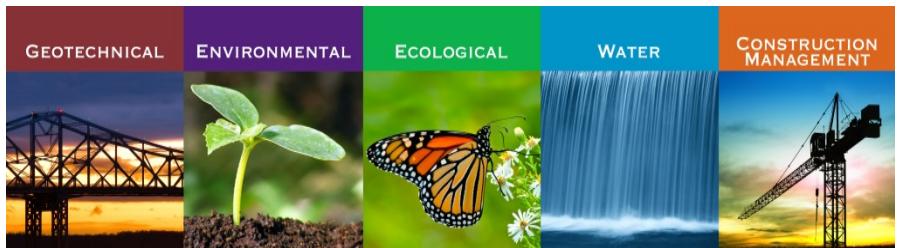




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GEOTECHNICAL DESIGN REPORT SHORTRIDGE BRIDGE NO. 2765 OVER SACO RIVER OVERFLOW MAINE DOT WIN 23849.00 FRYEBURG, MAINE

Prepared for:
Stantec
Scarborough, Maine

November 2018
09.0025967.00

Prepared by:
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VIA EMAIL

November 20, 2018
File No. 09.0025967.00

Ms. Sarah Williams, P.E.
Stantec
482 Payne Road Scarborough Court
Scarborough, Maine 04074

Re: Geotechnical Design Report
Reconstruction of Shortridge Bridge No. 2765 over Saco River Overflow
MaineDOT WIN 23849.00
Fryeburg, Maine

Dear Sarah:

We are pleased to provide this Geotechnical Design Report (GDR) to Stantec for Bridge No. 2765 over Saco River Overflow in Fryeburg, Maine. Our work was completed in accordance with Master Services Agreement 40839 between Stantec and GZA GeoEnvironmental, Inc. (GZA), and Stantec Task Order JN 195311568, dated February 27, 2018, which incorporates GZA's proposal No. 09.P000100.18, dated February 15, 2018, and the attached *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to Stantec, who is under contract with Maine Department of Transportation for preliminary and final design of the proposed bridge replacement.

It has been a pleasure serving Stantec on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Nicholas Williams, P.E.
Assistant Project Manager

Christopher L. Snow, P.E.
Consultant Reviewer



Andrew R. Blaisdell, P.E.
Associate Principal

NVW/CLS/ARB:erc

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1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the proposed replacement of Maine Department of Transportation (MaineDOT) Shortridge Road Bridge No. 2765 over Saco River Overflow in Fryeburg, Maine. Our work was completed in accordance with Master Services Agreement 40839 between Stantec and GZA, and Stantec Task Order JN 195311568, dated February 27, 2018, which incorporates GZA's proposal No. 09.P000100.18, dated February 15, 2018, and the attached *Limitations* included in **Appendix A** of this report.

GZA is providing geotechnical engineering services as a Subconsultant to Stantec, who is under contract with MaineDOT for design of the proposed bridge replacement.

1.1 BACKGROUND

Shortridge Bridge No. 2765 carries Route 302 over the Saco River Overflow in Fryeburg, Maine. The existing bridge was constructed in the 1930s and consists of an approximate 20-foot single-span bridge with a cast-in-place concrete deck. The bridge is supported on concrete abutments bearing on driven piles. Based on the pile capacities and our experience with similar-era bridges, we anticipate the bridge is supported on timber piles. The bottoms of the pile caps are approximately 16 feet below existing roadway elevation.

Based on our discussion with Stantec and the 90 percent PS&E plan set dated October 10, 2018, the replacement bridge is anticipated to be a 58-foot long, single-span bridge consisting of precast concrete beams with integral abutment substructures, the location of which is shown on **Figure 2, Boring Location Plan**. The abutment height will be approximately 10 feet, with bottom-of-pile cap elevations of El. 372 and El. 371 at Abutment 1 and Abutment 2, respectively. Elevations in this report are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

Each abutment will be supported on four piles. Based on information provided by Stantec, the bridge designer, we understand the maximum factored axial pile load is 322.5 kips for the strength loading condition and a 4-pile configuration. Stantec estimated that thermal deformation of the bridge superstructure could result in 0.27 inches of lateral deflection at each abutment.

The horizontal alignment of the roadway and bridge in the project area will not be significantly modified as part of the project. The approach roadway will be filled to approximately 3.5 feet above existing grades, and the roadway will be widened at the bridge approaches to match the increased bridge width of approximately 36 feet. Approach embankment modifications will also include placing additional fill along the sides of the approach embankments to facilitate the approach to the widened bridge and provide a slope inclination of 2 horizontal to 1 vertical (2H:1V) in the vicinity of the bridge, resulting in the new toe of the approach embankments up to about 25 feet outside of the current toe. The bridge opening at water level will be increased significantly, which will be achieved by removal of the existing abutments and excavation of soil on the water side of the new abutments to proposed grades. Riprap will be placed in front of each abutment and along the lower portion of embankment slopes.



The project plans call for traffic to be maintained on the existing bridge during staged construction, with one lane remaining open to alternating one-way traffic on Route 302.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and provide geotechnical design recommendations and construction considerations for bridge reconstruction. To meet these objectives, GZA completed the following Scope of Services:

- Conducted site visits to observe surficial conditions, traffic and boring access;
- Coordinated and observed a subsurface exploration program, consisting of two test borings, to evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate engineering properties of the site soils;
- Completed geotechnical evaluations for soil and bedrock properties; stability and settlement of approach embankments; frost susceptibility and drainage of approach embankments; AASHTO LRFD load and resistance factors associated with geotechnical design elements; nominal resistance of pile foundations; pile drivability; lateral pile evaluations; and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for driven piles, lateral earth pressures and seismic design parameters; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a subsurface exploration program consisting of two (2) test borings (BB-FSRO-101 and BB-FSRO-102). One boring was drilled behind each abutment in opposite lanes near planned replacement bridge abutment locations. The as-drilled boring locations were surveyed by MaineDOT and provided to GZA and are shown on **Figure 2**.

The borings were drilled to depths of approximately 69 to 70 feet below ground surface (BGS) and terminated approximately 5 feet into bedrock. New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. The drilling was completed between March 6 and March 16, 2018 using a truck-mounted drill rig. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**.

The borings were drilled using 3- and 4-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with a calibrated automatic hammer (NEBC No. 3, rated hammer efficiency factor = 0.869). Bedrock cores were obtained using NX2 coring equipment in each test boring. Wet and dry photographs of the rock core specimens are presented in **Appendix C**.

Upon completion, the borings were backfilled with cuttings and asphalt cold patch and marked for MaineDOT Survey. The as-drilled locations were surveyed by MaineDOT surveyors and provided to GZA.



3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete a soil testing program to assess the gradation and engineering characteristics of the soil. The testing program consisted of ten (10) gradation analysis/AASHTO Classification/Frost Classification assessments, one (1) hydrometer test, one (1) Atterberg limit evaluation, ten (10) moisture content determinations, and two (2) organic content determinations on split-spoon samples from the borings. Results of the testing are included in **Appendix D**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available surficial geology mapping¹, the site is mapped within a Stream Alluvium deposit that follows the general alignment of the Saco River, which is roughly 700 feet north of the site. This unit consists of silt, sand, gravel and organic material deposited by modern streams. An area of Till is mapped within the Stream Alluvium deposit to the west of the bridge. Large boulders are indicated in the mapped Till unit northwest of the bridge location (between the Saco River and Route 302). Lake Pigwacket lacustrine deposits are also mapped in the site vicinity, which contain sand, silt, clay and gravel deposited in successive stages during late-glacial to postglacial time.

Based on available bedrock geologic mapping², the site is underlain by granite of Carboniferous age, which is part of the Sebago Pluton.

4.2 SUBSURFACE PROFILE

Four soil units were encountered in the test borings beneath surficial pavement and overlying bedrock: Fill, Stream Alluvium, Lacustrine Silt, and Lacustrine Sand. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. The subsurface conditions are also shown in relation to the bridge alignment on the interpretive subsurface profile in **Figure 3**.

¹ Thompson, Woodrow B., 1999, Surficial geology of the Fryeburg 7.5-minute quadrangle, Oxford County, Maine: Maine Geological Survey, Open-File Report 99-8.

² Osberg, Philip H., Hussey II, Arthur M., Boone, Gary, M., 1985. Bedrock Geologic Map of Maine, Maine Geological Survey, Department of Conservation, map, scale 1:500,000.



Subsurface Unit	Approximate Encountered Thickness (ft)	Generalized Description
Asphalt Pavement	0.5 to 0.7	Asphalt Pavement Encountered in both borings
Fill	7 to 8	Variable <u>from</u> Brown, loose to very dense, fine to coarse SAND, little Silt (USCS: SP-SM, SM), <u>to</u> Sandy GRAVEL, trace Silt (USCS: GP-GM). - MaineDOT Frost Classification = I, II - Encountered in both borings
Stream Alluvium	7 to 20	Variable <u>from</u> Brown/Grey, loose to very loose, Silty fine to medium SAND, trace Gravel (USCS: SM), <u>to</u> very soft to soft Sandy SILT (USCS: ML). Traces of wood, wood fibers, organic fibers, and organic odor. - MaineDOT Frost Classification = IV - Encountered in both borings
Lacustrine Silt	3 to 10	Grey to Olive, stiff, Clayey SILT, trace to some fine Sand (USCS: ML). Encountered in both borings
Lacustrine Sand	26 to 45	Tan, medium dense to very dense, fine to coarse SAND, trace to little Silt, trace Gravel. (USCS: SP, SM) Encountered in both borings
Top of Bedrock Elevation	<u>Encountered Top of Rock: Approx. El. 313.7 to 316.2</u>	

4.2.1 Bedrock

Bedrock included Granite of the Sebago Pluton and an apparent Basalt dike, described as follows:

- Hard, fresh, fine grained to aphanitic, dark grey, BASALT. Joints are very close to closely spaced, low angle to horizontal, undulating, rough, discolored, partially open to open, found in BB-FSRO-101; and
- Hard, fresh, coarse grained, grey/pink/white GRANITE. Joints are close to moderately spaced, horizontal, planar, rough, fresh, partially open, found in BB-FSRO-102.

The Rock Quality Designation (RQD) ranged from 62 to 88 percent, averaging 75 percent, which indicates typically fair quality rock.

4.2.2 Groundwater

Groundwater was measured by GZA in the boreholes. The times, borehole depths, and casing conditions at the time of the measurements are provided in the boring logs. The measured groundwater level in BB-FSRO-101 and BB-FSRO-102 were approximately 12 and 18 feet below existing grade, respectively, corresponding to approximately El. 367.2 and El. 360.3, respectively. Water levels were generally measured immediately after drilling and were likely influenced by the wash boring drilling process. Soil samples were observed to be wet at and below 10 feet below grade (El. 368 to El. 369).

The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater will occur due to variations in temperature, season, precipitation, and construction activity in the area. Consequently, water levels during and after construction are likely to vary from those encountered at the time the observations were made.



5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations based on the 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition (AASHTO) and the MaineDOT Bridge Design Guide, 2003 Edition, with updates through 2018 (MaineDOT BDG). Supporting calculations developed by GZA for the project are attached in **Appendix E** of this report.

5.2 APPROACH EMBANKMENTS

The proposed bridge approaches will be constructed with maximum side slope angles of approximately 2H:1V, or flatter. The proposed approach embankments will be raised, requiring approximately 3.5 feet of fill placement above existing grade. The embankments will be constructed per MaineDOT standard specifications and details using engineered fill placed over the existing embankment/side slopes.

The anticipated generalized soil profile beneath each widened embankment is summarized below. The strength parameters for each soil unit were developed using the corrected blow count data and soil types from the borings. Supporting calculations are presented in **Appendix E**.

GENERALIZED SUBSURFACE CONDITIONS, PROPOSED APPROACH FILLS				
Soil Unit	Total Unit Weight (pcf)	Effective Friction Angle (deg) / Undrained Shear Strength (psf)	Estimated Thickness ¹ (feet)	
			West Approach (Abutment 1)	East Approach (Abutment 2)
New Fill (Sand, Gravel)	125	34	3	3
Road Base (Sand, Gravel)	125	39	4.5	4.5
Fill (Sand, Gravel)	125	30	4	4
Stream Alluvium (Silty Sand, Sandy Silt)	105	27	7	20
Lacustrine Silt (Clayey Silt)	120	Su = 1000 psf	3	10
Lacustrine Sand (Sand)	130	40	44.5	25.5

Note:

1. Estimated thickness is measured from existing approach embankment pavement grades.

5.2.1 SETTLEMENT

We anticipate that settlement from the planned 3.5 feet of new fill will occur elastically as the load is applied. Therefore, post-construction settlement is anticipated to be negligible.

5.2.2 GLOBAL STABILITY

GZA completed stability analyses to assess the factors of safety against rotational instability of the proposed slope modifications.

GZA evaluated the stability of the proposed approach embankments statically and pseudostatically, using the computer analytical software *Slope/W 2012*, developed by GeoSlope International, based on the Modified Bishop method. The analyzed cross-section is at Sta. 289+75 Left, corresponding to the longest and highest widened embankment area at the east approach. This location was selected as the critical



section because it also contains the greatest thickness of lower strength stream alluvium deposit. The proposed 2H:1V embankment slope inclination was modeled. A grid and radius search technique was used to identify the slip surface with the lowest factor of safety. The slope geometry and results of analyses are included in **Appendix E**.

The static analysis showed a factor of safety against rotational failure of 1.4 at Sta. 289+75 Left, which corresponds to an effective resistance factor of approximately 0.75 for Service conditions, meeting the requirements in Section 11.6.2.3 for slopes that do not contain structural elements.

The pseudostatic analysis included earthquake loading based on the Newmark method by incorporating a horizontal seismic coefficient, k_h , as defined as half of the maximum peak ground acceleration at the ground surface, in accordance with AASHTO 11.6.5.2.2. For this analysis, the stability is evaluated for the slip surface that is shown to have the lowest safety factor in the static analysis, and the basis of the calculation is that a factor of safety of at least 1 is adequate to protect the embankment from large deformations. The calculated factor of safety against rotational failure is approximately 1.1 for the critical slip surface, indicating that significant slope movement is not likely. This does not consider potential effects of liquefaction-induced movement and/or strength loss, which are discussed subsequently.

5.3 EVALUATION OF FOUNDATION TYPES

5.3.1 Abutment Foundations

Based on constructability and cost considerations, Stantec selected an integral abutment bridge supported on steel H-piles. Given the depth to rock and the nature of the overburden soils, we anticipate that the piles will likely be supported by a combination of skin friction in granular soil and end bearing on or near bedrock.

5.4 SEISMIC DESIGN CONSIDERATIONS

Seismic site class was evaluated in accordance with the 2017 AASHTO LRFD, along with consideration of the 2011 AASHTO Guide Specifications for LRFD Bridge Design (Seismic Guide Specification).

The subsurface profile for seismic design includes the proposed approach fills and soil strata described above. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1 using the average SPT N-value from the soil materials encountered in the borings. The average SPT N-value fell above 15 blows per foot for BB-FSRO-101 and below 15 blows per foot for BB-FSRO-102. Considering the very loose nature of the soil in BB-FSRO-102, we recommend that the site be assigned to Site Class E.

5.4.1 Liquefaction and Associated Hazards

Liquefaction evaluation is not specifically required in AASHTO for Seismic Design Category (SDC) B bridges. However, Article C6.8 of the AASHTO Seismic Guide Specification indicates that “a liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact bridge stability and is greater than or equal to 0.15 g.”

Based on liquefaction triggering analyses using the Boulanger and Idriss, 2008 empirical method, portions of the stream alluvium stratum sampled in the borings are susceptible to liquefaction during the design earthquake. Based on the SPT N-values and fines content of the stream alluvium at the west approach,



the minimum factor of safety against liquefaction triggering is approximately 1.1, and the estimated liquefaction-induced settlement of the 8.5-foot thick stratum is approximately ½ inch. The stream alluvium is weaker and thicker beneath the east approach embankment than the west approach, with factors of safety against liquefaction-triggering ranging from 0.9 to 1.1 within the 20-foot-thick stratum, including 10 feet of the stratum with safety factors below 1. Post-liquefaction ground surface settlement at the east approach could be on the order of 6 inches following the design event. Article 5.9.4 of the BDG states that a maximum allowable seismic settlement of 6 inches is acceptable. Therefore, we conclude liquefaction mitigation is not necessary to meet the requirements of the BDG.

Potentially liquefiable soils are present at the groundwater level in both borings, corresponding to approximately El. 368 to El. 369. This layer is partially within or directly below the elevation of the riverbank, indicating lateral displacement/spreading of the embankment slopes toward the stream is likely, following a design-level earthquake. The potentially liquefiable soil will be located adjacent to the approximately lower 2 to 3 feet of the integral abutments and upper 7 to 18 feet of the piles.

The primary effects of liquefaction on the bridge would be additional lateral load acting against the integral backwalls, soil strength loss within the stream alluvium stratum, and application of temporary downdrag loading on piles at Abutment 2 as the liquefied soil experiences settlement. Excavation and backfill for backwall construction will result in replacement of all of the liquefiable soil directly adjacent to the bridge. Therefore, the potential for lateral soil displacement to impose loading in excess of passive earth pressure on the backwalls is considered low. Downdrag loading is evaluated and discussed in **Sections 5.5.2 and 5.5.3.**

Based on these considerations, the implications of liquefaction are not anticipated to affect bridge design.

5.5 EVALUATION OF FOUNDATIONS

5.5.1 Foundation Type Assessment

GZA conducted preliminary evaluations for foundation support of the integral abutments in support of Stantec's preparation of the Preliminary Design Report. Based on the results of our preliminary evaluations, Stantec selected an integral abutment bridge supported on driven steel H-piles. Design considerations and evaluations are presented below.

5.5.2 Pile Design Considerations

The axial geotechnical resistance of piles was calculated using the computer analytical software *APile* by Ensoft based on the Nordlund method with a critical depth (10B) in accordance with LRFD Section 10.7. The results indicate that the piles will gain on the order of 213 to 305 kips in side resistance when driven to/near the top of rock and that the remainder will be developed in bearing on or near bedrock. To achieve a nominal resistance of 496 kips, we estimate the side friction to represent approximately 43 to 62 percent of the nominal capacity. Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. Axial tensile geotechnical (uplift) resistance was not evaluated because the structural loads provided by Stantec do not include uplift loading on the piles.

By utilizing steel H-piles for support of the abutments, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.



At Abutment 2, downdrag loading will temporarily be induced on piles following a design-level earthquake caused by approximately 3 to 6 inches of liquefaction-induced settlement. We evaluated the temporary downdrag load based on the Beta method, assuming $\beta = 0.35$ for the liquefied soil, and estimated approximately 50 kips of downdrag load could be imposed on HP14 piles. The effect of temporary liquefaction-induced downdrag is discussed subsequently.

5.5.3 Load and Resistance Factors

Piles should be designed for the geotechnical resistance of the piles. In GZA's experience for piles gaining a significant portion of their geotechnical resistance on bedrock, the drivability resistance will control the geotechnical static resistance of the pile. The piles will be driven to a nominal resistance calculated by dividing the maximum factored pile load (Strength I load case) by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1. Resistance factors for service and extreme limit state design should be taken as 1.0.

Structural resistance of the piles should be checked at the strength limit state considering a resistance factor $\phi_c=0.50$, per AASHTO LRFD Article 10.7.3.2.3 for hard driving condition. Since the piles will be subject to lateral loading, the piles should also be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per LRFD Article 6.5.4.2, the axial resistance factor $\phi_{cc}=0.75$ and the flexural resistance factor $\phi_r=1.0$ should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), live load surcharge (LS) loads, and components and attachments (DC) loads using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength limit state foundation design.

5.5.4 Pile Type and Loading

Four steel HP14x89 H-section piles are planned to support each abutment. Stantec provided a maximum factored axial load of 322.5 kips per pile for the strength condition; therefore, piles should be installed to a nominal axial resistance of at least 496 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.65 for piles installed under hard driving conditions. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted during construction in accordance with AASHTO requirements to assess nominal geotechnical pile resistance.

Liquefaction-induced downdrag is considered in the Extreme Event I load case in accordance with AASHTO LRFD. Therefore, the load case considering the downdrag load would include load factors of 1.0 for bridge dead loads, and the geotechnical resistance would use a resistance factor of 1.0 relative to the geotechnical axial compressive resistance. Therefore, we anticipate that the temporary downdrag load of 50 kips is unlikely to influence geotechnical design. Stantec should check that the Extreme Event I pile loads included a factored downdrag load of 50 kips (load factor of 1) and confirm this assumption, and Stantec should also check that the downdrag load does not control structural design.



5.5.5 Preliminary Pile Drivability Analysis

The pile-driving criteria are expected to be established based on dynamic pile testing with signal matching analysis. The piles should be driven to a nominal capacity calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per LRFD Table 10.5.5.2.3-1.

GZA completed preliminary wave equation analyses to assess the drivability of an HP 14x89 pile with a nominal geotechnical resistance of 496 kips. Analyses were completed using a Delmag D22-02 diesel hammer with a ram weight of 4,850 pounds and a maximum rated energy of 48,500 foot-pounds (ft-lbs).

The analyzed pile lengths were selected assuming that the piles would be driven as deep as the encountered top of rock elevation at each abutment. The calculated side resistance was used as an input in wave equation analyses, including 62 and 43 percent contribution of skin friction resistance to the maximum factored pile resistance at Abutment 1 and 2, respectively. The results are summarized below.

Pile Location and Type	Embedded Pile Length	Driving System ¹	Required Nominal Geotechnical Resistance (kips)	Max Driving Stress (ksi)	Final Penetration Resistance (blows per inch)
Abutment 1 HP 14x89	56 feet	Delmag D22-02	496	35	7
Abutment 2 HP 14x89	57 feet	Delmag D22-02	496	36	8

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculated penetration resistance is within the MaineDOT preferred range of 6 to 15 blows per inch, the analyzed hammer system is judged acceptable to install the piles to the required nominal resistance noted. Results of the preliminary wave equation analyses are provided in **Appendix E**.

5.5.6 Lateral Pile Analysis

GZA developed soil profiles for lateral pile evaluations at each abutment. The profiles reflect the soil conditions encountered in the test borings.

L-PILE® INPUT PARAMETERS						
ABUTMENT 1, PILE LENGTH = 56 FT (NEAREST BORING BB-FSRO-101)						
Stratum	Soil Model	Top of Layer Elevation (ft-NAVD 88)	Layer Thickness (ft)	k (pci) / E50 / UC (psi)	φ' (deg)/ Su (psf)	γ _e (pcf)
Fill	Reese Sand	372	1.5	130	39	63
Stream Alluvium	Reese Sand	370.5	7	10	27	43
Lacustrine Silt	Matlock Clay	363.5	3	E ₅₀ = 0.008	1000 psf	58
Lacustrine Sand	Reese Sand	360.5	44.5	145	40	68



L-PILE® INPUT PARAMETERS ABUTMENT 2, PILE LENGTH = 57 FT (NEAREST BORING BB-FSRO-102)						
Stratum	Soil Model	Top of Layer Elevation (ft-NAVD 88)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	ν_e (pcf)
Fill	Reese Sand	371	1.5	130	39	63
Stream Alluvium	Reese Sand	369.5	20	10	27	43
Lacustrine Silt	Matlock Clay	349.5	10	$E_{50} = 0.008$	1000 psf	58
Lacustrine Sand	Reese Sand	339.5	26	145	40	68

GZA conducted lateral pile analyses based on a maximum thermal deflection of 0.27 inches, as provided by Stantec. Lateral pile analysis used the Ensoft, Inc. LPILE version 2016.9.06 software. We assumed a fixed-head condition (zero rotation) and imposed the estimated thermal deflection at the pile head. The ground surface was assumed at the bottom of abutment/top of pile for thermal contraction. The analysis used ASTM A572 Grade 50, HP 14x89 piles aligned for weak-axis bending parallel to the bridge centerline. The assumed axial load was 322.5 kips, representing the maximum factored axial load provided by Stantec. Our results are summarized in the table below.

L-PILE® RESULTS				
Pile Type	Axial Load (kips)	Shear Force for Lateral deflection of 0.27 in. (kips)	Moment at Pile Head (ft-kips)	Total Stress at Pile Head (ksi)
Abutment 1 HP 14x89	322.5	12.5	-61.7	28.2
Abutment 2 HP 14x89	322.5	11.6	-54.1	13.8

The results indicate that a plastic hinge will not form in the piles due to the imposed lateral pile deformation.

5.5.7 Frost Protection

Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1400, and with low-moisture content (<20 percent) soils, the estimated depth of frost penetration is approximately 6.6 feet. Consequently, new foundation levels should be set at least 6.6 feet below ground surfaces exposed to freezing temperatures.

Granular fill soils encountered near the surface at the abutments typically were classified as AASHTO A-1-a and A-1-b with MaineDOT Frost Classification from I to II, indicating they are considered low to moderate frost susceptibility. Since there was no evidence of significant pavement distress or heave, these materials are judged to be suitable for continued use beneath the approach roadway after reconstruction. In accordance with MaineDOT Standards, new backfill placed behind abutments will consist of non-frost-susceptible materials.



5.5.8 Lateral Earth Pressures

Thermal expansion of the bridge will cause the backwalls and wingwalls of the integral abutment to move toward the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil.

Based on the estimated thermal bridge expansion of 0.27 inches and the abutment height of approximately 10 feet, the calculated abutment rotation is 0.0023 feet/foot. In accordance with the requirements of the BDG Section 5.4.2.11, integral abutment reinforcement is to be designed for full Coulomb passive pressure if the wall rotation is greater than 0.005 feet/foot. Therefore, we conclude that Rankine passive earth pressure is appropriate for design. We recommend using a Rankine Coefficient of Passive Earth Pressure, $K_p = 3.25$ for design of backwalls and wingwalls. Design lateral earth pressure recommendations are provided in **Section 6.3** of this report. AASHTO Commentary C3.10.9.1 specifies that single-span bridges in Seismic Zone 2 are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter, except in front of integral abutments where the typical 1.75H:1V slope angle will be used. Slopes should be provided with loam and seed for permanent erosion protection. If steeper slopes are required, additional means should be employed to limit surface erosion, such as fiber erosion control matting or riprap. Riprap should also be provided where the embankment side slopes will be near or below typical water levels in the Saco River Overflow to protect from scour.

6.2 SEISMIC DESIGN

The United States Geological Survey Online Design Maps Tool was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class E) for a 7 percent probability of exceedance in 75 years as follows:

SITE CLASS E SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	2.5
F _a	2.5
F _v	3.5
A _s (Period = 0.0 sec)	0.248 g
SDs (Period = 0.2 sec)	0.487 g
SD1 (Period = 1.0 sec)	0.172 g



Per AASHTO Article 4.7.4.2, bridges in Seismic Zone 1 need not be analyzed for seismic loads, but the minimum requirements apply for superstructure connections and support lengths as specified in AASHTO Articles 4.7.4.4 and 3.10.9.

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill between new abutments and wingwalls and a 1.5H:1V plane extending up from the bottom of the abutment to the pavement subgrade should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - Rankine Coefficient of Passive Earth Pressure, $K_p = 3.25$ (use for design of backwalls and wingwalls)
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center.

6.4 PILE DESIGN

- The proposed abutments may be supported on HP14x89 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed through a combination of side friction and end-bearing on or near the bedrock surface.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance of 496 kips, calculated by dividing the maximum factored pile load of 322.5 kips by a resistance factor of 0.65.
- Preliminary wave equation analyses indicate that the piles can be driven to a nominal resistance of 496 kips using a diesel hammer with a rated energy of about 48,500 ft-lbs for the anticipated 55- to 60-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi ($0.9F_y$ for 50 ksi steel). The final penetration resistance was 7 to 8 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings and shown on the interpretive subsurface profile (approximately El. 316 at Abutment 1, and approximately El. 314 at Abutment 2). A provision is recommended in the drawings for extra pile



length to account for variability in the top of rock surface and the potential for piles to penetrate a short distance into the bedrock.

- We recommend that one pile at each abutment be dynamically tested at the end of initial drive to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require a restrike test on each test pile. In the event that the initial driving results are favorable and show sufficient excess resistance to allow for some relaxation, the restrike tests may be waived.
- Piles shall be spliced in accordance with MaineDOT Section 501.047.

7.0 CONSTRUCTION CONSIDERATIONS

This section describes geotechnical-related issues that have the potential to impact design and cost considerations for bridge construction.

7.1 SUPPORT OF EXCAVATION AND DEWATERING

Excavations for abutment foundations are anticipated to extend approximately 8 feet below existing pavement grades. It is our understanding that Route 302 will remain in service during construction of the new bridge. Therefore, we anticipate it will be necessary to install temporary sheet pile excavation support to allow excavation adjacent to vehicular traffic areas. In areas where sufficient space is available and water conditions permit, the excavation slopes may consist of sloped, open-cuts. In all cases, temporary excavations should comply with OSHA excavation safety requirements.

Considering the proximity of the required abutment excavations to the stream water level, management of water will be related to stream water levels at the time of construction. Considering that bottoms of backwall elevations at approximately El. 371 to El. 372 are within a few feet of the streambed level, water levels may be at or above the bottom of excavation level during construction. However, utilizing the existing abutments for excavation support, if feasible, should make it easier to control water levels. It may be desirable to over-excavate and place an 8- to 12-inch-thick crushed stone working mat to improve accessibility and allow dewatering.

We anticipate that the inflow of groundwater or surface water to excavations could be handled by open pumping from sumps installed at the bottoms of excavations if cofferdams are installed. Stacked sand bags or a porta-dam-type system may be sufficient to limit inflow of surface water in lieu of a sheet pile cofferdam, given the relatively small anticipated head. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the subgrade and permit concrete placement in-the-dry. Discharge of pumped groundwater and surface water should comply with all local, State, and federal regulations.

7.2 PILE INSTALLATION CONTROL

We recommend that the pile installation be controlled using wave equation analysis and field logging of the pile installation and that final penetration resistance be based on dynamic pile testing with signal matching analysis. As previously noted, the piles should be driven to a nominal capacity calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1.



7.3 PILE OBSTRUCTIONS

Although no evidence of cobbles and boulders was seen in the Fill in the test borings, it is possible that pre-drilling, pre-excavation or spudding may be necessary to bypass potential obstructions, such as boulders, rock fill, timber cribbing or existing foundations.

7.4 REUSE OF ON-SITE MATERIALS

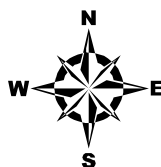
Based on the test boring results, two of the three fill samples tested had less than 10 percent passing the No. 200 sieve, indicating the fill may meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill and would be suitable for use as structural backfill. The material is also considered suitable for use as Common Borrow.

If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.

\\gza\port1\jobs\09 Jobs\0025900s\09.0025967.00 - MDOT Shortridge Fryeburg - Stantec\Report\GDR\FINAL 25967.00 Shortridge Bridge GDR 11202018.docx



FIGURES



USGS
QUADRANGLE
LOCATION

SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCSIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS MAP ALSO CONTAINS THE ESRI ARCGIS ONLINE USA COUNTIES WHICH PROVIDES DETAILED BOUNDARIES THAT ARE CONSISTENT WITH THE TRACT, BLOCK GROUP, AND STATE DATA SETS AND ARE EFFECTIVE AT REGIONAL AND STATE LEVELS.

0 1,000 2,000 4,000 6,000

SCALE IN FEET

Data Supplied by :



PROJ. MGR.: NVW
DESIGNED BY: NVW
REVIEWED BY: ARB
OPERATOR: ADM

DATE: 09-11-2018

LOCUS PLAN

MAINEDOT FRYEBURG SHORTRIDGE BRIDGE
#2765 WIN 23849.00

JOB NO.

09.0025967.00

FIGURE NO.

1

Date:11/29/2018

Username: common

Division: HIGHWAY

Filename: ... \GZA Figures\005_BLP.dgn

SHORTRIDGE BRIDGE NO. 2765 OVER SACO RIVER
OVERFLOW MAINEDOT WIN 23849.00
FRYEBURG, ME

BORING LOCATION PLAN

PREPARED BY:

GZA GeoEnvironmental, Inc.
Engineers and Scientists
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PREPARED FOR:

STANTEC

PROJ MGR:

NVW

REVIEWED BY:

ARB

CHECKED BY:

CLS

FIG

DESIGNED BY:

NVW

DRAWN BY:

NVW

SCALE:

AS SHOWN

DATE:

11/20/18

PROJECT NO.

09.0025967.00

REVISION NO.

0

SHEET NO.

1 OF 1

STATE OF MAINE

DEPARTMENT OF TRANSPORTATION

STP-2384(900)

WIN

23849.00

BRIDGE NO. 2765

BRIDGE PLANS



Andrew R. Baisell
SIGNATURE
10957
P.E. NUMBER
11/20/2018
DATE

PROJ. MGR.	DESIGN-DETAILED	CHECKED-REVIEWED	DESIGN-DETAILED	DESIGN-DETAILED	DESIGN-DETAILED	REVISIONS 1	REVISIONS 2	REVISIONS 3	REVISIONS 4	FIELD CHANGES
J. STILWELL	NVW	NVW	CLS	CLS	CLS					
DATE	NOV 2018	NOV 2018	NOV 2018	NOV 2018	NOV 2018					

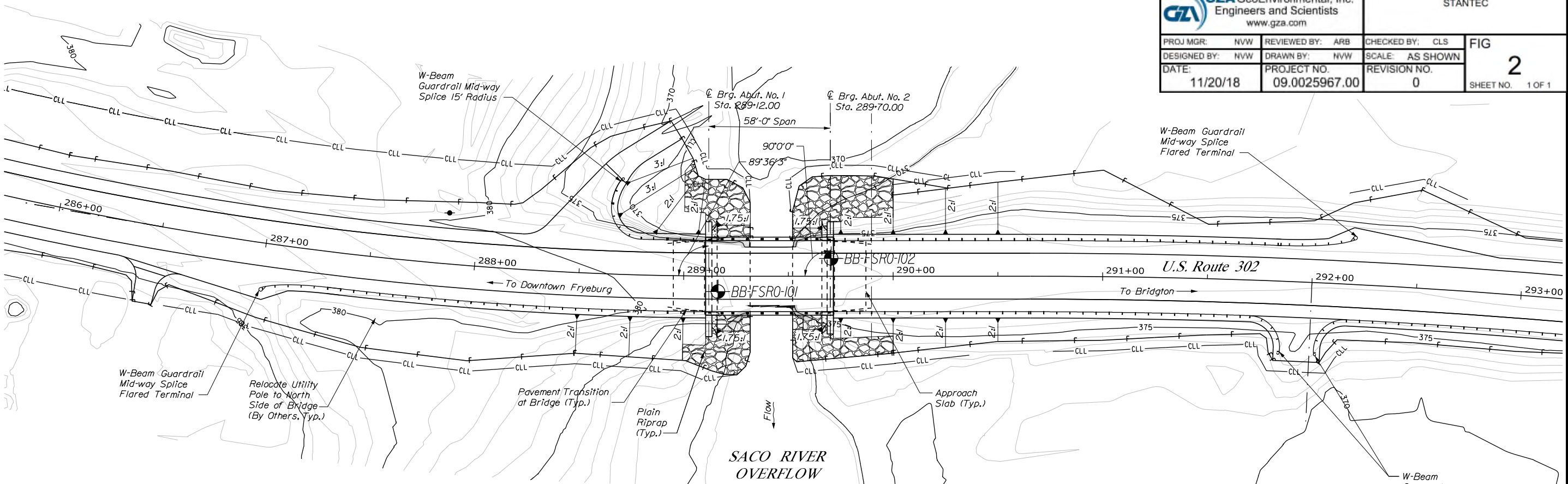
SHORTRIDGE BRIDGE
SACO RIVER OVERFLOW
FRYEBURG
OXFORD COUNTY
BORING LOCATION PLAN

SHEET NUMBER

A5

OF A20

PREPARED BY:



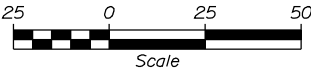
NOTES

- 1) Base map developed from electronic files (Text.dgn, Alignments.dgn, wetlands.dgn, Topo.dgn, and Contours.dgn) provided by Stantec on May 21, 2018, and from an electronic file (bridge.dgn) provided by Stantec on September 12, 2018.
- 2) The as-drilled locations of the test borings were surveyed by MaineDOT and provided by Stantec in an electronic file (Borings_03-29-18.dgn).
- 3) BB-FSR0-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between March 6 and 16, 2018.

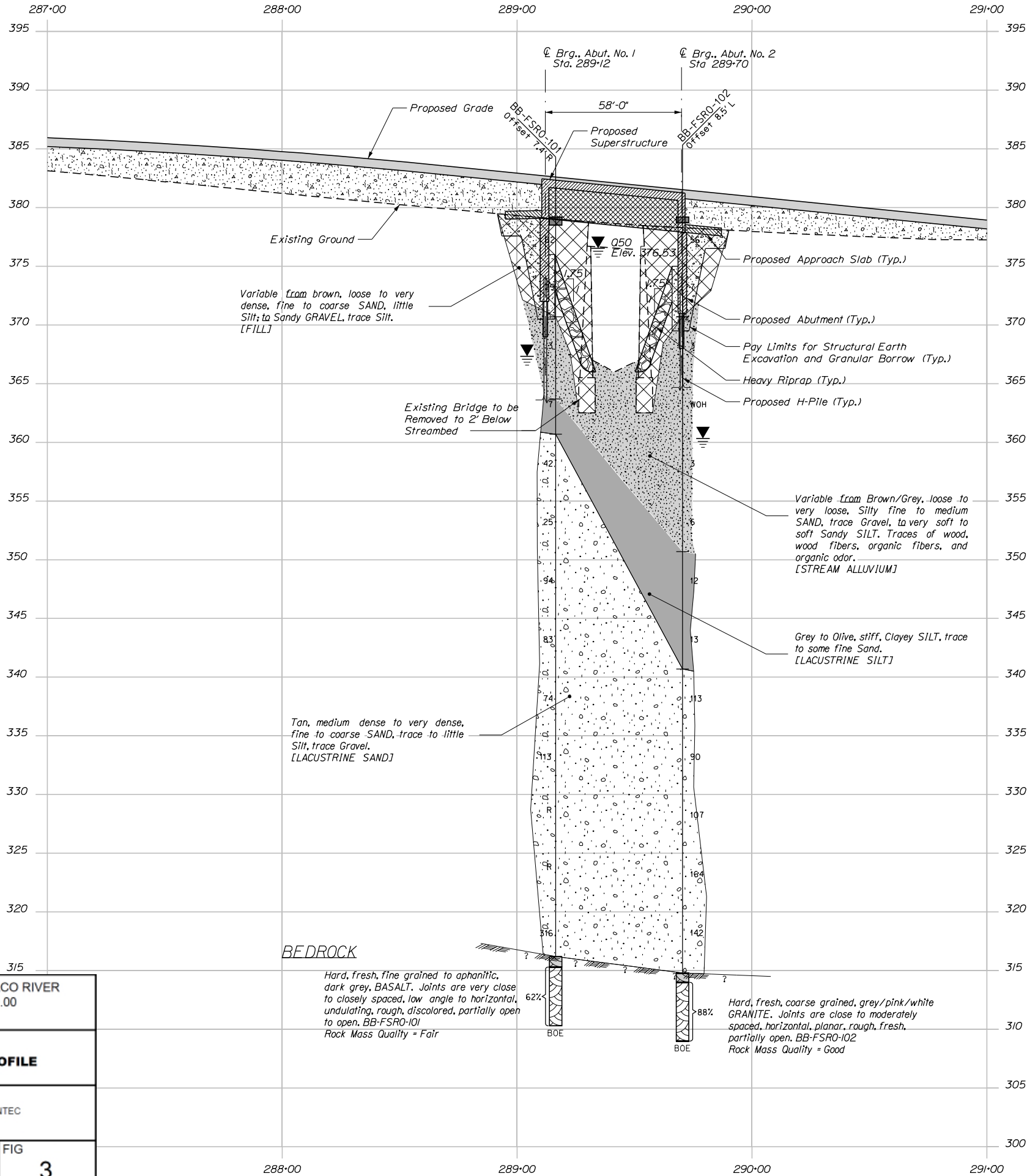
BORING LOCATION PLAN LEGEND

BB-FSR0-102 Location and designation of 100-Series cased wash boring

PLAN

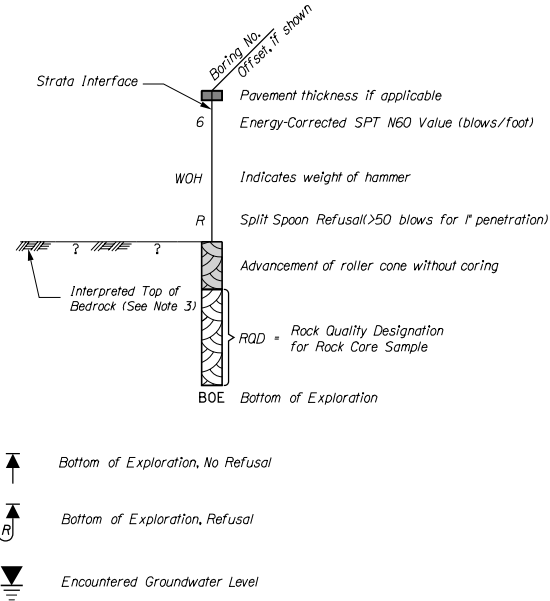


ELEVATION (FEET, NAVD 88)

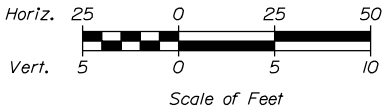


- NOTES**
- 1) Base map developed from electronic files provided by Stantec on May 25, 2018 (Files included Profiles.dgn)
 - 2) The as drilled locations of the test borings were surveyed by a MEDOT survey crew and supplied to GZA.
 - 3) BB-FSR0-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between March 6 and 16, 2018.
 - 4) This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

INTERPRETIVE SUBSURFACE PROFILE LEGEND



SHORTRIDGE BRIDGE PROFILE



SHORTRIDGE BRIDGE NO. 2765 OVER SACO RIVER
OVERFLOW MAINTENANCE WIN 23849.00
FRYEBURG, ME

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
STANTEC

PROJ MGR: NVW
DESIGNED BY: NVW
DATE: 11/20/18

REVIEWED BY: ARB
DRAWN BY: NVW
PROJECT NO. 09.0025967.00

CHECKED BY: CLS
SCALE: AS SHOWN
REVISION NO. 0

FIG
3
SHEET NO. 1 OF 1

PREPARED BY:
GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2384(900)

BRIDGE NO. 2765
WIN 23849.00
BRIDGE PLANS

DATE: 11/20/2018
P.E. NUMBER 10657
SIGNATURE: [Signature]

SHORTRIDGE BRIDGE
ROUTE 302
FRYEBURG
OXFORD COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
A6
OF 20



APPENDIX A – LIMITATIONS



GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the contract documents, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.
3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.
4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.
6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.



7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.
8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.
9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

11. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.



APPENDIX B – GZA BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER SYSTEM					
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES						
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Descriptive Term		Portion of Total (%)			
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace	0 - 10				
					little	11 - 20				
					some	21 - 35				
					adjective (e.g. sandy, clayey)	36 - 50				
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	TERMS DESCRIBING DENSITY/CONSISTENCY					
		GC	Clayey gravels, gravel-sand-clay mixtures.							
		CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	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).					
		(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	Density of Cohesionless Soils					
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.	Standard Penetration Resistance N-Value (blows per foot)					
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Very loose	0 - 4				
					Loose	5 - 10				
				Medium Dense	11 - 30					
				Dense	31 - 50					
	SILTS AND CLAYS (liquid limit greater than 50)	OL	Organic silts and organic silty clays of low plasticity.	Very Dense	> 50					
		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	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.						
		CH	Inorganic clays of high plasticity, fat clays.							
		OH	Organic clays of medium to high plasticity, organic silts.	Approximate Undrained Shear Strength (psf)						
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.	Consistency of Cohesive soils		SPT N-Value (blows per foot)		Field Guidelines			
			Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates				
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					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 (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$					
					*Minimum NQ rock core (1.88 in. OD of core)					
					Correlation of RQD to Rock Mass Quality					
					Rock Mass 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 mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))					Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information										

Maine Department of Transportation Soil/Rock Exploration Log <u>US CUSTOMARY UNITS</u>				Project: Shortridge Bridge Replacement #2765 Route 302 Over Saco River Overflow		Boring No. BB-FSRO-101	
				Location: Fryeburg, Maine		PIN: 23849.00	
Driller:		New England Boring Contractors		Elevation (ft.)		379.2	
Operator:		Mike Porter		Datum:		NAVD 88	
Logged By:		N. Williams		Rig Type:		Mobile Drill B-53 (Truck)	
Date Start/Finish:		03-15-18 / 03-16-18		Drilling Method:		SSA / Drive & Wash	
Boring Location:		N436263.6, E843982.5		Casing ID/OD:		4/4.5" 3/3.5"	
Hammer Efficiency Factor: 0.869				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 = Insitu Vane Shear Test MV = Unsuccessful Insitu 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 = weight of rods WO1P = Weight of one person			
				S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected			
				S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			

Sample Information										Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
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.)				
0							SSA	378.5		-ASPHALT-		
	1D	24/7	1.0 - 3.0	18-23-20-16	43	62				Brown, dry, very dense, Gravelly fine to medium SAND, trace Silt. -FILL- (SP-SM)	G#1 A-1-b, SP-SM WC=2.2	
5	2D	24/16	5.0 - 7.0	11-14-16-10	20	29	✓			Brown, dry, medium dense, Gravelly fine to medium SAND, little Silt. -FILL- (SM)		
								69				
								36				
								24				
10	3D	24/18	10.0 - 12.0	1-1-1-1	2	3				Brown, wet, soft, Sandy SILT, traces of wood and fibers. -STREAM ALLUVIUM- (ML)	G#2 A-4, ML WC=61.4	
								10				
								5				
								8				
								13				
15	4D	24/12	15.0 - 17.0	1-1-4-10	5	7				Top 6": Brown, wet, loose, fine to medium SAND, some Silt. -STREAM ALLUVIUM- (SM)	A-4, CL-ML LL=29 PL=22 PI=7 WC=27.5	
								30		Bottom 6": Grey, wet, stiff, Clayey SILT. -LACUSTRINE SILT- (CL-ML)		
								42				
								66				
								91				
20	5D	24/14	20.0 - 22.0	8-14-15-19	29	42				Tan, wet, dense, fine to coarse SAND, little Gravel, little Silt. -LACUSTRINE SAND- (SM) Increased resistance at 23.5 ft through possible gravel during roller cone advancement.		
								60				
								70				
								84				
								61				
25								58				

Remarks:

- SSA to 5.0 ft. Drove 4" casing to 35.0 ft.
- Washing ahead from 35.0 to 63.0 ft prior to casing advancement.
- Water level measured immediately after casing was removed from hole.

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.

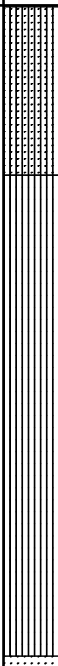
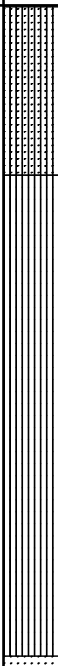
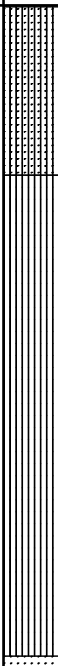
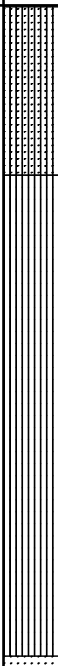
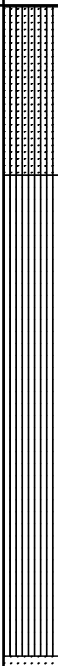
Page 1 of 3
 Boring No.: BB-FSRO-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Shortridge Bridge Replacement #2765 Route 302 Over Saco River Overflow				Boring No. BB-FSRO-101																																																																																																																																																																																																																																																											
				Location: Fryeburg, Maine				PIN: 23849.00																																																																																																																																																																																																																																																											
Driller: New England Boring Contractors				Elevation (ft.) 379.2				Auger ID/OD:																																																																																																																																																																																																																																																											
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55	12D	9/6	55.0 - 55.8	90-100/3"	R						Tan, wet, very dense, fine to medium SAND, little Silt. -LACUSTRINE SAND- (SM)																																																																																																																																																		
60	13D	24/10	60.0 - 62.0	77-97-122-120	218	316					Tan, wet, very dense, fine to medium SAND, little Silt. -LACUSTRINE SAND- (SM)																																																																																																																																																		
65	R1	60/50	63.9 - 68.9	RQD = 62%							316.2 315.3 63.0 63.9 65.1 67.0 67.5 68.9	Increase in resistance during roller cone advancement from 63.0 to 63.9 ft; possible weathered rock or cobbles/ boulder (blue gray rock chips in wash water). Practical refusal of roller cone at 63.9 ft; apparent top of rock, set up to core. R1: Hard, fresh, fine grained to aphanitic, dark grey, BASALT. Joints are very close to closely spaced, low angle to horizontal, undulating, rough, discolored, partially open to open. Two moderately wide joints with sand infilling at 67.0 and 67.5 ft. One high angle joint at 65.1 ft. Vertical contacts with GRANITE on side of core 63.9 to 64.1 ft, 66.4 to 66.5 ft, 67.1 to 67.7 ft. Rock Mass Quality = Fair -SEBAGO PLUTON- Recovery = 83% Rock Core Times (min:sec): 63.9-64.9' (2:52), 64.9-65.9' (2:29), 65.9-66.9' (2:12), 66.9-67.9' (2:54), 67.9-68.9' (3:17) Bottom of Exploration at 68.90 feet below ground surface.																																																																																																																																																	
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Shortridge Bridge Replacement #2765 Route 302 Over Saco River Overflow				Boring No. BB-FSRO-102																																																																																																																																																																																																																																																																																																																																																																																																										
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Shortridge Bridge Replacement #2765 Route 302 Over Saco River Overflow				Boring No. BB-FSRO-102							
				Location: Fryeburg, Maine				PIN: 23849.00							
Driller: New England Boring Contractors				Elevation (ft.) 378.1				Auger ID/OD:							
Operator: Mike Porter				Datum: NAVD 88				Sampler: Split Spoon							
Logged By: N. Williams				Rig Type: Mobile Drill B-53 (Truck)				Hammer Wt./Fall: 140/30							
Date Start/Finish: 03-06-18 / 03-12-18				Drilling Method: SSA / Drive & Wash				Core Barrel: NX							
Boring Location: N436293.6, E844030.2				Casing ID/OD: 4/4.5" 3/3.5"				Water Level*: 17.8'							
Hammer Efficiency Factor: 0.869				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 = Insitu Vane Shear Test MV = Unsuccessful Insitu 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 = weight of rods WQ1P = Weight of one person				S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected				S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	6D	24/0	25.0 - 27.0	1-2-2-2	4	6	23		349.6		No recovery.	
30	7D	24/18	30.0 - 32.0	2-5-3-11	8	12			339.6		Olive, wet, stiff, Clayey SILT, some fine Sand. -LACUSTRINE SILT- (ML)	
35	8D	24/0	35.0 - 37.0	5-5-4-5	9	13	25		339.6		No recovery.	
							25					
							42					
							37					
							77					
40	9D	24/18	40.0 - 42.0	12-40-38-37	78	113	96		339.6		Tan, wet, very dense, fine to medium SAND, some Silt, little Gravel. -LACUSTRINE SAND- (SM)	G#11 A-2-4, SM
							118					
							143					
							171					
							R/C					
45	10D	24/20	45.0 - 47.0	16-28-34-32	62	90			339.6		Tan, wet, very dense, medium to coarse SAND, trace Gravel, trace Silt. -LACUSTRINE SAND- (SP)	
50												

Remarks:

- SSA to 5.6 ft. Drove 4" casing to 44.0 ft.
- Fine gravel in washout return prior to 8D sampling.
- Groundwater level measured at approximately 12 hours after leaving site on 3/6/18 with casing at 64.0 ft.
- After advancing casing to 44.0 ft, used roller cone to wash ahead prior to casing advancement.
- Bottom of rock core did not break resulting in 53/60" of recovery.

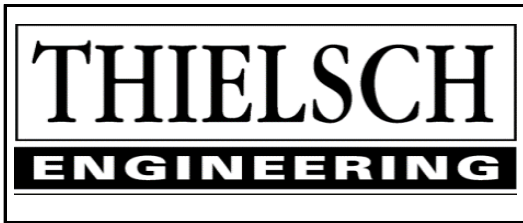
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
 * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 2 of 3
Boring No.: BB-FSRO-102

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Shortridge Bridge Replacement #2765 Route 302 Over Saco River Overflow			Boring No. BB-FSRO-102																																																																																																																																																																																														
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Logged By: N. Williams					Rig Type: Mobile Drill B-53 (Truck)			Hammer Wt./Fall: 140/30																																																																																																																																																																																														
Date Start/Finish: 03-06-18 / 03-12-18					Drilling Method: SSA / Drive & Wash			Core Barrel: NX																																																																																																																																																																																														
Boring Location: N436293.6, E844030.2					Casing ID/OD: 4/4.5" 3/3.5"			Water Level*: 17.8'																																																																																																																																																																																														
Hammer Efficiency Factor: 0.869					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 = Insitu Vane Shear Test MV = Unsuccessful Insitu 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 = weight of rods WO1P = Weight of one person			S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected			S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																																																																																																																																																																																											
<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="8">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing</th><th>Elevation (ft.)</th></tr><tr><td rowspan="3">50</td><td>11D</td><td>24/10</td><td>50.0 - 52.0</td><td>15-30-44-62</td><td>74</td><td>107</td><td></td><td></td><td rowspan="10"></td><td rowspan="10">Tan, wet, very dense, fine to medium SAND, trace Silt. Silt seams in sample. -LACUSTRINE SAND- (SP)</td><td rowspan="10"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">55</td><td>12D</td><td>24/14</td><td>55.0 - 57.0</td><td>26-53-60-79</td><td>113</td><td>164</td><td></td><td></td><td rowspan="10"></td><td rowspan="10">Tan, wet, very dense, fine to coarse SAND, trace Silt, trace Gravel. -LACUSTRINE SAND- (SP)</td><td rowspan="10"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">60</td><td>13D</td><td>24/14</td><td>60.0 - 62.0</td><td>23-44-54-49</td><td>98</td><td>142</td><td></td><td></td><td rowspan="10"></td><td rowspan="10">Tan, wet, very dense, fine to medium SAND, trace Silt. Silt seam in sample. -LACUSTRINE SAND- (SP)</td><td rowspan="10"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">65</td><td>R1</td><td>60/53</td><td>65.2 - 70.2</td><td>RQD = 88%</td><td></td><td></td><td>NX</td><td></td><td rowspan="10"></td><td rowspan="10">Increased roller cone resistance at 64.0 ft, possible cobbles. Casing refusal at 64.4 ft. Increased resistance during roller cone advancement at 64.4 ft, apparent top of rock. Practical refusal of roller cone at 65.2 ft, set up to core. R1: Hard, fresh, coarse grained, grey/ pink/white GRANITE. Joints are close to moderately spaced, horizontal, planar, rough, fresh, partially open. Rock Mass Quality = Good -SEBAGO PLUTON- Recovery = 88% Rock Core Times (min:sec): 65.2-66.2' (2:01), 66.2-67.2' (2:13), 67.2-68.2' (2:55), 68.2-69.2' (2:16), 69.2-70.2' (2:27)</td><td rowspan="10">Bottom of Exploration at 70.20 feet below ground surface.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">70</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="10"></td><td rowspan="10"></td><td rowspan="10"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">75</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="10"></td><td rowspan="10"></td><td rowspan="10"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>											Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Elevation (ft.)	50	11D	24/10	50.0 - 52.0	15-30-44-62	74	107				Tan, wet, very dense, fine to medium SAND, trace Silt. Silt seams in sample. -LACUSTRINE SAND- (SP)																		55	12D	24/14	55.0 - 57.0	26-53-60-79	113	164				Tan, wet, very dense, fine to coarse SAND, trace Silt, trace Gravel. -LACUSTRINE SAND- (SP)																		60	13D	24/14	60.0 - 62.0	23-44-54-49	98	142				Tan, wet, very dense, fine to medium SAND, trace Silt. Silt seam in sample. -LACUSTRINE SAND- (SP)																		65	R1	60/53	65.2 - 70.2	RQD = 88%			NX			Increased roller cone resistance at 64.0 ft, possible cobbles. Casing refusal at 64.4 ft. Increased resistance during roller cone advancement at 64.4 ft, apparent top of rock. Practical refusal of roller cone at 65.2 ft, set up to core. R1: Hard, fresh, coarse grained, grey/ pink/white GRANITE. Joints are close to moderately spaced, horizontal, planar, rough, fresh, partially open. Rock Mass Quality = Good -SEBAGO PLUTON- Recovery = 88% Rock Core Times (min:sec): 65.2-66.2' (2:01), 66.2-67.2' (2:13), 67.2-68.2' (2:55), 68.2-69.2' (2:16), 69.2-70.2' (2:27)	Bottom of Exploration at 70.20 feet below ground surface.																	70																												75																											
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Remarks: 1. SSA to 5.6 ft. Drove 4" casing to 44.0 ft. 2. Fine gravel in washout return prior to 8D sampling. 3. Groundwater level measured at approximately 12 hours after leaving site on 3/6/18 with casing at 64.0 ft. 4. After advancing casing to 44.0 ft, used roller cone to wash ahead prior to casing advancement. 5. Bottom of rock core did not break resulting in 53/60" of recovery.																																																																																																																																																																																																						
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.								Page 3 of 3 Boring No.: BB-FSRO-102																																																																																																																																																																																														



APPENDIX C – LABORATORY TEST RESULTS

	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 http://www.thielsch.com <i>Let's Build a Solid Foundation</i>	Client Information: Stantec Scarborough, Maine PM: Nicholas Williams Assigned By: Andrew Blaisdell Collected By: Nicholas Williams	Project Information: Shortridge Bridge Replacement #2765 WIN 23849.00 Fryeburg, ME GZA Project Number: 09.0025967.00 Summary Page: 1 of 1 Report Date: 03.30.2018
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LABORATORY TESTING DATA SHEET

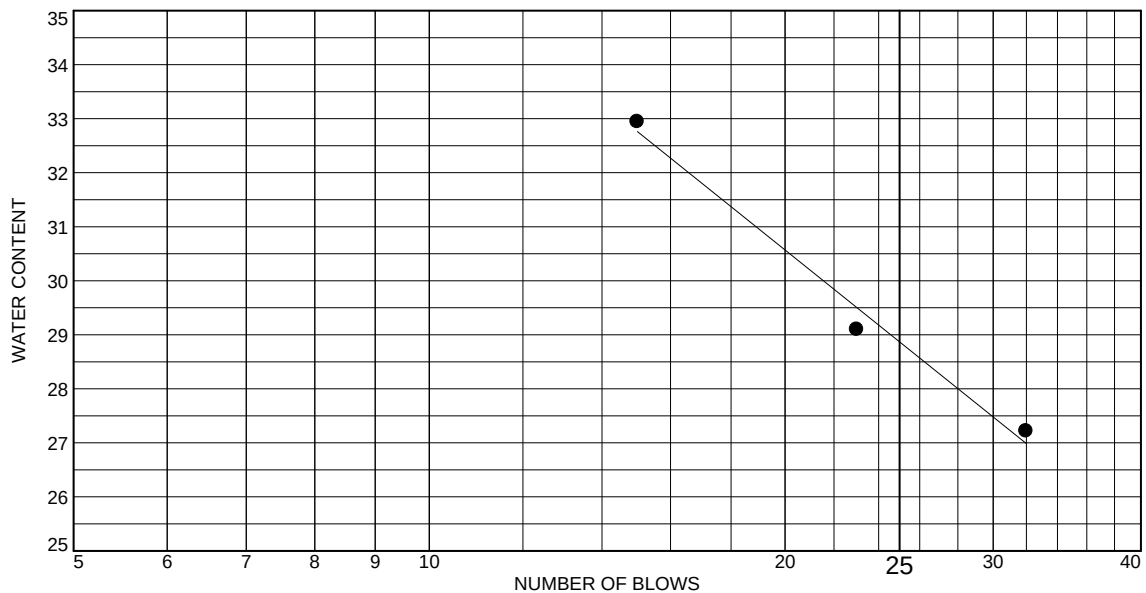
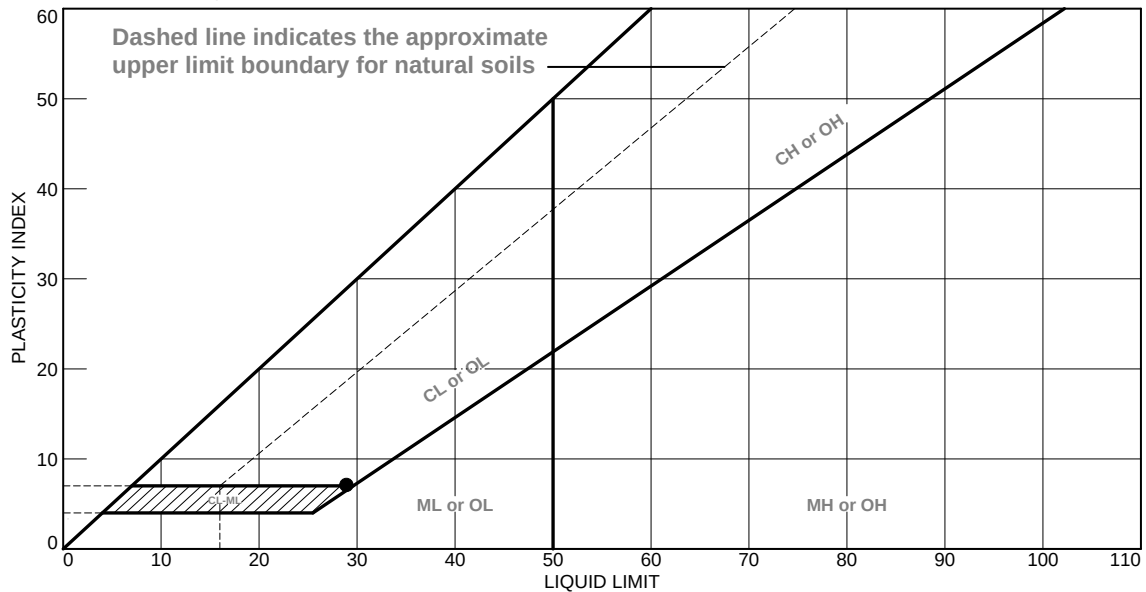
Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G _s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W _{opt} (%)	γ_d MAX (pcf) W _{opt} (%) (Corr.)	Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
BB-FSRO-101	1D	1-3	S-1	2.2			37.3	52.7	10.0											Brown Gravelly fine to medium Sand, trace Silt
BB-FSRO-101	3D	10-12	S-2	61.4			0.0	30.4	69.6											Brown Silt, some fine sand
BB-FSRO-101	4DB	15.5-17	S-3	27.5	29	22														Grayish Brown SILT & CLAY
BB-FSRO-101	6D	25-27	S-4	17.0			0.0	82.7	17.3											Grayish Brown fine to medium Sand, little Silt
BB-FSRO-101	9D	40-42	S-5	20.4			0	84.4	15.6											Brown fine to medium Sand, little Silt
BB-FRSO-102	1D	1-3	S-6	2.2			52.3	39.3	8.4											Brown Sandy fine to coarse Gravel, trace Silt
BB-FRSO-102	2D	5-7	S-7	5.8			0.0	84.3	15.7											Brown fine to coarse Sand, little Silt
BB-FRSO-102	3D	10-12	S-8	40.8			0.0	51.9	48.1											Brown Silty fine Sand
BB-FRSO-102	4D	15-17	S-9	51.1			0.0	37.6	62.4	2.5										Dark Gray Sandy Silt
BB-FRSO-102	5D	20-22	S-10	59.8			1.9	52.7	45.4	3.9										Dark Brown Silty fine Sand, trace fine Gravel
BB-FRSO-102	9D	40-42	S-11				12.8	60.0	27.2											Brown fine to medium Sand, some Silt, little Gravel

Reviewed By SKW

Date Reviewed 03.30.2018

State of Maine - Department of Transportation Laboratory Testing Summary Sheet

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish Brown SILT & CLAY	29	22	7			CL-ML

Project No. 09.0025967.00 **Client:** GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00

Fryeburge, ME

Source of Sample: BB-FSRO-101

Depth: 15.5-17'

Sample Number: 4DB

Thielsch Engineering Inc.

Cranston, RI

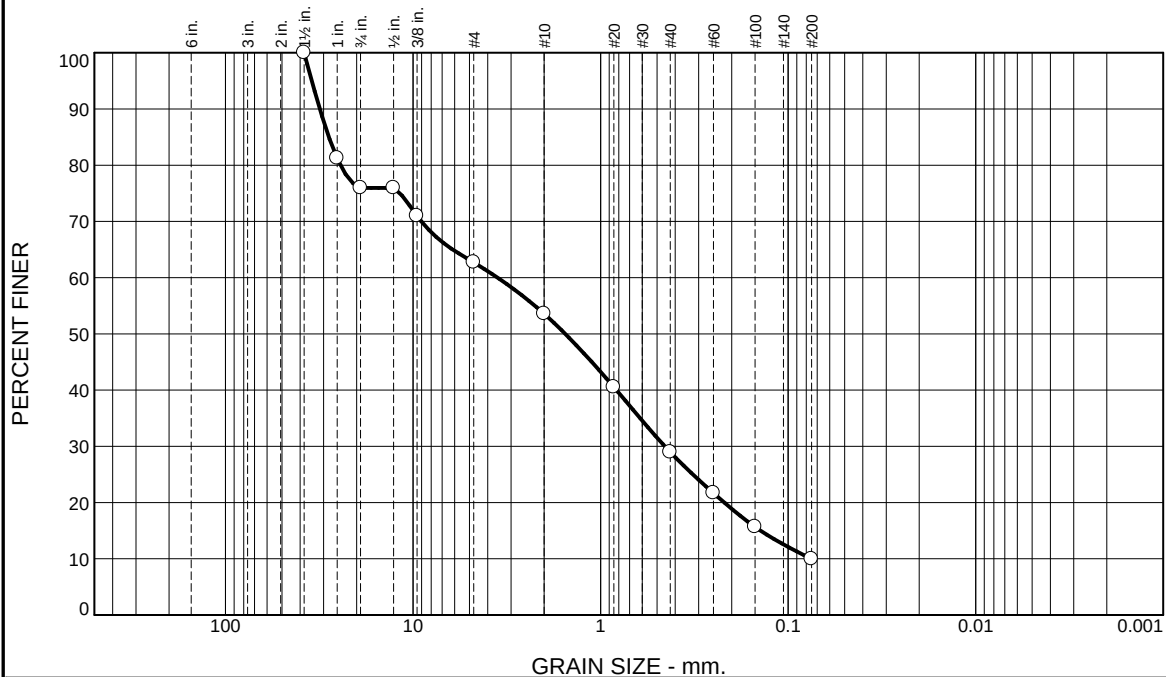
Remarks:

Figure L-3

Tested By: MN

Checked By: SA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	24.0	13.3	9.1	24.6	19.0	10.0	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	81.3		
0.75"	76.0		
0.5"	76.0		
0.375"	71.0		
#4	62.7		
#10	53.6		
#20	40.5		
#40	29.0		
#60	21.7		
#100	15.7		
#200	10.0		

* (no specification provided)

Material Description

Brown Gravelly fine to medium Sand, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP-SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 31.3809 D₈₅= 28.0937 D₆₀= 3.5421
D₅₀= 1.5428 D₃₀= 0.4542 D₁₅= 0.1402
D₁₀= 0.0754 C_u= 47.00 C_c= 0.77

Remarks

Sample had an as recieved moisture content of 2.2%.

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FSRO-101
Sample Number: 1D

Depth: 1-3'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

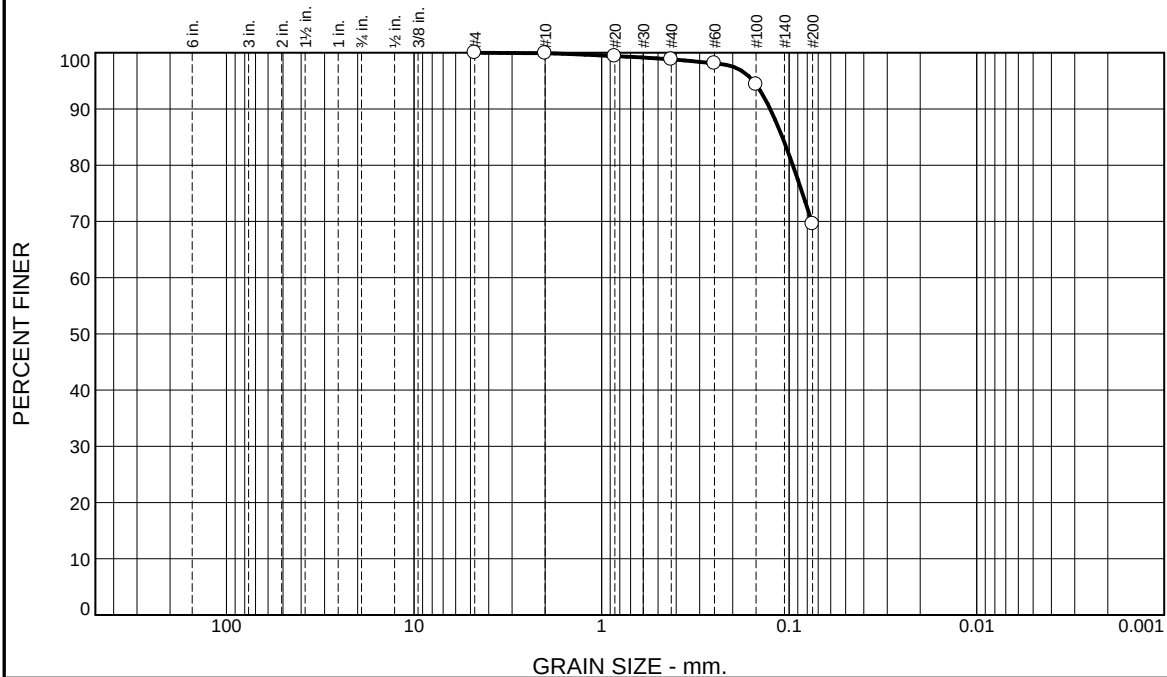
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	1.1	29.2	69.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.9		
#20	99.4		
#40	98.8		
#60	98.1		
#100	94.3		
#200	69.6		

* (no specification provided)

Material Description

Brown Silt, some fine sand

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.1261 D₈₅= 0.1087 D₆₀=
D₅₀= D₃₀= D₁₅=
D₁₀= C_u= C_c=

Remarks

Sample was visually classified as non-plastic.
Sample had an as recieved moisture content of 61.4%.

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FSRO-101
Sample Number: 3D

Depth: 10-12'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

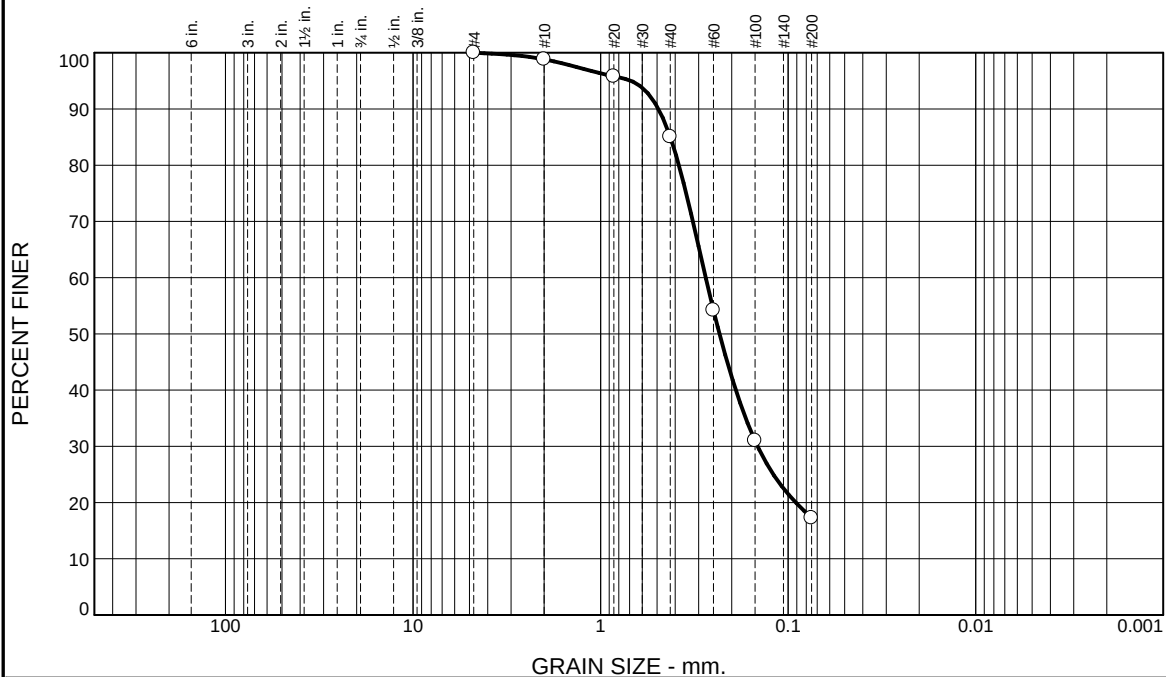
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.2	13.8	67.7	17.3	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	98.8		
#20	95.8		
#40	85.0		
#60	54.2		
#100	31.0		
#200	17.3		

* (no specification provided)

Material Description

Grayish Brown fine to medium Sand, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 0.4911 D₈₅= 0.4245 D₆₀= 0.2749
D₅₀= 0.2323 D₃₀= 0.1453 D₁₅=
D₁₀= C_u= C_c=

Remarks

Sample was visually classified as non-plastic.
Sample had an as recieved moisture content of 17.0%.

Date Received: 03.23.2018 Date Tested: 03.27.2018

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FSRO-101
Sample Number: 6D

Depth: 25-27'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

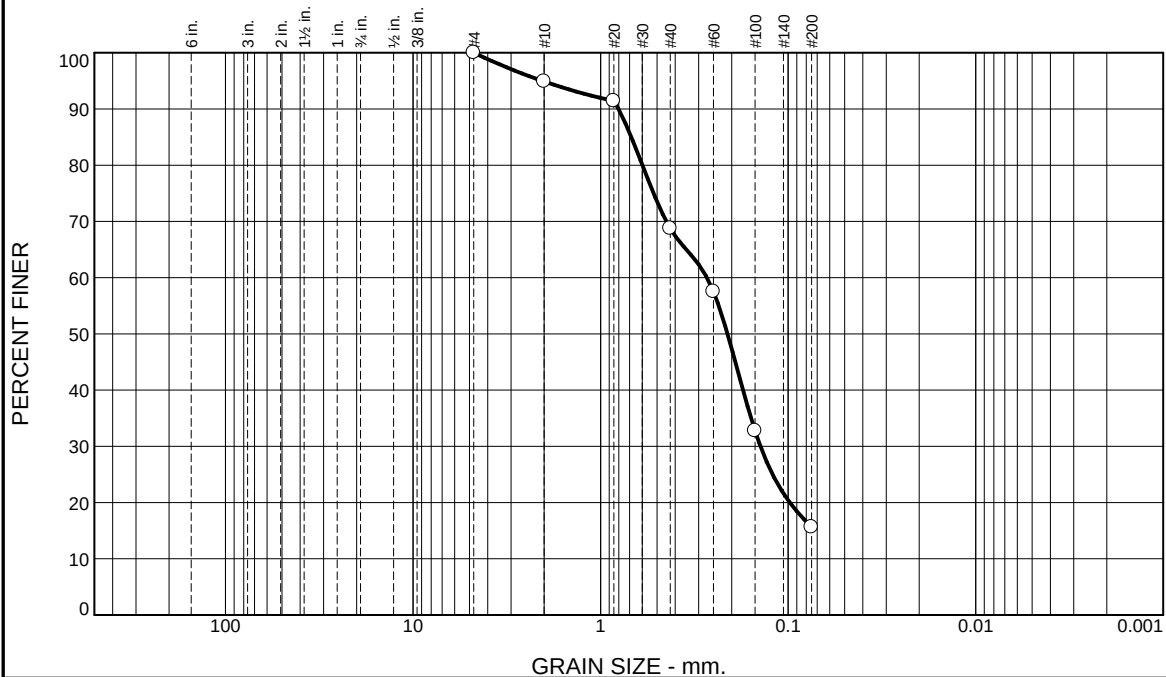
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	5.1	26.2	53.1	15.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	94.9		
#20	91.4		
#40	68.7		
#60	57.5		
#100	32.7		
#200	15.6		

* (no specification provided)

Material Description

Brown fine to medium Sand, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 0.8033 D₈₅= 0.6862 D₆₀= 0.2709
D₅₀= 0.2105 D₃₀= 0.1404 D₁₅=
D₁₀= C_u= C_c=

Remarks

Sample had an as recieved water content of 20.4%.

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FSRO-101
Sample Number: 9D

Depth: 40-42'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

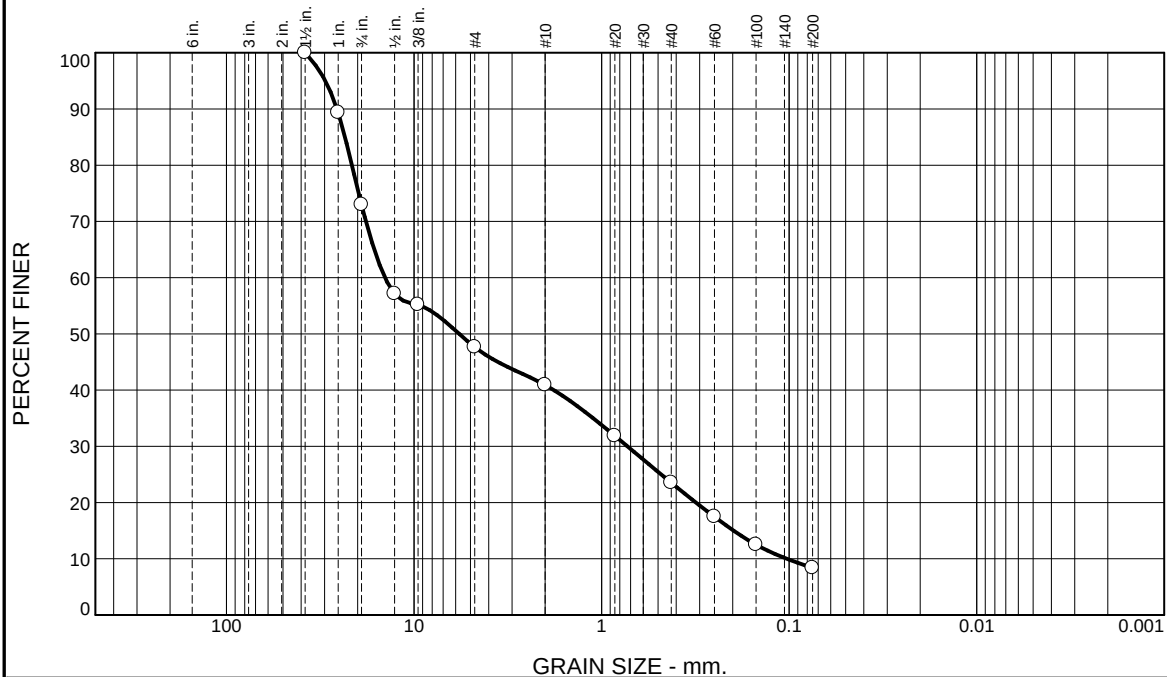
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-5

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	27.0	25.3	6.8	17.4	15.1	8.4	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	89.4		
0.75"	73.0		
0.5"	57.1		
0.375"	55.2		
#4	47.7		
#10	40.9		
#20	31.9		
#40	23.5		
#60	17.5		
#100	12.5		
#200	8.4		

* (no specification provided)

Material Description

Brown Sandy fine to coarse Gravel, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= GP-GM AASHTO (M 145)= A-1-a

Coefficients

D₉₀= 25.7613 D₈₅= 23.3418 D₆₀= 14.2969
D₅₀= 5.7430 D₃₀= 0.7283 D₁₅= 0.1977
D₁₀= 0.1028 C_u= 139.05 C_c= 0.36

Remarks

Sample had an as recieved water content of 2.2%.

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FRSO-102
Sample Number: 1D

Depth: 1-3'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

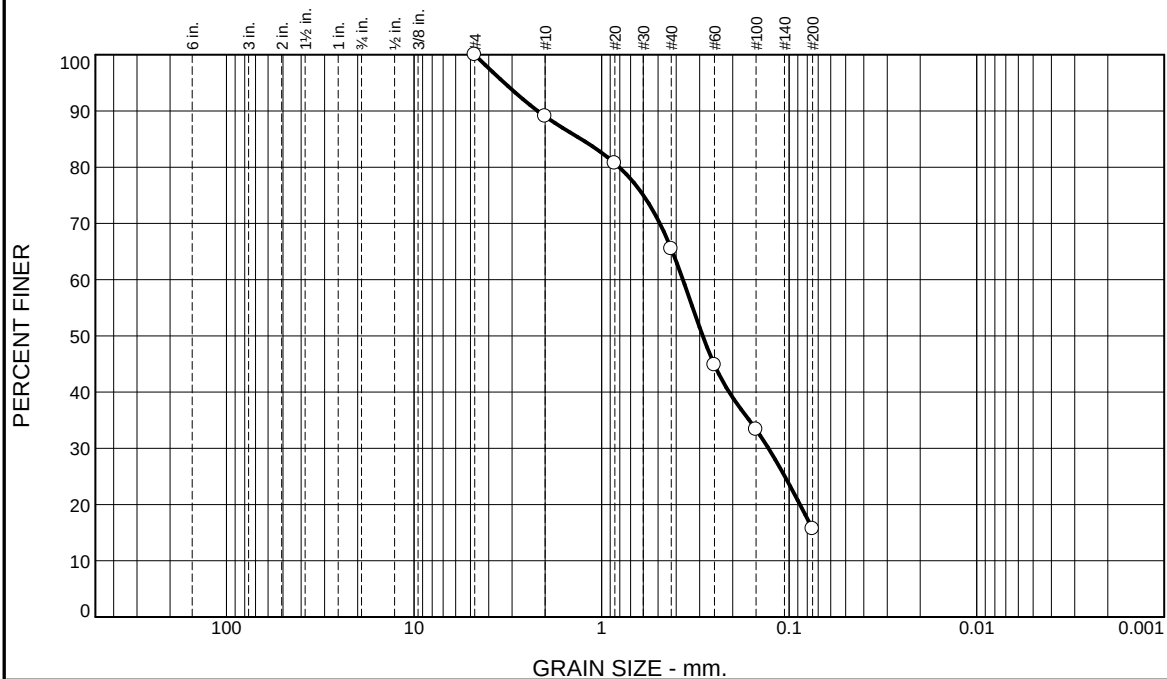
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburg, ME

Project No: 09.0025967.00

Figure S-6

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	11.0	23.5	49.8	15.7	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	89.0		
#20	80.7		
#40	65.5		
#60	44.8		
#100	33.3		
#200	15.7		

* (no specification provided)

Material Description

Brown fine to coarse Sand, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 2.1916 D₈₅= 1.2849 D₆₀= 0.3685
D₅₀= 0.2888 D₃₀= 0.1289 D₁₅=
D₁₀= C_u= C_c=

Remarks

Sample had an as recieved water content of 5.8%.

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FRSO-102
Sample Number: 2D

Depth: 5-7

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

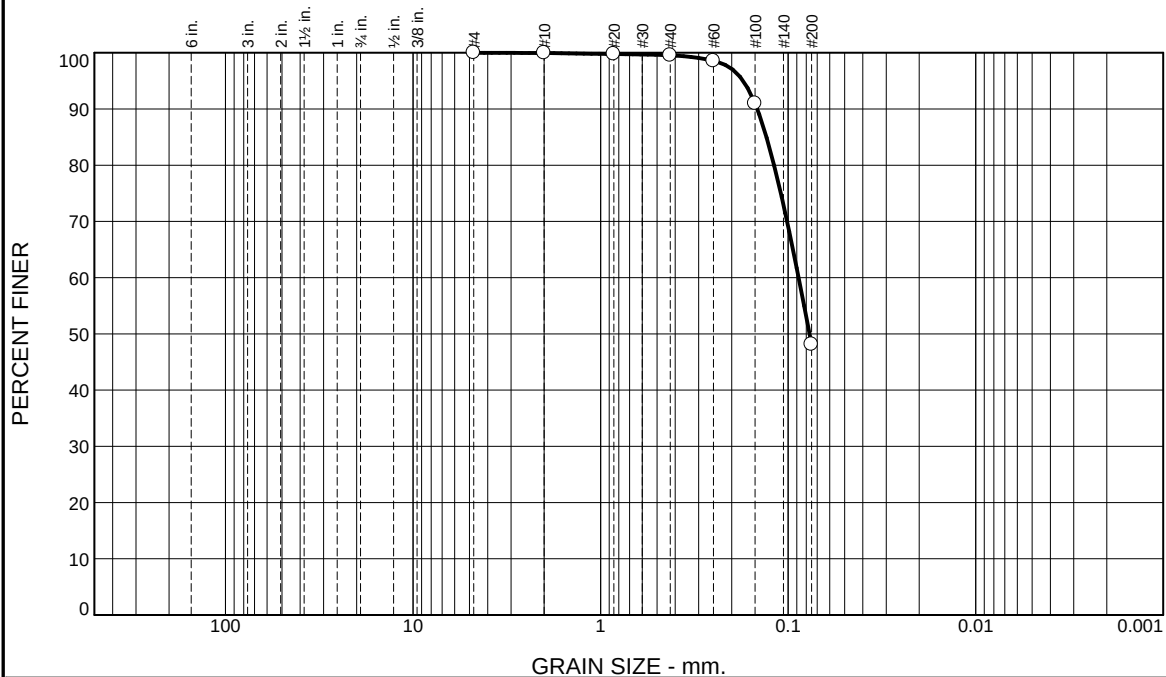
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.5	51.4	48.1	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	100.0		
#20	99.8		
#40	99.5		
#60	98.6		
#100	91.0		
#200	48.1		

* (no specification provided)

Material Description

Brown Silty fine Sand

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.1461 D₈₅= 0.1304 D₆₀= 0.0880
D₅₀= 0.0769 D₃₀= C_u=
D₁₀= C_c=

Remarks

Sample was visually classified as non-plastic.
Sample had an as recieved water content of 40.85.

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FRSO-102
Sample Number: 3D

Depth: 10-12'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

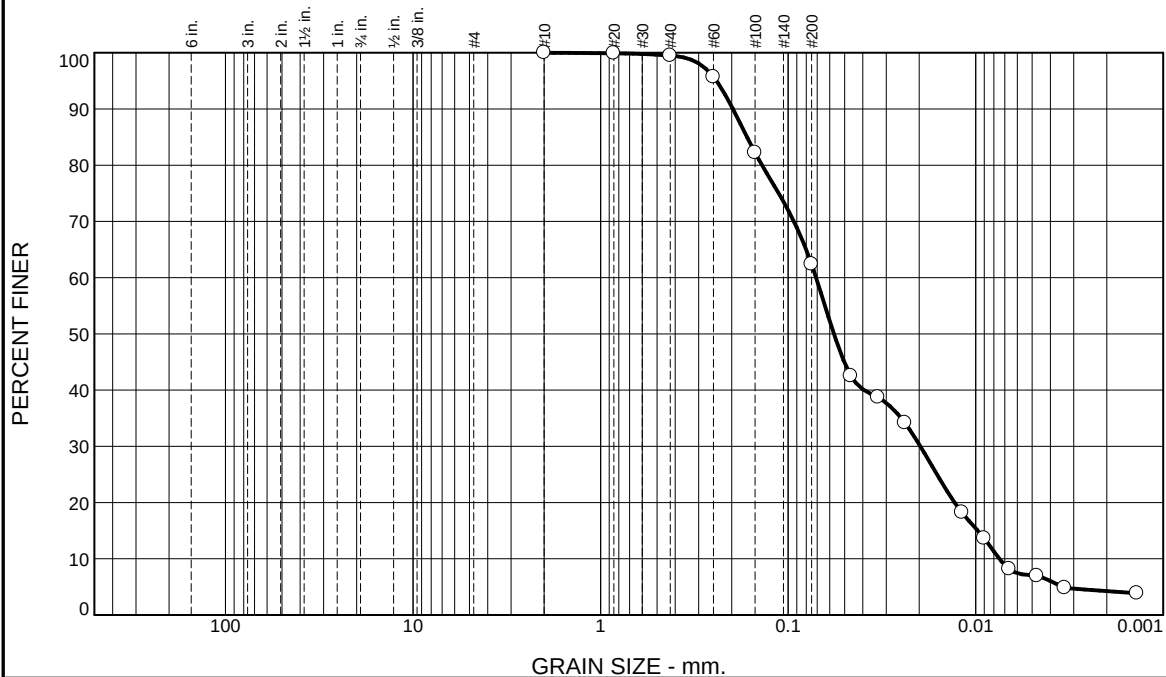
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.5	37.1	58.1	4.3

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#20	99.9		
#40	99.5		
#60	95.7		
#100	82.2		
#200	62.4		
0.0463 mm.	42.5		
0.0332 mm.	38.7		
0.0239 mm.	34.2		
0.0118 mm.	18.2		
0.0090 mm.	13.7		
0.0066 mm.	8.2		
0.0047 mm.	6.9		
0.0034 mm.	4.8		
0.0014 mm.	3.9		

* (no specification provided)

Material Description

Dark Gray Sandy Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.1969 D₈₅= 0.1656 D₆₀= 0.0709
D₅₀= 0.0570 D₃₀= 0.0197 D₁₅= 0.0097
D₁₀= 0.0075 C_u= 9.49 C_c= 0.73

Remarks

Sample had an as recieved water content of 51.1%

Date Received: 03.23.2018 Date Tested: 03.30.18

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: BB-FRSO-102
Sample Number: 4D

Depth: 15-17'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

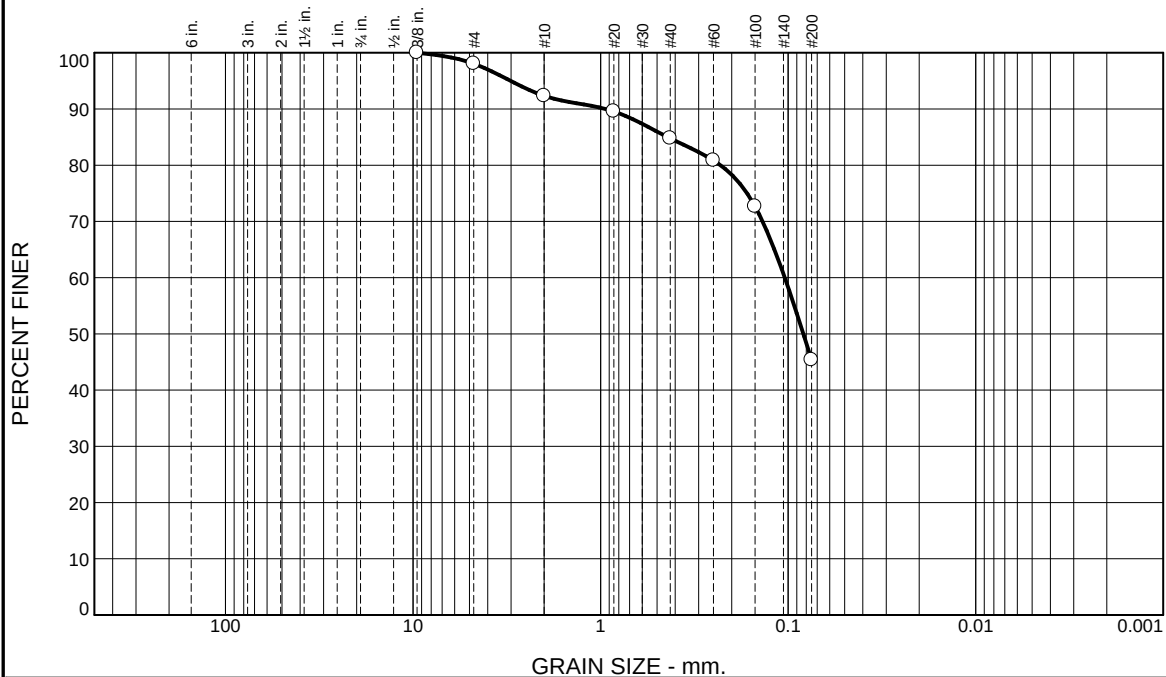
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-9

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.9	5.8	7.5	39.4	45.4	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375"	100.0		
#4	98.1		
#10	92.3		
#20	89.6		
#40	84.8		
#60	80.8		
#100	72.7		
#200	45.4		

* (no specification provided)

Material Description

Dark Brown Silty fine Sand, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.9401 D₈₅= 0.4384 D₆₀= 0.1043
D₅₀= 0.0830 D₃₀= C_u=
D₁₀= C_c=

Remarks

Sample had an as recieved water content of 59.8%.
Sample was visually classified as plastic, rolled to less than 1/4"

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FRSO-102
Sample Number: 5D

Depth: 20-22'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

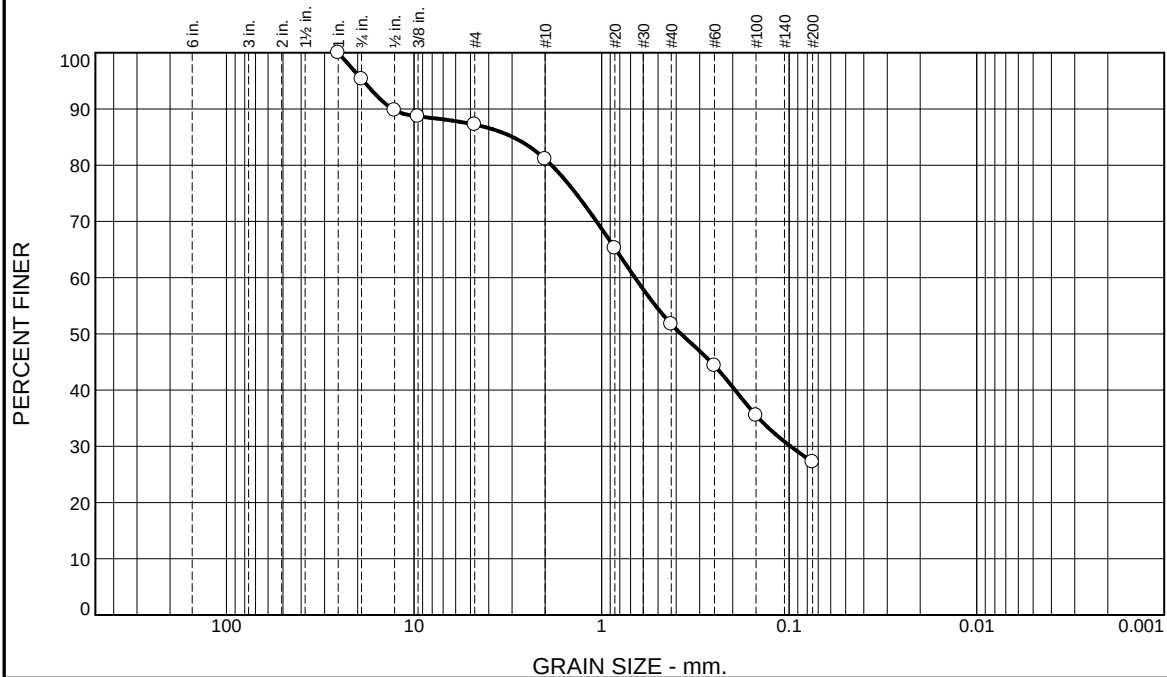
Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-10

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.7	8.1	6.1	29.4	24.5	27.2	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	95.3		
0.5"	89.8		
0.375"	88.7		
#4	87.2		
#10	81.1		
#20	65.2		
#40	51.7		
#60	44.4		
#100	35.5		
#200	27.2		

* (no specification provided)

Material Description

Brown fine to medium Sand, some Silt, little Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 13.0228 D₈₅= 2.9705 D₆₀= 0.6639
D₅₀= 0.3774 D₃₀= 0.0983 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 03.23.18 Date Tested: 03.27.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-FRSO-102
Sample Number: 9D

Depth: 40-42'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental / Stantec

Project: Shortridge Bridge Replacement #2765, Win 23849.00
Fryeburge, ME

Project No: 09.0025967.00

Figure S-11

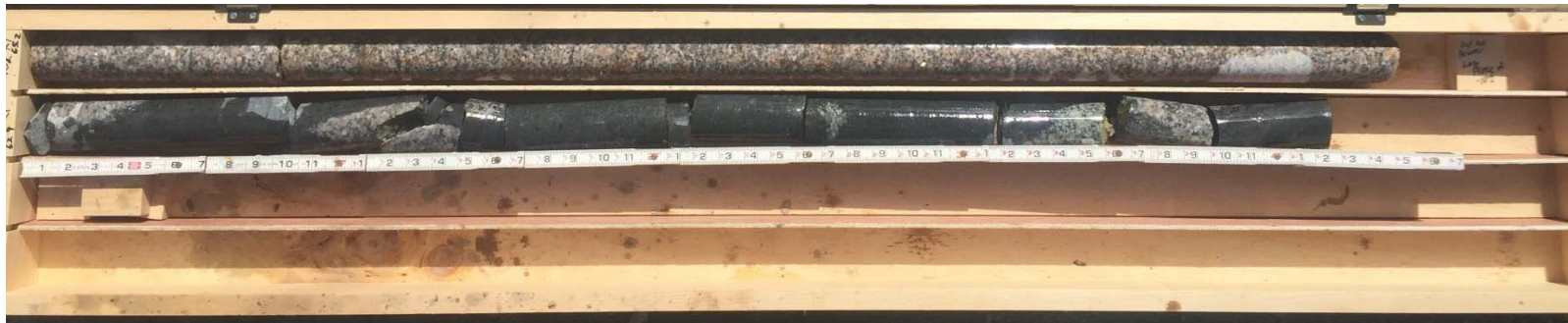


APPENDIX D – ROCK CORE PHOTOGRAPHS



Stantec – Shortridge Bridge
Bridge #2765 over Saco River Overflow
Fryeburg, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-FSRO-102	R1	65.2 - 70.2	53	88%	53	88%	GRANITE	1
BB-FSRO-101	R1	63.9 - 68.9	50	83%	37	62%	BASALT	2



- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.
 3. Transition between core runs within a row are marked by wood separators.



APPENDIX E – CALCULATIONS

SOIL ENGINEERING PROPERTIES



Correlation of SPT N-Values to Internal Friction Angle (ϕ) Using LRFD

Rev. 1

Project: Shortridge Bridge (Route 302) over Saco River Overflow
Location: Fryeburg, Maine
Calculated By: NVW Date: 4/5/2018
Checked By: ARB Date: _____

Objective: To estimate the internal friction angle (ϕ) of subsurface strata for the Shortridge Bridge (Route 302) Bridge based on Standard Penetration Test (SPT) measurements and empirical correlations using N_{field} , N_{corr} and $(N_1)_{60}$ for use in LRFD analyses.

Method: Correlations between SPT N-values and the soil internal friction angle are based on two (2) equations presented in NCHRP Report 651 and in Table 10.4.6.2.4-1 of the AASHTO, 8th Edition (2017). The correlated friction angle using these two (2) correlations are presented in this document. The recommended internal friction angle to be used for design is based on the correlation presented by Peck, Hanson and Thornburn (PHT, 1974) as mentioned in Kulhawy and Mayne (1990) and recommended in NCHRP Report 651 (below).

$$\phi \approx 54 - 27.6034 * e^{-0.014 * (N_1)_{60}}$$

Given

Information: SPT measurements and subsurface conditions in borings BB-FSRO-101, BB-FSRO-102 performed by New England Boring Contractors between March 6, 2018 and March 16, 2018 and observed by GZA.

Assumptions: Split spoon samples and SPTs were performed using an automatic hammer. For this calculation, however, the calibrated automatic hammer efficiency was calculated to be 86.9%.

Analysis

Results:

Layer	Range of SPT (N_1) ₆₀ - Values	Recomm. ϕ^1 (degrees)
Fill	5 to 43	39
Stream Alluvial	0 to 4	27
Lacustrine Silt	5 to 9	--
Lacustrine Sand	17 to 219	40

Notes:

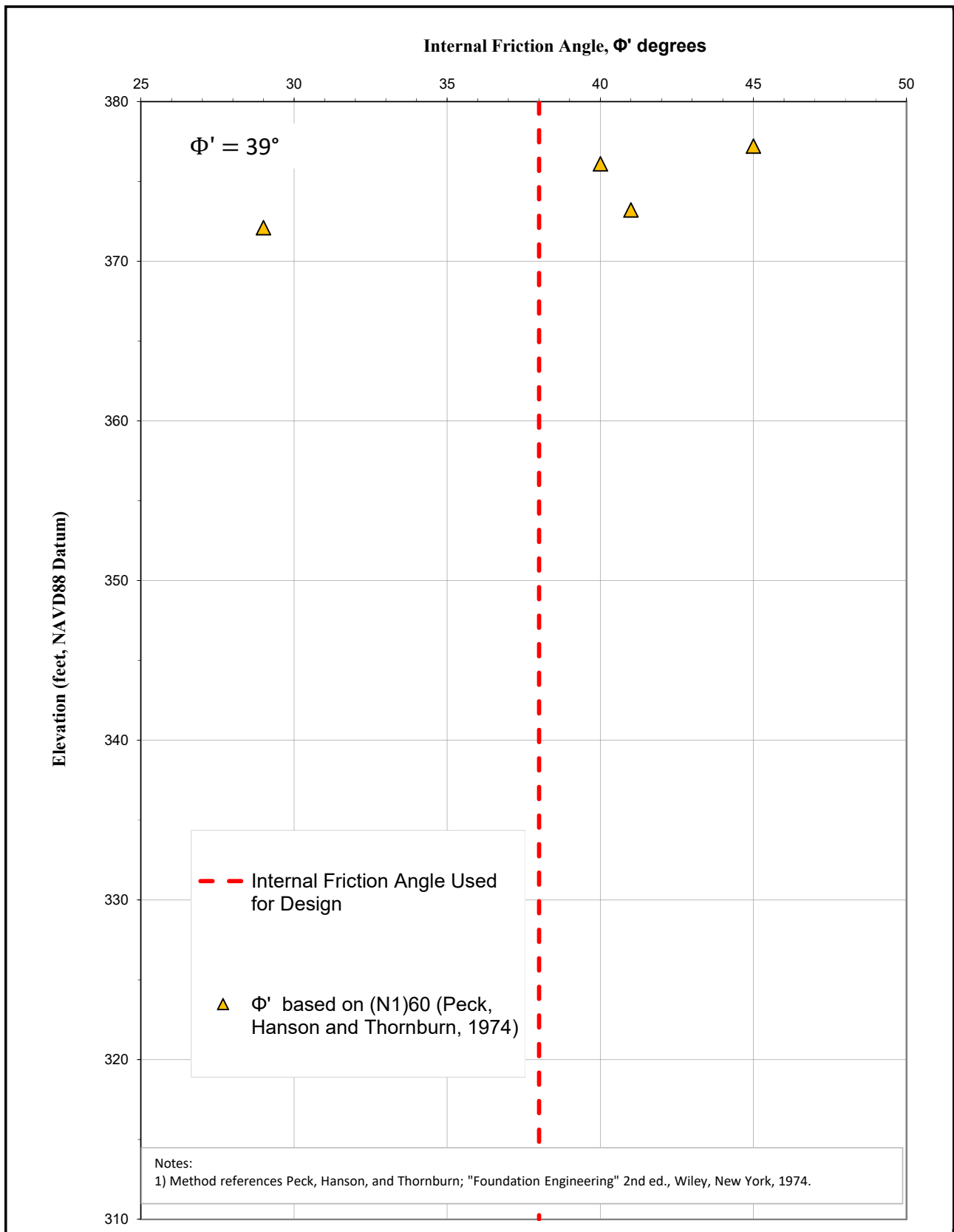
1) ϕ is based on the Peck, Hanson, & Thornburn (1974) empirical correlation, as mentioned by Kulhawy & Mayne (1990).

Attachments:

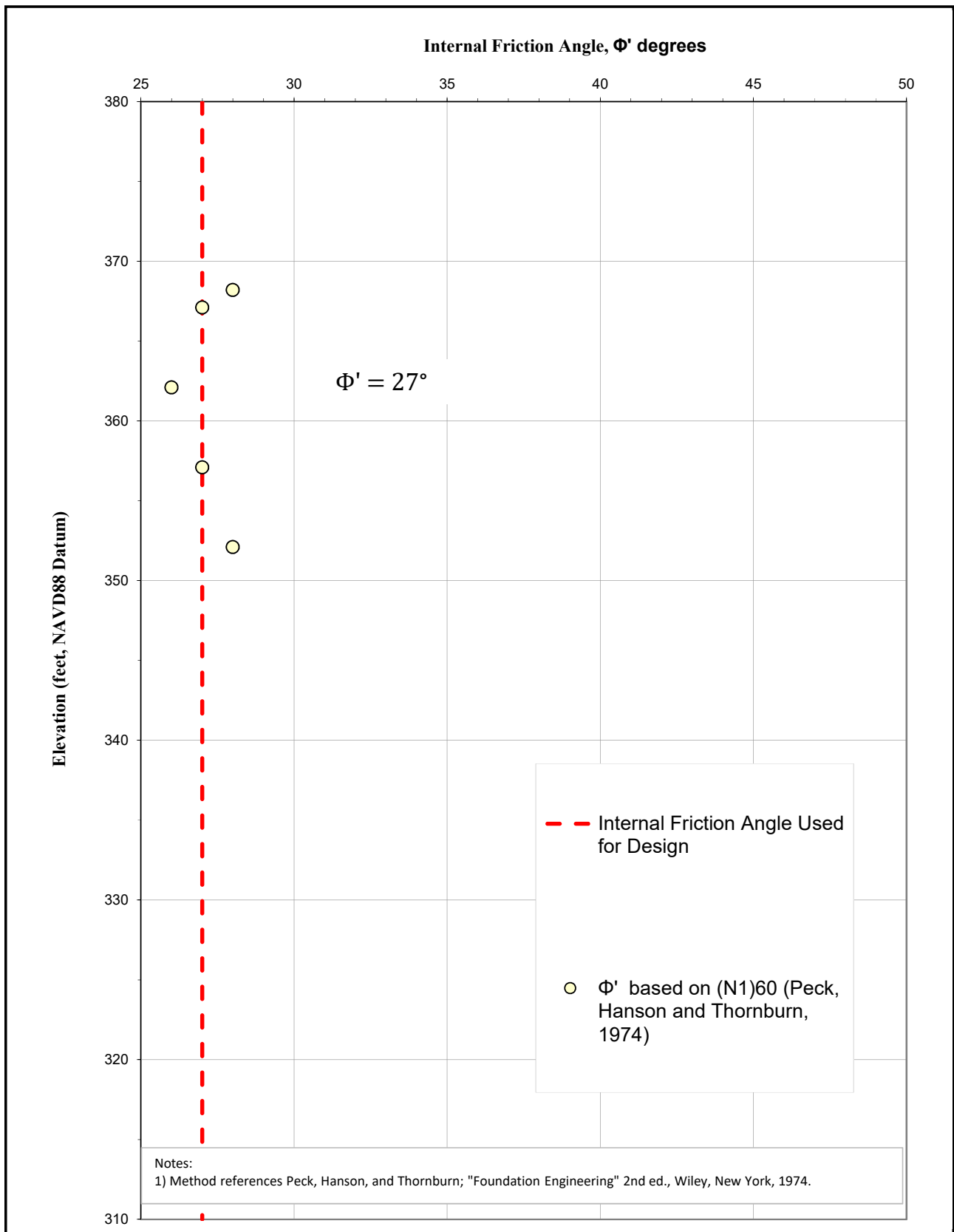
- 1) Figures
- 2) Calculations
- 3) Boring Logs BB-FSRO-101 and BB-FSRO-102:

References:

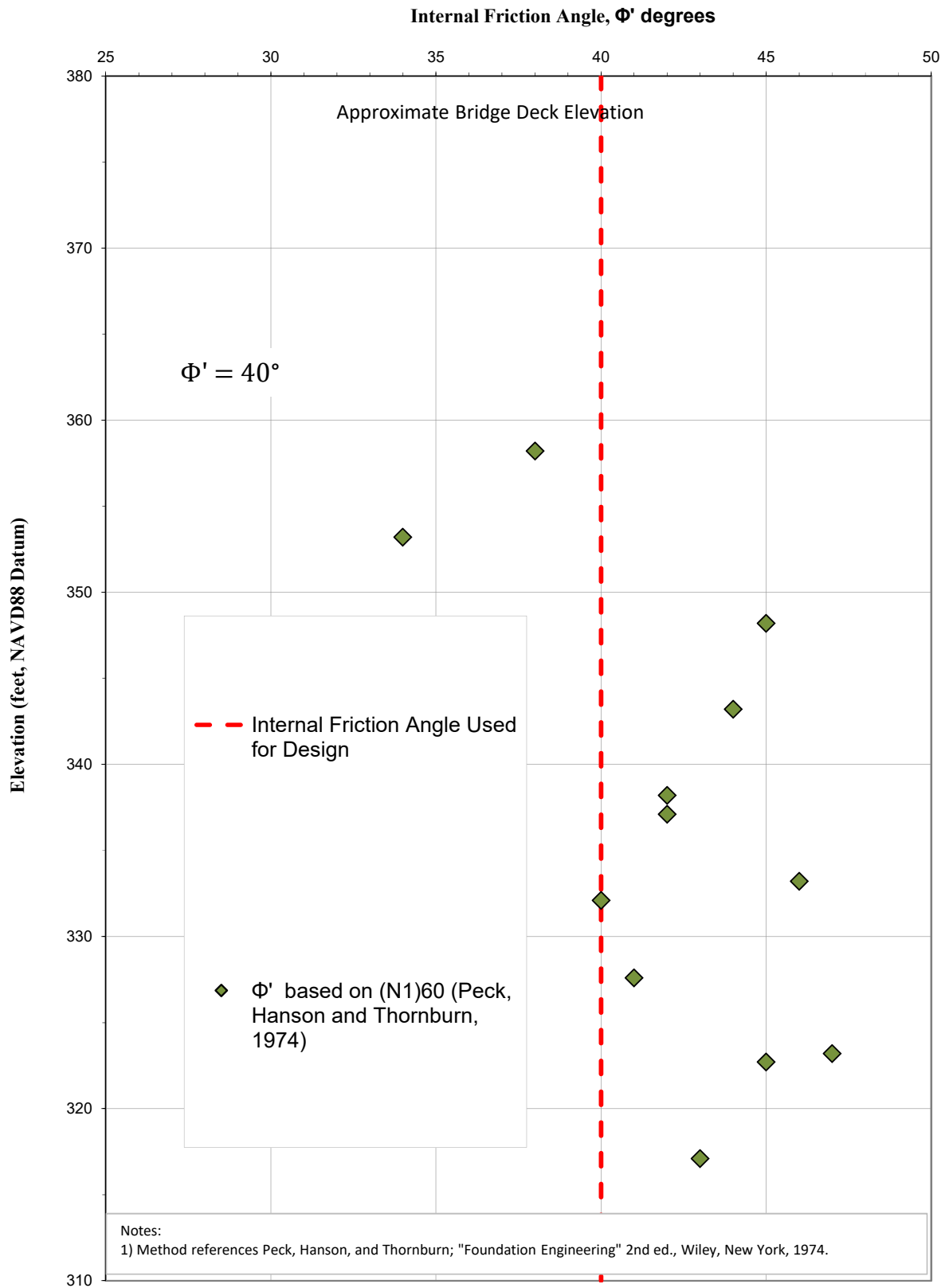
- 1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017.
- 2) Boring Logs BB-FSRO-101 and BB-FSRO-102.
- 2) NCHRP Report 651, "LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures," 2010.
- 3) FHWA NHI-06-088, "Soils and Foundations," Volume 1, December 2006.
- 4) Youd, T.L., et. al., "Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and GeoEnvironmental Engineering, October 2001.
- 5) Peck, Hanson, and Thornburn; "Foundation Engineering" 2nd ed., Wiley, New York, 1974
- 6) Kulhawy, F. and Mayne, P (1990). *Manual on Estimation of Soil Properties for Foundation Design*, Report EPRI-EL-6800. Electric Power Research Institute, Palo Alto, CA.



PROJ MGR: NVW DESIGNED BY: NVW REVIEWED BY:ARB	DRAWN BY: NVW DATE: 4/5/2018	SHORTRIDGE BRIDGE(ROUTE 302) OVER SACO RIVER OVERFLOW FRYEBURG, MAINE	FIGURE 2
			



PROJ MGR: NVW DESIGNED BY: NVW REVIEWED BY:ARB	DRAWN BY: NVW DATE: 4/5/2018	SHORTRIDGE BRIDGE(ROUTE 302) OVER SACO RIVER OVERFLOW FRYEBURG, MAINE	FIGURE 3
			



PROJ MGR: NVW
DESIGNED BY: NVW
REVIEWED BY: ARB

DRAWN BY: NVW
DATE: 4/5/2018



**SHORTRIDGE BRIDGE(ROUTE 302) OVER
SACO RIVER OVERFLOW
FRYEBURG, MAINE**

**INTERNAL FRICTION ANGLES BASED ON STANDARD
PENETRATION TEST (SPT) N-VALUES FOR
LACUSTRINE SAND**

FIGURE 5



Estimation of Horizontal Modulus of Subgrade Reaction (k)

Rev. 0

Project: Shortridge Bridge (Route 302) over Saco River Overflow

Location: Fryeburg, Maine

Calculated By: NVW

Date: 5/14/2018

Checked By: ARB

Date: 5/16/2018

Objective: To estimate the horizontal modulus of subgrade reaction (k) of subsurface strata for the Shortridge Bridge (Route 302) Bridge for use in lateral analyses. K values will be estimated using strata internal friction angles (ϕ).

Method(s): Correlations between the horizontal modulus of subgrade reaction and the soil internal friction angle of a given stratum are based on Figure 3-34 presented in the 2016 LPILE Technical Manual.

Given

Information: SPT measurements and subsurface conditions in borings BB-FSRO-101, BB-FSRO-102 performed by New England Boring Contractors between March 6, 2018 and March 16, 2018 and observed by GZA.

Layer	ϕ (degrees)
Fill	39
Stream Alluvial	27
Lacustrine Silt	--
Lacustrine Sand	40

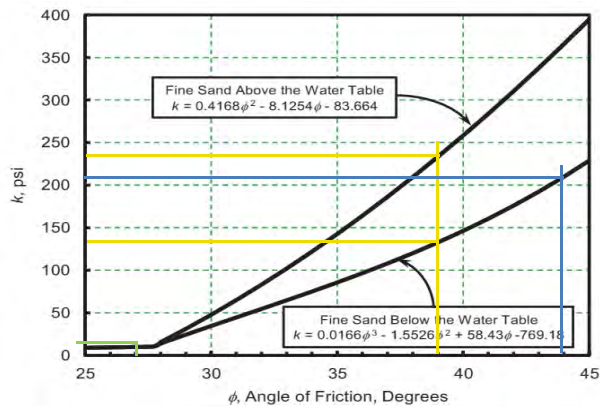


Figure 3-34 Value of k versus Friction Angle for Fine Sand Used in LPILE

Assumptions: Split spoon samples and SPTs were performed using an automatic hammer.

Analysis

Results:

Layer	Unit Weight above GWL (pcf)	Unit Weight below GWL (pcf)	Recomm. k above GWL ² (lb/in ³)	Recomm. k below GWL (lb/in ³)
Fill	125	63	225	130
Stream Alluvial	105	43	--	10
Lacustrine Silt	120	58	--	--
Lacustrine Sand	130	68	--	145

Notes:

- ϕ is based on the Peck, Hanson, & Thornburn (1974) empirical correlation, as mentioned by Kulhawy & Mayne (1990).
- GWL = groundwater level.
- Moduli for stream alluvial are the minimum values recommended in the Lpile Technical Manual.

References: 1) Technical Manual for LPILE 2016 by Ensoft, Inc., dated January 14, 2016.

GLOBAL STABILITY ANALYSIS



GZA
GeoEnvironmental, Inc
477 Congress Street, Suite 700
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
<http://www.gza.com>

*Engineers and
Scientists*

JOB: 09.0025967.00
SUBJECT: Shortridge Bridge Stability Eval
SHEET: 1 OF 3
CALCULATED BY NVW 9-21-18
CHECKED BY ARB 9-21-18

Objective

Evaluate global stability of the proposed approach embankments for the Shortridge Bridge replacement. The embankments will contain embankment fill over existing fill, Stream Alluvium, Lacustrine Silt, Lacustrine Sand and rock. The low-strength stream alluvium, with a friction angle of 27 deg., controls the global stability analyses. This stratum is thickest at BB-FSRO-102, beneath the east approach embankment (Abutment 2). Therefore, the worst-case section is through the north slope of the east approach embankment, at approximately Sta. 289+75. The proposed section at Sta. 289+75 consists of an approximately 12 foot tall embankment with a 2H:1V slope inclination.

Static Analysis

The minimum factor of safety for global stability is selected as 1.3. This corresponds approximately to a resistance factor of 0.75, which is specified in AASHTO LRFD Article 11.6.2.3 for a slope that does not contain or support a structural element.

Pseudostatic Analysis

The minimum factor of safety for global stability is selected as 1.0, which is recommended by Hynes-Griffin and Franklin in Geotechnical Earthquake Engineering. The pseudostatic analysis includes an additional earthquake load by incorporating the horizontal seismic coefficient, k_h , as defined in AASHTO 11.6.5.2.2 using the Newmark method as half of the maximum peak ground acceleration at the ground surface (see seismic analysis).

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017 with interims. (AASHTO LRFD)
2. Geotechnical Earthquake Engineering. Kramer, S.L. (1996) Upper Saddle River, NJ: Prentice-Hall.

Analysis

Soil Properties

Refer to the Soil Engineering Properties Sheet 1-6

GZA evaluated the stability of the proposed approach embankments statically and pseudostatically, using the computer analytical software *Slope/W 2012*, developed by Geo-Slope International, based on the Modified Bishop method.

Stability Station 289+75

A Slope-W model was developed with strata thicknesses based on boring BB-FSRO-102. The layering at that location was assumed to be representative of the largest thickness of weak stream alluvial deposit.

The proposed embankment at this location consists of a 20 foot wide travelway placed over the existing fill with a proposed side slope of 2H:1V with a 3 foot thick rip-rap cover.

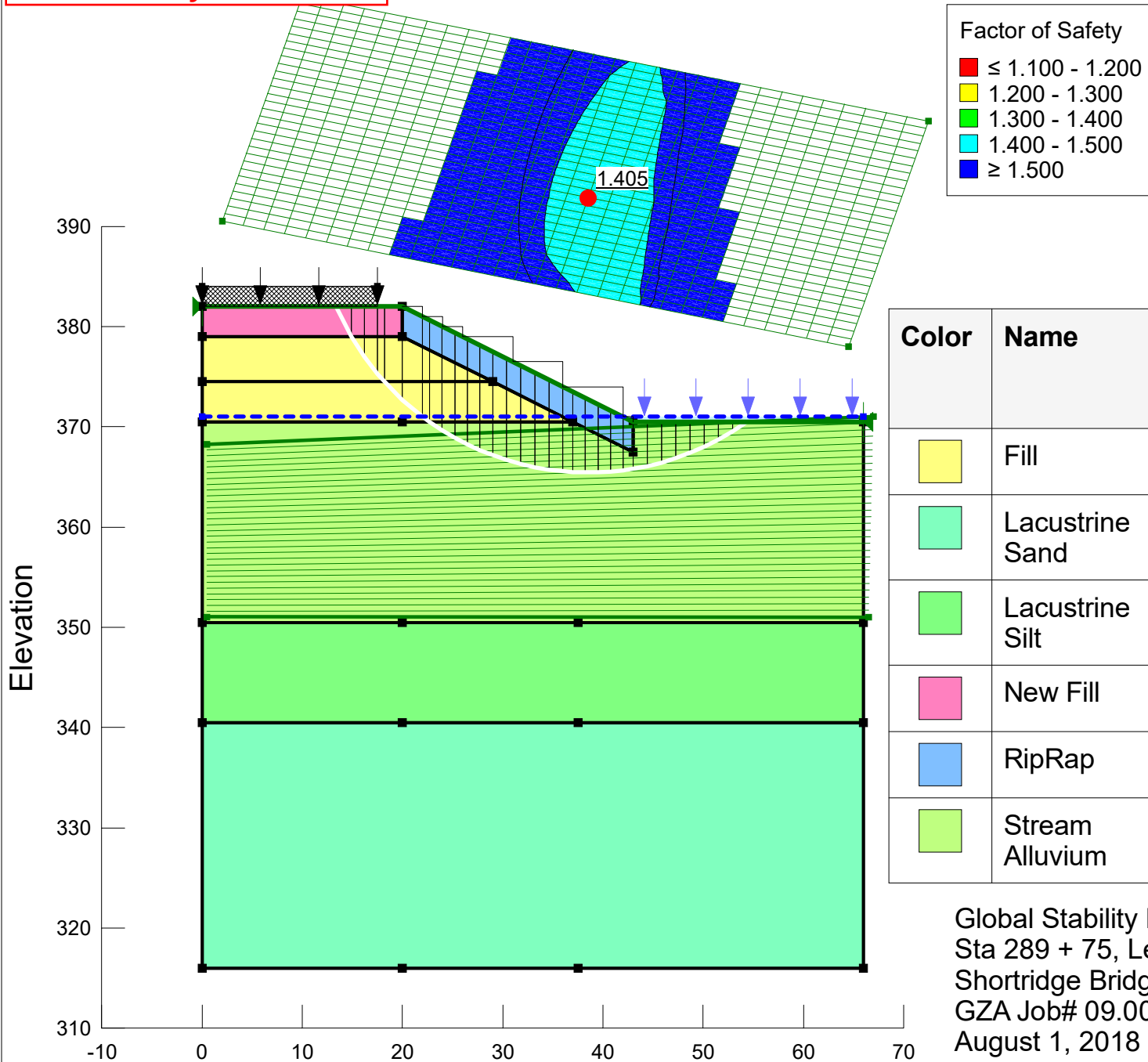
The attached Slope-W outputs (Sheets 2 and 3) shows that the proposed embankment provides the following minimum global factors of safety:

<u>Analysis</u>	<u>Factor of Safety Against Rotation</u>	<u>Required Factor of safety</u>
Static	1.4	1.3
Pseudostatic	1.1	1.0

Conclusion

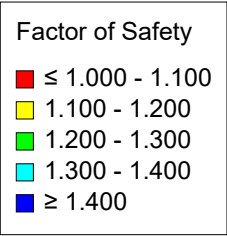
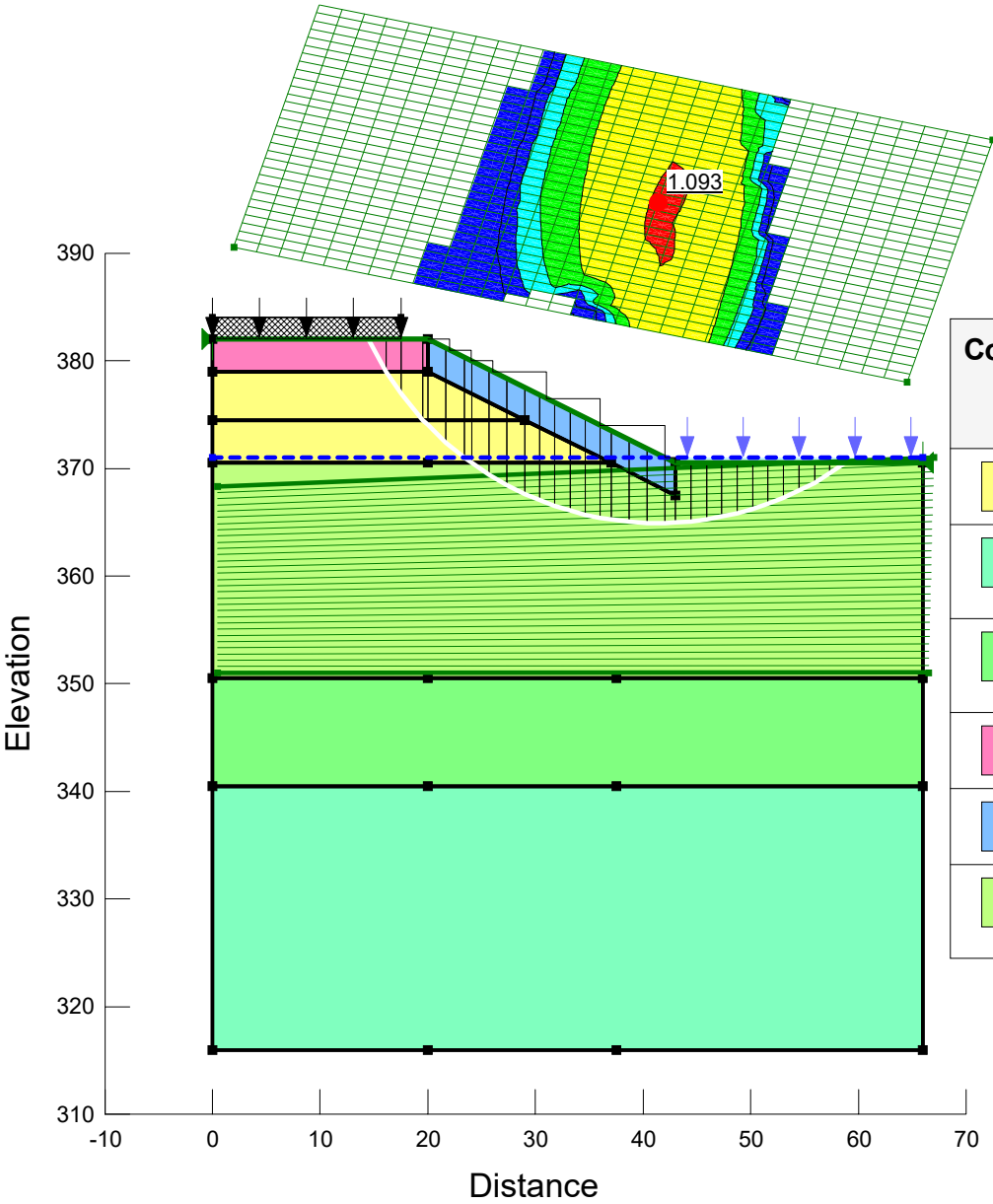
The global stability meets the minimum required criteria for both static and pseudostatic conditions. It is noted that this analysis does not consider strength loss due to liquefaction, which could result in lower safety factors.

Static Analysis



Global Stability Evaluation
Sta 289 + 75, Left
Shorridge Bridge - Fryeburg, Maine
GZA Job# 09.0025967.00
August 1, 2018

Pseudostatic Analysis



$K_h = a_{max} / 2 = 0.124g$

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
	Fill	125	0	39	0	39
	Lacustrine Sand	130	0	40	0	40
	Lacustrine Silt	120	1,000	0	1,000	0
	New Fill	125	0	34	0	34
	RipRap	140	0	40	0	40
	Stream Alluvium	105	0	27	0	27

Global Stability Evaluation
Sta 289 + 75, Left
Shortridge Bridge - Fryeburg, Maine
GZA Job# 09.0025967.00
August 1, 2018

LIQUEFACTION AND SEISMIC ANALYSIS



SPT-Based Liquefaction Susceptibility Calculation

Project: Shortridge Bridge	Project No: 09.0025967.00
Location: Fryeburg, Maine	
Calculated By: N. Williams	Date: 5/18/2018
Checked By: A. Blaisdell	Date: 5/18/2018

PURPOSE: Estimate exploration-specific factor of safety against liquefaction.

REFERENCES:

1. Idriss, I.M. and Boulanger, R.W. (2008). *Soil Liquefaction During Earthquakes*. Earthquake Engineering Research Institute. Oakland, California. EERI Publication No. MNO-12.
2. Idriss, I.M. and Boulanger, R.W. (2004). Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes. Proceedings of the Joint International Conference on Soil Dynamics & Earthquake Engineering and International Conference on Earthquake Geotechnical Engineering. Berkeley, California. January, 2004. pp.32-56.

INPUT PARAMETERS:

1. Use boring logs BB-FSRO-101 and -102 for subsurface and drilling data.
2. An automatic hammer was used to sample the borings with a hammer efficiency factor of 0.869.
3. The Site Class was determined to be Site Class E. Therefore, the peak ground acceleration corresponding to an earthquake with a 7% probability of exceedance in 75 years (975 year event) is 0.248 g.
4. Based on a USGS deaggregation of peak ground acceleration for Site Class B rock at the site, a Moment Magnitude, M, of 5.9 was selected.

SUMMARY OF RESULTS:

1. BB-FSRO-101/Abutment 1: The liquefaction analyses indicate that about 7 feet of Stream Alluvium at depths of 8.5-15.5' bgs have factors of safety of liquefaction between about 1.1 and 1.3. The predicted liquefaction settlement is approximately 1/2 inch, indicating minimal potential for liquefaction-induced embankment damage.
2. BB-FSRO-102/Abutment 2: The liquefaction analyses indicate that about 10 feet of Stream Alluvium at depths of 8.5-28.5' bgs have factors of safety of liquefaction between about 0.9 and 1.1. The predicted liquefaction settlement is approximately 6 inches.



SPT-Based Liquefaction Factor of Safety Calculation

Project Name: Shortridge Bridge
Project Location: Fryeburg, Maine
Project Number: 09.0025967.00
Calculations Reference: Idriss & Boulanger (2008)

Calculated By: N. Williams Date: 5/18/2018
Checked By: A. Blaisdell Date: 5/18/2018

Exploration ID: BB-FSRO-102

Elevation Data (at time of drilling):
Ground Surface Elevation: 379.2
Groundwater Elevation: 369.2
Groundwater Depth (ft): 10.0
Unit Weight of Water (pcf): 62.4

Drilling Data:
Typical rod stickup during SPT: 2 ft
Typical rod stickup during SPT: 0.6096 m
Borehole Diameter: 4 in

Material Properties:
Assumed Soil Weight Above Water Table, $\gamma = 112.5$ pcf
Assumed Soil Weight Below Water Table, $\gamma_{sat} = 113$ pcf
Atmospheric Pressure, $P_a = 2109$ psf

Seismic Assumptions:
Max Accel at Surface, a_{max} (g) = 0.248
Design Earthquake Magnitude, $M = 5.9$
Earthquake Return Period (yrs) 2475

Design Ground Surface Elevation: 382.7
Design Groundwater Elevation: 369.2
Design Groundwater Depth (ft): 13.5
Thickness of New Fill (ft): 3.5
Thickness of New Fill (m): 1.1
Assumed Soil Weight New Fill, $\gamma = 125$ pcf

SPT Correction Factors:

Split Spoon Type: 1 D=1-3/8 in - Standard Sampler
Hammer Type: Auto Hammer

Subsurface Data and Parameter Calculations																				Capacity/Demand Ratio (Factor of Safety) Calculations												Lateral Displacement Calculations					Vertical Settlement				
Depth (mid-SPT interval)	Elev.	Depth	Total Stress, σ _v	Effective Stress, σ' _v	N _{field}	Soil Type	Stratum	Flags	Hammer Energy Ratio, ER	C _E	C _R	C _B	C _s	N ₆₀	C _N	(N ₁) ₆₀	Fines Content	ΔN	(N ₁) _{0.005}	CRR (M=7.5, σ=1 atm)	MSF	C _α	K _α	CRR (M=7.5, σ=1 atm)	r _d	Design Total Stress, σ _v	Design Effective Stress, σ' _v	CSR	F _s	F _{ult}	γ _{lim}	γ _{max}	Layer Thickne ss	LDI	Vertical reconsol Strain, ε _v	ΔSI					
ft	ft	m	psf	psf	blows/ft	USCS			%					blows/ft		blows/ft	%	blows/ft	blows/ft							psf	psf					ft	ft		in						
2	377.2	0.61	225	225	43	Fill	Unsaturated		87	1.45	0.75	1.00	1.00	47	1.42	66	5	0	66	4.13	1.50	0.30	1.10	6.84	0.98	663	663	0.16	5.00	-2.99	0.00	0.000	4.3	0.08	0.000	0.43					
6	373.2	1.83	675	675	20	Fill	Unsaturated		87	1.45	0.75	1.00	1.00	22	1.49	32	15	3	36	1.24	1.50	0.23	1.10	2.06	0.96	1113	1113	0.15	5.00	-0.48	0.02	0.000	4.3	0.08	0.000	0.43					
11	368.2	3.35	1238	1175	2	Alluvial			87	1.45	0.80	1.00	1.00	2	1.46	3	50	6	9	0.11	1.50	0.07	1.04	0.17	0.93	1675	1613	0.16	1.12	0.93	0.53	0.011	5.0	0.08	0.006	0.43					
16	363.2	4.88	1800	1426	5	Alluvial/Lac Silt			87	1.45	0.85	1.00	1.00	6	1.25	8	43	6	13	0.14	1.50	0.08	1.03	0.22	0.90	2238	1863	0.17	1.28	0.82	0.33	0.010	2.0	0.02	0.004	0.09					
21	358.2	6.40	2363	1676	29	Lac Silt			87	1.45	0.95	1.00	1.00	40	1.07	43	60	6	48	4.13	1.50	0.30	1.07	6.65	0.86	2800	2114	0.18	5.00	-1.44	0.00	0.000	3.0	0.00	0.000	0.00					
26	353.2	7.92	2925	1927	17	Lac Sand			87	1.45	0.95	1.00	1.00	23	1.04	24	15	3	28	0.37	1.50	0.16	1.01	0.56	0.82	3363	2364	0.19	2.96	0.07	0.06	0.000	5.0	0.00	0.000	0.00					
31	348.2	9.45	3488	2177	65	Lac Sand			87	1.45	1.00	1.00	1.00	94	1.00	94	15	3	97	4.13	1.50	0.30	0.99	6.16	0.78	3925	2615	0.19	5.00	-5.81	0.00	0.000	5.0	0.00	0.000	0.00					
36	343.2	10.97	4050	2428	57	Lac Sand			87	1.45	1.00	1.00	1.00	83	0.99	82	15	3	85	4.13	1.50	0.30	0.96	5.96	0.74	4488	2865	0.19	5.00	-4.64	0.00	0.000	5.0	0.00	0.000	0.00					
41	338.2	12.50	4613	2678	51	Lac Sand			87	1.45	1.00	1.00	1.00	74	0.97	72	15	3	75	4.13	1.50	0.30	0.93	5.77	0.71	5050	3116	0.18	5.00	-3.72	0.00	0.000	5.0	0.00	0.000	0.00					
46	333.2	14.02	5175	2929	78	Lac Sand			87	1.45	1.00	1.00	1.00	113	1.01	114	15	3	118	4.13	1.50	0.30	0.90	5.61	0.67	5613	3366	0.18	5.00	-7.78	0.00	0.000	5.0	0.00	0.000	0.00					
50.5	328.7	15.39	5681	3154	100	Lac Sand			87	1.45	1.00	1.00	1.00	145	1.07	155	15	3	159	4.13	1.50	0.30	0.88	5.47	0.64	6119	3592	0.18	5.00	-11.89	0.00	0.000	5.0	0.00	0.000	0.00					
																														Total Thickness of Sublayers (ft)					48.50						

- Notes:
1. Design a_{max} was calculated using AASHTO LRFD based on assumed Site Class E.
 2. Design M was taken as mean magnitude from USGS Deaggregation for site location, PGA hazard level, 975-yr event.
 3. Factor of safety calculations limited to a maximum value of 5.0. Actual FS may be greater but shown as 5.0.
 4. Factor of safety presented as 5.0 for Unsaturated and Clay soils; not calculated.
 5. Design ground surface elevation was estimated based the Bridge Profile drawing.
 6. Fines content was estimated based on soil description provided on log where laboratory test results were not available.
 7. The automatic hammer was assigned ER=86.9%, corresponding to value provided by MaineDOT.



SPT-Based Liquefaction Factor of Safety Calculation

Project Name: Shortridge Bridge
Project Location: Fryeburg, Maine
Project Number: 09.0025967.00
Calculations Reference: Idriss & Boulanger (2008)

Calculated By: N. Williams Date: 5/18/2018
Checked By: A. Blaisdell Date: 5/18/2018

Exploration ID: BB-FSRO-102

Elevation Data (at time of drilling):
Ground Surface Elevation: 378.1
Groundwater Elevation: 368.1
Groundwater Depth (ft): 10.0
Unit Weight of Water (pcf): 62.4

Drilling Data:
Typical rod stickup during SPT: 2 ft
Typical rod stickup during SPT: 0.6096 m
Borehole Diameter: 4 in

Material Properties:
Assumed Soil Weight Above Water Table, $\gamma = 125$ pcf
Assumed Soil Weight Below Water Table, $\gamma_{sat} = 105$ pcf
Atmospheric Pressure, $P_a = 2109$ psf

Seismic Assumptions:
Max Accel at Surface, a_{max} (g) = 0.248
Design Earthquake Magnitude, $M = 5.9$
Earthquake Return Period (yrs) 975

Design Ground Surface Elevation: 381.6
Design Groundwater Elevation: 368.1
Design Groundwater Depth (ft): 13.5
Thickness of New Fill (ft): 3.5
Thickness of New Fill (m): 1.1
Assumed Soil Weight New Fill, $\gamma = 125$ pcf

SPT Correction Factors:

Split Spoon Type: 1 D=1-3/8 in - Standard Sampler
Hammer Type: Auto Hammer

Subsurface Data and Parameter Calculations																				Capacity/Demand Ratio (Factor of Safety) Calculations														Lateral Displacement Calculations					Vertical Settlement	
Depth (mid-SPT interval)	Elev.	Depth	Total Stress, σ _v	Effective Stress, σ' _v	N _{field}	Soil Type	Stratum	Flags	Hammer Energy Ratio, ER	C _E	C _R	C _B	C _s	N ₆₀	C _N	(N ₁) ₆₀	Fines Content	ΔN	(N ₁) _{60CS}	CRR (M=7.5, σ=1 atm)	MSF	C _c	K _c	CRR (M=7.5, σ=1 atm)	r _d	Design Total Stress, σ _v	Design Effective Stress, σ' _v	CSR	F _s	F _{cs}	γ _{lim}	γ _{max}	Layer Thickne ss	LDI	Vertical reconsol Strain, ε _v	ΔSi				
ft	ft	m	psf	psf	blows/ft	USCS			%					blows/ft		blows/ft	%	blows/ft	blows/ft							psf	psf					ft	ft		in					
2	376.1	0.61	250	250	39	Fill	Unsaturated		87	1.45	0.75	1.00	1.00	42	1.46	62	8	0	62	4.13	1.50	0.30	1.10	6.84	0.98	688	688	0.16	5.00	-2.63	0.00	0.000	4.3	6.18	0.000	6.17				
6	372.1	1.83	750	750	5	Fill	Unsaturated		87	1.45	0.75	1.00	1.00	5	1.70	9	16	4	13	0.14	1.50	0.09	1.09	0.23	0.96	1188	1188	0.15	5.00	0.84	0.35	0.000	4.3	6.18	0.000	6.17				
11	367.1	3.35	1355	1293	2	Alluvial			87	1.45	0.80	1.00	1.00	2	1.37	3	48	6	9	0.11	1.50	0.07	1.03	0.17	0.93	1793	1730	0.16	1.10	0.93	0.54	0.012	5.0	6.18	0.006	6.17				
16	362.1	4.88	1880	1506	1	Alluvial			87	1.45	0.85	1.00	1.00	1	1.26	2	62	6	7	0.10	1.50	0.06	1.02	0.15	0.90	2318	1943	0.17	0.89	0.95	0.65	0.653	5.0	6.12	0.045	5.80				
21	357.1	6.40	2405	1719	2	Alluvial			87	1.45	0.95	1.00	1.00	3	1.14	3	45	6	9	0.11	1.50	0.07	1.01	0.17	0.86	2843	2156	0.18	0.92	0.94	0.54	0.544	5.0	2.85	0.040	3.11				
26	352.1	7.92	2930	1932	4	Alluvial			87	1.45	0.95	1.00	1.00	6	1.05	6	45	6	11	0.13	1.50	0.08	1.01	0.19	0.82	3368	2369	0.19	1.03	0.88	0.41	0.027	5.0	0.13	0.012	0.69				
31	347.1	9.45	3455	2145	8	Lac Silt	Clay		87	1.45	1.00	1.00	1.00	12	0.99	11	75	6	17	0.17	1.50	0.10	1.00	0.26	0.78	3893	2582	0.19	5.00	0.66	0.22	0.000	5.0	0.00	0.000	0.00				
36	342.1	10.97	3980	2358	9	Lac Silt	Clay		87	1.45	1.00	1.00	1.00	13	0.94	12	75	6	18	0.18	1.50	0.10	0.99	0.27	0.74	4418	2795	0.19	5.00	0.63	0.20	0.000	5.0	0.00	0.000	0.00				
41	337.1	12.50	4505	2571	78	Lac Sand			87	1.45	1.00	1.00	1.00	113	1.01	114	27	5	119	4.13	1.50	0.30	0.94	5.85	0.71	4943	3008	0.19	5.00	-7.91	0.00	0.000	5.0	0.00	0.000	0.00				
46	332.1	14.02	5030	2784	62	Lac Sand			87	1.45	1.00	1.00	1.00	90	0.98	88	5	0	88	4.13	1.50	0.30	0.92	5.70	0.67	5468	3221	0.18	5.00	-4.96	0.00	0.000	5.0	0.00	0.000	0.00				
51	327.1	15.54	5555	2997	74	Lac Sand			87	1.45	1.00	1.00	1.00	107	1.00	108	5	0	108	4.13	1.50	0.30	0.89	5.56	0.64	5993	3434	0.18	5.00	-6.80	0.00	0.000	5.0	0.00	0.000	0.00				
																				Total Thickness of Sublayers (ft)		53.50																		

- Notes:
- Design a_{max} was calculated using AASHTO LRFD based on assumed Site Class E.
 - Design M was taken as mean magnitude from USGS Deaggregation for site location, PGA hazard level, 975-yr event.
 - Factor of safety calculations limited to a maximum value of 5.0. Actual FS may be greater but shown as 5.0.
 - Factor of safety presented as 5.0 for Unsaturated and Clay soils; not calculated.
 - Design ground surface elevation was estimated based the Bridge Profile drawing.
 - Fines content was estimated based on soil description provided on log where laboratory test results were not available.
 - The automatic hammer was assigned ER=86.9%, corresponding to value provided by MaineDOT.

 Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (44.04464°N, 70.87131°W)

Site Class E – “Soft Clay Soil”

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B.
Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 ^[1]	PGA = 0.099 g
From Figure 3.4.1-3 ^[2]	S _s = 0.195 g
From Figure 3.4.1-4 ^[3]	S ₁ = 0.049 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class E, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1-1 Site Class Definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5,000$	N/A	N/A
B	Rock	$2,500 < \bar{v}_s \leq 5,000$	N/A	N/A
C	Very dense soil and soft rock	$1,200 < \bar{v}_s \leq 2,500$	$\bar{N} > 50$	$> 2,000$ psf
D	Stiff soil profile	$600 \leq \bar{v}_s < 1,200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$< 1,000$ psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = E and PGA = 0.099 g, $F_{PGA} = 2.500$

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of S_s

For Site Class = E and $S_s = 0.195$ g, $F_a = 2.500$

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = E and $S_1 = 0.049$ g, $F_v = 3.500$

Used for liquefaction analysis

Equation (3.4.1-1):

$$A_S = F_{PGA} \text{ PGA} = 2.500 \times 0.099 = 0.248 \text{ g}$$

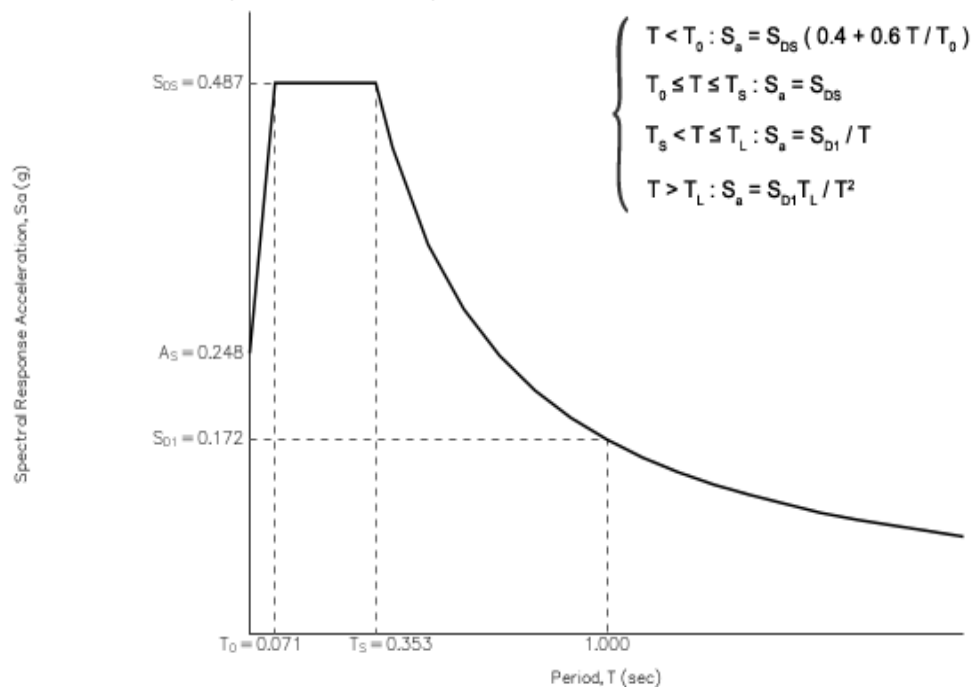
Equation (3.4.1-2):

$$S_{DS} = F_a S_S = 2.500 \times 0.195 = 0.487 \text{ g}$$

Equation (3.4.1-3):

$$S_{D1} = F_v S_1 = 3.500 \times 0.049 = 0.172 \text{ g}$$

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g \leq S_{D1}$	D

For $S_{D1} = 0.172\text{ g}$, Seismic Design Category = B

Seismic Design Category $\equiv$ “the design category in accordance with Table 3.5-1” = B

References

1. *Figure 3.4.1-2*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf>
2. *Figure 3.4.1-3*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf>
3. *Figure 3.4.1-4*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf>

Unified Hazard Tool



Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Conterminous U.S. 2008

Spectral Period

Peak ground acceleration

Latitude

Decimal degrees

44.029064

Time Horizon

Return period in years

975

Longitude

Decimal degrees, negative values for western long...

-70.908265

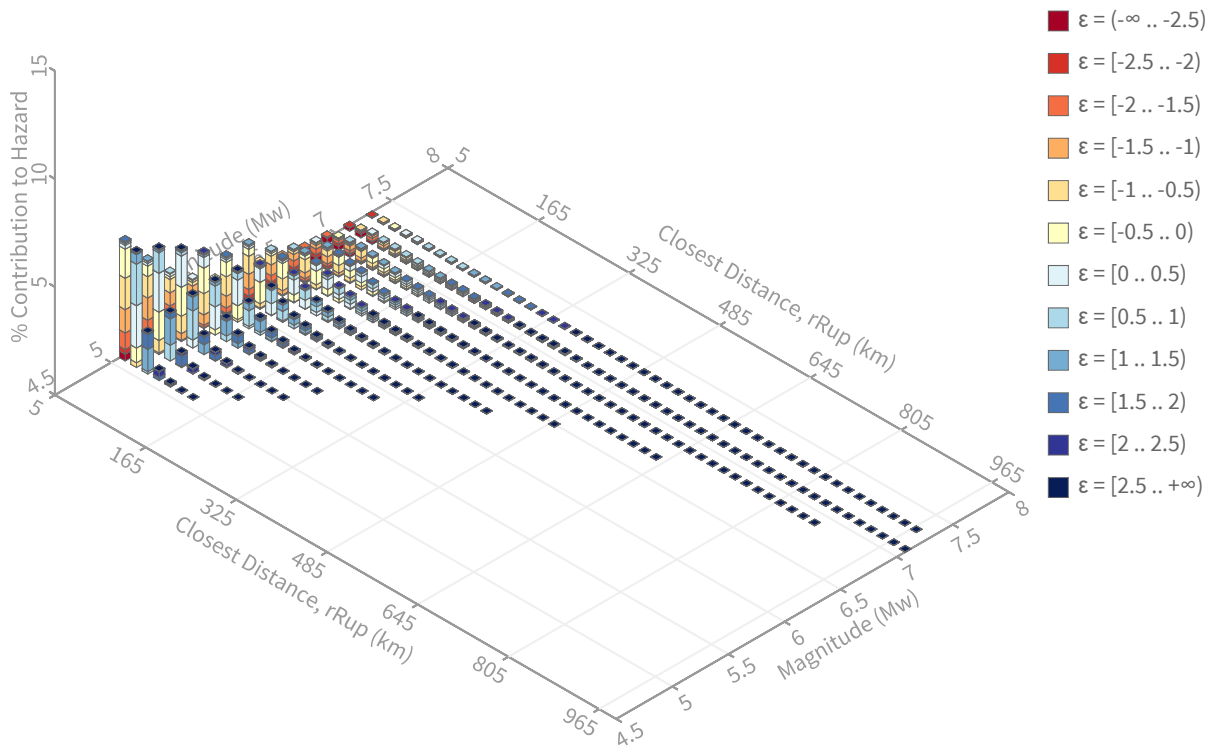
Site Class

760 m/s (B/C boundary)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 975 yrs

Exceedance rate: 0.001025641 yr⁻¹

PGA ground motion: 0.077609305 g

Recovered targets

Return period: 973.95142 yrs

Exceedance rate: 0.0010267453 yr⁻¹

Totals

Binned: 100 %

Residual: 0 %

Trace: 1.14 %

Used for liquefaction
analysis



Mean (for all sources)

r: 63.2 km

m: 5.94

ϵ_0 : -0.17 σ

Mode (largest r-m bin)

r: 13.28 km

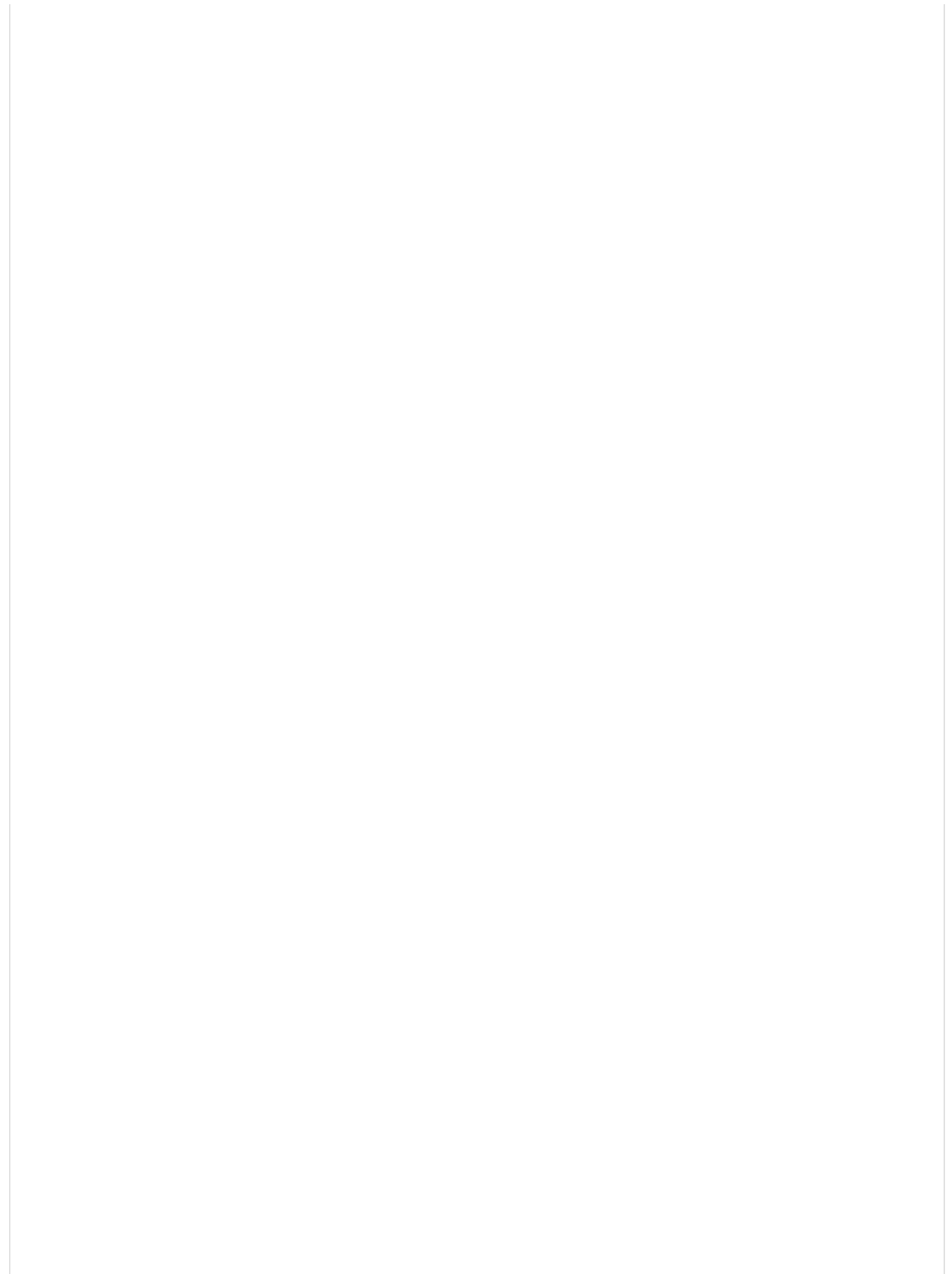
m: 5.1

ϵ_0 : -0.95 σ

Contribution: 5.5 %

Mode (largest ϵ_0 bin)

000000



Deaggregation Contributors

Source Set	Source	Type	r	m	ϵ_0	lon	lat	az	%
CEUS.2007all8.J.in (opt)		Grid							55.41
	PointSourceFinite: -70.908, 44.321		31.73	5.66	-0.42	70.908°W	44.321°N	0.00	4.41
	PointSourceFinite: -70.908, 44.186		17.62	5.51	-1.22	70.908°W	44.186°N	0.00	4.09
	PointSourceFinite: -70.908, 44.231		22.30	5.56	-0.90	70.908°W	44.231°N	0.00	3.89
	PointSourceFinite: -70.908, 44.411		41.11	5.78	-0.10	70.908°W	44.411°N	0.00	3.69
	PointSourceFinite: -70.908, 44.276		27.01	5.60	-0.63	70.908°W	44.276°N	0.00	3.26
	PointSourceFinite: -70.908, 44.366		36.43	5.72	-0.24	70.908°W	44.366°N	0.00	2.80
	PointSourceFinite: -70.908, 44.456		45.76	5.84	0.01	70.908°W	44.456°N	0.00	2.50
	PointSourceFinite: -70.908, 44.501		50.41	5.90	0.10	70.908°W	44.501°N	0.00	2.34
	PointSourceFinite: -70.908, 44.097		8.77	5.45	-2.16	70.908°W	44.097°N	0.00	2.31
	PointSourceFinite: -70.908, 44.546		55.03	5.96	0.18	70.908°W	44.546°N	0.00	2.07
	PointSourceFinite: -70.908, 44.141		13.04	5.48	-1.64	70.908°W	44.141°N	0.00	2.05
	PointSourceFinite: -70.908, 44.591		59.66	6.02	0.25	70.908°W	44.591°N	0.00	1.76
	PointSourceFinite: -70.908, 44.681		69.02	6.11	0.38	70.908°W	44.681°N	0.00	1.52
	PointSourceFinite: -70.908, 44.636		64.31	6.07	0.31	70.908°W	44.636°N	0.00	1.40
	PointSourceFinite: -70.908, 44.771		78.52	6.18	0.47	70.908°W	44.771°N	0.00	1.37
	PointSourceFinite: -70.908, 44.726		73.77	6.14	0.44	70.908°W	44.726°N	0.00	1.01
CEUS.2007all8.AB.in (opt)		Grid							44.59
	PointSourceFinite: -70.908, 44.186		17.53	5.54	-1.04	70.908°W	44.186°N	0.00	3.75
	PointSourceFinite: -70.908, 44.321		31.42	5.72	-0.27	70.908°W	44.321°N	0.00	3.71
	PointSourceFinite: -70.908, 44.231		22.15	5.59	-0.72	70.908°W	44.231°N	0.00	3.46
	PointSourceFinite: -70.908, 44.411		40.60	5.85	0.01	70.908°W	44.411°N	0.00	2.98
	PointSourceFinite: -70.908, 44.276		26.79	5.65	-0.47	70.908°W	44.276°N	0.00	2.82
	PointSourceFinite: -70.908, 44.366		36.03	5.78	-0.11	70.908°W	44.366°N	0.00	2.30
	PointSourceFinite: -70.908, 44.097		8.76	5.47	-1.93	70.908°W	44.097°N	0.00	2.26
	PointSourceFinite: -70.908, 44.456		45.15	5.92	0.10	70.908°W	44.456°N	0.00	1.99
	PointSourceFinite: -70.908, 44.141		13.00	5.50	-1.43	70.908°W	44.141°N	0.00	1.94
	PointSourceFinite: -70.908, 44.501		49.69	5.99	0.18	70.908°W	44.501°N	0.00	1.84
	PointSourceFinite: -70.908, 44.546		54.21	6.06	0.24	70.908°W	44.546°N	0.00	1.62
	PointSourceFinite: -70.908, 44.591		58.74	6.12	0.30	70.908°W	44.591°N	0.00	1.37
	PointSourceFinite: -70.908, 44.681		67.89	6.23	0.40	70.908°W	44.681°N	0.00	1.17
	PointSourceFinite: -70.908, 44.636		63.29	6.18	0.35	70.908°W	44.636°N	0.00	1.08
	PointSourceFinite: -70.908, 44.771		77.19	6.31	0.50	70.908°W	44.771°N	0.00	1.02

SUMMARY OF PILE REACTIONS



PROJECT: Fryeburg
CLIENT: Maine Dept. of Transportation
TASK: Integral Abutment Pile Reaction
BY: KLV
CHECK: LSF

JOB #: 195311568
WIN: 23849
DATE: 6/14/2018
DATE:

Summary of Pile Reactions

STRENGTH I	Fz (kips)	dx (+) (in)	dx (-) (in)
DC	168.75	-	-
DW	21.00	-	-
EV	24.30	-	-
LL	108.47	-	-
TU	-	0.27	0.16

Total for Strength I

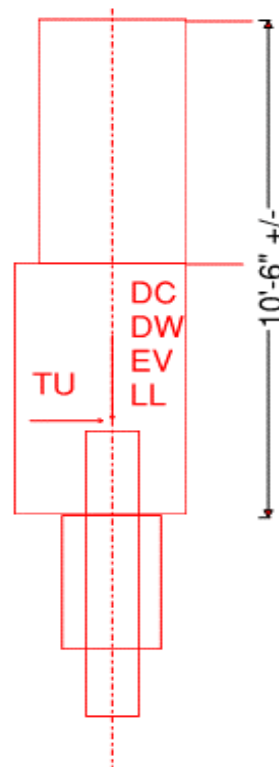
322.52 k

Max Deflection in inches

SERVICE I	Fz (kips)	dx (+) (in)	dx (-) (in)
DC	135	-	-
DW	14	-	-
EV	18	-	-
LL	52	-	-
TU	-	0.23	0.13

Total for Service I

219 k



The above reactions include the following load factors:

	STRENGTH I	SERVICE I
DC	1.25	1.0
DW	1.5	1.0
EV	1.35	1.0
LL	1.75	1.0
TU	1.2	1.0

DRIVABILITY ANALYSIS



GZA
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<http://www.gza.com>

*Engineers and
Scientists*

JOB: 09.0025967.00 Shortridge Bridge
SUBJECT: Axial Pile Resistance
SHEET: 1 OF 10
CALCULATED BY N. Williams, 11/15/18
REVIEWED BY A. Blaisdell, 11/15/18

Objective

Evaluate the axial geotechnical resistance of the abutment piles for the Shortridge Bridge Replacement in Fryeburg, ME

Methodology

Evaluate proposed pile section for governing factored axial compression resistance as follows.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017 with interims. (AASHTO LRFD)

Soil Properties

Consider Shortridge Bridge Interpretive Subsurface Profile (see Figure 2), subsurface layering and properties relative to pile design are presented in the Soil Engineering Properties calculation and summarized on Sheets 3-5 of this calculation.

Structural Properties

HP14x89

Yield Strength of Steel

$$F_y := 50 \text{ ksi}$$

Area of section

$$A_s := 26.1 \text{ in}^2$$

Young's Modulus of Steel

$$E_s := 30000 \cdot \text{ksi}$$



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JOB: 09.0025967.00 Shortridge Bridge
SUBJECT: Axial Pile Resistance
SHEET: 2 OF 10
CALCULATED BY N. Williams, 11/15/18
REVIEWED BY A. Blaisdell, 11/15/18

1. Nominal Structural Compressive Resistance P_n

Refer to the Elastic Zone Check in the Lateral Pile Analysis (Sheets 1-12)

Nominal Compressive Resistance:

$$P_{nAbut1} := 1155.5 \cdot \text{kip} \quad \text{AASHTO Eq. 6.9.5.1-1}$$

$$P_{nAbut2} := 1079.1 \cdot \text{kip}$$

2. Factored Structural Compressive Resistance - Strength Limit State:

Refer to the Elastic Zone Check

Factored Compressive Resistance for Strength Limit State:

$$P_{rAbut1} := 808.8 \cdot \text{kip}$$

$$P_{rAbut2} := 755.3 \cdot \text{kip}$$

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit
States:

From Article 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State:

$$P_{rAbut1E} := \phi \cdot P_{nAbut1} = 1155.5 \cdot \text{kip} \quad \text{AASHTO Eq. 6.9.2.1-1} \quad \text{pg. 6-71}$$

$$P_{rAbut2E} := \phi \cdot P_{nAbut2} = 1079.1 \cdot \text{kip}$$

4. Geotechnical Axial Resistance - Static Analysis

AASHTO Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state.

Refer to the attached USACE method used in Apile to estimate side friction resistance during driving. The estimated side friction is 305.4 kips for Abutment 1 piles and 212.5 kips for Abutment 2 piles. Friction estimates are attached.

Required nominal resistance of 496 kips for design of Abutments 1 and 2 based on a maximum factored pile load of 322.5 kips and a 0.65 resistance factor.

The estimated % skin friction resistance is about 62 and 43% at the required nominal pile resistance.



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JOB: 09.0025967.00 Shortridge Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 3 OF 10
 CALCULATED BY N. Williams, 11/15/18
 REVIEWED BY A. Blaisdell, 11/15/18

5. Geotechnical Axial Resistance - Drivability Analysis

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \text{AASHTO Eq. 10.7.8.1}$$

$$f_y := 50 \text{ ksi} \quad \text{yield Strength of steel}$$

$$\phi_{da} := 1.0 \quad \text{AASHTO Table 10.5.5.2.3-1 Refers to Article 6.5.4.2, Pg. 6-28}$$

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \sigma_{dr} = 45 \cdot \text{ksi} \quad \text{Driving Stress in pile cannot exceed 45 ksi}$$

Abutment 1 - Drive pile through 56 feet of soil to rock with toe quake representative of hard driving conditions (0.7in). Model pile length as 60 feet (4 foot stickup at end of drive).

Abutment 2 - Drive pile through 57 feet of soil to rock with toe quake representative of moderate driving conditions (0.7 in). Model pile length as 60 feet (3 foot stickup at end of drive).

Drive piles with a Delmag D 22-02 open-ended diesel hammer with a rated energy of 48,500 ft-lb. **The proposed hammer is sized to achieve the required nominal pile resistance; not necessarily the maximum driveability resistance for the pile section and profile.**

Allowable stress in piles during driving is $0.9 f_y = 45 \text{ ksi}$

GRLWEAP Output is attached on Sheets 7 through 10.

Abutment 1: $R_{ndr1} := 496 \text{ kip}$ Required nominal geotechnical resistance, pile driving stress=35 ksi, final penetration resistance=7 bpi.

Abutment 2: $R_{ndr2} := 496 \text{ kip}$ Required nominal geotechnical resistance, pile driving stress=36 ksi, final penetration resistance=8 bpi.

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

PDA, WEAP and CAPWAP used to establishing driving criteria

$$\phi_{dyn} := 0.65 \quad \text{AASHTO Table 10.5.5.2.3-1}$$

Abutment 1: $R_{ndr1_factored} := R_{ndr1} \cdot \phi_{dyn}$

~~$$R_{ndr1_factored} := 323 \cdot \text{kip}$$~~

Abutment 2: $R_{ndr2_factored} := R_{ndr2} \cdot \phi_{dyn}$

~~$$R_{ndr2_factored} := 323 \cdot \text{kip}$$~~



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JOB: 09.0025967.00 Shortridge Bridge
SUBJECT: Axial Pile Resistance
SHEET: 4 OF 10
CALCULATED BY N. Williams, 11/15/18
REVIEWED BY A. Blaisdell, 11/15/18

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States: $\phi_{serv_ext} := 1$

From Article 10.5.5.1 and 10.5.5.3

$$R_{ndr1_serv_ext} := R_{ndr1} \cdot \phi_{serv_ext}$$

$$R_{ndr1_serv_ext} = 496 \cdot \text{kip}$$

$$R_{ndr2_serv_ext} := R_{ndr2} \cdot \phi_{serv_ext}$$

$$R_{ndr2_serv_ext} = 496 \cdot \text{kip}$$

Summary of Results - Axial Loading:

ASTM A572 HP 14x89	Structural Resistance (kips)	Geotechnical Static Resistance (kips)	Required Geotechnical Resistance (kips)	Governing Resistance (kips)
Abutment 1: Strength Limit State, Design	1155.5	N/A	322.5	322.5
Abutment 1: Service/Extreme Limit State, Design	808.8	N/A	496	496
Abutment 2: Strength Limit State, Design	1079.1	N/A	322.5	322.5
Abutment 2: Service/Extreme Limit State, Design	755.3	N/A	496	496

Results indicate the selected hammer is suitable to meet the required nominal pile resistance.

Abutment 1 - Geotechnical Axial
Resistance - Apile Results

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=====

APILE for Windows, Version 2015.7.8

Serial Number : 653550831

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.

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This program is licensed to :

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Portland, OR

Path to file locations : P:\09 Jobs\0025900s\09.0025967.00 - MDOT Shortridge Fryeburg -
Stantec\Work\Calcs\APile\BBFSR0101\14x89\
Name of input data file : BBFSR0101 14X89.ap7d
Name of output file : BBFSR0101 14X89.ap7o
Name of plot output file : BBFSR0101 14X89.ap7p

Time and Date of Analysis

Date: July 02, 2018 Time: 13:45:13

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* INPUT INFORMATION *

Page 1

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New Pile

DESIGNER :

JOB NUMBER :

METHOD FOR UNIT LOAD TRANSFERS :

- USACE (U.S. Army Corps of Engineers)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)
- USACE (U.S. Army Corps of Engineers)
Critical Depth Method for Sand:
10 to 20 Pile Diameter based on the Density

TYPE OF LOADING :

- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 26.10 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 60.00 FT.
- PILE STICKUP LENGTH, PSL = 4.00 FT.

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- ZERO FRICTION LENGTH, ZFL = 0.00 FT.
 - PERIMETER OF PILE = 57.00 IN.
 - TIP AREA OF PILE = 26.10 IN2
 - INCREMENT OF PILE LENGTH USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.00	63.00	39.00	0.00
1.50	SAND	0.00	63.00	39.00	0.00
1.50	SAND	0.00	43.00	27.00	0.00
8.50	SAND	0.00	43.00	27.00	0.00
8.50	CLAY	0.00	58.00	0.00	0.00
11.50	CLAY	0.00	58.00	0.00	0.00
11.50	SAND	0.00	68.00	40.00	0.00
56.00	SAND	0.00	68.00	40.00	0.00
56.00	SAND	0.00	88.00	40.00	0.00
60.00	SAND	0.00	88.00	40.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOVED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

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0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
1.50	1.000	1.000
1.50	1.000	1.000
8.50	1.000	1.000
8.50	1.000	1.000
11.50	1.000	1.000
11.50	1.000	1.000
56.00	1.000	1.000
56.00	1.000	1.000
60.00	1.000	1.000

1

 * COMPUTATION RESULT *

 * FED. HWY. METHOD *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.5	0.5

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1.00	0.1	0.5	0.6
2.00	0.5	0.5	1.0
3.00	0.8	0.7	1.5
4.00	1.1	0.5	1.6
5.00	1.4	0.5	1.9
6.00	1.7	0.5	2.3
7.00	2.2	0.7	2.8
8.00	2.6	0.9	3.6
9.00	3.2	1.2	4.3
10.00	5.8	2.1	7.9
11.00	10.6	5.0	15.6
12.00	15.2	8.2	23.4
13.00	18.8	11.6	30.4
14.00	21.7	15.4	37.1
15.00	24.9	17.5	42.4
16.00	28.4	19.0	47.4
17.00	32.1	20.5	52.6
18.00	36.2	21.9	58.1
19.00	40.5	23.4	63.9
20.00	45.1	24.9	70.0
21.00	49.9	26.4	76.3
22.00	55.1	27.9	83.0
23.00	60.5	29.3	89.9
24.00	66.3	30.8	97.1
25.00	72.3	32.3	104.6
26.00	78.5	33.8	112.3
27.00	85.1	35.3	120.4
28.00	91.9	36.7	128.7
29.00	99.1	38.2	137.3
30.00	106.5	39.7	146.2
31.00	114.2	41.2	155.3
32.00	122.1	42.7	164.8
33.00	130.4	44.1	174.5
34.00	138.9	45.6	184.5
35.00	147.7	47.1	194.8
36.00	156.8	48.6	205.4
37.00	166.2	50.0	216.2
38.00	175.8	51.5	227.4
39.00	185.8	53.0	238.8
40.00	196.0	54.5	250.5
41.00	206.5	56.0	262.4

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42.00	217.3	57.4	274.7
43.00	228.3	58.9	287.2
44.00	239.7	60.4	300.1
45.00	251.3	61.9	313.2
46.00	263.2	63.4	326.5
47.00	275.4	64.8	340.2
48.00	287.8	66.3	354.1
49.00	300.6	67.8	368.4
50.00	313.6	69.3	382.9
51.00	326.9	70.8	397.7
52.00	340.5	72.2	412.7
53.00	354.4	73.5	427.9
54.00	368.5	74.5	443.1
55.00	383.0	75.2	458.2
56.00	397.7	75.6	473.3

 * ARMY CORPS METHOD *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.4	0.4
1.00	0.1	0.4	0.6
2.00	0.5	0.6	1.1
3.00	1.0	0.8	1.8
4.00	1.5	0.8	2.3
5.00	2.1	0.8	2.9
6.00	2.8	1.0	3.8
7.00	3.6	1.1	4.8
8.00	4.6	1.3	5.9
9.00	5.6	1.4	7.1
10.00	8.0	2.0	10.0
11.00	11.5	4.0	15.6
12.00	15.1	6.2	21.3
13.00	18.3	8.6	26.9
14.00	21.2	11.3	32.5
15.00	24.4	12.8	37.2

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16.00	27.9	13.9	41.8
17.00	31.6	15.0	46.7
18.00	35.7	16.1	51.8
19.00	40.0	17.2	57.2
20.00	44.7	18.3	62.9
21.00	49.6	19.3	68.9
22.00	54.7	20.4	75.2
23.00	60.2	21.5	81.7
24.00	66.0	22.6	88.6
25.00	72.0	23.7	95.7
26.00	78.3	24.8	103.1
27.00	84.9	25.9	110.8
28.00	91.8	26.9	118.7
29.00	98.9	28.0	127.0
30.00	106.4	29.0	135.4
31.00	114.0	29.4	143.4
32.00	121.7	29.6	151.2
33.00	129.3	29.5	158.8
34.00	137.0	29.2	166.2
35.00	144.6	29.1	173.7
36.00	152.3	29.1	181.4
37.00	159.9	29.1	189.1
38.00	167.6	29.1	196.7
39.00	175.3	29.1	204.4
40.00	182.9	29.1	212.0
41.00	190.6	29.1	219.7
42.00	198.2	29.1	227.3
43.00	205.9	29.1	235.0
44.00	213.5	29.1	242.6
45.00	221.2	29.1	250.3
46.00	228.8	29.1	257.9
47.00	236.5	29.1	265.6
48.00	244.1	29.1	273.3
49.00	251.8	29.1	280.9
50.00	259.5	29.1	288.6
51.00	267.1	29.1	296.2
52.00	274.8	29.1	303.9
53.00	282.4	29.1	311.5
54.00	290.1	29.1	319.2
55.00	297.7	29.1	326.8
56.00	305.4	29.1	334.5

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Does not include
downdrag (Strength I)

Nominal Capacity = $322.5/0.65 = 496.2$ kips
Side Friction = 305.4 kips
% of Nominal = $305.4/496.2 = 61.5\%$

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NOTES:

- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING *

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	-0.4000E+01	0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00	0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01 0.6000E-01 0.8000E-01 0.1200E+00 0.1600E+00 0.5000E+00 0.1000E+02
2	10	-0.2000E+01	0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00 0.0000E+00	0.0000E+00 0.1000E-01 0.2000E-01 0.4000E-01 0.6000E-01 0.8000E-01 0.1200E+00 0.1600E+00 0.5000E+00 0.1000E+02
3	10	-0.8333E-02		

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0.0000E+00 0.0000E+00
0.0000E+00 0.1000E-01
0.0000E+00 0.2000E-01
0.0000E+00 0.4000E-01
0.0000E+00 0.6000E-01
0.0000E+00 0.8000E-01
0.0000E+00 0.1200E+00
0.0000E+00 0.1600E+00
0.0000E+00 0.5000E+00
0.0000E+00 0.1000E+02

4 10 0.0000E+00

0.0000E+00 0.0000E+00
0.3848E+00 0.1000E-01
0.4548E+00 0.2000E-01
0.5003E+00 0.4000E-01
0.5176E+00 0.6000E-01
0.5266E+00 0.8000E-01
0.5360E+00 0.1200E+00
0.5409E+00 0.1600E+00
0.5510E+00 0.5000E+00
0.5557E+00 0.1000E+02

5 10 0.7750E+00

0.0000E+00 0.0000E+00
0.3848E+00 0.1000E-01
0.4548E+00 0.2000E-01
0.5003E+00 0.4000E-01
0.5176E+00 0.6000E-01
0.5266E+00 0.8000E-01
0.5360E+00 0.1200E+00
0.5409E+00 0.1600E+00
0.5510E+00 0.5000E+00
0.5557E+00 0.1000E+02

6 10 0.1458E+01

0.0000E+00 0.0000E+00
0.4161E+00 0.1000E-01
0.4992E+00 0.2000E-01
0.5545E+00 0.4000E-01
0.5758E+00 0.6000E-01
0.5871E+00 0.8000E-01
0.5988E+00 0.1200E+00
0.6048E+00 0.1600E+00

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0.6175E+00 0.5000E+00
0.6234E+00 0.1000E+02
0.0000E+00 0.0000E+00
0.2498E+00 0.1000E-01
0.3567E+00 0.2000E-01
0.4538E+00 0.4000E-01
0.4992E+00 0.6000E-01
0.5254E+00 0.8000E-01
0.5545E+00 0.1200E+00
0.5703E+00 0.1600E+00
0.6056E+00 0.5000E+00
0.6227E+00 0.1000E+02

7 10 0.1500E+01

8 10 0.5025E+01

0.0000E+00 0.0000E+00
0.3045E+00 0.1000E-01
0.4799E+00 0.2000E-01
0.6739E+00 0.4000E-01
0.7789E+00 0.6000E-01
0.8447E+00 0.8000E-01
0.9227E+00 0.1200E+00
0.9673E+00 0.1600E+00
0.1073E+01 0.5000E+00
0.1128E+01 0.1000E+02

9 10 0.8458E+01

10 10 0.8500E+01

0.0000E+00 0.0000E+00
0.4459E+00 0.1000E-01
0.9413E+00 0.2000E-01
0.1957E+01 0.4000E-01
0.2403E+01 0.6000E-01

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			BBFSR0101 14X89.ap7o	
			0.2477E+01	0.8000E-01
			0.2403E+01	0.1200E+00
			0.2304E+01	0.1600E+00
			0.2304E+01	0.5000E+00
			0.2304E+01	0.1000E+02
11	10	0.1003E+02	0.0000E+00	0.0000E+00
			0.9375E+00	0.1000E-01
			0.1979E+01	0.2000E-01
			0.4115E+01	0.4000E-01
			0.5052E+01	0.6000E-01
			0.5208E+01	0.8000E-01
			0.5052E+01	0.1200E+00
			0.4844E+01	0.1600E+00
			0.4844E+01	0.5000E+00
			0.4844E+01	0.1000E+02
12	10	0.1146E+02	0.0000E+00	0.0000E+00
			0.8861E+00	0.1000E-01
			0.1871E+01	0.2000E-01
			0.3889E+01	0.4000E-01
			0.4775E+01	0.6000E-01
			0.4923E+01	0.8000E-01
			0.4775E+01	0.1200E+00
			0.4578E+01	0.1600E+00
			0.4578E+01	0.5000E+00
			0.4578E+01	0.1000E+02
13	10	0.1150E+02	0.0000E+00	0.0000E+00
			0.9969E+00	0.1000E-01
			0.1658E+01	0.2000E-01
			0.2481E+01	0.4000E-01
			0.2972E+01	0.6000E-01
			0.3299E+01	0.8000E-01
			0.3706E+01	0.1200E+00
			0.3950E+01	0.1600E+00
			0.4563E+01	0.5000E+00
			0.4904E+01	0.1000E+02
14	10	0.3378E+02	0.0000E+00	0.0000E+00
			0.1124E+01	0.1000E-01

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			BBFSR0101 14X89.ap7o	
			0.2044E+01	0.2000E-01
			0.3456E+01	0.4000E-01
			0.4491E+01	0.6000E-01
			0.5281E+01	0.8000E-01
			0.6409E+01	0.1200E+00
			0.7176E+01	0.1600E+00
			0.9492E+01	0.5000E+00
			0.1109E+02	0.1000E+02
15	10	0.5596E+02	0.0000E+00	0.0000E+00
			0.1124E+01	0.1000E-01
			0.2044E+01	0.2000E-01
			0.3456E+01	0.4000E-01
			0.4491E+01	0.6000E-01
			0.5281E+01	0.8000E-01
			0.6409E+01	0.1200E+00
			0.7176E+01	0.1600E+00
			0.9492E+01	0.5000E+00
			0.1109E+02	0.1000E+02
16	10	0.5600E+02	0.0000E+00	0.0000E+00
			0.1124E+01	0.1000E-01
			0.2044E+01	0.2000E-01
			0.3456E+01	0.4000E-01
			0.4491E+01	0.6000E-01
			0.5281E+01	0.8000E-01
			0.6409E+01	0.1200E+00
			0.7176E+01	0.1600E+00
			0.9492E+01	0.5000E+00
			0.1109E+02	0.1000E+02
17	10	0.5803E+02	0.0000E+00	0.0000E+00
			0.1124E+01	0.1000E-01
			0.2044E+01	0.2000E-01
			0.3456E+01	0.4000E-01
			0.4491E+01	0.6000E-01
			0.5281E+01	0.8000E-01
			0.6409E+01	0.1200E+00
			0.7176E+01	0.1600E+00
			0.9492E+01	0.5000E+00
			0.1109E+02	0.1000E+02

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18	10	0.5996E+02	BBFSR0101 14X89.ap7o	
			0.0000E+00	0.0000E+00
			0.1124E+01	0.1000E-01
			0.2044E+01	0.2000E-01
			0.3456E+01	0.4000E-01
			0.4491E+01	0.6000E-01
			0.5281E+01	0.8000E-01
			0.6409E+01	0.1200E+00
			0.7176E+01	0.1600E+00
			0.9492E+01	0.5000E+00
			0.1109E+02	0.1000E+02

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.2983E+01	0.1000E-03
0.7931E+01	0.5000E-02
0.9432E+01	0.1000E-01
0.1410E+02	0.5000E-01
0.1677E+02	0.1000E+00
0.1995E+02	0.2000E+00
0.2508E+02	0.5000E+00
0.2983E+02	0.1000E+01
0.3547E+02	0.2000E+01

LOAD VERSUS SETTLEMENT CURVE

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.8714E+01	0.5284E-02	0.2983E+01	0.1000E-03
0.1635E+02	0.1017E-01	0.3892E+01	0.1000E-02
0.4735E+02	0.3132E-01	0.7931E+01	0.5000E-02
0.7100E+02	0.4961E-01	0.9432E+01	0.1000E-01

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0.1588E+03	0.1444E+00	0.1410E+02	0.5000E-01
0.2046E+03	0.2254E+00	0.1677E+02	0.1000E+00
0.2915E+03	0.6861E+00	0.2508E+02	0.5000E+00
0.2984E+03	0.1192E+01	0.2983E+02	0.1000E+01
0.3082E+03	0.2200E+01	0.3547E+02	0.2000E+01

BBFSR0102 14X89.ap7o

APILE for Windows, Version 2015.7.8

Serial Number : 653550831

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.

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This program is licensed to :

GZA GeoEnvironmental, Inc.
Portland, OR

Path to file locations : P:\09 Jobs\0025900s\09.0025967.00 - MDOT Shortridge Fryeburg -
Stantec\Work\Calcs\APile\BBFSR0102\14x89\
Name of input data file : BBFSR0102 14X89.ap7d
Name of output file : BBFSR0102 14X89.ap7o
Name of plot output file : BBFSR0102 14X89.ap7p

Time and Date of Analysis

Date: July 02, 2018 Time: 12:43:05

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* INPUT INFORMATION *

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Abutment 2 - 14x73

DESIGNER : N. Williams P.E.

JOB NUMBER : 09.0025967.00

METHOD FOR UNIT LOAD TRANSFERS :

- USACE (U.S. Army Corps of Engineers)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)
- USACE (U.S. Army Corps of Engineers)
Critical Depth Method for Sand:
10 to 20 Pile Diameter based on the Density

TYPE OF LOADING :
- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 26.10 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 60.00 FT.
- PILE STICKUP LENGTH, PSL = 2.50 FT.

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- ZERO FRICTION LENGTH, ZFL = 0.00 FT.
 - PERIMETER OF PILE = 57.00 IN.
 - TIP AREA OF PILE = 26.10 IN2
 - INCREMENT OF PILE LENGTH USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.00	63.00	39.00	0.00
1.50	SAND	0.00	63.00	39.00	0.00
1.50	SAND	0.00	43.00	27.00	0.00
21.50	SAND	0.00	43.00	27.00	0.00
21.50	CLAY	0.00	58.00	0.00	0.00
31.50	CLAY	0.00	58.00	0.00	0.00
31.50	SAND	0.00	68.00	40.00	0.00
57.50	SAND	0.00	68.00	40.00	0.00
57.50	SAND	0.00	88.00	40.00	0.00
62.00	SAND	0.00	88.00	40.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOVED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

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0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
1.50	1.000	1.000
1.50	1.000	1.000
21.50	1.000	1.000
21.50	1.000	1.000
31.50	1.000	1.000
31.50	1.000	1.000
57.50	1.000	1.000
57.50	1.000	1.000
62.00	1.000	1.000

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 * COMPUTATION RESULT *

 * FED. HWY. METHOD *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.5	0.5

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1.00	0.1	0.5	0.6
2.00	0.5	0.5	1.0
3.00	0.8	0.7	1.5
4.00	1.1	0.5	1.6
5.00	1.4	0.5	1.9
6.00	1.7	0.5	2.3
7.00	2.2	0.6	2.8
8.00	2.6	0.7	3.4
9.00	3.2	0.8	4.0
10.00	3.8	0.9	4.6
11.00	4.4	0.9	5.4
12.00	5.1	1.0	6.1
13.00	5.9	1.1	7.0
14.00	6.7	1.2	7.9
15.00	7.6	1.3	8.8
16.00	8.5	1.3	9.8
17.00	9.5	1.4	10.9
18.00	10.5	1.5	12.0
19.00	11.6	1.6	13.2
20.00	12.8	1.6	14.4
21.00	14.0	1.7	15.6
22.00	15.2	1.7	16.9
23.00	18.3	1.7	19.9
24.00	23.0	1.6	24.7
25.00	27.8	1.6	29.4
26.00	32.5	1.6	34.1
27.00	37.3	1.6	38.9
28.00	42.0	1.6	43.6
29.00	46.8	1.6	48.4
30.00	51.5	3.6	55.1
31.00	56.3	11.0	67.2
32.00	61.0	18.7	79.7
33.00	66.7	26.7	93.4
34.00	73.6	35.1	108.7
35.00	80.7	38.5	119.2
36.00	88.2	40.0	128.2
37.00	95.9	41.5	137.4
38.00	104.0	42.9	146.9
39.00	112.3	44.4	156.7
40.00	120.9	45.9	166.7
41.00	129.7	47.4	177.1

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42.00	138.9	48.9	187.7
43.00	148.3	50.3	198.6
44.00	158.0	51.8	209.8
45.00	168.0	53.3	221.3
46.00	178.3	54.8	233.0
47.00	188.8	56.2	245.1
48.00	199.6	57.7	257.4
49.00	210.8	59.2	270.0
50.00	222.1	60.7	282.8
51.00	233.8	62.2	296.0
52.00	245.8	63.6	309.4
53.00	258.0	65.1	323.1
54.00	270.5	66.6	337.1
55.00	283.3	68.1	351.4
56.00	296.4	69.6	366.0
57.00	309.8	71.1	380.9

 * ARMY CORPS METHOD *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.4	0.4
1.00	0.1	0.4	0.6
2.00	0.5	0.6	1.1
3.00	1.0	0.8	1.8
4.00	1.5	0.8	2.3
5.00	2.1	0.8	2.9
6.00	2.8	1.0	3.8
7.00	3.6	1.1	4.8
8.00	4.6	1.3	5.8
9.00	5.6	1.4	7.0
10.00	6.8	1.6	8.3
11.00	8.1	1.7	9.8
12.00	9.5	1.8	11.3
13.00	11.0	2.0	12.9
14.00	12.6	2.1	14.7

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15.00	14.3	2.3	16.5
16.00	16.1	2.3	18.4
17.00	17.9	2.3	20.2
18.00	19.7	2.3	22.0
19.00	21.5	2.3	23.8
20.00	23.2	2.3	25.5
21.00	25.0	2.1	27.1
22.00	26.8	2.0	28.8
23.00	29.5	1.8	31.3
24.00	33.1	1.7	34.7
25.00	36.6	1.6	38.3
26.00	40.2	1.6	41.8
27.00	43.8	1.6	45.4
28.00	47.3	1.6	48.9
29.00	50.9	1.6	52.5
30.00	54.4	2.9	57.4
31.00	58.0	7.7	65.7
32.00	61.6	12.5	74.1
33.00	66.4	17.3	83.7
34.00	72.5	22.1	94.5
35.00	78.6	23.3	101.9
36.00	84.6	23.3	108.0
37.00	90.7	23.3	114.1
38.00	96.8	23.3	120.2
39.00	102.9	23.3	126.3
40.00	109.0	23.3	132.3
41.00	115.1	23.3	138.4
42.00	121.2	23.3	144.5
43.00	127.3	23.3	150.6
44.00	133.3	23.3	156.7
45.00	139.4	23.3	162.8
46.00	145.5	23.3	168.9
47.00	151.6	23.3	174.9
48.00	157.7	23.3	181.0
49.00	163.8	23.3	187.1
50.00	169.9	23.3	193.2
51.00	175.9	23.3	199.3
52.00	182.0	23.3	205.4
53.00	188.1	23.3	211.5
54.00	194.2	23.3	217.5
55.00	200.3	23.3	223.6

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BBFSR0102 14X89.ap7o			
56.00	206.4	23.3	229.7
57.00	212.5	23.3	235.8

Nominal Capacity = $322.5/0.65 = 496.2\text{kips}$
Side Friction = 212.5 kips
% of Nominal = $212.5/496.2 = 42.8\%$

NOTES:

- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING *

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	-0.2500E+01	0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
2	10	-0.1250E+01	0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02

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			BBFSR0102 14X89.ap7o	
3	10	-0.8333E-02	0.0000E+00	0.1000E+02
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
4	10	0.0000E+00	0.0000E+00	0.0000E+00
			0.3848E+00	0.1000E-01
			0.4548E+00	0.2000E-01
			0.5003E+00	0.4000E-01
			0.5176E+00	0.6000E-01
			0.5266E+00	0.8000E-01
			0.5360E+00	0.1200E+00
			0.5409E+00	0.1600E+00
			0.5510E+00	0.5000E+00
			0.5557E+00	0.1000E+02
5	10	0.7750E+00	0.0000E+00	0.0000E+00
			0.3848E+00	0.1000E-01
			0.4548E+00	0.2000E-01
			0.5003E+00	0.4000E-01
			0.5176E+00	0.6000E-01
			0.5266E+00	0.8000E-01
			0.5360E+00	0.1200E+00
			0.5409E+00	0.1600E+00
			0.5510E+00	0.5000E+00
			0.5557E+00	0.1000E+02
6	10	0.1458E+01	0.0000E+00	0.0000E+00
			0.4161E+00	0.1000E-01
			0.4992E+00	0.2000E-01
			0.5545E+00	0.4000E-01
			0.5758E+00	0.6000E-01
			0.5871E+00	0.8000E-01

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			BBFSR0102 14X89.ap7o	
7	10	0.1500E+01	0.5988E+00	0.1200E+00
			0.6048E+00	0.1600E+00
			0.6175E+00	0.5000E+00
			0.6234E+00	0.1000E+02
			0.0000E+00	0.0000E+00
			0.2498E+00	0.1000E-01
			0.3567E+00	0.2000E-01
			0.4538E+00	0.4000E-01
			0.4992E+00	0.6000E-01
			0.5254E+00	0.8000E-01
8	10	0.1153E+02	0.5545E+00	0.1200E+00
			0.5703E+00	0.1600E+00
			0.6056E+00	0.5000E+00
			0.6227E+00	0.1000E+02
			0.0000E+00	0.0000E+00
			0.3480E+00	0.1000E-01
			0.5975E+00	0.2000E-01
			0.9314E+00	0.4000E-01
			0.1145E+01	0.6000E-01
			0.1293E+01	0.8000E-01
9	10	0.2146E+02	0.1484E+01	0.1200E+00
			0.1603E+01	0.1600E+00
			0.1917E+01	0.5000E+00
			0.2100E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.3695E+00	0.1000E-01
			0.6640E+00	0.2000E-01
			0.1104E+01	0.4000E-01
			0.1416E+01	0.6000E-01
			0.1650E+01	0.8000E-01
10	10	0.2150E+02	0.1976E+01	0.1200E+00
			0.2193E+01	0.1600E+00
			0.2824E+01	0.5000E+00
			0.3242E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.5881E+00	0.1000E-01
			0.1242E+01	0.2000E-01

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			BBFSR0102 14X89.ap7o	
11	10	0.2653E+02	0.2581E+01	0.4000E-01
			0.3169E+01	0.6000E-01
			0.3267E+01	0.8000E-01
			0.3169E+01	0.1200E+00
			0.3039E+01	0.1600E+00
			0.3039E+01	0.5000E+00
			0.3039E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.9375E+00	0.1000E-01
			0.1979E+01	0.2000E-01
12	10	0.3146E+02	0.4115E+01	0.4000E-01
			0.5052E+01	0.6000E-01
			0.5208E+01	0.8000E-01
			0.5052E+01	0.1200E+00
			0.4844E+01	0.1600E+00
			0.4844E+01	0.5000E+00
			0.4844E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.1104E+01	0.1000E-01
			0.2330E+01	0.2000E-01
13	10	0.3150E+02	0.4843E+01	0.4000E-01
			0.5947E+01	0.6000E-01
			0.6131E+01	0.8000E-01
			0.5947E+01	0.1200E+00
			0.5702E+01	0.1600E+00
			0.5702E+01	0.5000E+00
			0.5702E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.1038E+01	0.1000E-01
			0.1776E+01	0.2000E-01
14	10	0.4453E+02	0.2754E+01	0.4000E-01
			0.3373E+01	0.6000E-01
			0.3801E+01	0.8000E-01
			0.4352E+01	0.1200E+00
			0.4692E+01	0.1600E+00
			0.5583E+01	0.5000E+00
			0.6101E+01	0.1000E+02
			0.0000E+00	0.0000E+00
			0.1096E+01	0.1000E-01
			0.1952E+01	0.2000E-01

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			BBFSR0102 14X89.ap7o	
15	10	0.5746E+02	0.0000E+00	0.0000E+00
			0.1096E+01	0.1000E-01
			0.1952E+01	0.2000E-01
			0.3201E+01	0.4000E-01
			0.4070E+01	0.6000E-01
			0.4708E+01	0.8000E-01
			0.5585E+01	0.1200E+00
			0.6158E+01	0.1600E+00
			0.7789E+01	0.5000E+00
			0.8835E+01	0.1000E+02
16	10	0.5750E+02	0.0000E+00	0.0000E+00
			0.1096E+01	0.1000E-01
			0.1952E+01	0.2000E-01
			0.3201E+01	0.4000E-01
			0.4070E+01	0.6000E-01
			0.4708E+01	0.8000E-01
			0.5585E+01	0.1200E+00
			0.6158E+01	0.1600E+00
			0.7789E+01	0.5000E+00
			0.8835E+01	0.1000E+02
17	10	0.5978E+02	0.0000E+00	0.0000E+00
			0.1096E+01	0.1000E-01
			0.1952E+01	0.2000E-01
			0.3201E+01	0.4000E-01
			0.4070E+01	0.6000E-01
			0.4708E+01	0.8000E-01
			0.5585E+01	0.1200E+00
			0.6158E+01	0.1600E+00
			0.0000E+00	0.0000E+00
			0.1096E+01	0.1000E-01

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			BBFSR0102 14X89.ap7o	
			0.7789E+01	0.5000E+00
			0.8835E+01	0.1000E+02
18	10	0.6196E+02		
			0.0000E+00	0.0000E+00
			0.1096E+01	0.1000E-01
			0.1952E+01	0.2000E-01
			0.3201E+01	0.4000E-01
			0.4070E+01	0.6000E-01
			0.4708E+01	0.8000E-01
			0.5585E+01	0.1200E+00
			0.6158E+01	0.1600E+00
			0.7789E+01	0.5000E+00
			0.8835E+01	0.1000E+02

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.2392E+01	0.1000E-03
0.6362E+01	0.5000E-02
0.7566E+01	0.1000E-01
0.1131E+02	0.5000E-01
0.1345E+02	0.1000E+00
0.1600E+02	0.2000E+00
0.2012E+02	0.5000E+00
0.2392E+02	0.1000E+01
0.2845E+02	0.2000E+01

LOAD VERSUS SETTLEMENT CURVE

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.5833E+01	0.3863E-02	0.2392E+01	0.1000E-03
0.1140E+02	0.7894E-02	0.3122E+01	0.1000E-02

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			BBFSR0102 14X89.ap7o
0.3467E+02	0.2577E-01	0.6362E+01	0.5000E-02
0.5447E+02	0.4273E-01	0.7566E+01	0.1000E-01
0.1299E+03	0.1305E+00	0.1131E+02	0.5000E-01
0.1568E+03	0.2004E+00	0.1345E+02	0.1000E+00
0.2060E+03	0.6375E+00	0.2012E+02	0.5000E+00
0.2108E+03	0.1142E+01	0.2392E+02	0.1000E+01
0.2173E+03	0.2147E+01	0.2845E+02	0.2000E+01

WEAP Analysis - Abutment 1

GZA Geo Environmental, Inc.
Shortridge Bridge - Fryeburg Maine

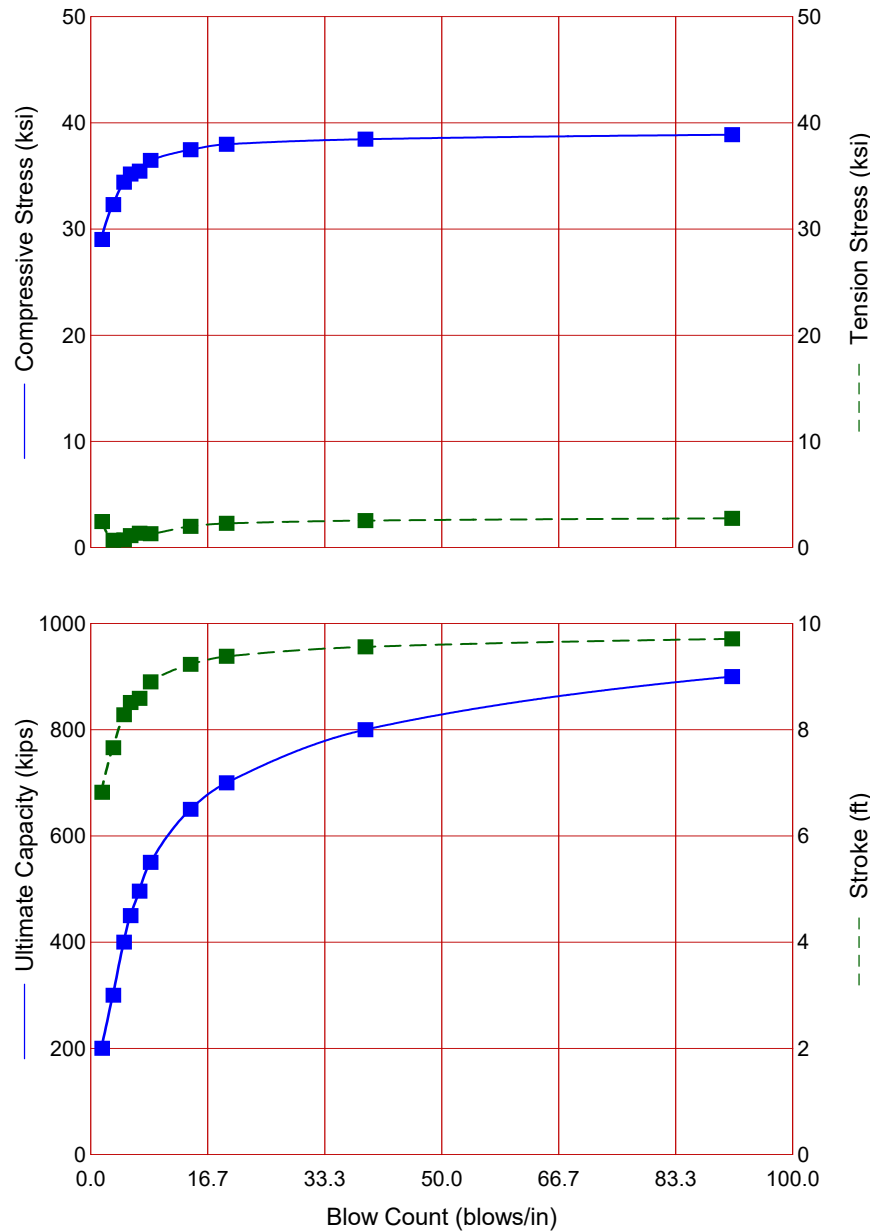
15-Nov-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
200.0	29.02	2.46	1.6	6.82	18.72
300.0	32.29	0.70	3.2	7.66	18.42
400.0	34.40	0.72	4.8	8.28	18.83
450.0	35.17	1.13	5.7	8.51	19.37
496.0	35.43	1.36	7.0	8.59	19.56
550.0	36.46	1.31	8.5	8.90	20.48
650.0	37.46	2.01	14.3	9.23	21.39
700.0	37.98	2.29	19.4	9.38	21.79
800.0	38.45	2.55	39.1	9.56	22.27
900.0	38.88	2.77	91.4	9.71	22.68

WEAP ANALYSIS - ABUTMENT 1

GZA Geo Environmental, Inc.
Shortridge Bridge - Fryeburg Maine

15-Nov-2018
GRLWEAP Version 2010

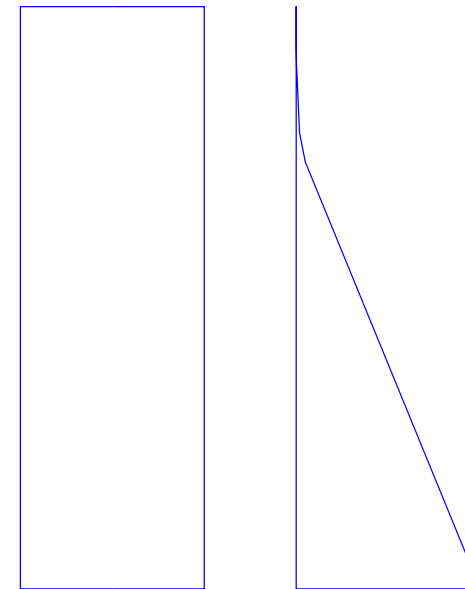


DELMAG D 22-02

Ram Weight	4.85 kips
Efficiency	0.800
Pressure	1000 (100%) psi
Helmet Weight	0.00 kips
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.070 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	60.00 ft
Pile Penetration	56.00 ft
Pile Top Area	26.10 in ²

Pile Model

Skin Friction
Distribution



Res. Shaft = 62 %
(Proportional)

WEAP Analysis - Abutment 2

GZA Geo Environmental, Inc.
Shortridge Bridge - Fryeburg Maine

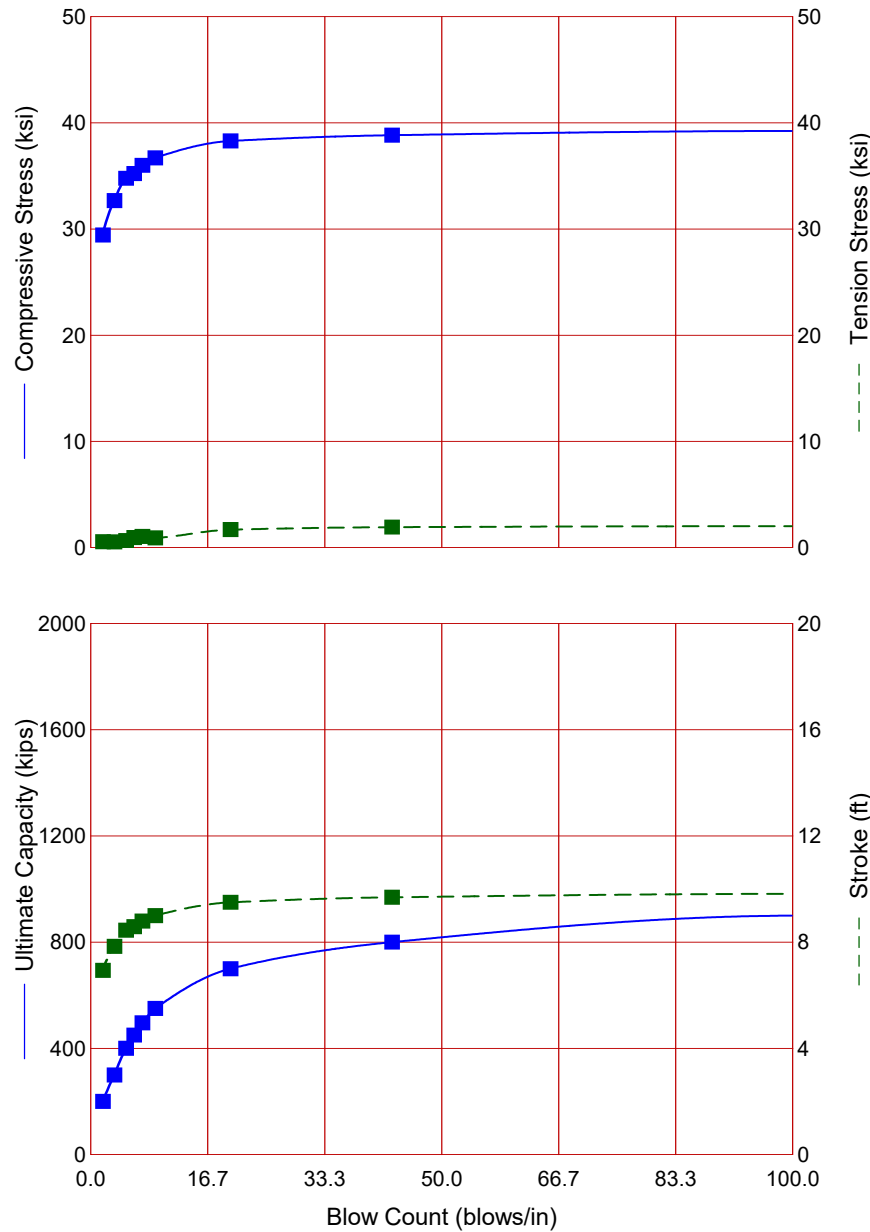
15-Nov-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
200.0	29.43	0.56	1.8	6.94	18.70
300.0	32.67	0.55	3.4	7.84	18.74
400.0	34.77	0.68	5.0	8.45	19.64
450.0	35.22	0.95	6.2	8.58	19.91
496.0	35.99	1.05	7.4	8.79	20.48
550.0	36.69	0.92	9.2	9.00	21.10
700.0	38.29	1.70	19.9	9.50	22.51
800.0	38.83	1.93	42.9	9.69	23.01
900.0	39.23	2.03	100.7	9.82	23.33
1000.0	39.86	1.97	9999.0	10.00	23.86

WEAP ANALYSIS - ABUTMENT 2

GZA Geo Environmental, Inc.
Shortridge Bridge - Fryeburg Maine

15-Nov-2018
GRLWEAP Version 2010



DELMAG D 22-02

Ram Weight	4.85 kips
Efficiency	0.800
Pressure	1000 (100%) psi
Helmet Weight	0.00 kips
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.070 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	60.00 ft
Pile Penetration	58.00 ft
Pile Top Area	26.10 in ²

Pile Model

Skin Friction
Distribution

Res. Shaft = 43 %
(Proportional)



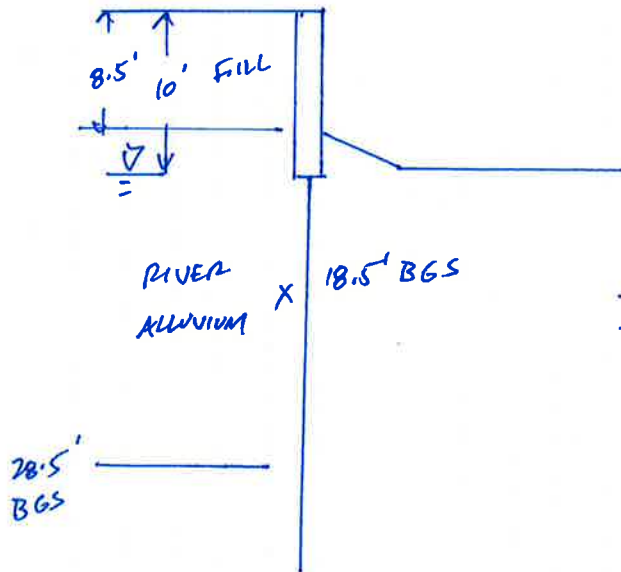
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<http://www.gza.com>

Engineers and
Scientists

JOB 09.0025967.00 Shattuck
SHEET NO. 1 OF 1
CALCULATED BY ARB DATE 3/16/18
CHECKED BY _____ DATE _____
SCALE _____

EVALUATE LIQUEFACTION-INDUCED DOWNDRAG



River Alluvium
 $t = 18.5'$ in along pile - subject to downdrag
Use β method, say $\beta = 0.35$
$$\sigma_v' = \frac{10}{8.5 \times 120 + 8.5 \times (115 - 62.4)}$$
$$= 1,650 \text{ psf}$$

$$f_{sdd} = \beta \sigma_v' = 580 \text{ psf}$$

Downdrag load on HP14 pile:
 $DD = 580 \text{ psf} \times 4 \times (14/12) \times 18.5' = 50 \text{ kips}$

Downdrag load on HP12 pile:
 $DD = 580 \text{ psf} \times 4 \times (12/12) \times 18.5' = 43 \text{ kips}$

LATERAL PILE ANALYSIS

Table 2 - LPile Input Parameters
Shortridge Bridge # Over Saco River Overflow Fryeburg, ME
 GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0025967.00
CALCULATED BY N. Williams
CHECKED BY A. Blaisdell

Abutment 1, Pile length = 56' (Boring: BB-FSRO-101)						
Stratum	Soil Model	Top of Layer Elevation (ft)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Fill	Reese Sand	372.0	1.5	130	39	63
Stream Alluvium	Reese Sand	370.5	7	10	27	43
Lacustrine Silt	Matlock Clay	363.5	3	$E_{50} = 0.008$	1000 psf	58
Lacustrine Sand	Reese Sand	360.5	44.5	145	40	68

Note:

1. Abutment 1 layering is based on stratification of boring BB-FSRO-101
2. S_u for SILT for was estimated from blow counts from -101 and 102 and experience with similar soils.

Abutment 2, Pile Length = 57', (Borings: BB-FSRO-102)						
Stratum	Soil Model	Top of Layer Elevation (ft)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Fill	Reese Sand	371	1.5	130	39	63
Stream Alluvium	Reese Sand	369.5	20	10	27	43
Lacustrine Silt	Matlock Clay	349.5	10	$E_{50} = 0.008$	1000 psf	58
Lacustrine Sand	Reese Sand	339.5	26	145	40	68

Note:

1. Abutment 2 layering is based on stratification of boring BB-FSRO-102.
2. S_u for SILT for was estimated from blow counts from -101 and 102 and experience with similar soils.

Table 1 - LPile Output Summary

Shortridge Bridge # Over Saco River Overflow Fryeburg, ME
GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0025967.00

CALCULATED BY N. Williams

CHECKED BY A. Blaisdell

Abutment I						
Lpile Summary Table						
Pile Type	Axial Load (kips)	Shear Force for Lateral deflection of 0.27 in. (kips)	Moment at Pile Head (ft-kips)	Total Stress at Pile Head (ksi)	Bending Stress at Pile Head (ksi)	Axial Stress at Pile Head (ksi)
HP 14x89	322.5	12.5	-61.7	28.2	15.8	12.4

Abutment II						
Lpile Summary Table						
Pile Type	Axial Load (kips)	Shear Force for Lateral deflection of 0.27 in. (kips)	Moment at Pile Head (ft-kips)	Total Stress at Pile Head (ksi)	Bending Stress at Pile Head (ksi)	Axial Stress at Pile Head (ksi)
HP 14x89	322.5	11.6	-54.1	13.8	1.4	12.4

Notes:

1. Soil layering and properties are presented in Table 2.
2. The axial load is the maximum Factored Axial Load, Strength Case.
3. The evaluated pile section is HP14x89, weak axis bending.

BBFSR0101 14x89.lp9o

LPILE for Windows, Version 2016-09.010

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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This copy of LPILE is being used by:

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GZA Portland

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is a violation of the software license agreement.

Files Used for Analysis

Path to file locations:

\09 Jobs\0025900s\09.0025967.00 - MDOT Shortridge Fryeburg - Stantec\Work\Calcs\Lpile\322.5 kip\BBFSR0101\14x89\

Name of input data file:

BBFSR0101 14x89.lp9d

Name of output report file:

BBFSR0101 14x89.lp9o

Name of plot output file:

BBFSR0101 14x89.lp9p

Page 1

BBFSR0101 14x89.lp9o

Name of runtime message file:

BBFSR0101 14x89.lp9r

Date and Time of Analysis

Date: July 2, 2018

Time: 9:31:20

Problem Title

Project Name: Shortridge Bridge

Job Number: 09.0025967.00

Client: Stantec

Engineer: Nicholas Williams

Description: Deep Foundation - Bridge Abutments

Program Options and Settings

Computational Options:

- Use unfactored loads in computations (conventional analysis)

Page 2

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
 - 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
 - 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 = 56.000 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.

Page 3

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	13.8000
2	56.000	13.8000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape = Weak H-Pile
 Length of section = 56.000000 ft
 Flange Width = 14.700000 in
 Section Depth = 13.800000 in
 Flange Thickness = 0.615000 in
 Web Thickness = 0.615000 in
 Section Area = 25.811550 sq. in
 Moment of Inertia = 325.837265 in^4
 Elastic Modulus = 29000000. psi

 Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
 = 0.000 radians

Pile Batter Angle = 0.000 degrees
 = 0.000 radians

Page 4

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

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

Distance from top of pile to top of layer	=	0.0000	ft
Distance from top of pile to bottom of layer	=	1.500000	ft
Effective unit weight at top of layer	=	63.000000	pcf
Effective unit weight at bottom of layer	=	63.000000	pcf
Friction angle at top of layer	=	39.000000	deg.
Friction angle at bottom of layer	=	39.000000	deg.
Subgrade k at top of layer	=	130.000000	pci
Subgrade k at bottom of layer	=	130.000000	pci

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

Distance from top of pile to top of layer	=	1.500000	ft
Distance from top of pile to bottom of layer	=	8.500000	ft
Effective unit weight at top of layer	=	43.000000	pcf
Effective unit weight at bottom of layer	=	43.000000	pcf
Friction angle at top of layer	=	27.000000	deg.
Friction angle at bottom of layer	=	27.000000	deg.
Subgrade k at top of layer	=	10.000000	pci
Subgrade k at bottom of layer	=	10.000000	pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	8.500000	ft
Distance from top of pile to bottom of layer	=	11.500000	ft
Effective unit weight at top of layer	=	58.000000	pcf
Effective unit weight at bottom of layer	=	58.000000	pcf
Undrained cohesion at top of layer	=	1000.000000	psf
Undrained cohesion at bottom of layer	=	1000.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

Page 5

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

Distance from top of pile to top of layer	=	11.500000	ft
Distance from top of pile to bottom of layer	=	56.000000	ft
Effective unit weight at top of layer	=	68.000000	pcf
Effective unit weight at bottom of layer	=	68.000000	pcf
Friction angle at top of layer	=	40.000000	deg.
Friction angle at bottom of layer	=	40.000000	deg.
Subgrade k at top of layer	=	145.000000	pci
Subgrade k at bottom of layer	=	145.000000	pci

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	0.00	63.0000	--	39.0000	--	130.0000
	(Reese, et al.)	1.5000	63.0000	--	39.0000	--	130.0000
2	Sand	1.5000	43.0000	--	27.0000	--	10.0000
	(Reese, et al.)	8.5000	43.0000	--	27.0000	--	10.0000
3	Stiff Clay	8.5000	58.0000	1000.0000	--	0.008000	--
	w/o Free Water	11.5000	58.0000	1000.0000	--	0.008000	--
4	Sand	11.5000	68.0000	--	40.0000	--	145.0000
	(Reese, et al.)	56.0000	68.0000	--	40.0000	--	145.0000

Static Loading Type

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

Page 6

 Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
1	5	y = 0.270000 in	S = 0.0000 in/in	322520.	N.A.

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:

Moment-curvature properties were derived from elastic section properties

 Layering Correction Equivalent Depths of Soil & Rock Layers

Page 7

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	1363.
2	1.5000	1.5000	Yes	No	1363.	17476.
3	8.5000	4.1074	No	No	18840.	20426.
4	11.5000	7.6506	No	No	39266.	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 = 0.270000 inches

Rotation of pile head = 0.000E+00 radians

Axial load on pile head = 322520.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb*2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	0.2718	-740362.	12520.	0.00	28173.	9.45E+09	0.00	0.00	0.00
0.5600	0.2700	-656635.	12238.	-4.97E-04	26400.	9.45E+09	-40.6090	1011.	0.00
1.1200	0.2651	-573731.	11795.	-9.34E-04	24645.	9.45E+09	-91.0739	2309.	0.00
1.6800	0.2574	-494055.	11315.	-0.00131	22957.	9.45E+09	-51.9007	1355.	0.00
2.2400	0.2474	-415961.	10917.	-0.00164	21304.	9.45E+09	-66.5101	1806.	0.00
2.8000	0.2354	-340229.	10441.	-0.00191	19700.	9.45E+09	-75.1609	2145.	0.00

Page 8

BBFSR0101 14x89.1p9o									
3.3600	0.2218	-267368.	9922.	-0.00212	18157.	9.45E+09	-79.3199	2403.	0.00
3.9200	0.2069	-197676.	9374.	-0.00229	16681.	9.45E+09	-83.9504	2727.	0.00
4.4800	0.1911	-131470.	8807.	-0.00240	15279.	9.45E+09	-84.6121	2976.	0.00
5.0400	0.1746	-68883.	8217.	-0.00248	13954.	9.45E+09	-90.9195	3500.	0.00
5.6000	0.1578	-10295.	7582.	-0.00250	12713.	9.45E+09	-98.0659	4177.	0.00
6.1600	0.1409	43880.	6903.	-0.00249	13424.	9.45E+09	-104.1765	4967.	0.00
6.7200	0.1243	93283.	6216.	-0.00244	14471.	9.45E+09	-100.2257	5419.	0.00
7.2800	0.1081	138016.	5562.	-0.00236	15418.	9.45E+09	-94.4276	5871.	0.00
7.8400	0.09255	178272.	4952.	-0.00225	16270.	9.45E+09	-87.0730	6322.	0.00
8.4000	0.07787	214321.	4396.	-0.00211	17034.	9.45E+09	-78.4889	6774.	0.00
8.9600	0.06420	246496.	3527.	-0.00195	17715.	9.45E+09	-179.9821	18838.	0.00
9.5200	0.05172	270163.	2321.	-0.00176	18216.	9.45E+09	-179.2164	23286.	0.00
10.0800	0.04053	285320.	1124.	-0.00156	18537.	9.45E+09	-176.8079	29317.	0.00
10.6400	0.03070	292053.	-49.6768	-0.00136	18680.	9.45E+09	-172.5902	37781.	0.00
11.2000	0.02226	290542.	-1188.	-0.00115	18648.	9.45E+09	-166.3298	50202.	0.00
11.7600	0.01522	281073.	-2404.	-9.48E-04	18447.	9.45E+09	-195.3771	86264.	0.00
12.3200	0.00952	262347.	-3618.	-7.55E-04	18051.	9.45E+09	-166.1343	117293.	0.00
12.8800	0.00507	235714.	-4558.	-5.78E-04	17487.	9.45E+09	-113.6317	150603.	0.00
13.4400	0.00175	203587.	-5078.	-4.22E-04	16806.	9.45E+09	-40.8982	157151.	0.00
14.0000	-6.00E-04	169298.	-5166.	-2.89E-04	16080.	9.45E+09	14.6072	163699.	0.00
14.5600	-0.00214	135409.	-4935.	-1.81E-04	15363.	9.45E+09	54.1918	170247.	0.00
15.1200	-0.00303	103758.	-4485.	-9.59E-05	14692.	9.45E+09	79.7513	176795.	0.00
15.6800	-0.00343	75549.	-3903.	-3.21E-05	14095.	9.45E+09	93.5210	183343.	0.00
16.2400	-0.00346	51446.	-3260.	1.30E-05	13585.	9.45E+09	97.8609	189891.	0.00
16.8000	-0.00325	31683.	-2611.	4.26E-05	13166.	9.45E+09	95.0827	196439.	0.00
17.3600	-0.00289	16166.	-1998.	5.96E-05	12838.	9.45E+09	87.3205	202987.	0.00
17.9200	-0.00245	4566.	-1448.	6.70E-05	12592.	9.45E+09	76.4443	209535.	0.00
18.4800	-0.00199	-3588.	-976.2395	6.73E-05	12571.	9.45E+09	64.0105	216083.	0.00
19.0400	-0.00155	-8846.	-588.9764	6.29E-05	12683.	9.45E+09	51.2464	222631.	0.00
19.6000	-0.00115	-11777.	-285.5495	5.56E-05	12745.	9.45E+09	39.0592	229179.	0.00
20.1600	-8.00E-04	-12925.	-60.0166	4.68E-05	12769.	9.45E+09	28.0637	235727.	0.00
20.7200	-5.17E-04	-12786.	96.8481	3.76E-05	12766.	9.45E+09	18.6222	242275.	0.00
21.2800	-2.94E-04	-11786.	196.0120	2.89E-05	12745.	9.45E+09	10.8908	248823.	0.00
21.8400	-1.28E-04	-10277.	248.9566	2.11E-05	12713.	9.45E+09	4.8665	255371.	0.00
22.4000	-1.11E-05	-8532.	266.7624	1.44E-05	12676.	9.45E+09	0.4328	261919.	0.00
22.9600	6.51E-05	-6754.	259.4810	8.94E-06	12638.	9.45E+09	-2.5999	268467.	0.00
23.5200	1.09E-04	-5083.	235.7592	4.73E-06	12603.	9.45E+09	-4.4602	275915.	0.00
24.0800	1.29E-04	-3606.	202.6688	1.64E-06	12572.	9.45E+09	-5.3882	281563.	0.00
24.6400	1.31E-04	-2366.	165.6960	-4.87E-07	12545.	9.45E+09	-5.6156	288111.	0.00
25.2000	1.22E-04	-1377.	128.8451	-1.82E-06	12524.	9.45E+09	-5.3519	294659.	0.00
25.7600	1.07E-04	-626.6466	94.8157	-2.53E-06	12508.	9.45E+09	-4.7759	301207.	0.00

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26.3200	8.81E-05	-91.3541	65.2196	-2.79E-06	12497.	9.45E+09	-4.0325	307754.	0.00
26.8800	6.91E-05	261.9782	40.8087	-2.72E-06	12501.	9.45E+09	-3.2326	314302.	0.00
27.4400	5.14E-05	468.9260	21.6961	-2.46E-06	12505.	9.45E+09	-2.4557	320850.	0.00
28.0000	3.60E-05	564.2574	7.5537	-2.10E-06	12507.	9.45E+09	-1.7534	327398.	0.00
28.5600	2.32E-05	579.5387	-2.2187	-1.69E-06	12507.	9.45E+09	-1.1550	333946.	0.00
29.1200	1.33E-05	541.7666	-8.3582	-1.29E-06	12507.	9.45E+09	-0.6722	340494.	0.00
29.6800	5.88E-06	472.8046	-11.6369	-9.31E-07	12505.	9.45E+09	-0.3036	347042.	0.00
30.2400	7.51E-07	389.4037	-12.7897	-6.25E-07	12503.	9.45E+09	-0.03951	353590.	0.00
30.8000	-2.52E-06	303.6186	-12.4693	-3.78E-07	12502.	9.45E+09	0.1348	360138.	0.00
31.3600	-4.33E-06	223.4550	-11.2220	-1.91E-07	12500.	9.45E+09	0.2364	366686.	0.00
31.9200	-5.08E-06	153.6221	-9.4796	-5.67E-08	12498.	9.45E+09	0.2822	373234.	0.00
32.4800	-5.09E-06	96.2944	-7.5642	3.22E-08	12497.	9.45E+09	0.2879	379782.	0.00
33.0400	-4.65E-06	51.8195	-5.6991	8.48E-08	12496.	9.45E+09	0.2672	386330.	0.00
33.6000	-3.95E-06	19.3314	-4.0245	1.10E-07	12496.	9.45E+09	0.2312	392878.	0.00
34.1600	-3.17E-06	-2.7471	-2.6151	1.16E-07	12495.	9.45E+09	0.1883	399426.	0.00
34.7200	-2.39E-06	-16.3187	-1.4964	1.09E-07	12496.	9.45E+09	0.1447	405974.	0.00
35.2800	-1.70E-06	-23.3323	-0.6598	9.52E-08	12496.	9.45E+09	0.1043	412522.	0.00
35.8400	-1.12E-06	-25.5988	-0.07547	7.78E-08	12496.	9.45E+09	0.06958	419070.	0.00
36.4000	-6.54E-07	-24.6837	0.2976	5.99E-08	12496.	9.45E+09	0.04145	425618.	0.00
36.9600	-3.11E-07	-21.8587	0.5041	4.33E-08	12496.	9.45E+09	0.02001	432166.	0.00
37.5200	-7.22E-08	-18.0966	0.5871	2.91E-08	12496.	9.45E+09	0.00471	438714.	0.00
38.0800	8.02E-08	-14.0938	0.5851	1.77E-08	12495.	9.45E+09	-0.00532	445262.	0.00
38.6400	1.65E-07	-10.3094	0.5299	8.99E-09	12495.	9.45E+09	-0.01111	451810.	0.00
39.2000	2.01E-07	-7.0110	0.4465	2.83E-09	12495.	9.45E+09	-0.01372	458358.	0.00
39.7600	2.03E-07	-4.3212	0.3531	-1.20E-09	12495.	9.45E+09	-0.01407	464906.	0.00
40.3200	1.85E-07	-2.2601	0.2622	-3.54E-09	12495.	9.45E+09	-0.01298	471454.	0.00
40.8800	1.56E-07	-0.7818	0.1813	-4.62E-09	12495.	9.45E+09	-0.01109	478002.	0.00
41.4400	1.23E-07	0.1970	0.1143	-4.83E-09	12495.	9.45E+09	-0.00887	484550.	0.00
42.0000	9.10E-08	0.7752	0.06215	-4.48E-09	12495.	9.45E+09	-0.00665	491098.	0.00
42.5600	6.28E-08	1.0518	0.02419	-3.83E-09	12495.	9.45E+09	-0.00465	497646.	0.00
43.1200	3.95E-08	1.1169	-0.00139	-3.06E-09	12495.	9.45E+09	-0.00297	504194.	0.00
43.6800	2.16E-08	1.0463	-0.01688	-2.29E-09	12495.	9.45E+09	-0.00164	510742.	0.00
44.2400	8.75E-09	0.8999	-0.02467	-1.60E-09	12495.	9.45E+09	-6.74E-04	517289.	0.00
44.8000	1.64E-10	0.7216	-0.02698	-1.02E-09	12495.	9.45E+09	-1.28E-05	523837.	0.00
45.3600	-4.98E-09	0.5417	-0.02570	-5.72E-10	12495.	9.45E+09	3.93E-04	530385.	0.00
45.9200	-7.53E-09	0.3787	-0.02236	-2.45E-10	12495.	9.45E+09	6.01E-04	536933.	0.00
46.4800	-8.27E-09	0.2422	-0.01810	-2.41E-11	12495.	9.45E+09	6.69E-04	543481.	0.00
47.0400	-7.85E-09	0.1356	-0.01369	1.10E-10	12495.	9.45E+09	6.43E-04	550029.	0.00
47.6000	-6.79E-09	0.05774	-0.00964	1.79E-10	12495.	9.45E+09	5.62E-04	556577.	0.00
48.1600	-5.45E-09	0.00520	-0.00622	2.01E-10	12495.	9.45E+09	4.56E-04	563125.	0.00
48.7200	-4.08E-09	-0.02673	-0.00352	1.94E-10	12495.	9.45E+09	3.46E-04	569673.	0.00

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49.2800	-2.84E-09	-0.04300	-0.00154	1.69E-10	12495.	9.45E+09	2.44E-04	576221.	0.00
49.8400	-1.81E-09	-0.04819	-1.94E-04	1.36E-10	12495.	9.45E+09	1.57E-04	582769.	0.00
50.4000	-1.01E-09	-0.04620	6.31E-04	1.03E-10	12495.	9.45E+09	8.86E-05	589317.	0.00
50.9600	-4.30E-10	-0.04015	0.00106	7.21E-11	12495.	9.45E+09	3.81E-05	595865.	0.00
51.5200	-4.09E-11	-0.03231	0.00120	4.64E-11	12495.	9.45E+09	3.67E-06	602413.	0.00
52.0800	1.94E-10	-0.02425	0.00115	2.63E-11	12495.	9.45E+09	-1.75E-05	608961.	0.00
52.6400	3.12E-10	-0.01695	9.96E-04	1.16E-11	12495.	9.45E+09	-2.86E-05	615509.	0.00
53.2000	3.50E-10	-0.01092	7.91E-04	1.71E-12	12495.	9.45E+09	-3.24E-05	622057.	0.00
53.7600	3.35E-10	-0.00633	5.77E-04	-4.42E-12	12495.	9.45E+09	-3.14E-05	628605.	0.00
54.3200	2.90E-10	-0.00314	3.79E-04	-7.79E-12	12495.	9.45E+09	-2.74E-05	635153.	0.00
54.8800	2.31E-10	-0.00119	2.13E-04	-9.33E-12	12495.	9.45E+09	-2.20E-05	641701.	0.00
55.4400	1.65E-10	-2.38E-04	8.57E-05	-9.84E-12	12495.	9.45E+09	-1.59E-05	648249.	0.00
56.0000	9.84E-11	0.00	0.00	-9.92E-12	12495.	9.45E+09	-9.58E-06	327398.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 0.27176911 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -740362. inch-lbs
 Maximum shear force = 12520. 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 = 6

 Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

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

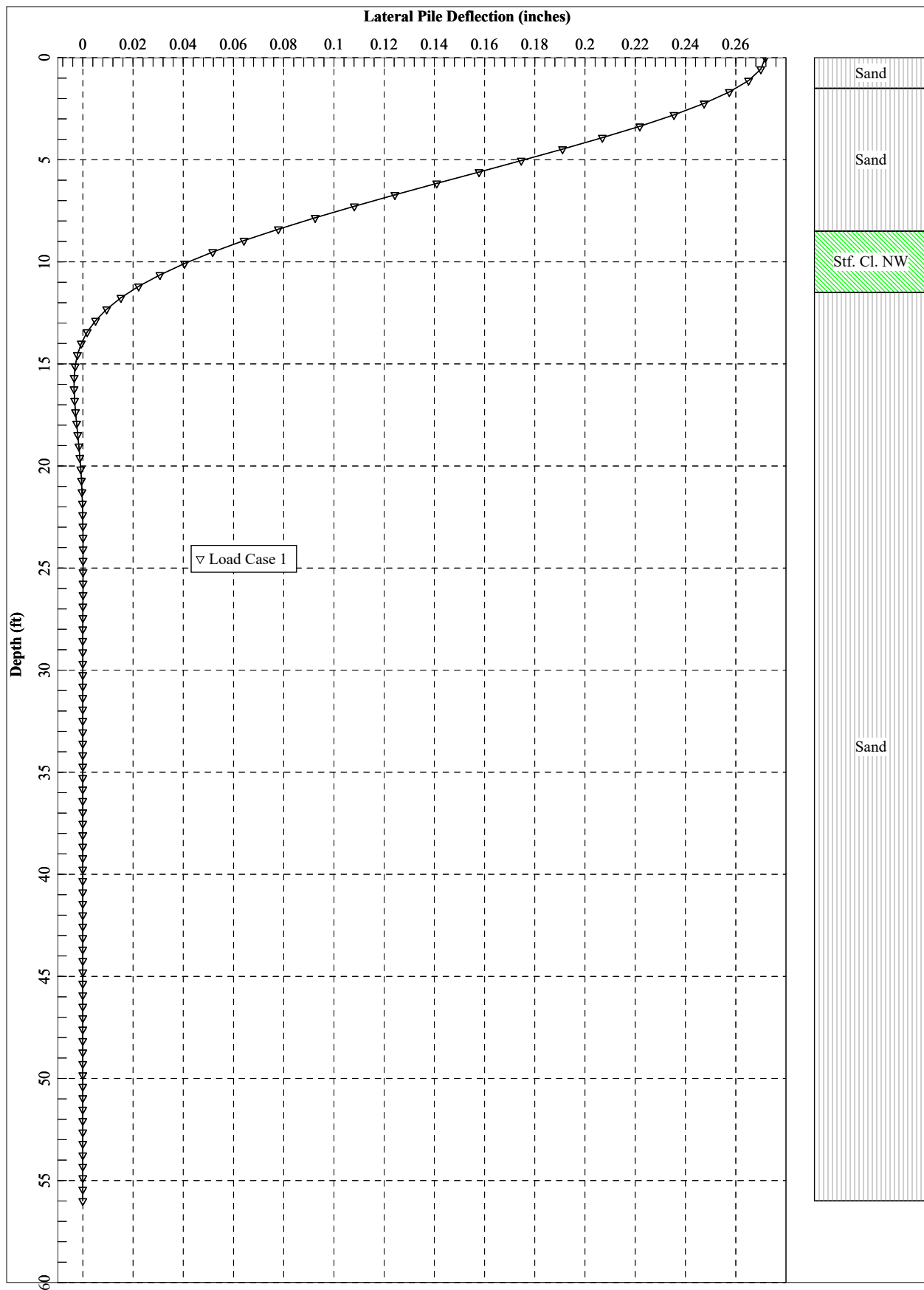
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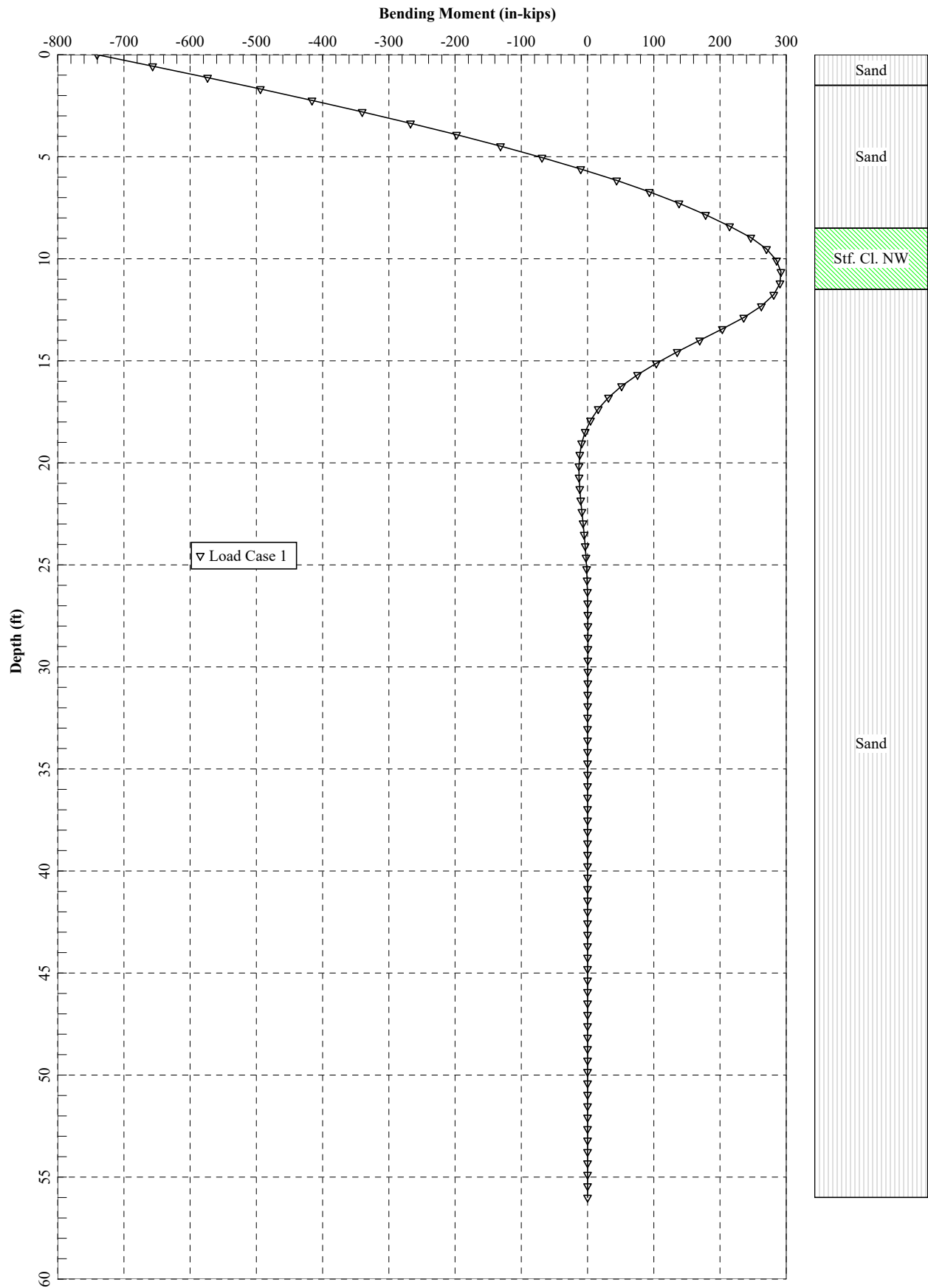
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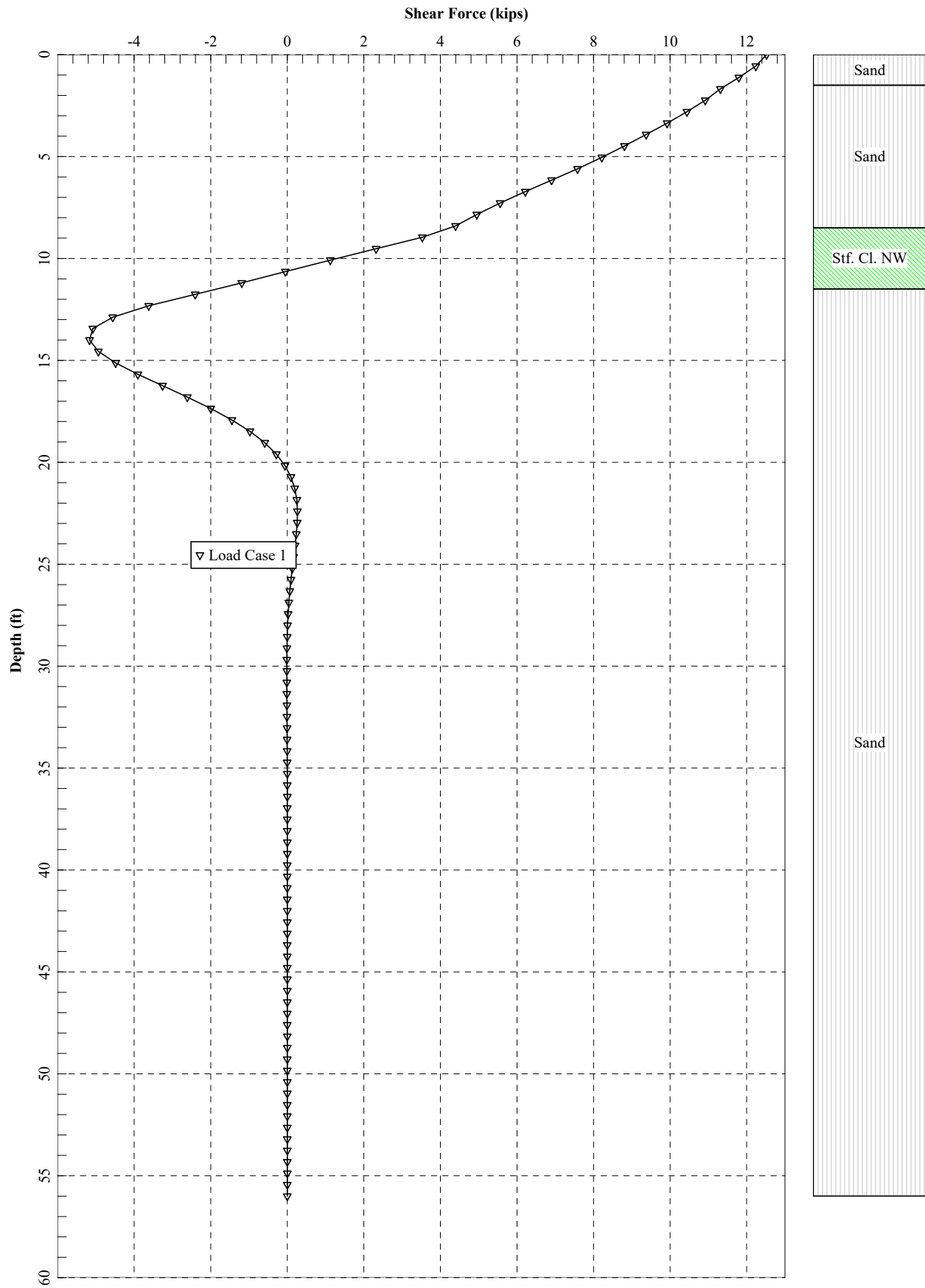
Load Case No.	Load Type	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	0.2700	S, rad	0.00	322520.	0.2718	0.00	12520.	-740362.

Maximum pile-head deflection = 0.2717691054 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.









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 SUBJECT: Pile Evaluation for Integral Abutment
 SHEET: 1 OF 9
 CALCULATED BY N. Williams
11/15/2018

Integral Abutment LRFD Pile Design

Subject: Pile Design for the Shortridge Bridge Replacement in Fryeburg, Maine.

Reference:

- AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017
- Maine BDG Chapter 5 - Substructures 2014
- VTRANS Integral Abutment Bridge Design Guidelines 2008

Design Steps - Maine BDG

Step 1 - Determine the foundation displacements and the load effects (P_u and M_u) from the superstructure and substructure designs.

$P_u := 322.5 \text{ kip}$ Maximum Factored Axial Load

0.27" thermal movement given by Stantec at each abutment.

Step 2 - If applicable, determine the magnitude of scour.

N/A

Step 3 - Select preliminary pile size.

HP14 x 89, Weak Axis Properties

Steel yield strength	$F_{y50} := 50 \text{ ksi}$
Modulus of elasticity for steel	$E := 29000 \text{ ksi}$
Cross sectional area of pile	$A_g := 26.1 \text{ in}^2$
Radius of gyration	$r_y := 3.53 \text{ in}$
Width of Flange	$b_f := 14.7 \text{ in}$
Thickness of Flange	$t_f := 0.615 \text{ in}$
Elastic Section Modulus	$S_y := 44.3 \text{ in}^3$
Plastic Section Modulus	$Z_y := 67.7 \text{ in}^3$

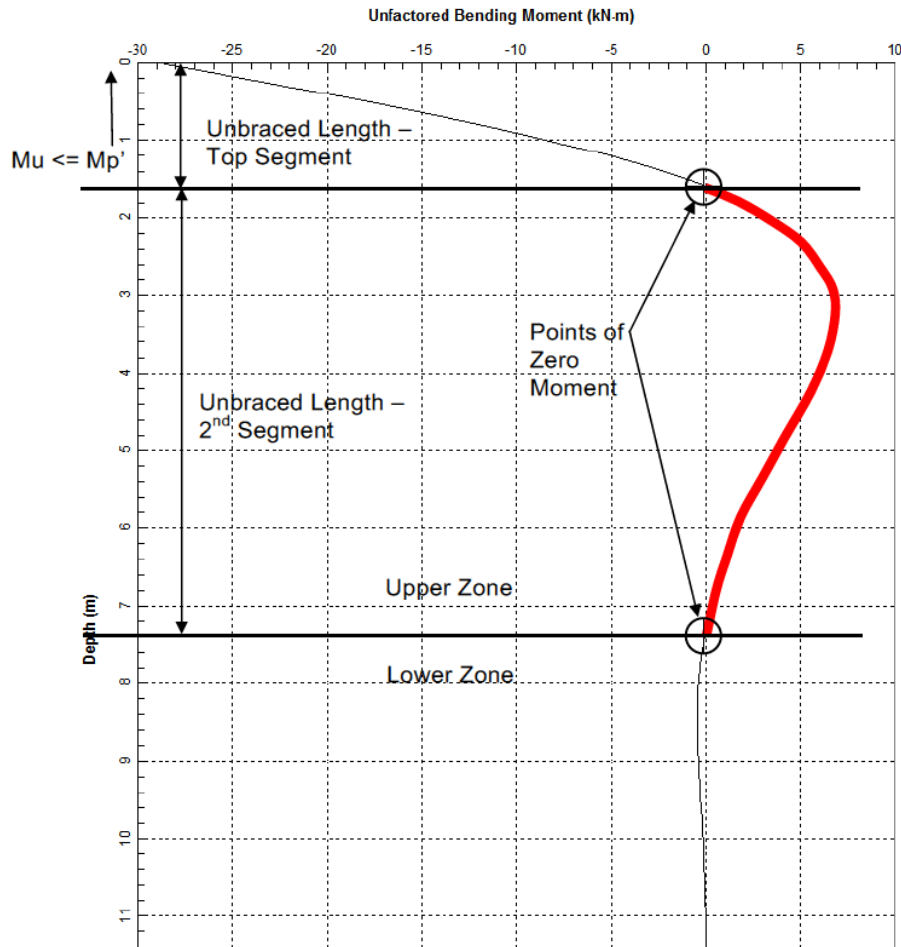


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Step 4 - Determine the pile unbraced length and maximum moment at the top of the pile by running LPile software for
top translation = 0.27 inches, $P_u = 322.5$ kip, and Live Load Rotation = 0 (Fixed against rotation)



Maximum moment from LPile output

$$M_u := 740.362 \text{ kip}\cdot\text{in} = 61.697 \cdot \text{kip}\cdot\text{ft}$$

Unbraced lengths from LPile output

Upper segment

$$L_1 := 6 \cdot \text{ft}$$

Lower segment

$$L_2 := 12 \cdot \text{ft}$$



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Step 5 - Determine if the applied moment on the pile will cause pile head plastic deformation considering the interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment.

See above for unbraced lengths (critical lengths L_1 and L_2).

Upper segment slenderness factor

$$\lambda_1 := \frac{(L_1)}{r_y} = 20.397$$

Lower segment slenderness factor

$$\lambda_2 := \frac{(L_2)}{r_y} = 40.793$$

b. Determine K values for top and bottom of the pile per LRFD Table C4.6.2.5-1

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of K when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0
End condition code		Rotation fixed Translation fixed Rotation free Translation fixed Rotation fixed Translation free Rotation free Translation free				

Upper segment K value (Type d)

$$K_1 := 1.2$$

Lower segment K value (Type c)

$$K_2 := 1$$



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c. Calculate the nominal and factored structural pile resistance P_n , per LRFD 6.9.4.1 using the λ values

Elastic critical buckling resistance, P_e , based on flexural buckling

$$\text{Upper } P_e \quad P_{e1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_1 \cdot \lambda_1)^2} = 12469.8 \cdot \text{kip}$$

$$\text{Lower } P_e \quad P_{e2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_2 \cdot \lambda_2)^2} = 4489.1 \cdot \text{kip}$$

$$\text{Nominal yield resistance, } P_o \quad P_o := F_{y50} \cdot A_g = 1305 \cdot \text{kip}$$

Check that the ratio of P_e to P_o is > 0.44

If $P_e/P_o > 0.44$ then use equation 6.9.4.1.1-1

If $P_e/P_o < 0.44$ then use equation 6.9.4.1.1-2

$$P_n = \left[0.658 \left(\frac{P_o}{P_e} \right) \right] P_o \quad (6.9.4.1.1-1)$$

$$P_n = 0.877 P_e \quad (6.9.4.1.1-2)$$

$$\frac{P_{e1}}{P_o} = 9.555 \quad \frac{P_{e2}}{P_o} = 3.44$$

Use eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{n1} := \left[0.658 \left(\frac{P_o}{P_{e1}} \right) \right] P_o = 1249.07 \cdot \text{kip}$$

$$P_{n2} := \left[0.658 \left(\frac{P_o}{P_{e2}} \right) \right] P_o = 1155.5 \cdot \text{kip}$$



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Factored structural Pile Resistance, $P_r = \phi_c (P_n)$

$$\phi_c := .7$$

$$P_{r1} := \phi_c \cdot P_{n1} = 874.4 \cdot \text{kip}$$

$$P_{r2} := \phi_c \cdot P_{n2} = 808.8 \cdot \text{kip}$$

d. Compare the ratio of P_u , the maximum factored axial load, to P_r , the structural resistance in the specified portion of the pile - the pile size should be such that the ratio is not less than 0.20.

Check for both segments

$$\frac{P_u}{P_{r1}} = 0.369$$

$$\frac{P_u}{P_{r2}} = 0.399$$

e. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)

Check slenderness ratio for flange, limiting slenderness ratio for compact flange, and limiting slenderness ratio for a noncompact flange.

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$$

slenderness ratio for flange eq. 6.12.2.2.1-3

$$\lambda_{pf} := 0.38 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 9.152$$

$$\lambda_{rf} := 0.83 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 19.989$$

If $\lambda_{pf} < \lambda_f < \lambda_{rf}$ Use equation 6.12.2.2.1-2 to find the nominal flexural resistance

$$M_n := \left[1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left[\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \left(\frac{E}{F_{y50}} \right)^{.5}} \right] \right] \cdot F_{y50} \cdot Z_y = 256.9 \cdot \text{ft} \cdot \text{kip}$$



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*Engineers and
Scientists*

JOB: 09.0025967.00
SUBJECT: Pile Evaluation for Integral Abutment
SHEET: 6 OF 9
CALCULATED BY N. Williams
11/15/2018

$\phi_f := 1.0$ for flexural resistance according to Maine BDG

$$M_r := M_n \cdot \phi_f = 256.9 \cdot \text{ft} \cdot \text{kip}$$

$M_u = 61.7 \cdot \text{ft} \cdot \text{kip}$ M_u from LPile > than M_r , exceeding the flexural resistance

$$\frac{P_u}{P_{r1}} + \frac{8}{9} \cdot \frac{M_u}{M_r} = 0.6$$

If less than 1, remains in elastic zone. The pile is not expected to yield at the base of the pile cap.

BBFSR0102 14x89.lp9o

LPIle for Windows, Version 2016-09.010

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:

\09 Jobs\0025900s\09.0025967.00 - MDOT Shortridge Fryeburg - Stantec\Work\Calcs\Lpile\322.5 kip\BBFSR0102\14x89\

Name of input data file:

BBFSR0102 14x89.lp9d

Name of output report file:

BBFSR0102 14x89.lp9o

Name of plot output file:

BBFSR0102 14x89.lp9p

Page 1

BBFSR0102 14x89.lp9o

Name of runtime message file:

BBFSR0102 14x89.lp9r

Date and Time of Analysis

Date: November 15, 2018

Time: 10:13:30

Problem Title

Project Name: Shortridge Bridge

Job Number: 09.0025967.00

Client: Stantec

Engineer: Nicholas Williams

Description: Deep Foundation - Bridge Abutments

Program Options and Settings

Computational Options:

- Use unfactored loads in computations (conventional analysis)

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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
 - 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
 - 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 = 57.000 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.

Page 3

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	13.8000
2	57.000	13.8000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape = Weak H-Pile
 Length of section = 57.000000 ft
 Flange Width = 14.700000 in
 Section Depth = 13.800000 in
 Flange Thickness = 0.615000 in
 Web Thickness = 0.615000 in
 Section Area = 25.811550 sq. in
 Moment of Inertia = 325.837265 in^4
 Elastic Modulus = 29000000. psi

 Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
 = 0.000 radians

Pile Batter Angle = 0.000 degrees
 = 0.000 radians

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Soil and Rock Layering Information

The soil profile is modelled using 4 layers

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

Distance from top of pile to top of layer	=	0.0000	ft
Distance from top of pile to bottom of layer	=	1.500000	ft
Effective unit weight at top of layer	=	63.000000	pcf
Effective unit weight at bottom of layer	=	63.000000	pcf
Friction angle at top of layer	=	39.000000	deg.
Friction angle at bottom of layer	=	39.000000	deg.
Subgrade k at top of layer	=	130.000000	pci
Subgrade k at bottom of layer	=	130.000000	pci

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

Distance from top of pile to top of layer	=	1.500000	ft
Distance from top of pile to bottom of layer	=	21.500000	ft
Effective unit weight at top of layer	=	43.000000	pcf
Effective unit weight at bottom of layer	=	43.000000	pcf
Friction angle at top of layer	=	27.000000	deg.
Friction angle at bottom of layer	=	27.000000	deg.
Subgrade k at top of layer	=	10.000000	pci
Subgrade k at bottom of layer	=	10.000000	pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	21.500000	ft
Distance from top of pile to bottom of layer	=	31.500000	ft
Effective unit weight at top of layer	=	58.000000	pcf
Effective unit weight at bottom of layer	=	58.000000	pcf
Undrained cohesion at top of layer	=	1000.000000	psf
Undrained cohesion at bottom of layer	=	1000.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

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

Distance from top of pile to top of layer	=	31.500000	ft
Distance from top of pile to bottom of layer	=	57.500000	ft
Effective unit weight at top of layer	=	68.000000	pcf
Effective unit weight at bottom of layer	=	68.000000	pcf
Friction angle at top of layer	=	40.000000	deg.
Friction angle at bottom of layer	=	40.000000	deg.
Subgrade k at top of layer	=	145.000000	pci
Subgrade k at bottom of layer	=	145.000000	pci

(Depth of the lowest soil layer extends 0.500 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	0.00	63.0000	--	39.0000	--	130.0000
	(Reese, et al.)	1.5000	63.0000	--	39.0000	--	130.0000
2	Sand	1.5000	43.0000	--	27.0000	--	10.0000
	(Reese, et al.)	21.5000	43.0000	--	27.0000	--	10.0000
3	Stiff Clay	21.5000	58.0000	1000.0000	--	0.008000	--
	w/o Free Water	31.5000	58.0000	1000.0000	--	0.008000	--
4	Sand	31.5000	68.0000	--	40.0000	--	145.0000
	(Reese, et al.)	57.5000	68.0000	--	40.0000	--	145.0000

Static Loading Type

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

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 Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
1	5	y = 0.270000 in	S = 0.0000 in/in	322.50000000	N.A.

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:

Moment-curvature properties were derived from elastic section properties

 Layering Correction Equivalent Depths of Soil & Rock Layers

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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	1363.
2	1.5000	1.5000	Yes	No	1363.	185824.
3	21.5000	21.5000	No	No	187187.	103500.
4	31.5000	31.5000	No	No	290687.	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 = 0.270000 inches

Rotation of pile head = 0.000E+00 radians

Axial load on pile head = 322.5 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb*2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	0.2716	-649208.	11638.	0.00	13760.	9.45E+09	0.00	0.00	0.00
0.5700	0.2700	-570576.	11354.	-4.41E-04	12095.	9.45E+09	-41.4600	1050.	0.00
1.1400	0.2656	-493882.	10894.	-8.27E-04	10471.	9.45E+09	-92.9773	2395.	0.00
1.7100	0.2587	-421538.	10395.	-0.00116	8939.	9.45E+09	-53.0832	1404.	0.00
2.2800	0.2497	-351676.	9980.	-0.00144	7460.	9.45E+09	-68.3249	1871.	0.00
2.8500	0.2390	-285011.	9486.	-0.00167	6048.	9.45E+09	-76.0290	2176.	0.00

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3.4200	0.2269	-221902.	8951.	-0.00185	4712.	9.45E+09	-80.3835	2423.	0.00
3.9900	0.2137	-162554.	8382.	-0.00199	3455.	9.45E+09	-85.9596	2752.	0.00
4.5600	0.1997	-107227.	7791.	-0.00209	2283.	9.45E+09	-86.8805	2976.	0.00
5.1300	0.1851	-55965.	7165.	-0.00215	1198.	9.45E+09	-96.2543	3557.	0.00
5.7000	0.1703	-9206.	6476.	-0.00217	207.4323	9.45E+09	-105.0143	4218.	0.00
6.2700	0.1554	32640.	5720.	-0.00216	703.6961	9.45E+09	-116.0286	5107.	0.00
6.8400	0.1407	69058.	4929.	-0.00213	1475.	9.45E+09	-115.4858	5614.	0.00
7.4100	0.1263	100072.	4149.	-0.00206	2132.	9.45E+09	-112.3312	6082.	0.00
7.9800	0.1125	125831.	3397.	-0.00198	2677.	9.45E+09	-107.6852	6550.	0.00
8.5500	0.09920	146551.	2681.	-0.00188	3116.	9.45E+09	-101.7803	7018.	0.00
9.1200	0.08667	162509.	2008.	-0.00177	3454.	9.45E+09	-94.8565	7486.	0.00
9.6900	0.07495	174029.	1386.	-0.00165	3698.	9.45E+09	-87.1548	7954.	0.00
10.2600	0.06409	181472.	817.6537	-0.00152	3855.	9.45E+09	-78.9104	8421.	0.00
10.8300	0.05413	185222.	307.1901	-0.00139	3935.	9.45E+09	-70.3480	8889.	0.00
11.4000	0.04509	185680.	-144.3365	-0.00126	3944.	9.45E+09	-61.6774	9357.	0.00
11.9700	0.03696	183253.	-536.8407	-0.00112	3893.	9.45E+09	-53.0900	9825.	0.00
12.5400	0.02974	178341.	-871.4753	-9.91E-04	3789.	9.45E+09	-44.7564	10293.	0.00
13.1100	0.02341	171335.	-1150.	-8.64E-04	3641.	9.45E+09	-36.8245	10761.	0.00
13.6800	0.01792	162606.	-1377.	-7.43E-04	3456.	9.45E+09	-29.4186	11229.	0.00
14.2500	0.01324	152501.	-1555.	-6.29E-04	3242.	9.45E+09	-22.6389	11696.	0.00
14.8200	0.00931	141336.	-1689.	-5.23E-04	3005.	9.45E+09	-16.5615	12164.	0.00
15.3900	0.00609	129396.	-1784.	-4.25E-04	2753.	9.45E+09	-11.2393	12632.	0.00
15.9600	0.00350	116930.	-1846.	-3.36E-04	2489.	9.45E+09	-6.7028	13100.	0.00
16.5300	0.00149	104150.	-1879.	-2.56E-04	2218.	9.45E+09	-2.9608	13568.	0.00
17.1000	1.19E-06	91231.	-1889.	-1.85E-04	1944.	9.45E+09	-0.00245	14036.	0.00
17.6700	-0.00104	78312.	-1881.	-1.24E-04	1671.	9.45E+09	2.2021	14504.	0.00
18.2400	-0.00169	65496.	-1861.	-7.16E-05	1399.	9.45E+09	3.7003	14971.	0.00
18.8100	-0.00202	52853.	-1833.	-2.88E-05	1132.	9.45E+09	4.5557	15439.	0.00
19.3800	-0.00208	40424.	-1801.	4.98E-06	868.5121	9.45E+09	4.8473	15907.	0.00
19.9500	-0.00195	28220.	-1768.	2.98E-05	610.0956	9.45E+09	4.6688	16375.	0.00
20.5200	-0.00168	16236.	-1738.	4.59E-05	356.3036	9.45E+09	4.1278	16843.	0.00
21.0900	-0.00132	4444.	-1712.	5.34E-05	106.6008	9.45E+09	3.3460	17311.	0.00
21.6600	-9.46E-04	-7191.	-1342.	5.24E-05	164.7760	9.45E+09	105.0027	759309.	0.00
22.2300	-6.05E-04	-13914.	-660.9608	4.48E-05	307.1331	9.45E+09	94.1068	1063499.	0.00
22.8000	-3.34E-04	-16233.	-60.8813	3.39E-05	356.2543	9.45E+09	81.3551	1668477.	0.00
23.3700	-1.42E-04	-14747.	352.4395	2.26E-05	324.7730	9.45E+09	39.4989	1900536.	0.00
23.9400	-2.38E-05	-11412.	510.1491	1.32E-05	254.1580	9.45E+09	6.6150	1900536.	0.00
24.5100	3.80E-05	-7768.	496.6263	6.23E-06	176.9889	9.45E+09	-10.5690	1900536.	0.00
25.0800	6.14E-05	-4618.	402.1125	1.75E-06	110.2906	9.45E+09	-17.0666	1900536.	0.00
25.6500	6.19E-05	-2267.	284.8844	-7.45E-07	60.5008	9.45E+09	-17.2107	1900536.	0.00
26.2200	5.12E-05	-720.9887	177.3367	-1.83E-06	27.7622	9.45E+09	-14.2360	1900536.	0.00

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26.7900	3.70E-05	158.9788	93.5280	-2.03E-06	15.8610	9.45E+09	-10.2695	1900536.	0.00
27.3600	2.35E-05	558.4832	36.1025	-1.77E-06	24.3210	9.45E+09	-6.5216	1900536.	0.00
27.9300	1.27E-05	652.8689	1.6846	-1.33E-06	26.3197	9.45E+09	-3.5421	1900536.	0.00
28.5000	5.26E-06	581.5343	-15.4252	-8.85E-07	24.8091	9.45E+09	-1.4608	1900536.	0.00
29.0700	6.46E-07	441.8563	-21.0347	-5.14E-07	21.8513	9.45E+09	-0.1794	1900536.	0.00
29.6400	-1.78E-06	293.7823	-19.9589	-2.48E-07	18.7156	9.45E+09	0.4940	1900536.	0.00
30.2100	-2.75E-06	168.8193	-15.6591	-8.06E-08	16.0694	9.45E+09	0.7633	1900536.	0.00
30.7800	-2.88E-06	79.5658	-10.3118	9.32E-09	14.1793	9.45E+09	0.8003	1900536.	0.00
31.3500	-2.62E-06	27.7536	-5.0856	4.82E-08	13.0821	9.45E+09	0.7278	1900536.	0.00
31.9200	-2.22E-06	9.9939	-2.1745	6.18E-08	12.7060	9.45E+09	0.1234	379899.	0.00
32.4900	-1.77E-06	-1.9936	-1.4096	6.47E-08	12.5366	9.45E+09	0.1003	386683.	0.00
33.0600	-1.34E-06	-9.2898	-0.8038	6.06E-08	12.6911	9.45E+09	0.07685	393467.	0.00
33.6300	-9.44E-07	-12.9903	-0.3520	5.26E-08	12.7695	9.45E+09	0.05525	400251.	0.00
34.2000	-6.17E-07	-14.1057	-0.03752	4.28E-08	12.7931	9.45E+09	0.03671	407035.	0.00
34.7700	-3.59E-07	-13.5037	0.1624	3.28E-08	12.7804	9.45E+09	0.02173	413819.	0.00
35.3400	-1.69E-07	-11.8849	0.2721	2.36E-08	12.7461	9.45E+09	0.01036	420603.	0.00
35.9100	-3.66E-08	-9.7812	0.3154	1.57E-08	12.7015	9.45E+09	0.00229	427386.	0.00
36.4800	4.68E-08	-7.5703	0.3131	9.46E-09	12.6547	9.45E+09	-0.00297	434170.	0.00
37.0500	9.28E-08	-5.4985	0.2825	4.73E-09	12.6108	9.45E+09	-0.00598	440954.	0.00
37.6200	1.12E-07	-3.7064	0.2370	1.40E-09	12.5729	9.45E+09	-0.00730	447738.	0.00
38.1900	1.12E-07	-2.2559	0.1866	-7.59E-10	12.5422	9.45E+09	-0.00744	454522.	0.00
38.7600	1.01E-07	-1.1533	0.1379	-1.99E-09	12.5188	9.45E+09	-0.00682	461306.	0.00
39.3300	8.46E-08	-0.3698	0.09473	-2.54E-09	12.5022	9.45E+09	-0.00579	468090.	0.00
39.9000	6.63E-08	0.1427	0.05917	-2.63E-09	12.4974	9.45E+09	-0.00461	474874.	0.00
40.4700	4.87E-08	0.4397	0.03169	-2.42E-09	12.5037	9.45E+09	-0.00343	481658.	0.00
41.0400	3.33E-08	0.5762	0.01182	-2.05E-09	12.5066	9.45E+09	-0.00238	488442.	0.00
41.6100	2.07E-08	0.6014	-0.00143	-1.62E-09	12.5071	9.45E+09	-0.00150	495226.	0.00
42.1800	1.11E-08	0.5566	-0.00935	-1.20E-09	12.5062	9.45E+09	-8.15E-04	502009.	0.00
42.7500	4.26E-09	0.4736	-0.01322	-8.30E-10	12.5044	9.45E+09	-3.17E-04	508793.	0.00
43.3200	-2.48E-10	0.3758	-0.01423	-5.22E-10	12.5024	9.45E+09	1.87E-05	515577.	0.00
43.8900	-2.89E-09	0.2788	-0.01342	-2.85E-10	12.5003	9.45E+09	2.21E-04	522361.	0.00
44.4600	-4.15E-09	0.1922	-0.01156	-1.15E-10	12.4985	9.45E+09	3.21E-04	529145.	0.00
45.0300	-4.46E-09	0.1207	-0.00927	-1.68E-12	12.4970	9.45E+09	3.50E-04	535929.	0.00
45.6000	-4.18E-09	0.06545	-0.00694	6.57E-11	12.4958	9.45E+09	3.31E-04	542713.	0.00
46.1700	-3.56E-09	0.02574	-0.00483	9.87E-11	12.4950	9.45E+09	2.86E-04	549497.	0.00
46.7400	-2.83E-09	-5.74E-04	-0.00306	1.08E-10	12.4944	9.45E+09	2.30E-04	556281.	0.00
47.3100	-2.09E-09	-0.01614	-0.00169	1.02E-10	12.4947	9.45E+09	1.72E-04	563065.	0.00
47.8800	-1.43E-09	-0.02366	-6.91E-04	8.73E-11	12.4949	9.45E+09	1.19E-04	569849.	0.00
48.4500	-8.94E-10	-0.02559	-2.44E-05	6.95E-11	12.4949	9.45E+09	7.54E-05	576633.	0.00
49.0200	-4.82E-10	-0.02399	3.74E-04	5.16E-11	12.4949	9.45E+09	4.11E-05	583416.	0.00
49.5900	-1.89E-10	-0.02047	5.71E-04	3.55E-11	12.4948	9.45E+09	1.63E-05	590200.	0.00

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50.1600	3.16E-12	-0.01619	6.25E-04	2.22E-11	12.4947	9.45E+09	-2.76E-07	596984.	0.00	
50.7300	1.15E-10	-0.01192	5.90E-04	1.20E-11	12.4947	9.45E+09	-1.02E-05	603768.	0.00	
51.3000	1.68E-10	-0.00812	5.04E-04	4.80E-12	12.4946	9.45E+09	-1.50E-05	610552.	0.00	
51.8700	1.81E-10	-0.00503	3.97E-04	0.00	12.4945	9.45E+09	-1.63E-05	617336.	0.00	
52.4400	1.68E-10	-0.00269	2.88E-04	-2.76E-12	12.4945	9.45E+09	-1.54E-05	624120.	0.00	
53.0100	1.43E-10	-0.00108	1.91E-04	-4.12E-12	12.4944	9.45E+09	-1.32E-05	630904.	0.00	
53.5800	1.12E-10	-8.62E-05	1.10E-04	-4.55E-12	12.4944	9.45E+09	-1.05E-05	637688.	0.00	
54.1500	8.08E-11	4.20E-04	4.80E-05	-4.42E-12	12.4944	9.45E+09	-7.61E-06	644472.	0.00	
54.7200	5.16E-11	5.71E-04	5.19E-06	-4.07E-12	12.4944	9.45E+09	-4.91E-06	651256.	0.00	
55.2900	2.52E-11	4.91E-04	-1.99E-05	-3.68E-12	12.4944	9.45E+09	-2.42E-06	658039.	0.00	
55.8600	1.21E-11	2.99E-04	-2.86E-05	-3.40E-12	12.4944	9.45E+09	-1.17E-07	664823.	0.00	
56.4300	-2.13E-11	1.00E-04	-2.18E-05	-3.25E-12	12.4944	9.45E+09	2.09E-06	671607.	0.00	
57.0000	-4.33E-11	0.00	0.00	-3.21E-12	12.4944	9.45E+09	4.29E-06	339196.	0.00	

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 0.27160719 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -649208. inch-lbs
 Maximum shear force = 11638. 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 = 7

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

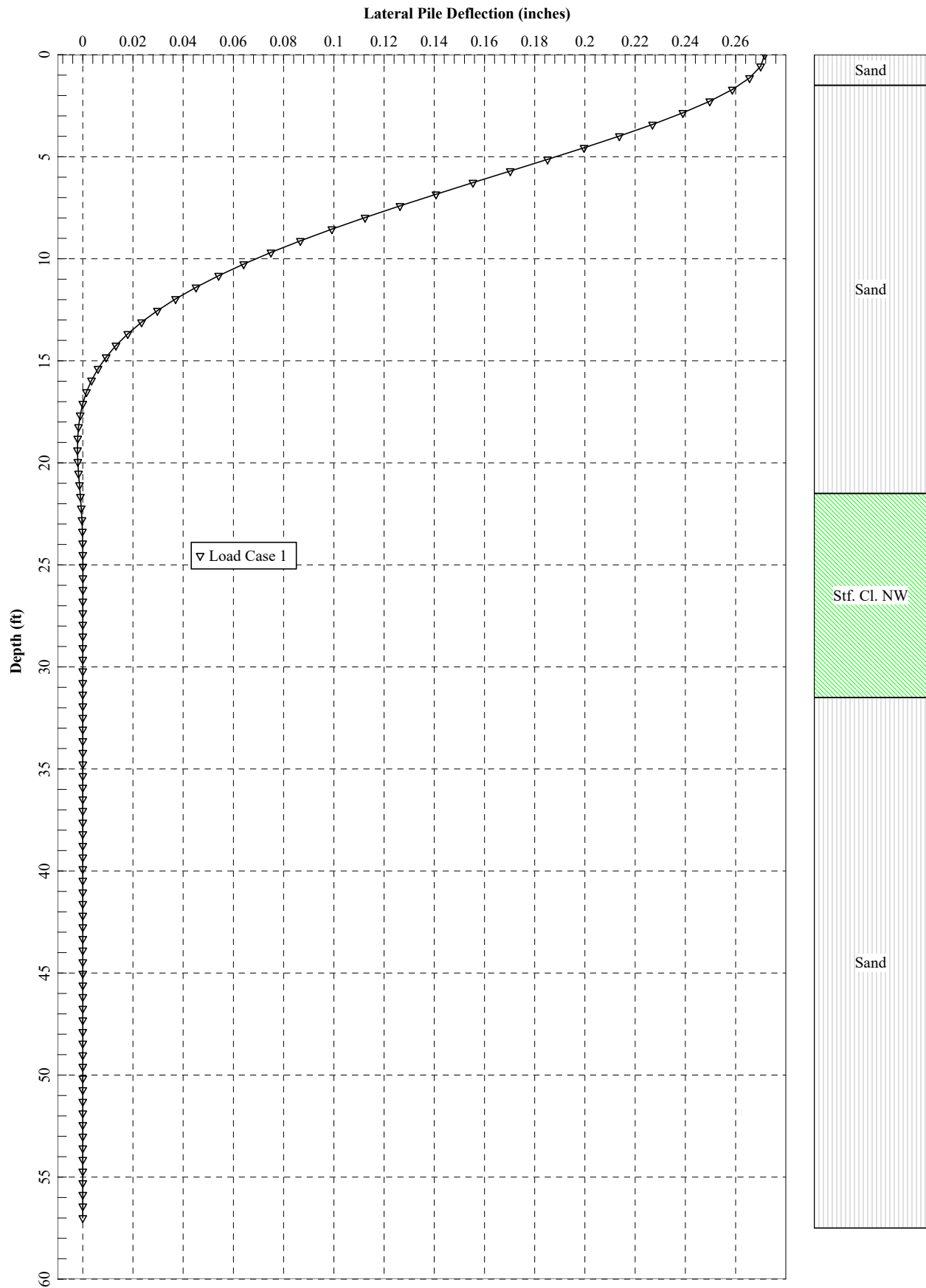
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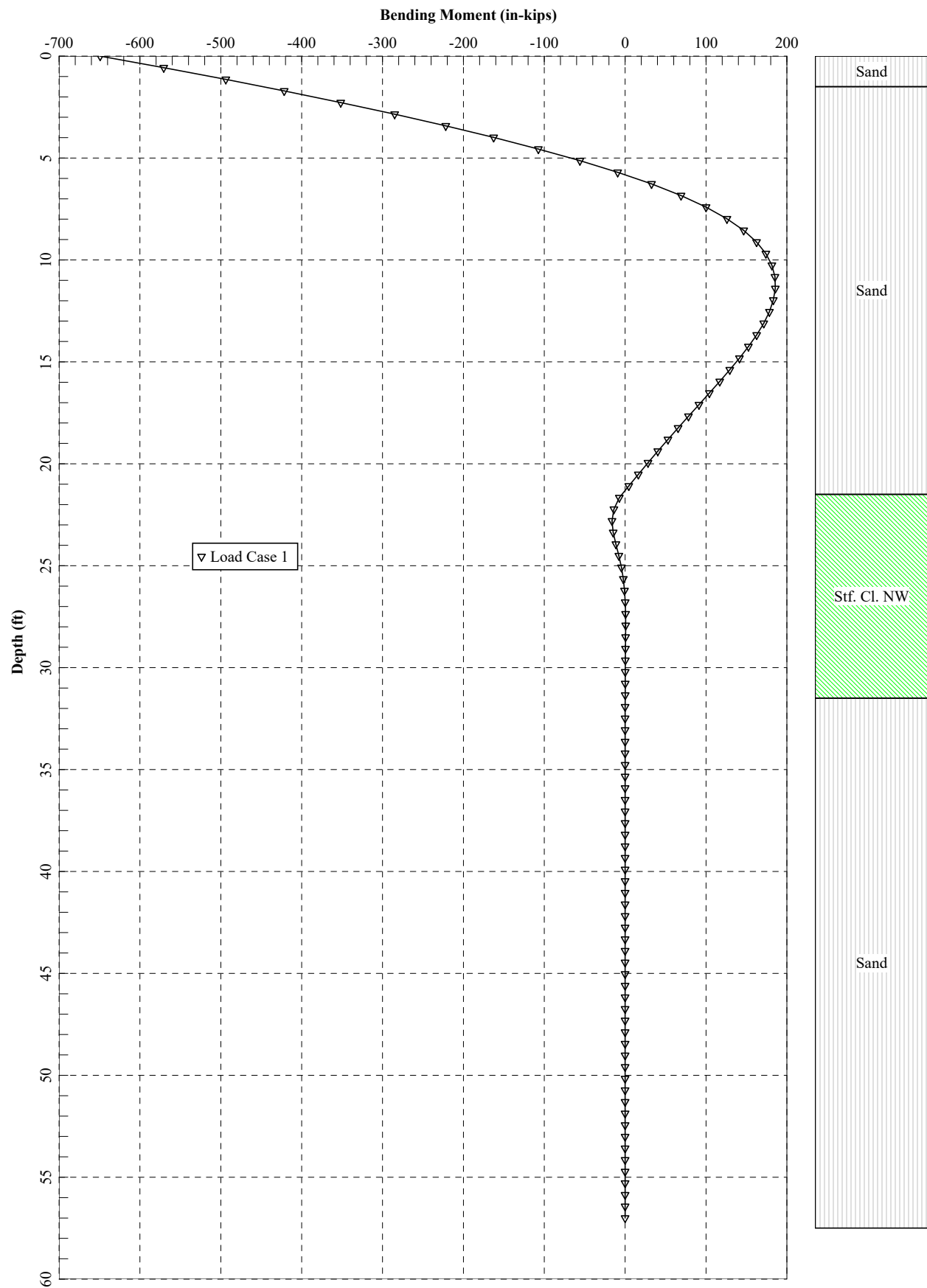
BBFSR0102 14x89.lp9o									
Load Case No.	Load Type	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	0.2700	S, rad	0.00	322.5000	0.2716	0.00	11638.	-649208.

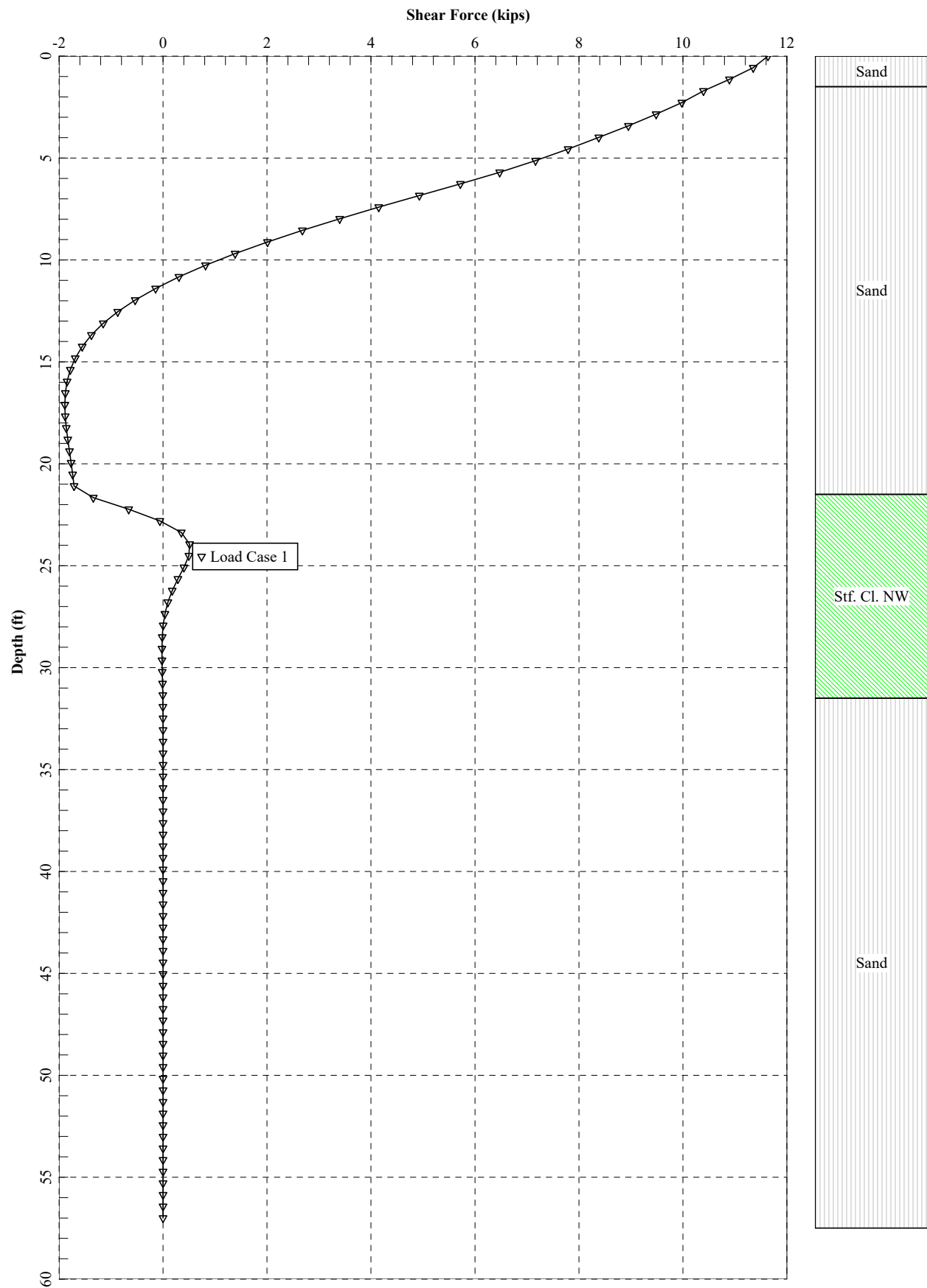
Maximum pile-head deflection = 0.2716071907 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.

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JOB: 09.0025967.00
 SUBJECT: Pile Evaluation for Integral Abutment
 SHEET: 1 OF 9
 CALCULATED BY N. Williams
11/15/2018

Integral Abutment LRFD Pile Design

Subject: Pile Design for the Shortridge Bridge Replacement in Fryeburg, Maine.

- Reference:**
- AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017
 - Maine BDG Chapter 5 - Substructures 2014
 - VTRANS Integral Abutment Bridge Design Guidelines 2008

Design Steps - Maine BDG

Step 1 - Determine the foundation displacements and the load effects (P_u and M_u) from the superstructure and substructure designs.

$P_u := 322.5 \text{ kip}$ Maximum Factored Axial Load

0.27" thermal movement given by Stantec at each abutment.

Step 2 - If applicable, determine the magnitude of scour.

N/A

Step 3 - Select preliminary pile size.

HP14 x 89, Weak Axis Properties

Steel yield strength	$F_{y50} := 50 \text{ ksi}$
Modulus of elasticity for steel	$E := 29000 \text{ ksi}$
Cross sectional area of pile	$A_g := 26.1 \text{ in}^2$
Radius of gyration	$r_y := 3.53 \text{ in}$
Width of Flange	$b_f := 14.7 \text{ in}$
Thickness of Flange	$t_f := 0.615 \text{ in}$
Elastic Section Modulus	$S_y := 44.3 \text{ in}^3$
Plastic Section Modulus	$Z_y := 67.7 \text{ in}^3$

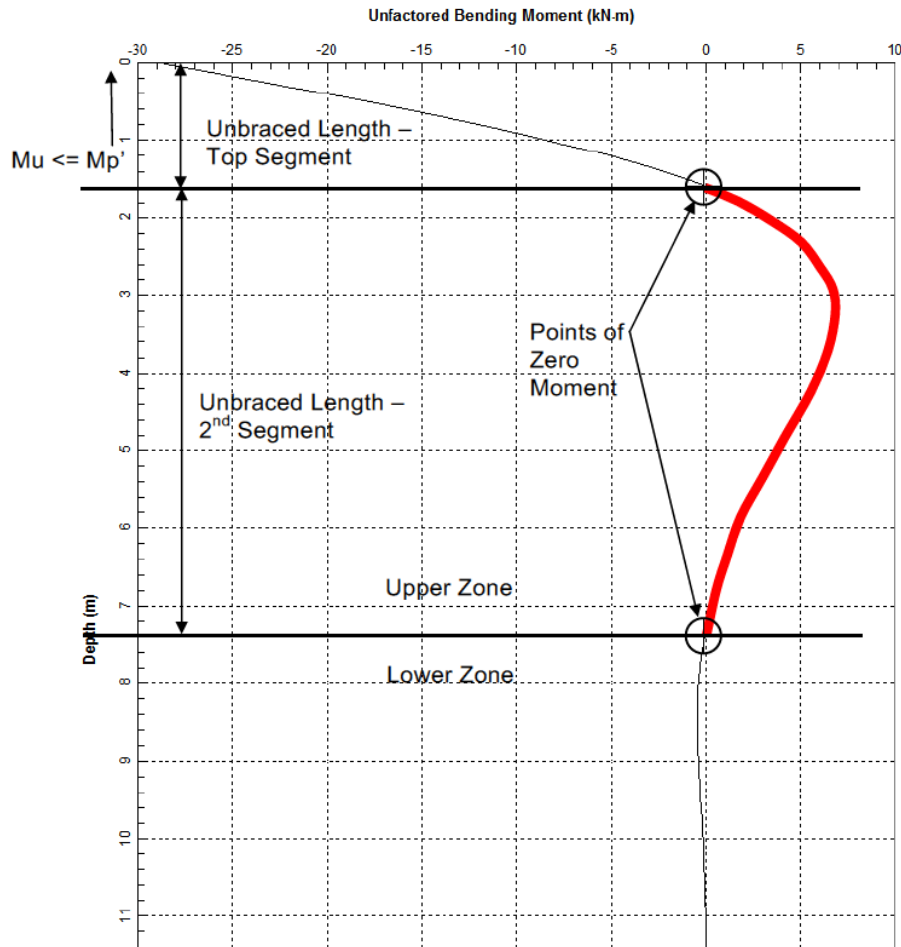


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Step 4 - Determine the pile unbraced length and maximum moment at the top of the pile by running LPile software for
top translation = 0.27 inches, $P_u = 322.5$ kip, and Live Load Rotation = 0 (Fixed against rotation)



Maximum moment from LPile output

$$M_u := 649.208 \text{ kip} \cdot \text{in} = 54.101 \cdot \text{kip} \cdot \text{ft}$$

Unbraced lengths from LPile output

Upper segment

$$L_1 := 6 \cdot \text{ft}$$

Lower segment

$$L_2 := 15 \cdot \text{ft}$$



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Step 5 - Determine if the applied moment on the pile will cause pile head plastic deformation considering the interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

- a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment.

See above for unbraced lengths (critical lengths L_1 and L_2).

Upper segment slenderness factor

$$\lambda_1 := \frac{(L_1)}{r_y} = 20.397$$

Lower segment slenderness factor

$$\lambda_2 := \frac{(L_2)}{r_y} = 50.992$$

- b. Determine K values for top and bottom of the pile per LRFD Table C4.6.2.5-1

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of K when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0
End condition code		Rotation fixed Translation fixed Rotation free Translation fixed Rotation fixed Translation free Rotation free Translation free				

Upper segment K value (Type d)

$$K_1 := 1.2$$

Lower segment K value (Type c)

$$K_2 := 1$$



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c. Calculate the nominal and factored structural pile resistance P_n , per LRFD 6.9.4.1 using the λ values

Elastic critical buckling resistance, P_e , based on flexural buckling

$$\text{Upper } P_e \quad P_{e1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_1 \cdot \lambda_1)^2} = 12469.8 \cdot \text{kip}$$

$$\text{Lower } P_e \quad P_{e2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_2 \cdot \lambda_2)^2} = 2873.0 \cdot \text{kip}$$

$$\text{Nominal yield resistance, } P_o \quad P_o := F_{y50} \cdot A_g = 1305 \cdot \text{kip}$$

Check that the ratio of P_e to P_o is > 0.44

If $P_e/P_o > 0.44$ then use equation 6.9.4.1.1-1

If $P_e/P_o < 0.44$ then use equation 6.9.4.1.1-2

$$P_n = \left[0.658 \left(\frac{P_o}{P_e} \right) \right] P_o \quad (6.9.4.1.1-1)$$

$$P_n = 0.877 P_e \quad (6.9.4.1.1-2)$$

$$\frac{P_{e1}}{P_o} = 9.555 \quad \frac{P_{e2}}{P_o} = 2.202$$

Use eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{n1} := \left[0.658 \left(\frac{P_o}{P_{e1}} \right) \right] P_o = 1249.07 \cdot \text{kip}$$

$$P_{n2} := \left[0.658 \left(\frac{P_o}{P_{e2}} \right) \right] P_o = 1079.06 \cdot \text{kip}$$



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 SUBJECT: Pile Evaluation for Integral Abutment
 SHEET: 5 OF 9
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Factored structural Pile Resistance, $P_r = \phi_c (P_n)$

$$\phi_c := .7$$

$$P_{r1} := \phi_c \cdot P_{n1} = 874.4 \cdot \text{kip}$$

$$P_{r2} := \phi_c \cdot P_{n2} = 755.3 \cdot \text{kip}$$

d. Compare the ratio of P_u , the maximum factored axial load, to P_r , the structural resistance in the specified portion of the pile - the pile size should be such that the ratio is not less than 0.20.

Check for both segments

$$\frac{P_u}{P_{r1}} = 0.369$$

$$\frac{P_u}{P_{r2}} = 0.427$$

e. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)

Check slenderness ratio for flange, limiting slenderness ratio for compact flange, and limiting slenderness ratio for a noncompact flange.

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$$

slenderness ratio for flange eq. 6.12.2.2.1-3

$$\lambda_{pf} := 0.38 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 9.152$$

$$\lambda_{rf} := 0.83 \cdot \left(\frac{E}{F_{y50}} \right)^{.5} = 19.989$$

If $\lambda_{pf} < \lambda_f < \lambda_{rf}$ Use equation 6.12.2.2.1-2 to find the nominal flexural resistance

$$M_n := \left[1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left[\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \left(\frac{E}{F_{y50}} \right)^{.5}} \right] \right] \cdot F_{y50} \cdot Z_y = 256.9 \cdot \text{ft} \cdot \text{kip}$$



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SUBJECT: Pile Evaluation for Integral Abutment
SHEET: 6 OF 9
CALCULATED BY N. Williams
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$\phi_f := 1.0$ for flexural resistance according to Maine BDG

$$M_r := M_n \cdot \phi_f = 256.9 \cdot \text{ft} \cdot \text{kip}$$

$M_u = 54.1 \cdot \text{ft} \cdot \text{kip}$ M_u from LPile > than M_r , exceeding the flexural resistance

$$\frac{P_u}{P_{r1}} + \frac{8}{9} \cdot \frac{M_u}{M_r} = 0.6$$

If less than 1, remains in elastic zone. The pile is not expected to yield at the base of the pile cap.

LATERAL EARTH PRESSURE



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JOB: 09.0025967.00 Shortridge Bridge
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 4
 CALCULATED BY N. Williams 9/21/18
 CHECKED BY A. Blaisdell 9/21/18

Subject: Evaluate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 8th Edition (2017)

Input Parameters:

$\beta := 0\text{deg}$ Angle of backfill to the horizontal

$\theta := 90\text{deg}$ Angle of backface of wall to the horizontal

$\phi := 32\text{deg}$ Effective angle of internal friction (*Granular borrow, Soil Type 4, BDG Table 3-3*)

$\delta_f := 19.5\text{deg}$ Average value, precast concrete against clean sand/silty sand-gravel mixture (*AASHTO LRFD Table 3.11.5.3-1*)

Passive Earth Pressure on Integral Backwall:

Per BDG Section 5.4.2.11, developing full passive pressure requires that ratio of lateral abutment movement (y) to abutment height (H_b) exceeds 0.005. If the calculated rotation is significantly less, Rankine earth pressure may be considered.

$y := 0.27\text{in}$

$H_b := 10\text{ft}$

$\frac{y}{H_b} = 0.0023$ Ratio of lateral movement to abutment height is less than 0.005, use Rankine Pressure

Rankine Passive Earth Pressure Coefficient

$$K_{pr} := \frac{1 + \sin(\phi)}{1 - \sin(\phi)}$$

$$K_{pr} = 3.25$$