641.1458   -0.00167     15201.   9.45E+09
0.00       0.00       0.00
   11.5710     0.2479    -37504.    641.1458   -0.00170     15064.   9.45E+09
0.00       0.00       0.00
   11.9700     0.2398    -31438.    641.1458   -0.00171     14928.   9.45E+09
0.00       0.00       0.00
   12.3690     0.2315    -25344.    641.1458   -0.00173     14790.   9.45E+09
0.00       0.00       0.00
   12.7680     0.2232    -19228.    641.1458   -0.00174     14652.   9.45E+09
0.00       0.00       0.00
   13.1670     0.2149    -13095.    641.1458   -0.00175     14514.   9.45E+09
0.00       0.00       0.00
   13.5660     0.2065     -6949.    641.1458   -0.00175     14375.   9.45E+09
0.00       0.00       0.00
   13.9650     0.1981   -798.107    641.1458   -0.00175     14236.   9.45E+09
0.00       0.00       0.00
   14.3640     0.1897      5354.    641.1458   -0.00175     14339.   9.45E+09
0.00       0.00       0.00
   14.7630     0.1813     11501.    641.1458   -0.00175     14478.   9.45E+09
0.00       0.00       0.00
   15.1620     0.1730     17638.    641.1458   -0.00174     14616.   9.45E+09
0.00       0.00       0.00
   15.5610     0.1646     23759.    641.1458   -0.00173     14754.   9.45E+09
0.00       0.00       0.00
   15.9600     0.1564     29860.    641.1458   -0.00172     14892.   9.45E+09
0.00       0.00       0.00
   16.3590     0.1482     35933.    641.1458   -0.00170     15029.   9.45E+09
0.00       0.00       0.00
   16.7580     0.1401     41975.    641.1458   -0.00168     15165.   9.45E+09
0.00       0.00       0.00
   17.1570     0.1321     47979.    641.1458   -0.00166     15301.   9.45E+09
0.00       0.00       0.00
   17.5560     0.1242     53940.    641.1458   -0.00163     15435.   9.45E+09
0.00       0.00       0.00
   17.9550     0.1165     59854.    641.1458   -0.00160     15569.   9.45E+09
0.00       0.00       0.00
   18.3540     0.1089     65714.    641.1458   -0.00157     15701.   9.45E+09
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0.00        0.00        0.00
   18.7530     0.1014     71516.   641.1458   -0.00154     15832.   9.45E+09
0.00        0.00        0.00
   19.1520    0.09416     77253.   641.1458   -0.00150     15961.   9.45E+09
0.00        0.00        0.00
   19.5510    0.08708     82923.   641.1458   -0.00146     16089.   9.45E+09
0.00        0.00        0.00
   19.9500    0.08020     88518.   641.1458   -0.00141     16215.   9.45E+09
0.00        0.00        0.00
   20.3490    0.07353     94034.   641.1458   -0.00137     16340.   9.45E+09
0.00        0.00        0.00
   20.7480    0.06709     99467.   641.1458   -0.00132     16462.   9.45E+09
0.00        0.00        0.00
   21.1470    0.06090    104811.   641.1458   -0.00127     16583.   9.45E+09
0.00        0.00        0.00
   21.5460    0.05495    110062.   641.1458   -0.00121     16701.   9.45E+09
0.00        0.00        0.00
   21.9450    0.04928    115215.   641.1458   -0.00116     16817.   9.45E+09
0.00        0.00        0.00
   22.3440    0.04388    120265.   628.7235   -0.00110     16931.   9.45E+09
-5.189   566.1805       0.00
   22.7430    0.03878    125089.   566.1412   -0.00103     17040.   9.45E+09
-20.952       2587.       0.00
   23.1420    0.03398    129322.   426.5832  -9.70E-04     17136.   9.45E+09
-37.342       5263.       0.00
   23.5410    0.02949    132583.   223.5842  -9.04E-04     17209.   9.45E+09
-47.452       7705.       0.00
   23.9400    0.02532    134638.    -16.597  -8.36E-04     17256.   9.45E+09
-52.874       9997.       0.00
   24.3390    0.02148    135362.   -275.193  -7.67E-04     17272.   9.45E+09
-55.145     12290.       0.00
   24.7380    0.01797    134700.   -538.258  -6.99E-04     17257.   9.45E+09
-54.740     14582.       0.00
   25.1370    0.01479    132664.   -794.095  -6.31E-04     17211.   9.45E+09
-52.126     16875.       0.00
   25.5360    0.01193    129315.     -1033.  -5.65E-04     17135.   9.45E+09
-47.751     19167.       0.00
   25.9350    0.00938    124756.     -1248.  -5.01E-04     17033.   9.45E+09
-42.043     21460.       0.00
   26.3340    0.00714    119121.     -1434.  -4.39E-04     16905.   9.45E+09
-35.396     23752.       0.00
   26.7330    0.00518    112570.     -1586.  -3.80E-04     16758.   9.45E+09
-28.171     26045.       0.00
   27.1320    0.00350    105272.     -1703.  -3.25E-04     16593.   9.45E+09
-20.690     28337.       0.00
   27.5310    0.00207     97406.     -1784.  -2.73E-04     16416.   9.45E+09
-13.231     30630.       0.00
   27.9300   8.77E-04     89150.     -1830.  -2.26E-04     16229.   9.45E+09
-6.030     32922.       0.00
   28.3290  -9.80E-05     80677.     -1843.  -1.83E-04     16038.   9.45E+09
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0.7207 35215. 0.00
28.7280 -8.77E-04 72148. -1825. -1.44E-04 15846. 9.45E+09
6.8718 37507. 0.00
29.1270 -0.00148 63713. -1779. -1.10E-04 15656. 9.45E+09
12.3140 39800. 0.00
29.5260 -0.00193 55503. -1709. -7.98E-05 15470. 9.45E+09
16.9757 42092. 0.00
29.9250 -0.00225 47632. -1618. -5.37E-05 15293. 9.45E+09
20.8196 44385. 0.00
30.3240 -0.00245 40197. -1511. -3.15E-05 15125. 9.45E+09
23.8383 46677. 0.00
30.7230 -0.00255 33272. -1392. -1.28E-05 14969. 9.45E+09
26.0504 48970. 0.00
31.1220 -0.00257 26915. -1263. 2.42E-06 14826. 9.45E+09
27.4957 51262. 0.00
31.5210 -0.00252 21165. -1130. 1.46E-05 14696. 9.45E+09
28.2308 53555. 0.00
31.9200 -0.00243 16042. -994.695 2.40E-05 14580. 9.45E+09
28.3245 55847. 0.00
32.3190 -0.00229 11555. -860.204 3.10E-05 14479. 9.45E+09
27.8541 58140. 0.00
32.7180 -0.00213 7696. -729.120 3.59E-05 14392. 9.45E+09
26.9010 60432. 0.00
33.1170 -0.00195 4447. -603.558 3.90E-05 14319. 9.45E+09
25.5476 62725. 0.00
33.5160 -0.00176 1779. -485.243 4.05E-05 14259. 9.45E+09
23.8742 65017. 0.00
33.9150 -0.00156 -342.390 -375.524 4.09E-05 14226. 9.45E+09
21.9563 67310. 0.00
34.3140 -0.00137 -1960. -275.410 4.03E-05 14263. 9.45E+09
19.8624 69602. 0.00
34.7130 -0.00118 -3121. -185.599 3.90E-05 14289. 9.45E+09
17.6528 71895. 0.00
35.1120 -9.92E-04 -3875. -106.523 3.73E-05 14306. 9.45E+09
15.3779 74187. 0.00
35.5110 -8.19E-04 -4273. -38.400 3.52E-05 14315. 9.45E+09
13.0778 76480. 0.00
35.9100 -6.55E-04 -4366. 18.7196 3.30E-05 14317. 9.45E+09
10.7819 78772. 0.00
36.3090 -5.03E-04 -4209. 64.9015 3.08E-05 14313. 9.45E+09
8.5088 81065. 0.00
36.7080 -3.60E-04 -3853. 100.2756 2.88E-05 14305. 9.45E+09
6.2673 83357. 0.00
37.1070 -2.27E-04 -3350. 124.9908 2.70E-05 14294. 9.45E+09
4.0565 85650. 0.00
37.5060 -1.02E-04 -2751. 139.1727 2.54E-05 14281. 9.45E+09
1.8674 87942. 0.00
37.9050 1.67E-05 -2107. 142.8877 2.42E-05 14266. 9.45E+09
-0.316 90235. 0.00
38.3040 1.30E-04 -1468. 136.1153 2.33E-05 14252. 9.45E+09

```
-2.513     92527.       0.00
   38.7030   2.40E-04   -885.334   118.7294   2.27E-05     14238.   9.45E+09
-4.749     94820.       0.00
   39.1020   3.47E-04   -410.780    90.4921   2.24E-05     14228.   9.45E+09
-7.046     97112.       0.00
   39.5010   4.54E-04    -97.393    51.0588   2.22E-05     14221.   9.45E+09
-9.426     99405.       0.00
   39.9000   5.60E-04       0.00       0.00   2.22E-05     14218.   9.45E+09
-11.902     50849.       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              =     0.39000000 inches
Computed slope at pile head       =       0.000000 radians
Maximum bending moment            =       -178670. inch-lbs
Maximum shear force               =         -1843. lbs
Depth of maximum bending moment   =       0.000000 feet below pile head
Depth of maximum shear force      =    28.32900000 feet below pile head
Number of iterations              =              6
Number of zero deflection points =              2
Pile deflection at ground         =     0.04582837 inches
```

```
--------------------------------------------------------------------------------
                 Computed Values of Pile Loading and Deflection
                   for Lateral Loading for Load Case Number 2
--------------------------------------------------------------------------------

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head                              =    -0.140000 inches
Rotation of pile head                                  =    1.000E-02 radians
Axial load on pile head                                =     367000.0 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	-0.140	-834517.	4072.	0.01000	33043.	9.45E+09
0.00	0.00	0.00				
0.3990	-0.09313	-832219.	4072.	0.00958	32991.	9.45E+09
0.00	0.00	0.00				
0.7980	-0.04828	-829181.	4072.	0.00916	32922.	9.45E+09
0.00	0.00	0.00				
1.1970	-0.00545	-825404.	4072.	0.00874	32837.	9.45E+09
0.00	0.00	0.00				
1.5960	0.03539	-820892.	4072.	0.00832	32736.	9.45E+09
0.00	0.00	0.00				
1.9950	0.07423	-815649.	4072.	0.00791	32617.	9.45E+09
0.00	0.00	0.00				
2.3940	0.1111	-809680.	4072.	0.00749	32483.	9.45E+09
0.00	0.00	0.00				
2.7930	0.1460	-802990.	4072.	0.00709	32332.	9.45E+09
0.00	0.00	0.00				
3.1920	0.1789	-795584.	4072.	0.00668	32165.	9.45E+09
0.00	0.00	0.00				
3.5910	0.2100	-787471.	4072.	0.00628	31982.	9.45E+09
0.00	0.00	0.00				
3.9900	0.2391	-778656.	4072.	0.00588	31783.	9.45E+09
0.00	0.00	0.00				
4.3890	0.2663	-769148.	4072.	0.00549	31568.	9.45E+09
0.00	0.00	0.00				
4.7880	0.2916	-758955.	4072.	0.00510	31338.	9.45E+09
0.00	0.00	0.00				
5.1870	0.3152	-748086.	4072.	0.00472	31093.	9.45E+09
0.00	0.00	0.00				
5.5860	0.3369	-736550.	4072.	0.00434	30833.	9.45E+09
0.00	0.00	0.00				
5.9850	0.3568	-724359.	4072.	0.00397	30558.	9.45E+09
0.00	0.00	0.00				
6.3840	0.3749	-711523.	4072.	0.00361	30268.	9.45E+09
0.00	0.00	0.00				
6.7830	0.3913	-698054.	4072.	0.00325	29965.	9.45E+09
0.00	0.00	0.00				
7.1820	0.4061	-683963.	4072.	0.00290	29647.	9.45E+09
0.00	0.00	0.00				
7.5810	0.4191	-669263.	4072.	0.00256	29315.	9.45E+09
0.00	0.00	0.00				
7.9800	0.4306	-653966.	4072.	0.00223	28970.	9.45E+09
0.00	0.00	0.00				
8.3790	0.4405	-638088.	4072.	0.00190	28612.	9.45E+09
0.00	0.00	0.00				
8.7780	0.4488	-621641.	4072.	0.00158	28241.	9.45E+09
0.00	0.00	0.00				
9.1770	0.4556	-604641.	4072.	0.00127	27857.	9.45E+09
0.00	0.00	0.00				
9.5760	0.4609	-587102.	4072.	9.66E-04	27462.	9.45E+09
0.00	0.00	0.00				

9.9750	0.4648	-569041.	4072.	6.73E-04	27054.	9.45E+09	0.00	0.00	0.00
10.3740	0.4674	-550473.	4072.	3.90E-04	26636.	9.45E+09	0.00	0.00	0.00
10.7730	0.4686	-531414.	4072.	1.16E-04	26206.	9.45E+09	0.00	0.00	0.00
11.1720	0.4685	-511883.	4072.	-1.49E-04	25765.	9.45E+09	0.00	0.00	0.00
11.5710	0.4671	-491895.	4072.	-4.03E-04	25314.	9.45E+09	0.00	0.00	0.00
11.9700	0.4646	-471470.	4072.	-6.47E-04	24854.	9.45E+09	0.00	0.00	0.00
12.3690	0.4609	-450625.	4072.	-8.81E-04	24383.	9.45E+09	0.00	0.00	0.00
12.7680	0.4562	-429378.	4072.	-0.00110	23904.	9.45E+09	0.00	0.00	0.00
13.1670	0.4504	-407749.	4072.	-0.00132	23416.	9.45E+09	0.00	0.00	0.00
13.5660	0.4436	-385758.	4072.	-0.00152	22920.	9.45E+09	0.00	0.00	0.00
13.9650	0.4358	-363422.	4072.	-0.00171	22416.	9.45E+09	0.00	0.00	0.00
14.3640	0.4272	-340763.	4072.	-0.00189	21905.	9.45E+09	0.00	0.00	0.00
14.7630	0.4178	-317801.	4072.	-0.00205	21387.	9.45E+09	0.00	0.00	0.00
15.1620	0.4076	-294555.	4072.	-0.00221	20863.	9.45E+09	0.00	0.00	0.00
15.5610	0.3966	-271047.	4072.	-0.00235	20333.	9.45E+09	0.00	0.00	0.00
15.9600	0.3851	-247298.	4072.	-0.00248	19797.	9.45E+09	0.00	0.00	0.00
16.3590	0.3729	-223329.	4072.	-0.00260	19256.	9.45E+09	0.00	0.00	0.00
16.7580	0.3602	-199161.	4072.	-0.00271	18711.	9.45E+09	0.00	0.00	0.00
17.1570	0.3469	-174815.	4072.	-0.00280	18162.	9.45E+09	0.00	0.00	0.00
17.5560	0.3333	-150314.	4072.	-0.00289	17609.	9.45E+09	0.00	0.00	0.00
17.9550	0.3193	-125679.	4072.	-0.00296	17053.	9.45E+09	0.00	0.00	0.00
18.3540	0.3050	-100932.	4072.	-0.00301	16495.	9.45E+09	0.00	0.00	0.00
18.7530	0.2905	-76095.	4072.	-0.00306	15935.	9.45E+09	0.00	0.00	0.00
19.1520	0.2757	-51191.	4072.	-0.00309	15373.	9.45E+09	0.00	0.00	0.00
19.5510	0.2609	-26241.	4072.	-0.00311	14810.	9.45E+09	0.00	0.00	0.00

19.9500	0.2460	-1267.	4072.	-0.00312	14247.	9.45E+09	0.00	0.00	0.00
20.3490	0.2310	23708.	4072.	-0.00311	14753.	9.45E+09	0.00	0.00	0.00
20.7480	0.2162	48661.	4072.	-0.00309	15316.	9.45E+09	0.00	0.00	0.00
21.1470	0.2014	73571.	4072.	-0.00306	15878.	9.45E+09	0.00	0.00	0.00
21.5460	0.1869	98416.	4072.	-0.00302	16438.	9.45E+09	0.00	0.00	0.00
21.9450	0.1725	123173.	4072.	-0.00296	16997.	9.45E+09	0.00	0.00	0.00
22.3440	0.1585	147821.	4055.	-0.00289	17553.	9.45E+09	-7.278	219.8591	0.00
22.7430	0.1448	172169.	3965.	-0.00281	18102.	9.45E+09	-30.332	1003.	0.00
23.1420	0.1316	195670.	3760.	-0.00272	18632.	9.45E+09	-55.305	2013.	0.00
23.5410	0.1188	217728.	3437.	-0.00261	19130.	9.45E+09	-79.314	3197.	0.00
23.9400	0.1065	237774.	3008.	-0.00250	19582.	9.45E+09	-100.168	4502.	0.00
24.3390	0.09486	255312.	2487.	-0.00237	19978.	9.45E+09	-117.485	5930.	0.00
24.7380	0.08380	269929.	1895.	-0.00224	20307.	9.45E+09	-129.705	7411.	0.00
25.1370	0.07340	281332.	1255.	-0.00210	20565.	9.45E+09	-137.463	8967.	0.00
25.5360	0.06368	289334.	593.8298	-0.00196	20745.	9.45E+09	-138.873	10441.	0.00
25.9350	0.05467	293894.	-58.569	-0.00181	20848.	9.45E+09	-133.641	11705.	0.00
26.3340	0.04636	295129.	-682.637	-0.00166	20876.	9.45E+09	-127.039	13120.	0.00
26.7330	0.03878	293189.	-1259.	-0.00151	20832.	9.45E+09	-113.606	14028.	0.00
27.1320	0.03190	288383.	-1760.	-0.00136	20724.	9.45E+09	-95.829	14383.	0.00
27.5310	0.02573	281123.	-2211.	-0.00122	20560.	9.45E+09	-92.552	17226.	0.00
27.9300	0.02023	271492.	-2640.	-0.00108	20343.	9.45E+09	-86.528	20478.	0.00
28.3290	0.01540	259635.	-3034.	-9.44E-04	20075.	9.45E+09	-77.942	24238.	0.00
28.7280	0.01119	245760.	-3393.	-8.16E-04	19762.	9.45E+09	-72.028	30814.	0.00
29.1270	0.00758	230015.	-3716.	-6.95E-04	19407.	9.45E+09	-63.038	39800.	0.00
29.5260	0.00453	212620.	-3962.	-5.83E-04	19015.	9.45E+09	-39.852	42092.	0.00

29.9250	0.00200	194122.	-4102.	-4.80E-04	18597.	9.45E+09	-18.527	44385.	0.00
30.3240	-6.50E-05	175026.	-4145.	-3.87E-04	18167.	9.45E+09	0.6333	46677.	0.00
30.7230	-0.00170	155790.	-4102.	-3.03E-04	17733.	9.45E+09	17.4263	48970.	0.00
31.1220	-0.00296	136813.	-3984.	-2.29E-04	17305.	9.45E+09	31.7416	51262.	0.00
31.5210	-0.00389	118443.	-3804.	-1.64E-04	16890.	9.45E+09	43.5514	53555.	0.00
31.9200	-0.00454	100966.	-3573.	-1.08E-04	16496.	9.45E+09	52.8987	55847.	0.00
32.3190	-0.00493	84611.	-3303.	-6.14E-05	16127.	9.45E+09	59.8856	58140.	0.00
32.7180	-0.00512	69554.	-3005.	-2.23E-05	15787.	9.45E+09	64.6612	60432.	0.00
33.1170	-0.00515	55918.	-2688.	9.47E-06	15480.	9.45E+09	67.4090	62725.	0.00
33.5160	-0.00503	43777.	-2363.	3.47E-05	15206.	9.45E+09	68.3361	65017.	0.00
33.9150	-0.00481	33163.	-2038.	5.42E-05	14967.	9.45E+09	67.6616	67310.	0.00
34.3140	-0.00451	24072.	-1719.	6.87E-05	14761.	9.45E+09	65.6073	69602.	0.00
34.7130	-0.00415	16462.	-1412.	7.90E-05	14590.	9.45E+09	62.3888	71895.	0.00
35.1120	-0.00376	10269.	-1124.	8.58E-05	14450.	9.45E+09	58.2085	74187.	0.00
35.5110	-0.00333	5400.	-856.862	8.97E-05	14340.	9.45E+09	53.2488	76480.	0.00
35.9100	-0.00290	1748.	-615.267	9.15E-05	14258.	9.45E+09	47.6684	78772.	0.00
36.3090	-0.00246	-813.040	-401.562	9.18E-05	14237.	9.45E+09	41.5984	81065.	0.00
36.7080	-0.00202	-2420.	-217.848	9.10E-05	14273.	9.45E+09	35.1411	83357.	0.00
37.1070	-0.00159	-3219.	-65.805	8.95E-05	14291.	9.45E+09	28.3688	85650.	0.00
37.5060	-0.00116	-3365.	53.1638	8.79E-05	14294.	9.45E+09	21.3258	87942.	0.00
37.9050	-7.44E-04	-3019.	137.8049	8.63E-05	14287.	9.45E+09	14.0298	90235.	0.00
38.3040	-3.35E-04	-2348.	186.8966	8.49E-05	14271.	9.45E+09	6.4763	92527.	0.00
38.7030	6.85E-05	-1527.	199.1541	8.39E-05	14253.	9.45E+09	-1.356	94820.	0.00
39.1020	4.68E-04	-735.933	173.1640	8.33E-05	14235.	9.45E+09	-9.500	97112.	0.00
39.5010	8.67E-04	-161.830	107.3529	8.31E-05	14222.	9.45E+09	-17.990	99405.	0.00

```
   39.9000    0.00126       0.00       0.00   8.31E-05     14218.   9.45E+09
-26.853     50849.       0.00

* This analysis computed pile response using nonlinear moment-curvature rela-
  tionships. 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 inter-
  polated from the output for nonlinear bending properties relative to the
  magnitude of bending moment developed in the pile.


Output Summary for Load Case No. 2:

Pile-head deflection               =     -0.1400000 inches
Computed slope at pile head        =     0.01000000 radians
Maximum bending moment             =       -834517. inch-lbs
Maximum shear force                =         -4145. lbs
Depth of maximum bending moment    =       0.000000 feet below pile head
Depth of maximum shear force       =    30.32400000 feet below pile head
Number of iterations               =             10
Number of zero deflection points =              3
Pile deflection at ground          =     0.16355375 inches


--------------------------------------------------------------------------------
            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 Load                Load                  Axial    Pile-head  Pile-head  Max
Shear Max Moment
Case Type   Pile-head    Type     Pile-head   Loading  Deflection  Rotation    in
Pile    in Pile
 No.  1      Load 1       2        Load 2       lbs      inches     radians      lbs
     in-lbs
---- ----- ---------- ---------- ---------- ---------- ---------- ----------
---------- ----------
  1  y, in     0.3900   S, rad        0.00    367000.     0.3900       0.00
-1843.   -178670.
  2  y, in     -0.140   S, rad     0.01000    367000.     -0.140    0.01000
-4145.   -834517.
```

```
Maximum pile-head deflection = 0.3900000000 inches
Maximum pile-head rotation   = 0.0100000000 radians = 0.572958 deg.

The analysis ended normally.
```

CALCULATIONS

Date:	4/15/2026	**Made by:**	DEB
Project No.:	US0040947.9162	**Checked by:**	ATM
Subject:	Pile Drivability at Abutment No. 2 - HP14x89 - Driving	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine if the proposed H-piles at Abutment No. 2 can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/aisc/publications/steel-construction-manual/15th-ed-steel-construction-manual/shapes-database-v150/
3. Smith Brook Bridge No. 3189 over Smith Brook, Bancroft TWP 60% Plans, Maine, prepared for MaineDOT, dated February 5, 2026.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled NBB Bridge 3189 Smith Brook Abutment pile Design, dated 8/6/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 10th Ed. 2024.
9. Email from Hoyle Tanner Structural team received on March 13, 2026 with subject line "RE: [External] NBB-Geotech-Ext: Bridge 3189 updated pile loads"

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on five (5) HP14x89 Grade 50 piles with a center-to-center pile spacing of 7.715 feet , Ref. 3, Sheet 28 and Ref. 9 Abutment No. 2. The piles are to be driven to bedrock.
2. The soil profile used in the analysis is based on Ref. 10 at the centerline of the proposed Abutment 2.
3. As per the MaineDOT Standard Specifications Section 501.042 (Ref. 4), the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material.
4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).
5. This analysis looks at resistance during driving and upon reaching the tip elevation in bedrock. A resistance distribution of 90% shaft resistance and 10% toe resistance is assumed during driving, and 10% shaft resistance and 90% toe resistance is assumed upon reaching the tip elevation.
6. Assume a pile cap embedment of 2-feet for the piles (Ref. 3 sheet 28).

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	4/15/2026	**Made by:**	DEB
Project No.:	US0040947.9162	**Checked by:**	ATM
Subject:	Pile Drivability at Abutment No. 2 - HP14x89 - Driving	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 7)
Elastic modulus =	285.0	ksi	(Ref. 7)
Cushion thickness =	3.5	in	(Ref. 7)
Coefficient of restitution =	0.80		(Ref. 7)
Helmet weight =	2.7	kips	(Ref. 7)

Pile Information:

For an HP14x89 pile section:

	Intact Section					
Section depth, d =	13.8	in	(Ref. 2)			
Section width, b_f =	14.7	in	(Ref. 2)			
Web thickness, t_w =	0.615	in	(Ref. 2)			
Flange thickness, t_f =	0.615	in	(Ref. 2)			
Cross-sectional area, A_s =	26.1	in^2	=	0.18	ft^2	(Ref. 2)

Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)		
90% of steel yield stress =	45	ksi			
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)		
Steel unit weight =	491	pcf			
Box perimeter = (d × 2) + (b_f × 2) =	57.0	in	=	4.8	ft
Pile head elevation =	342.2	ft	(Ref. 3, Sheet 25 of 38, with pile cap embedment)		
Pile tip elevation =	300.3	ft	(Ref. 6)		
Abutment embedment length =	2.0	ft	(Ref. 3, sheet 28)		
Pile length below abutment=	39.9	ft	(Ref. 6) (excluding pile cap embedment)		

Ultimate Capacities:

Axial load on pile, P_u =	367	kips	(Ref. 6)
φ_{dyn} =	0.65		(Ref. 8, Table 10.5.5.2.3-1)
Pile driving resistance, R_{ndr} =	565	kips	

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.05	s/ft	for non-cohesive soils (Ref. 1, included as Attachment 1)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

CALCULATIONS

Date:	4/15/2026	**Made by:**	DEB
Project No.:	US0040947.9162	**Checked by:**	ATM
Subject:	Pile Drivability at Abutment No. 2 - HP14x89 - Driving	**Reviewed by:**	MEL
Project Title:	Bancroft Road Bridge No. 3189 Crossing Smith Brook, Bancroft TWP, ME (WIN 028262.00)		

Soil Profile Input, to calculate Variable Resistance Distribution:
(Ref. 10)

Pile head elevation =	340.2	ft		Does not include embedment in pile cap	
Groundwater elevation =	326.0	ft	=	14.2	ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Existing Fill	340.2	335.7	0.0	4.5	125	36	-	-	-	Sand
Alluvial Sand & Gravel	335.7	330.3	4.5	9.9	125	31	-	-	-	Sand
Alluvial Silt	330.3	318.4	9.9	21.8	115	31	-	-	-	Sand
Glacial Till	318.4	300.3	21.8	39.9	130	36	-	-	-	Sand
Bedrock	300.3	-	39.9	-	165	-	-	24.0	1008.0	Rock Other

2. Analyze an open ended diesel (OED) hammer to determine the R_{ndr} for the abutment based on the stress and blow counts outline in Ref. 4.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
During Driving	90%	10%	APE D25-42	55,814	Fuel Setting 3 (90%)	565	5.4	28.13	56%
Reaching tip elevation	10%	90%	APE D25-42	55,814	Fuel Setting 3 (90%)	565	6.1	39.90	80%

CONCLUSIONS

The analysis indicates the APE D25-42 hammer operated at fuel setting 3 (90% of the combustion pressure) should be able to overcome the soil resistance in order to drive the plumb piles at Abutment No. 2 to bedrock and a nominal resistance of 565 kips while staying at a blow count of between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT Standard Specifications.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

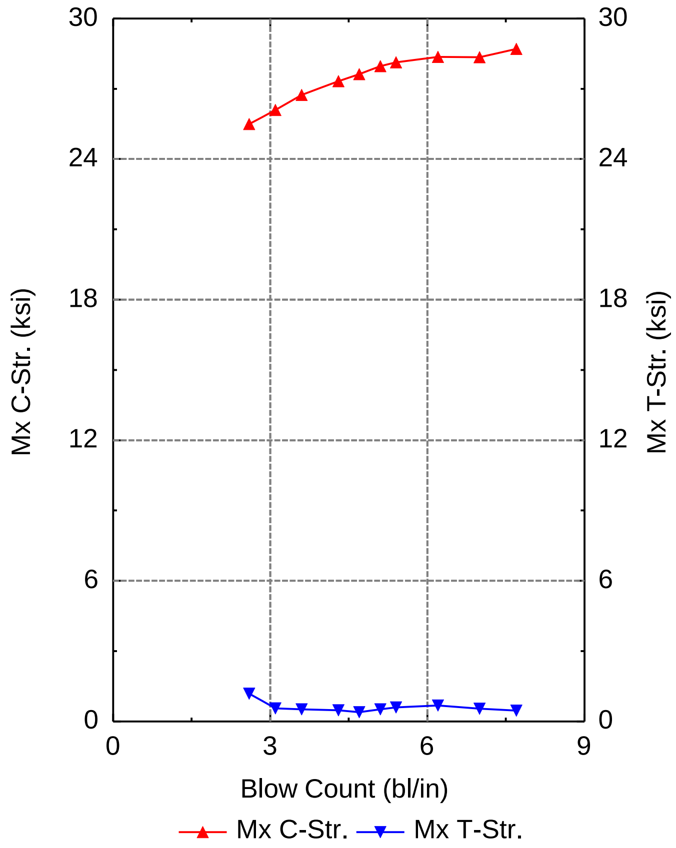
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

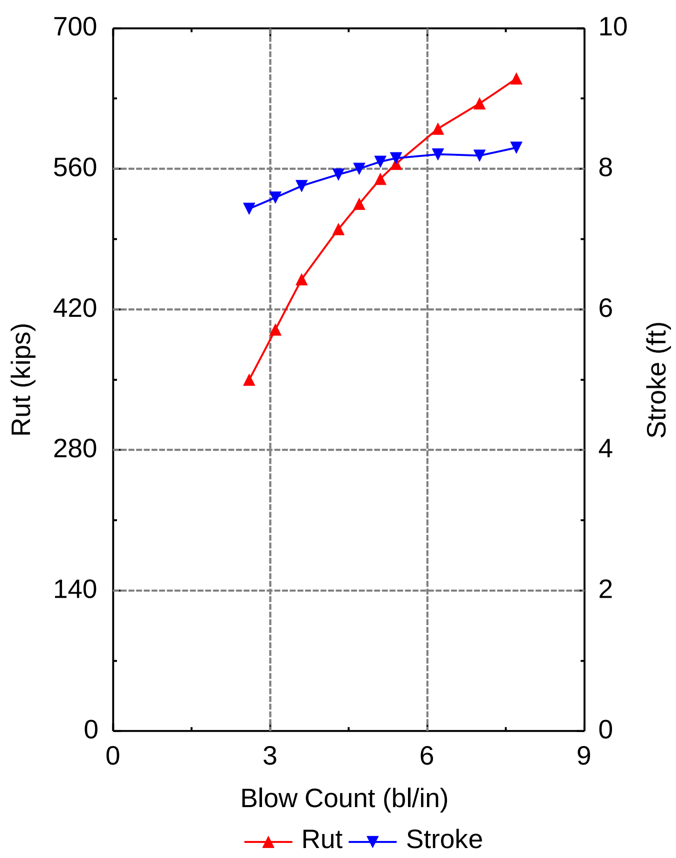
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

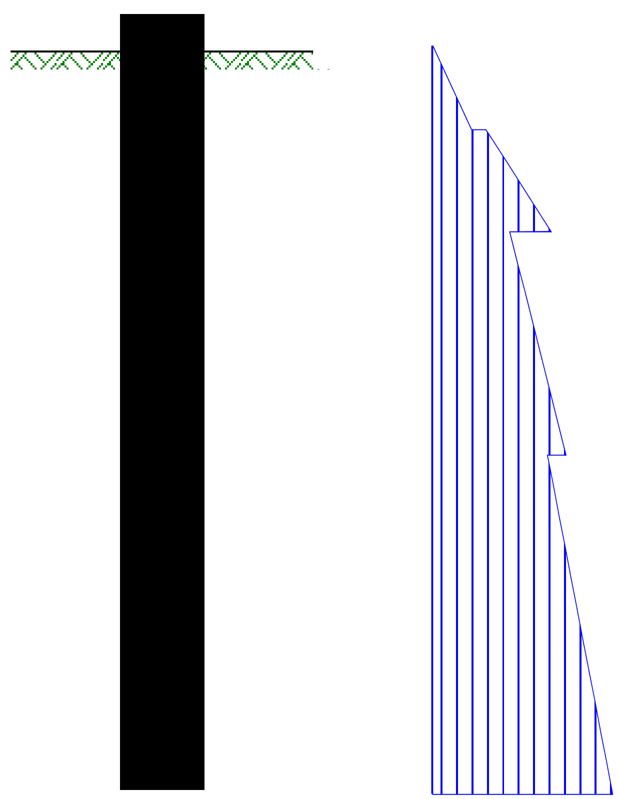
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1282.5 (90%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.123	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	41.90	ft
Pile Penetration	39.90	ft
Pile Top Area	26.10	in²
RSA	No	



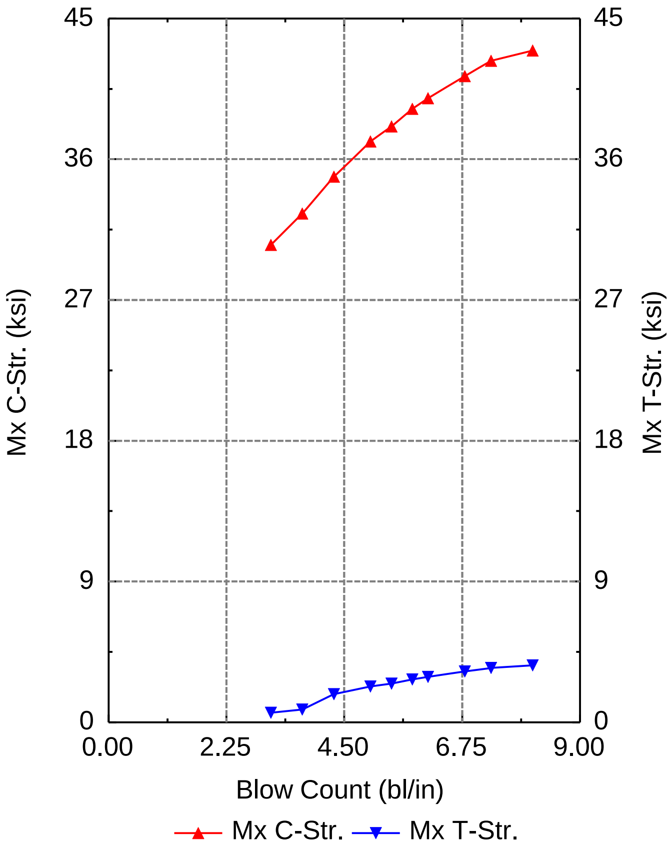


Pile Model

Shaft=90%
(Prop., SA)

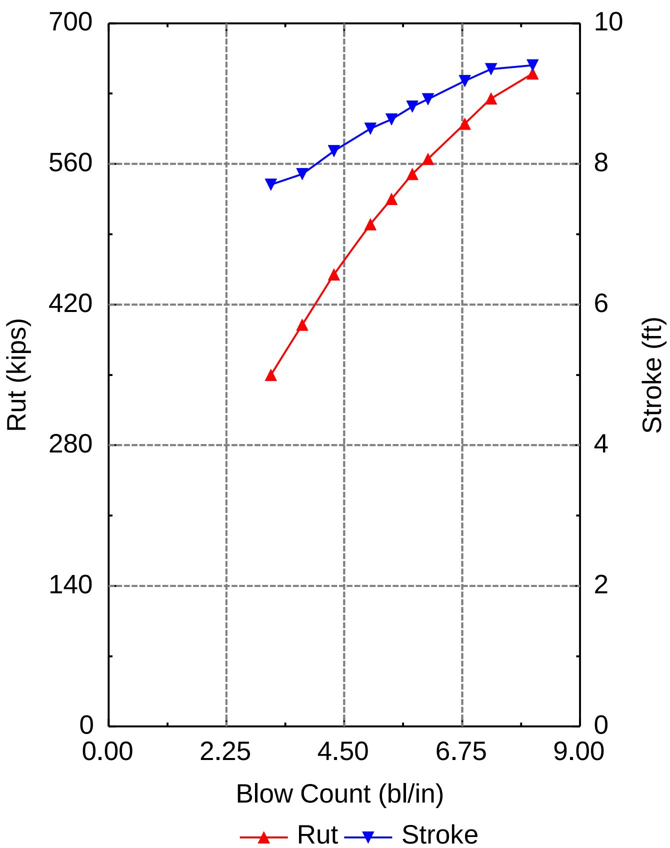
Bearing Graph Summary — APE D 25-42

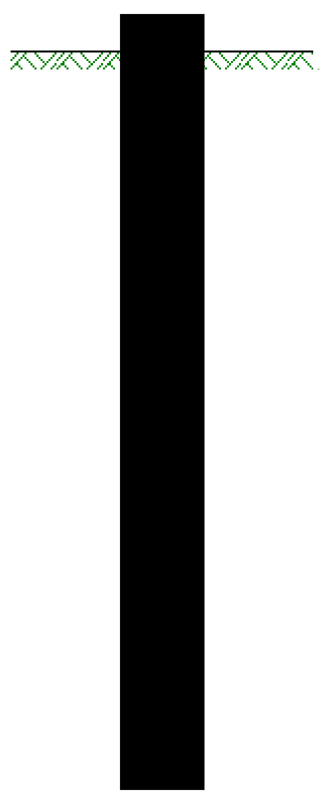
Rut kips	Mx C-Str. ksi	Top Str. ksi	Mx T-Str. ksi	Blow Ct bl/in	Stroke ft	ENTHRU kip-ft	Ham. APE	Pow. Trfr. R. %
350.0	25.50	25.03	1.18	2.6	7.43	19.63	D 25-42	31.7
400.0	26.09	25.58	0.55	3.1	7.59	19.14	D 25-42	30.9
450.0	26.74	26.19	0.52	3.6	7.76	19.11	D 25-42	30.8
500.0	27.32	26.73	0.48	4.3	7.92	19.38	D 25-42	31.3
525.0	27.62	27.03	0.40	4.7	8.00	19.48	D 25-42	31.4
550.0	27.97	27.35	0.51	5.1	8.10	19.63	D 25-42	31.7
565.0	28.13	27.50	0.59	5.4	8.15	19.68	D 25-42	31.7
600.0	28.36	27.72	0.67	6.2	8.21	19.63	D 25-42	31.7
625.0	28.35	27.67	0.54	7.0	8.19	19.34	D 25-42	31.2
650.0	28.70	28.00	0.45	7.7	8.30	19.54	D 25-42	31.5



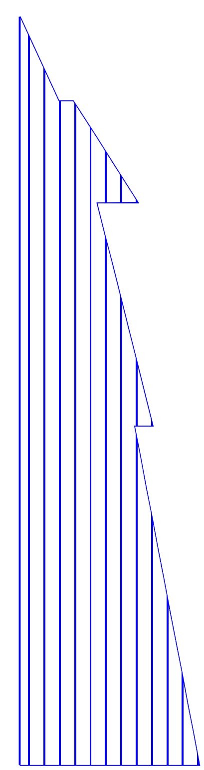
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1282.5 (90%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	41.90	ft
Pile Penetration	39.90	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model



Shaft=10%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham.	Pow. Trfr. R.
kips	ksi	ksi	ksi	bl/in	ft	kip-ft	APE	%
350.0	30.52	25.39	0.60	3.1	7.71	19.93	D 25-42	32.1
400.0	32.53	25.85	0.81	3.7	7.85	20.07	D 25-42	32.4
450.0	34.89	26.81	1.79	4.3	8.18	20.88	D 25-42	33.7
500.0	37.13	29.27	2.28	5.0	8.50	21.59	D 25-42	34.8
525.0	38.10	30.51	2.47	5.4	8.63	21.79	D 25-42	35.1
550.0	39.22	31.86	2.74	5.8	8.81	22.26	D 25-42	35.9
565.0	39.90	32.61	2.90	6.1	8.92	22.47	D 25-42	36.2
600.0	41.31	34.29	3.25	6.8	9.18	23.02	D 25-42	37.1
625.0	42.30	35.43	3.46	7.3	9.35	23.37	D 25-42	37.7
650.0	42.95	36.35	3.63	8.1	9.40	23.44	D 25-42	37.8

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