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Final Geotechnical Design Report

Tannery Brook Bridge (#3610) Replacement
Norway, Maine
MaineDOT WIN 23116.00

Prepared for:
TY LIN International

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1.0 INTRODUCTION

A multi-phase subsurface exploration program was conducted to evaluate the subsurface conditions at the two-span bridge that crosses the Tannery (Bird) Brook in Norway, Maine. The subsurface information collected was used to evaluate the existing bridge's substructure for possible reuse and to provide recommendations for the design and construction of the bridge substructure replacement.

1.1 Purpose of Report

This geotechnical report will serve to support the design and construction of the proposed bridge rehabilitation work. Data collected during the explorations and laboratory testing programs are provided in this document and were used in conjunction with available published subsurface information and historic site data to develop the geotechnical design recommendations and construction considerations presented herein.

1.2 Site Description

The existing bridge, identified as Bridge No. 3610, carries Main Street over the Tannery (Bird) Brook. The location of the site is presented below in Figure 1 Project Locus Map. The existing bridge was constructed in 1929 and consists of masonry abutments and a concrete center pier supporting a reinforced concrete deck. The bridge is approximately 61 feet wide, 22 feet long, and has a clear span of about 9 feet from the abutment to the center pier. According to an inspection report dated July 25, 2019, the existing bridge carries a daily average traffic of about 11,655 cars and is experiencing advanced deterioration of its deck and superstructure, while the substructure is deemed to be in fair condition with only minor section loss.

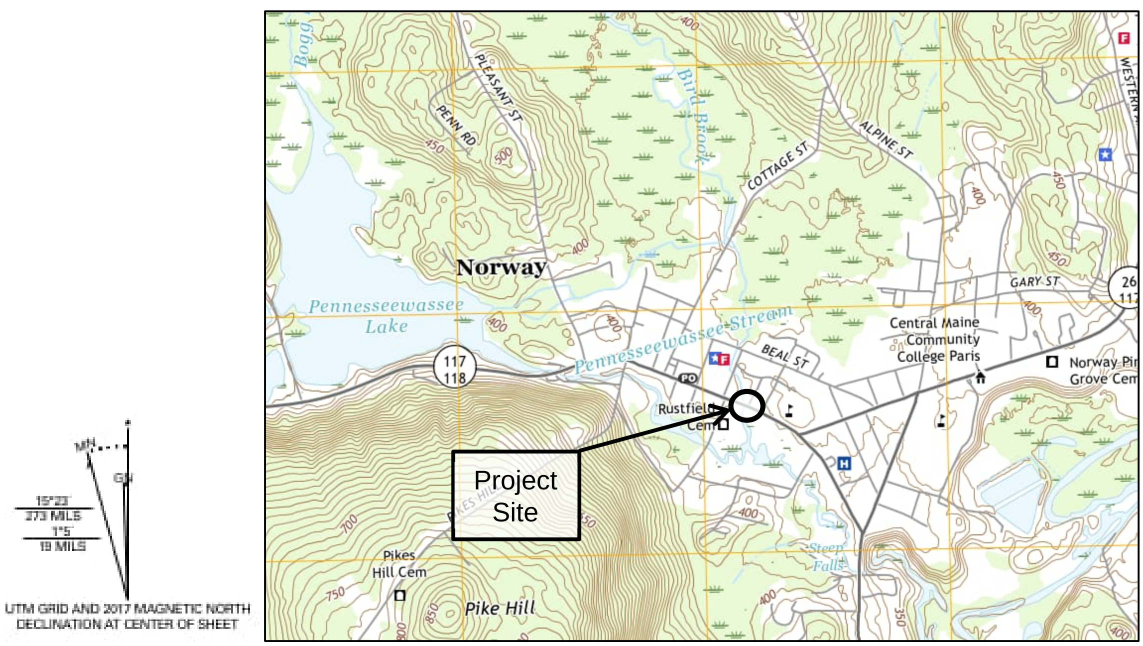

Figure 1. Project Locus Map

1.3 Proposed Construction

Given the deterioration of the existing bridge deck, as noted on the July 2019 inspection report, Maine DOT is evaluating a full replacement of the existing bridge. The proposed replacement bridge will be a single-span structural slab supported by deep foundations installed through the existing stacked granite abutments. The existing center pier will be removed, and the existing stacked granite abutment faces will be stabilized with a reinforced concrete facing. The proposed foundation support for the new bridge is drilled in micropiles or spun pipe piles socketed in bedrock. The new foundation elements are expected to carry primarily axial compression loads since the anticipated lateral loads are minimal.

1.4 Geotechnical Scope

A multi-phase study was conducted at the existing Tannery Brook Bridge. The first study was executed between April 2021 and January 2022. The purpose of this study was to perform probes at the existing Tannery Brook Bridge abutments and to collect data for analysis and reuse determination. AECOM performed the following tasks:

- Reviewed available surficial and regional geology information.
- Marked the boring and probe locations in the field.
- Engaged a drilling subcontractor, New England Boring Contractors of Derry, New Hampshire, to advance the boring and probes at the site.
- Provided a field representative to observe the explorations, collect soil samples, describe the soil samples, and prepare field logs.
- Selected soil samples for geotechnical laboratory testing to determine their classifications and engineering design properties.
- Performed engineering analyses and developed the geotechnical design recommendations; and
- Prepared this Geotechnical Design Report which provides and summarizes the data collected, presents the results of the foundation analyses, and provides the geotechnical recommendations and construction considerations. The Geotechnical Design Report was submitted in January 2022.

Upon review of the initial data collected, a supplemental study was authorized by MaineDOT to collect supplemental soil and rock samples for foundation evaluation. AECOM performed the following tasks from May 2022 to August 2022 as part of the supplemental study:

- Marked the boring and probe locations in the field.
- Engaged a drilling subcontractor, New England Boring Contractors of Derry, New Hampshire, to advance the supplemental borings at the site.
- Provided a field representative to observe the explorations, collect soil and rock samples, describe the soil and rock samples, and prepare field logs.
- Engaged a geophysics subconsultant, NDT Corporation of Sterling, Massachusetts, to perform a geophysical survey at the site.
- Revised the engineering analyses and modified the geotechnical design recommendations, as necessary; and
- Updated the Geotechnical Design Report with revised foundation analyses, recommendations, and construction considerations based on the data collected during the supplemental exploration. The Geotechnical Design Report, Revision 1 was submitted in August 2022.

In September 2022, Phase II of the proposed Tannery Brook Bridge replacement design was initiated, which included evaluation of alternative foundation types and the preparation of this Final Geotechnical Design Report that presents the results of the foundation analyses and provides the geotechnical recommendations and construction considerations for the foundation alternatives.

1.5 Limitations

The data presented herein represent the conditions encountered at the specific locations and at the specific times at which our explorations took place. It should be noted that, given that the subsurface exploration program comprised of relatively small diameter, widely spaced borings and probes, variations in soil and rock stratigraphy and characteristics, and groundwater conditions could appear evident at a later date. If that is the case, it may be necessary to reevaluate the recommendations of this report.

Background information and other data have been furnished to AECOM by third parties, which AECOM has used in preparing this report. AECOM has relied on this information as furnished and is neither responsible for nor has confirmed the accuracy of this information. The geotechnical information presented in this report is based on available information from previous investigations, and the data collected for this project. The geotechnical information presented in this report should not be used for other projects or purposes. Conclusions made from these data by others are their responsibility.

This geotechnical design report has been prepared for specific application to the proposed subject project as presented herein in accordance with generally accepted geotechnical engineering practices. The interpretations and evaluations presented in this report are based in part on information on the proposed work made available prior to submission of the final design documents.

In the event that any changes in the nature, elevations, design, or locations of the proposed structure are made during final design or during construction, the conclusions and recommendations presented herein will no longer be applicable unless the changes are reviewed, and our conclusions and recommendations are modified in writing.

2.0 SUBSURFACE CONDITIONS

2.1 Local Geologic Setting

Generalized descriptions of the surficial and bedrock geology that underlie the site area are provided below.

2.1.1 Surficial Geology

According to the Surficial Geology Map of the Norway Quadrangle (2008), glaciomarine deposits (Pmdo) form the banks of the Tannery Brook in the bridge area (see Figure 2). The glaciomarine deposits are mostly sand and gravel outwash underlain by glaciomarine clay and/or silt.

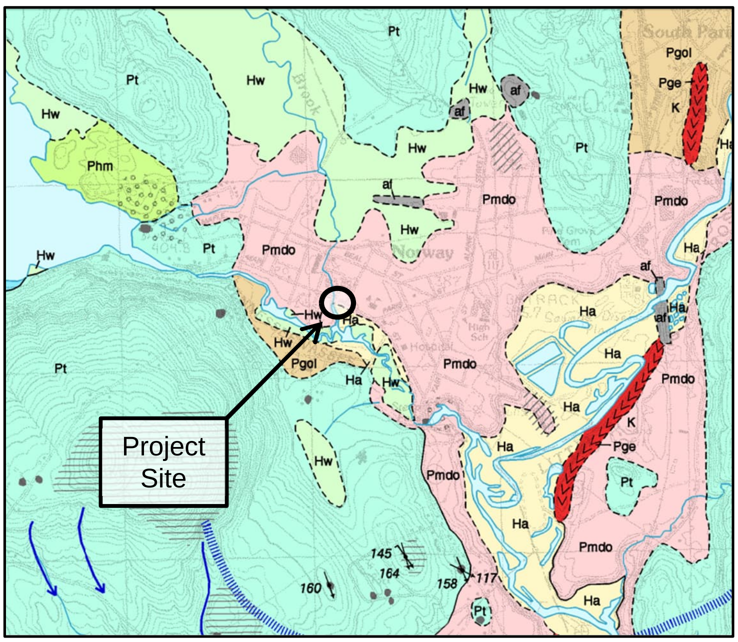

Figure 2. Surficial Geology Near Tannery Brook

The above map also indicates the area to be near stream alluvium deposits (Ha) which consist of sand, gravel, silt, and organic sediment which was deposited on flood plains of modern streams.

2.1.2 Bedrock Geology

According to the Bedrock Geologic Map of Maine (1985), the overburden in the Tannery Brook Bridge area is underlain by carboniferous muscovite-biotite granite. The bedrock geology in the Tannery Brook Bridge vicinity is illustrated in Figure 3.

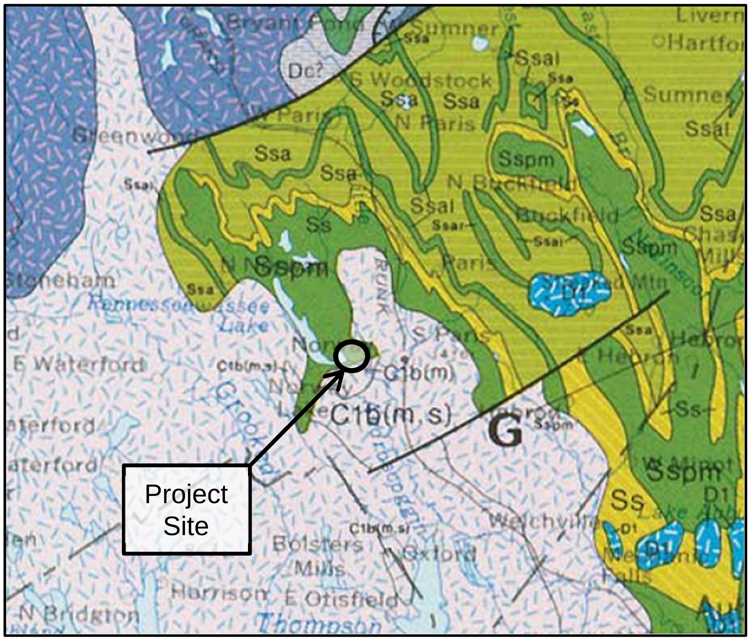

Figure 3. Bedrock Geology Near Tannery Brook

2.2 Subsurface Exploration Program

An exploration program was conducted from April 12 to 14, 2021 at the Tannery Brook Bridge to collect geotechnical data at the bridge. The program included six (6) non-sampling probes and two (2) test borings. The probes (BP-NTB-101 through BP-NTB-106) and test borings (BB-NTB-101 and BB-NTB-102) were drilled above or immediately adjacent to the bridge abutments. The locations of the probes and test borings are shown on the Boring Locations Plan in Appendix A.

The exploration drilling was performed by New England Boring Contractors of Derry, NH. A track-mounted Mobile B-53 drill rig equipped with an automatic hammer (ETR=86%) was used to perform the explorations. Drive and wash techniques were used to advance the borings. The collected soil samples were logged and preserved by an AECOM field representative. The test boring logs from the exploration program are provided in Appendix A and photographs of the boring locations are provided in Appendix D.

2.2.1 Vacuum Excavation

The non-coring probes and test boring locations were vacuum excavated to a depth between 4 and 5 feet below the existing ground surface in order to clear any near surface underground utilities prior to probing or drilling.

2.2.2 Non-Sampling Probes

The non-sampling probe locations are shown on the Boring Locations Plan in Appendix A and in photographs in Appendix D. The first probe of the eastern and western probe lines was located 3 feet away from the joint observed in the pavement, with each subsequent probe located 3 feet further away from the bridge.

Three (3) non-sampling probes (BP-NTB-101, BP-NTB-102, and BP-NTB-103) were performed on the eastern side of the bridge to determine the widths and inclination angles of the abutment batter walls. The eastern non-sampling probes were advanced by continuously driving hollow stem augers and were terminated after the surface of the posterior abutment wall was encountered. The non-sampling probe holes were backfilled with soil cuttings and the road surface was restored with asphalt cold patch.

Three (3) non-sampling probes (BP-NTB-104, BP-NTB-105, and BP-NTB-106) planned on the western side of the bridge each encountered vacuum excavation refusal at a depth of approximately 4 feet. The vacuum refusal occurred on large boulders.

2.2.3 Test Borings

A test boring was drilled on each side of the bridge. BB-NTB-101 was advanced in the northeast quadrant of the bridge and BB-NTB-102 was advanced in the northwest quadrant of the bridge. The test borings were sampled at 5-foot intervals using standard split-spoon samplers. Soil samples were described by the AECOM field representative using the procedures outlined in the Unified Soil Classification System and ASTM D2488 - *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. Representative soils from each collected split spoon sampler were stored in labelled glass jars.

Rock cores, sampled using an NX core barrel, were collected from both borings. Within BB-NTB-101 a total of 9.7 feet of rock was cored and within BB-NTB-102 a total of 10.0 feet of rock was cored. The total depths of borings at BB-NTB-101 and BB-NTB-102 were 26.7 and 24.0 feet, respectively. The test boring holes were backfilled using soil cuttings and the road surface was restored with asphalt cold patch. A photograph of the rock cores is presented in Appendix B.

2.2.4 Laboratory Testing

AECOM selected and submitted soil samples from the subsurface exploration for geotechnical laboratory testing. The soil samples submitted were tested for gradation analyses (ASTM D6913) and for organic content (ASTM D2974) where evidence of organics was observed. The laboratory testing was performed by GeoTesting Express of Acton, MA. The laboratory testing results are presented on the test data sheets in Appendix C.

2.3 Supplemental Subsurface Exploration Program

A supplemental exploration program was conducted on May 18 and 19, 2022 at the Tannery Brook Bridge to collect additional geotechnical data at the bridge abutments. The supplemental program included four (4) test borings (BB-NTB-103, BB-NTB-104, BB-NTB-105, and BB-NTB-106). The test borings were drilled through the existing bridge abutments and terminated within bedrock. The locations of the supplemental test borings are shown on the Boring Locations Plan in Appendix A.

The exploration drilling was performed by New England Boring Contractors of Derry, NH. A track-mounted Acker Soil Scout drill rig equipped with a manual drop hammer was used to perform the explorations. Drive and wash techniques were used to advance the borings. The collected soil samples were logged and preserved by an AECOM field representative. The test boring logs from the supplemental exploration program are provided in Appendix A and photographs of the boring locations are provided in Appendix D.

2.3.1 Test Borings

Four (4) test borings were drilled through the existing abutments, two on each side of the bridge. BB-NTB-104 and BB-NTB-106 were drilled on the west side of the bridge, while BB-NTB-103 and BB-NTB-105 were drilled on the east side of the bridge. Core drilling techniques were used through the concrete footings and stacked stone/boulder abutments. Abutment core samples were collected at each boring. The total depths of borings BB-NTB-103, BB-NTB-104, BB-NTB-105, and BB-NTB-106 were 17.0, 15.2, 19.5, and 17.1 feet, respectively. The test boring holes were backfilled using soil cuttings and the road surface was restored with asphalt cold patch. A photograph of the abutment core samples is presented in Appendix B.

Soil samples were described by the AECOM field representative using the procedures outlined in the Unified Soil Classification System and ASTM D2488 - *Standard Practice for Description and Identification* *of Soils (Visual-Manual Procedure)*. Representative soils from each collected split spoon sampler were stored in labelled glass jars.

2.3.2 Supplemental Laboratory Testing

AECOM selected and submitted rock samples from the supplemental subsurface exploration for uniaxial compressive strength testing with moduli, in accordance with ASTM D7012D. The laboratory testing was performed by GeoTesting Express of Acton, MA. The laboratory testing results are presented in Appendix C.

2.3.3 Geophysical Testing

Geophysical tests were performed by our geophysical subconsultant, NDT Corporation (NDT) of Sterling, Massachusetts. The services performed onsite included ground penetrating radar, seismic refraction, and sonic/ultrasonic reflection measurements at the bridge. The geophysical testing was performed on May 25, 2022. The results of the geophysical testing are described in NDT's report dated June 10, 2022, which is appended to this report.

2.4 Subsurface Conditions

2.4.1 Stratification

The surface and subsurface materials encountered during the subsurface exploration program are described in the following subsections and are shown on the Generalized Subsurface Profile in Appendix A.

Surficial Materials

Between 4 and 8 inches of asphalt were encountered at the ground surface at each of the boring locations. At probe locations, asphalt was observed to range between 8 and 12 inches in thickness.

Existing Fill

The asphalt pavements encountered at the ground surface at borings BB-NTB-101 and BB-NTB-102 were underlain by existing fill soils mainly comprised of silty sand or coarse sand, and poorly graded gravel. The sand in the samples was observed to be fine to coarse grained while the silt was noted as nonplastic. The gravel sample was observed to be fine to coarse grained with few fine to coarse grained sand. Traces of bricks, roots, and cobbles were noted. The existing fill was observed to extend between 4 and 9 feet below the existing ground surface. The only SPT N-Value for the fill was recorded as 73 blows per foot (bpf) within the poorly graded gravel.

Note that boring BB-NTB-102 encountered a 6-inch concrete slab within the existing fill at a depth of 1 foot from the existing ground surface.

Peat

A 5-foot-thick deposit of dark brown peat was encountered beneath the existing fill within boring BB-NTB-101. The peat extended to a depth of 9 feet. The SPT N-Value for the peat was recorded as 2 bpf, indicating a very soft deposit. Peat was not encountered in boring BB-NTB-102.

Glacial Till

A deposit consisting of light grey to grey sand, silt, and gravel, interpreted as glacial till, was encountered beneath the peat in boring BB-NTB-101 and the existing fill in boring BB-NTB-102. The surface of the glacial till was encountered at a depth of 9 feet in both borings. The glacial till samples were classified as silty sand or well-graded sand with silt and gravel. The sand in these samples was observed to be fine to coarse grained. The silt was nonplastic and the gravel, where present, was described as fine grained. The glacial till extended to depths between 14 and 16 feet below the ground surface (bgs).

The SPT N-Values for the glacial till varied between 22 and 135, indicating a medium dense to very dense deposit.

Bedrock

Bedrock was encountered in borings BB-NTB-101 and BB-NTB-102 at depths of 16 and 14 feet bgs, respectively. Two 5-foot rock cores were able to be retrieved from each of the borings. The rock was described as light grey to white, very hard, fresh Granite. The Rock Quality Designation (RQD) values were recorded as 37% and 91% for the cores from boring BB-NTB-101, and as 97% and 67% for the cores from boring BB-NTB-102. Borings BB-NTB-101 and BB-NTB-102 were terminated within the bedrock at depths of 26.7 and 24 feet, respectively.

Abutment Borings

At the abutment borings, BB-NTB-103 to BB-NTB-106, the asphalt pavements encountered at the ground surface were underlain by concrete from the stub abutment footings. The concrete generally extended to a depth ranging from about 4.9 to 5.3 feet bgs. The footings were resting on cobbles and boulders that appeared to extend to bedrock. These borings were advanced through the cobbles and boulders using core drilling techniques. Because the boulders and cobbles were not grouted, core recoveries were erratic and generally low. Due to the shifting of the cobbles and boulders during drilling, a roller bit was used to advance the boring in lieu of coring.

Each boring was advanced into bedrock with the roller bit. The depth to bedrock ranged from approximately 14.6 feet to 18 feet bgs and was inferred based on the rate of advancement of the roller bit and drill cuttings. The depth to bedrock in the abutment borings is consistent with the bedrock depths in borings BB-NTB-101 and BB-NTB-102.

2.4.2 Groundwater

Groundwater was encountered in boring BB-NTB-101 and was recorded at a depth of approximately 8.5 feet bgs at the time of drilling.

It should be noted that groundwater will fluctuate with precipitation, season, the water levels in Tannery Brook, construction activities, run-off controls, and other factors. As a result, water levels during construction may vary from those observed during the subsurface investigation.

3.0 GEOTECHNICAL RECOMMENDATIONS

Geotechnical engineering evaluations have been made on various aspects related to the substructure foundation options for the proposed replacement bridge. In general, these recommendations have been based on the results of subsurface investigations, laboratory testing results, and engineering evaluations in accordance with the requirements of the *AASHTO LRFD Bridge Design Manual, 9th Edition (AASHTO-9)* and the Maine DOT Bridge Design Manual (2003) with updates through 2018.

3.1 Foundation Recommendations

3.1.1 Abutment Geometry & Current Condition

Based on the probing findings, the existing abutments built in 1929 consist of mortared small granite blocks at the face of the abutments, with large boulder and cobble backfill behind them. The supplemental borings through the abutments substantiates the initial assessment that the abutments are constructed of stub footing supported on large boulders and cobbles extending to bedrock.

Based on the pictures included in the inspection report from July 2019 and observations made in the field, the existing abutments seem to be deteriorating as there is evidence of mortar loss and missing blocks from the face of the abutments. The original abutments have been in place for over 90 years.

3.1.2 Foundation Alternatives

Several foundation systems were evaluated as possible options for support of the new substructures, including precast box culverts with wingwalls, rock-socketed H-Piles, driven H-Piles, drilled micropiles, drilled spun pipe piles, and reuse of the existing shallow foundations. The site conditions, construction schedule, and proposed new bridge type limits the number of feasible foundations in consideration to support the replacement bridge.

As noted above, the condition of the existing abutments is poor and major repairs are necessary to meet current bridge design standards and extend the service life of the existing structure. Increased loads due to the elimination of the central pier may exceed the geotechnical capacity of the existing foundations. Therefore, we do not recommend supporting the new structure directly on the existing abutments.

A bridge type alternative consisting of precast concrete culvert with wingwalls is feasible, however, the installation would require large areas to be excavated, groundwater control, and the use of a temporary earth support system, making this foundation type inefficient in schedule and cost.

Driven H-piles are not recommended primarily for constructability concerns. First, the vibrations induced by the driving process might disturb the nearby structures and utilities. Second, overhead utilities will make installation of H-piles problematic. If H-piles are installed in a rock-socket, while feasible, it would be less cost-effective than drilled micropiles and will have low overhead clearance limitations during construction.

Two (2) drilled foundation alternatives were considered to support the new abutments; drilled micropiles and drilled spun pipe piles. Drilled micropiles socketed into bedrock can achieve the required axial and lateral geotechnical resistance to meet the anticipated load demand and can be drilled in place with relatively small equipment. Use of this foundation type would minimize impacts to adjacent utilities and structures while having minimal disruption to traffic.

Spun pipe piles are small diameter, tip bearing foundation elements consisting of a permanent casing bearing within bedrock and backfilled with grout. Spun pipe piles are designed to carry primarily axial compression loads and therefore do not include any central reinforcement. These piles are designed to provide tip resistance within bedrock and side resistance along the permanent casing is neglected. The permanent casing is usually socketed into intact bedrock. Spun pipe piles are typically installed in less time than traditional micropiles because a second drilling sequence is unnecessary due to the elimination of the internal bar. Spun pipe piles require less material while achieving the required geotechnical resistances.

As such, we recommend a new structure span over the existing bridge and be supported on drilled spun pipe piles. To maintain the stability of the abutments, we recommend the granite block and mortar abutment faces are reinforced with a new concrete fascia that are bolted to the existing granite abutment faces. The sections below provide recommendations for the preferred foundation option.

3.1.3 Drilled Spun Pipe Piles

Drilled spun pipe pile foundations may be installed through the existing stacked granite abutments and support the bridge foundation loads by means of end bearing in rock. The spun pipe pile geotechnical resistance may be computed using AASHTO-9 LRFD design guidance for drilled shaft end bearing resistance in jointed rock. The factored static geotechnical axial compression resistance should be computed using a resistance factor, φ_{stat} of 0.5, corresponding to micropile tip resistance on rock with no load test, from AASHTO-9 Table 10.5.5.2.5-1.

The factored geotechnical axial compressive resistance of spun pipe piles, R_R, may be calculated from the following formula from Article 10.8.3.5 of AASHTO-9, modified to neglect side resistance:

$$R_R = \varphi R_n = \varphi_{qp} R_p = \varphi_{qp} q_p A_p$$

where :

R_p = nominal pile tip resistance (kips)
q_p = unit tip resistance of pile (ksf)
φ_{qp} = resistance factor
A_p = area of pile tip (ft^2)

The nominal tip resistance was estimated using procedures presented in Article 10.8.3.5.4c of AASHTO-9 for jointed rock. The drilled shaft tip resistance on rock is governed by the frequency and condition of joints in the bedrock. When the rock is jointed the tip resistance is computed using the geological strength index (GSI) and Hoek-Brown failure criterion. Based on the condition of the rock cores collected at the Tannery Brook Bridge, the rock is considered jointed and the GSI was determined by an AECOM Geologist. The estimated GSI ranged from 30 to 82, so an average value of 50 was used in the rock strength evaluation.

The rock mass strength was estimated using the GSI in combination with the rock type (granite) and the uniaxial compressive strength determined from laboratory testing. The laboratory uniaxial compressive strength ranged from 14,719 psi to 19,098 psi. The lower uniaxial compressive strength value was used in the static axial compression analysis.

The factored tip resistance on jointed rock was computed for a range of spun pipe pile sizes. The side resistance of the spun pipe pile is neglected in the axial resistance because the socket installation utilizes a ring bit that is larger than the outside diameter of the casing and no grout is placed between the casing and wall of the socket. The tip resistance was computed for the shortest spun pipe pile anticipated at the site, which corresponds to the west abutment. Table 1 presents the estimated spun pipe pile axial compression resistance in jointed rock for various pile sizes.

Table 1: Estimated Spun Pipe Pile Axial Resistance – Jointed Rock

Tip Diameter (inch)	GSI	Hoek Brown Parameter, m_i	Estimated UCS, Intact Rock (psi)	Nominal Tip Resistance (kips)	Factored Tip Resistance (kips)
8.625	50	30	14,719	643	321
9.625	50	30	14,719	801	400
10.75	50	30	14,719	1000	500

Lateral foundation analyses were completed using the LPILE software by Ensoft, Inc., in conjunction with factored single pile axial compression load of 230 kips, provided by the Project Structural Engineer, and subsurface information obtained from the borings. The anticipated lateral loads on the proposed spun pipe piles are minimal, so the lateral response evaluation was executed to verify that the pile stress stays within the elastic range under conventional lateral loads. The spun pipe pile was modeled as a grout-filled steel casing socketed 5 feet into rock and subjected to 0.5-inch pile top displacement and 230 kips factored axial load. The diameter of the pile was 9.625-inch with a wall thickness of 0.545-inch. Table 2 provides the lateral response of 9.625-inch spun pipe piles subject to the above-mentioned loads.

Table 2: 9.625-inch Spun Pipe Pile Lateral Response

Pile Type	Pile-Head Deflection (in)	Depth to Bedrock (ft)	Max. Shear Force (kips)	Max. Moment (in-kips)	Total Stress in Pile (ksi)
9.625" x 0.545" wall	0.5	14.0	22.7	642.8	38.7
9.625" x 0.545" wall	1.0	14.0	53.2	1182.4	57.1

The casing was assumed to be N80 grade with a yield strength of 80 ksi. The grout was assumed to have a yield strength of 4 ksi. The lateral response analysis demonstrates that the stress in the spun pipe pile casing will be less than 90% of the casing yield strength. Fixity in the piles occurs approximately 2 feet below the bedrock interface.

The bending moment capacity at the threaded casing joint was evaluated based on guidance provided in the Federal Highway Administration reference manual *NHI-05-039 Micropile Design and Construction*, dated December 2005. The maximum bending moment capacity at a joint location was computed and compared to the estimated maximum bending moment at the approximate depth of threaded joint developed in the lateral response analysis. For threaded joint located at 10 feet (typical) below the top of pile the joint bending moment capacity is exceeded when the pile head deflection is greater than 0.5-inch. If higher bending resistance is required, a larger casing section may be specified, or a smaller pipe may be placed inside the casing and extended below the location of the moment at the joint.

3.1.4 Drilled Micropiles

A micropile is a steel casing advanced into bedrock, then cleaned out, and a secondary drilling process takes place to install an internal bar into bedrock in a smaller diameter hole. The micropile transfers axial load through the casing plunge length to the grout-to-ground or grout-to-bedrock bond resistance.

Micropiles are a viable alternative to spun pipe piles at the Tannery Brook Bridge. Micropiles would derive their resistance from side friction in the bedrock, as the tip resistance is typically ignored due to the micropile's small diameter. A minimum rock socket length of 7 feet below the casing is recommended, which would result in pile tips approximately 20 to 25 feet bgs.

The micropile factored axial resistance is governed by either the pile structural resistance or the geotechnical resistance of the soil and bedrock supporting the pile.

The factored geotechnical axial compressive resistance of embedded micropiles, R_R, may be calculated from the following formula from Article 10.9.3.5 of AASHTO-9:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s = \varphi_{qs} q_s A_s + \varphi_{qp} q_p A_p$$

where:

R_p = nominal pile tip resistance (kips)
R_s = nominal grout-to-ground/rock bond resistance (kips)
q_s = unit grout-to-ground/rock bond resistance (ksf)
q_p = unit tip resistance of pile (ksf)
φ = resistance factor
A_p = area of micropile tip (ft^2)
A_s = area of the grout-to-ground bond surface (ft^2)

Micropiles are frequently designed utilizing either side resistance or end bearing. In our experience, mobilizing side resistance requires axial deformations on the order of 0.25 inches or less to mobilize full side resistance. Design using side resistance also provides more flexibility in the field as rock sockets are easily lengthened if additional resistance is required. End bearing resistance requires greater deformations to mobilize resistance, often on the order of an inch of movement or more. End bearing micropiles also require careful construction to properly clean the micropile socket of soil, enabling good contact between the cement grout and the bedrock surface. Due to the differences in strain compatibility, we do not recommend combining side resistance and end bearing resistance in drilled micropiles.

According to AASHTO Table 10.5.5.2.5-1, the factor for side resistance in rock is 0.55, the factor for uplift resistance is 0.55, and the factor for axial end bearing resistance is 0.5. Each load factor may be increased to 0.7 if a verification load test is performed. An axial analysis was performed based on the information from the borings.

Our analyses of micropiles designed for side resistance uses a grout to ground bond resistance value of 200 psi within the rock socket. The factored single pile axial compression load is 230 kips. Assuming an approximate 8.5-inch diameter rock socket, 7 feet long, results in an axial factored compressive resistance of approximately 247 kips, and a factored uplift resistance of 247 kips. We recommend a verification test is performed to confirm the design capacity of the micropile. Using the increased resistance factor of 0.7 for load testing, the compression resistance is approximately 269 kips and the required socket length is 6 feet. Additional load test considerations are provided in Section 4 of this report. Rock socketed micropiles should incorporate a minimum plunge length of two feet into rock. Side resistance within the plunge length is neglected.

Our analyses of micropiles designed for end bearing assumes the bearing value is limited due to the strength of the cement grout, typically 5,000 psi, due to the quality of the bedrock encountered at the site. Assuming an approximate 8.5-inch diameter bearing surface results in an axial factored compressive resistance of approximately 355 kips. With an increased resistance factor of 0.7 for load testing, the compression resistance increases to 497 kips.

Piles supporting the bridge substructures will be required to support lateral loads and overturning moments resulting from design loads. Preliminary lateral foundation analyses were completed for a 9.625-inch diameter micropile using the LPILE software by Ensoft, Inc., in conjunction with preliminary axial compression loads and subsurface information obtained from the borings. The lateral response analysis indicates that micropiles can achieve a lateral load resistance of 9.0 and 15.5 kips, corresponding to displacement values of 0.5 inches and 1 inch at the pile head.

The bending moment capacity at the threaded casing joint was evaluated based on guidance provided in the Federal Highway Administration reference manual *NHI-05-039 Micropile Design and Construction*, dated December 2005. The maximum bending moment capacity at a joint location was computed and compared to the estimated maximum bending moment at the approximate depth of threaded joint developed in the lateral response analysis. For threaded joint located at 10 feet (typical) below the top of pile the joint bending moment capacity is exceeded when the pile head deflection is greater than 0.5-inch.

If higher bending resistance is required, a larger casing section may be specified, or a smaller pipe may be placed inside the casing and extended below the location of the moment at the joint.

Note that the cast-in-place wall positioned in front of the existing abutments is also proposed along with the micropile option. The CIP wall would reduce the lateral load demand on the proposed micropiles.

3.2 Lateral Earth Pressures

Abutments will be subject to soil lateral pressures from earth, line loads, traffic loads, seismic loads and other loads, and hydrostatic pressures. The design lateral pressures should be calculated by adding earth and water pressures, and surcharge pressures from structures near the proposed wall. Walls braced at the top such that lateral deflections are not permitted or restricted, should be designed for an at-rest earth pressure. Free standing walls not braced at the top may be designed for an active earth pressure, provided lateral movements at the top of the wall are acceptable throughout the life of each wall.

Lateral earth pressure design parameters presented in the following table assume horizontal grades in front of and behind the walls:

Table 3: Retaining Wall Design Parameters

Material	Total Unit Weight (pcf)	Friction Angle (degrees)	K_o, At-Rest Earth Pressure Coefficient	K_a, Active Earth Pressure Coefficient	K_p, Passive Earth Pressure Coefficient
Granular Borrow or Granular Underwater Backfill	125	32	0.47	0.31	3.25

3.3 Seismic Design Criteria

In accordance with AASHTO-9, Section 3.10, the seismic criteria are as follows:

- Site Class: D
- Peak Ground Acceleration (PGA): 0.09 g
- Spectral Response Acceleration at short period (Ss): 0.19 g
- Spectral Response Acceleration at 1 sec. (S1): 0.048 g
- Site Coefficient Fpga (Table 3.10.3.2-1): 1.6
- Site Coefficient Fa (Table 3.10.3.2-2): 1.6
- Site Coefficient Fv (Table 3.10.3.2-3): 2.4
- Adjusted PGA, As 0.144
- Adjusted Spectral Response SDS: 0.304 g
- Adjusted Spectral Response SD1: 0.115 g

Based on the boring information, the site soils are not susceptible to liquefaction. The site classifies as Seismic Zone 1 with an $S_{D1} \leq 0.15$g.

3.4 Seismic Pressure

In accordance with AASHTO LRFD Bridge Design Manual, Table 3.10.6-1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.15 will be assigned to Seismic Zone 1. According to Section 3.10.9.1, a seismic analysis is not required for single-span bridges and bridge located in Seismic Zone 1.

4.0 CONSTRUCTION CONSIDERATIONS

The purpose of this section is to discuss geotechnical related construction issues for the planned bridge replacement.

4.1 Foundation Installation

The spun pipe piles may be installed using similar equipment and procedures as those used for micropile construction. The Contractor may be required to use a drilling technique that can penetrate obstructions. Based on the results of the geotechnical exploration, the granular deposits encountered on site will become unstable in unsupported drill holes. Therefore, the use of steel casings during drilling to control ground losses will be required.

The Contractor should submit their proposed installation procedures, including construction procedures and sequencing, material specifications, equipment specifications, quality control documentation method and criteria, and safety monitoring procedures, for approval by the Geotechnical Engineer of record a minimum of 10 business days prior to the start of construction. The rock bearing surface should be verified by an experienced geologist prior to grouting and the entirety of the pile installation should be inspected by a qualified geotechnical engineer or geologist.

The grout level should be maintained at the top of the element and monitored to ensure that the grout flow is unobstructed and completely fills the drill hole. Piles should be grouted as soon as possible and on the same day after drilling is completed. Subsequent piles should not be drilled near elements that have been grouted until the grout has had sufficient time to harden.

At a minimum, the following information should be recorded during installation:

- Measurements of pile lengths (including top and tip elevations of constructed piles), casing lengths, and diameters.
- Measurement of grout volumes.
- Depth of bearing layer surface, and embedment into the bearing layer.

During pile construction, all pile installation records should be provided to the Geotechnical Engineer of record on a daily basis.

If micropiles are selected and installed as the preferred foundation type, center bar reinforcing steel should be inserted in the open drill hole prior to grouting. If the micropile holes are over drilled, the annulus between the design pile diameter and the drill hole diameter must be filled with grout placed during the installation of the micropile, in order to ensure that full contact is established between the grout and surrounding ground.

Should the final design utilize micropiles deriving capacity through side resistance, an uplift load could be performed. This method is advantageous as the pile test load is developed by jacking the pile against the ground.

4.2 Utility Replacement

We understand that Norway Water is considering replacement of the existing 10-inch diameter, cast iron water line that is buried beneath Tannery Brook with a larger 12-inch diameter water line. In our opinion, this water line could be replaced safely and with less disruption to the bridge and roadway if pipe bursting is used. This method opens the existing pipe and forces it outward using a bursting tool, which is pulled through the existing pipe while pulling the replacement pipe behind it. This method can pull in a variety of replacement pipe types including ductile iron pipe, concrete pipe, and HDPE pipe. Ideally, the water line would be replaced prior to installation of the foundations. If the replacement pipe is installed after foundation installations are completed, we recommend that a clear space of at least 18 inches be maintained between the foundations and the water line.

The existing gravity sewer may also be replaced by Norway Wastewater. The existing 14-inch diameter asbestos sewer is concrete encased and passes through the stream channel. Due to the proximity of the sewer to the center line of the road and the relatively shallow depth of the sewer, an open cut would be required to replace the existing pipe. An open cut may require the partial removal of the existing granite block abutment, the diversion of Tannery Brook to allow construction in the dry, and installation of shoring along the roadway and possibly at the abutment to limit the excavation size and impact on adjacent utilities and structures. Due to the vibration associated with any shoring installation and partial removal of the abutment walls there is risk that the mortar between blocks left in place could crack and de-bond from the blocks, leaving the walls in a weakened state.

If the sewer is replaced in an open cut, we recommend the following:

- Pre-construction and post-construction survey of each bridge abutment and all structures within 250 feet of construction.
- Vibration monitoring during construction using seismographs.
- Installation of deformation monitoring points on the existing abutments and the adjacent structures.

One alternative for the replacement sewer could be to install the replacement sewer in an open cut at the downstream end of the bridge. This will reduce the disruption to the roadway and eliminate the need to demolish the existing abutment walls in the vicinity of the existing sewer main. To avoid issues with fish passage, an inverted siphon design would be required to allow passage of the sewer below Tannery Brook. We recommend this alternative be considered along with other site constraints to determine feasibility.

4.3 Subgrade Preparation

Loose or soft soils identified during the compaction of the subgrade should be excavated to a suitable bearing stratum as determined by the field representative of AECOM. Grades should be restored by backfilling with Granular Borrow, Gravel Borrow, or crushed stone.

To reduce the potential of increasing lateral pressures on either existing abutment walls to remain in place or new abutment walls, fill placed within 3 feet of walls should be compacted using a small plate compactor imparting a maximum dynamic effort of 4 kips. The fill within 3 feet of the abutment wall should be placed in maximum 8-inch loose lifts.

When crushed stone is required in the drawings or it is used for the convenience of the contractor, it should be wrapped in a geotextile fabric for separation except where introduction of the geotextile promotes sliding.

4.4 Subgrade Protection

The onsite soils are anticipated to be frost susceptible. If construction takes place during freezing weather, special measures such as heat blankets or other measures should be taken to prevent the subgrade from freezing. Excavations should be backfilled as soon as possible after construction. Soil used as backfill should be free of frozen material, as should be the ground on which it is placed. Fill placement should be halted during freezing weather.

4.5 Water Control

Temporary excavations that extend below El. 367.5 will require dewatering so that the work is conducted in the dry. It is expected that dewatering may be accomplished by well points or with local filtered sump pumps installed in low points of the excavation. Groundwater levels should be maintained at a minimum of 2 feet below the bottom of excavations during construction. To reduce the potential for sinkholes developing over sump pump pits after the sump pumps are removed, the crushed stone placed in the sump pump pits should be wrapped in a geotextile fabric. Alternatively, the crushed stone should be entirely removed after the sump pump is no longer in use and the sump pump pit should be restored with suitable backfill.

Dewatering systems for bridge construction should be designed by the Contractor's professional engineer registered in the State of Maine and submitted to the geotechnical engineer of record for review at least two weeks prior to the start of construction. Discharge water must be managed in accordance with local, state, and federal government requirements.

4.6 Soil Excavations

The Contractor will be responsible for the excavation in accordance with the applicable federal and state laws and regulations, including OSHA.

The site soils should generally be considered Type "C" in accordance with OSHA and should have a maximum allowable slope of 1.5 horizontal:1 vertical for excavations less than 20 feet deep. In areas where the horizontal distance between the edge of any excavation and the bottom of existing structures is less than twice the vertical distance between the bottom of the excavation and the bottom of the existing structure, excavation support systems will be required to prevent the undermining of existing structures. Major utilities may also be protected with temporary sheeting as determined by the contractor on a case-by-case basis. Any excavation support system, if required, should be designed by the Contractor's professional engineer registered in the State of Maine.

4.7 Temporary Earth Support

Construction of a new bridge may require a temporary earth support system. The contractor should select the excavation support type to fit the construction sequence, including the support of any utilities crossing the bridge. The temporary earth support system should be designed by a professional engineer registered in the state of Maine and engaged by the Contractor. The design should be submitted to the Geotechnical Engineer of record for review at least two weeks prior to the start of construction.

4.8 Protection of Existing Utilities

Existing utilities may be encountered in the vicinity of the work. There are two existing utilities, a gravity sewer and a waterline, that must remain in place in their current locations at the bottom of Tannery Brook. Proper planning and protection measures should be implemented to protect the existing utilities and minimize impacts accordingly.

4.9 Construction Monitoring

It is recommended that AECOM be retained to provide geotechnical engineering observation and consultation services during construction to observe compliance with design and construction recommendations and specifications. The field representative would undertake the following responsibilities:

- Verify foundation installation procedures.
- Observe and document foundation installation.
- Monitor all excavation activities.
- Monitor all SOE operations, if required.
- Monitor all dewatering operations, if any.
- Provide recommendations regarding re-use of on-site soils.
- Observe and document the placement and compaction of fill materials.

Additionally, the field representative would be present to verify and provide timely responses to the project team in the event that the actual conditions encountered differ from those described herein.

5.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), “LRFD Bridge Design Specifications, Ninth Edition”, AASHTO, Washington, DC, 2020.

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ASTM D2974-20e1, Standard Test Methods for Determining the Water (Moisture) Content, Ash Content, and Organic Material of Peat and Other Organic Soils, ASTM International, West Conshohocken, PA, 2020, www.astm.org.

Cardali et al. 2018, “Innovative Socketed Pile for Accelerated Bridge Construction in Naples, Maine”, prepared for the 69th Highway Geology Symposium, September 2018.

Department of Transportation, State of Maine, “Bridge Design Guide”, 2003, Amended 2018.

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Federal Highway Administration, “Publication No. FHWA NHI-05-039 Micropile Design and Construction”, December 2005.

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Prepared by:

STATE OF MAINE
TODD DOUGLAS DWYER
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09/15/23

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AECOM

Appendix A

Boring Locations Plan, Generalized Subsurface Profile,

Test Boring Logs and Probe Logs

BORING PLAN

10 0 10 20

Scale of Feet

KEY:

BB = Bridge Boring

BP = Bridge Probe

NOTES:

1. Borings BB-NTB-101, 102 and probes BP-NTB-101 through 106 performed by New England Boring Contractors between April 12 and April 14, 2022.
2. Supplemental explorations consisting of boring BB-NTB-103 through BB-NTB-106 performed by New England Boring Contractors between May 18 and May 19, 2022.

LEGEND:

Cased Wash Boring

Solid Stem Auger

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2311600

WIN
023116.00

BRIDGE PLANS

BRIDGE NO. 3610

SIGNATURE
P.E. NUMBER 13187
DATE 10/27/2023

		BY	DATE
PROJ. MANAGER	JULIE BRASK		
DESIGN-DETAILED	B. Reyes	S. Morgan	8/2022
CHECKED-REVIEWED	T. Dwyer	B. Reyes	8/2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

TANNERY BROOK BRIDGE
OVER TANNERY BROOK
NORWAY OXFORD COUNTY

BORING LOCATION PLAN

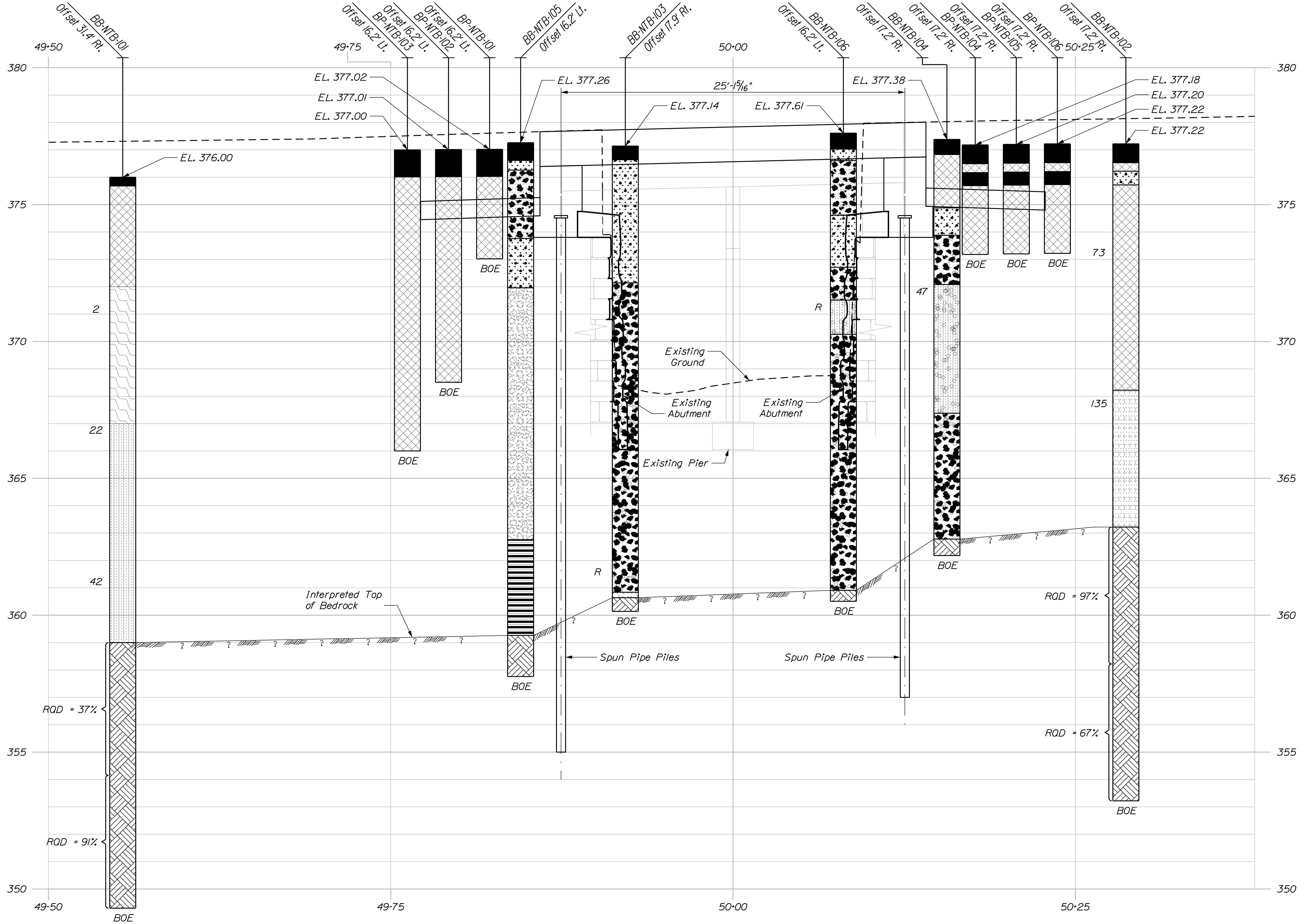
SHEET NUMBER

5

OF 31

TYLin

Filename: ...\xxx_Boring_Location_Plan.dgn Division: HIGHWAY Username: Date:10/27/2023



Legend:

- Boring No.
- Offset, if shown
- Pavement Thickness if applicable
- 6 — Uncorrected SPT N Value (blows/foot) for specified number of inches for penetration between 6" and 18"
- WOH — Indicates weight of hammer
- R — Split Spoon Refusal (> 50 blows for 1" penetration)
- Advancement of core barrel through apparent boulder or till
- Interpreted Top of Bedrock
- RQD — Rock Quality Designation for Rock Core Sample
- BOE — Bottom of Exploration

STRATA SYMBOLS

- Paving
- FILL
- Low Plasticity Organic Silts
- Silty Sand
- Bedrock
- Concrete
- Well graded sand with silt
- Granite and boulders
- Poorly graded sand
- Variable sand and silt mix
- Well graded gravel and sand
- Boulders and cobbles with sand
- Weathered Bedrock

PROFILE - U.S. ROUTE 2

Horiz. 0 2 4 6 8 10
Vert. 0 1 2 3 4 5
Scale of Feet

NOTE:

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

TYLin

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2311600

WIN
023116.00

BRIDGE NO. 3610

STATE OF MAINE
TODD DOUGLAS DWYER
No. 13817
LICENSED PROFESSIONAL ENGINEER

SIGNATURE
P.E. NUMBER 13817
DATE 10/27/2023

	BY	DATE
PROJ. MANAGER	JULIE BRASK	
DESIGN-DETAILED	B. Reyes	S. Morgan	8/2022
CHECKED-REVIEWED	T. Dwyer	B. Reyes	8/2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

TANNERY BROOK BRIDGE
OVER TANNERY BROOK
NORWAY OXFORD COUNTY

GENERALIZED SOIL PROFILE

Table 2: Probe and Boring Depths and Locations

Probe / Boring No.	Distance from Bridge (ft)	Probe / Boring Depth (ft)
BB-NTB-101	45	26.7
BP-NTB-101	3	4
BP-NTB-102	6	8.5
BP-NTB-103	9	11
BB-NTB-102	12	24
BP-NTB-104	3	4
BP-NTB-105	6	4
BP-NTB-106	9	4
BB-NTB-103	On Bridge Footing	17
BB-NTB-104	On Bridge Footing	15.2
BB-NTB-105	On Bridge Footing	19.5
BB-NTB-106	On Bridge Footing	17.1

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BB-NTB-101

WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	376.00	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	2" SPT
Logged By:	J. Loomis	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140 lbs./ 30 in.
Date Start/Finish:	04/13/21	**Drilling Method:**	Wash and Drive	**Core Barrel:**	2.15" (NX)
Boring Location:	N 502678.3573 E 941696.8687	**Casing ID/OD:**	HW 4" ID 4.5" OD	**Water Level*:**	8.5 feet
Hammer Efficiency Factor: 0.86		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

T_v = Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WD	375.7		4 inches of Asphalt. (0.3)	
										Brown, dry, fine to coarse Silty SAND, some non-plastic fines, some cobbles (FILL).	
								372.0		(4.0) Dark grey to brown, moist, Organic SILT, non-plastic.	
5	1D	24/16	5.00 - 7.00	1/ 1/ 1/ 1	2	3				Dark brown, moist, soft PEAT.	Organic Content = 95%. Gravel= 15.4%, Sand= 83.2% , Fines= 1.4%
	2D	24/10	9.00 - 11.00	11/ 9/ 13/ 30	22	32		367.0		(9.0) Light grey, wet, dense, fine to coarse Silty SAND, some non-plastic fines.	
10											
	3D	24/13	14.00 - 16.00	11/ 18/ 24/ 30	42	60				Similar to 2D, except very dense.	
15											
	R1	60/50	17.00 - 22.00	RQD = 37%			NX	359.0		(17.0) Top of BEDROCK at El. 359.0. Light grey to white GRANITE, very hard fresh, (Islesboro Formation). Rock Quality= Poor. Rock Core Rate (min/ft): 2, 2, 1.75, 1.75, 1.75.	
20											
	R2	60/51	22.00 - 27.00	RQD = 91%						Light grey to white GRANITE, very hard, fresh(Islesboro Formation) Rock Quality= Excellent Rock Core Rate (min/ft): 1.75, 1.75, 1.75, 1.75, 1.25	
25											

Remarks:

WD= Wash and Drive.
NX= Core Barrel.
Mobile Drill B-53 SN: D-23.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BB-NTB-101

WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	376.00	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	2" SPT
Logged By:	J. Loomis	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140 lbs./ 30 in.
Date Start/Finish:	04/13/21	**Drilling Method:**	Wash and Drive	**Core Barrel:**	2.15" (NX)
Boring Location:	N 502678.3573 E 941696.8687	**Casing ID/OD:**	HW 4" ID 4.5" OD	**Water Level*:**	8.5 feet
Hammer Efficiency Factor:	0.86	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
								349.3		26.7	
										Bottom of Exploration at 26.7 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

WD= Wash and Drive.
NX= Core Barrel.
Mobile Drill B-53 SN: D-23.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BB-NTB-102
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.22	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	2" SPT
Logged By:	J. Loomis	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140 lbs / 30 inches
Date Start/Finish:	04/14/21	**Drilling Method:**	Wash and Drive	**Core Barrel:**	2.15" (NX)
Boring Location:	N 502691.9226 E 941623.5105	**Casing ID/OD:**	HW 4" ID 4.5" OD	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.86	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WD	376.5		8 inches of ASPHALT. (0.7)	
								376.2		Grey, dry, coarse SAND, poorly-graded (FILL). (1.0)	
								375.7		6-inch CONCRETE SLAB. (1.5)	
										Grey, dry, coarse SAND, poorly-graded (FILL).	
	1D	16/4	4.75 - 6.08	1/ 45/ 28/ 5	73	105					
5										Grey, dry, very dense, fine to coarse GRAVEL, few fine to coarse Sand, poorly-graded.	
								368.2		(9.0)	
	2D	24/11	9.00 - 11.00	32/ 23/ 112/ 50	135	194				Grey, dry, very dense, fine to coarse SAND, some fine Gravel, few non-plastic fines, well- graded.	
10											
								363.2		(14.0)	
	R1	60/60	14.00 - 19.00	RQD = 97%			NX			Top of BEDROCK at El. 363.2. Light grey to white GRANITE, very hard fresh, (Islesboro Formation). Rock Quality= Excellent. Rock Core Rate (min/ft): 3.5, 3.5, 3.5, 3.25, 2.5.	
15											
	R2	60/55	19.00 - 24.00	RQD = 67%						Light grey to white GRANITE, very hard, fresh, (Islesboro Formation). Rock Quality= Fair. Rock Core Rate (min/ft): 2.5, 3.25, 3.25, 3.0, 3.5.	
20											
								353.2		(24.0)	
25										**Bottom of Exploration at 24.0 feet below ground surface.**	

Remarks:

WD= Wash and Drive.
NX= Core Barrel.
Mobile Drill B-53 SN: D-23.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BB-NTB-103
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.14	**Auger ID/OD:**	N/A
Operator:	Sam Cooley	**Datum:**	NAVD88	**Sampler:**	2" SPT
Logged By:	R. Munschauer	**Rig Type:**	Acker Soil Scout	**Hammer Wt./Fall:**	140 lbs./30 in.
Date Start/Finish:	5/18/22-5/19/22	**Drilling Method:**	Wash and Drive	**Core Barrel:**	2.15" (NX)
Boring Location:	N 502679.1729 E 941657.7734	**Casing ID/OD:**	HW 4" ID 4.5" OD	**Water Level*:**	Unable to measure
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WD	376.6		6 inches of Asphalt. (0.5)	
										Concrete and Boulders.	
5								372.1		(5.0)	
	R1		5.00 - 7.30				NX			Granite & Gabbro Boulders Coring time / ft (min): 2, 2.	
							WD			Advanced from 7.3' to 11.3' with roller bit. Roller bit through strong material from 7.3' to 9'. High drill chatter due to boulders and cobbles.	
10											
	R2	60/28	11.30 - 16.30				NX			Boulders Coring time / ft (min): 2.25, 2.5, 3, 2, 2.5.	
15											
								360.8		(16.3)	
	1D	3/3	16.30 - 16.55	50/3"	--		WD	360.6		Light brown to grey, very dense Poorly-graded SAND with Gravel, fine to coarse grained sand, some fine to coarse grained subangular gravel (possible drill wash). Driller indicated no pressure on drill bit - potential soil. (16.5)	
								360.1		Top of BEDROCK at El. 360.6 ft. Roller bit action indicated very dense material. Rate of drill 2"/ min. (17.0)	
										Bottom of Exploration at 17.0 feet below ground surface.	
20											
25											

Remarks:

Water level unable to be measured since water was introduced at the time of drilling and it did not stabilize at the end of drilling.
WD= Wash and Drive.
NX= Core Barrel.
Acker Soil Scout SN: 0907.

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.

Boring No.: BB-NTB-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BB-NTB-104
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.38	**Auger ID/OD:**	N/A
Operator:	Sam Cooley	**Datum:**	NAVD88	**Sampler:**	2" SPT
Logged By:	R. Munschauer	**Rig Type:**	Acker Soil Scout	**Hammer Wt./Fall:**	140 lbs./30 in.
Date Start/Finish:	5/18/22	**Drilling Method:**	Wash and Drive	**Core Barrel:**	2.15" (NX)
Boring Location:	N 502687.3661 E 941635.7583	**Casing ID/OD:**	HW 4" ID 4.5" OD	**Water Level*:**	Unable to measure
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

T_v = Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WD	376.8		7 inches of Asphalt. 0.6	
										Sand Sub-base.	
	R1	34/34	2.50 - 5.33				NX	374.9		2.5	
								373.9		Concrete. Coring time/ft (min): 2, 2, 2. 3.5	
										Granite and Gabbro Boulders. No mortar observed.	
5	1D	24/0	5.30 - 7.30	10/ 29/ 18/ 12	47	47	WD	372.1		5.3	
										Very dense, Glacial Outwash with Cobbles SPT sampler seized up. Driller unable to open it. Angular gravel at tip.	
10								367.4		10.0	
										Boulders. Increased bit chatter indicating harder material.	
										12.5 - 13 - No resistance on tri-cone roller bit.	
										14-14.6 - No resistance on tri-cone roller bit.	
								362.8		14.6	
15								362.2		Top of BEDROCK at El. 362.8 ft. Rate of drill 1"/3 min. 15.2	
										Bottom of Exploration at 15.2 feet below ground surface.	
20											
25											

Remarks:

Water level unable to be measured since water was introduced at the time of drilling and it did not stabilize at the end of drilling.
WD= Wash and Drive.
NX= Core Barrel.
Acker Soil Scout SN: 0907.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BB-NTB-105
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.26	**Auger ID/OD:**	N/A
Operator:	Sam Cooley	**Datum:**	NAVD88	**Sampler:**	2" SPT
Logged By:	R. Munschauer	**Rig Type:**	Acker Soil Scout	**Hammer Wt./Fall:**	140 lbs./30 in.
Date Start/Finish:	5/19/22 to 5/20/22	**Drilling Method:**	Wash and Drive	**Core Barrel:**	2.15" (NX)
Boring Location:	N 502644.6882 E 941652.5041	**Casing ID/OD:**	HW 4" ID 4.5" OD	**Water Level*:**	Unable to measure
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WD	376.6		8 inches of Asphalt. 0.7	
								376.3		Concrete. 1.0	
										Sand & Grave Sub-base.	
	R1	36/28	3.50 - 6.50				NX	373.8		3.5	
										Concrete. Coring time/ft (min): 2, 2, 1.75.	
5							WD	372.0		5.3 Boulders and Cobbles with Sand.	
	R2		9.70 - 10.70				NX				
10										Decomposed wood encountered. Coring time/ft (min): 4.	
	R3	60/12	11.00 - 16.00							Coring time/ft (min):1.75, 2.25, 2.5, 1.5, 1. 11-11.5 - No resistance on tri-cone roller bit. Boulders. 11.5-13.5 - Increase in tri-cone roller bitresistance.	
										13.5-14.5 - No resistance on tri-cone roller bit.	
								362.8		14.5 Weathered Bedrock.	
15							WD				
								359.3		18.0 Top of BEDROCK at El. 359.3 ft. Drill rate 3min/1".	
								357.8		19.5	
20										**Bottom of Exploration at 19.5 feet below ground surface.**	
25											

Remarks:

Water level unable to be measured since water was introduced at the time of drilling and it did not stabilize at the end of drilling.
WD= Wash and Drive.
NX= Core Barrel.
Acker Soil Scout SN: 0907.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BB-NTB-106
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.61	**Auger ID/OD:**	N/A
Operator:	Sam Cooley	**Datum:**	NAVD88	**Sampler:**	2" SPT
Logged By:	R. Munschauer	**Rig Type:**	Acker Soil Scout	**Hammer Wt./Fall:**	140 lbs./30 in.
Date Start/Finish:	5/19/22	**Drilling Method:**	Wash & Drive	**Core Barrel:**	2.15" (NX)
Boring Location:	N 502653.2722 E 941630.5488	**Casing ID/OD:**	HW 4" ID 4.5" OD	**Water Level*:**	Unable to measure
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WD	377.0		7 inches of Asphalt. (0.6)	
								376.6		Concrete. (1.0)	
										Sand & Grave Sub-base. (3.0)	
	R1	37/34	3.00 - 6.08				NX	374.6		Concrete. Coring time/ft (min): 2.5, 2.25, 2.5. (4.9)	
5								372.7		Boulders. No mortar observed. (6.1)	
	1D	15/2	6.10 - 7.35	17/ 5/ 50/3"	R		WD	371.5		Dark brown, very dense Silty SAND with Gravel, some fine to coarse grained gravel, little fines. (7.4)	
								370.3		Boulders. Roller bit down to 16.7'. Increased drill chatter. Boulder.	
	R2	54/19	8.00 - 12.50				NX				
10											
							WD				
15											
								360.9		(16.7)	
								360.5		Top of BEDROCK at El. 360.9 ft. Drill rate 2"/min. (17.1)	
										Bottom of Exploration at 17.1 feet below ground surface.	
20											
25											

Remarks:

Water level unable to be measured since water was introduced at the time of drilling and it did not stabilize at the end of drilling.
WD= Wash and Drive.
NX= Core Barrel.
Acker Soil Scout SN: 0907.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BP-NTB-101
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.02	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Loomis	**Rig Type:**	N/A	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	04/13/21	**Drilling Method:**	Vacuum	**Core Barrel:**	N/A
Boring Location:	N 502643.8633 E 941654.6142	**Casing ID/OD:**	N/A	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	N/A	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							V			12 inches of ASPHALT.	
								376.0		1.0 Grey, dry, coarse SAND, trace cobbles, poorly-graded (FILL).	
										Grey, dry, fine Silty SAND, little non plastic fines, trace roots (FILL).	
								373.0		4.0 **Bottom of Exploration at 4.0 feet below ground surface.**	
5											
10											
15											
20											
25											

Remarks:

Top of abutment encountered at 4 feet.
V= Vacuum.

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Boring No.: BP-NTB-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook Location: Norway, ME	Boring No.: BP-NTB-102 WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.01	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Loomis	**Rig Type:**	N/A	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	04/13/21	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	N 502642.7445 E 941657.3977	**Casing ID/OD:**	N/A	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	N/A	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
0							V			12 inches of ASPHALT.		
								376.0		Grey, dry, coarse SAND, trace cobbles, poorly-graded (FILL).	1.0	
										Grey, dry, fine Silty SAND, little non plastic fines, trace roots, trace cobbles (FILL).		
5												
								368.5		**Bottom of Exploration at 8.5 feet below ground surface.**	8.5	
10												
15												
20												
25												

Remarks:

Refusal on possible boulder at 8.5 feet.
V= Vacuum.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BP-NTB-103
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.00	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Loomis	**Rig Type:**	N/A	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	04/13/21	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	N 502641.6257 E 941660.1813	**Casing ID/OD:**	N/A	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	N/A	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							V			12 inches of ASPHALT.	
								376.0		1.0 Grey, dry, coarse SAND, trace cobbles, poorly-graded (FILL).	
										Grey, dry, fine Silty SAND, little non- plastic fines, trace roots, trace cobbles (FILL).	
5											
10											
								366.0		11.0 **Bottom of Exploration at 11.0 feet below ground surface.**	
15											
20											
25											

Remarks:

V= Vacuum.

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.

Boring No.: BP-NTB-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BP-NTB-104

WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.18	**Auger ID/OD:**	
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Loomis	**Rig Type:**	N/A	**Hammer Wt./Fall:**	
Date Start/Finish:	04/12/21	**Drilling Method:**	Vacuum	**Core Barrel:**	N/A
Boring Location:	N 502687.3661 E 941635.583	**Casing ID/OD:**	N/A	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	N/A	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

T_v = Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							V			8 inches of ASPHALT.	
								376.5		0.7	
								376.2		Grey, dry, coarse SAND, trace cobbles,trace bricks, poorly-graded. 1.0	
								375.7		6 inches of CONCRETE. 1.5	
										Grey, dry, coarse SAND, trace cobbles, trace brick, poorly-graded.	
								373.2		4.0	
										Bottom of Exploration at 4.0 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

Top of rubble abutment encountered at 4 feet.
V= Vacuum.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BP-NTB-105
WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.20	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Loomis	**Rig Type:**	N/A	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	04/12/21	**Drilling Method:**	Vacuum	**Core Barrel:**	N/A
Boring Location:	N 502688.9918 E 941630.9543	**Casing ID/OD:**	N/A	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	N/A	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							V			8 inches of ASPHALT.	
								376.5		0.7	
								376.2		Grey, dry, coarse SAND, trace cobbles,trace bricks, poorly-graded. 1.0	
								375.7		6 inches of CONCRETE. 1.5	
										Grey, dry, coarse SAND, trace cobbles, trace brick, poorly-graded.	
								373.2		4.0	
										Bottom of Exploration at 4.0 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

Top of rubble abutment encountered at 4 feet.
V= Vacuum.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tannery Brook Bridge #3610 over Tannery (Bird) Brook
Location: Norway, ME

Boring No.: BP-NTB-106

WIN: 23116.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	377.22	**Auger ID/OD:**	N/A
Operator:	Brad / Garret	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Loomis	**Rig Type:**	N/A	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	04/12/21	**Drilling Method:**	Vacuum	**Core Barrel:**	N/A
Boring Location:	N 502690.0908 E 941628.1629	**Casing ID/OD:**	N/A	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	N/A	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

T_v = Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							V			8 inches of ASPHALT.	
								376.5		0.7	
								376.2		Grey, dry, coarse SAND, trace cobbles,trace bricks, poorly-graded. 1.0	
								375.7		6 inches of CONCRETE. 1.5	
										Grey, dry, coarse SAND, trace cobbles, trace brick, poorly-graded.	
								373.2		4.0	
										Bottom of Exploration at 4.0 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

Top of rubble abutment encountered at 4 feet.
V= Vacuum.

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

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

Appendix B

Rock Core Photograph

Photograph of Rock Cores from Borings BB-NTB-101 and BB-NTB-102

Notes: 1) The top of the core run is located on the Right side of the core box.

BB-NTB-104 R1 TOP

BB-NTB-103 R1 TOP

BB-NTB-106 1 TOP

BB-NTB-105 R1 TOP

BB-NTB-103 R2 BOT

BB-NTB-106 R2 BOT

BB-NTB-105 R3 BOT

BB-NTB-105 R2

Photograph of Rock Cores from Borings BB-NTB-103 through BB-NTB-106

Notes: 1) The top of the core runs are located on the Left side of the core box.

Appendix C

Laboratory Testing Results

Client:	AECOM				
Project:	Tannery Brook Bridge				
Location:	Norway, ME			Project No:	GTX-313512
Boring ID:	B-1	Sample Type:	jar	Tested By:	cam
Sample ID:	S-1	Test Date:	04/22/21	Checked By:	bfs
Depth :	5-7	Test Id:	616182		

Test Comment: ---
Visual Description: Moist, very dark brown peat
Sample Comment: ---

Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content,%	Ash Content,%	Organic Matter,%
B-1	S-1	5-7	Moist, very dark brown peat	237	5.4	94.6

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

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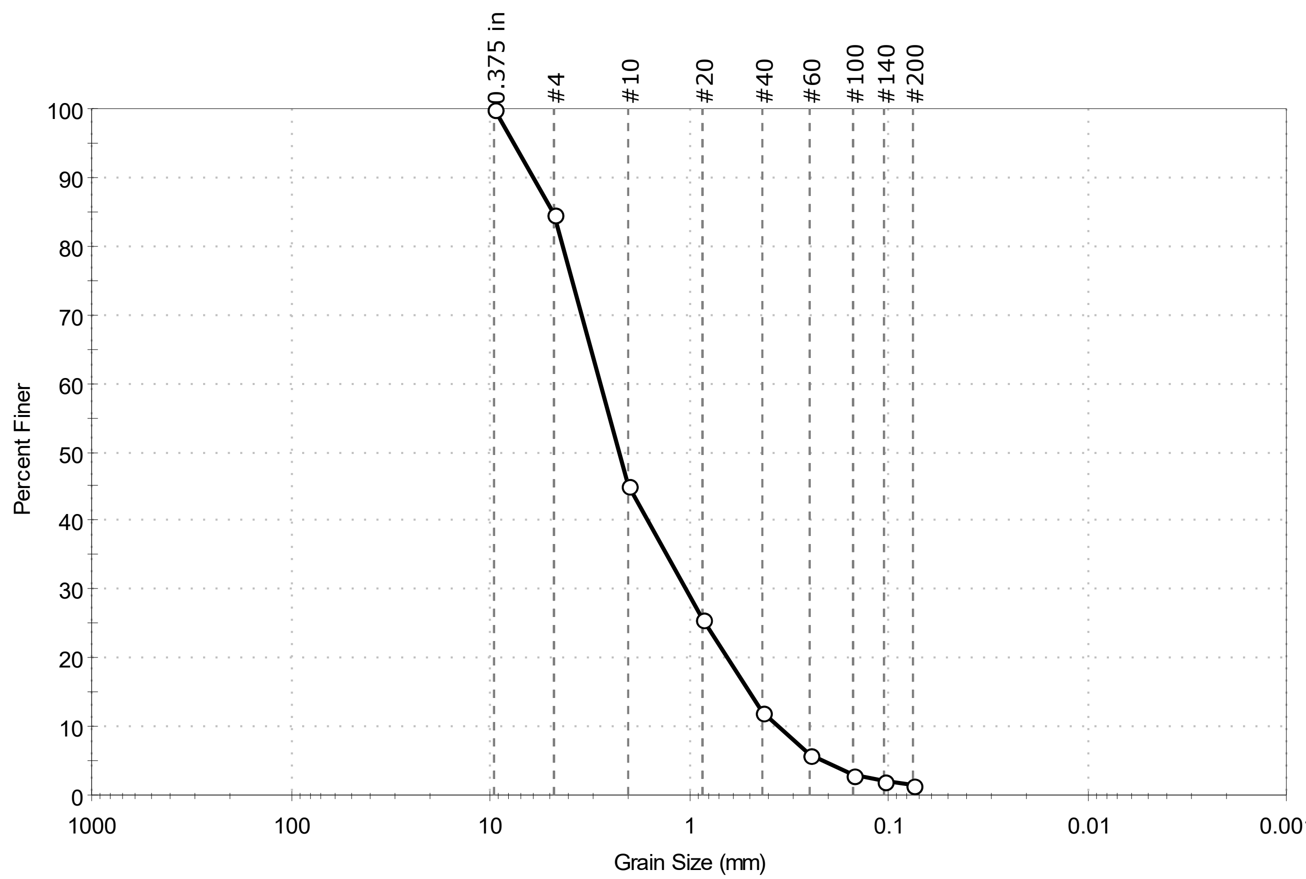
Client:	AECOM				
Project:	Tannery Brook Bridge				
Location:	Norway, ME			Project No:	GTX-313512
Boring ID:	B-1	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-1	Test Date:	04/22/21	Checked By:	bfs
Depth :	5-7	Test Id:	616179		
Test Comment:	---				
Visual Description:	Moist, very dark brown peat				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	15.4	83.2	1.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	85		
#10	2.00	45		
#20	0.85	26		
#40	0.42	12		
#60	0.25	6		
#100	0.15	3		
#140	0.11	2		
#200	0.075	1.4		

Coefficients

D_{85} =4.8296 mm D_{30} =1.0266 mm

D_{60} =2.7667 mm D_{15} =0.4930 mm

D_{50} =2.2216 mm D_{10} =0.3547 mm

C_u =7.800 C_c =1.074

Classification

ASTM Well-graded SAND with Gravel (SW)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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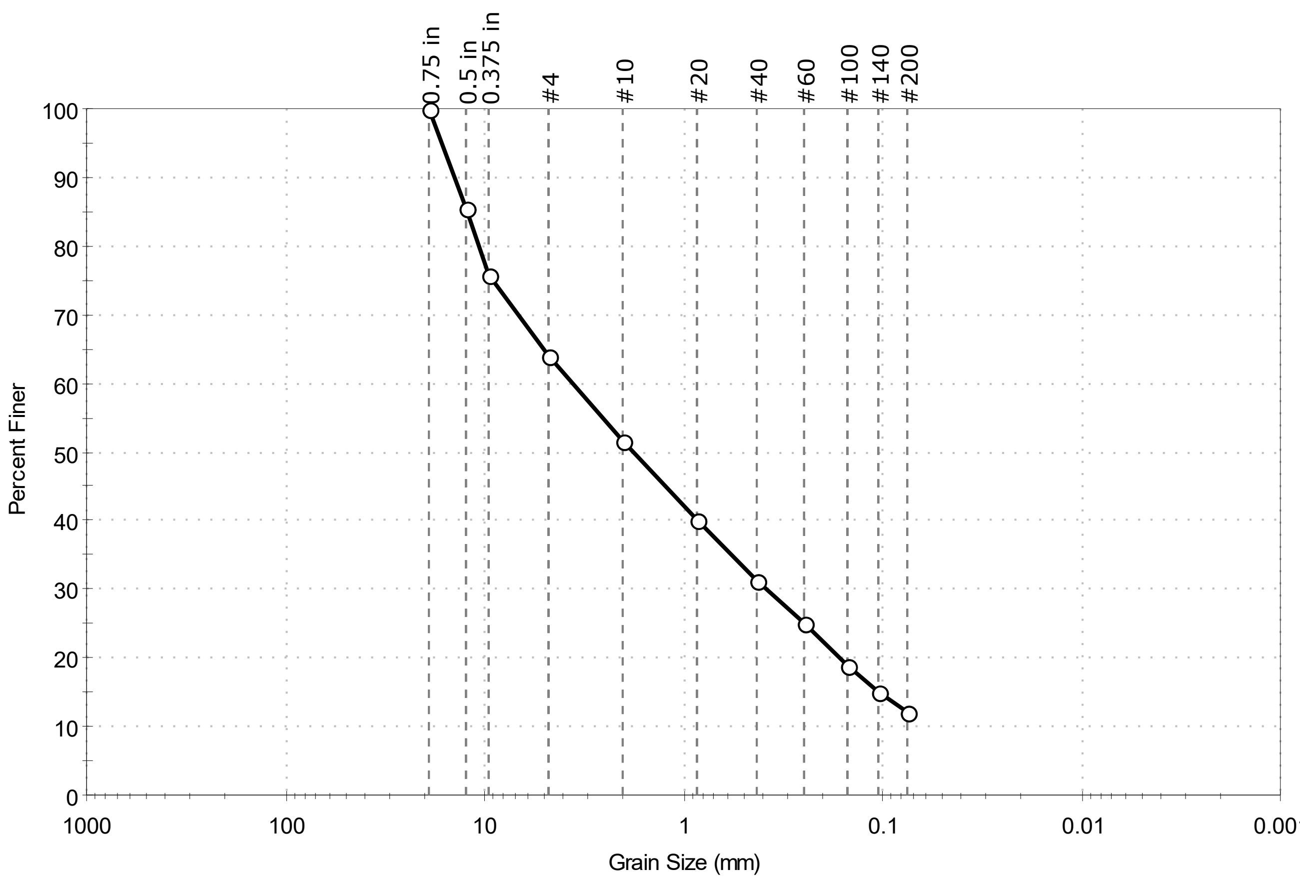
Client:	AECOM				
Project:	Tannery Brook Bridge				
Location:	Norway, ME			Project No:	GTX-313512
Boring ID:	B-2	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	04/22/21	Checked By:	bfs
Depth :	9-11	Test Id:	616181		

Test Comment: ---

Visual Description: Moist, yellowish brown silty sand with gravel

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	36.1	51.7	12.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	86		
0.375 in	9.50	76		
#4	4.75	64		
#10	2.00	52		
#20	0.85	40		
#40	0.42	31		
#60	0.25	25		
#100	0.15	19		
#140	0.11	15		
#200	0.075	12		

Coefficients

D_{85} =12.2822 mm
D_{30} =0.3794 mm
D_{60} =3.6102 mm
D_{15} =0.1039 mm
D_{50} =1.7692 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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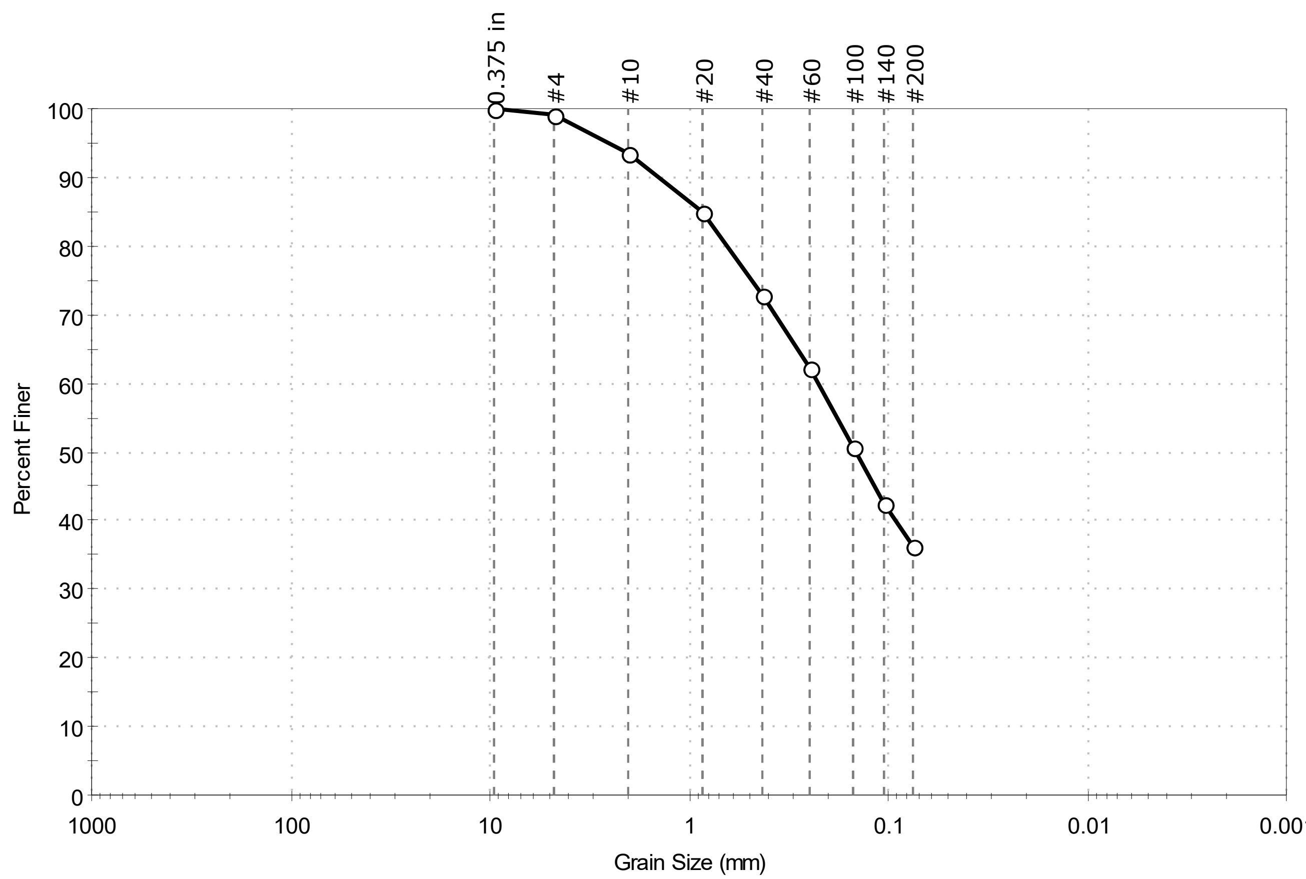
Client:	AECOM				
Project:	Tannery Brook Bridge				
Location:	Norway, ME			Project No:	GTX-313512
Boring ID:	B-1	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-3	Test Date:	04/22/21	Checked By:	bfs
Depth :	14-16	Test Id:	616180		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.0	62.6	36.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	93		
#20	0.85	85		
#40	0.42	73		
#60	0.25	62		
#100	0.15	51		
#140	0.11	42		
#200	0.075	36		

Coefficients

D_{85} =0.8658 mm　D_{30} =N/A
D_{60} =0.2264 mm　D_{15} =N/A
D_{50} =0.1456 mm　D_{10} =N/A
C_u =N/A　C_c =N/A

Classification

ASTM　N/A

AASHTO　Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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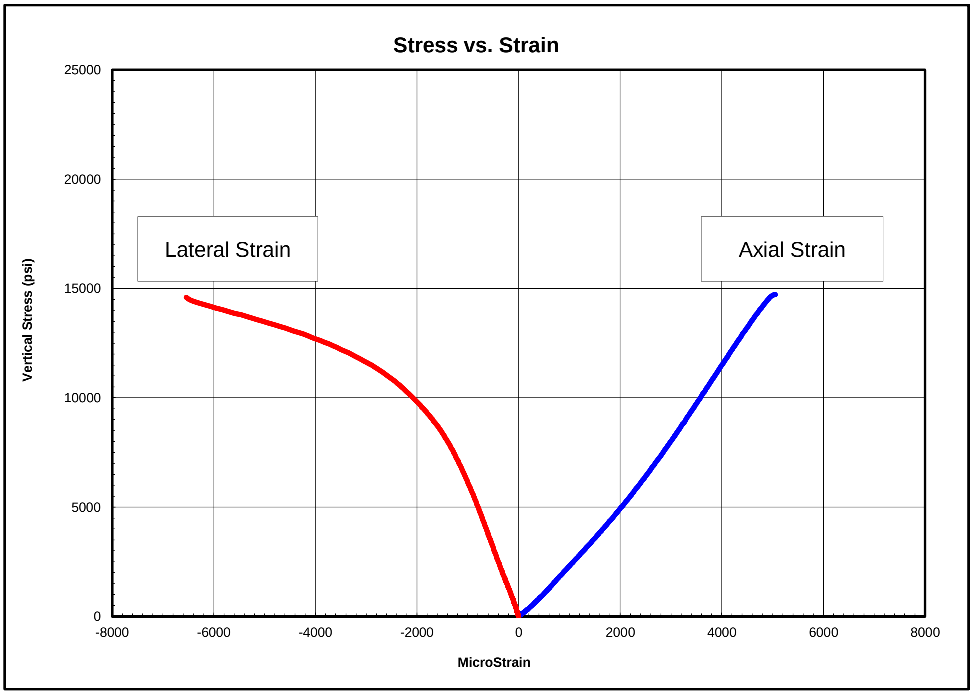
Client:	AECOM
Project Name:	Tannery Brook Bridge
Project Location:	Norway, ME
GTX #:	317606
Test Date:	8/23/2023
Tested By:	te
Checked By:	jsc
Boring ID:	B-1
Sample ID:	C1
Depth, ft:	21.1-22.1
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 14,719 psi

The strain values recorded within the second and third stress ranges for this test produce values of Poisson's Ratio that exceed maximum values found in rocks.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1500-5400	2,600,000	0.42
5400-9300	3,210,000	---
9300-13200	3,510,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	AECOM	Test Date:	8/21/2023
Project Name:	Tannery Brook Bridge	Tested By:	te
Project Location:	Norway, ME	Checked By:	smd
GTX #:	317606		
Boring ID:	B-1		
Sample ID:	C1		
Depth:	21.1-22.1 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.08	4.08	4.08	
Specimen Diameter, in:	1.94	1.94	1.94	
Specimen Mass, g:	519.21			
Bulk Density, lb/ft^3	164	**Minimum Diameter Tolerence Met?**		**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**		**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? **YES**

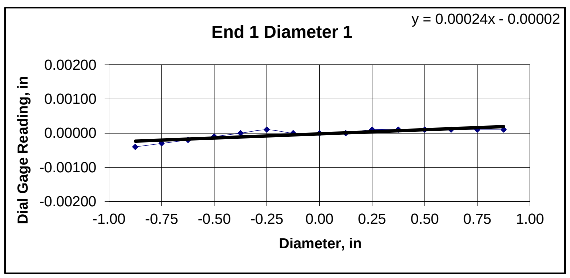
END FLATNESS AND PARALLELISM (Procedure FP1)

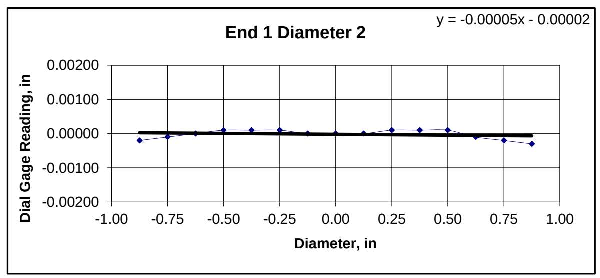
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	0.00000	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	-0.00010	-0.00020	-0.00030

Difference between max and min readings, in:
0° = 0.00050 90° = 0.00040

END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010

Difference between max and min readings, in:
0° = 0.0005 90° = 0.0003
Maximum difference must be < 0.0020 in. Difference = ± 0.00025
Flatness Tolerance Met? **YES**





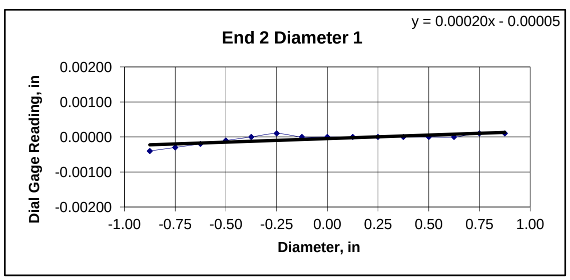
DIAMETER 1

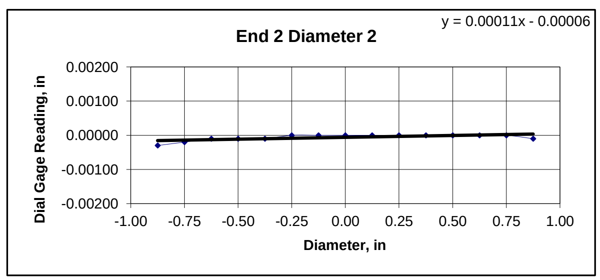
End 1:
Slope of Best Fit Line 0.00024
Angle of Best Fit Line: 0.01391

End 2:
Slope of Best Fit Line 0.00020
Angle of Best Fit Line: 0.01162

Maximum Angular Difference: 0.00229

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00005
Angle of Best Fit Line: 0.00295

End 2:
Slope of Best Fit Line 0.00011
Angle of Best Fit Line: 0.00622

Maximum Angular Difference: 0.00327

Parallelism Tolerance Met? **YES**
Spherically Seated

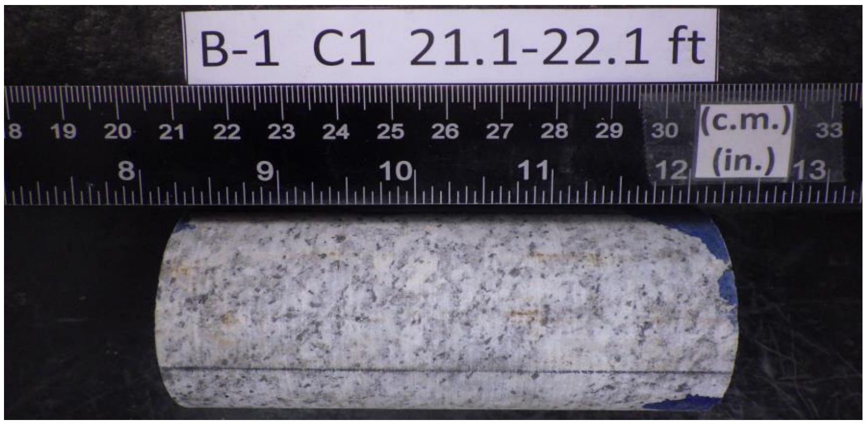
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	1.940	0.00026	0.015	YES
Diameter 2, in (rotated 90°)	0.00040	1.940	0.00021	0.012	YES
END 2					
Diameter 1, in	0.00050	1.940	0.00026	0.015	YES
Diameter 2, in (rotated 90°)	0.00030	1.940	0.00015	0.009	YES

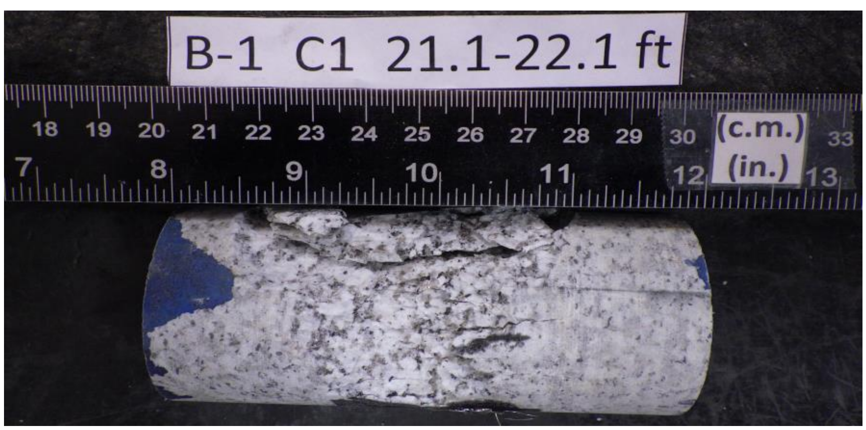
Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	AECOM
Project Name:	Tannery Brook Bridge
Project Location:	Norway, ME
GTX #:	317606
Test Date:	8/23/2023
Tested By:	te
Checked By:	smd
Boring ID:	B-1
Sample ID:	C1
Depth, ft:	21.1-22.1



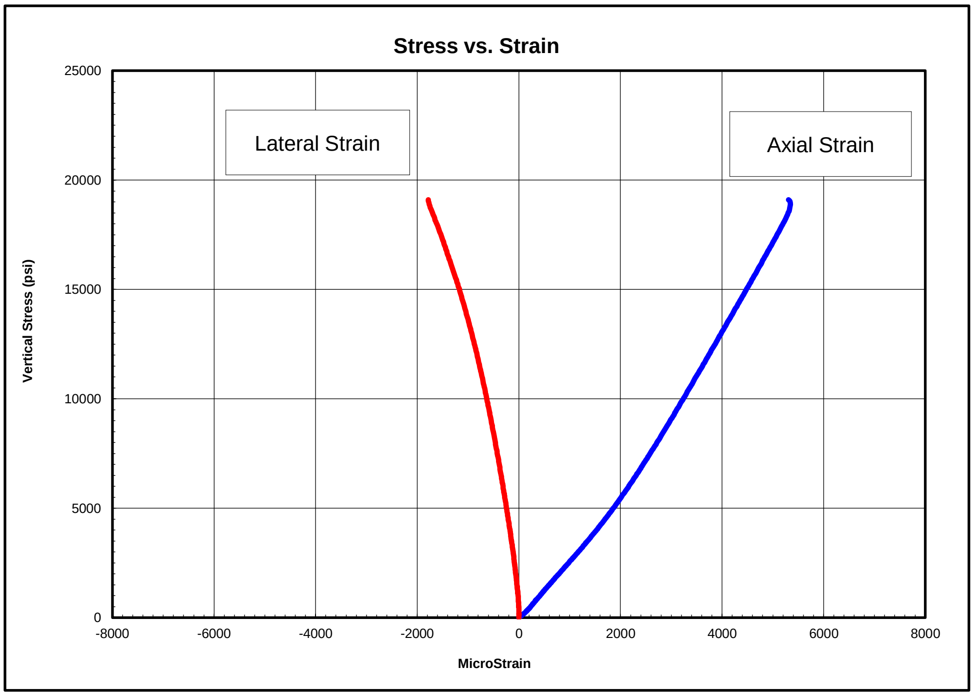
After cutting and grinding



After break

Client:	AECOM
Project Name:	Tannery Brook Bridge
Project Location:	Norway, ME
GTX #:	317606
Test Date:	8/23/2023
Tested By:	te
Checked By:	jsc
Boring ID:	B-2
Sample ID:	C1
Depth, ft:	18-19
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 19,098 psi

The strain values recorded within the third stress range for this test produce values of Poisson's Ratio that exceed maximum values found in rocks.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1900-7000	2,960,000	0.19
7000-12100	3,880,000	0.34
12100-17200	4,070,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	AECOM	Test Date:	8/21/2023
Project Name:	Tannery Brook Bridge	Tested By:	te
Project Location:	Norway, ME	Checked By:	smd
GTX #:	317606		
Boring ID:	B-2		
Sample ID:	C1		
Depth:	18-19 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.15	4.15	4.15
Specimen Diameter, in:	1.96	1.96	1.96
Specimen Mass, g:	538.23		
Bulk Density, lb/ft^3	163	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010

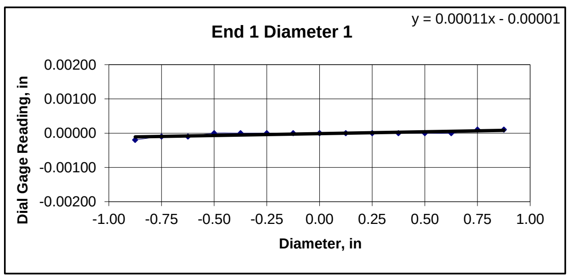
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00030

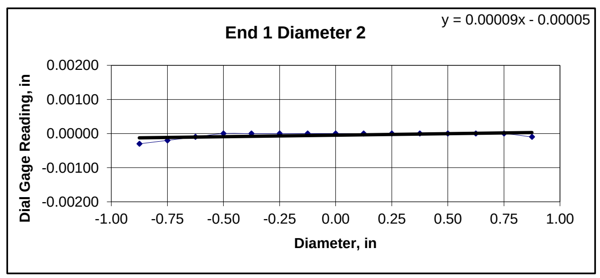
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030

Difference between max and min readings, in:
0° = 0.0001 90° = 0.0004

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00020

Flatness Tolerance Met? **YES**

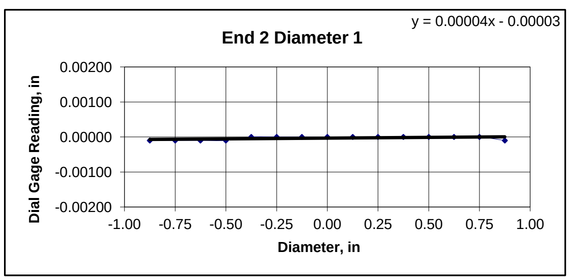


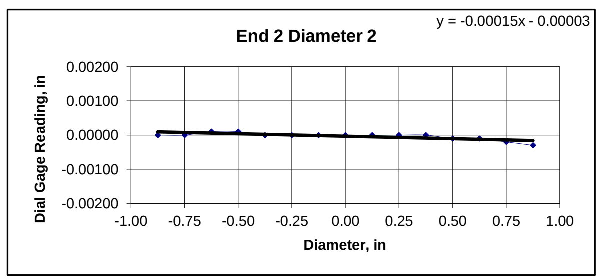


DIAMETER 1

End 1:	Slope of Best Fit Line	0.00011
	Angle of Best Fit Line:	0.00622
End 2:	Slope of Best Fit Line	0.00004
	Angle of Best Fit Line:	0.00246
Maximum Angular Difference:		0.00377

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:	Slope of Best Fit Line	0.00009
	Angle of Best Fit Line:	0.00507
End 2:	Slope of Best Fit Line	0.00015
	Angle of Best Fit Line:	0.00835
Maximum Angular Difference:		0.00327

Parallelism Tolerance Met? **YES**
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

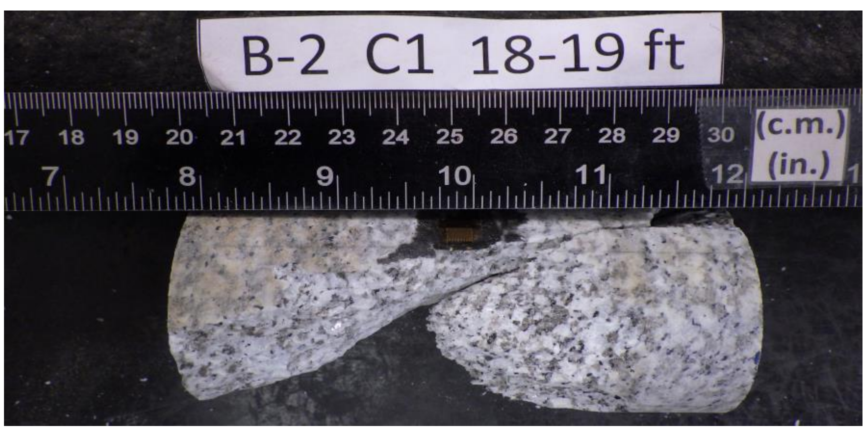
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	1.960	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00030	1.960	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00010	1.960	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00040	1.960	0.00020	0.012	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	AECOM
Project Name:	Tannery Brook Bridge
Project Location:	Norway, ME
GTX #:	317606
Test Date:	8/23/2023
Tested By:	te
Checked By:	smd
Boring ID:	B-2
Sample ID:	C1
Depth, ft:	18-19

After cutting and grinding



After break

Appendix D

Exploration Photographs

Photograph of non-sampling probes P2A (BP-NTB-104), P2B (BP-NTB-105), P2C (BP-NTB-106), and boring BB-NTB-102

Photograph of non-sampling probes P1A-ALT (BP-NTB-101), P1B-ALT (BP-NTB-102), and P1C-ALT (BP-NTB-103)

Photograph of boring BB-NTB-101

Photograph of boring BB-NTB-101

Photograph of boring BB-NTB-103

Photograph of boring BB-NTB-104

Photograph of boring BB-NTB-105

Photograph of boring BB-NTB-106

Appendix E

Calculations

Lateral Earth Pressure Coefficients

Surcharge Soil Prop.		Bearing Soil Properties	
Φ' =	32 degs	Φ' =	32.0 degs
γ =	125 pcf	γ =	125 pcf
c_u =	0 psf	c_u =	0 psf

Active Earth Pressure Coefficient

$k_a = \tan^2(45 - \phi/2)$

k_a =	0.31

Passive Earth Pressure Coefficient

$k_p = 1 / k_a$

k_p =	3.25

At-Rest Earth Pressure Coefficient

$k_o = 1 - \sin\phi$

k_o =	0.47

Seismic Site Classification

Reference: AASHTO LRFD Bridge Design Specifications 9th Edition, Section 3.10

Data from Boring No. BB-NTB-101

Layer No.	Depth Range Start (ft)	End (ft)	Thickness, d_i (ft)	N_i (blows/ft)	di/Ni	Stratum
1	**0**	**4**	4	**5**	0.80	Fill
2	**4**	**9**	5	**2**	2.50	Organics
3	**9**	**16**	7	**32**	0.22	Silty Sand
4	**16**	**100**	84	**50**	1.68	Rock
Σ =			100		5.20	

N bar=	19.24	Site Class:	D

Data from Boring No. BB-NTB-102

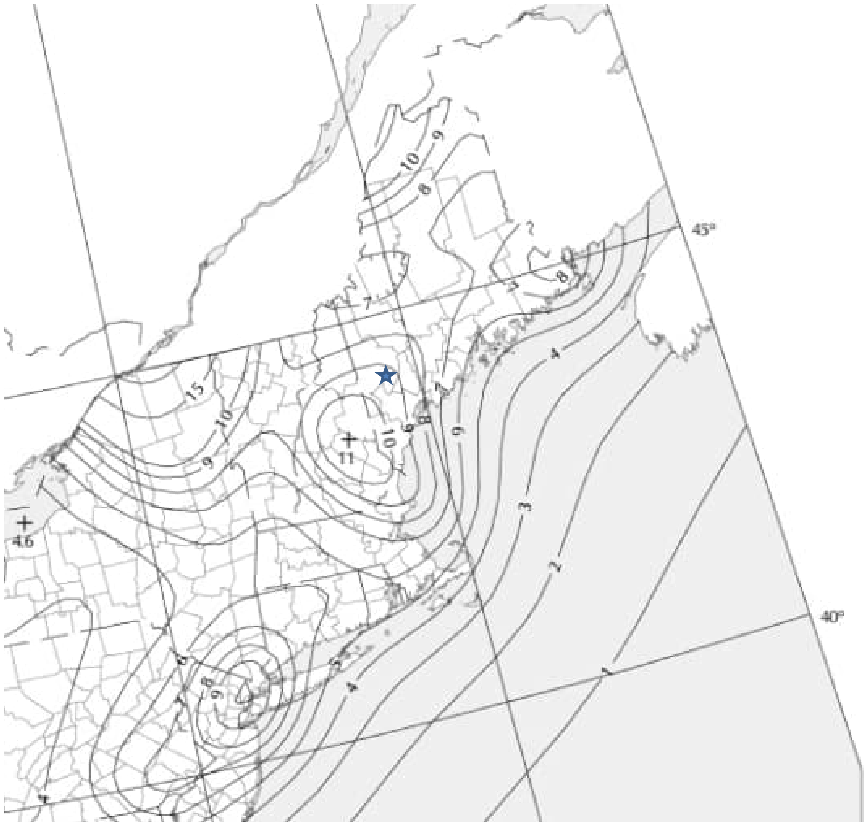
Layer No.	Depth Range Start (ft)	End (ft)	Thickness, d_i (ft)	N_i (blows/ft)	di/Ni	Stratum
1	**0**	**9**	9	**73**	0.12	Fill
2	**9**	**14**	5	**135**	0.04	Sand
3	**14**	**100**	86	**50**	1.72	Rock
Σ =			100		1.88	

N bar=	53.18	Site Class:	C

Table 3.10.3.1-1—Site Class Definitions

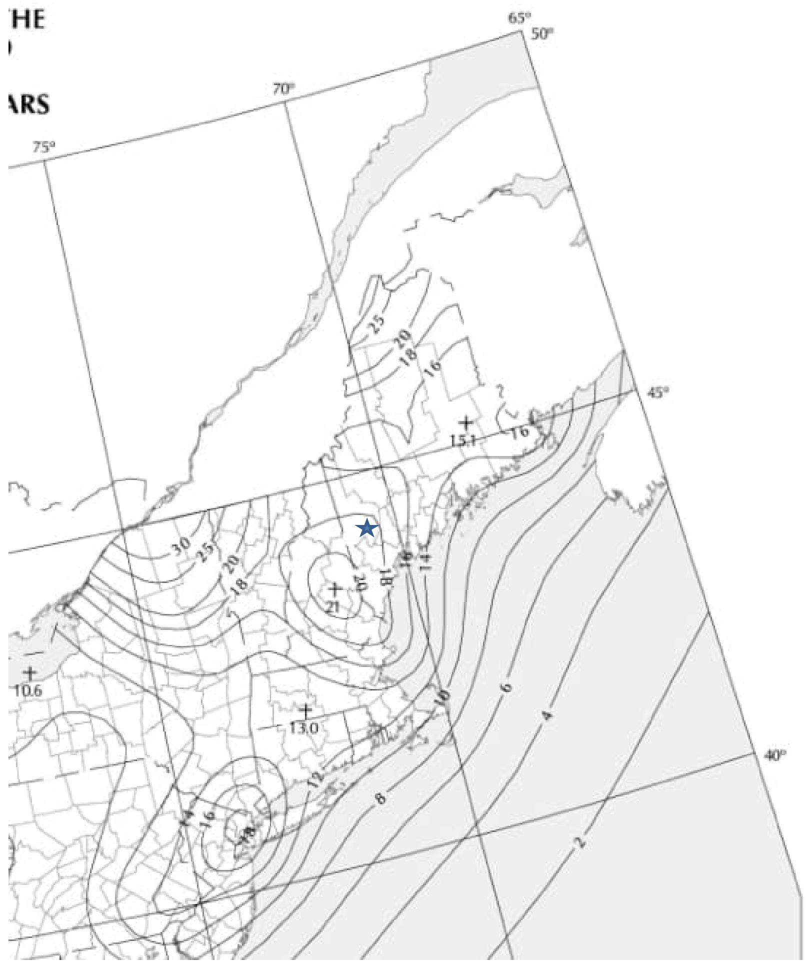
Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s < 5{,}000$ ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s < 2{,}500$ ft/s, or with either $\bar{N} > 50$ blows/ft, or $\bar{s}_u > 2.0$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

Peak Ground Acceleration, PGA, determined from Figure 3.10.2.1-1 for 7% Exceedance in 75 Years



Excerpt from Figure 3.10.2.1-1

Spectral Response Acceleration at short period, Ss, determined from Figure 3.10.2.1-2 for 7% Exceedance in 75 Years



Excerpt from Figure 3.10.2.1-2

Spectral Response Acceleration at 1 second, S1, determined from Figure 3.10.2.1-3 for 7% Exceedance in 75 Years

Excerpt from Figure 3.10.2.1-3

Site modification coefficients are selected based on PGA and Site Class D

Table 3.10.3.2-1—Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Site Class	Peak Ground Acceleration Coefficient (PGA)[1]				
	$PGA <$ 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[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of PGA.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Table 3.10.3.2-2—Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]				
	$S_S <$ 0.25	$S_S =$ 0.50	$S_S =$ 0.75	$S_S =$ 1.00	$S_S >$ 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[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_s.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Table 3.10.3.2-3—Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]				
	$S_1 <$ 0.1	$S_1 =$ 0.2	$S_1 =$ 0.3	$S_1 =$ 0.4	$S_1 >$ 0.5
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[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_1.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

$$A_S = F_{pga}\, PGA \qquad (3.10.4.2\text{-}2)$$

$$S_{DS} = F_a\, S_S \qquad (3.10.4.2\text{-}3)$$

$$S_{D1} = F_v\, S_1 \qquad (3.10.4.2\text{-}6)$$

PGA (g)	Ss (g)	S_1 (g)
0.09	0.19	0.048

As (g)	S_{DS} (g)	S_{D1} (g)
0.144	0.304	0.115

Spun Pipe Pile Calculations

Spun Pipe Pile Resistance

- Reference: AASHTO LRFD Bridge Design Specifications 9th Ed.
  - Drilled Shaft Tip Resistance on Jointed-Rock

- Purpose: compute tip resistance on jointed rock using Article 10.8.3.5.4c-2 and GSI calculation.
  - Neglect side resistance

- Rock Info:
  Top of Rock: BB-NTB-101 = 17', BB-NTB-102 = 14'
  Granite, "poor" to "excellent" quality.
  RQD ranges: 37%, 91%, 97%, 67%
  Lab UCS: $\boxed{\text{B-1 C1 } 14{,}719 \text{ psi}}$ $\boxed{\text{B-2 C1 } 19{,}098 \text{ psi}}$

Tip Resistance

Eq. 10.8.3.5.4c-2
$$q_p = A + q_u\left(m_b\left(\frac{A}{q_u}\right) + s\right)^a$$

Eq. 10.8.3.5.4c-3
$$A = \sigma'_{vb} + q_u\left(m_b\frac{\sigma'_{vb}}{q_u} + s\right)^a$$

Eq. 10.4.6.4-2
$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$

Eq. 10.4.6.4-3
$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$

D = 0 for undisturbed excavation method

AECOM

JOB TITLE Tannery Brook Bridge
JOB NO. 60656103
CALCULATION NO.
ORIGINATOR K. Scancarello
DATE 9/1/2023
REVIEWER T. Dwyer
DATE 9/10/2023
SCALE

Spun Pipe pile Resistance cont'd.

Eq. 10.4.6.4-4 $$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$

m_i estimated from Table 10.4.6.4-1 for Granite
$\Rightarrow m_i = 30$

GSI: see GSI calculation. A value of 50 was selected based on rounding the average GSI down from 52.

$$m_b = 30\, e^{\left(\frac{50-100}{28-\underset{0}{14D}}\right)} = 5.03$$

$$s = e^{\left(\frac{50-100}{9-0}\right)} = 0.004$$

$$a = 0.5 + \frac{1}{6}\left(e^{\frac{-50}{15}} - e^{\frac{-20}{3}}\right) = 0.506$$

Effective Stress:

- Min. depth to rock is 14'. Assume min. embedment of 3 ft for a depth of 17 feet. bearing.
- Assume groundwater at top of rock.

$\gamma_{soil} = 125$ pcf $H = 17'$, $3'$ in rock $\gamma_{rock} = 110$ pcf

$$\sigma'_{vb} = 125(14') + 110(3') = 2080 \text{ psf} \Rightarrow 2.08 \text{ ksf}$$

$$A = 2.08 + \frac{14719 * 144 in^2}{1000 \frac{lb}{kip}}\left[5.03\left(\frac{2.08}{14719\left(\frac{144}{1000}\right)}\right) + 0.004\right]^{0.506} = 195.6 \text{ ksf}$$

Grid: 4x4 = 1"

Spun Pipe Pile Resistance Cont'd

$$q_p = 195.6 + 14719(0.144)\left(5.03\left(\frac{195.6}{14719(0.144)}\right) + 0.004\right)^{0.5057}$$

$$q_p = 1639.5 \text{ ksf}$$

$\phi_{stat} = 0.5$ Table 10.5.5.2.5-1 for micropile tip on Rock

Factored Unit Tip Resistance, $\phi_{stat} q_p$
819.7 ksf

9 5/8" Factored Tip Resistance, $\phi_{stat} R_p = \phi_{stat} q_p A_p$

$$A_p = \pi(0.25)(9.625^2)/144 \frac{in^2}{ft^2} = 0.5053 \text{ ft}^2$$

$$0.5(1639.5)(0.5053) = 414.2 \text{ ksf}$$

The value of the constant m_i should be estimated from Table 10.4.6.4-1, based on lithology. Relationships between GSI and the parameters m_b, s, and a, according to Hoek et al. (2002) are as follows:

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad (10.4.6.4\text{-}4)$$

Table 10.4.6.4-1—Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000; with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
IGNEOUS	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
IGNEOUS	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

Disturbance to the foundation excavation caused by the rock removal methodology should be considered through the disturbance factor, D, in Eqs. 10.4.6.4-2 through 10.4.6.4-4.

The disturbance factor, D, ranges from 0.0 (undisturbed) to 1.0 (highly disturbed), and is an adjustment for the rock mass disturbance induced by the excavation method. Suggested values for various tunnel and slope excavations can be found in Hoek et al. (2002).

Computation of Geologic Strength Index, GSI

Reference:

Hoek, E., Carter, T.G., and Diederichs, M.S., 2013, “Quantification of the Geological Strength Index Chart”, 47th US Rock Mechanics / Geomechanics Symposium, ARMA Paper 13-672, 9pp.

AASHTO LRFD Bridge Design Specifications 9th Edition, Section 10.4.6.4

Project:	Tannery Brook Bridge
Date:	9/5/2023

Summary of Intact Bedrock Data from Borings				
Boring ID	Core Run ID	Run Depth (ft bgs)	Rock Description	Rock Quality Designation, RQD
BB-NTB-101	R1	17-22	GRANITE: Very Hard, Fresh, Poor	37
BB-NTB-101	R2	22-26.7	GRANITE: Very Hard, Fresh, Excellent	91
BB-NTB-102	R1	14-19	GRANITE: Very Hard, Fresh, Excellent	97
BB-NTB-102	R2	19-24	GRANITE: Very Hard, Fresh, Fair	67

GSI is based on the JCond89 rating for joint condition ratings and RQD. From Hoek,et al. 2013:

$$GSI = 1.5\ JCond_{89} + RQD/2 \qquad (1)$$

Definition of JCond89 from Bieniawski, Z.T. 1989. "Engineering rock mass classification." New York, Wiley Interscience

Table 1: Definition of $JCond_{89}$, after Bieniawski (1989) [5].

Condition of discontinuities	Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1 – 5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous
Rating	30	25	20	10	0

Guidelines for classification of discontinuity conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 to 3 m 4	3 to10 m 2	10 to 20 m 1	More than 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 – 1.0 mm 4	1 – 5 mm 1	More than 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard infilling < 5 mm 4	Hard filling > 5 mm 2	Soft infilling < 5 mm 2	Soft infilling > 5 mm 0
Weathering Rating	Unweathered 6	Slightly weathered 5	Moderate weathering 3	Highly weathered 1	Decomposed 0

Typical Description of Granite from Logs: White GRANITE, very hard, medium to coarse grained, slightly foliated, slightly weathered, occasionally pegmati

JCond89 Granite

Condition	Rating	JCond89
Discontinuity Length, Persistence	4	22
Separation, Aperture	4	
Roughness	5	
Infilling, Gouge	4	
Weathering Rating	5	

Boring ID	Core Run ID	Run Depth (ft bgs)	Rock Quality Designation, RQD (%)	1.5*JCond89	RQD/2 (%)	GSI
BB-NTB-101	R1	17-22	37	33	18.5	51.5
BB-NTB-101	R2	22-26.7	91	33	45.5	78.5
BB-NTB-102	R1	14-19	97	33	48.5	81.5
BB-NTB-102	R2	19-24	67	33	33.5	66.5

Sample GSI Calc: GSI = 1.5*22+37/2 = 51.5

Required Shaft Diameter and Embedment on Rock

Reference: AASHTO LRFD Bridge Design Specifications 9th Edition, Section 10

Project:	Tannery Brook Bridge
Date:	9/1/2023
Structure:	Spun Pipe Pile on Rock
Boring	BB-NTB-102
El. Bottom of Pile Cap (ft NAVD88)	374
Min. El. of Sound Rock (ft NAVD88)	363.2
Min. Depth of Sound Rock (ft bgs)	10.8
Socket Length (ft)	3
Tip Bearing Depth (ft bgs)	13.8
Groundwater Depth (ft bgs)	Not Encountered, assume at top of rock

TIP RESISTANCE

Jointed Rock

Based on Drilled Shaft Tip Resistance: Article 10.8.3.5.4c

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a \qquad (10.8.3.5.4c\text{-}2)$$

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u} + s\right]^a \qquad (10.8.3.5.4c\text{-}3)$$

Hoek-Brown Parameters: Article 10.4.6.4-4

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad (10.4.6.4\text{-}2)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad (10.4.6.4\text{-}3)$$

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad (10.4.6.4\text{-}4)$$

See Table 10.4.6.4-1 for values for Constant m_i for Granite

See separate calculation for computing GSI.

See hand calculation for sample computation of all parameters.

where:

e	=	2.718 (natural or Naperian log base)
D	=	disturbance factor (dim)
σ'_1 and σ'_3	=	principal effective stresses (ksf)
q_u	=	average unconfined compressive strength of rock core (ksf)
m_b, s, and a	=	empirically determined parameters

INPUTS						
Laboratory UCS, q_u (psi)	Tip Diameter, B (in)	GSI	Υ_{soil} (pcf)	Υ'_{rock} (pcf)	Disturbance Factor, D	Resistance Factor, Tip φ_{stat}
14719	8.625	50	125	110	0	0.5

HOEK-BROWN PARAMETERS			
mi	m_b	s	a
30	5.03	0.0039	0.5057

σ'_{vb} (ksf)	A (ksf)	Tip Area, A_p (in^2)
1.68	184.4	58.43

TIP RESISTANCE			
q_p (ksf)	Nominal Tip Resistance q_p A_p (ksf)	Factored Unit Resistance, φ_{stat} q_p (ksf)	Factored Tip Resistance, φ_{stat} q_p A_p (ksf)
1586.0	643.5	793.0	321.7

Required Shaft Diameter and Embedment on Rock

Reference: AASHTO LRFD Bridge Design Specifications 9th Edition, Section 10

Project:	Tannery Brook Bridge
Date:	9/1/2023
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TIP RESISTANCE

Jointed Rock

Based on Drilled Shaft Tip Resistance: Article 10.8.3.5.4c

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a \qquad (10.8.3.5.4c\text{-}2)$$

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u} + s\right]^a \qquad (10.8.3.5.4c\text{-}3)$$

Hoek-Brown Parameters: Article 10.4.6.4-4

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad (10.4.6.4\text{-}2)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad (10.4.6.4\text{-}3)$$

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See Table 10.4.6.4-1 for values for Constant m_i for Granite

See separate calculation for computing GSI.

See hand calculation for sample computation of all parameters.

where:

e	=	2.718 (natural or Naperian log base)
D	=	disturbance factor (dim)
σ'_1 and σ'_3	=	principal effective stresses (ksf)
q_u	=	average unconfined compressive strength of rock core (ksf)
m_b, s, and a	=	empirically determined parameters

INPUTS						
Laboratory UCS, q_u (psi)	Tip Diameter, B (in)	GSI	Υ_{soil} (pcf)	Υ'_{rock} (pcf)	Disturbance Factor, D	Resistance Factor, Tip φ_{stat}
14719	9.625	50	125	110	0	0.5

HOEK-BROWN PARAMETERS			
mi	m_b	*s*	*a*
30	5.03	0.0039	0.5057

σ'_{vb} (ksf)	A (ksf)	Tip Area, A_p (in^2)
1.68	184.4	72.76

TIP RESISTANCE			
q_p (ksf)	Nominal Tip Resistance $q_p A_p$ (ksf)	Factored Unit Resistance, φ_{stat} q_p (ksf)	Factored Tip Resistance, φ_{stat} q_p A_p (ksf)
1586.0	801.4	793.0	400.7

Required Shaft Diameter and Embedment on Rock

Reference: AASHTO LRFD Bridge Design Specifications 9th Edition, Section 10

Project:	Tannery Brook Bridge
Date:	9/1/2023
Structure:	Spun Pipe Pile on Rock
Boring	BB-NTB-102
El. Bottom of Pile Cap (ft NAVD88)	374
Min. El. of Sound Rock (ft NAVD88)	363.2
Min. Depth of Sound Rock (ft bgs)	10.8
Socket Length (ft)	3
Tip Bearing Depth (ft bgs)	13.8
Groundwater Depth (ft bgs)	Not Encountered, assume at top of rock

TIP RESISTANCE

Jointed Rock

Based on Drilled Shaft Tip Resistance: Article 10.8.3.5.4c

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right)+s\right]^a \qquad (10.8.3.5.4c\text{-}2)$$

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u}+s\right]^a \qquad (10.8.3.5.4c\text{-}3)$$

Hoek-Brown Parameters: Article 10.4.6.4-4

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad (10.4.6.4\text{-}2)$$

$$a = \frac{1}{2}+\frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad (10.4.6.4\text{-}3)$$

where:

e	=	2.718 (natural or Naperian log base)
D	=	disturbance factor (dim)
σ'_1 and σ'_3	=	principal effective stresses (ksf)
q_u	=	average unconfined compressive strength of rock core (ksf)
m_b, s, and a	=	empirically determined parameters

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad (10.4.6.4\text{-}4)$$

See Table 10.4.6.4-1 for values for Constant m_i for Granite

See separate calculation for computing GSI.

See hand calculation for sample computation of all parameters.

INPUTS						
Laboratory UCS, q_u (psi)	Tip Diameter, B (in)	GSI	Υ_{soil} (pcf)	Υ'_{rock} (pcf)	Disturbance Factor, D	Resistance Factor, Tip φ_{stat}
14719	10.75	50	125	110	0	0.5

HOEK-BROWN PARAMETERS			
mi	m_b	*s*	*a*
30	5.03	0.0039	0.5057

σ'_{vb} (ksf)	A (ksf)	Tip Area, A_p (in^2)
1.68	184.4	90.76

TIP RESISTANCE			
q_p (ksf)	Nominal Tip Resistance $q_p A_p$ (ksf)	Factored Unit Resistance, φ_{stat} q_p (ksf)	Factored Tip Resistance, φ_{stat} $q_p A_p$ (ksf)
1586.0	999.6	793.0	499.8

Tannery Brook Bridge

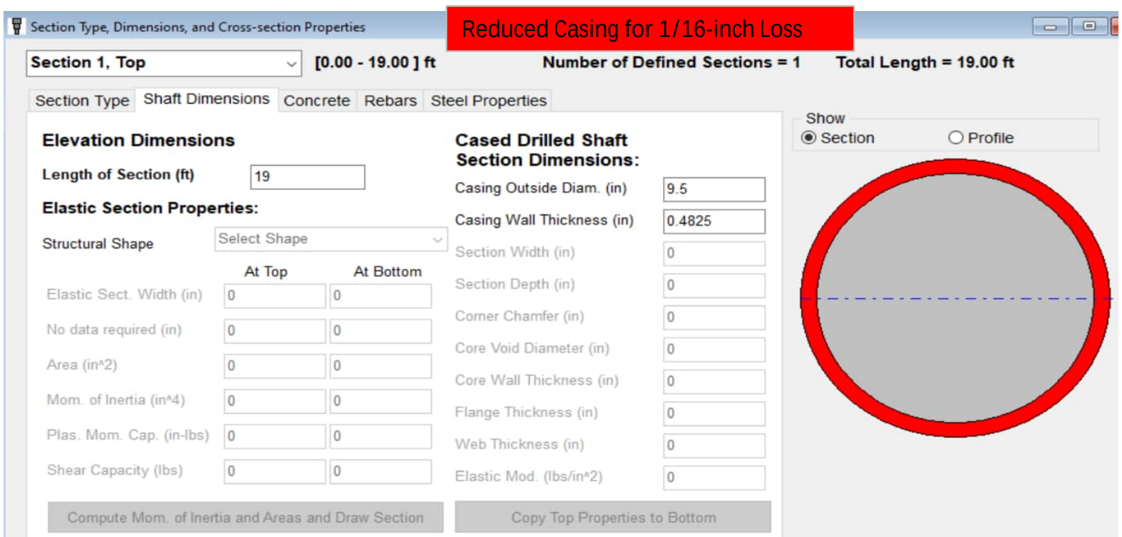
Spun Pipe Pile

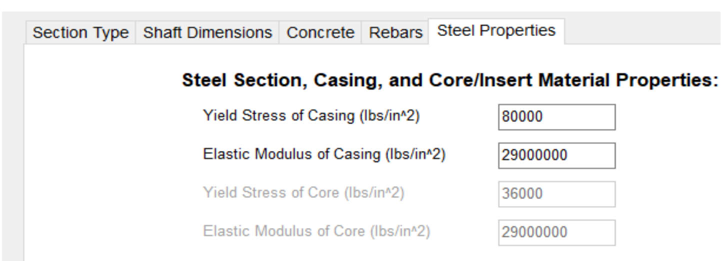
Lateral Pile Analysis

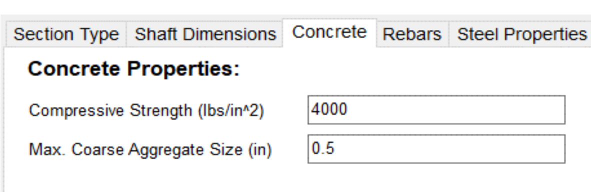
Top of Pile EL/Bot. of Cap (ft NAVD88)	374
Min. EL of Sound Rock (ft NAVD88)	360
Pile Tip EL (ft NAVD88)	355
Total Pile Length (ft)	19.0

9.625" Spun Pipe Pile bearing in Rock

Section	OD Casing No Reduction (in)	OD Reduced due to Couplings or Corrosion (in)	ID casing (in)	Length of Section (ft)	Casing Wall Thickness (in)	Yield Strength of Casing (ksi)	Elastic Modulus of Casing (psi)
Steel Casing - Corrosion	9.625	9.500	8.535	19.0	0.4825	80	2.90E+07







Bridge No.	Tannery Brook
Substructure	Spun Pipe Pile
Available Borings	BB-NTB-101
Substructure Stationing	NA
Bottom of Pile Cap El (ft NAVD88)	374.00
Top of Pile Cap El (ft NAVD88)	375.70

Soil Profile Boring	BB-NTB-101
Ground Surface EL at Boring (ft NAVD88)	376.0
GW EL (ft NAVD88)	376.0
Min. EL of Sound Rock (ft NAVD88)	360.0
Pile Tip Bearing EL (ft NAVD88)	355.00
Pile Top El (ft NAVD88)	374.00
Total Pile Length (ft)	19.00

SUBSURFACE PROFILE FOR PILE LATERAL ANALYSES

Soil Properties	Layer No.	*Depth to Top of Layer (ft bgs)	EL at Top of Layer (ft NAVD88)	Depth to Bottom of Layer (ft bgs)	EL at Bottom of Layer (ft NAVD88)	Layer Thickness (ft)	GROUP soil Layer	Total Unit Weight, γ (pcf)	Effective Unit Weight, γ' (pcf)	Friction Angle (deg)	Soil Modulus Param. k (pci)	Cohesion Top/Bot, c (psf)	ε_{50}	Rock Qu (psi)
FILL - Silty SAND	1	0	374.0	2.0	372.0	2.0	Sand (Reese)	120	57.6	32	27	--	--	--
Organic SILT/PEAT	2	2.0	372.0	7.0	367.0	5.0	Soft Clay	110	47.6	--	--	50	0.02	--
Silty SAND	3	7.0	367.0	14.0	360.0	7.0	Sand (Reese)	125	62.6	34	60	--	--	--
GRANITE	4	14.0	360.0	34.0	340.0	20.0	Vuggy Limestone	155	92.6	40	--	--	--	14719
			GW EL	**376.0**		**GW Depth**	**-2.0**							

*layer depths are computed in relation to the bottom of pile cap

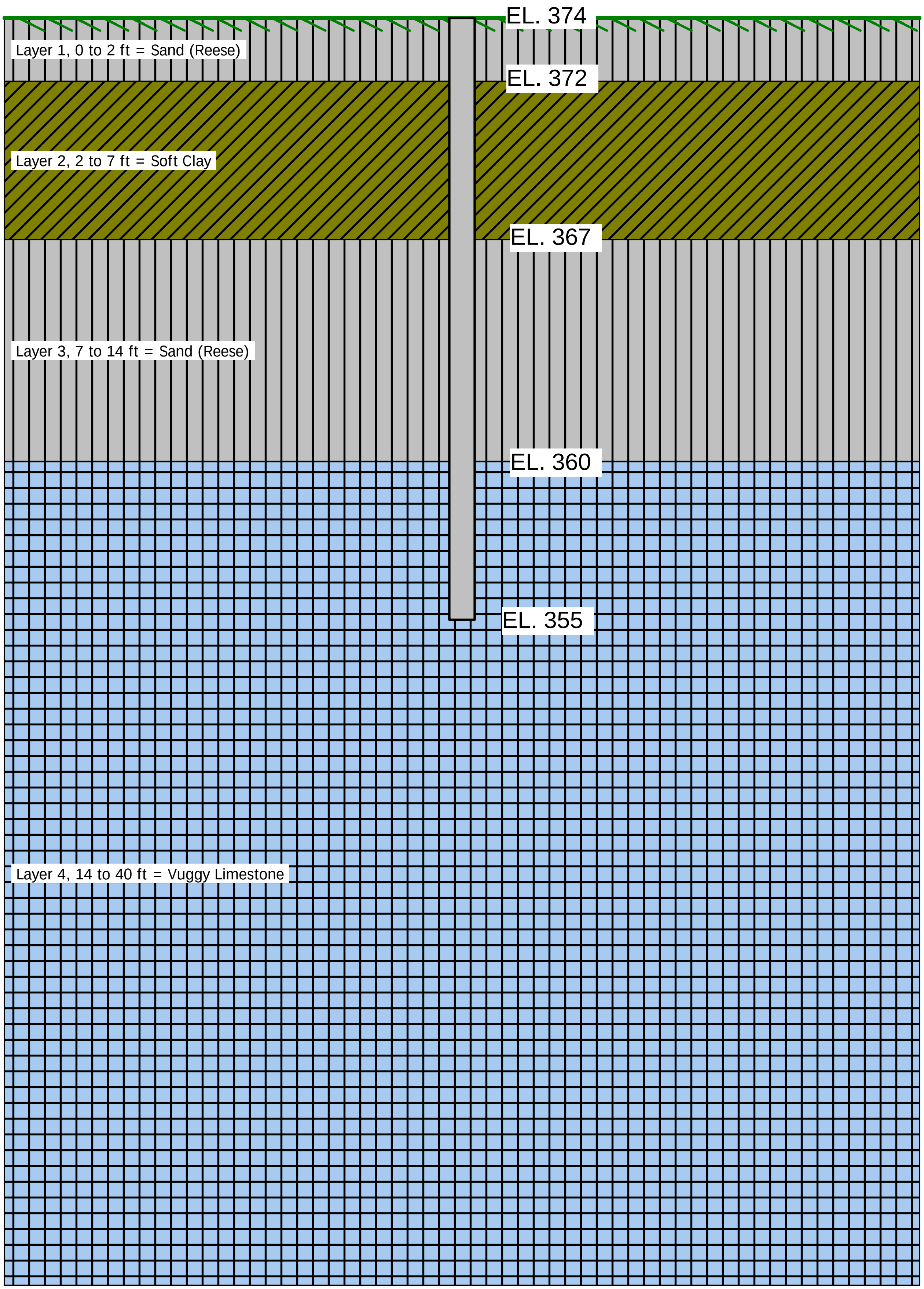
Notes:

1. Assume piles are socketed within rock.

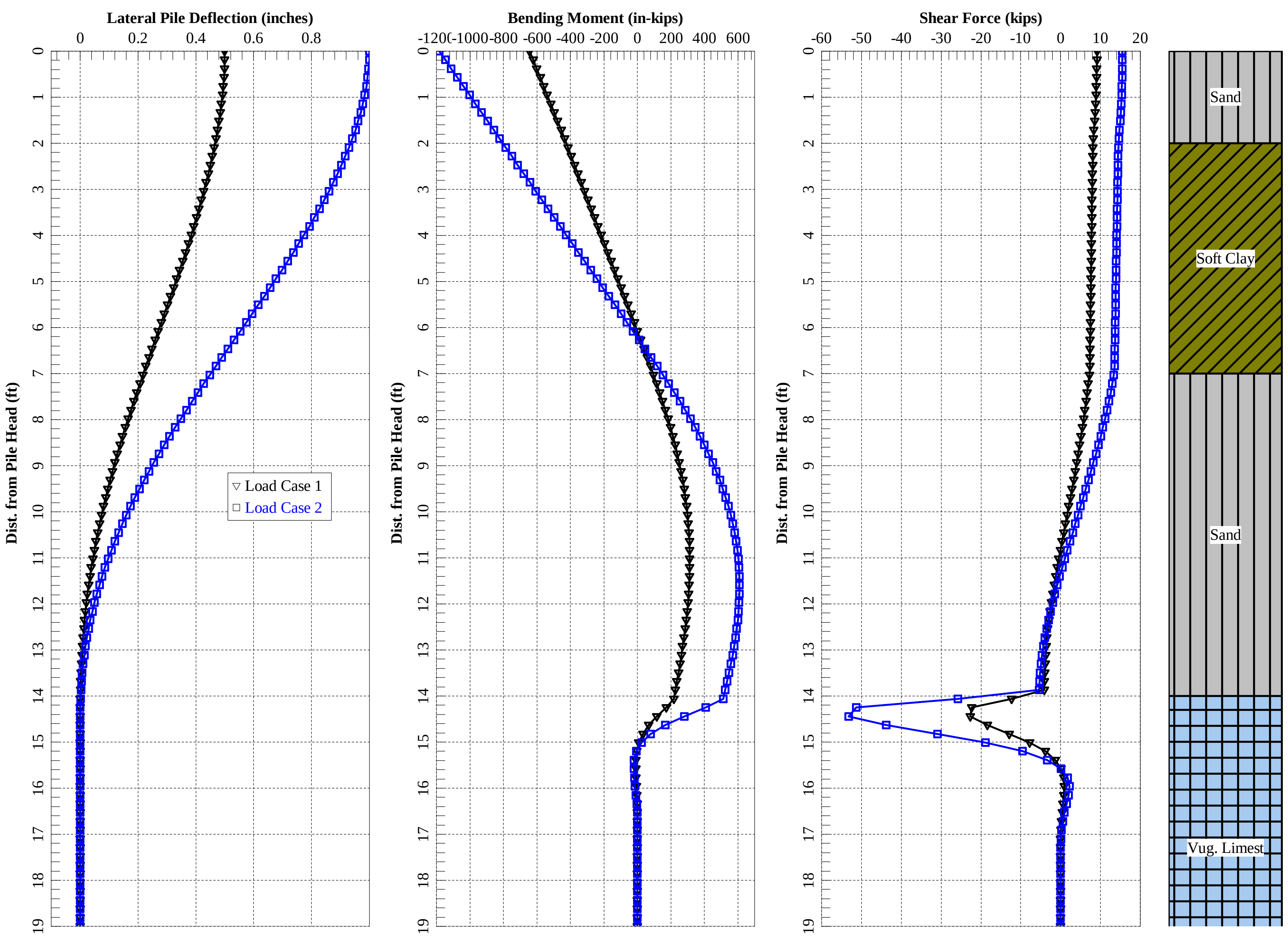
LPILE Model Results

Model	Load Case	Axial Compression (kip)	Axial Tension (kip)	Max Moment (in-kips)	Section Modulus, S (in^3)	Bending Stress at Max Moment (ksi)	Axial Stress at Pile Head (ksi)	Total Stress (ksi)	Depth of Fixity, below Pile Top (ft)
Spun Pipe Pile with Corrosion Loss	0.5-inch	230	0	-642.8	29.3	21.9	16.8	38.7	16.0
	1.0-inch	230	0	-1182.4	29.3	40.3	16.8	57.1	16.0

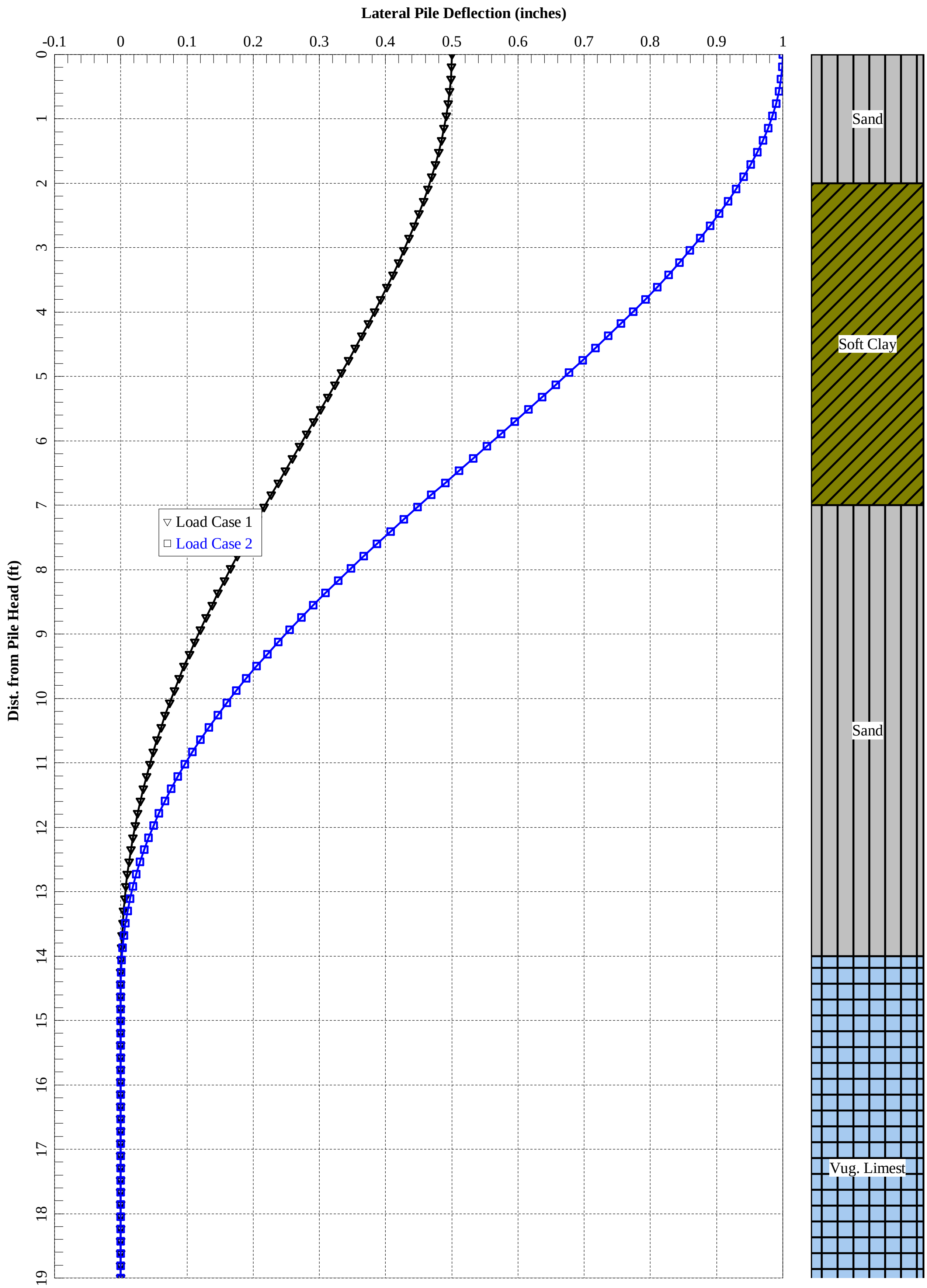
LPILE Soil-Pile Profile: 9.625-in x 0.545-in Spun Pipe Pile



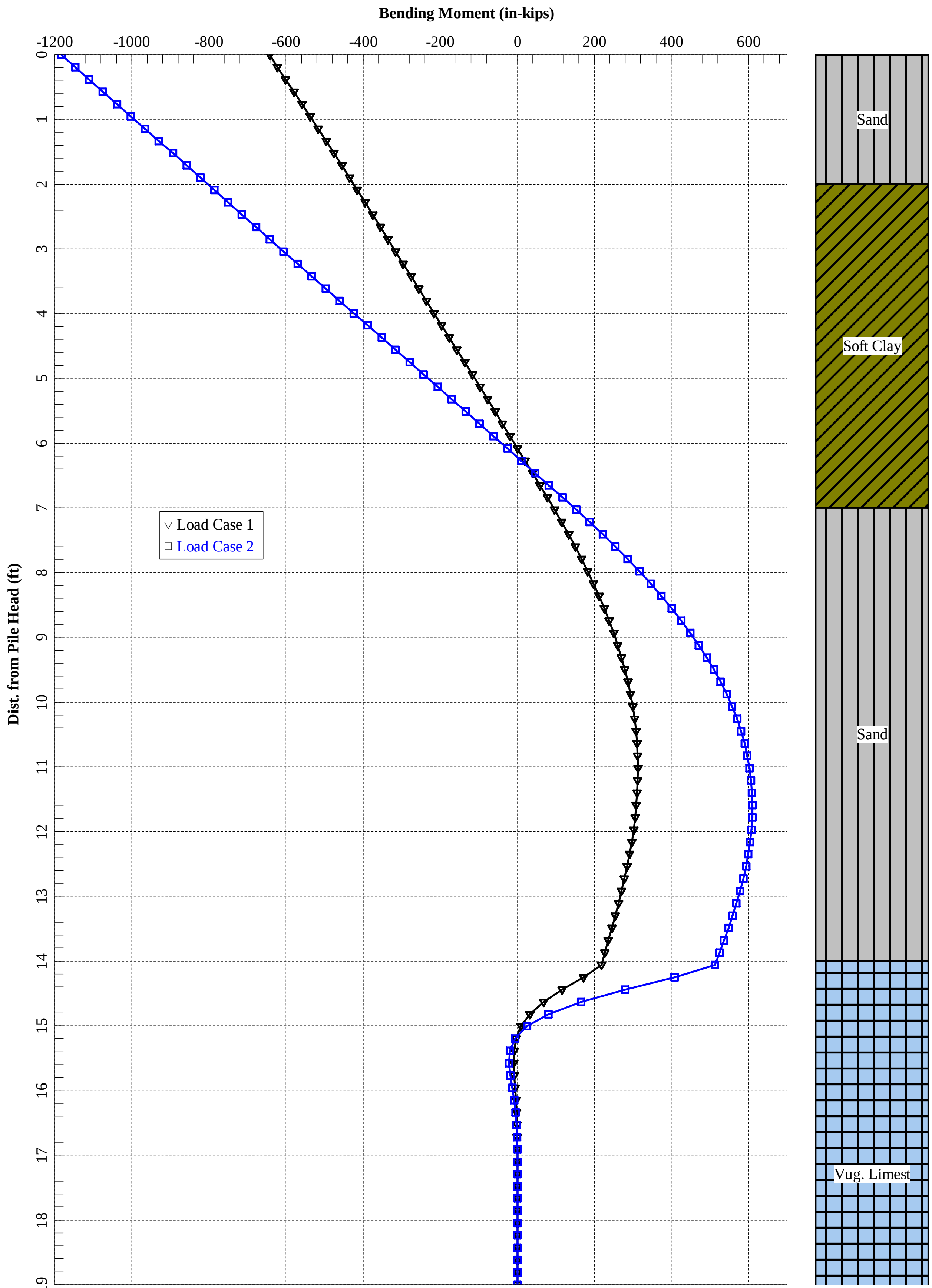
9.625-in x 0.545-in Spun Pipe Pile



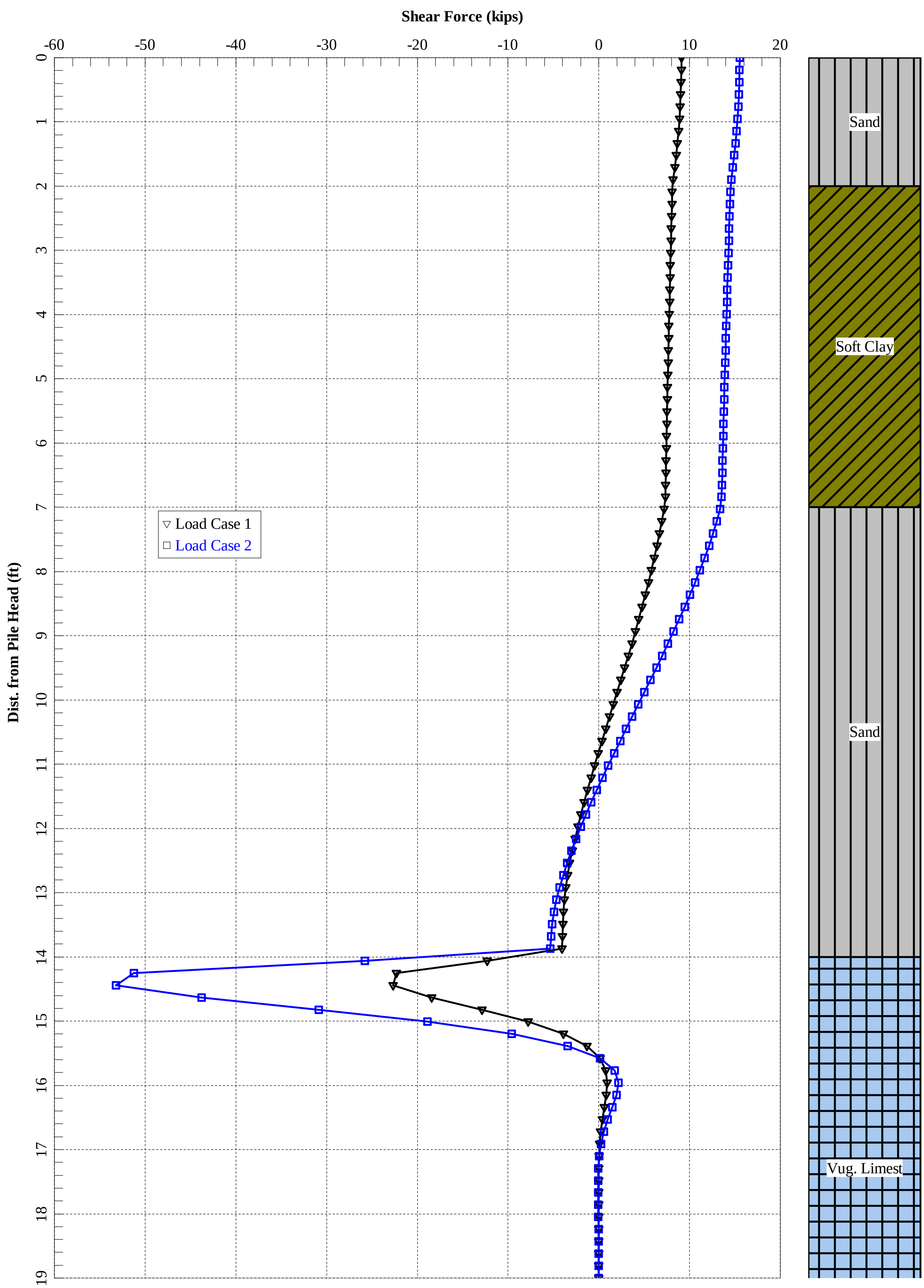
9.625-in x 0.545-in Spun Pipe Pile



9.625-in x 0.545-in Spun Pipe Pile



9.625-in x 0.545-in Spun Pipe Pile



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LPile for Windows, Version 2022-12.002

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

==

This copy of LPile is being used by:

Katie
AECOM

Serial Number of Security Device: 235486653

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Files Used for Analysis

Path to file locations:
\Chelmsford-USCHL1\Secure_DCS\Resources\Legacy\Private\AE_Depts\Geot\PROJECT FILES (GEOTECH)\Tannery Brook Bridge Norway ME - 60656153\Calculations\Spun Pipe Pile\LPILE\

Name of input data file:
Tannery Brook Spun Pipe Pile_R1 Corrosion.lp12d

Name of output report file:
Tannery Brook Spun Pipe Pile_R1 Corrosion.lp12o

Name of plot output file:
Tannery Brook Spun Pipe Pile_R1 Corrosion.lp12p

Name of runtime message file:
Tannery Brook Spun Pipe Pile_R1 Corrosion.lp12r

Date and Time of Analysis

Date: September 15, 2023   Time: 13:22:04

Problem Title

Project Name: Tannery Brook Bridge Replacement

Job Number: 60656153

Client: TY Linn

Engineer: B Reyes

Description: Lateral Analysis of Micropiles

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

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

Loading Type and Number of Cycles of Loading:
 - Static loading specified

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

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

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 19.000 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.

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	9.5000
2	19.000	9.5000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

```
   Section 1 is a drilled shaft with permanent casing
   Length of section                                   =    19.000000 ft
   Casing outside diameter                             =     9.500000 in
```

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

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

```
   Distance from top of pile to top of layer           =       0.0000 ft
   Distance from top of pile to bottom of layer        =     2.000000 ft
   Effective unit weight at top of layer               =    57.600000 pcf
   Effective unit weight at bottom of layer            =    57.600000 pcf
   Friction angle at top of layer                      =    32.000000 deg.
   Friction angle at bottom of layer                   =    32.000000 deg.
   Subgrade k at top of layer                          =    27.000000 pci
   Subgrade k at bottom of layer                       =    27.000000 pci
```

Layer 2 is soft clay, p-y criteria by Matlock, 1970

```
   Distance from top of pile to top of layer           =     2.000000 ft
   Distance from top of pile to bottom of layer        =     7.000000 ft
   Effective unit weight at top of layer               =    47.600000 pcf
   Effective unit weight at bottom of layer            =    47.600000 pcf
   Undrained cohesion at top of layer                  =    50.000000 psf
   Undrained cohesion at bottom of layer               =    50.000000 psf
   Epsilon-50 at top of layer                          =     0.020000
   Epsilon-50 at bottom of layer                       =     0.020000
```

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

```
   Distance from top of pile to top of layer           =     7.000000 ft
   Distance from top of pile to bottom of layer        =    14.000000 ft
   Effective unit weight at top of layer               =    62.600000 pcf
   Effective unit weight at bottom of layer            =    62.600000 pcf
   Friction angle at top of layer                      =    34.000000 deg.
   Friction angle at bottom of layer                   =    34.000000 deg.
   Subgrade k at top of layer                          =    60.000000 pci
   Subgrade k at bottom of layer                       =    60.000000 pci
```

Layer 4 is strong rock (vuggy limestone)

```
   Distance from top of pile to top of layer           =    14.000000 ft
   Distance from top of pile to bottom of layer        =    40.000000 ft
   Effective unit weight at top of layer               =    92.600000 pcf
   Effective unit weight at bottom of layer            =    92.600000 pcf
   Uniaxial compressive strength at top of layer       =       14719. psi
   Uniaxial compressive strength at bottom of layer    =       14719. psi
```

(Depth of the lowest soil layer extends 21.000 ft below the pile tip)

Summary of Input Soil Properties

Layer	Soil Type	Layer	Effective	Cohesion	Angle of	Uniaxial	E50	
Num.	Name	Depth	Unit Wt.		Friction	qu	or	kpy

	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	57.6000	--	32.0000	--	--	27.0000
	(Reese, et al.)	2.0000	57.6000	--	32.0000	--	--	27.0000
2	Soft	2.0000	47.6000	50.0000	--	--	0.02000	--
	Clay	7.0000	47.6000	50.0000	--	--	0.02000	--
3	Sand	7.0000	62.6000	--	34.0000	--	--	60.0000
	(Reese, et al.)	14.0000	62.6000	--	34.0000	--	--	60.0000
4	Strong Rock	14.0000	92.6000	--	--	14719.	--	--
	(Vuggy Limestone)	40.0000	92.6000	--	--	14719.	--	--

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.500000 in	S = 0.0000 in/in	230000.	N.A.	Yes
2	5	y = 1.000000 in	S = 0.0000 in/in	230000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

----

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

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Drilled Shaft (Bored Pile) with Permanent Casing:

```
Length of Section                                  =    19.000000 ft
Outer Diameter of Casing                           =     9.500000 in
Casing Wall Thickness                              =     0.482500 in
```

```
Moment of Inertia of Steel Casing                      =   139.334052 in^4
Yield Stress of Casing                                 =       80000. psi
Elastic Modulus of Casing                              =    29000000. psi
Number of Reinforcing Bars                             =            0 bars
Area of Single Reinforcing Bar                         =       0.0000 sq. in.
Offset of Center of Rebar Cage from Center of Pile     =       0.0000 in
Yield Stress of Reinforcing Bars                       =       0.0000 psi
Modulus of Elasticity of Reinforcing Bars              =       0.0000 psi
Gross Area of Pile                                     =    70.882184 sq. in.
Area of Concrete                                       =    57.213291 sq. in.
Cross-sectional Area of Steel Casing                   =    13.668893 sq. in.
Area of All Steel (Casing and Bars)                    =    13.668893 sq. in.
Area Ratio of All Steel to Gross Area of Pile          =        19.28 percent
```

Axial Structural Capacities:

```
Nom. Axial Structural Capacity = 0.85 Fc Ac + Fy As    =     1288.037 kips
Tensile Load for Cracking of Concrete                  =      -69.528 kips
Nominal Axial Tensile Capacity                         =    -1093.511 kips
```

Concrete Properties:

```
Compressive Strength of Concrete                       =        4000. psi
Modulus of Elasticity of Concrete                      =     3604997. psi
Modulus of Rupture of Concrete                         =   -474.34165 psi
Compression Strain at Peak Stress                      =     0.001886
Tensile Strain at Fracture of Concrete                 =   -0.0001154
Maximum Coarse Aggregate Size                          =     0.500000 in
```

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

Number	Axial Thrust Force kips
1	230.000

Definitions of Run Messages and Notes:

C = concrete in section has cracked in tension.
Y = stress in reinforcing steel has reached yield stress.
T = ACI 318 criteria for tension-controlled section met, tensile strain in reinforcement exceeds 0.005 while simultaneously compressive strain in concrete more than 0.003. See ACI 318-14, Section 21.2.3.
Z = depth of tensile zone in concrete section is less than 10 percent of section depth.

Bending Stiffness (EI) = Computed Bending Moment / Curvature.
Position of neutral axis is measured from edge of compression side of pile.
Compressive stresses and strains are positive in sign.
Tensile stresses and strains are negative in sign.

Axial Thrust Force =   230.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Comp Strain in/in	Max Tens Strain in/in	Max Conc Stress ksi	Max Steel Stress ksi	Max Casing Stress ksi	Run Msg
0.00000125	6.1583956	4926716.	303.9403763	0.0003799	0.0003681	1.4488544	0.00000	11.0161168	

0.00000250	12.3167877	4926715.	154.3461175	0.0003859	0.0003621	1.4687330	0.00000	11.1866498
0.00000375	18.4751731	4926713.	104.4817775	0.0003918	0.0003562	1.4885390	0.00000	11.3572277
0.00000500	24.6335483	4926710.	79.5499174	0.0003977	0.0003502	1.5082725	0.00000	11.5278505
0.00000625	30.7919098	4926706.	64.5910491	0.0004037	0.0003443	1.5279332	0.00000	11.6985183
0.00000750	36.9502544	4926701.	54.6186767	0.0004096	0.0003384	1.5475213	0.00000	11.8692310
0.00000875	43.1085785	4926695.	47.4957307	0.0004156	0.0003325	1.5670365	0.00000	12.0399886
0.00001000	49.2668788	4926688.	42.1536760	0.0004215	0.0003265	1.5864789	0.00000	12.2107911
0.00001125	55.4251519	4926680.	37.9988823	0.0004275	0.0003206	1.6058484	0.00000	12.3816385
0.00001250	61.5833943	4926672.	34.6751712	0.0004334	0.0003147	1.6251449	0.00000	12.5525308
0.00001375	67.7416028	4926662.	31.9558839	0.0004394	0.0003088	1.6443684	0.00000	12.7234681
0.00001500	73.8997738	4926652.	29.6899144	0.0004453	0.0003028	1.6635187	0.00000	12.8944503
0.00001625	80.0579040	4926640.	27.7726509	0.0004513	0.0002969	1.6825959	0.00000	13.0654774
0.00001750	86.2159899	4926628.	26.1293707	0.0004573	0.0002910	1.7015999	0.00000	13.2365494
0.00001875	92.3740282	4926615.	24.7052771	0.0004632	0.0002851	1.7205306	0.00000	13.4076664
0.00002000	98.5320155	4926601.	23.4592727	0.0004692	0.0002792	1.7393880	0.00000	13.5788282
0.00002125	104.6899483	4926586.	22.3599300	0.0004751	0.0002733	1.7581719	0.00000	13.7500350
0.00002250	110.8478233	4926570.	21.3828053	0.0004811	0.0002674	1.7768824	0.00000	13.9212868
0.00002375	117.0056371	4926553.	20.5086010	0.0004871	0.0002615	1.7955194	0.00000	14.0925834
0.00002500	123.1633864	4926535.	19.7218792	0.0004930	0.0002555	1.8140827	0.00000	14.2639244
0.00002625	129.3210675	4926517.	19.0101422	0.0004990	0.0002496	1.8325724	0.00000	14.4353109
0.00002750	135.4786771	4926497.	18.3631650	0.0005050	0.0002437	1.8509884	0.00000	14.6067423
0.00002875	141.6362118	4926477.	17.7725005	0.0005110	0.0002378	1.8693306	0.00000	14.7782187
0.00003000	147.7936682	4926456.	17.2311097	0.0005169	0.0002319	1.8875990	0.00000	14.9497400
0.00003125	153.9510430	4926433.	16.7330797	0.0005229	0.0002260	1.9057935	0.00000	15.1213062
0.00003250	160.1083327	4926410.	16.2734074	0.0005289	0.0002201	1.9239140	0.00000	15.2929174
0.00003375	166.2655339	4926386.	15.8478309	0.0005349	0.0002142	1.9419605	0.00000	15.4645735
0.00003500	172.4226433	4926361.	15.4526969	0.0005408	0.0002083	1.9599329	0.00000	15.6362745
0.00003625	178.5796573	4926335.	15.0848563	0.0005468	0.0002025	1.9778312	0.00000	15.8080204
0.00003750	184.7365727	4926309.	14.7415797	0.0005528	0.0001966	1.9956552	0.00000	15.9798113
0.00003875	190.8933860	4926281.	14.4204899	0.0005588	0.0001907	2.0134050	0.00000	16.1516472
0.00004000	197.0500937	4926252.	14.1195071	0.0005648	0.0001848	2.0310805	0.00000	16.3235279
0.00004125	203.2066926	4926223.	13.8368031	0.0005708	0.0001789	2.0486815	0.00000	16.4954536
0.00004250	209.3631791	4926192.	13.5707653	0.0005768	0.0001730	2.0662081	0.00000	16.6674243
0.00004375	215.5195499	4926161.	13.3199651	0.0005827	0.0001671	2.0836602	0.00000	16.8394399
0.00004500	221.6758016	4926129.	13.0831327	0.0005887	0.0001612	2.1010377	0.00000	17.0115004
0.00004625	227.8319308	4926096.	12.8591355	0.0005947	0.0001554	2.1183406	0.00000	

								17.1836059
0.00004750	233.9879340	4926062.	12.6469603	0.0006007	0.0001495	2.1355688	0.00000	17.3557563
0.00004875	240.1438079	4926027.	12.4456976	0.0006067	0.0001436	2.1527222	0.00000	17.5279517
0.00005125	252.4551540	4925954.	12.0727161	0.0006187	0.0001319	2.1868044	0.00000	17.8724773
0.00005375	264.7659419	4925878.	11.7345459	0.0006307	0.0001201	2.2205869	0.00000	18.2171827
0.00005625	277.0761443	4925798.	11.4265456	0.0006427	0.0001084	2.2540690	0.00000	18.5620680
0.00005875	289.3857338	4925715.	11.1448636	0.0006548	0.00009664	2.2872504	0.00000	18.9071331
0.00006125	301.6946834	4925627.	10.8862773	0.0006668	0.00008491	2.3201305	0.00000	19.2523781
0.00006375	314.0029656	4925537.	10.6480696	0.0006788	0.00007319	2.3527088	0.00000	19.5978030
0.00006625	326.3105532	4925442.	10.4279335	0.0006909	0.00006148	2.3849848	0.00000	19.9434078
0.00006875	338.6174190	4925344.	10.2238975	0.0007029	0.00004977	2.4169580	0.00000	20.2891925
0.00007125	350.9235356	4925243.	10.0342669	0.0007149	0.00003807	2.4486279	0.00000	20.6351571
0.00007375	363.2288757	4925137.	9.8575768	0.0007270	0.00002637	2.4799941	0.00000	20.9813017
0.00007625	375.5334121	4925028.	9.6925543	0.0007391	0.00001468	2.5110560	0.00000	21.3276263
0.00007875	387.8371175	4924916.	9.5380882	0.0007511	0.00000300	2.5418131	0.00000	21.6741309
0.00008125	400.1399645	4924800.	9.3932042	0.0007632	-0.00000868	2.5722650	0.00000	22.0208155
0.00008375	412.4419258	4924680.	9.2570440	0.0007753	-0.00002035	2.6024111	0.00000	22.3676802
0.00008625	424.7429741	4924556.	9.1288493	0.0007874	-0.00003201	2.6322509	0.00000	22.7147250
0.00008875	437.0430822	4924429.	9.0079467	0.0007995	-0.00004367	2.6617839	0.00000	23.0619498
0.00009125	449.3421679	4924298.	8.8937367	0.0008116	-0.00005532	2.6910097	0.00000	23.4093542
0.00009375	461.6399152	4924159.	8.7856827	0.0008237	-0.00006697	2.7199273	0.00000	23.7569345
0.00009625	473.9358351	4924009.	8.6833030	0.0008358	-0.00007861	2.7485359	0.00000	24.1046853
0.00009875	486.2293906	4923842.	8.5861642	0.0008479	-0.00009024	2.7768344	0.00000	24.4525999
0.0001013	498.5200385	4923655.	8.4938760	0.0008600	-0.000102	2.8048219	0.00000	24.8006718
0.0001038	510.8072470	4923443.	8.4060854	0.0008721	-0.000113	2.8324973	0.00000	25.1488941
0.0001063	522.0125155	4913059.	8.3180268	0.0008838	-0.000126	2.8587899	0.00000	25.4835607 C
0.0001088	533.9328542	4909727.	8.2369011	0.0008958	-0.000137	2.8854853	0.00000	25.8273231 C
0.0001113	545.8023770	4906089.	8.1592515	0.0009077	-0.000149	2.9118231	0.00000	26.1705385 C
0.0001138	557.6246887	4902195.	8.0848601	0.0009197	-0.000161	2.9378073	0.00000	26.5132420 C
0.0001163	569.4038094	4898097.	8.0135311	0.0009316	-0.000173	2.9634431	0.00000	26.8554828 C
0.0001188	581.1331869	4893753.	7.9450376	0.0009435	-0.000185	2.9887233	0.00000	27.1971456 C
0.0001213	592.8127082	4889177.	7.8791971	0.0009554	-0.000197	3.0136475	0.00000	27.5382053 C
0.0001238	604.4587991	4884516.	7.8159189	0.0009672	-0.000208	3.0382356	0.00000	27.8789135 C
0.0001263	616.0645926	4879720.	7.7550181	0.0009791	-0.000220	3.0624800	0.00000	28.2191508 C
0.0001288	627.6234427	4874745.	7.6963204	0.0009909	-0.000232	3.0863726	0.00000	28.5587835 C
0.0001313	639.1598981	4869790.	7.6398084	0.0010027	-0.000244	3.1099436	0.00000	28.8982243 C

0.0001338	650.6465231	4864647.	7.5852332	0.0010145	-0.000256	3.1331601	0.00000	29.2369830 C
0.0001363	662.1098156	4859522.	7.5326029	0.0010263	-0.000268	3.1560545	0.00000	29.5755133 C
0.0001388	673.5373153	4854323.	7.4817558	0.0010381	-0.000280	3.1786122	0.00000	29.9135873 C
0.0001413	684.9356124	4849102.	7.4326224	0.0010499	-0.000292	3.2008413	0.00000	30.2513080 C
0.0001438	696.3056033	4843865.	7.3851170	0.0010616	-0.000304	3.2227436	0.00000	30.5886914 C
0.0001463	707.6472264	4838614.	7.3391518	0.0010734	-0.000316	3.2443191	0.00000	30.9257186 C
0.0001488	718.9614865	4833355.	7.2946534	0.0010851	-0.000328	3.2655697	0.00000	31.2624085 C
0.0001588	763.9769940	4812454.	7.1300355	0.0011319	-0.000376	3.3473674	0.00000	32.6062232 C
0.0001688	808.6628649	4792076.	6.9841498	0.0011786	-0.000425	3.4241259	0.00000	33.9462305 C
0.0001788	853.0300574	4772196.	6.8538190	0.0012251	-0.000473	3.4958771	0.00000	35.2822564 C
0.0001888	897.1526355	4753127.	6.7368279	0.0012716	-0.000522	3.5627317	0.00000	36.6157091 C
0.0001988	941.0540286	4734863.	6.6312228	0.0013180	-0.000570	3.6247327	0.00000	37.9469329 C
0.0002088	984.7552194	4717390.	6.5354230	0.0013643	-0.000619	3.6819177	0.00000	39.2762646 C
0.0002188	1028.	4700692.	6.4481424	0.0014105	-0.000668	3.7343209	0.00000	40.6040758 C
0.0002288	1072.	4684750.	6.3683247	0.0014568	-0.000716	3.7819724	0.00000	41.9307715 C
0.0002388	1115.	4669544.	6.2950953	0.0015030	-0.000765	3.8248974	0.00000	43.2567886 C
0.0002488	1158.	4655024.	6.2276915	0.0015491	-0.000814	3.8631093	0.00000	44.5823575 C
0.0002588	1201.	4641057.	6.1653561	0.0015953	-0.000863	3.8965985	0.00000	45.9068633 C
0.0002688	1244.	4627744.	6.1077115	0.0016414	-0.000912	3.9254055	0.00000	47.2317739 C
0.0002788	1286.	4614956.	6.0541774	0.0016876	-0.000961	3.9495161	0.00000	48.5564796 C
0.0002888	1329.	4602675.	6.0043618	0.0017338	-0.001009	3.9689362	0.00000	49.8812725 C
0.0002988	1372.	4590881.	5.9579353	0.0017799	-0.001058	3.9836684	0.00000	51.2065349 C
0.0003088	1414.	4579516.	5.9145451	0.0018261	-0.001107	3.9937065	0.00000	52.5320562 C
0.0003188	1456.	4568554.	5.8739249	0.0018723	-0.001156	3.9990481	0.00000	53.8580165 C
0.0003288	1498.	4558009.	5.8359019	0.0019186	-0.001205	3.9997615	0.00000	55.1851772 C
0.0003388	1541.	4547751.	5.8000890	0.0019648	-0.001253	3.9999958	0.00000	56.5119972 C
0.0003488	1583.	4537869.	5.7664911	0.0020111	-0.001302	3.9993363	0.00000	57.8404468 C
0.0003588	1625.	4528286.	5.7348367	0.0020574	-0.001351	3.9996997	0.00000	59.1696305 C
0.0003688	1666.	4518945.	5.7049328	0.0021037	-0.001399	3.9998812	0.00000	60.4991732 C
0.0003788	1708.	4509881.	5.6767682	0.0021501	-0.001448	3.9999588	0.00000	61.8304755 C
0.0003888	1750.	4501063.	5.6501941	0.0021965	-0.001497	3.9999848	0.00000	63.1633737 C
0.0003988	1791.	4492406.	5.6249427	0.0022429	-0.001545	3.9999885	0.00000	64.4961538 C
0.0004088	1833.	4483984.	5.6010711	0.0022894	-0.001594	3.9999766	0.00000	65.8306443 C
0.0004188	1874.	4475788.	5.5784763	0.0023360	-0.001642	3.9999322	0.00000	67.1667851 C
0.0004288	1916.	4467810.	5.5570614	0.0023826	-0.001691	3.9998152	0.00000	68.5045099 C
0.0004388	1957.	4459985.	5.5366001	0.0024292	-0.001739	3.9995599	0.00000	

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69.8419327 C
    0.0004488        1998.      4452373.     5.5171635     0.0024758     -0.001787     3.9990806       0.00000
71.1808306 C
    0.0004588        2039.      4444968.     5.4986794     0.0025225     -0.001836     3.9986455       0.00000
72.5211261 C
    0.0004688        2080.      4437758.     5.4810869     0.0025693     -0.001884     3.9999410       0.00000
73.8628205 C
    0.0004788        2121.      4430740.     5.4643251     0.0026160     -0.001932     3.9995199       0.00000
75.2058444 C
    0.0004888        2162.      4423877.     5.4482305     0.0026628     -0.001980     3.9985434       0.00000
76.5486037 C
    0.0004988        2203.      4417194.     5.4328511     0.0027096     -0.002028     3.9999621       0.00000
77.8923726 C
    0.0005088        2244.      4410684.     5.4181584     0.0027565     -0.002077     3.9993572       0.00000
79.2373524 C
    0.0005188        2284.      4403724.     5.4043965     0.0028035     -0.002125     3.9982578       0.00000
80.0000000 CY
    0.0005288        2324.      4395198.     5.3920607     0.0028511     -0.002172     3.9997561       0.00000
80.0000000 CY
    0.0005388        2362.      4384456.     5.3814423     0.0028993     -0.002219     3.9985240       0.00000
80.0000000 CY
    0.0005488        2399.      4371170.     5.3726906     0.0029483     -0.002265     3.9999496       0.00000
80.0000000 CY
    0.0006088        2581.      4240033.     5.3581912     0.0032618     -0.002521     3.9980789       0.00000
80.0000000 CY
    0.0006688        2725.      4074579.     5.3799585     0.0035978     -0.002755     3.9985806       0.00000
80.0000000 CY
    0.0007288        2839.      3895040.     5.4127719     0.0039446     -0.002979     3.9968102       0.00000
80.0000000 CY
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Summary of Results for Nominal Moment Capacity for Section 1

Moment values interpolated at maximum compressive strain = 0.003
or maximum developed moment if pile fails at smaller strains.

Load No.	Axial Thrust kips	Nominal Mom. Cap. in-kip	Max. Comp. Strain	Max. Tens. Strain
1	230.000	2428.784	0.00300000	-0.00230718

Note that the values of moment capacity in the table above are not
factored by a strength reduction factor (phi-factor).

In ACI 318, the value of the strength reduction factor depends on whether
the transverse reinforcing steel bars are tied hoops (0.65) or spirals (0.75).

The above values should be multiplied by the appropriate strength reduction
factor to compute ultimate moment capacity according to ACI 318,
or the value required by the design standard being followed.

The following table presents factored moment capacities and corresponding
bending stiffnesses computed for common resistance factor values used for
reinforced concrete sections.

Axial Load No.	Resist. Factor	Nominal Ax. Thrust kips	Nominal Moment Cap in-kips	Ult. (Fac) Ax. Thrust kips	Ult. (Fac) Moment Cap in-kips	Bend. Stiff. at Ult Mom kip-in^2
1	0.65	230.000000	2429.	149.500000	1579.	4538780.
1	0.75	230.000000	2429.	172.500000	1822.	4486266.
1	0.90	230.000000	2429.	207.000000	2186.	4419999.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	987.4977
2	2.0000	3.8933	No	No	987.4977	1781.
3	7.0000	3.3614	No	No	2769.	53164.
4	14.0000	14.0000	No	Yes	N.A.	N.A.

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

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 0.500000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 230000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.5000	-642802.	9112.	0.00	0.00	4.87E+09	0.00	0.00	0.00
0.1900	0.4997	-621968.	9095.	-2.96E-04	0.00	4.87E+09	-6.717	30.6505	0.00
0.3800	0.4986	-601017.	9071.	-5.82E-04	0.00	4.89E+09	-14.558	66.5640	0.00
0.5700	0.4970	-579994.	9028.	-8.57E-04	0.00	4.89E+09	-23.037	105.6820	0.00
0.7600	0.4947	-558949.	8966.	-0.00112	0.00	4.90E+09	-31.745	146.2961	0.00
0.9500	0.4919	-537933.	8883.	-0.00138	0.00	4.91E+09	-40.430	187.4009	0.00
1.1400	0.4885	-516996.	8782.	-0.00162	0.00	4.92E+09	-48.652	227.0964	0.00
1.3300	0.4845	-496187.	8662.	-0.00186	0.00	4.92E+09	-56.854	267.5568	0.00
1.5200	0.4800	-475552.	8523.	-0.00208	0.00	4.92E+09	-64.535	306.5480	0.00
1.7100	0.4750	-455137.	8368.	-0.00230	0.00	4.92E+09	-71.641	343.8808	0.00
1.9000	0.4695	-434984.	8198.	-0.00250	0.00	4.92E+09	-77.838	377.9871	0.00
2.0900	0.4636	-415131.	8092.	-0.00270	0.00	4.92E+09	-14.724	72.4157	0.00
2.2800	0.4572	-395253.	8059.	-0.00289	0.00	4.92E+09	-14.656	73.0874	0.00
2.4700	0.4504	-375355.	8025.	-0.00307	0.00	4.93E+09	-14.583	73.8205	0.00
2.6600	0.4432	-355442.	7992.	-0.00324	0.00	4.93E+09	-14.505	74.6166	0.00
2.8500	0.4357	-335518.	7959.	-0.00340	0.00	4.93E+09	-14.422	75.4778	0.00
3.0400	0.4277	-315588.	7926.	-0.00355	0.00	4.93E+09	-14.334	76.4065	0.00
3.2300	0.4195	-295655.	7894.	-0.00369	0.00	4.93E+09	-14.241	77.4052	0.00
3.4200	0.4109	-275725.	7861.	-0.00382	0.00	4.93E+09	-14.144	78.4770	0.00
3.6100	0.4021	-255801.	7829.	-0.00394	0.00	4.93E+09	-14.042	79.6250	0.00
3.8000	0.3929	-235888.	7797.	-0.00406	0.00	4.93E+09	-13.934	80.8529	0.00
3.9900	0.3836	-215991.	7766.	-0.00416	0.00	4.93E+09	-13.823	82.1648	0.00
4.1800	0.3740	-196113.	7734.	-0.00426	0.00	4.93E+09	-13.706	83.5652	0.00
4.3700	0.3642	-176258.	7703.	-0.00434	0.00	4.93E+09	-13.586	85.0591	0.00
4.5600	0.3542	-156432.	7672.	-0.00442	0.00	4.93E+09	-13.460	86.6519	0.00
4.7500	0.3440	-136637.	7642.	-0.00449	0.00	4.93E+09	-13.330	88.3496	0.00
4.9400	0.3337	-116879.	7612.	-0.00455	0.00	4.93E+09	-13.196	90.1590	0.00
5.1300	0.3233	-97160.	7582.	-0.00460	0.00	4.93E+09	-13.057	92.0874	0.00
5.3200	0.3127	-77486.	7552.	-0.00464	0.00	4.93E+09	-12.914	94.1429	0.00
5.5100	0.3021	-57861.	7523.	-0.00467	0.00	4.93E+09	-12.766	96.3345	0.00
5.7000	0.2915	-38288.	7494.	-0.00469	0.00	4.93E+09	-12.614	98.6719	0.00
5.8900	0.2808	-18770.	7465.	-0.00470	0.00	4.93E+09	-12.457	101.1663	0.00
6.0800	0.2700	686.3319	7437.	-0.00471	0.00	4.93E+09	-12.297	103.8296	0.00
6.2700	0.2593	20079.	7409.	-0.00470	0.00	4.93E+09	-12.131	106.6751	0.00
6.4600	0.2486	39404.	7382.	-0.00469	0.00	4.93E+09	-11.962	109.7178	0.00

6.6500	0.2379	58657.	7355.	-0.00467	0.00	4.93E+09	-11.788	112.9740	0.00
6.8400	0.2273	77834.	7328.	-0.00463	0.00	4.93E+09	-11.611	116.4618	0.00
7.0300	0.2168	96933.	7193.	-0.00459	0.00	4.93E+09	-106.406	1119.	0.00
7.2200	0.2064	115454.	6943.	-0.00454	0.00	4.93E+09	-113.122	1250.	0.00
7.4100	0.1961	133360.	6678.	-0.00449	0.00	4.93E+09	-119.439	1389.	0.00
7.6000	0.1859	150612.	6399.	-0.00442	0.00	4.93E+09	-125.316	1537.	0.00
7.7900	0.1759	167176.	6105.	-0.00435	0.00	4.93E+09	-132.371	1716.	0.00
7.9800	0.1661	183012.	5796.	-0.00427	0.00	4.93E+09	-139.210	1911.	0.00
8.1700	0.1564	198080.	5471.	-0.00418	0.00	4.93E+09	-145.806	2125.	0.00
8.3600	0.1470	212341.	5131.	-0.00408	0.00	4.93E+09	-152.047	2358.	0.00
8.5500	0.1378	225761.	4778.	-0.00398	0.00	4.93E+09	-157.848	2611.	0.00
8.7400	0.1289	238305.	4412.	-0.00387	0.00	4.93E+09	-163.160	2887.	0.00
8.9300	0.1201	249943.	4035.	-0.00376	0.00	4.93E+09	-167.937	3187.	0.00
9.1200	0.1117	260648.	3647.	-0.00364	0.00	4.93E+09	-172.136	3514.	0.00
9.3100	0.1035	270394.	3250.	-0.00352	0.00	4.93E+09	-175.717	3870.	0.00
9.5000	0.09565	279161.	2846.	-0.00339	0.00	4.93E+09	-178.641	4258.	0.00
9.6900	0.08806	286932.	2436.	-0.00326	0.00	4.93E+09	-180.874	4683.	0.00
9.8800	0.08077	293693.	2022.	-0.00313	0.00	4.93E+09	-182.384	5148.	0.00
10.0700	0.07379	299435.	1606.	-0.00299	0.00	4.93E+09	-183.143	5658.	0.00
10.2600	0.06713	304152.	1188.	-0.00285	0.00	4.93E+09	-183.125	6219.	0.00
10.4500	0.06079	307843.	771.5045	-0.00271	0.00	4.93E+09	-182.310	6837.	0.00
10.6400	0.05478	310512.	357.6939	-0.00257	0.00	4.93E+09	-180.681	7520.	0.00
10.8300	0.04909	312166.	-51.459	-0.00242	0.00	4.93E+09	-178.225	8278.	0.00
11.0200	0.04373	312817.	-454.059	-0.00228	0.00	4.93E+09	-174.933	9120.	0.00
11.2100	0.03870	312484.	-848.196	-0.00213	0.00	4.93E+09	-170.801	10061.	0.00
11.4000	0.03401	311187.	-1232.	-0.00199	0.00	4.93E+09	-165.829	11118.	0.00
11.5900	0.02964	308952.	-1603.	-0.00185	0.00	4.93E+09	-160.024	12311.	0.00
11.7800	0.02559	305810.	-1961.	-0.00170	0.00	4.93E+09	-153.395	13665.	0.00
11.9700	0.02187	301797.	-2302.	-0.00156	0.00	4.93E+09	-145.959	15215.	0.00
12.1600	0.01847	296951.	-2625.	-0.00142	0.00	4.93E+09	-137.737	17003.	0.00
12.3500	0.01538	291318.	-2929.	-0.00129	0.00	4.93E+09	-128.757	19086.	0.00
12.5400	0.01260	284944.	-3206.	-0.00115	0.00	4.93E+09	-113.759	20586.	0.00
12.7300	0.01012	277910.	-3441.	-0.00102	0.00	4.93E+09	-92.745	20898.	0.00
12.9200	0.00793	270327.	-3631.	-8.97E-04	0.00	4.93E+09	-73.780	21209.	0.00
13.1100	0.00603	262294.	-3780.	-7.74E-04	0.00	4.93E+09	-56.910	21521.	0.00
13.3000	0.00440	253902.	-3893.	-6.54E-04	0.00	4.93E+09	-42.169	21833.	0.00
13.4900	0.00305	245229.	-3975.	-5.39E-04	0.00	4.93E+09	-29.587	22145.	0.00
13.6800	0.00195	236342.	-4030.	-4.27E-04	0.00	4.93E+09	-19.182	22457.	0.00
13.8700	0.00110	227299.	-4065.	-3.20E-04	0.00	4.93E+09	-10.967	22769.	0.00
14.0600	4.89E-04	218143.	-12279.	-2.17E-04	0.00	4.93E+09	-7195.	3.36E+07	0.00
14.2500	1.10E-04	171532.	-22321.	-1.27E-04	0.00	4.93E+09	-1613.	3.36E+07	0.00
14.4400	-8.86E-05	116491.	-22674.	-6.00E-05	0.00	4.93E+09	1304.	3.36E+07	0.00
14.6300	-1.64E-04	68201.	-18438.	-1.72E-05	0.00	4.93E+09	2412.	3.36E+07	0.00
14.8200	-1.67E-04	32432.	-12883.	6.05E-06	0.00	4.93E+09	2461.	3.36E+07	0.00
15.0100	-1.36E-04	9449.	-7791.	1.57E-05	0.00	4.93E+09	2006.	3.36E+07	0.00
15.2000	-9.54E-05	-3109.	-3903.	1.72E-05	0.00	4.93E+09	1405.	3.36E+07	0.00
15.3900	-5.78E-05	-8365.	-1331.	1.46E-05	0.00	4.93E+09	851.2279	3.36E+07	0.00
15.5800	-2.91E-05	-9194.	127.1691	1.05E-05	0.00	4.93E+09	427.8661	3.36E+07	0.00
15.7700	-1.00E-05	-7796.	782.8446	6.56E-06	0.00	4.93E+09	147.2879	3.36E+07	0.00
15.9600	8.30E-07	-5631.	936.8295	3.45E-06	0.00	4.93E+09	-12.213	3.36E+07	0.00
16.1500	5.72E-06	-3528.	826.8439	1.33E-06	0.00	4.93E+09	-84.265	3.36E+07	0.00
16.3400	6.90E-06	-1862.	615.0371	8.37E-08	0.00	4.93E+09	-101.530	3.36E+07	0.00
16.5300	6.11E-06	-723.175	396.8286	-5.14E-07	0.00	4.93E+09	-89.881	3.36E+07	0.00
16.7200	4.55E-06	-51.667	217.9842	-6.94E-07	0.00	4.93E+09	-67.000	3.36E+07	0.00
16.9100	2.94E-06	271.5599	92.2224	-6.43E-07	0.00	4.93E+09	-43.317	3.36E+07	0.00
17.1000	1.62E-06	369.5413	15.6501	-4.95E-07	0.00	4.93E+09	-23.852	3.36E+07	0.00
17.2900	6.88E-07	343.4431	-23.083	-3.30E-07	0.00	4.93E+09	-10.125	3.36E+07	0.00
17.4800	1.18E-07	264.6270	-36.601	-1.89E-07	0.00	4.93E+09	-1.733	3.36E+07	0.00
17.6700	-1.73E-07	176.7391	-35.670	-8.67E-08	0.00	4.93E+09	2.5499	3.36E+07	0.00
17.8600	-2.78E-07	102.0638	-28.103	-2.22E-08	0.00	4.93E+09	4.0876	3.36E+07	0.00
18.0500	-2.74E-07	48.6127	-18.837	1.27E-08	0.00	4.93E+09	4.0402	3.36E+07	0.00
18.2400	-2.20E-07	16.1527	-10.540	2.76E-08	0.00	4.93E+09	3.2379	3.36E+07	0.00
18.4300	-1.48E-07	0.5204	-4.359	3.15E-08	0.00	4.93E+09	2.1846	3.36E+07	0.00
18.6200	-7.63E-08	-3.755	-0.587	3.08E-08	0.00	4.93E+09	1.1233	3.36E+07	0.00
18.8100	-8.17E-09	-2.191	0.8303	2.94E-08	0.00	4.93E+09	0.1203	3.36E+07	0.00
19.0000	5.77E-08	0.00	0.00	2.89E-08	0.00	4.93E+09	-0.849	1.68E+07	0.00

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

polated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     0.50000000 inches
Computed slope at pile head      =       0.000000 radians
Maximum bending moment           =       -642802. inch-lbs
Maximum shear force              =        -22674. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =    14.44000000 feet below pile head
Number of iterations             =              7
Number of zero deflection points =              4
```

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

```
Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head                          =     1.000000 inches
Rotation of pile head                              =    0.000E+00 radians
Axial load on pile head                            =     230000.0 lbs
```

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	1.0000	-1182394.	15523.	0.00	0.00	4.65E+09	0.00	0.00	0.00
0.1900	0.9993	-1146872.	15505.	-5.71E-04	0.00	4.65E+09	-6.717	15.3248	0.00
0.3800	0.9974	-1111090.	15481.	-0.00112	0.00	4.67E+09	-14.558	33.2788	0.00
0.5700	0.9942	-1075099.	15438.	-0.00166	0.00	4.68E+09	-23.037	52.8299	0.00
0.7600	0.9898	-1038953.	15376.	-0.00217	0.00	4.70E+09	-31.745	73.1215	0.00
0.9500	0.9843	-1002708.	15294.	-0.00267	0.00	4.71E+09	-40.430	93.6483	0.00
1.1400	0.9777	-966419.	15192.	-0.00314	0.00	4.72E+09	-48.652	113.4591	0.00
1.3300	0.9700	-930138.	15072.	-0.00360	0.00	4.74E+09	-56.854	133.6381	0.00
1.5200	0.9613	-893917.	14933.	-0.00404	0.00	4.75E+09	-64.535	153.0676	0.00
1.7100	0.9516	-857808.	14778.	-0.00446	0.00	4.77E+09	-71.641	171.6515	0.00
1.9000	0.9410	-821856.	14608.	-0.00486	0.00	4.79E+09	-77.838	188.6061	0.00
2.0900	0.9294	-786103.	14498.	-0.00524	0.00	4.80E+09	-18.566	45.5443	0.00
2.2800	0.9171	-750251.	14456.	-0.00560	0.00	4.82E+09	-18.483	45.9529	0.00
2.4700	0.9039	-714309.	14414.	-0.00595	0.00	4.84E+09	-18.394	46.3985	0.00
2.6600	0.8899	-678285.	14372.	-0.00628	0.00	4.85E+09	-18.299	46.8821	0.00
2.8500	0.8753	-642190.	14330.	-0.00659	0.00	4.87E+09	-18.198	47.4046	0.00
3.0400	0.8599	-606032.	14289.	-0.00688	0.00	4.88E+09	-18.091	47.9675	0.00
3.2300	0.8439	-569819.	14248.	-0.00715	0.00	4.90E+09	-17.978	48.5721	0.00
3.4200	0.8273	-533561.	14207.	-0.00741	0.00	4.91E+09	-17.859	49.2200	0.00
3.6100	0.8101	-497266.	14166.	-0.00765	0.00	4.92E+09	-17.735	49.9133	0.00
3.8000	0.7924	-460942.	14126.	-0.00787	0.00	4.92E+09	-17.605	50.6538	0.00
3.9900	0.7742	-424598.	14086.	-0.00807	0.00	4.92E+09	-17.469	51.4439	0.00
4.1800	0.7556	-388241.	14046.	-0.00826	0.00	4.92E+09	-17.328	52.2863	0.00
4.3700	0.7366	-351880.	14007.	-0.00843	0.00	4.93E+09	-17.181	53.1838	0.00
4.5600	0.7171	-315524.	13968.	-0.00859	0.00	4.93E+09	-17.029	54.1394	0.00
4.7500	0.6974	-279179.	13929.	-0.00873	0.00	4.93E+09	-16.871	55.1568	0.00
4.9400	0.6773	-242854.	13891.	-0.00885	0.00	4.93E+09	-16.708	56.2396	0.00
5.1300	0.6570	-206557.	13853.	-0.00895	0.00	4.93E+09	-16.539	57.3921	0.00
5.3200	0.6365	-170295.	13816.	-0.00904	0.00	4.93E+09	-16.365	58.6189	0.00
5.5100	0.6158	-134078.	13779.	-0.00911	0.00	4.93E+09	-16.186	59.9250	0.00
5.7000	0.5950	-97912.	13742.	-0.00916	0.00	4.93E+09	-16.001	61.3162	0.00
5.8900	0.5740	-61806.	13706.	-0.00920	0.00	4.93E+09	-15.811	62.7986	0.00
6.0800	0.5530	-25766.	13670.	-0.00922	0.00	4.93E+09	-15.616	64.3789	0.00
6.2700	0.5320	10198.	13634.	-0.00922	0.00	4.93E+09	-15.415	66.0649	0.00
6.4600	0.5110	46080.	13599.	-0.00921	0.00	4.93E+09	-15.210	67.8648	0.00
6.6500	0.4900	81871.	13565.	-0.00918	0.00	4.93E+09	-14.998	69.7880	0.00
6.8400	0.4691	117565.	13531.	-0.00913	0.00	4.93E+09	-14.782	71.8446	0.00
7.0300	0.4484	153153.	13342.	-0.00907	0.00	4.93E+09	-150.671	766.2066	0.00
7.2200	0.4277	187921.	12981.	-0.00899	0.00	4.93E+09	-166.230	886.0430	0.00
7.4100	0.4073	221779.	12584.	-0.00890	0.00	4.93E+09	-182.486	1021.	0.00

7.6000	0.3872	254635.	12148.	-0.00879	0.00	4.93E+09	-199.446	1175.	0.00
7.7900	0.3673	286392.	11672.	-0.00866	0.00	4.93E+09	-218.255	1355.	0.00
7.9800	0.3477	316945.	11157.	-0.00852	0.00	4.93E+09	-233.485	1531.	0.00
8.1700	0.3284	346207.	10614.	-0.00837	0.00	4.93E+09	-242.797	1686.	0.00
8.3600	0.3095	374123.	10051.	-0.00820	0.00	4.93E+09	-251.393	1852.	0.00
8.5500	0.2910	400641.	9469.	-0.00802	0.00	4.92E+09	-259.236	2031.	0.00
8.7400	0.2729	425715.	8869.	-0.00783	0.00	4.92E+09	-266.298	2225.	0.00
8.9300	0.2553	449301.	8255.	-0.00763	0.00	4.92E+09	-272.560	2434.	0.00
9.1200	0.2381	471361.	7628.	-0.00742	0.00	4.92E+09	-278.011	2662.	0.00
9.3100	0.2215	491861.	6988.	-0.00719	0.00	4.92E+09	-282.652	2910.	0.00
9.5000	0.2053	510772.	6340.	-0.00696	0.00	4.92E+09	-286.490	3181.	0.00
9.6900	0.1897	528070.	5683.	-0.00672	0.00	4.91E+09	-289.544	3480.	0.00
9.8800	0.1747	543735.	5020.	-0.00647	0.00	4.91E+09	-291.841	3809.	0.00
10.0700	0.1602	557749.	4353.	-0.00622	0.00	4.90E+09	-293.418	4176.	0.00
10.2600	0.1463	570103.	3683.	-0.00595	0.00	4.90E+09	-294.101	4582.	0.00
10.4500	0.1331	580788.	3013.	-0.00569	0.00	4.89E+09	-293.530	5029.	0.00
10.6400	0.1204	589806.	2346.	-0.00541	0.00	4.89E+09	-291.662	5523.	0.00
10.8300	0.1084	597163.	1685.	-0.00514	0.00	4.89E+09	-288.467	6068.	0.00
11.0200	0.09699	602875.	1032.	-0.00486	0.00	4.89E+09	-283.918	6674.	0.00
11.2100	0.08624	606963.	391.6746	-0.00457	0.00	4.88E+09	-277.998	7349.	0.00
11.4000	0.07614	609457.	-233.836	-0.00429	0.00	4.88E+09	-270.695	8106.	0.00
11.5900	0.06668	610395.	-841.112	-0.00400	0.00	4.88E+09	-262.003	8958.	0.00
11.7800	0.05788	609822.	-1427.	-0.00372	0.00	4.88E+09	-251.926	9924.	0.00
11.9700	0.04972	607789.	-1988.	-0.00344	0.00	4.88E+09	-240.475	11027.	0.00
12.1600	0.04221	604358.	-2522.	-0.00315	0.00	4.88E+09	-227.671	12296.	0.00
12.3500	0.03535	599595.	-3025.	-0.00287	0.00	4.89E+09	-213.542	13773.	0.00
12.5400	0.02912	593575.	-3494.	-0.00259	0.00	4.89E+09	-198.131	15512.	0.00
12.7300	0.02352	586380.	-3927.	-0.00232	0.00	4.89E+09	-181.492	17590.	0.00
12.9200	0.01855	578099.	-4321.	-0.00205	0.00	4.89E+09	-163.693	20118.	0.00
13.1100	0.01419	568825.	-4660.	-0.00178	0.00	4.90E+09	-133.960	21521.	0.00
13.3000	0.01044	558716.	-4927.	-0.00152	0.00	4.90E+09	-99.936	21833.	0.00
13.4900	0.00727	547951.	-5121.	-0.00126	0.00	4.91E+09	-70.639	22145.	0.00
13.6800	0.00469	536686.	-5254.	-0.00101	0.00	4.91E+09	-46.196	22457.	0.00
13.8700	0.00268	525050.	-5337.	-7.62E-04	0.00	4.91E+09	-26.722	22769.	0.00
14.0600	0.00122	513146.	-25791.	-5.21E-04	0.00	4.92E+09	-17915.	3.36E+07	0.00
14.2500	3.01E-04	407988.	-51257.	-3.08E-04	0.00	4.92E+09	-4424.	3.36E+07	0.00
14.4400	-1.85E-04	279736.	-53189.	-1.48E-04	0.00	4.93E+09	2729.	3.36E+07	0.00
14.6300	-3.76E-04	165601.	-43767.	-4.53E-05	0.00	4.93E+09	5536.	3.36E+07	0.00
14.8200	-3.92E-04	80207.	-30876.	1.15E-05	0.00	4.93E+09	5772.	3.36E+07	0.00
15.0100	-3.23E-04	24796.	-18868.	3.58E-05	0.00	4.93E+09	4761.	3.36E+07	0.00
15.2000	-2.29E-04	-5870.	-9604.	4.02E-05	0.00	4.93E+09	3366.	3.36E+07	0.00
15.3900	-1.40E-04	-19040.	-3417.	3.45E-05	0.00	4.93E+09	2061.	3.36E+07	0.00
15.5800	-7.15E-05	-21489.	132.6856	2.51E-05	0.00	4.93E+09	1053.	3.36E+07	0.00
15.7700	-2.57E-05	-18461.	1763.	1.58E-05	0.00	4.93E+09	377.6865	3.36E+07	0.00
15.9600	7.15E-07	-13465.	2182.	8.45E-06	0.00	4.93E+09	-10.530	3.36E+07	0.00
16.1500	1.29E-05	-8521.	1954.	3.37E-06	0.00	4.93E+09	-189.626	3.36E+07	0.00
16.3400	1.61E-05	-4560.	1468.	3.38E-07	0.00	4.93E+09	-236.388	3.36E+07	0.00
16.5300	1.44E-05	-1827.	956.4125	-1.14E-06	0.00	4.93E+09	-212.329	3.36E+07	0.00
16.7200	1.09E-05	-197.681	532.0800	-1.61E-06	0.00	4.93E+09	-159.892	3.36E+07	0.00
16.9100	7.09E-06	600.7370	230.8044	-1.52E-06	0.00	4.93E+09	-104.385	3.36E+07	0.00
17.1000	3.95E-06	856.3756	45.4495	-1.18E-06	0.00	4.93E+09	-58.207	3.36E+07	0.00
17.2900	1.72E-06	809.2218	-49.783	-7.92E-07	0.00	4.93E+09	-25.330	3.36E+07	0.00
17.4800	3.41E-07	630.1969	-84.382	-4.59E-07	0.00	4.93E+09	-5.020	3.36E+07	0.00
17.6700	-3.74E-07	424.9219	-83.833	-2.15E-07	0.00	4.93E+09	5.5020	3.36E+07	0.00
17.8600	-6.40E-07	248.1456	-66.816	-5.95E-08	0.00	4.93E+09	9.4250	3.36E+07	0.00
18.0500	-6.45E-07	120.3039	-45.248	2.58E-08	0.00	4.93E+09	9.4940	3.36E+07	0.00
18.2400	-5.23E-07	41.7866	-25.653	6.33E-08	0.00	4.93E+09	7.6947	3.36E+07	0.00
18.4300	-3.56E-07	3.2593	-10.900	7.37E-08	0.00	4.93E+09	5.2464	3.36E+07	0.00
18.6200	-1.87E-07	-7.996	-1.787	7.26E-08	0.00	4.93E+09	2.7474	3.36E+07	0.00
18.8100	-2.53E-08	-4.967	1.7695	6.96E-08	0.00	4.93E+09	0.3727	3.36E+07	0.00
19.0000	1.31E-07	0.00	0.00	6.85E-08	0.00	4.93E+09	-1.925	1.68E+07	0.00

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

Output Summary for Load Case No. 2:

```
Pile-head deflection             =     1.00000000 inches
Computed slope at pile head      =       0.000000 radians
Maximum bending moment           =      -1182394. inch-lbs
Maximum shear force              =        -53189. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =    14.44000000 feet below pile head
Number of iterations             =              8
Number of zero deflection points =              4
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

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

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	0.5000	S, rad	0.00	230000.	0.5000	0.00	-22674.	-642802.
2	y, in	1.0000	S, rad	0.00	230000.	1.0000	0.00	-53189.	-1182394.

Maximum pile-head deflection = 1.0000000000 inches
Maximum pile-head rotation   = 0.0000000000 radians = 0.000000 deg.

Summary of Warning Messages

The following warning was reported 2970 times

**** Warning ****

This warning is for an input value for uniaxial compressive strength that
has been specified for a soil defined using the vuggy limestone criteria.
The input value is outside of the range of 1,000 to 2,500 psi
(6,895 to 17,237 kPa) which were used in actual field tests
on which this theory is based. Higher or lower values may be applicable
but the user is warned about the theoretical and testing limitations.

The analysis ended normally.

AECOM Imagine it. Delivered.

JOB TITLE Tannery Brook Bridge Replacement (#3610)
PROJECT/JOB NO. 60690153 CALCULATION NO.
COMPUTED BY B. Reyes DATE 9/15/23
VERIFIED BY K. Scancarello DATE 9/19/23
SCALE N/A SHEET NO. 1 OF 3

- Flexural Stiffness of Casing with Threaded Joint Reduction

  - From FHWA Micropile Manual, Section 5.18.3: Reduce wall thickness by 50% at threaded joints

∴ For 9.625" outer casing with 0.545"-thick wall, ignoring corrosion reduction,

$D = 9.625'' - 0.545'' = 9.08 \text{ in}$

$d = 9.625'' - 2(0.545'') = 8.535 \text{ in}$

Joint moment of inertia, I_{joint}

$$I_{joint} = \frac{\pi}{64}(D^4 - d^4) = \frac{\pi}{64}(9.08 \text{ in}^4 - 8.535 \text{ in}^4) = 73.18 \text{ in}^4$$

From FHWA Micropile Manual, the reduced section modulus (Eq. 5-26)

$$S_{joint} = \frac{I_{joint}}{D/2} = \frac{73.18 \text{ in}^4}{\frac{9.08 \text{ in}}{2}} = 16.12 \text{ in}^3$$

Maximum bending moment at a joint (Eq. 5-27)

$$M_{max\,joint} = S_{joint} \times \left(1 - \frac{f_a}{F_a}\right) \times \left(1 - \frac{f_a}{F_e'}\right) F_b$$

where, f_a = axial stress = P_c / A_{casing}

F_a = allowable axial stress that would be permitted if axial force alone existed $\doteq 0.47 F_{y\,casing}$

F_e' = Euler buckling stress = $\dfrac{\pi^2 E}{FS\,(Kl/r)^2}$

5x5 = 1 in

AECOM Imagine it. Delivered.

JOB TITLE: Tannery Brook Bridge Replacement (#3610)
PROJECT/JOB NO.: 60656153 CALCULATION NO.:
COMPUTED BY: B. Reyes DATE: 9/15/23
VERIFIED BY: K. Scancarello DATE: 9/15/23
SCALE: N/A SHEET NO. 2 OF 3

F_b = allowable bending stress that would be permitted if bending moment alone existed

$= 0.55\ F_y$ casing

So,

Axial stress,

$$f_a = \frac{146\text{ kips}}{15.55\text{ in}^2} = 9.39\text{ ksi}$$

Allowable axial stress,

$$F_a = 0.47\,(80\text{ ksi}) = 37.6\text{ ksi}$$

Euler buckling stress,

$$F_e' = \frac{\pi^2\,(29{,}000\text{ ksi})}{2.12\,(kl/r)^2}$$

since l = unsupported length of the micropile = 0

$$F_e' = \infty$$

Allowable bending stress,

$$F_b = 0.55\,(80\text{ ksi}) = 44\text{ ksi}$$

And so,

$$M_{max\ joint} = 16.12\text{ in}^3\left(1-\frac{9.39\text{ ksi}}{37.6\text{ ksi}}\right)\left(1-\cancelto{0}{\frac{9.39\text{ ksi}}{\infty}}\right)(44\text{ ksi})$$

$$M_{max\ joint} = 16.12\text{ in}^3\,(0.75)\,(1)\,(44\text{ ksi})$$

$$M_{max\ joint} = \underline{\underline{532\text{ kip-in}}}$$

5x5 = 1 in

AECOM Imagine it. Delivered.

JOB TITLE Tannery Brook Bridge Replacement (#3610)
PROJECT/JOB NO. 60656153 CALCULATION NO.
COMPUTED BY B. Reyes DATE 9/15/23
VERIFIED BY K. Scancarello DATE 9/15/23
SCALE N/A

Micropile casing sections are typically 10 feet long, so threaded joint would be 10 feet below the top of the pile.

From LPile results, at a depth of 10 feet

Maximum moment for 0.5 inches of deflection,

$M_{max\,0.5} = 300$ kip-in

$M_{max\,0.5} < M_{max\,joint}$ ✓ OK

Maximum moment for 1 inch of deflection,

$M_{max\,1} = 560$ Kip-in

$M_{max\,1} > M_{max\,joint}$ ✗ Need more steel

5x5 = 1 in

Micropile Calculations

Axial Capacity of Drilled Micropile with Rock Socket based on Micropile Tip Resistance

Assumptions: - Bearing on competent, intact granite

Micropile Diameter (ft)	Area of Micropile Tip, Ap (ft^2)	Uniaxial Rock Strength (psi)
0.71	0.3941	5000

Notes: 1) Tip resistance for micropile founded on competent granite, strength limited to the compressive strength of the grout, 5,000 psi

Nominal End Bearing Resistance (R_p)

$Rp = Ap * q_p$

where,

Ap = Area of pile tip

q_p = Unit tip resistance= 2.5 * Uniaxial Rock Strength (Eq. 10.8.3.5.4c-1)

Rp= 709.3 kips

From AASHTO-9 Table 10.5.5.2.5-1,

For Tip Resistance in Compression on Rock:

φ= 0.5 Assumes no Load Test

For Tip Resistance in Compression on Rock, with Load Test:

φ= 0.7 Assumes Load Test, therefore values in AASHTO-9 Table 10.5.5.2.3-1 apply, but no greater than 0.7

Factored Axial End Bearing, without Load Test

R_{SFC}=	354.7	kips

Factored Axial End Bearing, with Load Test

R_{SFU}=	496.5	kips

AECOM Imagine it. Delivered.

JOB TITLE Tannery Brook Bridge Replacement (#3610)
PROJECT/JOB NO. 60656153 CALCULATION NO.
COMPUTED BY B. Reyes DATE 9/7/23
VERIFIED BY K. Scancarello DATE 9/8/23
SCALE N/A SHEET NO. 1 OF 1

Axial Capacity of Drilled Micropile with Rock Socket based on Micropile Tip Resistance

Assumptions: - Bearing on Competent, Intact Rock

Mp diameter = 0.71 ft

Area of Mp Tip, $A_p = \frac{(0.71\text{ ft})^2}{4}(\pi) = 0.3959\text{ ft}^2$

Uniaxial Rock Strength $= 5{,}000 \frac{\text{lb}}{\text{in}^2}$

Nominal End Bearing Resistance (R_p),

$$R_p = A_p(q_p)$$

$$q_p = 2.5 \times \text{Uniaxial rock strength}$$

$$R_p = (0.3959\text{ ft}^2)\left(\frac{144\text{ in}^2}{1\text{ ft}^2}\right)(2.5)\left(5{,}000\frac{\text{lb}}{\text{in}^2}\right)\left(\frac{1\text{ kip}}{1{,}000\text{ lb}}\right)$$

$$R_p = 712.6\text{ kips}$$

Resistance factors per AASHTO-9,

Without Load Test, $\varphi_1 = 0.5$

With Load Test, $\varphi_2 = 0.7$

Factored Axial End Bearing, without Load Test

$R_{SFC} = \varphi_1 R_p = 0.5(712.6\text{ kips}) = 356\text{ kips}$

Factored Axial End Bearing, with Load Test

$R_{SFU} = \varphi_2 R_p = 0.7(712.6\text{ kips}) = 498\text{ kips}$

5x5 = 1 in

Axial Capacity of Drilled Micropile with Rock Socket and Load Test

Assumptions: Minimum 2-foot plunge length
Neglect 2-foot plunge length contribution to axial resistance
Neglect end bearing
Load test will be performed

Micropile Diameter, d_b (ft)	Length of Rock Socket, L_b (ft)	Nominal Grout-to-Ground Bond Strength, α_b (psi)
0.71	6	200

Notes: 1) Grout-to-Ground Bond Strength based on values provided in AASHTO-9 Table C10.9.3.5.2-1

Nominal grout-to-ground bond resistance(Rs)

$$Rs = \pi * d_b * \alpha_b * L_b \quad \text{(Eq. 10.9.3.5.2-1)}$$

where,

d_b= Diameter of micropile drill hole through bonded length
α_b= Nominal micropile grout-to-ground bond strength
L_b= micropile bonded length

Rs= 384.5 kips

From AASHTO-9 Table 10.5.5.2.5-1,
For Side Resistance in Compression in Rock,

φ= 0.7 Assumes Load Test, therefore values in AASHTO-9 Table 10.5.5.2.3-1 apply, but no greater than 0.7

For Side Resistance in Uplift in Rock,

φ= 0.6 Assumes Load Test, therefore values in AASHTO-9 Table 10.5.5.2.3-1 apply, but no greater than 0.7

Factored Axial Compressive Strength, with Load Test

R_{SFC}= 269.2 kips

Factored Axial Uplift Strength, with Load Test

R_{SFU}= 230.7 kips

AECOM Imagine it. Delivered.

JOB TITLE Tannery Brook Bridge Replacement (#5610)
PROJECT/JOB NO. 60656153 CALCULATION NO.
COMPUTED BY B. Reyes DATE 9/7/23
VERIFIED BY K. Scarcarello DATE 9/8/23
SCALE N/A SHEET NO. 1 OF 1

Axial Capacity of Drilled Micropile with Rock Socket and Load Test

Assumptions:
- Minimum 2-foot plunge length
- Neglect 2-foot plunge length contribution
- Neglect end bearing
- Load Test will be performed

Mp diameter, $d_b = 0.71$ ft

Length of rock socket, $L_b = 7$ ft

Nominal grout-to-ground bond strength, $\alpha_b = 200$ psi

Nominal Grout-to-Ground Bond Resistance (R_s),

$$R_s = \pi (d_b)(\alpha_b)(L_b)$$

$$R_s = \pi (0.71\,ft)\left(\frac{12\,in}{1\,ft}\right)\left(200\,\frac{lb}{in^2}\right)\left(\frac{1\,kip}{1{,}000\,lb}\right)(7\,ft)\left(\frac{12\,in}{1\,ft}\right)$$

$$R_s = 449 \text{ kips}$$

Resistance factors per AASHTO-9, (with Load Test)

For side resistance in compression in rock, $\varphi = 0.7$

For side resistance in uplift in rock, $\varphi_{up} = 0.6$

Factored Axial Compressive Strength, with Load Test

$$R_{SFC} = \varphi R_s = 0.7(449 \text{ kips}) = \underline{314 \text{ kips}}$$

Factored Axial Uplift Strength, with Load Test

$$R_{SFU} = \varphi_{up} R_s = 0.6(449 \text{ kips}) = \underline{269 \text{ kips}}$$

5x5 = 1 in

Axial Capacity of Drilled Micropile with Rock Socket, No Load Test

Assumptions: Minimum 2-foot plunge length
Neglect 2-foot plunge length contribution to axial resistance
Neglect end bearing
Presumptive values, Load Test will not be performed

Micropile Diameter, d_b (ft)	Length of Rock Socket, L_b (ft)	Nominal Grout-to-Ground Bond Strength, α_b (psi)
0.71	7	200

Notes: 1) Grout-to-Ground Bond Strength based on values provided in AASHTO-9 Table C10.9.3.5.2-1

Nominal grout-to-ground bond resistance(Rs)

$$R_s = \pi * d_b * \alpha_b * L_b \quad \text{(Eq. 10.9.3.5.2-1)}$$

where,

d_b= Diameter of micropile drill hole through bonded length
α_b= Nominal micropile grout-to-ground bond strength
L_b= micropile bonded length

R_S= 448.6 kips

From AASHTO-9 Table 10.5.5.2.5-1,
For Side Resistance in Compression in Rock,
φ= 0.55 Based on Presumptive Values - Assumes no Load Test

For Side Resistance in Uplift in Rock,
φ= 0.55 Based on Presumptive Values - Assumes no Load Test

Factored Axial Compressive Strength, without Load Test

R_{SFC}=	246.7	kips

Factored Axial Uplift Strength, without Load Test

R_{SFU}=	246.7	kips

AECOM Imagine it. Delivered.

JOB TITLE: Tannery Brook Bridge Replacement (#3611

PROJECT/JOB NO.: 60656153 CALCULATION NO.:

COMPUTED BY: B. Reyes DATE: 9/7/23

VERIFIED BY: K. Scancarello DATE: 9/18/23

SCALE: N/A SHEET NO. 1 OF 1

Axial Capacity of Drilled Micropile with Rock Socket, No Load Test

Assumptions:
- Minimum 2-foot plunge length
- Neglect 2-foot plunge length
- Neglect end bearing
- Load Test will NOT be performed

Mp diameter, $d_b = 0.71$ ft

Length of rock socket, $L_b = 7$ ft

Nominal grout-to-ground bond strength, $\alpha_b = 200$ psi

Nominal Grout-to-Ground Bond Resistance (R_s),

$$R_s = \pi (d_b)(\alpha_b)(L_b)$$

$$R_s = \pi (0.71\ ft)\left(\frac{12\ in}{1\ ft}\right)\left(200\ \frac{lb}{in^2}\right)\left(\frac{1\ kip}{1{,}000\ lb}\right)(7\ ft)\left(\frac{12\ in}{1\ ft}\right)$$

$$R_s = 449\ \text{kips}$$

Resistance factors per AASHTO-9, with presumptive values (NO Load Test)

For side resistance in compression in rock, $\phi = 0.55$

For side resistance in uplift in rock, $\phi_{up} = 0.55$

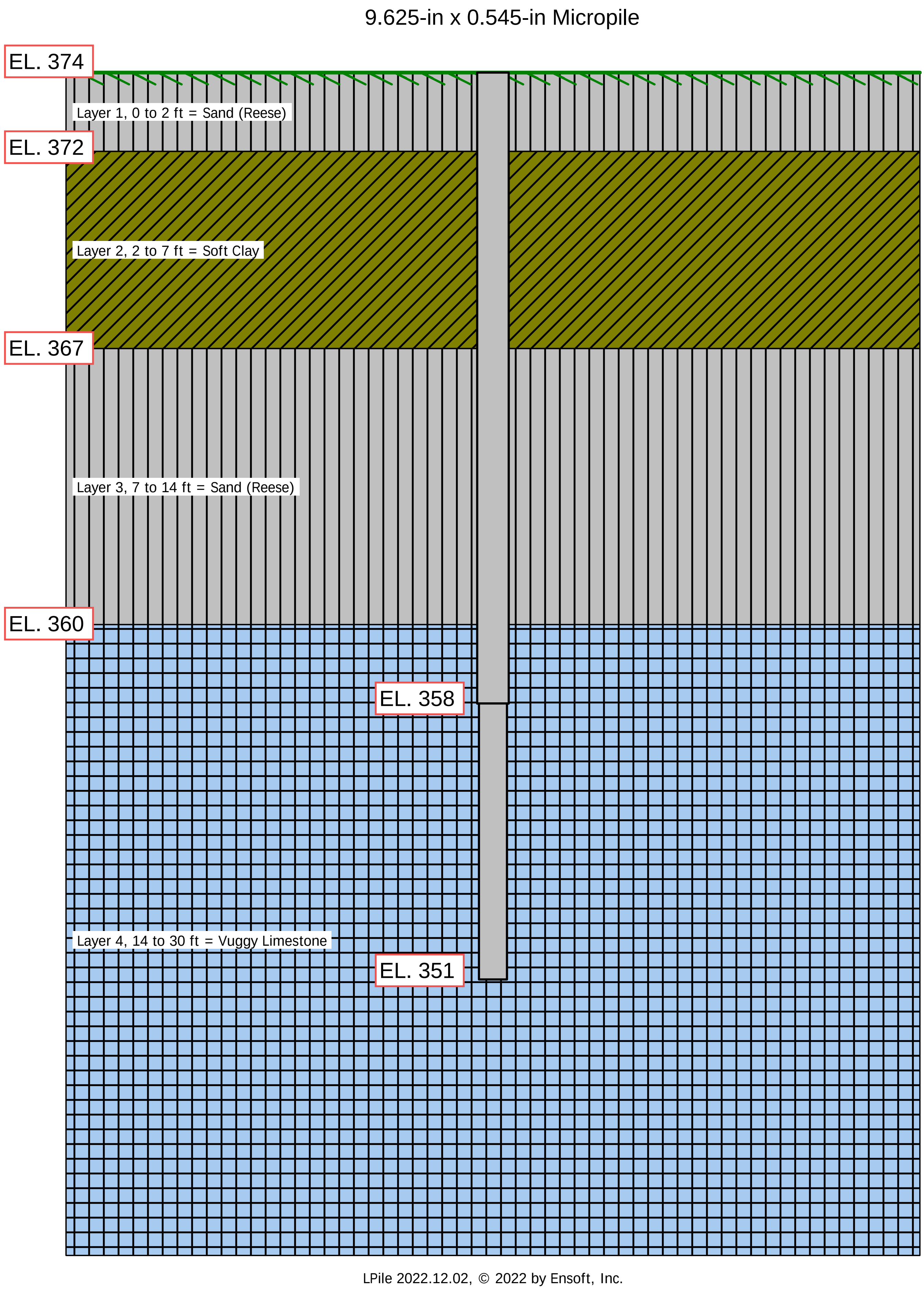
Factored Axial Compressive Strength, NO Load Test

$$R_{SFC} = \phi R_s = 0.55\,(449\ \text{kips}) = 246\ \text{kips}$$

Factored Axial Uplift Strength, NO Load Test

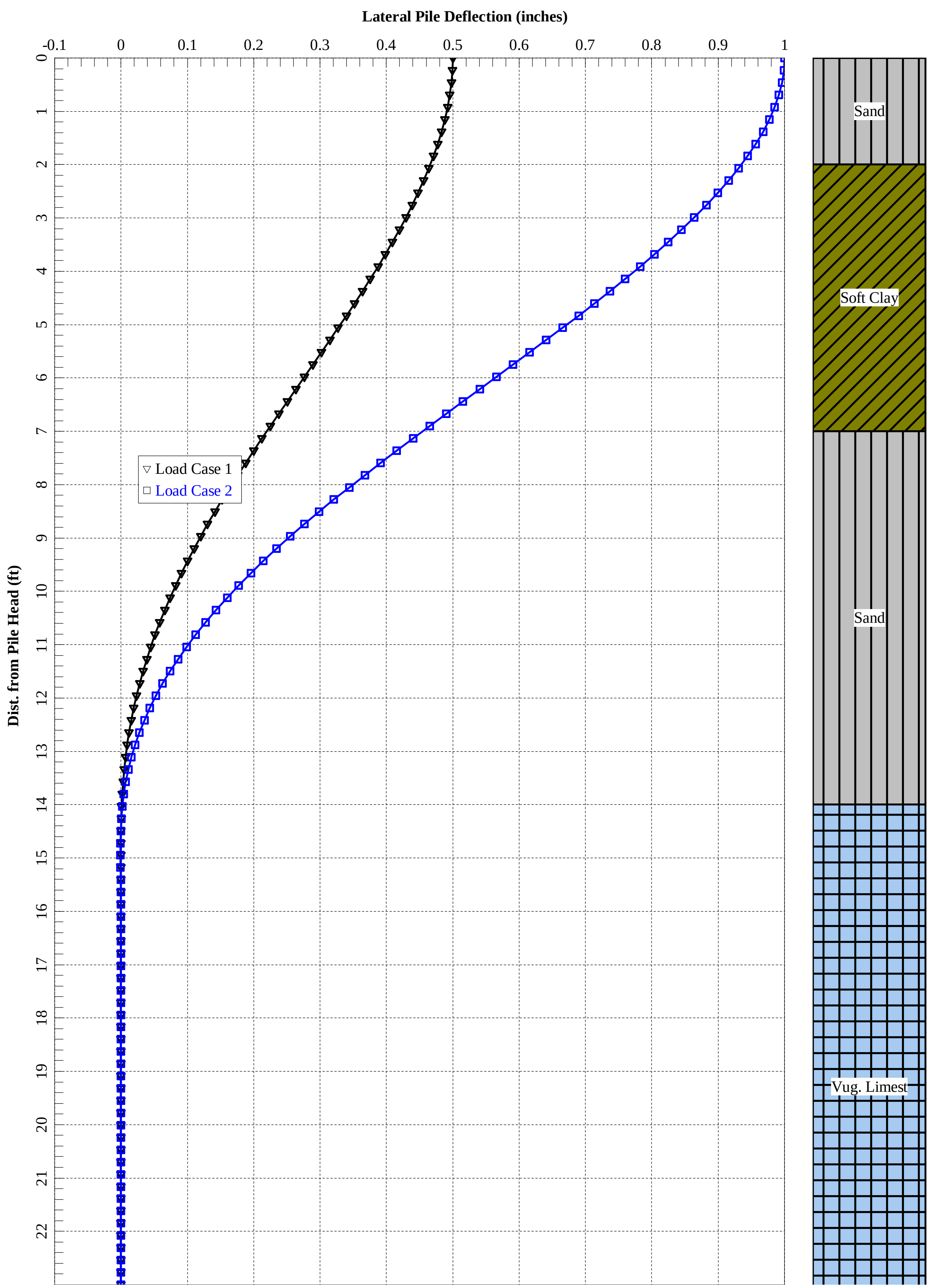
$$R_{SFU} = \phi_{up} R_s = 0.55\,(449\ \text{kips}) = 246\ \text{kips}$$

5x5 = 1 in

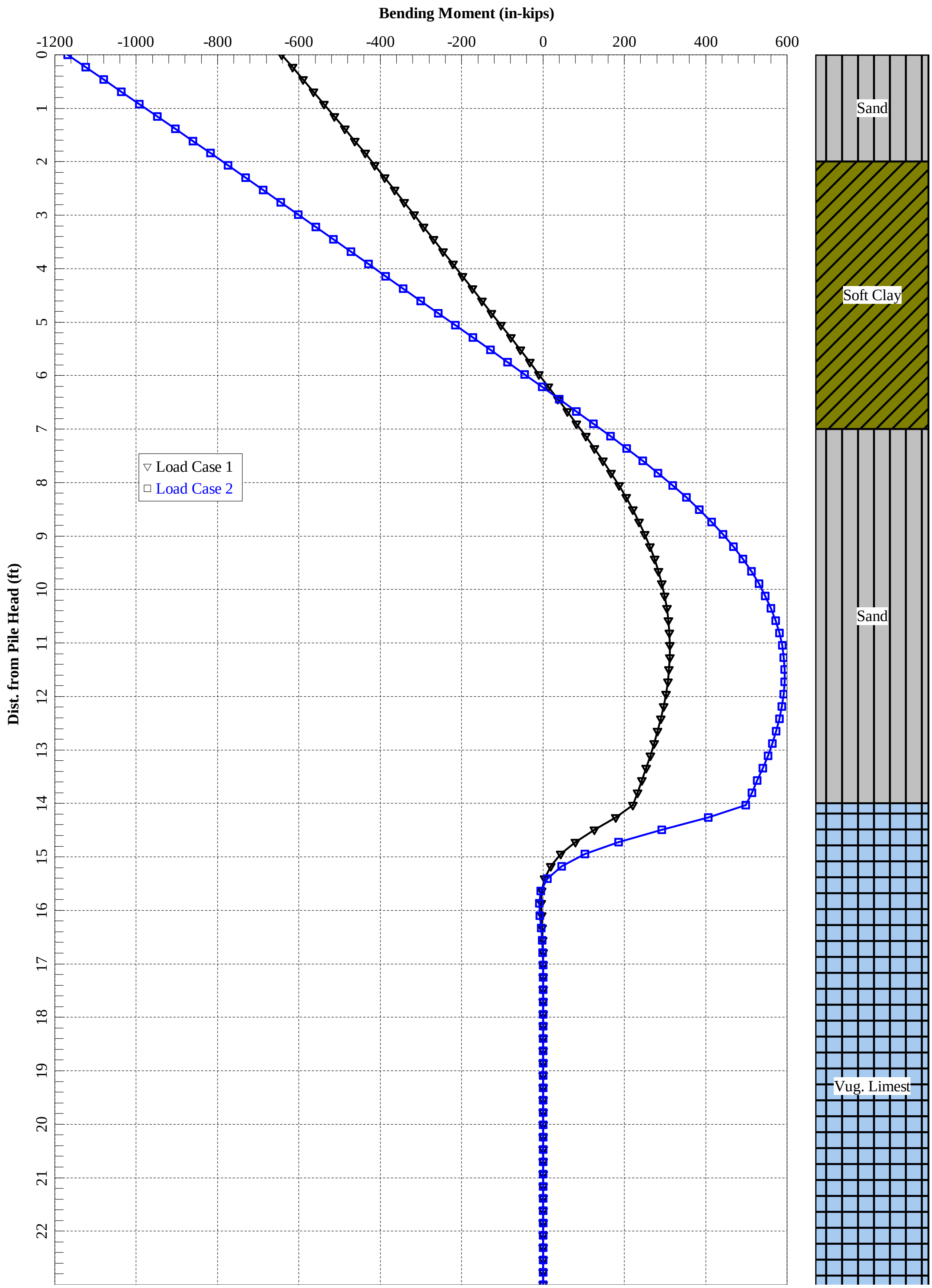


Note: Elevations correspond to NAVD88 datum

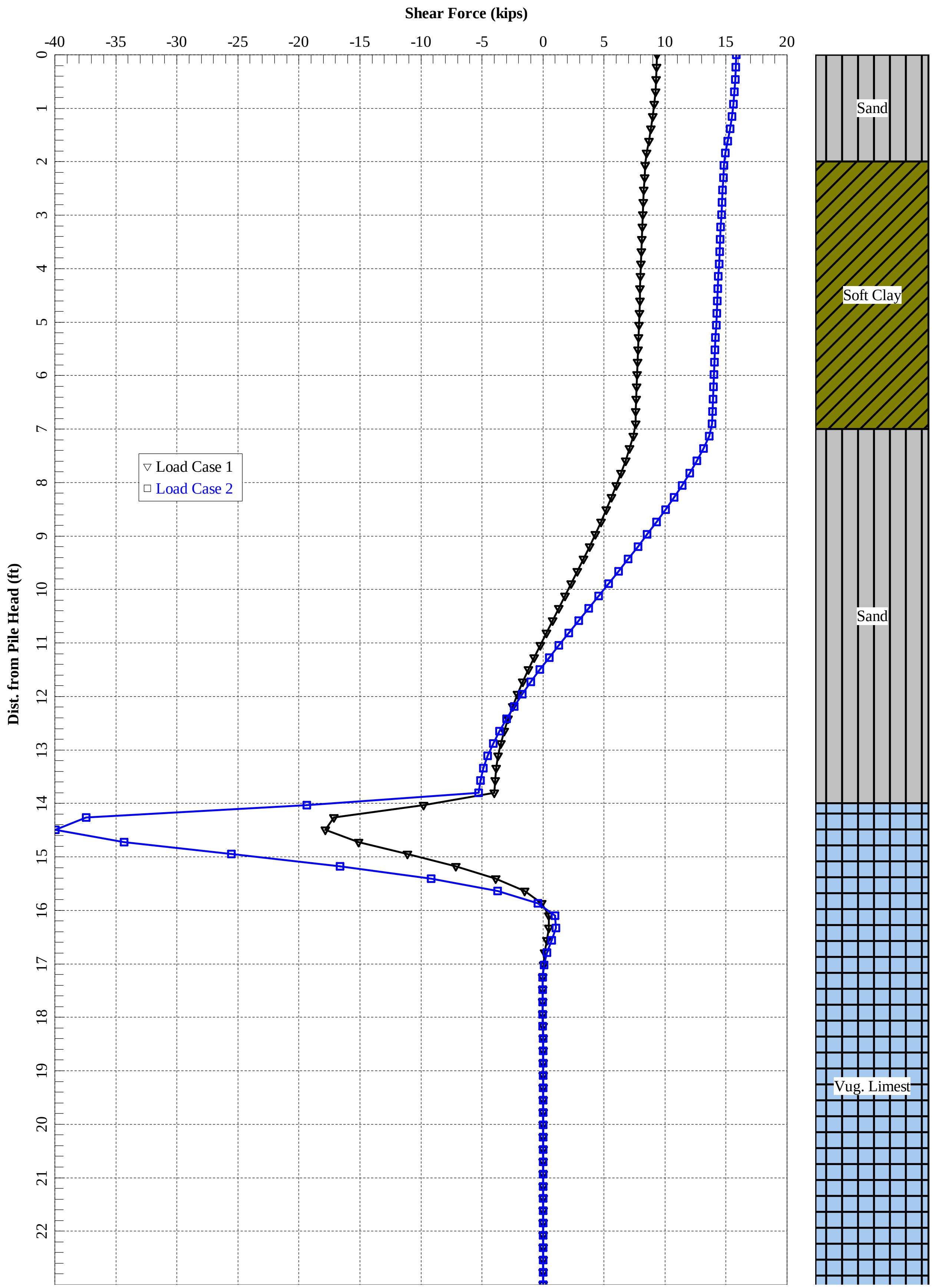
9.625-in x 0.545-in Micropile



9.625-in x 0.545-in Micropile



9.625-in x 0.545-in Micropile



```
===============================================================================

                    LPile for Windows, Version 2022-12.002

                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:
\Secure_DCS\Resources\Legacy\Private\AE_Depts\Geot\PROJECT FILES (GEOTECH)\Tannery Brook Bridge Norway ME  -
60656153\Calculations\Micropiles\

Name of input data file:
Tannery Brook Micropile Lateral Capacity_R1 Corrosion.lp12d

Name of output report file:
Tannery Brook Micropile Lateral Capacity_R1 Corrosion.lp12o

Name of plot output file:
Tannery Brook Micropile Lateral Capacity_R1 Corrosion.lp12p

Name of runtime message file:
Tannery Brook Micropile Lateral Capacity_R1 Corrosion.lp12r



-------------------------------------------------------------------------------
                           Date and Time of Analysis
-------------------------------------------------------------------------------

               Date:  September 15, 2023          Time:  14:37:46



-------------------------------------------------------------------------------
                                 Problem Title
-------------------------------------------------------------------------------

Project Name: Tannery Brook Bridge Replacement


Job Number: 60656153


Client: TY Linn


Engineer: B Reyes
```

Description: Lateral Analysis of Micropiles based on Boring BB-NTB-101

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

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

Loading Type and Number of Cycles of Loading:
- Static loading specified

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

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

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 23.000 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

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

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	9.5000
2	16.000	9.5000
3	16.000	8.5000
4	23.000	8.5000

Input Structural Properties for Pile Sections:

```
---------------------------------------------

Pile Section No. 1:

   Section 1 is a drilled shaft with permanent casing
   Length of section                                  =    16.000000 ft
   Casing outside diameter                            =     9.500000 in

Pile Section No. 2:

   Section 2 is a round drilled shaft, bored pile, or CIDH pile
   Length of section                                  =     7.000000 ft
   Shaft Diameter                                     =     8.500000 in



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

The soil profile is modelled using 4 layers

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

   Distance from top of pile to top of layer          =       0.0000 ft
   Distance from top of pile to bottom of layer       =     2.000000 ft
   Effective unit weight at top of layer              =    57.600000 pcf
   Effective unit weight at bottom of layer           =    57.600000 pcf
   Friction angle at top of layer                     =    32.000000 deg.
   Friction angle at bottom of layer                  =    32.000000 deg.
   Subgrade k at top of layer                         =    27.000000 pci
   Subgrade k at bottom of layer                      =    27.000000 pci


Layer 2 is soft clay, p-y criteria by Matlock, 1970

   Distance from top of pile to top of layer          =     2.000000 ft
   Distance from top of pile to bottom of layer       =     7.000000 ft
   Effective unit weight at top of layer              =    47.600000 pcf
   Effective unit weight at bottom of layer           =    47.600000 pcf
   Undrained cohesion at top of layer                 =    50.000000 psf
   Undrained cohesion at bottom of layer              =    50.000000 psf
   Epsilon-50 at top of layer                         =     0.020000
   Epsilon-50 at bottom of layer                      =     0.020000


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

   Distance from top of pile to top of layer          =     7.000000 ft
   Distance from top of pile to bottom of layer       =    14.000000 ft
   Effective unit weight at top of layer              =    62.600000 pcf
   Effective unit weight at bottom of layer           =    62.600000 pcf
   Friction angle at top of layer                     =    34.000000 deg.
   Friction angle at bottom of layer                  =    34.000000 deg.
   Subgrade k at top of layer                         =    60.000000 pci
   Subgrade k at bottom of layer                      =    60.000000 pci


Layer 4 is strong rock (vuggy limestone)

   Distance from top of pile to top of layer          =    14.000000 ft
   Distance from top of pile to bottom of layer       =    30.000000 ft
   Effective unit weight at top of layer              =    92.600000 pcf
   Effective unit weight at bottom of layer           =    92.600000 pcf
   Uniaxial compressive strength at top of layer      =        5000. psi
   Uniaxial compressive strength at bottom of layer   =        5000. psi


 (Depth of the lowest soil layer extends 7.000 ft below the pile tip)
```

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	57.6000	--	32.0000	--	--	27.0000
	(Reese, et al.)	2.0000	57.6000	--	32.0000	--	--	27.0000
2	Soft	2.0000	47.6000	50.0000	--	--	0.02000	--
	Clay	7.0000	47.6000	50.0000	--	--	0.02000	--
3	Sand	7.0000	62.6000	--	34.0000	--	--	60.0000
	(Reese, et al.)	14.0000	62.6000	--	34.0000	--	--	60.0000
4	Strong Rock	14.0000	92.6000	--	--	5000.	--	--
	(Vuggy Limestone)	30.0000	92.6000	--	--	5000.	--	--

Static Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 0.500000 in	S = 0.0000 in/in	230000.	N.A.	Yes
2	5	y = 1.000000 in	S = 0.0000 in/in	230000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

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

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Dimensions and Properties of Drilled Shaft (Bored Pile) with Permanent Casing:

```
Length of Section                                      =    16.000000 ft
Outer Diameter of Casing                               =     9.500000 in
Concrete Cover Thickness Inside Casing                 =     3.000000 in
Casing Wall Thickness                                  =     0.482500 in
Moment of Inertia of Steel Casing                      =   139.334052 in^4
Yield Stress of Casing                                 =       80000. psi
Elastic Modulus of Casing                              =    29000000. psi
Number of Reinforcing Bars                             =            1 bar
Area of Single Reinforcing Bar                         =     0.790000 sq. in.
Edge-to-Edge Bar Spacing                               =     -1.00000 in
Maximum Concrete Aggregate Size                        =     0.500000 in
Ratio of Bar Spacing to Aggregate Size                 =        -2.00
Offset of Center of Rebar Cage from Center of Pile     =       0.0000 in
Yield Stress of Reinforcing Bars                       =       75000. psi
Modulus of Elasticity of Reinforcing Bars              =    29000000. psi
Gross Area of Pile                                     =    70.882184 sq. in.
Area of Concrete                                       =    56.423291 sq. in.
Cross-sectional Area of Steel Casing                   =    13.668893 sq. in.
Area of All Steel (Casing and Bars)                    =    14.458893 sq. in.
Area Ratio of All Steel to Gross Area of Pile          =        20.40 percent
```

Axial Structural Capacities:

```
Nom. Axial Structural Capacity = 0.85 Fc Ac + Fy As    =     1344.601 kips
Tensile Load for Cracking of Concrete                  =      -71.843 kips
Nominal Axial Tensile Capacity                         =    -1152.761 kips
```

Reinforcing Bar Dimensions and Positions Used in Computations:

Bar Number	Bar Diam. inches	Bar Area sq. in.	X inches	Y inches
1	1.000000	0.790000	0.00000	0.00000

NOTE: The positions of the above rebars were computed by LPile

Concrete Properties:

```
Compressive Strength of Concrete                       =        4000. psi
Modulus of Elasticity of Concrete                      =     3604997. psi
Modulus of Rupture of Concrete                         =   -474.34165 psi
Compression Strain at Peak Stress                      =     0.001886
Tensile Strain at Fracture of Concrete                 =   -0.0001154
Maximum Coarse Aggregate Size                          =     0.500000 in
```

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

Number	Axial Thrust Force kips
1	230.000

Definitions of Run Messages and Notes:

C = concrete in section has cracked in tension.
Y = stress in reinforcing steel has reached yield stress.
T = ACI 318 criteria for tension-controlled section met, tensile strain in
    reinforcement exceeds 0.005 while simultaneously compressive strain in

concrete more than 0.003. See ACI 318-14, Section 21.2.3.
Z = depth of tensile zone in concrete section is less than 10 percent of section depth.

Bending Stiffness (EI) = Computed Bending Moment / Curvature.
Position of neutral axis is measured from edge of compression side of pile.
Compressive stresses and strains are positive in sign.
Tensile stresses and strains are negative in sign.

Axial Thrust Force = 230.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Comp Strain in/in	Max Tens Strain in/in	Max Conc Stress ksi	Max Steel Stress ksi	Max Casing Stress ksi	Run Msg
0.00000125	6.1582342	4926587.	294.2148963	0.0003678	0.0003559	1.4075122	10.5120431	10.6635681	
0.00000250	12.3271323	4930853.	149.4833687	0.0003737	0.0003500	1.4275515	10.5310505	10.8341005	
0.00000375	18.4960238	4932273.	101.2399245	0.0003796	0.0003440	1.4475180	10.5501012	11.0046762	
0.00000500	24.6649052	4932981.	77.1185011	0.0003856	0.0003381	1.4674118	10.5691952	11.1752952	
0.00000625	30.8337734	4933404.	62.6458860	0.0003915	0.0003322	1.4872328	10.5883325	11.3459575	
0.00000750	37.0026250	4933683.	52.9976751	0.0003975	0.0003262	1.5069810	10.6075131	11.5166631	
0.00000875	43.1714569	4933881.	46.1062666	0.0004034	0.0003203	1.5266562	10.6267370	11.6874120	
0.00001000	49.3402656	4934027.	40.9378595	0.0004094	0.0003144	1.5462584	10.6460043	11.8582043	
0.00001125	55.5090480	4934138.	36.9181202	0.0004153	0.0003085	1.5657876	10.6653149	12.0290399	
0.00001250	61.6778007	4934224.	33.7024482	0.0004213	0.0003025	1.5852436	10.6846687	12.1999187	
0.00001375	67.8465204	4934292.	31.0715524	0.0004272	0.0002966	1.6046266	10.7040659	12.3708409	
0.00001500	74.0152040	4934347.	28.8792389	0.0004332	0.0002907	1.6239362	10.7235064	12.5418064	
0.00001625	80.1838480	4934391.	27.0242962	0.0004391	0.0002848	1.6431726	10.7429903	12.7128153	
0.00001750	86.3524493	4934426.	25.4344308	0.0004451	0.0002789	1.6623357	10.7625174	12.8838674	
0.00001875	92.5210045	4934454.	24.0566270	0.0004511	0.0002729	1.6814254	10.7820879	13.0549629	
0.00002000	98.6895104	4934476.	22.8511235	0.0004570	0.0002670	1.7004416	10.8017017	13.2261017	
0.00002125	104.8579636	4934492.	21.7875141	0.0004630	0.0002611	1.7193842	10.8213588	13.3972838	
0.00002250	111.0263609	4934505.	20.8421500	0.0004689	0.0002552	1.7382533	10.8410592	13.5685092	
0.00002375	117.1946991	4934514.	19.9963607	0.0004749	0.0002493	1.7570488	10.8608029	13.7397779	
0.00002500	123.3629750	4934519.	19.2352102	0.0004809	0.0002434	1.7757705	10.8805896	13.9110896	
0.00002625	129.5311849	4934521.	18.5466071	0.0004868	0.0002375	1.7944185	10.9004200	14.0824450	
0.00002750	135.6993258	4934521.	17.9206586	0.0004928	0.0002316	1.8129927	10.9202937	14.2538437	
0.00002875	141.8673944	4934518.	17.3491923	0.0004988	0.0002257	1.8314930	10.9402107	14.4252857	
0.00003000	148.0353874	4934513.	16.8253981	0.0005048	0.0002198	1.8499194	10.9601711	14.5967711	
0.00003125	154.2033016	4934506.	16.3435552	0.0005107	0.0002139	1.8682718	10.9801748	14.7682998	
0.00003250	160.3711336	4934496.	15.8988231	0.0005167	0.0002080	1.8865501	11.0002218		

								14.9398718
0.00003375	166.5388801	4934485.	15.4870783	0.0005227	0.0002021	1.9047543	11.0203121	15.1114871
0.00003500	172.7065379	4934473.	15.1047866	0.0005287	0.0001962	1.9228843	11.0404458	15.2831458
0.00003625	178.8741038	4934458.	14.7489011	0.0005346	0.0001903	1.9409401	11.0606228	15.4548478
0.00003750	185.0415743	4934442.	14.4167811	0.0005406	0.0001844	1.9589216	11.0808431	15.6265931
0.00003875	191.2089463	4934424.	14.1061268	0.0005466	0.0001785	1.9768288	11.1011067	15.7983817
0.00004000	197.3762164	4934405.	13.8149257	0.0005526	0.0001726	1.9946615	11.1214137	15.9702137
0.00004125	203.5433814	4934385.	13.5414094	0.0005586	0.0001667	2.0124198	11.1417640	16.1420890
0.00004250	209.7104380	4934363.	13.2840175	0.0005646	0.0001608	2.0301035	11.1621577	16.3140077
0.00004375	215.8773829	4934340.	13.0413678	0.0005706	0.0001549	2.0477127	11.1825947	16.4859697
0.00004500	222.0442128	4934316.	12.8122318	0.0005766	0.0001491	2.0652472	11.2030750	16.6579750
0.00004625	228.2109245	4934290.	12.5955140	0.0005825	0.0001432	2.0827069	11.2235986	16.8300236
0.00004750	234.3775146	4934263.	12.3902337	0.0005885	0.0001373	2.1000919	11.2441657	17.0021157
0.00004875	240.5439798	4934235.	12.1955113	0.0005945	0.0001314	2.1174021	11.2647760	17.1742510
0.00005125	252.8765228	4934176.	11.8346500	0.0006065	0.0001197	2.1517978	11.3061267	17.5186517
0.00005375	265.2085269	4934112.	11.5074685	0.0006185	0.0001079	2.1858934	11.3476508	17.8632258
0.00005625	277.5399661	4934044.	11.2094760	0.0006305	0.00009616	2.2196885	11.3893483	18.2079733
0.00005875	289.8708140	4933971.	10.9369464	0.0006425	0.00008442	2.2531826	11.4312193	18.5528943
0.00006125	302.2010443	4933895.	10.6867617	0.0006546	0.00007269	2.2863752	11.4732636	18.8979886
0.00006375	314.5306307	4933814.	10.4562932	0.0006666	0.00006096	2.3192659	11.5154814	19.2432564
0.00006625	326.8595470	4933729.	10.2433088	0.0006786	0.00004924	2.3518541	11.5578727	19.5886977
0.00006875	339.1877668	4933640.	10.0459012	0.0006907	0.00003753	2.3841394	11.6004374	19.9343124
0.00007125	351.5152638	4933548.	9.8624307	0.0007027	0.00002582	2.4161214	11.6431757	20.2801007
0.00007375	363.8420118	4933451.	9.6914800	0.0007147	0.00001412	2.4477995	11.6860874	20.6260624
0.00007625	376.1679844	4933351.	9.5318177	0.0007268	0.00000243	2.4791732	11.7291727	20.9721977
0.00007875	388.4931554	4933246.	9.3823687	0.0007389	-0.00000926	2.5102422	11.7724315	21.3185065
0.00008125	400.8174983	4933138.	9.2421902	0.0007509	-0.00002095	2.5410058	11.8158639	21.6649889
0.00008375	413.1409869	4933027.	9.1104520	0.0007630	-0.00003262	2.5714636	11.8594699	22.0116449
0.00008625	425.4635948	4932911.	8.9864203	0.0007751	-0.00004430	2.6016152	11.9032495	22.3584745
0.00008875	437.7852072	4932791.	8.8694433	0.0007872	-0.00005596	2.6314599	11.9472018	22.7054768
0.00009125	450.1054427	4932662.	8.7589397	0.0007993	-0.00006762	2.6609969	11.9913225	23.0526475
0.00009375	462.4237800	4932520.	8.6543894	0.0008113	-0.00007928	2.6902254	12.0356059	23.3999809
0.00009625	474.7396594	4932360.	8.5553263	0.0008235	-0.00009092	2.7191442	12.0800453	23.7474703
0.00009875	487.0525196	4932177.	8.4613311	0.0008356	-0.000103	2.7477524	12.1246341	24.0951091
0.0001013	499.3618176	4931969.	8.3720262	0.0008477	-0.000114	2.7760490	12.1693655	24.4428905
0.0001038	510.4731781	4920223.	8.2821932	0.0008593	-0.000126	2.8028594	12.1995581	24.7761331 C

0.0001063	522.3981352	4916688.	8.1998330	0.0008712	-0.000138	2.8301638	12.2397504	25.1193754	C
0.0001088	534.2732959	4912858.	8.1210958	0.0008832	-0.000150	2.8571131	12.2794281	25.4621031	C
0.0001113	546.0877868	4908654.	8.0456825	0.0008951	-0.000162	2.8836947	12.3184116	25.8041366	C
0.0001138	557.8554175	4904223.	7.9734285	0.0009070	-0.000174	2.9099243	12.3568819	26.1456569	C
0.0001163	569.5816802	4899627.	7.9041507	0.0009189	-0.000186	2.9358086	12.3949088	26.4867338	C
0.0001188	581.2710138	4894914.	7.8376787	0.0009307	-0.000197	2.9613535	12.4325533	26.8274283	C
0.0001213	592.9005772	4889902.	7.7737313	0.0009426	-0.000209	2.9865319	12.4694361	27.1673611	C
0.0001238	604.4992956	4884843.	7.7122789	0.0009544	-0.000221	3.0113782	12.5060007	27.5069757	C
0.0001263	616.0667669	4879737.	7.6531714	0.0009662	-0.000233	3.0358929	12.5422397	27.8462647	C
0.0001288	627.5822089	4874425.	7.5961736	0.0009780	-0.000245	3.0600506	12.5777855	28.1848605	C
0.0001313	639.0781577	4869167.	7.5413079	0.0009898	-0.000257	3.0838913	12.6131816	28.5233066	C
0.0001338	650.5291337	4863769.	7.4883386	0.0010016	-0.000269	3.1073843	12.6479781	28.8611531	C
0.0001363	661.9564251	4858396.	7.4372524	0.0010133	-0.000281	3.1305559	12.6825346	29.1987596	C
0.0001388	673.3498449	4852972.	7.3879014	0.0010251	-0.000293	3.1533940	12.7166656	29.5359406	C
0.0001413	684.7168334	4847553.	7.3402226	0.0010368	-0.000305	3.1759079	12.7504905	29.8728155	C
0.0001438	696.0544709	4842118.	7.2941154	0.0010485	-0.000317	3.1980947	12.7839534	30.2093284	C
0.0001463	707.3695506	4836715.	7.2495261	0.0010602	-0.000329	3.2199628	12.8171685	30.5455935	C
0.0001488	718.6533178	4831283.	7.2063382	0.0010719	-0.000341	3.2415017	12.8499638	30.8814388	C
0.0001588	763.5748362	4809920.	7.0466628	0.0011187	-0.000389	3.3244965	12.9787214	32.2223964	C
0.0001688	808.1747748	4789184.	6.9051534	0.0011652	-0.000438	3.4024740	13.1037670	33.5596420	C
0.0001788	852.4951362	4769204.	6.7788696	0.0012117	-0.000486	3.4755038	13.2256625	34.8937375	C
0.0001888	896.5673472	4750026.	6.6654707	0.0012581	-0.000535	3.5436411	13.3448425	36.2251175	C
0.0001988	940.4324116	4731735.	6.5631460	0.0013044	-0.000584	3.6069504	13.4620800	37.5545550	C
0.0002088	984.1107124	4714303.	6.4703614	0.0013507	-0.000632	3.6654669	13.5777228	38.8823978	C
0.0002188	1028.	4697699.	6.3858685	0.0013969	-0.000681	3.7192225	13.6921508	40.2090258	C
0.0002288	1071.	4681894.	6.3086417	0.0014431	-0.000730	3.7682446	13.8057746	41.5348496	C
0.0002388	1114.	4666779.	6.2377394	0.0014893	-0.000779	3.8125353	13.9183956	42.8596706	C
0.0002488	1157.	4652336.	6.1724531	0.0015354	-0.000828	3.8521164	14.0304061	44.1838811	C
0.0002588	1200.	4638588.	6.1122420	0.0015815	-0.000877	3.8870147	14.1426338	45.5083088	C
0.0002688	1243.	4625393.	6.0564546	0.0016277	-0.000925	3.9172139	14.2544159	46.8322909	C
0.0002788	1286.	4612759.	6.0046882	0.0016738	-0.000974	3.9427292	14.3663457	48.1564207	C
0.0002888	1328.	4600647.	5.9565520	0.0017200	-0.001023	3.9635622	14.4786502	49.4809252	C
0.0002988	1371.	4588992.	5.9116531	0.0017661	-0.001072	3.9797063	14.5910819	50.8055569	C
0.0003088	1413.	4577779.	5.8697215	0.0018123	-0.001121	3.9911630	14.7040413	52.1307163	C
0.0003188	1456.	4566993.	5.8305113	0.0018585	-0.001170	3.9979275	14.8178362	53.4567112	C
0.0003288	1498.	4556537.	5.7936834	0.0019047	-0.001218	3.9991262	14.9316024		

```
54.7826774 C
    0.0003388        1540.      4546481.     5.7591882     0.0019509     -0.001267     3.9997194    15.0469230
56.1101980 C
    0.0003488        1582.      4536714.     5.7266889     0.0019972     -0.001316     3.9999714    15.1624232
57.4378982 C
    0.0003588        1624.      4527243.     5.6960789     0.0020435     -0.001365     3.9989713    15.2787289
58.7664039 C
    0.0003688        1666.      4518080.     5.6672777     0.0020898     -0.001413     3.9993435    15.3966241
60.0964991 C
    0.0003788        1708.      4509144.     5.6400667     0.0021362     -0.001462     3.9995523    15.5152694
61.4273444 C
    0.0003888        1750.      4500407.     5.6143123     0.0021826     -0.001511     3.9996509    15.6345684
62.7588434 C
    0.0003988        1791.      4491904.     5.5899955     0.0022290     -0.001559     3.9996700    15.7555525
64.0920275 C
    0.0004088        1833.      4483625.     5.5670078     0.0022755     -0.001608     3.9996159    15.8781876
65.4268626 C
    0.0004188        1874.      4475525.     5.5451661     0.0023220     -0.001656     3.9994698    16.0014056
66.7622806 C
    0.0004288        1915.      4467610.     5.5244055     0.0023686     -0.001705     3.9991912    16.1253981
68.0984731 C
    0.0004388        1957.      4459904.     5.5047080     0.0024152     -0.001753     3.9987174    16.2508759
69.4361509 C
    0.0004488        1998.      4452402.     5.4859976     0.0024618     -0.001801     3.9999847    16.3777742
70.7752492 C
    0.0004588        2039.      4445089.     5.4682115     0.0025085     -0.001850     3.9998952    16.5061170
72.1157920 C
    0.0004688        2080.      4437940.     5.4512025     0.0025553     -0.001898     3.9994921    16.6347064
73.4565814 C
    0.0004788        2121.      4430968.     5.4349552     0.0026020     -0.001946     3.9986443    16.7640070
74.7980820 C
    0.0004888        2162.      4424182.     5.4194578     0.0026488     -0.001994     3.9999852    16.8945134
76.1407884 C
    0.0004988        2203.      4417567.     5.4046676     0.0026956     -0.002043     3.9995564    17.0262598
77.4847348 C
    0.0005088        2244.      4411132.     5.3905311     0.0027424     -0.002091     3.9983795    17.1590712
78.8297462 C
    0.0005188        2285.      4404674.     5.3770968     0.0027894     -0.002139     3.9999168    17.2942490
80.0000000 CY
    0.0005288        2325.      4396986.     5.3648485     0.0028367     -0.002186     3.9990469    17.4398182
80.0000000 CY
    0.0005388        2364.      4387142.     5.3541386     0.0028845     -0.002234     3.9999258    17.6023139
80.0000000 CY
    0.0005488        2401.      4374575.     5.3453125     0.0029332     -0.002280     3.9994696    17.7885897
80.0000000 CY
    0.0006088        2587.      4249189.     5.3282178     0.0032436     -0.002540     3.9999965    19.4317986
80.0000000 CY
    0.0006688        2734.      4087513.     5.3467184     0.0035756     -0.002778     3.9981913    21.7058440
80.0000000 CY
    0.0007288        2849.      3908839.     5.3736289     0.0039160     -0.003007     3.9998226    24.2220045
80.0000000 CY


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   Summary of Results for Nominal Moment Capacity for Section 1
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Moment values interpolated at maximum compressive strain = 0.003
or maximum developed moment if pile fails at smaller strains.

 Load          Axial Thrust        Nominal Mom. Cap.      Max. Comp.      Max. Tens.
  No.              kips                 in-kip              Strain           Strain
 ----         ----------------     ------------------     ------------    ------------
   1              230.000              2440.595            0.00300000     -0.00233575

Note that the values of moment capacity in the table above are not
factored by a strength reduction factor (phi-factor).

In ACI 318, the value of the strength reduction factor depends on whether
the transverse reinforcing steel bars are tied hoops (0.65) or spirals (0.75).
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The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to ACI 318, or the value required by the design standard being followed.

The following table presents factored moment capacities and corresponding bending stiffnesses computed for common resistance factor values used for reinforced concrete sections.

Axial Load No.	Resist. Factor	Nominal Ax. Thrust kips	Nominal Moment Cap in-kips	Ult. (Fac) Ax. Thrust kips	Ult. (Fac) Moment Cap in-kips	Bend. Stiff. at Ult Mom kip-in^2
1	0.65	230.000000	2441.	149.500000	1586.	4535764.
1	0.75	230.000000	2441.	172.500000	1830.	4484071.
1	0.90	230.000000	2441.	207.000000	2197.	4418654.

Pile Section No. 2:

Dimensions and Properties of Drilled Shaft (Bored Pile):

Length of Section	=	7.000000 ft
Shaft Diameter	=	8.500000 in
Concrete Cover Thickness (to edge of long. rebar)	=	3.000000 in
Number of Reinforcing Bars	=	1 bar
Yield Stress of Reinforcing Bars	=	75000. psi
Modulus of Elasticity of Reinforcing Bars	=	29000000. psi
Gross Area of Shaft	=	56.745017 sq. in.
Total Area of Reinforcing Steel	=	0.790000 sq. in.
Area Ratio of Steel Reinforcement	=	1.39 percent
Edge-to-Edge Bar Spacing	=	-1.00000 in
Maximum Concrete Aggregate Size	=	0.500000 in
Ratio of Bar Spacing to Aggregate Size	=	-2.00
Offset of Center of Rebar Cage from Center of Pile	=	0.0000 in

Axial Structural Capacities:

Nom. Axial Structural Capacity = 0.85 Fc Ac + Fy As	=	249.497 kips
Tensile Load for Cracking of Concrete	=	-25.915 kips
Nominal Axial Tensile Capacity	=	-59.250 kips

Reinforcing Bar Dimensions and Positions Used in Computations:

Bar Number	Bar Diam. inches	Bar Area sq. in.	X inches	Y inches
1	1.000000	0.790000	0.00000	0.00000

NOTE: The positions of the above rebars were computed by LPile

Concrete Properties:

Compressive Strength of Concrete	=	4000. psi
Modulus of Elasticity of Concrete	=	3604997. psi
Modulus of Rupture of Concrete	=	-474.34165 psi
Compression Strain at Peak Stress	=	0.001886
Tensile Strain at Fracture of Concrete	=	-0.0001154
Maximum Coarse Aggregate Size	=	0.500000 in

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

Number	Axial Thrust Force kips
1	230.000

Definitions of Run Messages and Notes:

C = concrete in section has cracked in tension.
Y = stress in reinforcing steel has reached yield stress.
T = ACI 318 criteria for tension-controlled section met, tensile strain in reinforcement exceeds 0.005 while simultaneously compressive strain in concrete more than 0.003. See ACI 318-14, Section 21.2.3.
Z = depth of tensile zone in concrete section is less than 10 percent of section depth.

Bending Stiffness (EI) = Computed Bending Moment / Curvature.
Position of neutral axis is measured from edge of compression side of pile.
Compressive stresses and strains are positive in sign.
Tensile stresses and strains are negative in sign.

Axial Thrust Force = 230.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Comp Strain in/in	Max Tens Strain in/in	Max Conc Stress ksi	Max Steel Stress ksi	Run Msg
0.00000125	0.4004984	320399.	1028.	0.0012851	0.0012745	3.5936088	37.1303877	
0.00000250	0.8390452	335618.	516.1651310	0.0012904	0.0012692	3.6007064	37.1477408	
0.00000375	1.2775519	340681.	345.5329081	0.0012957	0.0012639	3.6077535	37.1653569	
0.00000500	1.7159990	343200.	260.2186115	0.0013011	0.0012586	3.6147497	37.1832362	
0.00000625	2.1543668	344699.	209.0314856	0.0013064	0.0012533	3.6216949	37.2013787	
0.00000750	2.5926357	345685.	174.9079453	0.0013118	0.0012481	3.6285886	37.2197844	
0.00000875	3.0307861	346376.	150.5350254	0.0013172	0.0012428	3.6354305	37.2384533	
0.00001000	3.4687984	346880.	132.2562435	0.0013226	0.0012376	3.6422204	37.2573857	
0.00001125	3.9066530	347258.	118.0402206	0.0013280	0.0012323	3.6489578	37.2765814	
0.00001250	4.3443301	347546.	106.6681290	0.0013334	0.0012271	3.6556426	37.2960406	
0.00001375	4.7818102	347768.	97.3643515	0.0013388	0.0012219	3.6622742	37.3157633	
0.00001500	5.2190736	347938.	89.6118097	0.0013442	0.0012167	3.6688525	37.3357498	
0.00001625	5.6561006	348068.	83.0525263	0.0013496	0.0012115	3.6753770	37.3560000	
0.00001750	6.0928715	348164.	77.4308034	0.0013550	0.0012063	3.6818476	37.3765140	
0.00001875	6.5293666	348233.	72.5591291	0.0013605	0.0012011	3.6882637	37.3972921	
0.00002000	6.9655662	348278.	68.2968695	0.0013659	0.0011959	3.6946252	37.4183344	
0.00002125	7.4014506	348304.	64.5364810	0.0013714	0.0011908	3.7009317	37.4396409	
0.00002250	7.8370000	348311.	61.1943188	0.0013769	0.0011856	3.7071828	37.4612118	
0.00002375	8.2721946	348303.	58.2043473	0.0013824	0.0011805	3.7133783	37.4830474	
0.00002500	8.7070147	348281.	55.5137381	0.0013878	0.0011753	3.7195178	37.5051477	
0.00002625	9.1414403	348245.	53.0797254	0.0013933	0.0011702	3.7256009	37.5275129	
0.00002750	9.5754518	348198.	50.8673191	0.0013989	0.0011651	3.7316274	37.5501433	
0.00002875	10.0090292	348140.	48.8476142	0.0014044	0.0011600	3.7375970	37.5730390	
0.00003000	10.4421525	348072.	46.9965232	0.0014099	0.0011549	3.7435092	37.5962002	
0.00003125	10.8748020	347994.	45.2938128	0.0014154	0.0011498	3.7493638	37.6196273	
0.00003250	11.3069576	347906.	43.7223623	0.0014210	0.0011447	3.7551604	37.6433203	
0.00003375	11.7385995	347810.	42.2675875	0.0014265	0.0011397	3.7608988	37.6672795	
0.00003500	12.1697075	347706.	40.9169878	0.0014321	0.0011346	3.7665785	37.6915052	
0.00003625	12.6002616	347593.	39.6597866	0.0014377	0.0011295	3.7721992	37.7159976	
0.00003750	13.0302419	347473.	38.4866443	0.0014432	0.0011245	3.7777606	37.7407570	
0.00003875	13.4596281	347345.	37.3894264	0.0014488	0.0011195	3.7832624	37.7657836	
0.00004000	13.8884002	347210.	36.3610153	0.0014544	0.0011144	3.7887043	37.7910778	
0.00004125	14.3165379	347068.	35.3951560	0.0014601	0.0011094	3.7940858	37.8166399	
0.00004250	14.7440211	346918.	34.4863296	0.0014657	0.0011044	3.7994067	37.8424701	
0.00004375	15.1708295	346762.	33.6296477	0.0014713	0.0010994	3.8046666	37.8685687	
0.00004500	15.5969428	346599.	32.8207652	0.0014769	0.0010944	3.8098653	37.8949361	
0.00004625	16.0223407	346429.	32.0558067	0.0014826	0.0010895	3.8150022	37.9215727	
0.00004750	16.4470029	346253.	31.3313048	0.0014882	0.0010845	3.8200772	37.9484787	
0.00004875	16.8709088	346070.	30.6441477	0.0014939	0.0010795	3.8250899	37.9756546	
0.00005125	17.7163701	345685.	29.3709388	0.0015053	0.0010696	3.8349268	38.0308172	
0.00005375	18.5585605	345276.	28.2168631	0.0015167	0.0010598	3.8445103	38.0870637	
0.00005625	19.3973147	344841.	27.1660384	0.0015281	0.0010500	3.8538377	38.1443971	

0.00005875	20.2324670	344382.	26.2052855	0.0015396	0.0010402	3.8629060	38.2028210
0.00006125	21.0638506	343900.	25.3235774	0.0015511	0.0010304	3.8717127	38.2623388
0.00006375	21.8912976	343393.	24.5116165	0.0015626	0.0010207	3.8802548	38.3229544
0.00006625	22.7146394	342862.	23.7615092	0.0015742	0.0010111	3.8885295	38.3846715
0.00006875	23.5337063	342308.	23.0665096	0.0015858	0.0010014	3.8965340	38.4474944
0.00007125	24.3483275	341731.	22.4208192	0.0015975	0.0009919	3.9042654	38.5114272
0.00007375	25.1583310	341130.	21.8194254	0.0016092	0.0009823	3.9117208	38.5764745
0.00007625	25.9635438	340505.	21.2579735	0.0016209	0.0009728	3.9188974	38.6426409
0.00007875	26.7637916	339858.	20.7326614	0.0016327	0.0009633	3.9257921	38.7099312
0.00008125	27.5588986	339186.	20.2401553	0.0016445	0.0009539	3.9324021	38.7783506
0.00008375	28.3486880	338492.	19.7775196	0.0016564	0.0009445	3.9387243	38.8479042
0.00008625	29.1329815	337774.	19.3421591	0.0016683	0.0009351	3.9447558	38.9185975
0.00008875	29.9115991	337032.	18.9317709	0.0016802	0.0009258	3.9504935	38.9904361
0.00009125	30.6843596	336267.	18.5443048	0.0016922	0.0009165	3.9559344	39.0634261
0.00009375	31.4510801	335478.	18.1779292	0.0017042	0.0009073	3.9610753	39.1375734
0.00009625	32.2115761	334666.	17.8310031	0.0017162	0.0008981	3.9659132	39.2128844
0.00009875	32.9656612	333829.	17.5020514	0.0017283	0.0008890	3.9704449	39.2893657
0.0001013	33.7131474	332969.	17.1897452	0.0017405	0.0008798	3.9746671	39.3670241
0.0001038	34.4538448	332085.	16.8928835	0.0017526	0.0008708	3.9785768	39.4458665
0.0001063	35.1875617	331177.	16.6103783	0.0017649	0.0008617	3.9821706	39.5259004
0.0001088	35.9141041	330245.	16.3412421	0.0017771	0.0008527	3.9854453	39.6071332
0.0001113	36.6332762	329288.	16.0845759	0.0017894	0.0008438	3.9883975	39.6895728
0.0001138	37.3448800	328307.	15.8395600	0.0018017	0.0008349	3.9910238	39.7732272
0.0001163	38.0487150	327301.	15.6054453	0.0018141	0.0008260	3.9933209	39.8581048
0.0001188	38.7445788	326270.	15.3815458	0.0018266	0.0008172	3.9952853	39.9442142
0.0001213	39.4322661	325215.	15.1672320	0.0018390	0.0008084	3.9969135	40.0315644
0.0001238	40.1115694	324134.	14.9619258	0.0018515	0.0007997	3.9982020	40.1201646
0.0001263	40.7822786	323028.	14.7650945	0.0018641	0.0007910	3.9991471	40.2100244
0.0001288	41.4441807	321897.	14.5762471	0.0018767	0.0007823	3.9997453	40.3011536
0.0001313	42.0970599	320740.	14.3949301	0.0018893	0.0007737	3.9999929	40.3935625
0.0001338	42.7392242	319546.	14.2207476	0.0019020	0.0007651	3.9999981	40.4873532
0.0001363	43.3687831	318303.	14.0533365	0.0019148	0.0007566	3.9999998	40.5826433
0.0001388	43.9847990	317008.	13.8923461	0.0019276	0.0007482	3.9999956	40.6794935
0.0001413	44.5867944	315659.	13.7374432	0.0019404	0.0007398	3.9999640	40.7779366
0.0001438	45.1747247	314259.	13.5883118	0.0019533	0.0007314	4.0000000	40.8779782
0.0001463	45.7484334	312810.	13.4446588	0.0019663	0.0007232	3.9999993	40.9796313
0.0001488	46.3080893	311315.	13.3062062	0.0019793	0.0007149	3.9999964	41.0828879
0.0001588	48.4114361	304954.	12.7994180	0.0020319	0.0006825	3.9998977	41.5116366
0.0001688	50.3110782	298140.	12.3576712	0.0020854	0.0006510	3.9999110	41.9647445
0.0001788	52.0270462	291060.	11.9697867	0.0021396	0.0006202	3.9998226	42.4408477
0.0001888	53.5773454	283853.	11.6269659	0.0021946	0.0005902	3.9999769	42.9386460
0.0001988	54.9817537	276638.	11.3221377	0.0022503	0.0005609	3.9997528	43.4565872
0.0002088	56.2534224	269477.	11.0496666	0.0023066	0.0005322	3.9997677	43.9936107
0.0002188	57.4087259	262440.	10.8048891	0.0023636	0.0005042	3.9997321	44.5482816
0.0002288	58.4601325	255563.	10.5839994	0.0024211	0.0004767	3.9999637	45.1194475
0.0002388	59.4153816	248860.	10.3838931	0.0024792	0.0004498	3.9999624	45.7063940
0.0002488	60.2874626	242362.	10.2018799	0.0025377	0.0004233	3.9997610	46.3078020
0.0002588	61.0840644	236074.	10.0357532	0.0025968	0.0003974	3.9999791	46.9228490
0.0002688	61.8115124	229996.	9.8836517	0.0026562	0.0003719	3.9996297	47.5508516
0.0002788	62.4762380	224130.	9.7439752	0.0027161	0.0003468	3.9998573	48.1910760
0.0002888	63.0858101	218479.	9.6153240	0.0027764	0.0003220	3.9999636	48.8426108
0.0002988	63.6445678	213036.	9.4965284	0.0028371	0.0002977	3.9999976	49.5049143
0.0003088	64.1556684	207792.	9.3865865	0.0028981	0.0002737	3.9995443	50.1775907
0.0003188	64.6237975	202741.	9.2845984	0.0029595	0.0002501	3.9993469	50.8600228
0.0003288	65.0533735	197881.	9.1897742	0.0030211	0.0002268	3.9993087	51.5516011
0.0003388	65.4476717	193203.	9.1014317	0.0030831	0.0002037	3.9994048	52.2518563
0.0003488	65.8093470	188701.	9.0189751	0.0031454	0.0001810	3.9999993	52.9603956
0.0003588	66.1397228	184362.	8.9418929	0.0032079	0.0001585	3.9999736	53.6770342
0.0003688	66.4427266	180184.	8.8696922	0.0032707	0.0001363	3.9998791	54.4011624
0.0003788	66.7212163	176162.	8.8019438	0.0033337	0.0001144	3.9996616	55.1323170
0.0003888	66.9777872	172290.	8.7382651	0.0033970	0.00009263	3.9992514	55.8700578
0.0003988	67.2138697	168561.	8.6783278	0.0034605	0.00007111	3.9999962	56.6141283
0.0004088	67.4281927	164962.	8.6218773	0.0035242	0.00004982	3.9998269	57.3647621
0.0004188	67.6261514	161495.	8.5685837	0.0035881	0.00002872	3.9993267	58.1210052
0.0004288	67.8089149	158155.	8.5182080	0.0036522	0.00000781	3.9999971	58.8826109
0.0004388	67.9729807	154924.	8.4706020	0.0037165	-0.00001290	3.9997181	59.6502385
0.0004488	68.1249731	151811.	8.4254809	0.0037809	-0.00003344	3.9988777	60.4225944
0.0004588	68.1249731	148501.	8.3934572	0.0038505	-0.00004888	3.9999724	61.3430231

Summary of Results for Nominal Moment Capacity for Section 2

Moment values interpolated at maximum compressive strain = 0.003
or maximum developed moment if pile fails at smaller strains.

Load No.	Axial Thrust kips	Nominal Mom. Cap. in-kip	Max. Comp. Strain	Max. Tens. Strain
1	230.000	64.906	0.00300000	0.00023476

Note that the values of moment capacity in the table above are not factored by a strength reduction factor (phi-factor).

In ACI 318, the value of the strength reduction factor depends on whether the transverse reinforcing steel bars are tied hoops (0.65) or spirals (0.75).

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to ACI 318, or the value required by the design standard being followed.

The following table presents factored moment capacities and corresponding bending stiffnesses computed for common resistance factor values used for reinforced concrete sections.

Axial Load No.	Resist. Factor	Nominal Ax. Thrust kips	Nominal Moment Cap in-kips	Ult. (Fac) Ax. Thrust kips	Ult. (Fac) Moment Cap in-kips	Bend. Stiff. at Ult Mom kip-in^2
1	0.65	230.000000	64.906136	129.738471*	42.188989	320569.
1	0.75	230.000000	64.906136	159.054375*	48.679602	303992.
1	0.90	230.000000	64.906136	207.000000	58.415523	255855.

* Maximum axial load including effects of accidental eccentricity (ACI 318)

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	987.4977
2	2.0000	3.8933	No	No	987.4977	1781.
3	7.0000	3.3614	No	No	2769.	53164.
4	14.0000	14.0000	No	Yes	N.A.	N.A.

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

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 0.500000 inches

Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 230000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.5000	-639082.	9022.	0.00	0.00	4.87E+09	0.00	0.00	0.00
0.2300	0.4995	-614102.	8998.	-3.55E-04	0.00	4.87E+09	-8.288	45.7937	0.00
0.4600	0.4980	-588964.	8961.	-6.95E-04	0.00	4.89E+09	-18.075	100.1682	0.00
0.6900	0.4957	-563753.	8897.	-0.00102	0.00	4.90E+09	-28.536	158.8986	0.00
0.9200	0.4924	-538557.	8804.	-0.00133	0.00	4.91E+09	-39.077	219.0291	0.00
1.1500	0.4883	-513467.	8682.	-0.00163	0.00	4.92E+09	-49.064	277.3125	0.00
1.3800	0.4834	-488568.	8533.	-0.00191	0.00	4.93E+09	-58.961	336.6180	0.00
1.6100	0.4778	-463945.	8358.	-0.00217	0.00	4.93E+09	-67.915	392.3112	0.00
1.8400	0.4714	-439674.	8159.	-0.00243	0.00	4.93E+09	-76.018	445.0424	0.00
2.0700	0.4644	-415826.	8034.	-0.00267	0.00	4.93E+09	-14.733	87.5570	0.00
2.3000	0.4567	-391943.	7993.	-0.00289	0.00	4.93E+09	-14.651	88.5354	0.00
2.5300	0.4484	-368032.	7953.	-0.00310	0.00	4.93E+09	-14.562	89.6221	0.00
2.7600	0.4396	-344101.	7913.	-0.00330	0.00	4.93E+09	-14.465	90.8210	0.00
2.9900	0.4302	-320158.	7873.	-0.00349	0.00	4.93E+09	-14.362	92.1366	0.00
3.2200	0.4203	-296211.	7834.	-0.00366	0.00	4.93E+09	-14.251	93.5742	0.00
3.4500	0.4100	-272267.	7795.	-0.00382	0.00	4.93E+09	-14.133	95.1399	0.00
3.6800	0.3993	-248335.	7756.	-0.00397	0.00	4.93E+09	-14.009	96.8406	0.00
3.9100	0.3881	-224421.	7717.	-0.00410	0.00	4.93E+09	-13.877	98.6845	0.00
4.1400	0.3766	-200532.	7679.	-0.00422	0.00	4.93E+09	-13.739	100.6803	0.00
4.3700	0.3648	-176678.	7642.	-0.00432	0.00	4.93E+09	-13.594	102.8384	0.00
4.6000	0.3528	-152864.	7604.	-0.00441	0.00	4.93E+09	-13.443	105.1701	0.00
4.8300	0.3405	-129098.	7567.	-0.00449	0.00	4.93E+09	-13.284	107.6884	0.00
5.0600	0.3280	-105388.	7531.	-0.00456	0.00	4.93E+09	-13.120	110.4078	0.00
5.2900	0.3153	-81740.	7495.	-0.00461	0.00	4.93E+09	-12.949	113.3445	0.00
5.5200	0.3025	-58161.	7459.	-0.00465	0.00	4.93E+09	-12.771	116.5171	0.00
5.7500	0.2896	-34660.	7424.	-0.00468	0.00	4.93E+09	-12.587	119.9462	0.00
5.9800	0.2767	-11242.	7390.	-0.00469	0.00	4.93E+09	-12.397	123.6552	0.00
6.2100	0.2638	12086.	7356.	-0.00469	0.00	4.93E+09	-12.201	127.6706	0.00
6.4400	0.2508	35316.	7323.	-0.00468	0.00	4.93E+09	-11.998	132.0220	0.00
6.6700	0.2379	58443.	7290.	-0.00465	0.00	4.93E+09	-11.789	136.7433	0.00
6.9000	0.2252	81458.	7258.	-0.00461	0.00	4.93E+09	-11.574	141.8727	0.00
7.1300	0.2125	104357.	7089.	-0.00456	0.00	4.93E+09	-110.398	1434.	0.00
7.3600	0.2000	126378.	6774.	-0.00449	0.00	4.93E+09	-118.298	1633.	0.00
7.5900	0.1877	147453.	6437.	-0.00442	0.00	4.93E+09	-125.546	1846.	0.00
7.8200	0.1756	167519.	6079.	-0.00433	0.00	4.93E+09	-134.088	2107.	0.00
8.0500	0.1638	186504.	5697.	-0.00423	0.00	4.93E+09	-142.368	2399.	0.00
8.2800	0.1523	204338.	5293.	-0.00412	0.00	4.93E+09	-150.260	2724.	0.00
8.5100	0.1411	220956.	4869.	-0.00400	0.00	4.93E+09	-157.564	3083.	0.00
8.7400	0.1302	236294.	4425.	-0.00387	0.00	4.93E+09	-164.177	3481.	0.00
8.9700	0.1197	250298.	3964.	-0.00374	0.00	4.93E+09	-170.015	3921.	0.00
9.2000	0.1095	262918.	3487.	-0.00359	0.00	4.93E+09	-175.005	4409.	0.00
9.4300	0.09983	274112.	2999.	-0.00344	0.00	4.93E+09	-179.076	4951.	0.00
9.6600	0.09054	283844.	2500.	-0.00329	0.00	4.93E+09	-182.163	5553.	0.00
9.8900	0.08168	292087.	1995.	-0.00313	0.00	4.93E+09	-184.210	6225.	0.00
10.1200	0.07328	298824.	1485.	-0.00296	0.00	4.93E+09	-185.165	6974.	0.00
10.3500	0.06533	304044.	974.0872	-0.00279	0.00	4.93E+09	-184.988	7815.	0.00
10.5800	0.05786	307747.	465.3784	-0.00262	0.00	4.93E+09	-183.642	8760.	0.00
10.8100	0.05086	309942.	-37.969	-0.00245	0.00	4.93E+09	-181.103	9828.	0.00
11.0400	0.04434	310647.	-532.639	-0.00228	0.00	4.93E+09	-177.353	11040.	0.00
11.2700	0.03830	309890.	-1015.	-0.00210	0.00	4.93E+09	-172.386	12423.	0.00
11.5000	0.03274	307711.	-1483.	-0.00193	0.00	4.93E+09	-166.204	14013.	0.00
11.7300	0.02765	304156.	-1931.	-0.00176	0.00	4.93E+09	-158.820	15854.	0.00
11.9600	0.02303	299284.	-2358.	-0.00159	0.00	4.93E+09	-150.257	18006.	0.00
12.1900	0.01888	293160.	-2759.	-0.00142	0.00	4.93E+09	-140.551	20551.	0.00
12.4200	0.01517	285862.	-3132.	-0.00126	0.00	4.93E+09	-129.748	23601.	0.00
12.6500	0.01191	277474.	-3461.	-0.00110	0.00	4.93E+09	-108.495	25138.	0.00
12.8800	0.00908	268160.	-3727.	-9.51E-04	0.00	4.93E+09	-84.195	25595.	0.00
13.1100	0.00666	258111.	-3930.	-8.04E-04	0.00	4.93E+09	-62.864	26052.	0.00
13.3400	0.00464	247490.	-4078.	-6.63E-04	0.00	4.93E+09	-44.560	26509.	0.00
13.5700	0.00300	236443.	-4180.	-5.28E-04	0.00	4.93E+09	-29.319	26966.	0.00
13.8000	0.00173	225088.	-4244.	-3.98E-04	0.00	4.93E+09	-17.163	27423.	0.00
14.0300	8.01E-04	213522.	-9797.	-2.76E-04	0.00	4.93E+09	-4007.	1.38E+07	0.00
14.2600	2.05E-04	171360.	-16740.	-1.68E-04	0.00	4.93E+09	-1025.	1.38E+07	0.00
14.4900	-1.27E-04	121329.	-17280.	-8.63E-05	0.00	4.93E+09	634.1533	1.38E+07	0.00

14.7200	-2.71E-04	76085.	-14532.	-3.11E-05	0.00	4.93E+09	1357.	1.38E+07	0.00
14.9500	-2.98E-04	41150.	-10601.	1.71E-06	0.00	4.93E+09	1492.	1.38E+07	0.00
15.1800	-2.62E-04	17565.	-6735.	1.81E-05	0.00	4.93E+09	1310.	1.38E+07	0.00
15.4100	-1.98E-04	3951.	-3559.	2.42E-05	0.00	4.93E+09	991.6229	1.38E+07	0.00
15.6400	-1.29E-04	-2111.	-1303.	2.47E-05	0.00	4.93E+09	643.0494	1.38E+07	0.00
15.8700	-6.22E-05	-3273.	13.2121	2.32E-05	0.00	4.93E+09	310.7930	1.38E+07	0.00
16.1000	-7.68E-07	-2067.	447.4078	1.40E-05	0.00	3.44E+08	3.8415	1.38E+07	0.00
16.3300	1.49E-05	-821.171	349.8391	2.30E-06	0.00	3.35E+08	-74.543	1.38E+07	0.00
16.5600	1.19E-05	-138.851	164.6545	-1.68E-06	0.00	3.20E+08	-59.648	1.38E+07	0.00
16.7900	5.65E-06	89.8511	43.3592	-1.89E-06	0.00	3.20E+08	-28.247	1.38E+07	0.00
17.0200	1.51E-06	102.8891	-6.008	-1.06E-06	0.00	3.20E+08	-7.527	1.38E+07	0.00
17.2500	-1.92E-07	58.0295	-15.067	-3.65E-07	0.00	3.20E+08	0.9624	1.38E+07	0.00
17.4800	-5.11E-07	20.1838	-10.215	-2.83E-08	0.00	3.20E+08	2.5531	1.38E+07	0.00
17.7100	-3.49E-07	1.6763	-4.285	6.58E-08	0.00	3.20E+08	1.7444	1.38E+07	0.00
17.9400	-1.47E-07	-3.552	-0.861	5.77E-08	0.00	3.20E+08	0.7364	1.38E+07	0.00
18.1700	-3.02E-08	-3.152	0.3630	2.89E-08	0.00	3.20E+08	0.1508	1.38E+07	0.00
18.4000	1.21E-08	-1.585	0.4878	8.46E-09	0.00	3.20E+08	-0.06028	1.38E+07	0.00
18.6300	1.66E-08	-0.469	0.2903	-3.86E-10	0.00	3.20E+08	-0.08287	1.38E+07	0.00
18.8600	9.93E-09	0.01758	0.1074	-2.33E-09	0.00	3.20E+08	-0.04964	1.38E+07	0.00
19.0900	3.70E-09	0.1264	0.01338	-1.71E-09	0.00	3.20E+08	-0.01850	1.38E+07	0.00
19.3200	4.78E-10	0.09359	-0.01546	-7.64E-10	0.00	3.20E+08	-0.00239	1.38E+07	0.00
19.5500	-5.19E-10	0.04205	-0.01517	-1.80E-10	0.00	3.20E+08	0.00260	1.38E+07	0.00
19.7800	-5.17E-10	0.01005	-0.00803	4.42E-11	0.00	3.20E+08	0.00258	1.38E+07	0.00
20.0100	-2.75E-10	-0.00232	-0.00257	7.75E-11	0.00	3.20E+08	0.00138	1.38E+07	0.00
20.2400	-8.87E-11	-0.00421	-5.73E-05	4.94E-11	0.00	3.20E+08	4.43E-04	1.38E+07	0.00
20.4700	-2.56E-12	-0.00270	5.72E-04	1.96E-11	0.00	3.20E+08	1.28E-05	1.38E+07	0.00
20.7000	1.93E-11	-0.00108	4.57E-04	3.28E-12	0.00	3.20E+08	-9.66E-05	1.38E+07	0.00
20.9300	1.56E-11	-1.85E-04	2.16E-04	-2.16E-12	0.00	3.20E+08	-7.79E-05	1.38E+07	0.00
21.1600	7.41E-12	1.16E-04	5.73E-05	-2.46E-12	0.00	3.20E+08	-3.70E-05	1.38E+07	0.00
21.3900	1.99E-12	1.34E-04	-7.52E-06	-1.39E-12	0.00	3.20E+08	-9.95E-06	1.38E+07	0.00
21.6200	0.00	7.59E-05	-1.96E-05	0.00	0.00	3.20E+08	1.20E-06	1.38E+07	0.00
21.8500	0.00	2.65E-05	-1.33E-05	0.00	0.00	3.20E+08	3.32E-06	1.38E+07	0.00
22.0800	0.00	2.33E-06	-5.59E-06	0.00	0.00	3.20E+08	2.29E-06	1.38E+07	0.00
22.3100	0.00	-4.43E-06	-1.06E-06	0.00	0.00	3.20E+08	9.86E-07	1.38E+07	0.00
22.5400	0.00	-3.64E-06	5.79E-07	0.00	0.00	3.20E+08	2.05E-07	1.38E+07	0.00
22.7700	0.00	-1.28E-06	6.65E-07	0.00	0.00	3.20E+08	-1.43E-07	1.38E+07	0.00
23.0000	0.00	0.00	0.00	0.00	0.00	3.20E+08	-3.39E-07	6900000.	0.00

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

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     0.50000000 inches
Computed slope at pile head      =       0.000000 radians
Maximum bending moment           =       -639082. inch-lbs
Maximum shear force              =        -17280. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =    14.49000000 feet below pile head
Number of iterations             =              7
Number of zero deflection points =              8
```

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

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

```
Displacement of pile head                        =     1.000000 inches
Rotation of pile head                            =    0.000E+00 radians
Axial load on pile head                          =     230000.0 lbs
```

Depth X	Deflect. y	Bending Moment	Shear Force	Slope S	Total Stress	Bending Stiffness	Soil Res. p	Soil Spr. Es*H	Distrib. Lat. Load

feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
0.00	1.0000	-1174186.	15361.	0.00	0.00	4.65E+09	0.00	0.00	0.00
0.2300	0.9990	-1131608.	15335.	-6.85E-04	0.00	4.65E+09	-8.288	22.8960	0.00
0.4600	0.9962	-1088667.	15299.	-0.00134	0.00	4.68E+09	-18.075	50.0770	0.00
0.6900	0.9916	-1045455.	15234.	-0.00197	0.00	4.69E+09	-28.536	79.4248	0.00
0.9200	0.9853	-1002071.	15141.	-0.00257	0.00	4.71E+09	-39.077	109.4562	0.00
1.1500	0.9774	-958611.	15019.	-0.00315	0.00	4.72E+09	-49.064	138.5437	0.00
1.3800	0.9680	-915169.	14870.	-0.00369	0.00	4.74E+09	-58.961	168.1167	0.00
1.6100	0.9570	-871839.	14695.	-0.00421	0.00	4.76E+09	-67.915	195.8568	0.00
1.8400	0.9447	-828705.	14497.	-0.00470	0.00	4.78E+09	-76.018	222.0860	0.00
2.0700	0.9311	-785846.	14366.	-0.00517	0.00	4.80E+09	-18.577	55.0676	0.00
2.3000	0.9162	-742842.	14315.	-0.00561	0.00	4.82E+09	-18.477	55.6626	0.00
2.5300	0.9001	-699708.	14264.	-0.00602	0.00	4.84E+09	-18.369	56.3229	0.00
2.7600	0.8830	-656462.	14214.	-0.00641	0.00	4.86E+09	-18.251	57.0505	0.00
2.9900	0.8648	-613117.	14163.	-0.00677	0.00	4.88E+09	-18.125	57.8479	0.00
3.2200	0.8456	-569691.	14114.	-0.00710	0.00	4.90E+09	-17.990	58.7180	0.00
3.4500	0.8256	-526198.	14064.	-0.00741	0.00	4.92E+09	-17.847	59.6642	0.00
3.6800	0.8047	-482653.	14015.	-0.00769	0.00	4.93E+09	-17.696	60.6905	0.00
3.9100	0.7831	-439072.	13966.	-0.00795	0.00	4.93E+09	-17.536	61.8012	0.00
4.1400	0.7609	-395468.	13918.	-0.00818	0.00	4.93E+09	-17.368	63.0016	0.00
4.3700	0.7380	-351856.	13871.	-0.00839	0.00	4.93E+09	-17.192	64.2975	0.00
4.6000	0.7145	-308250.	13823.	-0.00858	0.00	4.93E+09	-17.008	65.6954	0.00
4.8300	0.6906	-264665.	13777.	-0.00874	0.00	4.93E+09	-16.816	67.2027	0.00
5.0600	0.6663	-221113.	13731.	-0.00887	0.00	4.93E+09	-16.617	68.8277	0.00
5.2900	0.6417	-177610.	13685.	-0.00898	0.00	4.93E+09	-16.409	70.5797	0.00
5.5200	0.6167	-134168.	13640.	-0.00907	0.00	4.93E+09	-16.194	72.4690	0.00
5.7500	0.5916	-90802.	13596.	-0.00913	0.00	4.93E+09	-15.971	74.5075	0.00
5.9800	0.5663	-47526.	13552.	-0.00917	0.00	4.93E+09	-15.740	76.7082	0.00
6.2100	0.5410	-4352.	13509.	-0.00919	0.00	4.93E+09	-15.502	79.0862	0.00
6.4400	0.5156	38705.	13466.	-0.00918	0.00	4.93E+09	-15.255	81.6582	0.00
6.6700	0.4903	81632.	13424.	-0.00914	0.00	4.93E+09	-15.002	84.4432	0.00
6.9000	0.4652	124415.	13383.	-0.00909	0.00	4.93E+09	-14.741	87.4626	0.00
7.1300	0.4402	167043.	13144.	-0.00900	0.00	4.93E+09	-158.772	995.5317	0.00
7.3600	0.4155	208401.	12679.	-0.00890	0.00	4.93E+09	-178.141	1183.	0.00
7.5900	0.3911	248328.	12159.	-0.00877	0.00	4.93E+09	-198.520	1401.	0.00
7.8200	0.3670	286655.	11580.	-0.00862	0.00	4.93E+09	-221.305	1664.	0.00
8.0500	0.3435	323195.	10945.	-0.00845	0.00	4.93E+09	-238.569	1917.	0.00
8.2800	0.3204	357802.	10272.	-0.00826	0.00	4.93E+09	-249.624	2150.	0.00
8.5100	0.2979	390381.	9569.	-0.00805	0.00	4.93E+09	-259.604	2405.	0.00
8.7400	0.2760	420843.	8840.	-0.00782	0.00	4.93E+09	-268.454	2685.	0.00
8.9700	0.2547	449112.	8089.	-0.00758	0.00	4.93E+09	-276.138	2992.	0.00
9.2000	0.2341	475117.	7317.	-0.00732	0.00	4.93E+09	-282.637	3332.	0.00
9.4300	0.2143	498800.	6530.	-0.00705	0.00	4.93E+09	-287.953	3709.	0.00
9.6600	0.1952	520112.	5730.	-0.00676	0.00	4.92E+09	-292.108	4130.	0.00
9.8900	0.1769	539014.	4919.	-0.00647	0.00	4.91E+09	-295.146	4604.	0.00
10.1200	0.1595	555476.	4102.	-0.00616	0.00	4.91E+09	-297.127	5141.	0.00
10.3500	0.1429	569476.	3281.	-0.00584	0.00	4.90E+09	-297.757	5749.	0.00
10.5800	0.1273	581003.	2461.	-0.00552	0.00	4.90E+09	-296.541	6432.	0.00
10.8100	0.1125	590064.	1647.	-0.00519	0.00	4.89E+09	-293.419	7200.	0.00
11.0400	0.09862	596679.	843.7682	-0.00485	0.00	4.89E+09	-288.343	8070.	0.00
11.2700	0.08569	600883.	57.6874	-0.00451	0.00	4.89E+09	-281.281	9060.	0.00
11.5000	0.07370	602729.	-706.133	-0.00417	0.00	4.89E+09	-272.212	10194.	0.00
11.7300	0.06265	602285.	-1442.	-0.00383	0.00	4.89E+09	-261.134	11505.	0.00
11.9600	0.05253	599636.	-2145.	-0.00349	0.00	4.89E+09	-248.059	13032.	0.00
12.1900	0.04336	594883.	-2809.	-0.00316	0.00	4.89E+09	-233.017	14833.	0.00
12.4200	0.03511	588141.	-3428.	-0.00282	0.00	4.89E+09	-216.057	16987.	0.00
12.6500	0.02777	579543.	-3999.	-0.00249	0.00	4.90E+09	-197.251	19605.	0.00
12.8800	0.02134	569234.	-4515.	-0.00217	0.00	4.90E+09	-176.695	22857.	0.00
13.1100	0.01579	557376.	-4964.	-0.00185	0.00	4.90E+09	-149.021	26052.	0.00
13.3400	0.01110	544184.	-5317.	-0.00154	0.00	4.91E+09	-106.656	26509.	0.00
13.5700	0.00727	529986.	-5562.	-0.00124	0.00	4.91E+09	-70.990	26966.	0.00
13.8000	0.00425	515057.	-5719.	-9.49E-04	0.00	4.92E+09	-42.216	27423.	0.00
14.0300	0.00203	499624.	-19780.	-6.64E-04	0.00	4.93E+09	-10147.	1.38E+07	0.00
14.2600	5.82E-04	406716.	-37797.	-4.11E-04	0.00	4.93E+09	-2909.	1.38E+07	0.00
14.4900	-2.38E-04	291508.	-40169.	-2.15E-04	0.00	4.93E+09	1189.	1.38E+07	0.00
14.7200	-6.07E-04	185256.	-34336.	-8.21E-05	0.00	4.93E+09	3037.	1.38E+07	0.00
14.9500	-6.91E-04	102076.	-25377.	-1.73E-06	0.00	4.93E+09	3455.	1.38E+07	0.00
15.1800	-6.17E-04	45179.	-16351.	3.95E-05	0.00	4.93E+09	3085.	1.38E+07	0.00
15.4100	-4.73E-04	11767.	-8828.	5.54E-05	0.00	4.93E+09	2366.	1.38E+07	0.00
15.6400	-3.11E-04	-3623.	-3414.	5.77E-05	0.00	4.93E+09	1557.	1.38E+07	0.00

15.8700	-1.55E-04	-7154.	-196.852	5.46E-05	0.00	4.93E+09	774.9510	1.38E+07	0.00
16.1000	-9.71E-06	-4779.	939.5894	3.37E-05	0.00	3.48E+08	48.5573	1.38E+07	0.00
16.3300	3.09E-05	-2010.	793.4250	6.66E-06	0.00	3.44E+08	-154.474	1.38E+07	0.00
16.5600	2.70E-05	-407.366	393.7296	-3.15E-06	0.00	3.21E+08	-135.161	1.38E+07	0.00
16.7900	1.35E-05	167.6400	114.0678	-4.18E-06	0.00	3.20E+08	-67.493	1.38E+07	0.00
17.0200	3.95E-06	227.5974	-6.331	-2.48E-06	0.00	3.20E+08	-19.753	1.38E+07	0.00
17.2500	-1.86E-07	135.8412	-32.305	-9.14E-07	0.00	3.20E+08	0.9309	1.38E+07	0.00
17.4800	-1.09E-06	50.4331	-23.477	-1.11E-07	0.00	3.20E+08	5.4662	1.38E+07	0.00
17.7100	-8.01E-07	6.3884	-10.405	1.33E-07	0.00	3.20E+08	4.0062	1.38E+07	0.00
17.9400	-3.57E-07	-7.174	-2.411	1.30E-07	0.00	3.20E+08	1.7867	1.38E+07	0.00
18.1700	-8.40E-08	-7.086	0.6342	6.85E-08	0.00	3.20E+08	0.4201	1.38E+07	0.00
18.4000	2.08E-08	-3.760	1.0701	2.18E-08	0.00	3.20E+08	-0.104	1.38E+07	0.00
18.6300	3.63E-08	-1.207	0.6757	4.07E-10	0.00	3.20E+08	-0.182	1.38E+07	0.00
18.8600	2.31E-08	-0.03054	0.2657	-4.92E-09	0.00	3.20E+08	-0.115	1.38E+07	0.00
19.0900	9.14E-09	0.2663	0.04330	-3.91E-09	0.00	3.20E+08	-0.04572	1.38E+07	0.00
19.3200	1.53E-09	0.2134	-0.03034	-1.84E-09	0.00	3.20E+08	-0.00764	1.38E+07	0.00
19.5500	-1.01E-09	0.1012	-0.03388	-4.86E-10	0.00	3.20E+08	0.00507	1.38E+07	0.00
19.7800	-1.15E-09	0.02700	-0.01893	6.65E-11	0.00	3.20E+08	0.00576	1.38E+07	0.00
20.0100	-6.48E-10	-0.00341	-0.00651	1.68E-10	0.00	3.20E+08	0.00324	1.38E+07	0.00
20.2400	-2.24E-10	-0.00913	-4.87E-04	1.14E-10	0.00	3.20E+08	0.00112	1.38E+07	0.00
20.4700	-1.80E-11	-0.00624	0.00119	4.79E-11	0.00	3.20E+08	9.02E-05	1.38E+07	0.00
20.7000	4.00E-11	-0.00264	0.00104	9.66E-12	0.00	3.20E+08	-2.00E-04	1.38E+07	0.00
20.9300	3.53E-11	-5.42E-04	5.16E-04	-4.04E-12	0.00	3.20E+08	-1.76E-04	1.38E+07	0.00
21.1600	1.77E-11	2.15E-04	1.51E-04	-5.44E-12	0.00	3.20E+08	-8.84E-05	1.38E+07	0.00
21.3900	5.21E-12	2.96E-04	-7.48E-06	-3.24E-12	0.00	3.20E+08	-2.61E-05	1.38E+07	0.00
21.6200	0.00	1.78E-04	-4.20E-05	-1.20E-12	0.00	3.20E+08	1.08E-06	1.38E+07	0.00
21.8500	-1.42E-12	6.62E-05	-3.06E-05	0.00	0.00	3.20E+08	7.11E-06	1.38E+07	0.00
22.0800	-1.05E-12	8.66E-06	-1.36E-05	0.00	0.00	3.20E+08	5.27E-06	1.38E+07	0.00
22.3100	0.00	-8.86E-06	-3.00E-06	0.00	0.00	3.20E+08	2.39E-06	1.38E+07	0.00
22.5400	0.00	-8.12E-06	1.09E-06	0.00	0.00	3.20E+08	5.71E-07	1.38E+07	0.00
22.7700	0.00	-2.98E-06	1.48E-06	0.00	0.00	3.20E+08	-2.86E-07	1.38E+07	0.00
23.0000	0.00	0.00	0.00	0.00	0.00	3.20E+08	-7.88E-07	6900000.	0.00

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

Output Summary for Load Case No. 2:

```
Pile-head deflection             =     1.00000000 inches
Computed slope at pile head      =       0.000000 radians
Maximum bending moment           =      -1174186. inch-lbs
Maximum shear force              =         -40169. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =    14.49000000 feet below pile head
Number of iterations             =              8
Number of zero deflection points =              8
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

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

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	0.5000	S, rad	0.00	230000.	0.5000	0.00	-17280.	-639082.

```
  2  y, in     1.0000   S, rad         0.00    230000.     1.0000       0.00    -40169.  -1174186.

Maximum pile-head deflection = 1.0000000000 inches
Maximum pile-head rotation   = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.
```

∘ Flexural Stiffness of Casing with Threaded Joint Reduction

- From FHWA Micropile Manual, Section 5.18.3: Reduce wall thickness by 50% at threaded joints

∴ For 9.625" outer casing with 0.545"-thick wall, ignoring corrosion reduction,

$D = 9.625'' - 0.545'' = 9.08 \text{ in}$

$d = 9.625'' - 2(0.545'') = 8.535 \text{ in}$

Joint moment of inertia, I_{joint}

$$I_{joint} = \frac{\pi}{64}(D^4 - d^4) = \frac{\pi}{64}(9.08\text{ in}^4 - 8.535\text{ in}^4) = 73.18\text{ in}^4$$

From FHWA Micropile Manual, the reduced section modulus (Eq. 5-26)

$$S_{joint} = \frac{I_{joint}}{D/2} = \frac{73.18\text{ in}^4}{\frac{9.08\text{ in}}{2}} = 16.12\text{ in}^3$$

Maximum bending moment at a joint (Eq. 5-27)

$$M_{max\,joint} = S_{joint} \times \left(1 - \frac{f_a}{F_a}\right) \times \left(1 - \frac{f_a}{F_e'}\right) F_b$$

where, f_a = axial stress = P_c / A_{casing}

F_a = allowable axial stress that would be permitted if axial force alone existed
$\doteq 0.47\, F_{y\,casing}$

F_e' = Euler buckling stress = $\dfrac{\pi^2 E}{FS\,(Kl/r)^2}$

AECOM Imagine it. Delivered.

JOB TITLE Tannery Brook Bridge Replacement (#3610)
PROJECT/JOB NO. 60656153 CALCULATION NO.
COMPUTED BY B. Reyes DATE 9/15/23
VERIFIED BY K. Scancarello DATE 9/15/23
SCALE N/A

F_b = allowable bending stress that would be permitted if bending moment alone existed

$= 0.55\ F_y$ casing

So,

Axial stress,

$$f_a = \frac{146\ \text{kips}}{15.55\ \text{in}^2} = 9.39\ \text{ksi}$$

Allowable axial stress,

$$F_a = 0.47\,(80\ \text{ksi}) = 37.6\ \text{ksi}$$

Euler buckling stress,

$$F_e' = \frac{\pi^2\,(29{,}000\ \text{ksi})}{2.12\,(kl/r)^2}$$

since l = unsupported length of the micropile = 0

$$F_e' = \infty$$

Allowable bending stress,

$$F_b = 0.55\,(80\ \text{ksi}) = 44\ \text{ksi}$$

And so,

$$M_{max\ joint} = 16.12\ \text{in}^3 \left(1 - \frac{9.39\ \text{ksi}}{37.6\ \text{ksi}}\right)\left(1 - \overset{0}{\cancel{\frac{9.39\ \text{ksi}}{\infty}}}\right)(44\ \text{ksi})$$

$$M_{max\ joint} = 16.12\ \text{in}^3\,(0.75)(1)(44\ \text{ksi})$$

$$M_{max\ joint} = \underline{\underline{532\ \text{kip-in}}}$$

5x5 = 1 in

AECOM Imagine it. Delivered.

JOB TITLE Tannery Brook Bridge Replacement (#3610)
PROJECT/JOB NO. 60656153 CALCULATION NO.
COMPUTED BY B. Reyes DATE 9/15/23
VERIFIED BY K. Scancarello DATE 9/15/23
SCALE N/A SHEET NO. 3 OF 3

Micropile casing sections are typically 10 feet long, so threaded joint would be 10 feet below the top of the pile.

From LPile results, at a depth of 10 feet

Maximum moment for 0.5 inches of deflection,

$M_{max\ 0.5} = 300$ kip-in

$M_{max\ 0.5} < M_{max\ joint}$ ✓ OK

Maximum moment for 1 inch of deflection,

$M_{max\ 1} = 560$ Kip-in

$M_{max\ 1} > M_{max\ joint}$ ✗ Need more steel

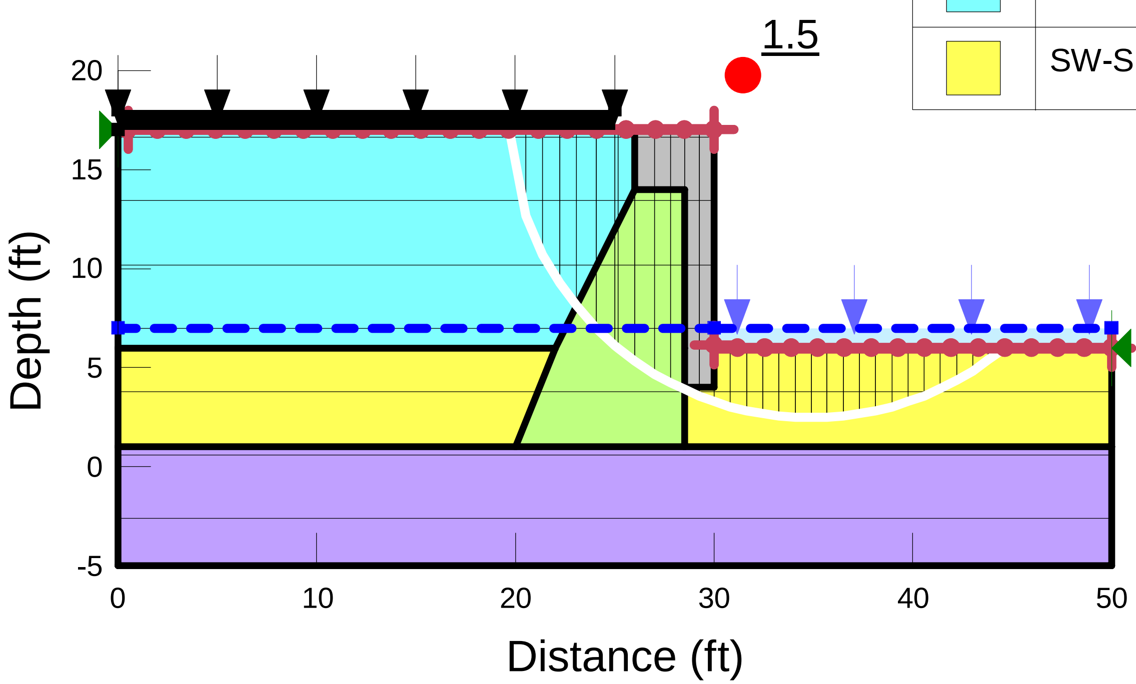
5x5 = 1 in

Stability Analysis

Computed By: B. Reyes Date: 6/28/2022
Checked By: T. Dwyer Date: 6/30/2022

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fascia	High Strength	150		
	Gravel Fill	Mohr-Coulomb	135	0	40
	Reg Fill	Mohr-Coulomb	125	0	32
	SW-SM	Mohr-Coulomb	130	0	36

Tannery Brook Bridge
Fascia Stability Analysis



Tannery Brook Bridge

File Information

File Version: 8.16
Created By: Reyes, Bivian
Last Edited By: Reyes, Bivian
Revision Number: 25
Date: 6/28/2022
Time: 2:22:43 PM
Tool Version: 8.16.5.15361
File Name: Tannery Brook Facia.gsz
Directory: \\na.aecomnet.com\lfs\AMER\Chelmsford-USCHL1\Secure_DCS\Resources\Legacy\Private\AE_Depts\Geot\PROJECT FILES (GEOTECH)\Tannery Brook Bridge Norway ME - 60656153\Calculations\Slope W\
Last Solved Date: 6/28/2022
Last Solved Time: 2:22:46 PM

Project Settings

Length(L) Units: Feet
Time(t) Units: Seconds
Force(F) Units: Pounds
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

Tannery Brook Bridge

- Description: Stability Analysis for Existing Bridge with fascia
- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - PWP Conditions Source: Piezometric Line
  - Apply Phreatic Correction: No
  - Use Staged Rapid Drawdown: No
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Resisting Side Maximum Convex Angle: 1 °
  - Driving Side Maximum Convex Angle: 5 °
  - Optimize Critical Slip Surface Location: No
  - Tension Crack
    - Tension Crack Option: (none)
- F of S Distribution
  - F of S Calculation Option: Constant
- Advanced
  - Number of Slices: 30
  - F of S Tolerance: 0.01

Minimum Slip Surface Depth: 0.1 ft
Search Method: Root Finder
Tolerable difference between starting and converged F of S: 3
Maximum iterations to calculate converged lambda: 20
Max Absolute Lambda: 2

Materials

Fascia

Model: High Strength
Unit Weight: 150 pcf
Pore Water Pressure
Piezometric Line: 1

Gravel Fill

Model: Mohr-Coulomb
Unit Weight: 135 pcf
Cohesion': 0 psf
Phi': 40 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

SW-SM

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion': 0 psf
Phi': 36 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Bedrock

Model: Bedrock (Impenetrable)
Pore Water Pressure
Piezometric Line: 1

Reg Fill

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 0 psf
Phi': 32 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (0.5, 17) ft
Left-Zone Right Coordinate: (30, 17) ft
Left-Zone Increment: 20
Right Projection: Range
Right-Zone Left Coordinate: (30, 6.140566) ft
Right-Zone Right Coordinate: (50, 6) ft
Right-Zone Increment: 15
Radius Increments: 15

Slip Surface Limits

Left Coordinate: (0, 17) ft
Right Coordinate: (50, 6) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
Coordinate 1	0	7
Coordinate 2	30	7
Coordinate 3	50	7

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 250 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	0	17
	0	18
	25	18

Points

	X (ft)	Y (ft)
Point 1	0	1
Point 2	20	1
Point 3	30	6
Point 4	28.5	6
Point 5	28.5	17
Point 6	30	17
Point 7	50	1
Point 8	50	6
Point 9	0	17
Point 10	0	6
Point 11	0	-5
Point 12	50	-5
Point 13	30	7
Point 14	50	7
Point 15	0	7
Point 16	27	6
Point 17	27.8	6
Point 18	28.5	14
Point 19	26	14
Point 20	26	17
Point 21	22	6
Point 22	28.5	1

Point 23	28.5	4
Point 24	30	4

Regions

	Material	Points	Area (ft²)
Region 1	Fascia	5,18,4,3,13,6	16.5
Region 2	Bedrock	1,11,12,7,22,2	300
Region 3	Reg Fill	9,10,21,19,20	270
Region 4	Gravel Fill	21,16,17,4,18,19	36
Region 5	Fascia	20,19,18,5	7.5
Region 6	Gravel Fill	21,2,22,23,4,17,16	37.5
Region 7	Fascia	4,23,24,3	3
Region 8	SW-SM	23,22,7,8,3,24	104.5
Region 9	SW-SM	21,10,1,2	105

Current Slip Surface

Slip Surface: 3,520
F of S: 1.5
Volume: 140.40124 ft³
Weight: 18,791.524 lbs
Resisting Moment: 194,155.91 lbs-ft
Activating Moment: 129,793.32 lbs-ft
Resisting Force: 10,204.145 lbs
Activating Force: 6,838.5172 lbs
F of S Rank (Analysis): 1 of 5,376 slip surfaces
F of S Rank (Query): 1 of 5,376 slip surfaces
Exit: (44.629182, 6) ft
Entry: (19.675, 17) ft
Radius: 15.229583 ft
Center: (34.888166, 17.706955) ft

Slip Slices

	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	20.097873	14.827737	-488.45082	153.81636	96.115132	0
Slice 2	20.943619	11.677483	-291.87496	406.95321	254.29259	0
Slice 3	21.789365	9.9824099	-186.10238	554.10244	346.24163	0
Slice 4	22.635111	8.6906487	-105.49648	674.79335	421.65768	0
Slice 5	23.557789	7.5579846	-34.818237	745.90321	625.88711	0
Slice 6	24.528796	6.5620102	27.330565	900.63596	732.79024	0
Slice 7	25.073575	6.0620102	58.530565	832.20864	649.19299	0
Slice 8	25.573575	5.6700176	82.990904	908.0126	692.27541	0
Slice 9	26.5	5.0097205	124.19344	1,098.9759	817.9396	0
Slice 10	27.4	4.4534084	158.90731	1,236.0072	903.79411	0
Slice 11	28.15	4.0546666	183.78881	1,355.4286	983.1225	0
Slice 12	28.875	3.7206969	204.62852	1,600.3378	1,014.0422	0
Slice 13	29.625	3.4213129	223.31008	1,745.9506	1,106.263	0
Slice 14	30.406366	3.1579734	239.74246	343.21571	75.177715	0
Slice	31.219099	2.9318805	253.85066	404.43413	109.4053	0

15						
Slice 16	32.031831	2.7533471	264.99114	466.57586	146.45987	0
Slice 17	32.844563	2.620679	273.26963	528.78539	185.64306	0
Slice 18	33.657296	2.5326696	278.76142	589.5592	225.8078	0
Slice 19	34.470028	2.4885418	281.51499	646.6875	265.31336	0
Slice 20	35.28276	2.4879127	281.55425	697.26532	302.03177	0
Slice 21	36.095493	2.5307769	278.87952	737.81066	333.43299	0
Slice 22	36.908225	2.6175062	273.46762	764.51727	356.76846	0
Slice 23	37.720957	2.7488657	265.27078	773.64453	369.35515	0
Slice 24	38.53369	2.9260492	254.21453	762.01207	368.93651	0
Slice 25	39.346422	3.1507355	240.1941	727.5266	354.06779	0
Slice 26	40.159154	3.4251746	223.0691	669.63656	324.45025	0
Slice 27	40.971887	3.7523135	202.65564	589.59809	281.13014	0
Slice 28	41.784619	4.1359804	178.71482	490.45185	226.49021	0
Slice 29	42.597352	4.5811576	150.93577	376.6587	163.99731	0
Slice 30	43.410084	5.0943981	118.90956	253.41975	97.727373	0
Slice 31	44.222816	5.6844837	82.08822	125.82156	31.774134	0

Appendix F

Geophysical Test Results

Geophysical Investigation
Main St. (Rt. 117/118) ME Bridge No. 3610

over Tannery (Bird) Brook

Norway, ME

Prepared for

AECOM

June, 2022

NDT CORPORATION
Concrete & Geophysical Testing

NDT CORPORATION
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June 10, 2022

Todd Dwyer, PE
Geotechnical Department Manager
AECOM
250 Apollo Drive
Chelmsford, MA 01824
todd.dwyer@aecom.com

Subject: NDT Report for Nondestructive Testing Investigation to identify the abutment geometry and depth to rock Bridge # 3610 carrying Main St. (Rt. 117) over Tannery (Bird) Brook located in Norway, Maine.

Dear Mr. Dwyer:

NDT Corporation (NDT) is pleased to submit this report to AECOM for nondestructive testing including ground penetrating radar (GPR), seismic refraction and sonic/ultrasonic reflection measurements at the above referenced project location. Based on information provided to NDT by AECOM, the objective of these measurements is to better define the abutment geometry and the bearing strata/bearing elevation (rock or soil) to assist AECOM to evaluate this structure.

We thank you for the opportunity to perform this work and look forward to being of service to you in the future. If you have any questions or require additional information, call the undersigned at 978-573-1327.

Sincerely,

William Horne

William Horne
NDT Corporation

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1.0 INTRODUCTION AND PURPOSE:

The geophysical investigation conducted by NDT Corporation on May 25th, 2022, used ground penetrating radar (GPR), sonic/ultrasonic reflection and seismic refraction measurements to determine the abutment geometry and the top of rock profile along a 100 foot line of coverage along the east abutment of the bridge. The results of this testing will assist AECOM with their evaluation of this structure.

2.0 LOCATION AND SURVEY CONTROL

The general location of the project area is shown in Figure 1. GPR data was collected on top of the bridge in both directions (east to west and west to east) to determine the construction of the abutments. Lines were collected 25-30 feet past each abutment to ensure that the back of the abutment was located. A Seismic line of data was collected along a 100-foot-long line located in the streambed in between the east abutment and center pier. Figure 2 shows the location of each of the GPR and seismic line of coverage. Sonic/ultrasonic measurements were made on the on both the masonry blocks and the concrete portion of the abutment to help determine the thickness of each section.

Elevation data is referenced to the low chord measurement taken at the upstream center pier, low chord measurements, top of water to top of bridge slab, were made at both the up and downstream center pier (upstream 7’, downstream 7’9”). These measurements in addition to the depth to each seismic hydrophone are added to the results of the seismic profile to get a depth of rock profile.

3.0 METHODS OF INVESTIGATION

3.1 Ground Penetrating Radar (GPR)

The ground penetrating radar (GPR) method uses a pulsed electromagnetic signal that is transmitted to and reflected by a “target” (re-bar, wire, moisture, etc.) back to the point of transmission. The wave transmission and reflection are dependent on the contrast in electrical properties (dielectric constant and conductivity) of the material(s) being investigated. Since these electrical properties are affected by moisture content in the concrete; saturated or moist conditions reflect energy, cause high attenuation and limit signal penetration. Metal reinforcing typically are easily detected because it is high conductivity in comparison to concrete which produces strong reflections. GPR targets indicate the location and approximate depths of metal targets; it cannot determine the size of reinforcing.

GPR detects reinforcing perpendicular to the direction of the line of coverage, therefore, to detect longitudinal reinforcing a transverse line of data would be collected and to detect transverse reinforcing, a longitudinal line of data would be collected. From this data an average spacing and time to reflector (rebar) is determined. Using the average time to reflector (rebar) an average signal velocity is used to calculate the depth of cover. Appendix 1 has a more detailed discussion of the GPR survey method.

3.2 Sonic/Ultrasonic Pulse Velocity Testing

The sonic/ultrasonic data were acquired using a system developed by NDT Corporation specifically for testing concrete. This system uses a projectile impact energy source, a 2.5 foot long four sensor array with sensors spaced 0, 0.5, 1.5, and 2.5 feet from the source and a PC for onsite data display for quality assurance as the data is acquired and to archive the data for future playback and processing.

The projectile impact produces stress waves in the concrete; these stress waves are detected by the array of sensors which measure the amplitude of the stress wave in time (time domain data). This data is used to determine the time required for a compressional and shear wave to travel from the impact point to each of the sensors and the frequency content of the recorded signals. Compressional and shear wave velocity values are calculated using the travel times and the distance between the impact point and sensors.

Sonic/ultrasonic, compressional and shear wave transmission velocity values are used to determine the mechanical characteristics of concrete. The transmission velocity values determine the elastic deformational characteristics of the concrete, including the Young's, Bulk, and shear modulus values as well as Poisson's ratio and a calculated strength value. These values are principally controlled by the presence of cracking, voiding or weaker strength concrete. Strength values are expected to be within 500 psi of values determined from laboratory compressive strength test. This accuracy is dependent on the degree and attitude of fracturing. Appendix 2 has a more detailed description of the sonic/ultrasonic testing method.

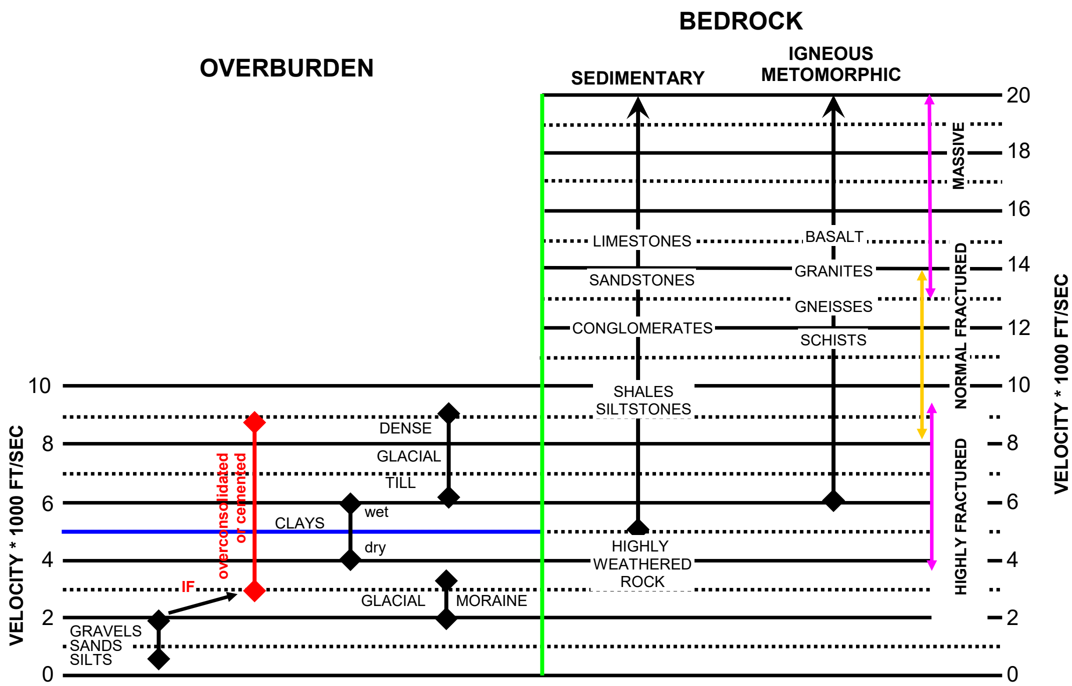
3.3 SEISMIC REFRACTION:

Seismic refraction data was acquired with a 12-channel system with 5 and 10 foot geophone spacing and seismic energy generated approximately every 50 feet with a “seisgun” or sledge hammer energy source. Seismic Refraction utilizes the natural energy transmitting properties of the soils and rocks and is based on the principle that the velocity at which seismic waves travel through the earth is a function of the physical properties (elastic moduli and Poisson’s ratio) of the materials. Refracted compressional wave data are used to evaluate material types and thickness, profile top of bedrock, and to determine the approximate depth to layer interfaces. A more complete discussion of the seismic refraction survey method is included in Appendix 1.

The seismic refraction data were interpreted using the critical distance method. Delayed bedrock wave arrivals were used to portray the bedrock surface more accurately between critical distance depth calculations. The delayed arrivals at individual geophone locations are an indicator of variability in the rock surface. Delayed arrivals indicate thicker overburden over the bedrock. Variations of 3-5 feet are not accurately profiled, particularly in shallow (less than 10 feet deep) bedrock areas.

Overburden with a 1,000-1,500+/- ft/sec velocity is consistent with normally consolidated soils/sands/fill material typical of natural soils, fluvial deposits, and/or construction fill. Till with a 2,000 to 2,600 ft/sec velocity value is consistent with unstratified glacial drift or ground moraine. These tills are typically deposited by receding glaciers consisting of an admixture of clays, sands and gravels with occasional and sometimes frequent boulders associated with an ablation till. Overburden and Till layers with “dry” velocities of less than 5,000 ft/sec are indistinguishable when saturated and assume the velocity of water 5,000+/- ft/sec when saturated. These overburden/soils/tills are susceptible to scour.

Bedrock velocities of less than 10,000 ft/sec are indicative of highly weathered and/or fractured rock typical of sedimentary and low-grade metamorphic rocks such as shales, silt stones and schists. Bedrock with a velocity of 10,000 to 15,000 ft/sec is indicative of competent bedrock that will require drilling and blasting for removal. This velocity range is typical of competent sedimentary and metamorphic rocks such as sandstones, limestones, schists, and gneisses. Bedrock velocities greater than 15,000 ft/sec are indicative of massive bedrock typical in metamorphic and igneous rocks such as gneisses, granites and basalts.



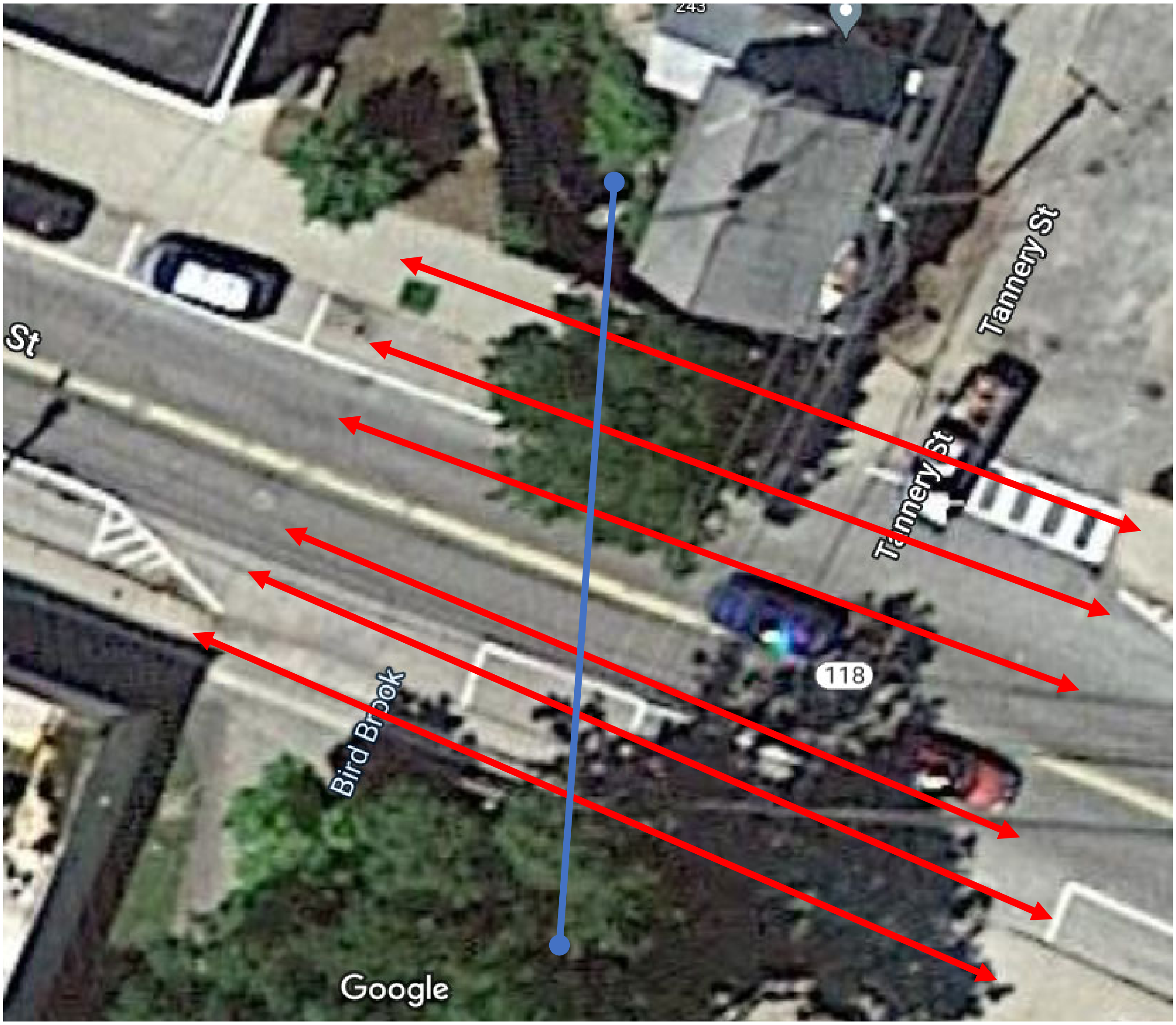
5.0 RESULTS:

Interpretation of the GPR and Sonic/ultrasonic data determined that the masonry portion of the abutments ranges from 54-72 inches in total thickness, and the concrete section of the abutment is 24 inches thick. The GPR data has an approximate depth of penetration 10-12 feet and does not give an indication of a batter or a stepped-out back of abutment to this depth. Figure 3 is an annotated GPR record which shows the abutment thicknesses.

Figure 4 is a seismic refraction profile with the depth of rock. Seismic Refraction results indicate that there is a 5-6 feet of saturated fill/soil materials with a velocity of 5,000+/-ft/sec overlying a bedrock with a seismic velocity of 14,000+/- ft/sec. Measurements from the top of the existing bridge slab to the water/river surface was 7-7.75 feet below this elevation. Seismic data indicates the top of rock surface ranging from 14 to 16 feet from the top of the existing bridge slab (5 to 6 feet below the water surface at the time of the survey).

FIGURES

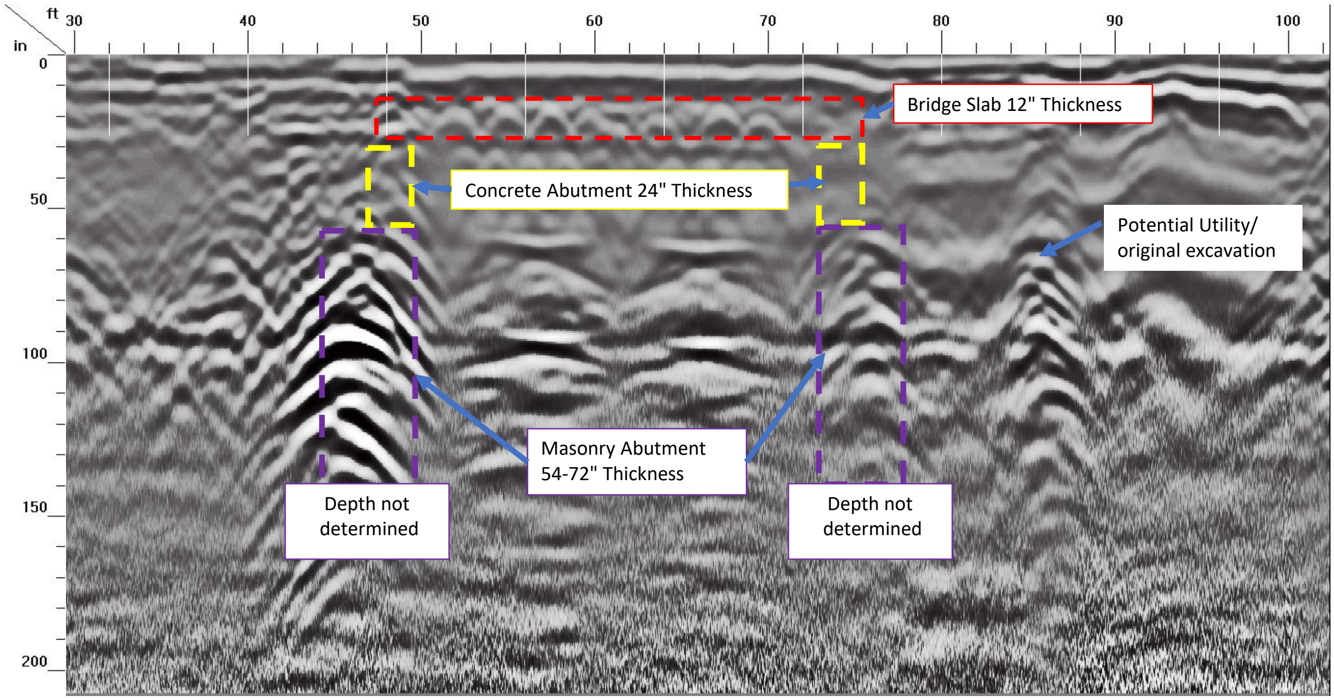
Geophysical Top of Rock Survey Main St. (Routes 117 & 118) crossing Bird Brook Norway, ME prepared for AECOM by NDT Corporation	Area of Investigation	
	June 2022	Figure 1



GPR Line of Coverage

Seismic Refraction Line of Coverage

<table>
<tr><td rowspan="2">Geophysical Top of Rock Survey
Main St. (Routes 117 & 118)
crossing Bird Brook
Norway, ME
prepared for
AECOM
by
NDT Corporation</td><td colspan="2">Lines of Coverage
Map</td></tr>
<tr><td>June
2022</td><td>Figure 2</td></tr>
</table>



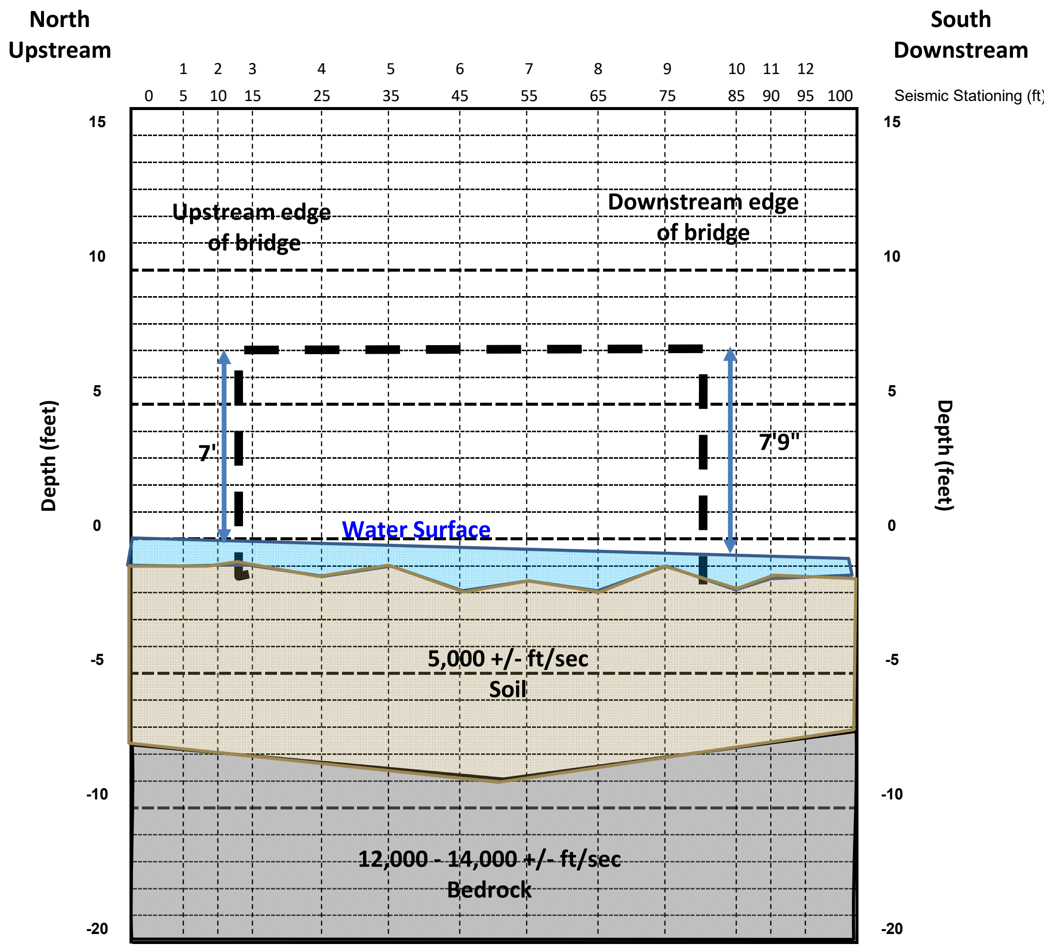
Geophysical Top of Rock Survey Main St. (Routes 117 & 118) crossing Bird Brook Norway, ME prepared for AECOM by NDT Corporation	Annotated GPR Record with Thicknesses	
	June 2022	Figure 3

Main Street over Bird Brook, Norway, ME

Line 1 - Centered between the East abutment and the center pier
0+00 US to Sta. 1+00 DS

Date of Survey: 5/25/22

Field Notes:



Geophysical Top of Rock Survey Main St. (Routes 117 & 118) crossing Bird Brook Norway, ME prepared for AECOM by NDT Corporation	Line 1 Seismic Profile near East Abutment	
	June 2022	Figure 4

APPENDIX 1

GPR Method of Investigation

APPENDIX: GROUND PENETRATING RADAR

Ground Penetrating Radar (GPR) is an electrical geophysical method for evaluating subsurface conditions by transmitting high frequency electromagnetic waves into the ground and detecting the energy reflected back to the surface. Electromagnetic signals are transmitted from the antenna (transmitter and receiver) at ground surface and reflected back to the antenna from interfaces with differing electrical (dielectric constant and conductivity) properties. The greater the contrast in the electrical properties between two materials, the more energy that is reflected to the surface and the more defined results are.

GPR SYSTEM:

GPR systems consist of: Control unit (pulse transmitter, digital recorder, data storage, monitor); and an antenna(s) and survey wheel.

The GPR control unit is a computer which controls data acquisition parameters, such as sampling rate, range, gain control, filtering, etc. The Control Unit also visually displays the data, digitally archives the data, and allows for play back of the data.

Coaxial cable connects the control unit to the antenna. The antenna(s) are sealed and shielded in fiberglass housing.. Selection of the antenna is dictated by the requirements of the survey. For high resolution, near-surface data, a high frequency antenna is used; for deeper penetration investigation, a lower frequency antenna is used. Typically the 100 to 400 MHz antennas are used for geologic surveys; 400 to 900MHz are used for utility, near surface voiding settlement, foundation, etc surveys while the 900 to 1500 MHz are used for concrete reinforcing assessment.

APPLICATIONS

Ground Penetrating Radar (GPR) can be used to locate underground pipes, buried drums, foundations, voids in rock and concrete, soil settlement, determine stratigraphy, depth to water table, buried artifacts, filled excavations, and locate voids/settlement behind walls and under floor slabs, etc. GPR is also a good tool for evaluating concrete structures such as bridges, walls, beams, ceilings, etc where the GPR can locate rebar and conduits, quantify rebar spacing, cover variability over reinforcing, and concrete thickness.

GPR reflections typically occur at subsurface discontinuities such as:

- Buried metal objects (utilities, tanks, reinforcing)
- Open and water filled voids
- Water table
- Soil stratification
- Seepage paths
- Bedrock fractures

DETPH OF PENETRATION AND LIMITATIONS

The depth of penetration of GPR is site specific, limited by the attenuation of the electromagnetic energy. Signal attenuation is controlled by four different mechanisms:

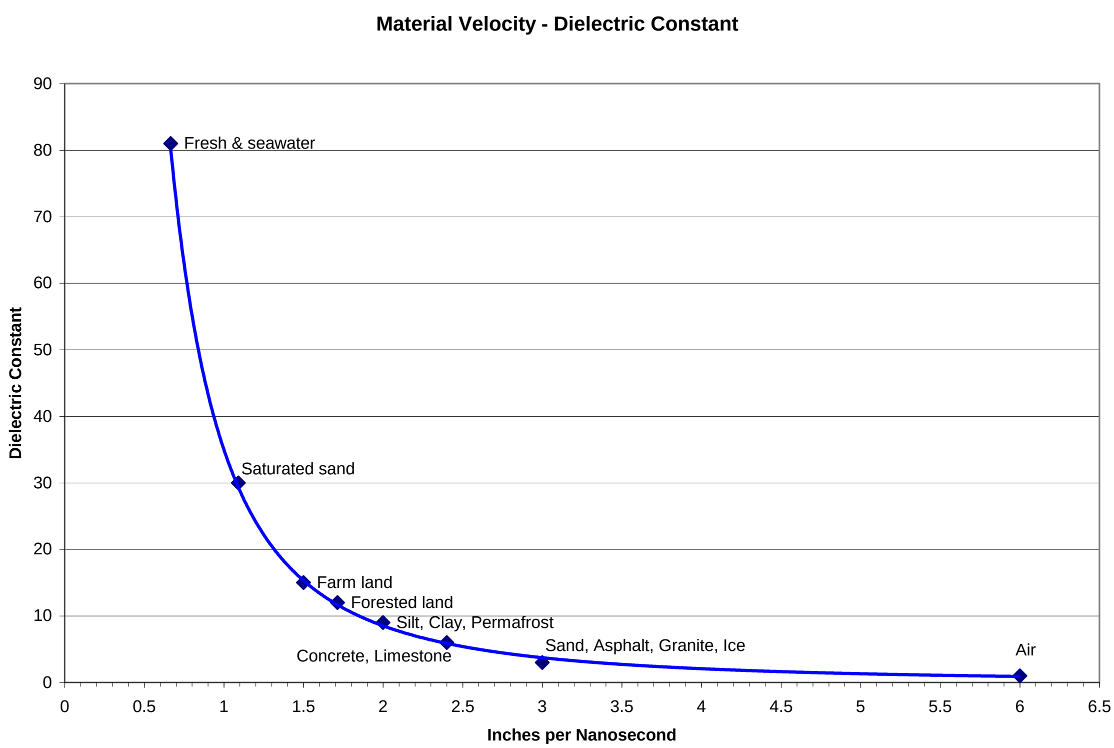
- Scattering: energy losses due to scattering occur when signals are dispersed in random directions, away from the receiving antenna, by closely spaced rebar or large irregular shaped objects, such as boulders or tree stumps.
- High conductivity layers: the greater the conductivity values of materials at a site, the more signal attenuation or less penetration. (Mineral content, high moisture content, water table, metal plates, etc.)

Signal penetration is also dependent on the frequency of the antenna. High frequency antennas have shallow penetration and high resolution. Low frequency antennas have greater depths of penetration, but the resolution of small and near surface targets is reduced. Listed below are antenna frequency, approximate depths of penetration and typical application. (Depths of penetration are in ideal conditions if a highly conductive layer, such as a brackish water table, steel plate, etc., is present all antennas will be limited to the depth of this layer.)

1500 and 1600 MHz	+/-2 feet	Asphalt/Concrete thickness Wire mesh/rebar/conduit location Voiding within and behind structures
900 MHz	3-5 feet	Concrete thickness Rebar and utility location Voiding within and behind structures
400 MHz	10-15 feet	Concrete/Masonry thickness Utility location Soil settlement/sinkhole development Geologic and Environmental mapping Archaeological Surveys
200 MHz	25-30	Soil settlement/sinkhole development

		Geologic and Environmental mapping Archaeological Surveys
100 MHz	+/-50	Soil settlement/sinkhole development Geologic and Environmental mapping Archaeological Surveys

Depth of investigation can be estimated using material dielectric constants and the diagram shown below. Typically 2 inches per nanosecond can be used as an average signal velocity for most materials and sites. When available an onsite depth calibration can be conducted to determine the electrical properties (speed of the signal) of the materials at the site. Depth calibrations typically consist of collecting GPR data over a metal target with a known depth. Known utilities, and buried metal plates are good targets for calibrations. GPR surveys can be very effective when coupled with other geophysical surveys and/or ground truth methods to verify, correlate and extrapolate GPR results. GPR surveys are a fast and cost effective method to collect data over large or obstructed sites, and isolate anomalies and areas where borings or other methods can be focused for the best interest of a project.

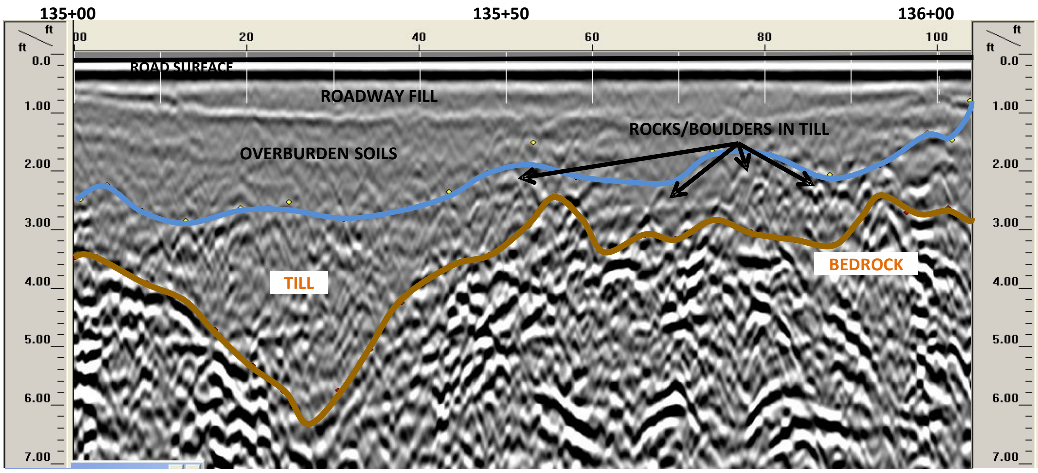


<u>ACQUISITION AND INTERPRETATION:</u>

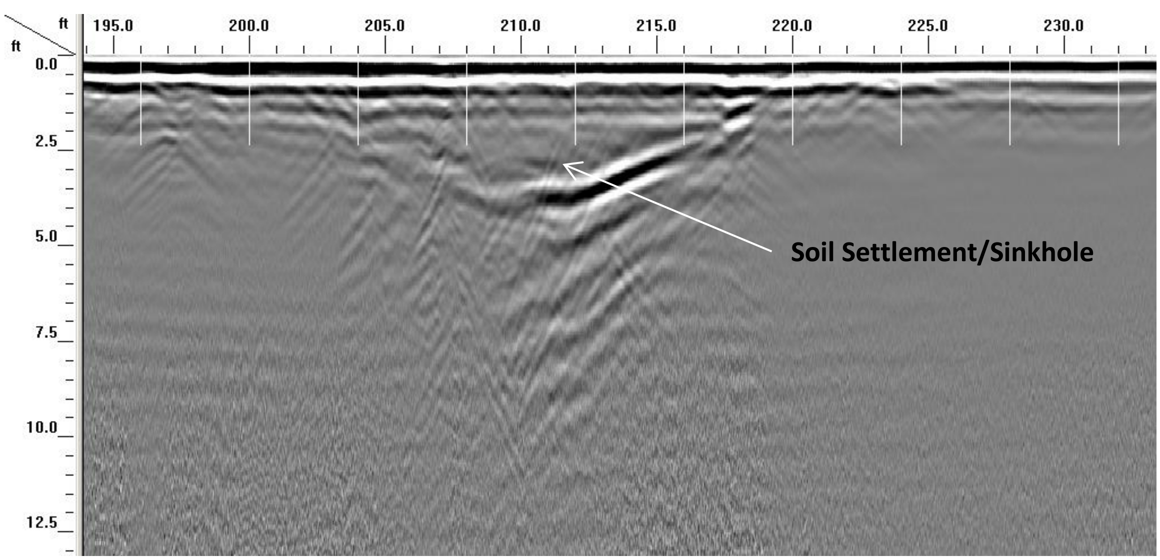
Radar data are typically acquired at a slow walking speed along a grid pattern of survey lines or a series of parallel lines. Data is displayed on LCD screen for field verification and quality control of results and digitally saved. Calibrated measuring wheels are used to automatically added footage/station markers to the digital data. The saved data can be printed or post processed.

Interpretation of GPR data is subjective. GPR results should be verified with borings or test pits. GPR lines indicate a cross-section in time/depth along a survey line.

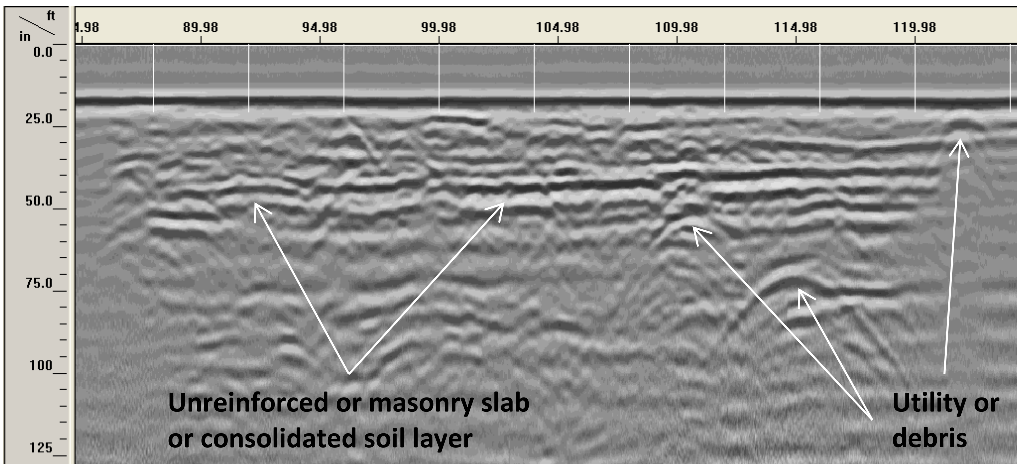
Natural soils or fill placed in lifts during construction retain moisture between material interfaces and typically have horizontal or near horizontal bedding planes. These conditions cause a change in conductivity which shows as continuous reflective layers on GPR data. The strength of a reflected signal and/or the continuity of the reflector across the record may be indicative of a stratigraphic contact, water table, top of rock, back of wall/slab.

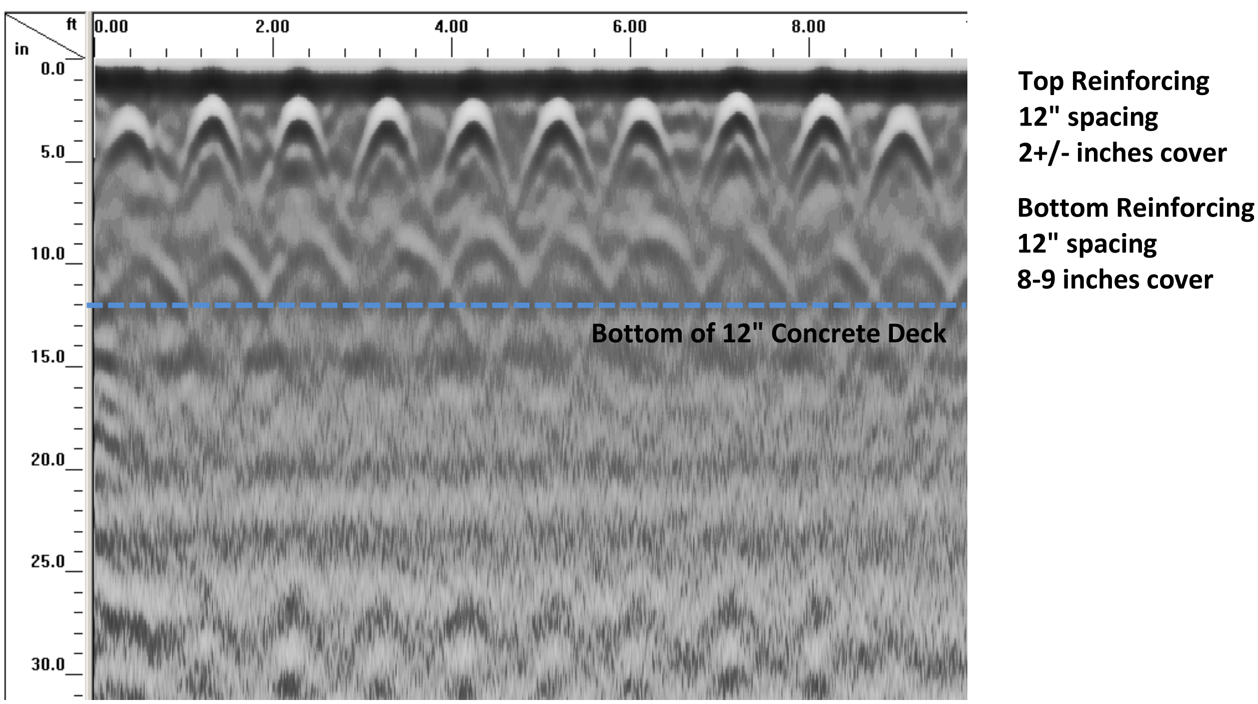


Locations where GPR data indicate theses horizontal bedding planes/layers are sloping, draped or disturbed can be indication of soil settlement, trenching and/or voiding. Areas where GPR data is less reflective, indicating fine soil materials (clays and silts) have been washed or eroded away or areas that are more reflective, indicating loose soil conditions where moisture has accumulated are also indicative of and associated with settlement, sinkholes, and voiding.



Often point targets, such as reinforcing, buried utilities, boulders, create a distinctive parabolic feature on GPR records. Point targets trending perpendicular to the direction of the line of coverage are detected, therefore to detect longitudinal reinforcing a transverse line of data would be collected and to detect transverse reinforcing, a longitudinal line of data would be collected. Plotting point targets of similar signal strength, depth, and shape located along the grid of GPR lines give the trend and location of individual utilities and/or reinforcing.





APPENDIX 2

SONIC/ULTRASONIC METHOD OF INVESTIGATION

APPENDIX
SONIC/ULTRASONIC NONDESTRUCTIVE TESTING OF CONCRETE

The sonic/ultrasonic measurements made to determine the characteristics of concrete (or rock) are generated by a relatively low energy source as a single discrete wide band impulse with a pulsed transducer, projectile, mechanical hammer, etc. Practical problems and the condition of the concrete surface largely determine the source(s) to be used. A rough concrete surface that has deposits of organic materials or mineral deposits generally requires a more powerful energy source whereas a relatively new or wet concrete may be inspected by the use of a pulsed transducer or other higher frequency source. In general high frequency sources that may work well in the laboratory may be unusable for field conditions. High frequency sources have the advantage of high resolution but the disadvantage of low penetration. While metals can be tested in the megahertz range, such signals in concrete will not have measurable signals for more than an inch in thickness. The energy source should be sufficient to maximize the resolution, have sufficient penetration to examine the concrete being tested and enough energy to excite the fundamental frequencies being sought.

The transmitted energy is in the form of three principal wave types: compressional (contraction/expansion, spring-like particle motion), shear (traction-sliding motion), and surface waves (combination of motions). Each boundary that has density and or velocity contrast will reflect and/or refract these waves. Compressional and shear wave velocity values are determined by the Young's, shear and bulk moduli values as well as the density and Poisson's Ratio. In turn the velocity can be used to determine the moduli values and Poisson's Ratio given that the density is known or can be assumed. The moduli values measured are the dynamic moduli values at low strain. In general, the difference between the dynamic values and the static values is almost entirely controlled by the crack densities of the concrete. Using the modulus values, a reasonable estimate of the unconfined compressive strength can be determined. The strength is largely dependent on the crack density of the concrete and for static tests, the orientation of the cracks. For static testing, cracks perpendicular to the axis of the core and perpendicular to the directed stress will produce a strength (static) that is not greatly different from uncracked concrete. The applied stress closes or compresses the cracks. Cracks that are near 45º to the direction of stress will result in lowest static strength. The approximate orientation of the cracks can be determined with dynamic measurements of the velocity values in different directions.

NDT Corporation makes several determinations from one energy impact. The velocity is measured directly from the energy point of impact to a linear array(s) of sensors on the surface. The array length is usually in excess of the thickness of the concrete being tested. In addition to the velocity measurements, reflections are measured individually or determined from a frequency analysis of the time domain recordings. Each reflecting surface (change of density and/or velocity) produces a multi-path reflection in the layer it bounds. A generated wave will travel to a delamination surface and reflect back to the surface of the concrete where it is reflected back to the delamination, resulting in multi-reflections that are apparent in the frequency domain. These reverberations (echoes) are particularly diagnostic of delaminations and thickness of the concrete. They will readily distinguish the presence of local delaminations, cracked or decomposed inclusions by the particular frequency band generated at the mechanical discontinuity. If a delamination is severe or large in area, the reflected signals are strong resulting in a low frequency, high amplitude, and long duration "ringing" signal or a drum head effect that is usually quite distinguishable. This is the basis of the 'chain drag" using the human ear as the sensor to recognize frequency differences. The ear however is limited in its perception and will only distinguish within the hearing range.

DIRECT AND REFRACTED ENERGY

One of the advantages of the sonic/ultrasonic method is its ability to "look through" overlying materials coatings, particularly decomposed "softer layers" when the array(s) is configured properly. This is done using refracted waves associated with the different layer velocities or by careful examination of the resonant frequencies associated with such layering.

The diagram below shows the wave path for refracted energy generated for a softer (1) lower velocity layer over a harder (2) higher velocity layer. For example asphalt (1) over good concrete (2). The wave is bent (similar to the appearance of a stick in water) and travels along the boundary between the lower velocity layer and the higher velocity layer and radiates back to the surface. The higher velocity of the good concrete allows the refracted wave to overtake the direct wave in layer 1 at some distance designated as $D_{1|2}$. To the left of this point the lower velocity of layer 1 will be measured and beyond it the velocity of the deeper layer 2 is measured.

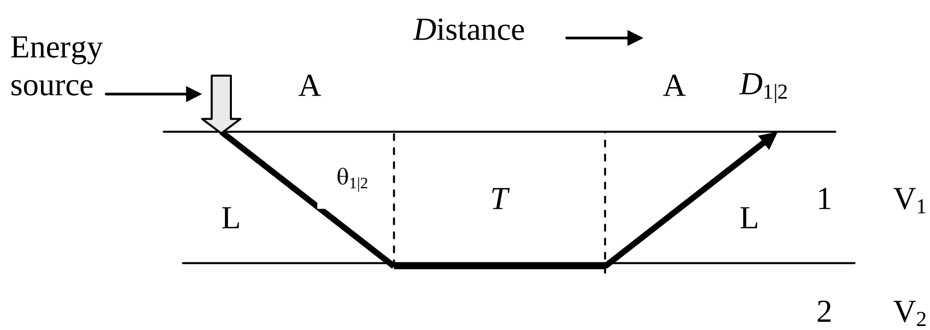


Figure A1

The time for the direct path is D/V_1; the refracted path time is $2L/V_1+(D_{1|2}-2A)/V_2$.
The array of sensors is placed in the distance direction and the time elapsed (travel time) from the time of energy impact to the sensor distance is measured. The velocity is determined from this time-distance measurement(s). The angle θ is the angle between the perpendicular to the layer and the incident wave that is critically refracted. The sine of this angle is the velocity of the first layer divided by that of the second layer (Snell's Law). The distance shown $D_{1|2}$ is the point on the surface where the refracted time arrival equals that of the direct wave (the refracted wave travels at a higher velocity than the direct wave).

$$\frac{D}{T} = \frac{D - 2\tan\Theta}{V_2} + \frac{2T}{V_1 \cos\Theta}$$

The thickness is expressed as:

$$T = \frac{D_{1|2}}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$

D is the distance and *T* is the thickness. Since the times as well as the distances are measured, then V_1 and V_2 are determined. If a plot of distance versus time is made then the resulting graph will look like Figure A2.

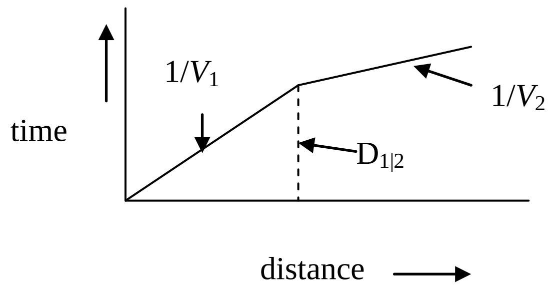


Figure A2

If the concrete has no overlay then the concrete velocity is simply D/T.

The resonant frequencies are determined by the thickness and velocity of the material. Since the velocity is measured as above, then the thickness can be determined directly.

The resonance of a simple beam is given by:

$$f = \frac{nV}{2L} \qquad (\mathit{fixed} - \mathit{fixed}, \quad \mathit{free} - \mathit{free}) \quad \mathit{where}\ n = 1,2,3---$$

$$f = \frac{nV}{4L} \qquad (\mathit{open} - \mathit{fixed}), \quad \mathit{where} \quad n = 1,3,5,7---$$

Since the frequency and velocity are measured, the thickness is determined. This thickness can be the thickness of the concrete floor, deck slabs, or column being measured or it can be the thickness of concrete overlying a delamination.

While the ref**ra**cted wave is dependent only on a contrast in velocity, a reflection can take place where there is a change in velocity or density or both. The impedance (RF reflection coefficient) which causes a wave to be reflected is given by:

$$RF = \frac{\rho_2 V_2 - \rho_1 V_1}{\rho_2 V_2 + \rho V_{11}}$$

Where ρ is the density and V is the velocity of the material. The impedence determines the strength of the reflection. The contrast between an air filled void at the back of or within the concrete is significant; the velocity in air is 1,000ft/sec. and the velocity of good concrete is 13,000ft/sec. The density differences are of course very large between the concrete and air. A similar difference exists for a water filled void where the velocity in water is 5,000ft/sec. and concrete is nearly a factor of 2.5 denser. Voiding behind a liner or under a slab is usually well distinguished by a distinct " ringing" resonant frequency, reffered to above as a drum head effect..

MODULI VALUES AND STRENGTH

The moduli values as stated above are determined from the velocity values using an assumed or measured density. The density is usually the best known or best estimated value for the concrete, its variance generally does not affect the calculations significantly.

The relationships for Young's modulus versus the compressional velocity are shown in Figure A4; shear modulus versus the shear velocity Figure A5; Poisson's Ratio versus the compressional and shear wave velocities Figure A6; and finally a relationship between the velocity values (compressional and shear) and the unconfined compressiive strength of concrete, Figure A7.

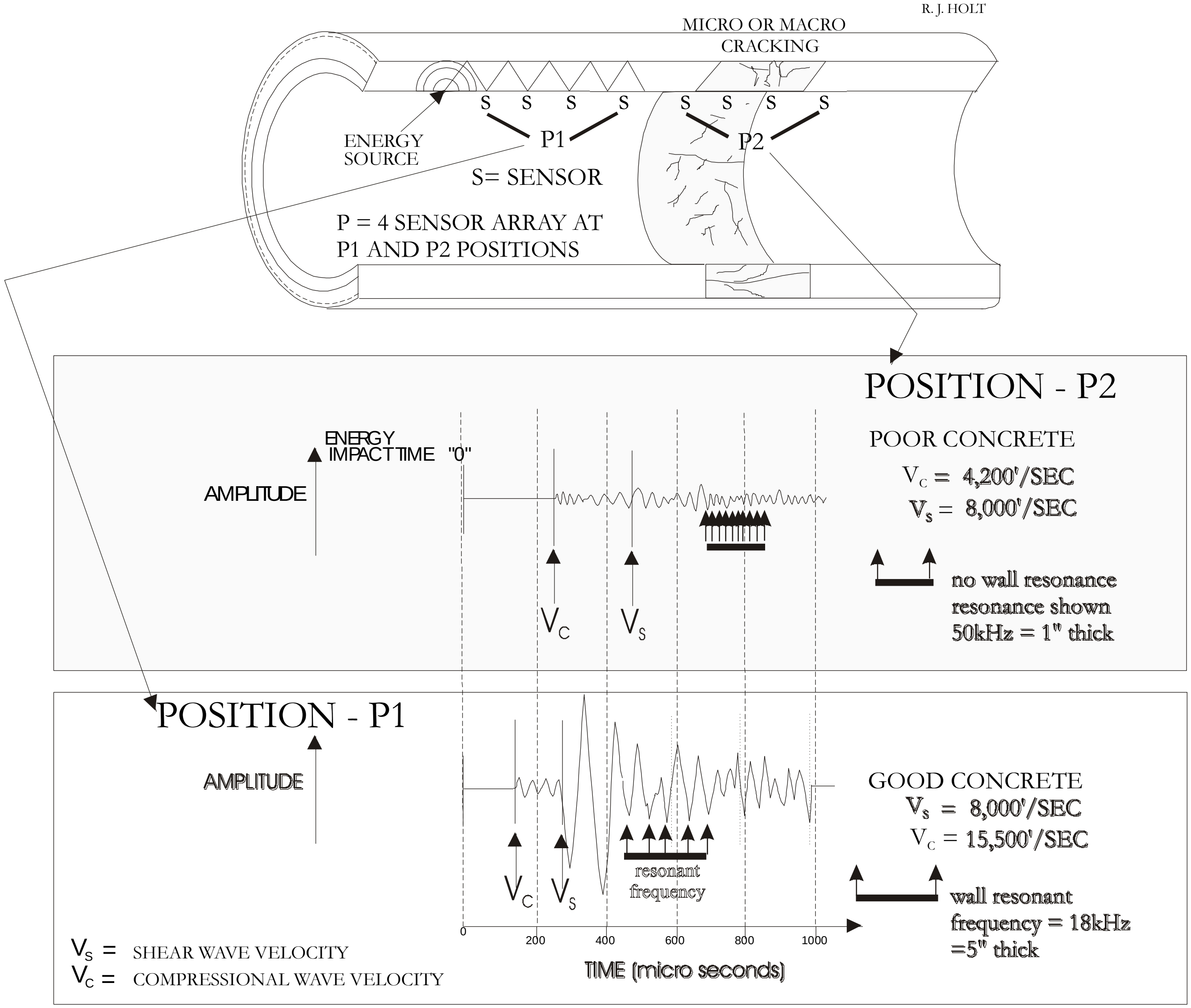
Figure A3 is illustrative of a tunnel liner or pipe investigation where there has been circumferential damage, perhaps at a construction joint or an outside zone of weakness (rock shear or fault, soil washout etc.) that has affected the integrity of the liner. The damage need not be visible; there can be a 20% reduction in the strength of the concrete from micro-cracking that is not visible to the naked eye. The process of deteroration of most concrete starts at the micro level and with continued stress the micro cracks coalesce into macro cracks and finally to spalling. The ability to measure at the micro level well in advance of future needed repairs provides a management tool for establishing proirities for repair, projected budgets, and asset valuation

SONIC/ULTRASONIC TUNNEL & PIPE LINE TESTING

Circumferential (face to back) Cracking

NDT ENGINEERING, INC.

R. J. HOLT



SONIC/ULTRASONIC TIME AMPLITUDE RECORDS
FOR FOURTH SENSOR - POOR AND GOOD CONCRETE

FIGURE A3

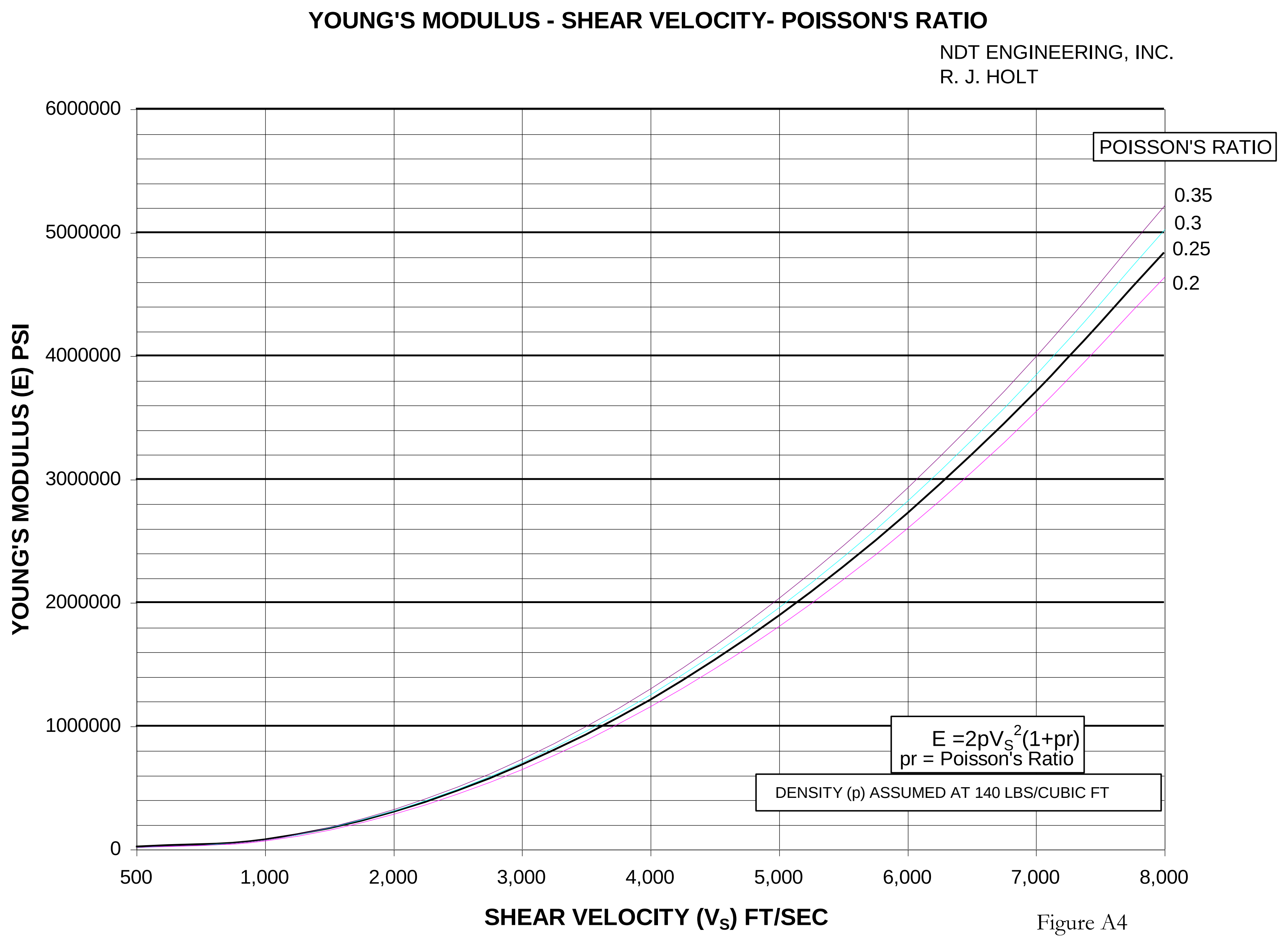

YOUNG'S MODULUS - SHEAR VELOCITY- POISSON'S RATIO
NDT ENGINEERING, INC.
R. J. HOLT
POISSON'S RATIO
0.35
0.3
0.25
0.2
6000000
5000000
4000000
3000000
2000000
1000000
0
YOUNG'S MODULUS (E) PSI
E =2pV$_S^2$(1+pr)
pr = Poisson's Ratio
DENSITY (p) ASSUMED AT 140 LBS/CUBIC FT
500
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
SHEAR VELOCITY (V$_S$) FT/SEC

Figure A4

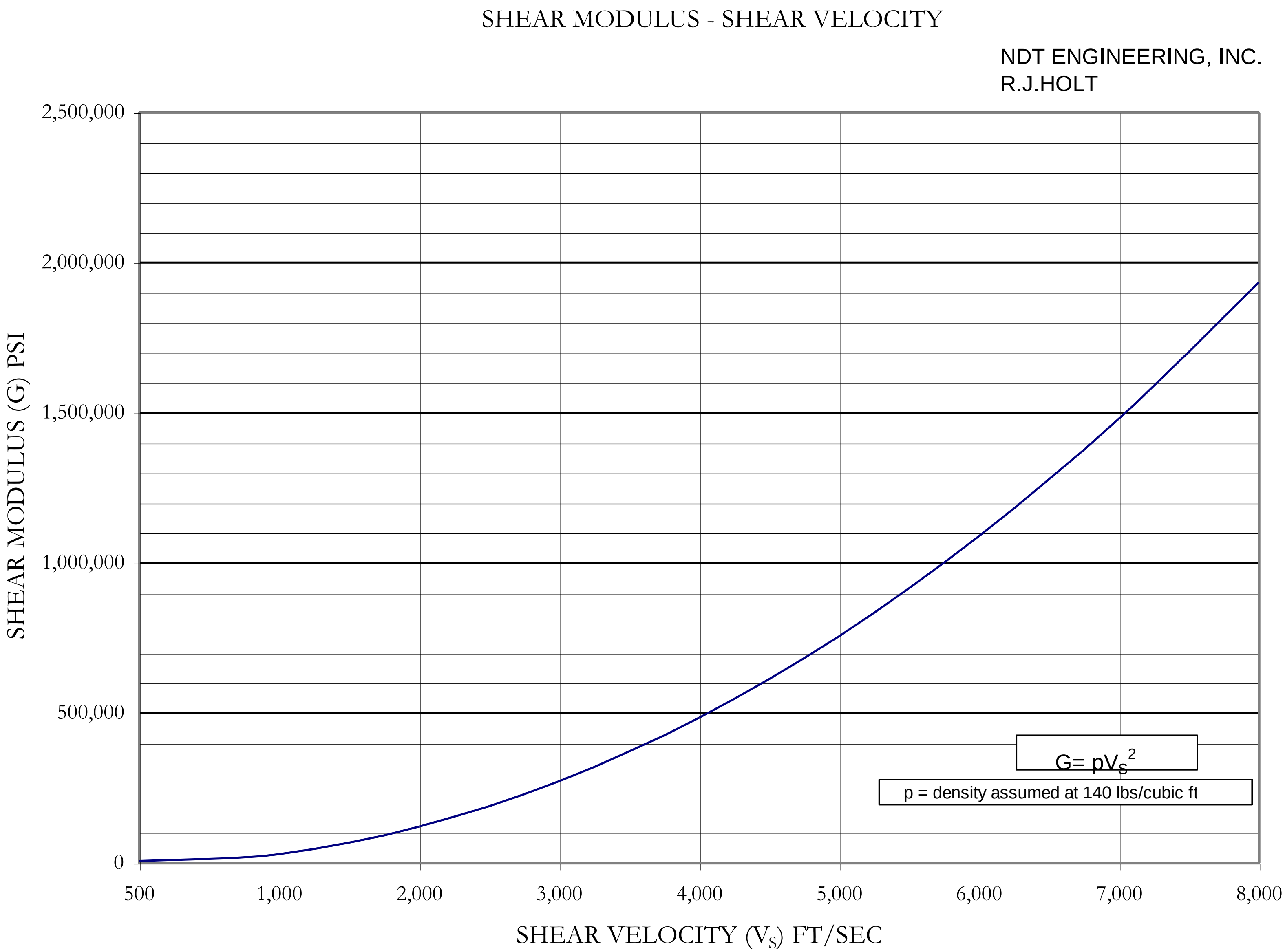

SHEAR MODULUS - SHEAR VELOCITY
NDT ENGINEERING, INC.
R.J.HOLT
2,500,000
2,000,000
1,500,000
1,000,000
500,000
0
SHEAR MODULUS (G) PSI
500
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
SHEAR VELOCITY (V_S) FT/SEC
$G= pV_S^2$
p = density assumed at 140 lbs/cubic ft

Figure A5

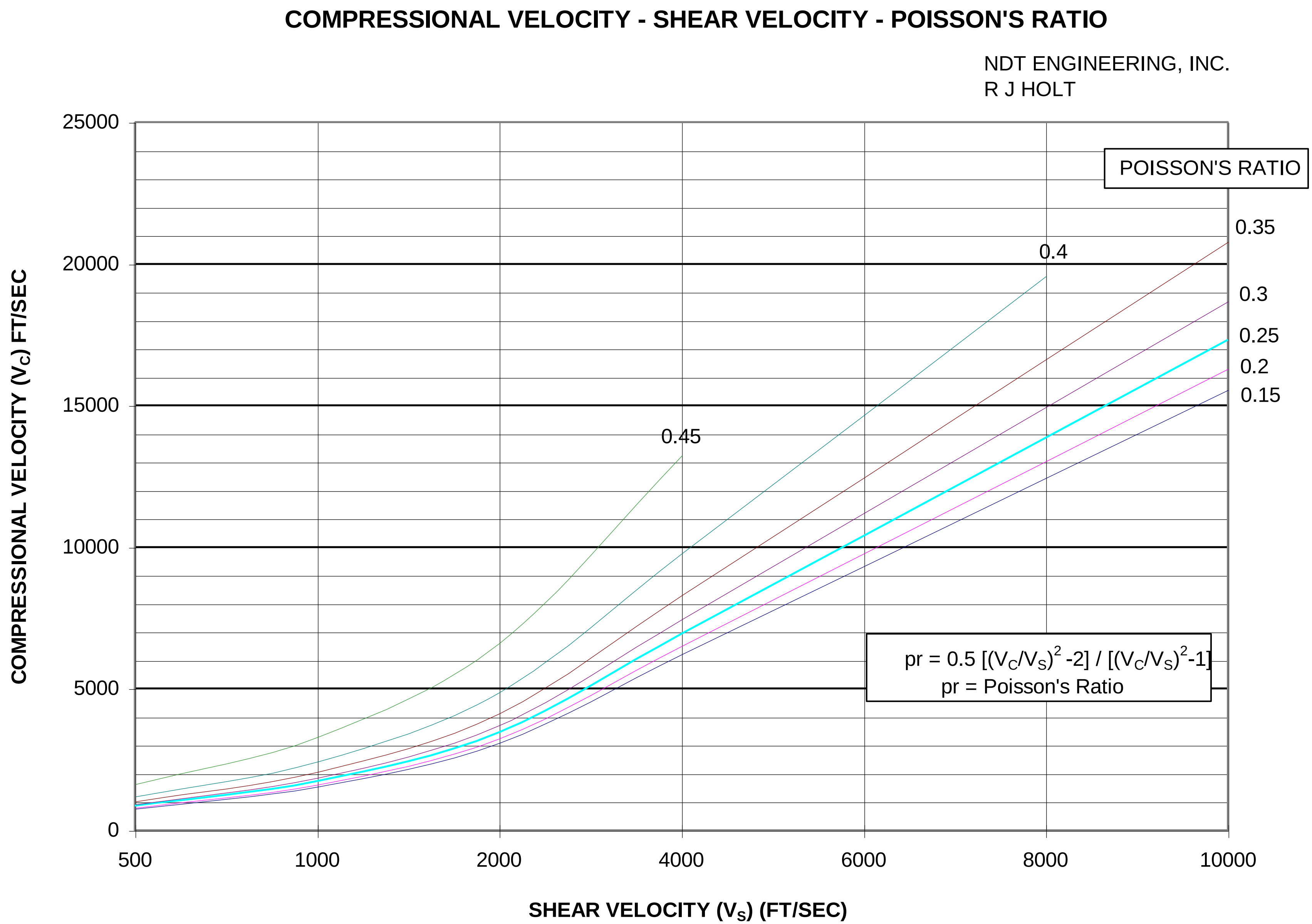
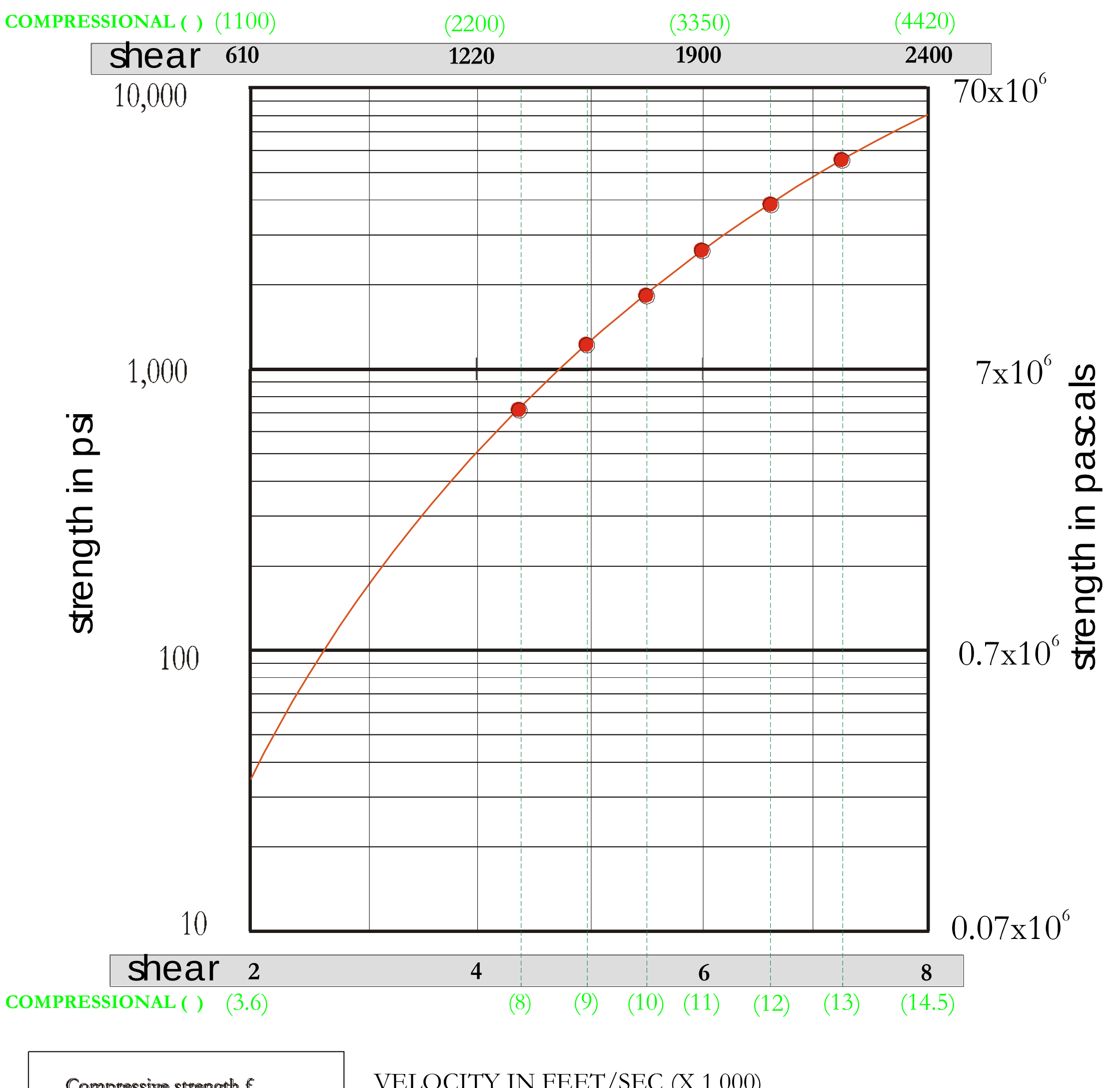
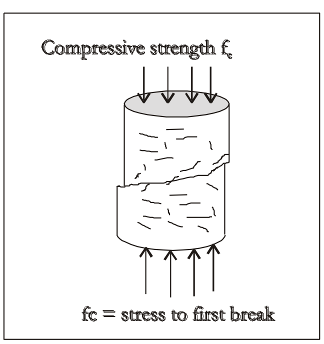

COMPRESSIONAL VELOCITY - SHEAR VELOCITY - POISSON'S RATIO
NDT ENGINEERING, INC.
R J HOLT
POISSON'S RATIO
0.45
0.4
0.35
0.3
0.25
0.2
0.15
pr = 0.5 [(V_C/V_S)^2 -2] / [(V_C/V_S)^2-1]
pr = Poisson's Ratio
COMPRESSIONAL VELOCITY (V_C) FT/SEC
SHEAR VELOCITY (V_S) (FT/SEC)
25000
20000
15000
10000
5000
0
500
1000
2000
4000
6000
8000
10000

Figure A6

strength of concrete versus velocity

r. j. holt





CURVE FOR RATIO: $V_{SHEAR} / V_{COMPRESSIONAL} = 0.55$
EQUALS POISSON'S RATIO OF 0.28

FIGURE A7

NDT ENGINEERING, INC.

APPENDIX 3

SEISMIC REFRACTION

APPENDIX: SEISMIC REFRACTION

OVERVIEW

Seismic exploration methods utilize the natural energy transmitting properties of the soils and rocks and are based on the principle that the velocity at which seismic waves travel through the earth is a function of the physical properties (elastic moduli and Poisson's ratio) of the materials. Energy is generated at the ends and at the center of the seismic spread. The geophone/hydrophone is in direct contact with the earth/water and converts the earth's motion resulting from the energy generation into electric signals with a voltage proportional to the particle velocity of the ground motion. The field operator can amplify and filter the seismic signals to minimize background noise. Data are recorded on magnetic disk and can be printed in the field. Interpretations are based on the time required for a seismic wave to travel form a source to a series of geophones/hydrophones located at specific intervals along the ground surface. The resultant seismic velocities are used for:

* Material identification.
* Stratigraphic correlation.
* Depth determinations.
* Calculation of elastic moduli values and Poison's ratio.

A variety of seismic wave types, differing in resultant particle motion, are generated by a near surface seismic energy source. The two types of seismic waves for seismic exploration are the compressional (P) wave and the shear (S) wave. Particle motion resulting from a (P-wave) is an oscillation, consisting of alternating compression and dilatation, orientated parallel to the direction of propagation. An S-wave causes particle motion transverse to the direction of propagation. The P-wave travels with a higher velocity of the two waves and is of greater importance for seismic surveying. The following discussions are concerned principally with P-waves.

Possible seismic wave paths include a direct wave path, a reflected wave path or a refracted wave path. These wave paths are illustrated in FIGURE A1. The different paths result in different travel times, so that the recorded seismic waveform will theoretically show three distinct wave arrivals. The direct and refracted wave paths are important to seismic refraction exploration while the reflected wave path is important for seismic reflection studies.

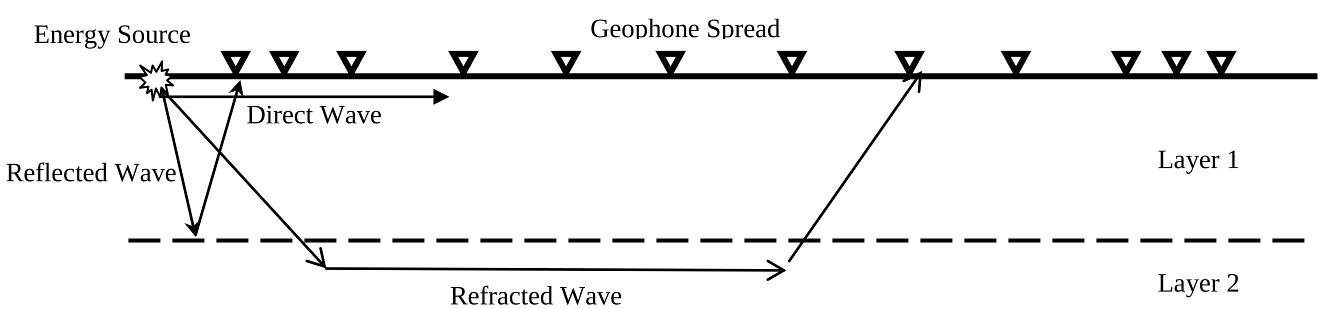


FIGURE A1:
SEISMIC WAVE PATHS FOR DIRECT WAVE, REFLECTED WAVE AND REFRACTED WAVE ILLUSTRATING EFFECTS OF A BOUNDARY BETWEEN MATERIALS WITH DIFFERENT ELASTIC PROPERTIES

Seismic waves incident on the interface between materials of different elastic properties at what is termed the critical angle are refracted and travel along the top of the lower layer. The critical angle is a function of the seismic velocities of the two materials. These same waves are then refracted back to the surface at the same angle. The recorded arrival times of these refracted waves, because they depend on the properties and geometry of the subsurface, can be analyzed to produce a vertical profile of the subsurface. Information such as the number, thickness and depths of stratigraphic layers, as well as clues to the composition of these units can be ascertained.

The first arrivals at the geophones/hydrophones located near the energy source are direct waves that travel through the near surface. At greater distances, the first arrival is a refracted wave. Lower layers typically are higher velocity materials, therefore the refracted wave will overtake both the direct wave and the reflected wave, because of the time gained travelling through the higher velocity material compensates for the longer wave path. Depth computations are based on the ratio of the layer velocities and the distance from the energy source to the point where refracted wave arrivals over take direct arrivals.

Although not the usual case, a constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole data are available.

APPLICATIONS

Seismic refraction technique is an accurate and effective method for determining the thickness of subsurface geologic layers. Applications for engineering design, assessment, and remediation as well as ground water and hydrogeologic studies include:

* Continuous profiling of subsurface layers including the bedrock surface
* Water-table depth determinations
* Mapping and general identification of significant stratigraphic layers
* Detection of sinkholes and cavities
* Detection of bedrock fracture zones
* Detection of filled-in areas
* Elastic moduli and Poisson’s ratio values for subsurface layers

Seismic refraction investigations are particularly useful because seismic velocities can be used for material identification. FIGURE A2 presents a guide to material identification based on P-wave seismic velocities. In rocks and compacted overburden material, the seismic waves travel from grain to grain so that the measured seismic velocity value is a direct function of the solid material. In porous or fractured rock and most overburden materials the seismic waves travel partly or wholly though the fluid between the grains.

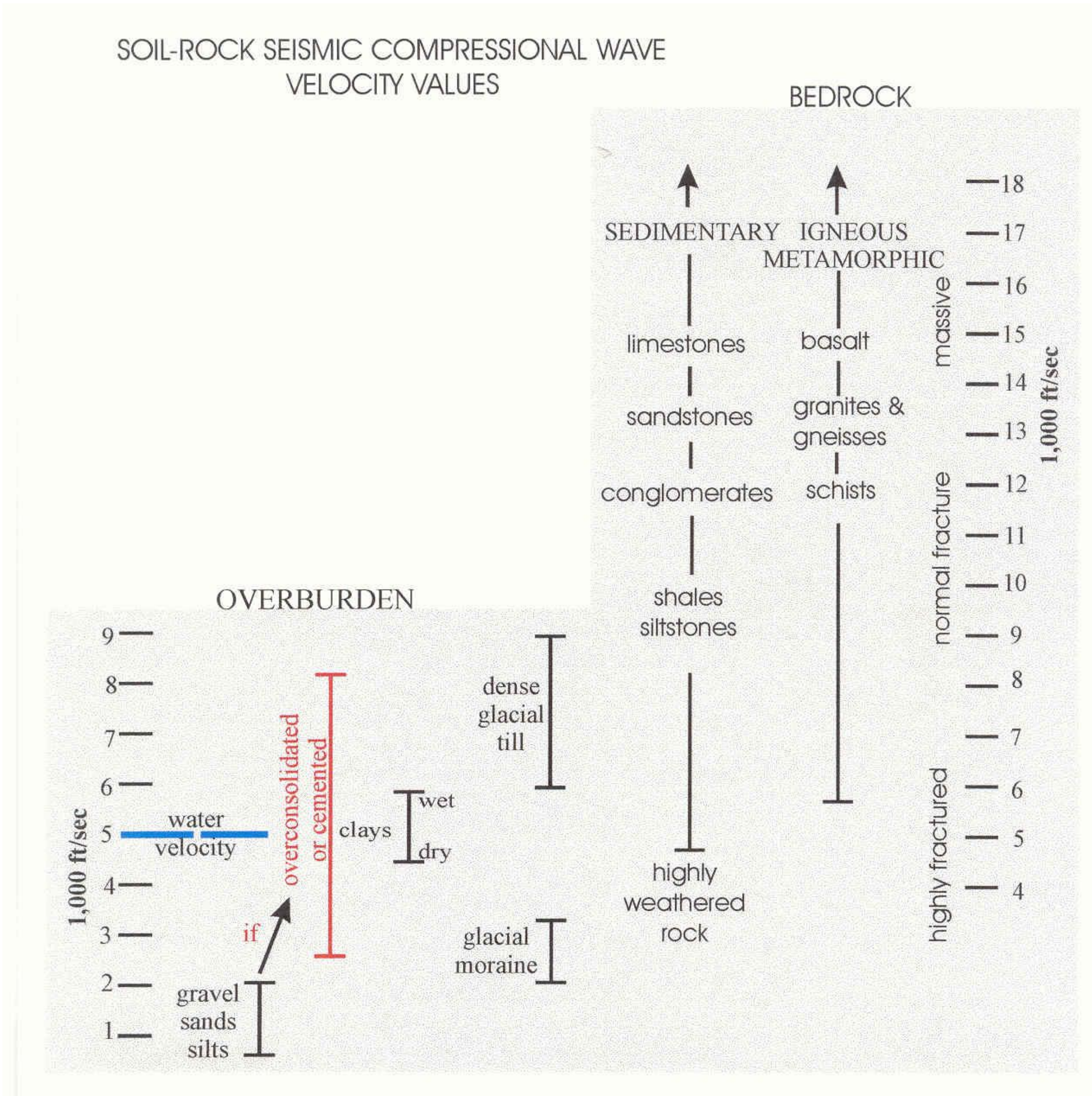


FIGURE A2:
GUIDE TO MATERIAL IDENTIFICATION BY P-WAVE VELOCITY

Seismic compressional wave velocities in unconsolidated deposits are significantly affected by water saturation. The seismic velocity values of unsaturated overburden materials such gravels, sands and silts generally fall in the range of 1,000 to 2,000 ft/sec. When these materials are water saturated, that is when the space between individual grains are 100% filled with water, the seismic velocities range from 4,800 to 5,100 ft/sec, equivalent to the compressional P-wave velocity of sound in water. This is because the seismic wave assumes the velocity of the faster medium, that of water. Even a small decrease in the saturation level will substantially lower the measured P-wave velocity of the material. Because of this velocity contrast between saturated and unsaturated materials, the water table acts as a strong refractor.

Seismic investigations over unconsolidated deposits are used to map stratigraphic discontinuities and to unravel the gross stratigraphy of the subsurface. These can be vertically as in the case of a dense till layer beneath a layer of saturated material or horizontally as in the case of the boundaries of a fill material. Often these boundaries represent significant hydrologic boundaries, such as those between aquifers and aquicludes.

A common use of seismic refraction is the determination of the thickness of a saturated layer in unconsolidated sediments and the depth to relatively impermeable bedrock or dense glacial till. Continuous subsurface profiles and even contour maps of the top of a particular horizon or layer of interest can be developed from a suite of seismic refraction data.

Bedrock velocities FIGURE A2 vary over a broad range depending on variables, which include:

* Rock type
* Density
* Degree of jointing/fracturing
* Degree of weathering

Fracturing and weathering generally reduce seismic velocity values in bedrock. Low velocity zones in seismic data must be evaluated carefully to determine if they are due to overburden conditions or fractured/weathered or perhaps even faulted bedrock.

<u>EQUIPMENT:</u>

The basic equipment necessary to conduct a seismic refraction investigation consists of:

* Energy source
* Seismometers (Geophones/Hydrophones)
* Seismic cables
* Seismograph

Energy sources used for seismic surveys are categorized as either non-explosive or explosive. The energy for a non-explosive seismic signal can be provided by one of the following:

* Sledge Hammer (very shallow penetration)
* Weight Drop
* Seisgun
* Airgun
* Sparker
* Vibrators (for reflection surveys)

Explosive sources can be categorized as:

* Dynamite
* Primers
* Blasting Agents

Choice of energy source is dependent on site conditions, depth of investigation, and seismic technique chosen as well as local restrictions. Explosive sources may be prohibited in urban areas where non-explosive sources can be routinely used. Deeper investigations usually require a larger energy source: therefore, explosives may be required for sufficient penetration.

Geophones/Hydrophones are sensitive vibration detectors, which convert ground motion to an electric voltage for recording the seismic wave arrivals. Seismic cables, which link the geophones/hydrophones and seismograph are generally fabricated with pre-measured locations for the geophones/hydrophones and shot point definitions.

The seismograph can be single channel or multi-channel, although, multi-channel seismographs (12 to 24 channels) are preferred and necessary for all but the simplest of very shallow surveys. The seismograph, amplifies (increases the voltage output of the geophones), conditions/filters the data, and produces analog and digital archives of the data. The analog archive is in the form of a thermal print of the data, which can be printed directly after acquisition in the field. The digital archive is stored on magnetic disk and can be used for subsequent computer processing and enable more extensive and detailed interpretation of seismic data.

<u>ACQUISITION CONSIDERATIONS:</u>

Several concerns arise before data collection, which must be addressed before of any seismic survey:

* Geophone spacing and Spread length
* Energy Source (discussed above)
* On-site utilities and cultural features (buildings, high tension lines, buried utilities, etc.)
* Vibration generating activities
* Geology
* Topography

To acquire seismic refraction data, a specific number of geophones are spaced at regular intervals along a straight line on the ground surface; this line is commonly referred to as a seismic spread. The length of spread determines the depth of penetration; a longer spread is required for a greater depth of penetration. Spread length should be approximately three to five times the required depth of penetration. Required resolution will control the number of geophones in each spread and the distance between each geophone. Closer spacings and more geophones usually result in more detail and greater resolution.

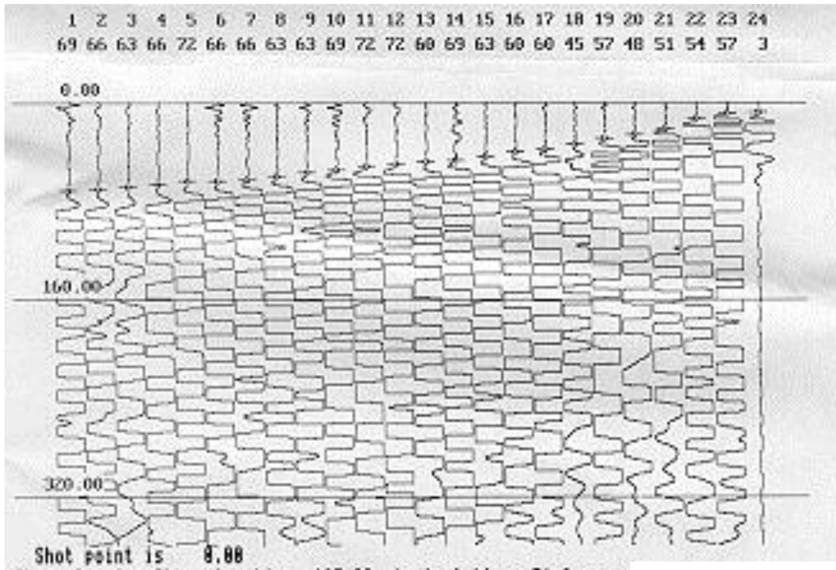
Cultural effects such as vibration generating activities, on-site utilities, and building affect where data can be acquired, and where lines/spreads are located. High volume traffic areas may require nighttime acquisition. If the survey is to be conducted near a building where vibration-sensitive manufacturing is conducted, data acquisition may be constrained to particular time intervals and appropriate energy sources must be used. Over head and buried utilities must be located an avoided, for both safety and induced electrical noise concerns. Since the seismic method measures ground vibration, it is inherently sensitive to noise from a variety of sources such as traffic, wind, rain etc. Signal Enhancement, such as record stacking, accomplished by adding a number of seismic signals from a repeated source, causes the seismic signal to "grow" out of the noise level, permitting operation in noisier environments and at greater source to phone spacings.

Knowledge of site geology can be used to determine the energy source. Some geologic materials, such as loose, unsaturated alluvium, do not transmit seismic energy as well and a powerful energy source may be required. Geologic conditions also dictate whether or not drilled shotholes are required. Site geology can also dictate the positioning of seismic lines/spreads. Where a bedrock depression of a feature is suspected, seismic lines should be orientated perpendicular to the suspected trend of the feature. Seismic cross profiles may be necessary to confirm depths to a particular refracting horizon.

The topography of a site dictates whether or not surveyed elevations are required. If possible, refraction profile lines should be positioned along level topography. For highly variable topography, a continuous elevation profile may be required to ensure sufficiently accurate cross-sections and to permit the use of time corrections in the interpretation of the refraction data.

DATA PRESENTATION AND INTERPETATION:

Interpretation of seismic refraction data involves solving a number of mathematical equations with the refraction data as it is presented on a travel-time versus distance chart. Seismic refraction data FIGURE A3 can be processed by plotting the "First Arrival" travel times at each geophone location. The preferred format of data presentation is a graph (Travel Time Plot) illustrated in FIGURE A4, in which travel time in milliseconds is plotted against source-receiver distance. From such a chart, the velocities of each layer can be obtained directly from the increase slope of each straight-line segment. Using the velocities the critical angle of refraction for each boundary can be calculated using Snell's Law. Then, utilizing these velocities, and angles and the recorded distances to crossover points (where line segments cross); the depths and thickness of each layer can be calculated using simple geometric relationships.



Shot point is 0.00

Channel 1	Phone location	195.00	Arrival time	71.3	msec
Channel 2	Phone location	190.00	Arrival time	70.3	msec
Channel 3	Phone location	185.00	Arrival time	68.5	msec
Channel 4	Phone location	175.00	Arrival time	66.3	msec
Channel 5	Phone location	165.00	Arrival time	63.8	msec
Channel 6	Phone location	155.00	Arrival time	63.8	msec-
Channel 7	Phone location	145.00	Arrival time	62.0	msec
Channel 8	Phone location	135.00	Arrival time	59.0	msec
Channel 9	Phone location	125.00	Arrival time	56.8	msec
Channel 10	Phone location	115.00	Arrival time	54.8	msec
Channel 11	Phone location	110.00	Arrival time	51.5	msec
Channel 12	Phone location	105.00	Arrival time	49.3	msec
Channel 13	Phone location	95.00	Arrival time	47.3	msec
Channel 14	Phone location	90.00	Arrival time	46.0	msec
Channel 15	Phone location	85.00	Arrival time	43.0	msec
Channel 16	Phone location	75.00	Arrival time	39.0	msec
Channel 17	Phone location	65.00	Arrival time	37.0	msec
Channel 18	Phone location	55.00	Arrival time	33.0	msec
Channel 19	Phone location	45.00	Arrival time	28.0	msec
Channel 20	Phone location	35.00	Arrival time	25.0	msec
Channel 21	Phone location	25.00	Arrival time	20.0	msec
Channel 22	Phone location	15.00	Arrival time	12.0	msec
Channel 23	Phone location	10.00	Arrival time	8.0	msec
Channel 24	Phone location	5.00	Arrival time	7.0	msec

FIGURE A3:
TYPICAL 24 CHANNEL ANALOG SEISMIC REFRACTION RECORD, WITH FIRST ARRIVAL TIMES

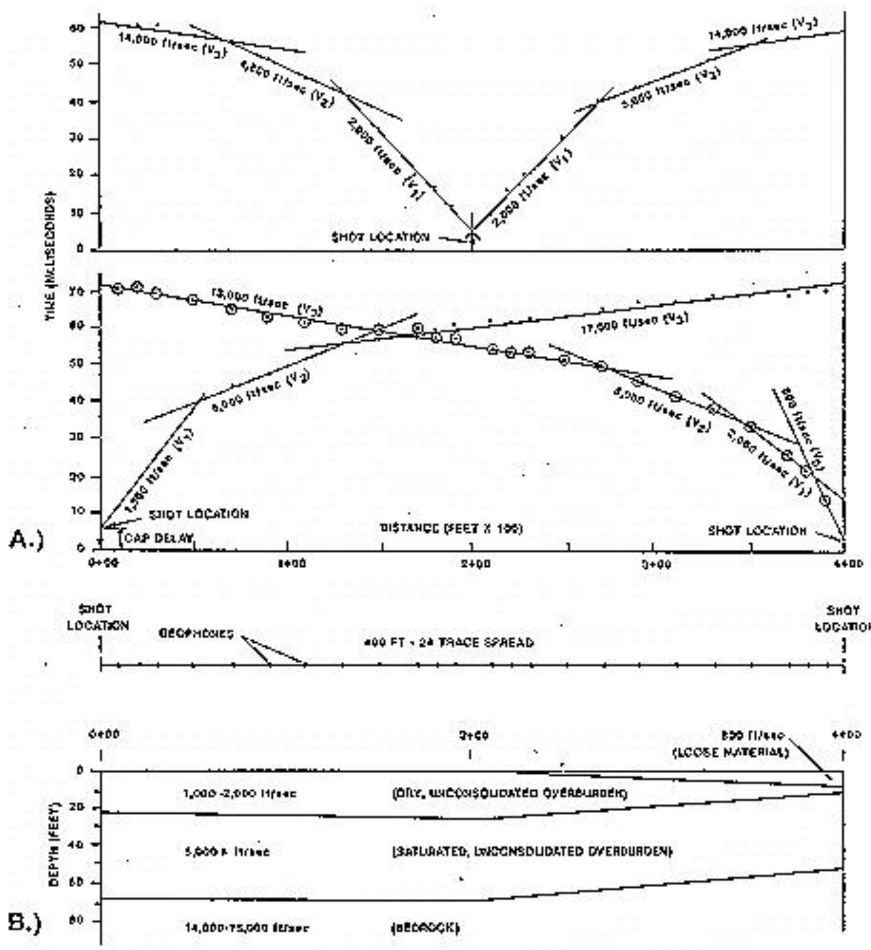


FIGURE A4:
A: TRAVEL-TIME PLOTS; UPPER PLOT IS A CENTER SHOT, LOWER PLOT IS TWO END SHOTS
B: RESULTING PROFILE OF SUBSURFACE MATERIALS SHOWING INTERFACE BETWEEN DIFFERENT SEISMIC VELOCITY LAYERS

The results of any seismic survey, refraction or reflection are usually presented in profile form showing elevations of seismic horizons/layers. Data acquired on a grid basis can be contoured and used to construct isopach maps. Seismic velocities and therefore, generalized material identifications should be presented on refraction profiles along with any test borings used for correlation to establish confidence in the overall subsurface data, both seismic and borings.

Where profiles indicate dipping boundaries, calculation of dips, true depths and true velocities involve more complicated equations. Further more, corrections for differing elevations and varying thicknesses of weathered zones must often be made. Fracturing and weathering generally reduce seismic velocity values in bedrock. Consequently, travel-time plots with late arrivals must be evaluated carefully to determine if the late arrival times (slower velocities) are due to overburden conditions or fractured/weathered bedrock.

Very thin layers or low velocity zones often complicate the travel-time chart as well. Although not the usual case, one constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole velocity data are available.

ADVANTAGES AND LIMITATIONS:

The seismic refraction technique, when properly employed, is the most accurate of the geophysical methods for determining subsurface layering and materials. It is extremely effective in that as much as 2,000 linear feet or more of profiling can be acquired in a field day. The resulting profiles can be used to minimize drilling and place drilling at locations where borehole information will be maximized resulting in cost-effective exploration. A standard drilling program runs the risk of missing key locations due to drillhole spacing. This risk is substantially reduced when refraction is used.

In summary, the advantages and limitations of the seismic techniques are:

Advantages:

* Material identification
* Subsurface data over broader areas at less cost than drilling
* Relatively accurate depth determination
* Correlation between drillholes
* Preliminary results available almost immediately
* Rapid data processing

Limitations:

* As depth of interest and geophone spacing increases, resolution decreases
* Thin layers may be undetected
* Velocity inversions may add uncertainty to calculations
* Susceptible to noise interference in urban areas, which require use of grounded cables and equipment, signal enhancement and alternative energy sources.