GEOTECHNICAL DESIGN REPORT

NORTH PARISH BRIDGE NO. 2619
North Parish Road over Nezinscot River
Turner, Maine
MaineDOT WIN 18749.00

FOR

Maine Department of Transportation
Augusta, ME

BY

NOBIS ENGINEERING, INC.

(800) 394-4182
www.nobiseng.com

Nobis Project No. 92270.02
November 6, 2017

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November 6, 2017
Project No. 92270.02

Ms. Laura Krusinski, P.E.
Senior Geotechnical Engineer
Maine Department of Transportation
State House Station 16
Augusta, Maine 04333-016
Laura.Krusinski@maine.gov

Re: Geotechnical Design Report
North Parish Bridge No. 2619
North Parish Road over Nezinscot River
Turner, Maine
WIN 18749.00

Dear Ms. Krusinski,

We are pleased to provide this Geotechnical Design Report to MaineDOT for the North Parish Bridge No. 2619 which carries the North Parish Road (State Route 117) over the Nezinscot River in Turner, Maine. This report has been completed in accordance with the Project-Specific Task Order No, 20150610000000000814 dated October 26, 2016, and our proposal dated October 24, 2016.

It has been a pleasure serving the MaineDOT and HNTB on this project. Please let us know if you have any questions regarding this report.

Very truly yours,

Nobis Engineering, Inc.

STATE OF MAINE
KURT JELINEK
No. 11746
LICENSED
PROFESSIONAL ENGINEER
11/6/2017

Brien Waterman
Project Manager

Kurt Jelinek, P.E.
Director, Transportation Services

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1.0 INTRODUCTION

This geotechnical design report presents Nobis Engineering Inc.'s (Nobis) recommendations for the MaineDOT North Parish Bridge WIN 18749.00 located in Turner, Maine. This report is subject to the limitations contained in Appendix A. Nobis performed geotechnical services in accordance with our proposal, dated October 24, 2016.

2.0 PROJECT AND SITE DESCRIPTION

Nobis understands that MaineDOT intends to replace the bridge carrying North Parish Road over the Nezinscot River in Turner, Maine (the Site). A Site Locus Plan is included as Figure 1. The proposed replacement bridge will be located to the east of the existing bridge.

North Parish Bridge – Turner, Maine

During the preliminary design phase of this project there was discussion of replacing the bridge with a two-span option, however, we understand the current proposed bridge includes one-span supported by piles.

This report provides geotechnical recommendations to be used by the project structural engineer to design the proposed substructures.

The existing three-span bridge was constructed in 1941 and has a total length of approximately 220 feet and a curb-to-curb roadway width of approximately 22 feet with no sidewalks. The existing foundations consist of concrete abutments and piers founded on shallow footings.

The top of the roadway at the existing bridge is at approximate elevation (El.) 290 and the bed of the river is at approximate El. 265. Elevations referenced in this report are relative to the North American Vertical Datum of 1988 (NAVD 88).

3.0 SUBSURFACE EXPLORATIONS

New England Boring Contractors, of Hermon, Maine drilled seven (7) test borings (BB-TNR-101 through BB-TNR-107) in December 2016. Three test borings were performed near the footprint of the proposed bridge and four test borings were performed in the approximate alignment of the proposed approach embankment. An additional test boring (BB-TNR-108) was performed within the footprint of the north abutment in October 2017. Information encountered in BB-TNR-108 are not included in this report, and will be provided in an addendum.

The test borings were advanced using drive and wash methods with an automatic hammer, and were observed and logged by Nobis personnel. The borings were drilled to depths ranging from approximately 24 to 45 feet below existing grades. Bedrock cores were collected from the three test borings completed near the footprint of the proposed bridge (i.e. BB-TNR-103, -104, and -105). Boring logs are included in Appendix B, photo logs of the rock core samples are included as Appendix C, and the approximate test boring locations are shown on Figure 3, Boring Location Plan.

4.0 LABORATORY TESTING

Samples of rock/soil were selected by Nobis and submitted to GeoTesting Express of Acton, Massachusetts for laboratory testing. Laboratory testing included:

- Eight (8) moisture content analyses (in accordance with ASTM D2216);
- Three (3) grain size analyses – sieve only (in accordance with ASTM D422);
- Three (3) grain size analyses – sieve and hydrometer (in accordance with ASTM D422);
- Three (3) Atterberg Limit analyses (in accordance with ASTM D4318); and
- Four (4) compressive strength and elastic moduli of rock test (in accordance with ASTM D7012).

The laboratory test results are included in Appendix D.

5.0 SUBSURFACE CONDITIONS

Based on a review of a surficial geology map titled “Surficial Geology of the Turner Center Quadrangle, Maine” dated 2008, the site consists of stream alluvium and/or Presumpscot Formation deposits. The surficial geology map is included as Figure 2.

The generalized subsurface conditions encountered in the test borings varied across the site, as summarized below.

The borings completed in the area of the proposed south approach embankment and south abutment (i.e. borings BB-TNR-101, -102, and -103) generally encountered 6 to 8 inches of topsoil, overlying approximately 5 to 10 feet of fill, overlying approximately 4 to 10 feet of alluvial deposits, overlying glacial till. Decomposed bedrock and bedrock was encountered within boring BB-TNR-103 at a depth of approximately 21 and 23 feet below grades, respectively.

The boring completed in the general vicinity of the proposed pier during the preliminary design phase (boring BB-TNR-104) generally encountered glacial till, starting at the mudline, overlying decomposed bedrock at a depth of approximately 10.6 feet below the mudline, overlying bedrock at a depth of approximately 14 feet below the mudline. Based on site observations, cobbles and boulders are located in the Nezinscot River near the location of the proposed bridge. Boring BB-TNR-104 was drilled through the existing bridge deck. The concrete bridge deck was observed to be approximately 9” thick and was repaired in-kind upon completion of the boring.

The borings completed in the area of the proposed north approach embankment and north abutment (i.e. borings BB-TNR-105, -106, and -107) generally encountered 4 to 6 inches of topsoil, overlying approximately 5 feet of fill (encountered only in boring BB-TRN-105), overlying approximately 5 to 10 feet of clay deposits (Presumpscot Formation), overlying glacial till. Bedrock was encountered within boring BB-TNR-105 at a depth of approximately 33.5 feet below grade.

A summary of the stratums encountered is included below. Refer to the boring logs prepared by Nobis, included in Appendix B, for more detailed subsurface conditions.

5.1 Fill

The fill encountered generally consisted of very loose to medium dense fine to coarse sand, with varying amounts of gravel, silt, and clay. Several samples had asphalt, wood, and root fragments present. SPT-N values generally ranged from 3 to 24 blows-per-foot (bpf) with a typical value of 8 bpf.

5.2 Alluvial Deposits

The alluvial deposits encountered generally consisted of very loose to medium dense brown/gray fine to medium sand, with varying amounts of silt and gravel. One sample had root fragments present with an organic odor. SPT-N values of 6 bpf to 8 bpf were observed.

5.3 Clay Deposit (Presumpscot Formation)

The clay deposit encountered generally consisted of very soft to medium stiff clay, with varying amounts of silt and trace amounts of fine sand. SPT-N values generally ranged from weight of hammer (WOH) to 7 bpf, with a typical value of 4 bpf. The lowest elevation this stratum was encountered was El. 276.9, which was generally above the groundwater table encountered during drilling. Eight (8) moisture content tests were performed on various samples in this stratum, with values ranging from 27.1% to 36.4%. Three (3) Atterberg Limits were performed on various samples in this stratum, with liquid limit (LL) values ranging from 35 to 38, plastic limit (PL) values ranging from 20 to 23, plasticity index (PI) values ranging from 14 to 17, and liquidity index (LI) values ranging from 0.4 to 0.8.

5.4 Glacial Till

The glacial till encountered generally consisted of both granular and cohesive soils. The primarily granular soils generally consisted of medium dense to very dense fine to coarse sand, with varying amounts of gravel, silt, and clay. SPT-N values generally ranged from 25 bpf to refusal. The primarily cohesive soils generally consisted of very stiff to hard silt and clay or clay and silt, with little to trace amounts of gravel and sand. SPT-N values generally ranged from 25 bpf to refusal.

5.5 Decomposed Bedrock/Bedrock

A thin layer of decomposed bedrock was observed above the bedrock in borings BB-TRN-103 and BB-TRN-104, with a thickness of 2.1 feet and 3.4 feet, respectively. Bedrock was encountered in three of the test borings (BB-TNR-103, -104, and -105) between El. 253.5 and El. 251.0. The bedrock encountered below the north abutment and middle pier generally consisted of moderately hard metasandstone. The bedrock encountered below the south abutment consisted of hard granite.

The rock quality designation (RQD) of the collected metasandstone rock cores ranged from 13% to 92%, with the RQD of the collected granite rock cores ranged from 13% to 100%. Two (2) laboratory uniaxial compressive strength (UCS) tests were performed on the metasandstone, with values ranging from 3,314 psi to 5,920 psi. Two (2) laboratory uniaxial compressive strength (UCS) tests were performed on the granite, with values ranging from 10,060 psi to 12,286 psi.

Refer to the photo logs prepared by Nobis, included in Appendix C, for photos of the rock core samples collected during the subsurface investigation.

5.6 Groundwater

The groundwater elevations encountered during the subsurface explorations ranged from approximate El. 276.1 feet to El. 266.9 feet. Note that fluctuations in groundwater levels will occur due to variations in precipitation, river water level, temperature and other factors different from those existing at the time the measurements were made.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 Integral Abutment Design Recommendations

Loads

Factored loads and pile head movement were provided by HNTB via email on August 10 and 23, 2017. The pile factored axial demand is 450 kips per pile and the pile head movement is 0.96 inches. Additionally, the limiting pile head moment is 2,300 kip-inches. The loads are provided in Appendix E.

Axial Pile Resistance

Nobis anticipates the capacity of the piles will be achieved through end bearing resistance on or in bedrock. Calculations to determine the geotechnical axial pile resistance were not performed because we anticipate that the piles will be driven through the decomposed bedrock, where present, to competent bedrock. Based on the subsurface explorations, hard bedrock was encountered at the site, therefore, the structural resistance of the piles controls in accordance with AASHTO Article 10.7.3.2.3. By utilizing end bearing (primary) piles, total and differential settlements will be limited to elastic compression of the piles and are anticipated to be ≤ 0.5-inch.

Lateral Pile Evaluation

Lateral analyses were performed using lateral pile analysis software, LPile v2016, by Ensoft, Inc. The objective of our lateral evaluation was to determine the depth of fixity for the HP 14x102 piles, based on the provided loads.

Based on the provided loads for the north and south abutments, the HP 14x102 piles reach fixity at approximately 17 feet for the proposed south abutment and 16 feet for the proposed north abutment. The depth of fixity is relative to the depth below bottom of the proposed concrete jacket and was taken as the second point of zero deflection. The results of the lateral evaluation are provided in Appendix E.

Nominal Geotechnical Driving Resistance

The piles will need to be driven to the required total nominal geotechnical driving resistance in order to support the factored loads. Based on the loads provided by HNTB, the maximum axial compression for the strength load case is 450 kips. Therefore, the total nominal geotechnical driving resistance, using a resistance factor of 0.65 assuming dynamic testing is performed, as described below, is 692 kips.

We recommend using a structural resistance factor (AASHTO Article 6.5.4.2) of 0.5 for H-piles subject to damage due to severe driving conditions. We recommend that the H-piles be protected with a driving shoe to protect the piles during driving.

Estimated Pile Tip Elevation

The proposed piles should be driven a minimum to five (5) feet past point of fixity (≥ 22 feet for south abutment and ≥ 21 feet for the north abutment). In locations where bedrock is less than five (5) feet below the elevation of the theoretical point of fixity, the piles should be driven to the top of bedrock. Additionally, the piles need to be driven to the required total nominal geotechnical driving resistance, as discussed above.

Based on the borings, the top of bedrock varies at the proposed abutment locations. We recommend using an estimated pile tip elevation, for quantity estimating purposes, of El. 252 for the south abutment and for the north abutment.

Corrosion Considerations

The proposed H-piles are anticipated to be driven through the fill, alluvial deposits or Presumpscot formation, and glacial till deposits. A corrosive environment has not been identified for this project, however, we recommend a design sacrificial thickness of 1/16-inch over 75 years be used. Alternatively, the steel piles could be protected with electrostatically applied epoxies, or metalized zinc and aluminum with a protective topcoat.

Drainage

The proposed integral abutments should be designed to drain in accordance with Section 5.4.1.9 of the MaineDOT Bridge Design Guide. We recommend 4 inch diameter drain piles (weep holes) at nominal 10 foot maximum spacing be installed at the base of the proposed integral abutment pile caps.

6.2 Lateral Earth Pressures

We recommend a passive earth pressure coefficient of 6.89, assuming the abutment moves laterally at least 2 percent of the wall height (0.020H for dense soils). For relative movement of less than 0.020H, Section 3.10.8 of the MassDOT LRFD Bridge Manual could be used. Based on Figure 3.10.8-1 (MassDOT LRFD Bridge Manual), with a height of 13 feet with a displacement of 0.96-inches, the passive pressure coefficient would be 2.20.

6.3 Approach Embankments

Global Slope Stability

The 2D limit equilibrium software, Slide 6.0, by RocScience, Inc. was used to evaluate the global slope stability. The required resistance factor against global stability is 0.65 (AASHTO Section 11.6.2.3) for the abutments, which corresponds to a factor of safety of 1.5, and 0.75 for the embankments, which corresponds to a factor of safety of 1.3.

The results of our global stability evaluation indicate a safety factor above 1.5 is obtained at both the north and south abutments of the proposed bridge. Additionally, the results of our global stability evaluation indicate a safety factor above 1.3 is obtained at the approach embankments. Our global stability evaluations are included in Appendix E.

Estimated Settlement

Based on the borings, approximately 5 to 10 feet of clay was encountered in the area of the proposed north approach embankment. We performed a settlement evaluation for the proposed north approach embankment, where approximately 5 to 10 feet of fill will be placed. We estimate settlement of the order of 3 inches under the influence of the proposed fill. Based on the borings, the clay is above the groundwater table, and therefore, is unsaturated. We anticipate the settlement of the unsaturated clay to be elastic and occur during placement of the fill. The settlement estimate is included in Appendix E.

6.4 Seismic Design Considerations

Based on the SPT-N values, and using Method B (Table C3.10.3.1-1), the SPT blow count is approximately 25 to 31 in the area of the abutments, corresponding to a Site Class “D”.

The seismic parameters developed for the proposed bridge are provided below per the AASHTO 7th Edition:

Mapped Ground and Spectral Response Coefficients (USGS Seismic Design Maps):

- Horizontal Peak Ground Acceleration (PGA): 0.088
- Horizontal Response Spectral Acceleration, 0.2 Sec (S_S): 0.177
- Horizontal Response Spectral Acceleration, 1.0 Sec (S_1): 0.047

Site Class: D (AASHTO Table 3.10.1-1):

- Site Factors for Site Class “D” (AASHTO Tables 3.10.3.2-1, -2, and -3): Zero-Period (F_{pga}) = 1.6, Short-Period, 0.2 Sec (F_a) = 1.6, and Long Period, 1.0 Sec (F_v) = 2.4.

- Design Spectral Response Parameters for Site Class “D”:
  A_S = 0.140, S_{DS} = 0.283, S_{D1} = 0.114.

Per AASHTO Article 3.10.6 the site is assigned Seismic Zone 1 based on a calculated S_{D1} of 0.114.

6.5 Construction Considerations

Dynamic Testing and Driveability

We recommend that piles be driven with an impact hammer utilizing fixed-lead pile installation equipment. Pile followers and pile jetting should not be allowed.

We performed a preliminary WEAP analysis to estimate pile driving feasibility. Based on our preliminary WEAP evaluation, HP 14x102 H-piles can be driven to the required total nominal driving resistance without overstressing the piles. However, a WEAP analysis shall be performed before construction by a Professional Engineer modeling the actual hammer and equipment to be used by the pile driving contractor. Our preliminary driveability calculation is included in Appendix E.

We recommend that dynamic pile testing using Pile Driving Analyzer (PDA) is used to confirm that piles are driven to the total nominal driving resistance which we anticipate to be on or within bedrock. We recommend piles be driven to a nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 (AASHTO Table 10.5.5.2.3-1).

We recommend that 1 indicator pile at each abutment be tested during the end of initial driving with a PDA followed by a CAPWAP. This test is used to determine the pile driving acceptance criteria including blows per inch (or pile set in inches per blow), the hammer energy, and hammer stroke. Pile driving installation criteria, including refusal criteria, must be determined by the Contractor’s geotechnical engineer and submitted to Nobis for review.

We also recommend that a re-strike test with a PDA and CAPWAP analysis be completed on previously tested piles a minimum of 24 hours after installation to verify that piles have been driven to the required nominal driving resistance. We recommend that the indicator piles are monitored to ensure that pile driving stresses in the piles are within allowable limits.

Additional Pile Driving Considerations

The glacial till may contain cobbles and boulders, therefore, the pile driving contractor should be prepared to pre-drill through potential obstructions within the glacial till materials. We recommend a maximum pre-drill hole diameter approximately equal to the diagonal width of the H-piles.

After pile driving the hole should be backfilled with lean concrete using tremie methods grouting from the bottom of the pre-drill hole to the ground surface. Alternatively, the pre-drill hole could be backfilled with sand.

The contractor may have to use casing to keep the pre-drill hole from collapsing. If casing is used, then the inside diameter of the casing should be slightly larger than the diagonal width of the H-piles. The pre-drill hole should be backfilled with lean concrete using tremie methods, or sand, as the casing is removed.

Subgrade Preparation Procedures

We recommend the following subgrade preparation procedures for the proposed integral abutments:

- Excavations should be completed in the dry, as discussed in the Temporary Dewatering section. The use of a smooth edged bucket to limit disturbance of the clayey natural materials during excavations is recommended.

- Granular Borrow for Underwater Backfill should be used as backfill behind the proposed abutments, a minimum of 1.5 feet directly behind the heel of the proposed pile caps sloping up at a minimum 1.5 Horizontal to 1 Vertical (1.5H:1V).

- Granular Borrow for Underwater Backfill should be placed in maximum 12 inch thick loose lifts, and compacted to at least 95 percent of its maximum dry density as determined by ASTM D-1557, Method C (Modified Proctor).

Re-Use of On-site Soil

Based on soils encountered in test borings, the existing fill material consisted of sand with more than 20% silt and clay. Materials with fines up to 20% can be difficult to reuse if wet. We recommend those materials be reused in landscape areas or be disposed of offsite.

Temporary Dewatering

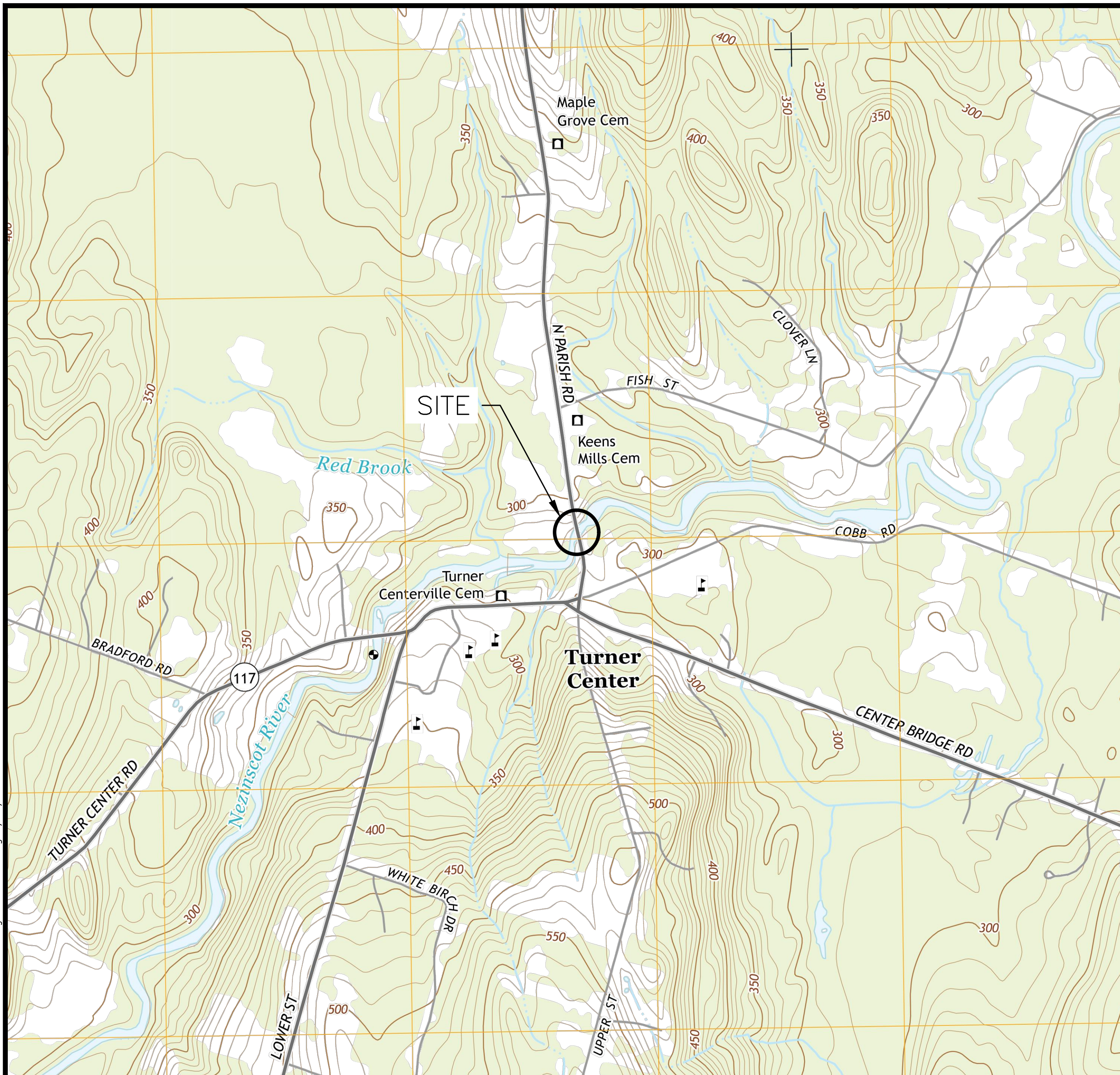
In general, groundwater was encountered below the bottom of the proposed abutments. However, groundwater may be encountered near the bottom of proposed abutments depending on the river stage at the time of construction. If needed, temporary excavation dewatering should be performed so that the work conducted is completed in the dry. It is likely that dewatering may be accomplished by well points or by pumping from filtered sumps installed in low points of the excavation. Discharge water should be managed in accordance with local, state and federal government requirements.

Excavation and Temporary Lateral Support

Excavation geometry should conform to OSHA excavation regulations contained in 29 CFR Part 1926, latest edition. In general, temporary soil slopes of 1.5H:1V (Soil Profile Type C), or flatter, appear appropriate but should be confirmed during construction based on conditions at the time of excavation.

Temporary earth support may be needed for construction of the bridge adjacent to the existing bridge. If needed, temporary earth support systems must be designed by a professional engineer registered in the state of Maine.

FIGURES



NORTH

2014 USGS TOPOGRAPHIC MAP

TURNER CENTER QUADRANGLE
TURNER, MAINE
CONTOUR INTERVAL 10 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)

APPROXIMATE SCALE
1 INCH = 2,000 FEET

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QUADRANGLE LOCATION

FIGURE 1

SITE LOCUS PLAN
NORTH PARISH BRIDGE (#2619)
NORTH PARISH ROAD OVER NEZINSCOT RIVER
TURNER, MAINE

PROJECT NO. 92270.02

DATE: FEBRUARY 2017

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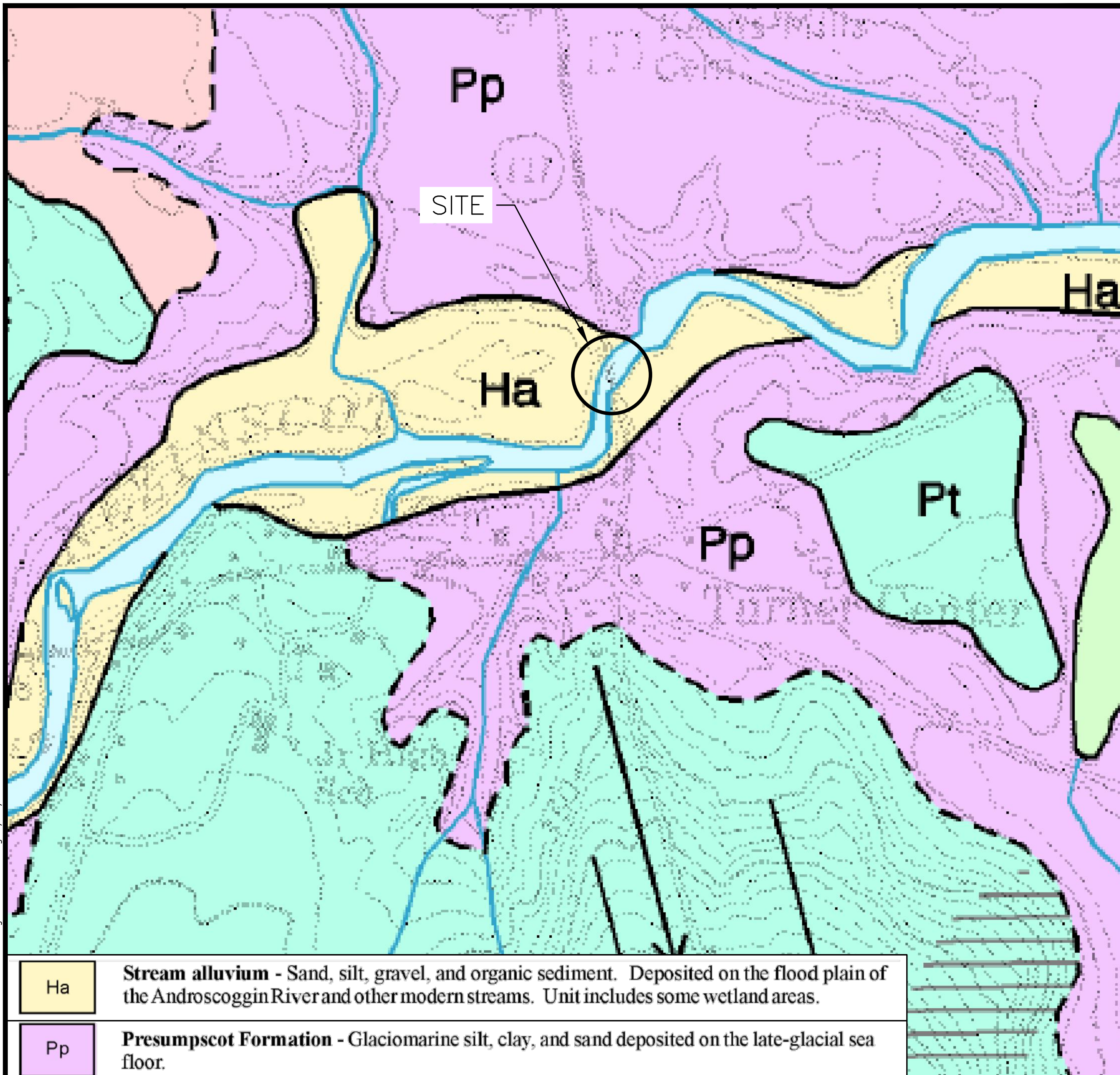

NORTH

2008 USGS SURFICIAL GEOLOGIC MAP

"SURFICIAL GEOLOGY OF THE TURNER CENTER QUADRANGLE, MAINE"
(SMITH, TOLMAN, MARVINNEY, TUCKER)
NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)
CONTOUR INTERVAL 10 FEET
ORIGINAL SCALE 1:24,000

NOT TO SCALE

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QUADRANGLE LOCATION

FIGURE 2

SURFICIAL GEOLOGY MAP

NORTH PARISH BRIDGE (#2619)
NORTH PARISH ROAD OVER NEZINSCOT RIVER
TURNER, MAINE

PROJECT NO. 92270.02

DATE: FEBRUARY 2017

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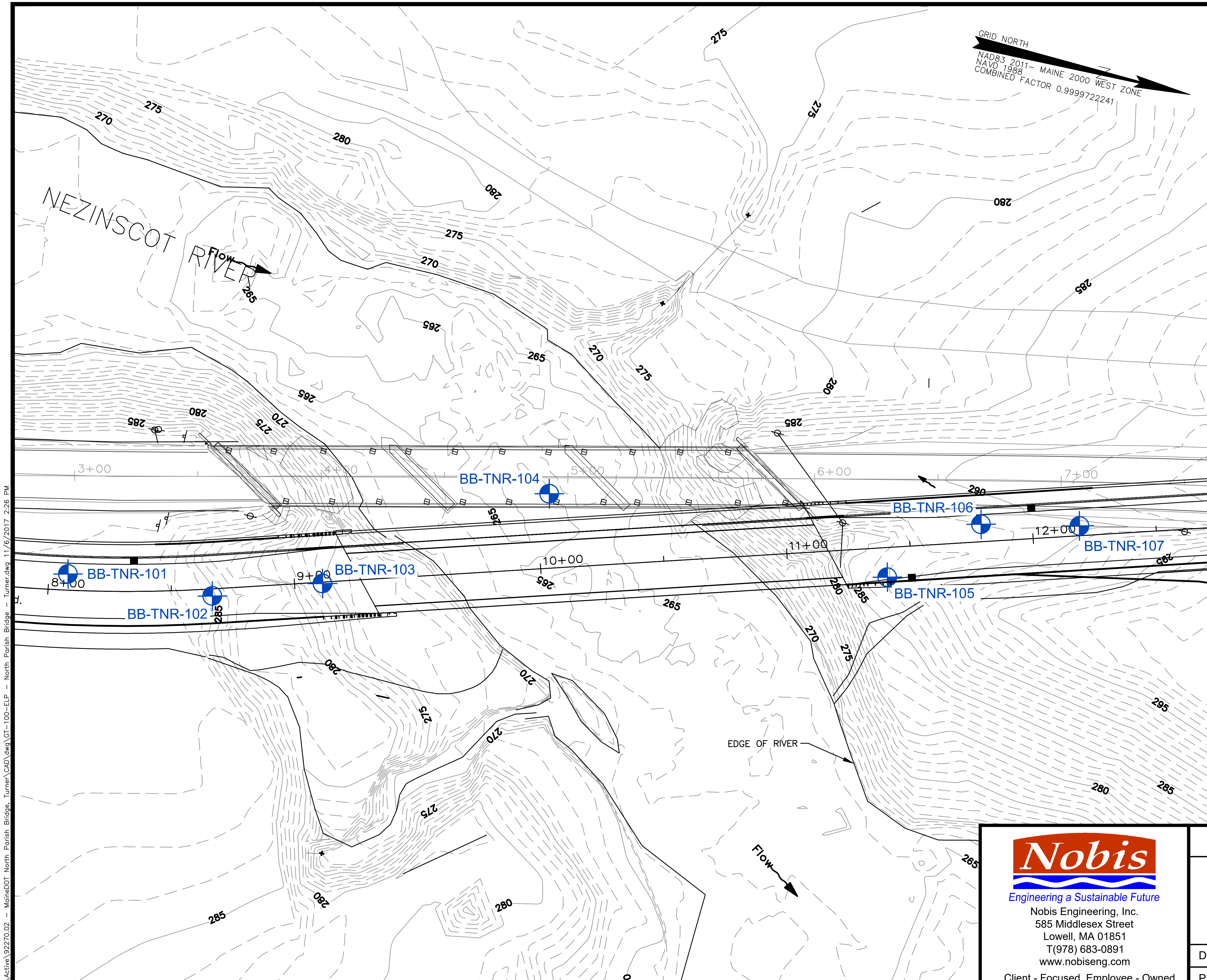

NOTES:

1. THE BASE PLAN WAS DEVELOPED FROM .DGN FILES PROVIDED BY HNTB ON SEPTEMBER 1, 2017.
2. LOCATIONS AND SITE FEATURES DEPICTED ARE APPROXIMATE AND GIVEN FOR ILLUSTRATIVE PURPOSES.
3. SOIL BORINGS WERE DRILLED BY NEW ENGLAND BORING CONTRACTORS OF DERRY, NEW HAMPSHIRE, AND OBSERVED BY NOBIS BETWEEN DECEMBER 8 AND DECEMBER 15, 2016.

LEGEND

BB-TNR-101 — APPROXIMATE BORING LOCATION OBSERVED BY NOBIS

0 40' 80'

GRAPHIC SCALE

Nobis

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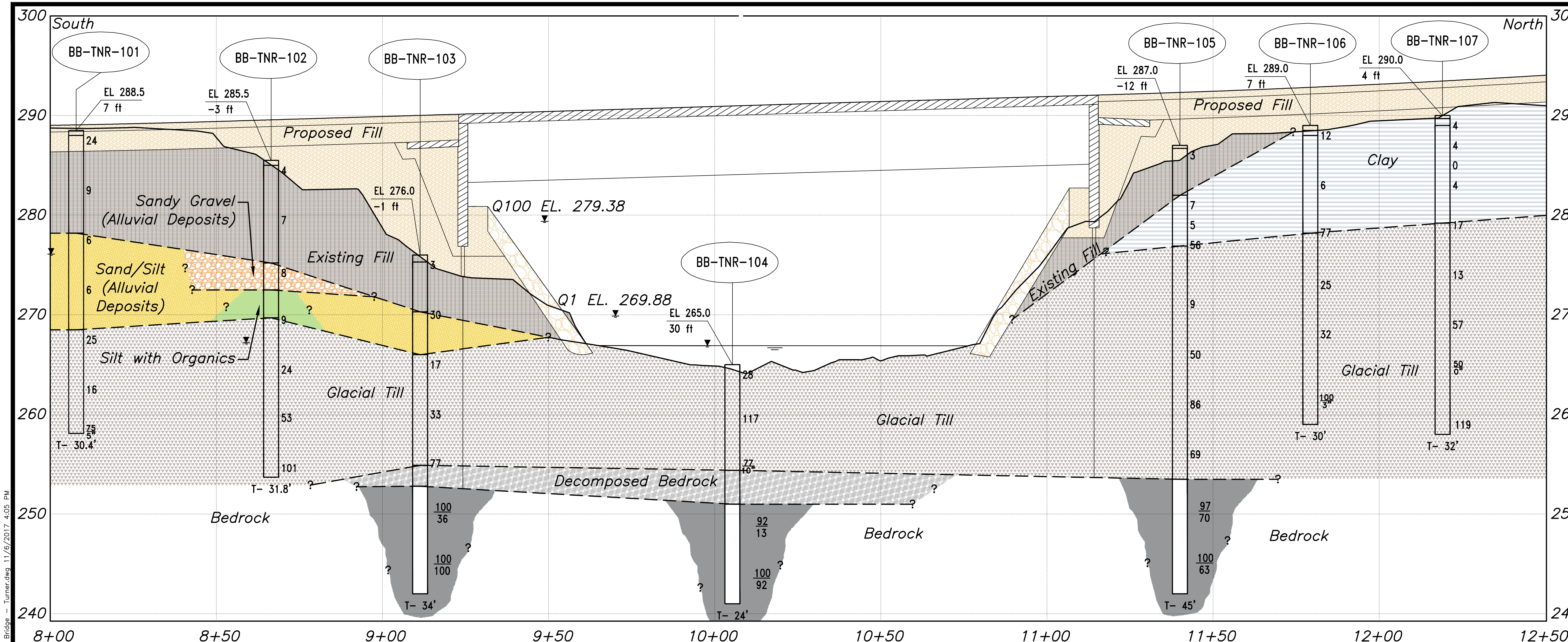
FIGURE 3

BORING LOCATION PLAN
NORTH PARISH BRIDGE (#2619)
NORTH PARISH ROAD OVER NEZINSCOT RIVER
TURNER, MAINE

DRAWN BY:	KK	CHECKED BY:	BTW
PROJECT NO.	92270.02	DATE:	NOVEMBER 2017



Interpretive Subsurface Profile

Station along Centerline of Proposed Bridge #2619 (feet)

NOTES:

1. THIS SUBSURFACE PROFILE WAS DEVELOPED FROM A TOPOGRAPHIC SURVEY .DGN FILE PROVIDED BY MAINEDOT ON NOVEMBER 7, 2016.
2. LINES REPRESENTING LIMITS OF STRATA ARE INTERPOLATED FROM SUBSURFACE EXPLORATION LOGS. THE SUBSURFACE EXPLORATIONS ARE WIDELY SPACED AND ARE BASED ON LIMITED SUBSURFACE INFORMATION COLLECTED DURING DRILLING. OTHER INTERPRETATIONS ARE POSSIBLE AND ACTUAL CONDITIONS MAY VARY FROM THOSE PRESENTED.
3. TOP OF BEDROCK SHOULD BE CONSIDERED APPROXIMATE AND CAN VARY SIGNIFICANTLY OVER SHORT DISTANCES.
4. WATER LEVELS PRESENTED WERE COLLECTED DURING DRILLING AND MAY NOT REPRESENT STABILIZED GROUNDWATER CONDITIONS. GROUNDWATER LEVELS WILL FLUCTUATE WITH SEASON, PRECIPITATION, AND NEARBY ACTIVITIES.
5. EXISTING AND PROPOSED SITE FEATURES DEPICTED ARE APPROXIMATE AND GIVEN FOR ILLUSTRATIVE PURPOSES.
6. SOIL BORINGS WERE DRILLED BY NEW ENGLAND BORING CONTRACTORS OF DERRY, NEW HAMPSHIRE, AND OBSERVED BY NOBIS BETWEEN DECEMBER 8 AND DECEMBER 18, 2016.
7. ELEVATIONS ARE PROVIDED IN FEET, AND ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
8. REFER TO FIGURE 3 FOR SUBSURFACE EXPLORATION LOCATIONS AND A PLAN VIEW OF THE SITE.

EXPLORATION LEGEND

BB-TNR-101	EXPLORATION DESIGNATION
EL 288.5	APPROX. GROUND SURFACE ELEVATION
1 ft	APPROX. OFFSET FROM BASELINE (FEET)*
	APPROXIMATE EXISTING GROUND SURFACE
24	SPT N-VALUE
75/5"	NUMBER OF BLOWS/DEPTH SAMPLER
▼	OBSERVED WATER LEVEL
	MAJOR STRATUM BOUNDARY BETWEEN DIFFERENT SOIL TYPES
100	ROCK CORE RECOVERY (%)
92	ROCK QUALITY DESIGNATION (%)
T- 30.4'	T=TERMINATION DEPTH

*POSITIVE OFFSETS ARE IN THE WEST DIRECTION. NEGATIVE OFFSETS ARE IN THE EAST DIRECTION.

STRATA LEGEND

- FILL
- SANDY GRAVEL
- SAND / SILT
- SILT WITH ORGANICS
- GLACIAL TILL
- DECOMPOSED BEDROCK

HORIZONTAL SCALE IN FEET: 0, 30', 60'

VERTICAL SCALE IN FEET: 0, 10', 20'

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FIGURE 4

SUBSURFACE PROFILE
NORTH PARISH BRIDGE (#2619)
NORTH PARISH ROAD OVER NEZINSCOT RIVER
TURNER, MAINE

DRAWN BY:	PCC/KK	CHECKED BY:	BTW
PROJECT NO.	92270.02	DATE:	NOVEMBER 2017

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APPENDIX A – Limitations

GEOTECHNICAL LIMITATIONS

Explorations and Subsurface Conditions

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

   In preparing this report, Nobis relied on certain information provided by the Client and other parties referenced therein which were made available to Nobis at the time of our evaluation. Nobis did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

2. The generalized soil profile described in the text is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretations of widely spaced explorations and samples; actual soil transitions are probably more erratic. For specific information, refer to the exploration logs.

3. Water level readings have been made in the explorations at times and under conditions stated on the logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, and other factors occurring since the time measurements were made. The water table encountered in the course of the work may differ from that indicated in the Report.

   Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

4. Nobis' geotechnical services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

Additional Services

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

Use of Report

6. Nobis prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in our proposal and/or report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Reliance by any party not expressly identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to Nobis.

   This report is for design purposes only and is not sufficient to prepare an accurate construction bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to design considerations only.

7. Nobis' findings and conclusions are based on the work conducted as part of the scope of work set forth in our proposal and/or report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions considering the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the project design has been altered in any way, Nobis shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.

8. Nobis' services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

Compliance with Codes and Regulations

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

Opinion of Cost

10. This report may contain or be based on comparative cost opinions for the purpose of evaluating alternative foundation schemes. These opinions may also involve approximate quantity evaluations. It should be noted that quantity estimates may not be accurate enough for construction bids. In addition, since we are not professional estimators of labor and materials cost, the evaluation of construction costs should be considered as approximate guidelines and could vary significantly from actual costs. Nobis does not guarantee the accuracy of our cost opinions as compared to contractor's bids for construction costs.

END OF LIMITATIONS

APPENDIX B – Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-101
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	288.5	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 14, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 4+06, 2 feet Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	12.4' bgs
Hammer Efficiency Factor: 0.63		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.00 - 2.00	7/9/15/15	24	25	RC	288.00		1D-A: (6") Dark brown, dry, fine to medium Silty SAND, very few roots, (Topsoil).	
										0.50	
										1D-B: (14") Brown, dry, medium dense, fine to coarse SAND, some clayey silt, trace fine gravel, few asphalt fragments, (Fill).	
5	2D	24/17	5.00 - 7.00	3/5/4/4	9	9				Grey-brown, dry, loose, fine to medium Silty SAND, trace fine gravel, (Fill).	A-4, SM
10	3D	24/8	10.00 - 12.00	4/3/3/4	6	6		278.20		3D-A: (3") Brown, wet, loose, fine to medium SAND, some silt, few wood fragments, (Fill).	
										10.30	
										3D-B: (5") Grey, wet, loose, fine to medium SAND, little silt, (Alluvial Deposit).	
15	4D	24/14	15.00 - 17.00	4/2/4/6	6	6				4D-A: (11") Grey-brown with orange staining, wet, loose, Clayey SILT, little fine to medium sand, (Alluvial Deposit).	A-4, ML
										4D-B: (3") Grey, wet, loose, fine to medium SAND, little silt, (Alluvial Deposit).	
20	5D	24/7	20.00 - 22.00	17/12/13/13	25	26		268.50		20.00	
										Grey, wet, medium dense, fine to coarse Sandy GRAVEL, little silt, (Glacial Till).	
25											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-101

WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	288.5	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 14, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 4+06, 2 feet Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	12.4' bgs
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.00 - 27.00	5/7/9/8	16	17	RC			6D-A: (4") Grey, wet, medium dense, fine to medium Silty SAND, little fine gravel, (Glacial Till).	
										6D-B: (8") Grey, wet, very stiff, Clayey SILT, some fine sand, (Glacial Till).	
30	7D	5/4	30.00 - 30.42	75/5"	---			258.10		Grey, wet, hard, Silty CLAY, some fine to coarse sand, (Glacial Till). 30.40	
										Bottom of Exploration at 30.40 feet below ground surface.	
35											
40											
45											
50											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River Location: Turner, Maine	Boring No.: BB-TNR-102 WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	285.5	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 14, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 4+65, 5 feet Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	18.3' bgs
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.00 - 2.00	WOH/2/2/4	4	4	RC	285.00		1D-A: (6") Dark brown, dry, fine to medium Silty SAND, very few roots, (Topsoil).	
										0.50	
										1D-B: (7") Brown, dry, very loose, fine to coarse Silty SAND, trace fine gravel, very few roots, (Fill).	
5	2D	24/9	5.00 - 7.00	3/3/4/10	7	7				Brown, dry, loose, fine to coarse Clayey SAND, trace fine gravel, very few wood fragments, (Fill).	
10								275.20		10.30	
	3D	24/2	10.30 - 12.30	5/3/5/6	8	8				Brown, wet, loose, fine to coarse GRAVEL, some fine to coarse sand, trace silt, (Alluvial Deposit).	
								272.50		13.00	
										Dark brown cuttings with organic odor.	
15	4D	24/13	15.00 - 17.00	WOH/1/8/20	9	9				4D-A: (9") Grey-brown, wet, very loose, Sandy SILT, very few root fibers, faint organic odor, (Alluvial Deposit).	A-4, ML
								269.70		15.80	
										4D-B: (4") Grey, wet, medium dense, fine Sandy GRAVEL, trace silt, (Glacial Till).	
20	5D	24/11	20.00 - 22.00	10/11/13/8	24	25				Grey, wet, medium dense, fine to coarse Sandy GRAVEL, some silt, (Glacial Till).	
25											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-102
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	285.5	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 14, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 4+65, 5 feet Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	18.3' bgs
Hammer Efficiency Factor: 0.63		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	20/8	25.00 - 26.67	21/25/28/50/2"	53	56	RC	260.50		25.00 Grey, wet, very dense, fine to coarse GRAVEL, some fine to coarse sand, little silt, little clay, (Glacial Till).	
30	7D	21/17	30.00 - 31.75	23/58/43/50/3"	101	106		255.50		30.00 Grey, wet, hard, Silty CLAY, some fine to coarse sand, trace fine gravel, (Glacial Till).	
								253.70		31.80 **Bottom of Exploration at 31.80 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-103
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	276.0	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 13, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2
Boring Location:	Sta 5+10, 1 foot Rt.	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	1/1/2/1	3	3	RC	275.30		Brown, dry, fine to coarse SAND, some silt, few wood fragments, (Topsoil). 0.70	
5	2D	24/12	5.00 - 7.00	11/20/10/10	30	32		270.30		2D:-A: (8") Brown, wet, medium dense, Gravelly fine to coarse SAND, little silt, (Fill). 5.70 2D-B: (4") Grey, wet, medium dense, fine to medium Silty SAND, (Alluvial Deposit).	
10	3D	24/9	10.00 - 12.00	6/7/10/9	17	18		266.00		10.00 Grey, wet, very stiff, Clayey SILT, little fine to coarse sand, trace fine gravel, (Glacial Till).	
15	4D	24/14	15.00 - 17.00	8/14/19/22	33	35				Grey, wet, dense, fine to coarse Silty SAND, trace fine gravel, (Glacial Till).	
20	5D	22/15	20.00 - 21.83	14/26/51/50/4"	77	81		254.90		5D-A: (14") Grey, wet, very dense, fine to coarse Silty SAND, little fine gravel, (Glacial Till). 21.10 5D-B: (1") Grey, wet, DECOMPOSED BEDROCK.	
								252.80		23.20 Top of Bedrock Elevation at 252.8 feet. Roller coned from 23.2' to 24.0' bgs to set core barrel.	
25	R1	60/60	24.00 - 29.00	RQD = 36%			NQ2			R1-A (42"): Bedrock: Dark grey, fine to coarse-grained,	

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-103
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	276.0	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 13, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2
Boring Location:	Sta 5+10, 1 foot Rt.	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										METASANDSTONE, moderately hard, slightly to moderately weathered, horizontal to moderately-dipping joints, very close to close joints. R1-B (18"): Bedrock: Grey-white, medium to coarse-grained GRANITE, hard, fresh to slightly weathered, horizontal to moderately-dipping joints, very close to moderately-close joints. Sangerville Formation. Rock Mass Quality = Poor. R1: Core Times (min:sec) 24-25 feet (2:30) 25-26 feet (1:00) 26-27 feet (1:00) 27-28 feet (1:00) 28-29 feet (2.15)	
	R2	60/60	29.00 - 34.00	RQD = 100%							UCT q_p= 12,286 psi
30										R2: Bedrock: Grey-white, medium to coarse-grained GRANITE, hard, fresh to slightly weathered, horizontal to moderately dipping joints, close to wide joints. Sangerville Formation. Rock Mass Quality = Excellent. R2: Core Times (min:sec) 29-30 feet (1:30) 30-31 feet (1:00) 31-32 feet (1:15) 32-33 feet (1:15) 33-34 feet (1:30)	
								242.00		34.00	
35										**Bottom of Exploration at 34.00 feet below ground surface.**	
40											
45											
50											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-104
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	265	**Auger ID/OD:**	N/A
Operator:	C. Dupois	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Truck/ B-53 Mobile	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 8, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2
Boring Location:	Sta 6+04, 30 feet Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	1.9 feet above mudline
Hammer Efficiency Factor: 0.572		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	8/14/14/17	28	27	RC			Grey, wet, very stiff, Silty CLAY, some fine to coarse sand, little fine gravel, (Glacial Till).	
5	2D	22/6	4.50 - 6.33	50/65/52/60/4"	117	112				2D-A: (5") Grey, wet, hard, Silty CLAY, some fine to coarse sand, little fine gravel, (Glacial Till). 2D-B: (1") Grey, wet, very dense, cobble/boulder fragments, (Glacial Till).	
10	3D	16/14	9.50 - 10.83	18/22/50/4"	---			254.40		3D-A: (13") Grey, wet, hard, Silty CLAY , some fine to coarse sand, little fine gravel, (Glacial Till). 10.60 3D-B: (1") Grey, wet, DECOMPOSED BEDROCK.	
15	R1	60/55	14.00 - 19.00	RQD = 13%			NQ2	251.00		14.00 Top of Bedrock Elevation 251.0 feet. R1-A (18"): Bedrock: Grey-white, medium to coarse grained GRANITE, hard, slightly to moderately weathered, horizontal to low-angle joints, very close to close joints. R1-B (42"): Bedrock: Dark grey, fine to coarse-grained, METASANDSTONE, moderately hard, slightly to moderately weathered, low-angle joints, very close to close joints. Sangerville Formation. Rock Mass Quality = Very Poor. R1: Core Times (min:sec) 14-15 feet (2:30) 15-16 feet (3:30) 16-17 feet (2:30) 17-18 feet (3:15) 18-19 feet (2:15)	UCT q_p= 10,060 psi
20	R2	60/60	19.00 - 24.00	RQD = 92%				241.00		R2: Bedrock: Dark grey-grey, fine to coarse-grained METASANDSTONE, moderately hard, slightly to moderately weathered, horizontal to steeply-dipping, close to moderately-close joints. Sangerville Formation. Rock Mass Quality = Excellent. R2: Core Times (min:sec) 19-20 feet (2:00)	UCT q_p= 5,920 psi
25											

Remarks:

-Borehole backfilled with native soils.
-Cored through deck to mudline approximately 23.5 feet below roadway. Concrete deck was 9" thick, 3'7" to bottom of I Beams. Deck was repaired and patched after boring completion.
-bgs = below ground surface.
-Automatic Hammer ID # AH1.
-No Casing Blow Counts were recorded.

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:	North Parish Bridge (#2619) over Nezinscot River	**Boring No.:**	BB-TNR-104		
Location:	Turner, Maine	**WIN:**	18749.00		
Driller:	New England Boring Contractors	**Elevation (ft.)**	265	**Auger ID/OD:**	N/A
Operator:	C. Dupois	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Truck/ B-53 Mobile	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 8, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2
Boring Location:	Sta 6+04, 30 feet Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	1.9 feet above mudline
Hammer Efficiency Factor:	0.572	**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
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										20-21 feet (2:00) 21-22 feet (1:45) 22-23 feet (2:15) 23-24 feet (3:30)	
										24.00	
										Bottom of Exploration at 24.00 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

-Borehole backfilled with native soils.
-Cored through deck to mudline approximately 23.5 feet below roadway. Concrete deck was 9" thick, 3'7" to bottom of I Beams. Deck was repaired and patched after boring completion.
-bgs = below ground surface.
-Automatic Hammer ID # AH1.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-105
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	287	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 14, 2016/Dec. 15, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2
Boring Location:	Sta 7+38, 13 feet Rt.	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/17	0.00 - 2.00	1/1/2/4	3	3	RC	286.70		1D-A: (4") Dark brown, dry, fine to medium Silty SAND, very few roots, (Topsoil). 0.30 1D-B: (13") Grey-brown, dry, soft, Silty CLAY, some fine sand, very few root fibers, (Fill).	
5	2D	24/24	5.00 - 7.00	4/3/4/4	7	7		282.00		5.00 Olive-brown, moist, medium stiff, Clayey SILT, trace fine sand, (Presumpscot Formation).	A-6, CL WC=30.0% LL=37 PL= 23 PI=14
	3D	24/24	7.00 - 9.00	1/2/3/2	5	5				Olive-brown, moist, medium stiff, CLAY, trace fine sand, (Presumpscot Formation).	WC=33.2%
	4D	24/20	9.00 - 11.00	1/5/51/50	56	59				4D-A: (13") Olive-brown, wet, CLAY, trace fine sand, (Presumpscot Formation).	WC=32.0%
10								276.90		10.10 4D-B: (7") Grey with orange-brown, wet, very dense, fine to coarse GRAVEL, some fine to coarse sand, trace silt, (Glacial Till).	
15	5D	24/16	15.00 - 17.00	5/4/5/7	9	9				Grey, wet, stiff, Silty CLAY, some fine to coarse sand, trace fine gravel, (Glacial Till).	
20	6D	24/19	20.00 - 22.00	10/22/28/39	50	53				Grey, wet, hard, Clayey SILT, some fine to coarse gravel, some fine to coarse sand, (Glacial Till).	
25											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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-TNR-105

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-105
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	287	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 14, 2016/Dec. 15, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2
Boring Location:	Sta 7+38, 13 feet Rt.	**Casing ID/OD:**	3"/3.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/24	25.00 - 27.00	24/27/59/52	86	90	RC			Grey, wet, hard, Clayey SILT, some fine to coarse sand, little fine to coarse gravel, (Glacial Till).	
30	8MD	24/0	30.00 - 32.00	22/29/40/44	69	72				Failed sample attempt.	
								253.50		33.50 Top of Bedrock at Elevation 253.5 feet. Roller coned from 33.5' to 35.0' bgs to set core barrel.	
	9MD	0/0	34.99 - 34.99	50/0"	---					Failed sample attempt.	
35	R1	60/58	35.00 - 40.00	RQD = 70%			NQ2			R1: Bedrock: Grey, fine to coarse-grained METASANDSTONE, moderately hard, slightly to moderately weathered, close to moderately-close joints, horizontal to low-angle joints. Sangerville Formation. Rock Mass Quality = Fair. R1: Core Times (min:sec) 35-36 feet (3:00) 36-37 feet (2:30) 37-38 feet (2:15) 38-39 feet (2:45) 39-40 feet (3:00)	UCT q_p= 3,314 psi
40	R2	60/60	40.00 - 45.00	RQD = 63%						R2: Bedrock: Dark grey, fine to coarse-grained METASANDSTONE, moderately hard, moderately weathered, horizontal to steeply-dipping joints, very close to moderately close joints. Sangerville Formation. Rock Mass Quality = Fair. R2: Core Times (min:sec) 40-41 feet (1:15) 41-42 feet (1:00) 42-43 feet (1:00) 43-44 feet (1:15) 44-45 feet (1:15)	
45								242.00		45.00 **Bottom of Exploration at 45.00 feet below ground surface.**	
50											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-106
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	289	**Auger ID/OD:**	N/A
Operator:	C. Dupois	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Truck/ B-53 Mobile	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 9, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 7+78, 7 feet Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.572	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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 = 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.00 - 2.00	5/5/7/7	12	11	RC	288.50 288.00		1D-A: (6") Brown, dry, fine to coarse SAND, little silt, very few root fibers, (Topsoil). 0.50 1D-B:(10") Orange-brown, dry, medium dense, fine to coarse SAND, some silt, very few root fibers, (Subsoil). 1.00	
5	2D	24/20	5.00 - 7.00	4/3/3/5	6	6				Grey, moist, medium stiff, Clayey SILT, trace fine sand, (Presumpscot Formation).	A-6, CL WC=27.1% LL=38 PL=21 PI=17
10	3D	18/17	10.00 - 11.50	1/27/50/6"	---			278.20		3D-A: (10") Grey, wet, CLAY, (Presumpscot Formation). 10.80 3D-B: (7") Grey, wet, very dense, fine Silty GRAVEL, some fine to coarse sand, (Glacial Till).	WC=33.0%
15	4D	24/8	15.00 - 17.00	18/13/12/14	25	24				Grey, wet, medium dense, fine to coarse Silty SAND, trace fine gravel, (Glacial Till).	
20	5D	23/16	20.00 - 21.92	15/15/17/50/5"	32	31				Grey, wet, hard, Clayey SILT, some fine to coarse sand, little fine to coarse gravel, (Glacial Till).	
	R1	60/12	22.50 - 27.50				NQ2			Attempted bedrock core: grey, Clayey SILT, some fine to coarse sand, trace fine gravel, with inferred boulder fragments, (Glacial Till).	
25											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # AH1.
-No Casing Blow Counts were recorded.

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* 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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-106
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	289	**Auger ID/OD:**	N/A
Operator:	C. Dupois	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Truck/ B-53 Mobile	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 9, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 7+78, 7 feet Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.572	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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 = 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											
	6D	3/3	27.50 - 27.75	100/3"	---		RC			Grey, wet, fine to coarse Gravelly SAND, some clayey silt, (Glacial Till). Roller coned through inferred cobble from 27.8 feet to 29 feet bgs.	
30								259.00		30.00 **Bottom of Exploration at 30.00 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # AH1.
-No Casing Blow Counts were recorded.

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-TNR-106

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-107
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	290	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 15, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 8+18, 4 feet Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor: 0.63		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.00 - 2.00	3/2/2/3	4	4	RC	289.70		1D-A: (4") Brown, dry, fine to coarse SAND, some silt, very few root fibers, (Topsoil). 0.30	
								289.00		1D-B: (8") Brown, moist, very loose, fine to coarse SAND, little silt, very few root fibers, (Subsoil). 1.00	
	2D	24/21	2.00 - 4.00	WOH/2/2/2	4	4				1D-C: (8") Brown, moist, soft, Silty CLAY, trace fine sand, (Presumpscot Formation). Olive-brown, wet, soft, Silty CLAY, (Presumpscot Formation).	
	3D	24/24	4.00 - 6.00	WOH/WOH/WOH/1	---					Grey, wet, very soft, Silty CLAY, (Presumpscot Formation).	WC=34.1%
5											
	4D	24/24	6.00 - 8.00	1/2/2/2	4	4				Grey, wet, very soft, Clayey SILT, trace fine sand, (Presumpscot Formation) .	A-6, CL WC=31.5% LL=35 PL=20 PI=15
	V1		8.00 - 8.43	S_u=843/155 psf						65x130mm vane raw torque readings: V1: 30/5.5 ft-lbs V2: 24.5/4 ft-lbs	
	V2		9.00 - 9.43	S_u=689/112 psf							
10	5D	24/15	10.00 - 12.00	WOH/2/15/18	17	18		279.20		5D-A: (10") Grey, wet, medium stiff, Silty CLAY, (Presumpscot Formation). 10.80	WC=36.4%
										5D-B: (5") Grey, wet, medium dense, fine to coarse GRAVEL, some fine to coarse sand, little silt, (Glacial Till).	
15	6D	24/14	15.00 - 17.00	6/6/7/8	13	14				Grey, wet, stiff, Silty CLAY, some fine to coarse sand, trace fine gravel, (Glacial Till).	
20	7D	24/20	20.00 - 22.00	12/16/41/44	57	60				Grey, wet, hard, Clayey SILT, some fine to coarse sand, trace fine gravel, (Glacial Till).	
25											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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: North Parish Bridge (#2619) over Nezinscot River
Location: Turner, Maine

Boring No.: BB-TNR-107
WIN: 18749.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	290	**Auger ID/OD:**	N/A
Operator:	B. Enos	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	Diedrich D-50 Track Rig	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	Dec. 15, 2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	Sta 8+18, 4 feet Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	8D	6/4	25.00 - 25.50	50-50/0"	---		RC			Grey, wet, Clayey SILT, trace fine to coarse sand, coarse piece of gravel in spoon tip, (Glacial Till).	
30	9D	24/18	30.00 - 32.00	45/47/72/67	119	125				Grey, wet, very dense, fine to coarse Silty SAND, little fine gravel, (Glacial Till).	
								258.00		32.00 **Bottom of Exploration at 32.00 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

-Borehole backfilled with native soils.
-bgs = below ground surface.
-Automatic Hammer ID # MTBAH3.
-No Casing Blow Counts were recorded.

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-TNR-107

APPENDIX C – Photo Logs of Rock Core Samples

Engineering a Sustainable Future

92270.02 North Parish Bridge (#2619) – Turner, ME – Photo Log No. 1 – Boring BB-TNR-103 Rock Cores – December 13, 2016

Photo 1: Boring BB-TNR-103 Rock Core Samples R-1 (Row 1) and R-2 (Row 2); top sections of each sample.

Photo 2: Boring BB-TNR-103 Rock Core Samples R-1 (Row 1) and R-2 (Row 2); mid sections of each sample.

Nobis Engineering, Inc.
585 Middlesex Street
Lowell, MA 01851
T (978) 683-0891

Photo 3: Boring BB-TNR-103 Rock Core Samples R-1 (Row 1) and R-2 (Row 2); bottom sections of each sample.

Engineering a Sustainable Future

92270.02 North Parish Bridge (#2619) – Turner, ME – Photo Log No. 2 – Boring BB-TNR-104 Rock Cores – December 8, 2016

Photo 1: Boring BB-TNR-104 Rock Core Samples R-1 (Row 1) and R-2 (Row 2); top sections of each sample.

Photo 2: Boring BB-TNR-104 Rock Core Samples R-1 (Row 1) and R-2 (Row 2); mid to bottom sections of each sample.

92270.02 North Parish Bridge (#2619) – Turner, ME – Photo Log No. 3 – Boring BB-TNR-105 Rock Cores – December 14 and December 15, 2016

Photo 1: Boring BB-TNR-105 Rock Core Samples R-1 (Row 3) and R-2 (Row 4); top sections of each sample.

Photo 2: Boring BB-TNR-105 Rock Core Samples R-1 (Row 3) and R-2 (Row 4); top to mid sections of each sample.

Photo 3: Boring BB-TNR-105 Rock Core Samples R-1 (Row 3) and R-2 (Row 4); mid to bottom sections of each sample.

Photo 4: Boring BB-TNR-105 Rock Core Samples R-1 (Row 3) and R-2 (Row 4); bottom sections of each sample.

Photo 5: Boring BB-TNR-105 Rock Core Samples R-1 (Row 3) and R-2 (Row 4); entire sections of each sample.

APPENDIX D – Laboratory Test Results

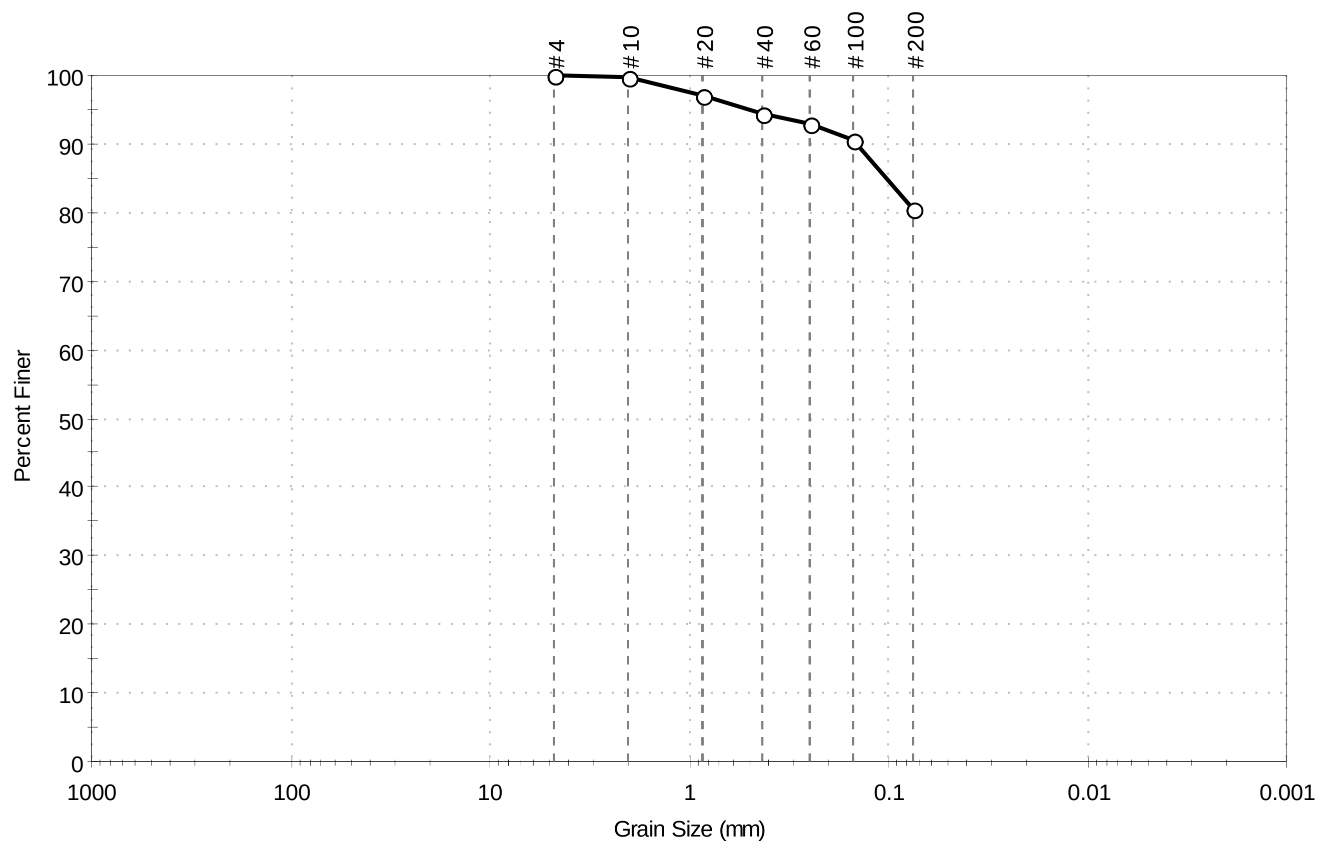
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-TNR-105	2D	---	Moist, olive brown clay	30.0
BB-TNR-105	3D	---	Moist, brown clay	33.2
BB-TNR-105	4D- A	---	Moist, olive brown clay	32.0
BB-TNR-106	2D	---	Moist, brown clay	27.1
BB-TNR-106	3D- A	---	Moist, olive clay	33.0
BB-TNR-107	3D	---	Moist, olive clay	34.1
BB-TNR-107	4D	---	Moist, gray clay	31.5
BB-TNR-107	5D- A	---	Moist, olive clay	36.4

Notes: Temperature of Drying : 110° Celsius

Client:	Nobis Engineering, Inc.				
Project:	N. Parish Rd Bridge				
Location:	Turner, ME			Project No:	GTX-305852
Boring ID:	BB-TNR-101	Sample Type:	jar	Tested By:	jbr
Sample ID:	4D-A	Test Date:	01/16/17	Checked By:	emm
Depth :	---	Test Id:	401994		
Test Comment:	---				
Visual Description:	Moist, gray silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	19.4	80.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	97		
#40	0.42	94		
#60	0.25	93		
#100	0.15	91		
#200	0.075	81		

Coefficients

D_{85} = 0.1019 mm D_{30} = N/A

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

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

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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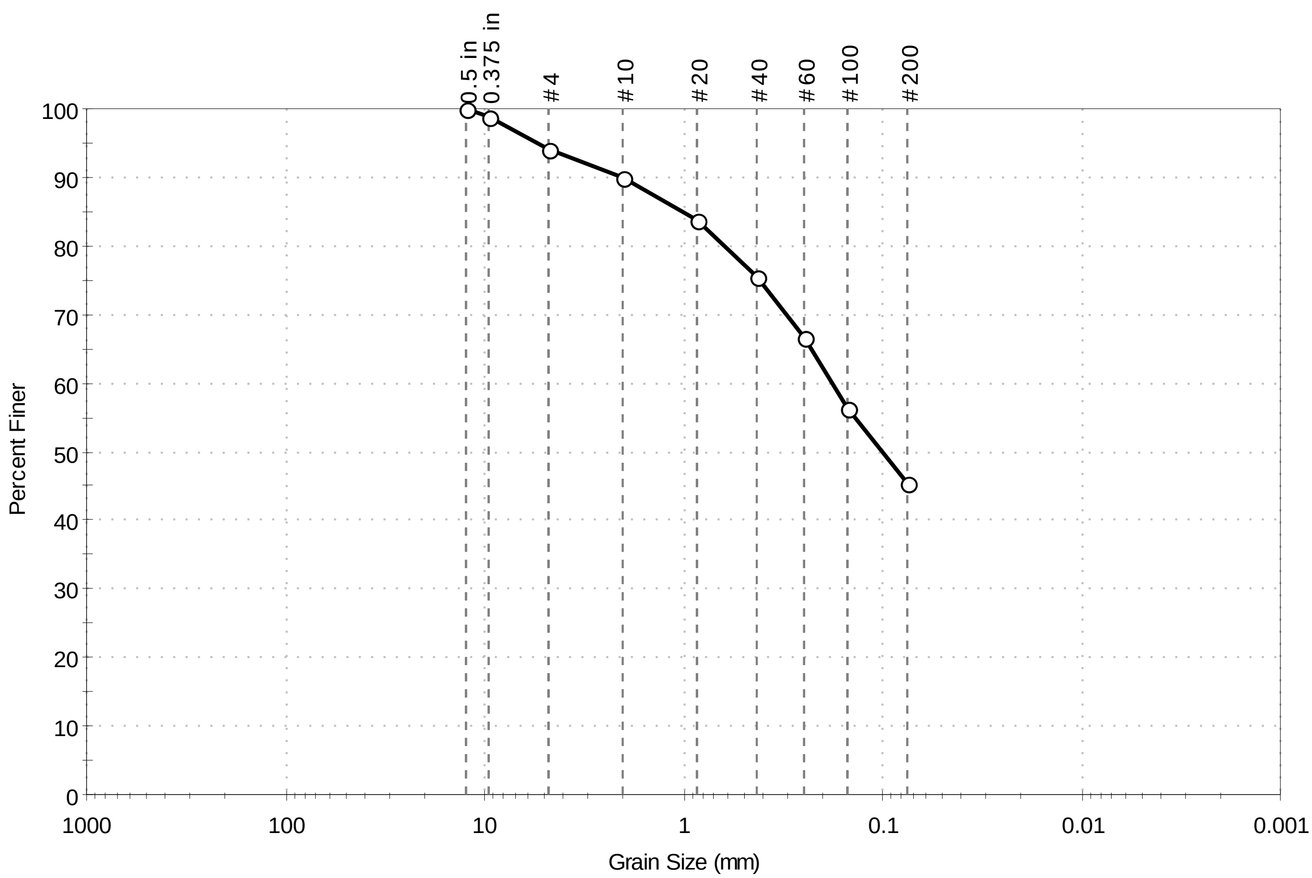
GeoTesting EXPRESS

Client: Nobis Engineering, Inc.			
Project: N. Parish Rd Bridge			
Location: Turner, ME		Project No:	GTX-305852
Boring ID: BB-TNR-101	Sample Type: jar	Tested By:	jbr
Sample ID: S-2D	Test Date: 01/16/17	Checked By:	emm
Depth : ---	Test Id: 401993		
Test Comment: ---			
Visual Description: Moist, grayish brown silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	5.9	48.7	45.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	94		
#10	2.00	90		
#20	0.85	84		
#40	0.42	75		
#60	0.25	67		
#100	0.15	56		
#200	0.075	45		

Coefficients

D_{85} = 0.9952 mm D_{30} = N/A

D_{60} = 0.1801 mm D_{15} = N/A

D_{50} = 0.1003 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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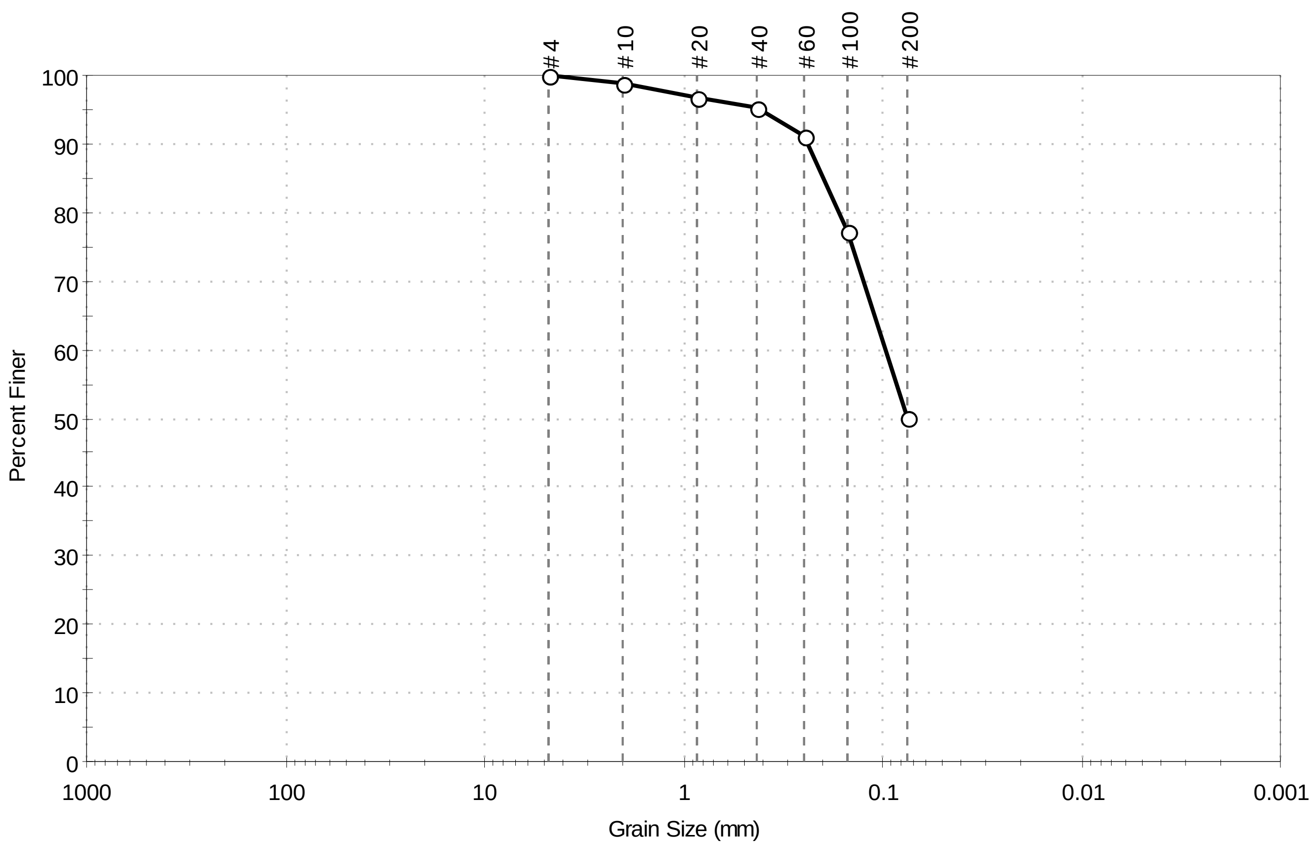
Client:	Nobis Engineering, Inc.		
Project:	N. Parish Rd Bridge		
Location:	Turner, ME	Project No:	GTX-305852
Boring ID: BB-TNR-102	Sample Type: jar	Tested By:	jbr
Sample ID: 4D-A	Test Date: 01/16/17	Checked By:	emm
Depth : ---	Test Id: 401995		
Test Comment:	---		
Visual Description:	Moist, dark gray sandy silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	49.8	50.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	97		
#40	0.42	95		
#60	0.25	91		
#100	0.15	77		
#200	0.075	50		

Coefficients

D_{85} = 0.2000 mm D_{30} = N/A

D_{60} = 0.0964 mm D_{15} = N/A

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

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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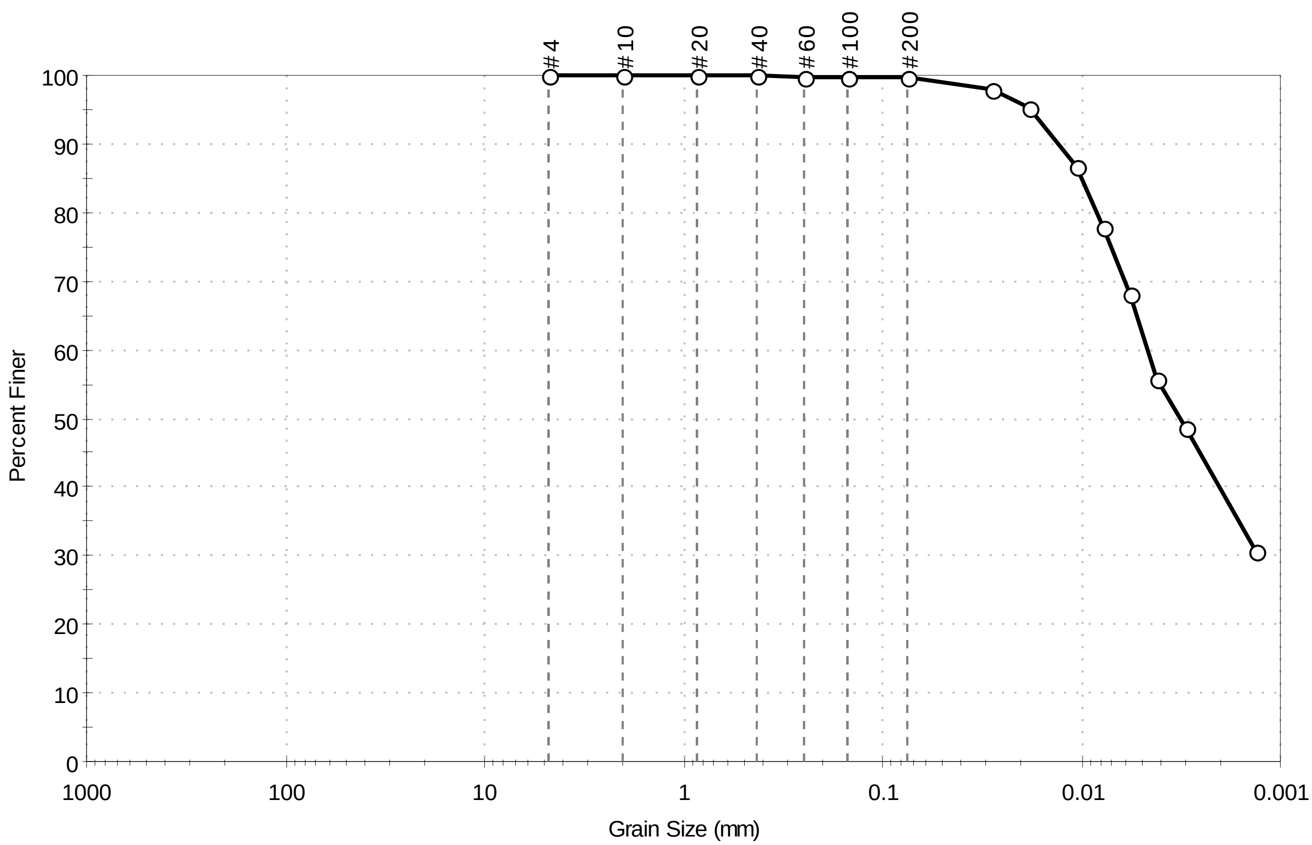
Client:	Nobis Engineering, Inc.		
Project:	N. Parish Rd Bridge		
Location:	Turner, ME	Project No:	GTX-305852
Boring ID: BB-TNR-105	Sample Type: jar	Tested By:	jbr
Sample ID: 2D	Test Date: 01/16/17	Checked By:	emm
Depth : ---	Test Id: 401996		
Test Comment:	---		
Visual Description:	Moist, olive brown clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	0.4	99.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0284	98		
---	0.0182	95		
---	0.0107	87		
---	0.0078	78		
---	0.0057	68		
---	0.0042	56		
---	0.0030	49		
---	0.0013	31		

Coefficients

D_{85} = 0.0100 mm
D_{30} = N/A
D_{60} = 0.0046 mm
D_{15} = N/A
D_{50} = 0.0032 mm
D_{10} = N/A
C_u = N/A
C_c = N/A

Classification

ASTM Lean clay (CL)

AASHTO Clayey Soils (A-6 (15))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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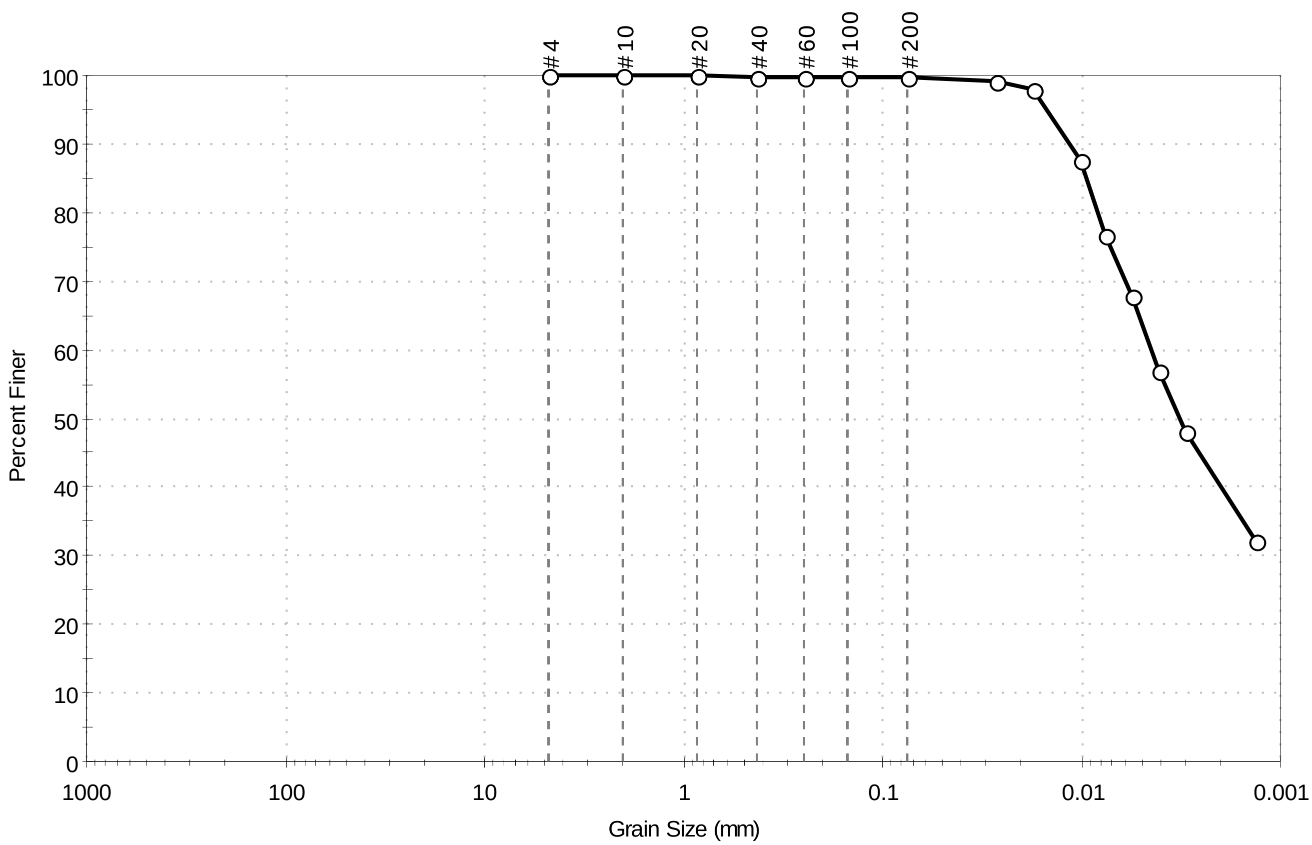
Client:	Nobis Engineering, Inc.		
Project:	N. Parish Rd Bridge		
Location:	Turner, ME	Project No:	GTX-305852
Boring ID: BB-TNR-106	Sample Type: jar	Tested By:	jbr
Sample ID: 2D	Test Date: 01/16/17	Checked By:	emm
Depth : ---	Test Id: 401997		
Test Comment:	---		
Visual Description:	Moist, brown clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	0.3	99.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0268	99		
---	0.0174	98		
---	0.0102	88		
---	0.0076	77		
---	0.0056	68		
---	0.0041	57		
---	0.0030	48		
---	0.0013	32		

Coefficients

D_{85} = 0.0095 mm	D_{30} = N/A
D_{60} = 0.0045 mm	D_{15} = N/A
D_{50} = 0.0032 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean clay (CL)

AASHTO Clayey Soils (A-6 (18))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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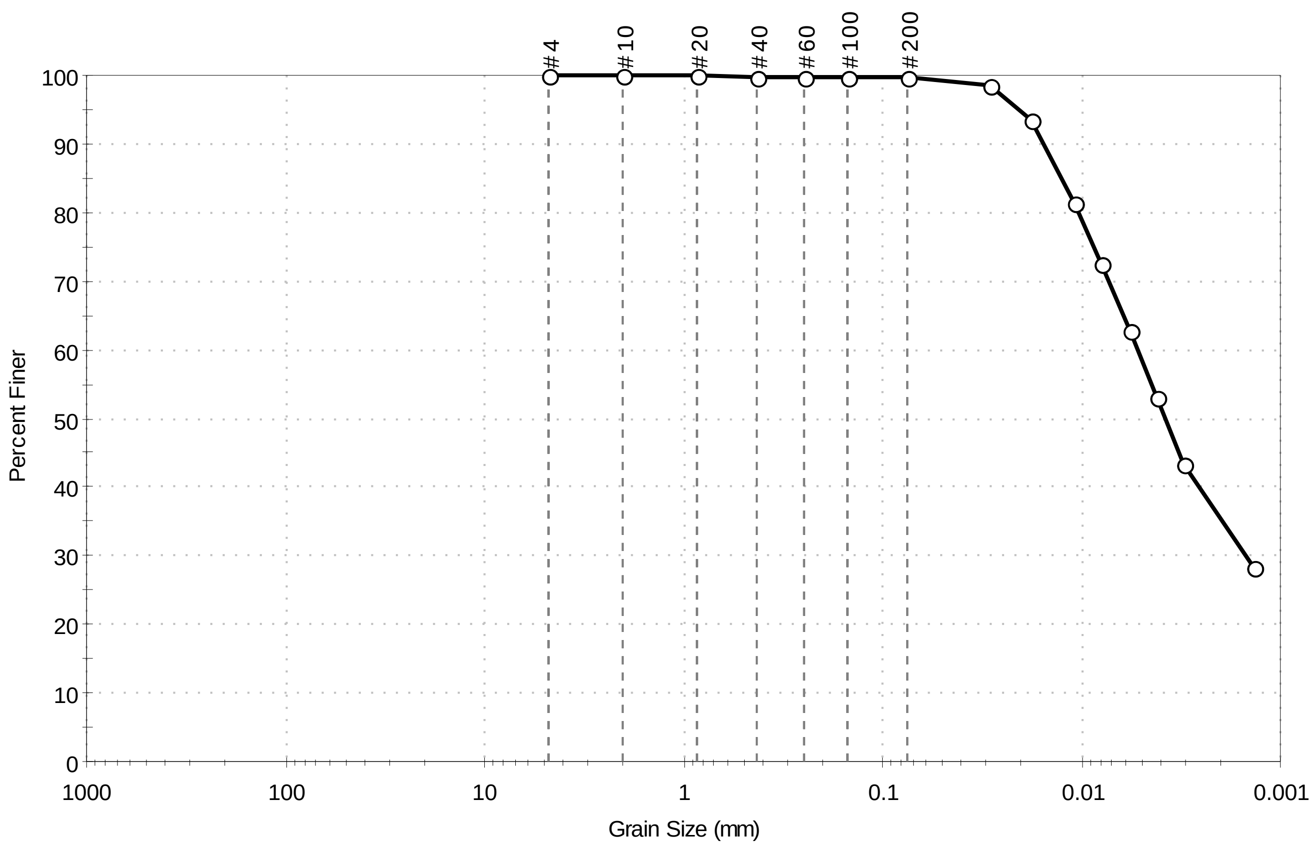
GeoTesting EXPRESS

Client:	Nobis Engineering, Inc.				
Project:	N. Parish Rd Bridge				
Location:	Turner, ME			Project No:	GTX-305852
Boring ID:	BB-TNR-107	Sample Type:	jar	Tested By:	jbr
Sample ID:	4D	Test Date:	01/16/17	Checked By:	emm
Depth :	---	Test Id:	401998		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	0.4	99.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0287	98		
---	0.0179	94		
---	0.0109	82		
---	0.0080	73		
---	0.0058	63		
---	0.0042	53		
---	0.0031	43		
---	0.0014	28		

Coefficients

D_{85} = 0.0126 mm
D_{30} = 0.0015 mm
D_{60} = 0.0053 mm
D_{15} = N/A
D_{50} = 0.0038 mm
D_{10} = N/A
C_u = N/A
C_c = N/A

Classification

ASTM Lean clay (CL)

AASHTO Clayey Soils (A-6 (16))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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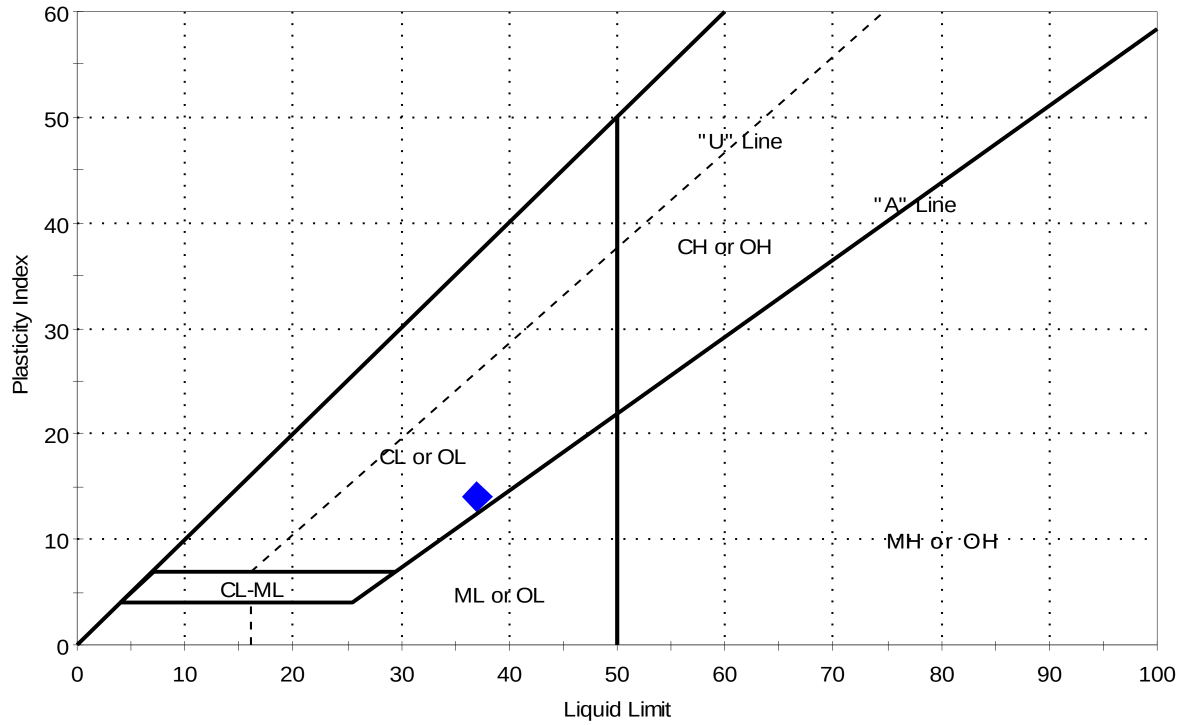
Client:	Nobis Engineering, Inc.		
Project:	N. Parish Rd Bridge		
Location:	Turner, ME	Project No:	GTX-305852
Boring ID: BB-TNR-105	Sample Type: jar	Tested By:	cam
Sample ID: 2D	Test Date: 01/16/17	Checked By:	emm
Depth : ---	Test Id: 402007		
Test Comment:	---		
Visual Description:	Moist, olive brown clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-TNR-10	---	30	37	23	14	0.5	Lean clay (CL)

Sample Prepared using the WET method

0% Retained on #40 Sieve

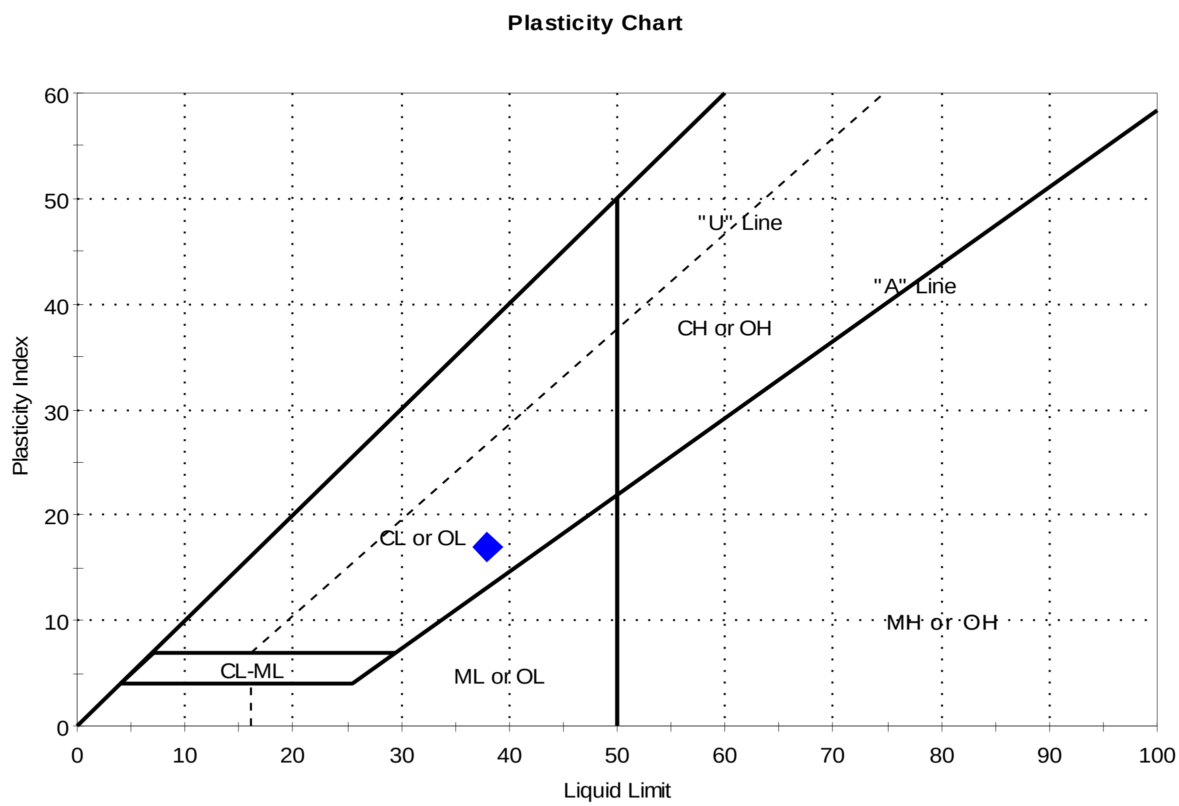
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Nobis Engineering, Inc.				
Project:	N. Parish Rd Bridge				
Location:	Turner, ME			Project No:	GTX-305852
Boring ID:	BB-TNR-106	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	01/16/17	Checked By:	emm
Depth :	---	Test Id:	402008		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-TNR-10	---	27	38	21	17	0.4	Lean clay (CL)

Sample Prepared using the WET method

0% Retained on #40 Sieve

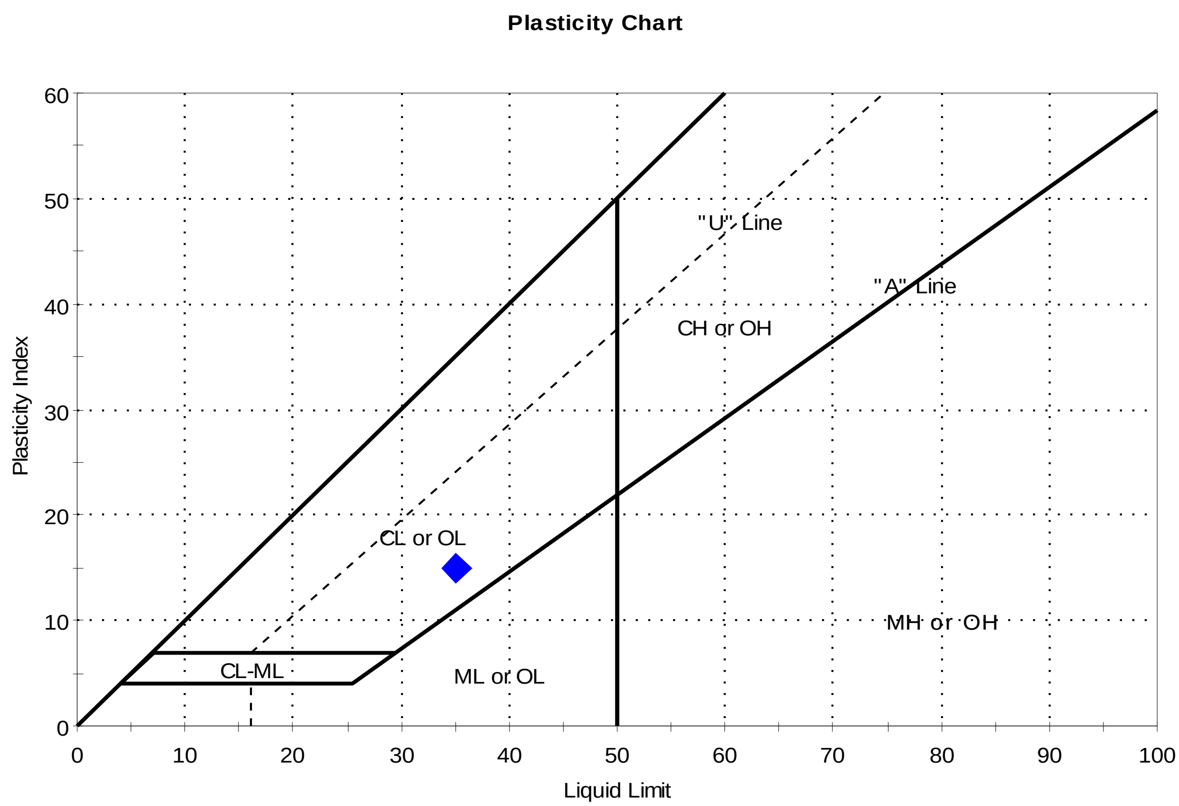
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Nobis Engineering, Inc.				
Project:	N. Parish Rd Bridge				
Location:	Turner, ME			Project No:	GTX-305852
Boring ID:	BB-TNR-107	Sample Type:	jar	Tested By:	cam
Sample ID:	4D	Test Date:	01/13/17	Checked By:	emm
Depth :	---	Test Id:	402009		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-TNR-10	---	31	35	20	15	0.8	Lean clay (CL)

Sample Prepared using the WET method

0% Retained on #40 Sieve

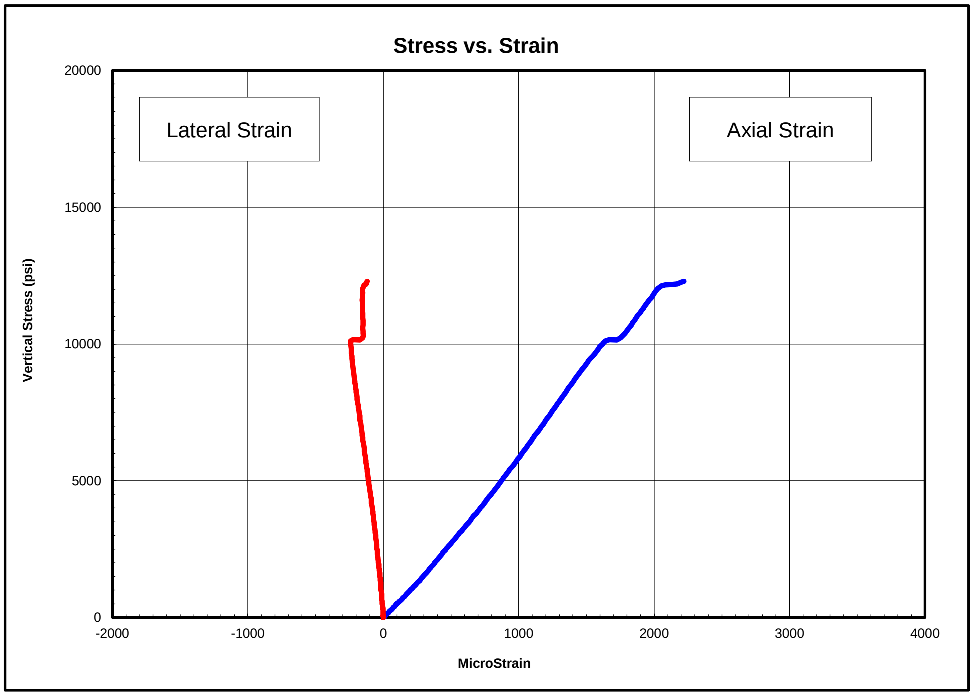
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-103
Sample ID:	R2
Depth, ft:	29.0-30.0
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: 12,286 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1200-4500	5,930,000	0.14
4500-7800	6,750,000	0.19
7800-11100	5,060,000	---

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:	Nobis Engineering, Inc.	Test Date:	1/11/2017
Project Name:	N. Parish Rd Bridge	Tested By:	daa/rlc
Project Location:	Turner, ME	Checked By:	jsc
GTX #:	305852		
Boring ID:	BB-TNR-103		
Sample ID:	R2		
Depth:	29.0-30.0 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:	4.47	4.47	4.47		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	591.94				
Bulk Density, lb/ft^3	162	**Minimum Diameter Tolerence Met?**		YES	
Length to Diameter Ratio:	2.2	**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	-0.00060	-0.00060	-0.00060	-0.00060	-0.00060	-0.00040	-0.00020	0.00000	0.00000	0.00000	-0.00060	-0.00070	-0.00070	-0.00070	-0.00050
Diameter 2, in (rotated 90°)	-0.00040	-0.00080	-0.00080	-0.00080	-0.00070	-0.00060	-0.00050	0.00000	-0.00010	-0.00060	-0.00060	-0.00060	-0.00050	-0.00050	-0.00030

Difference between max and min readings, in:

0° = 0.00070 90° = 0.00080

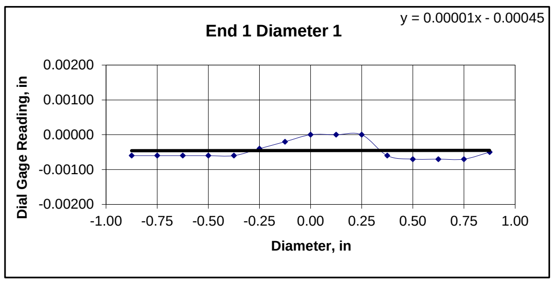
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00040	0.00040	0.00040	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	0.00020	0.00040	0.00050	0.00050	0.00050	0.00040
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	0.00020	0.00030	0.00030	0.00030	0.00020	0.00000	0.00000	0.00000	0.00050	0.00050	0.00050	0.00030	0.00010

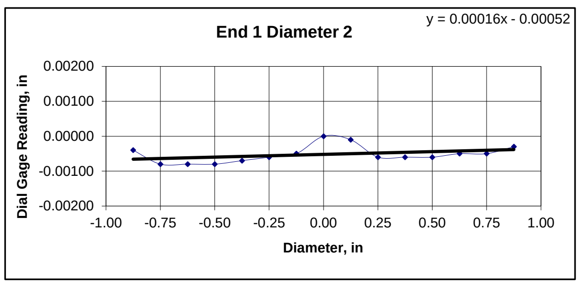
Difference between max and min readings, in:

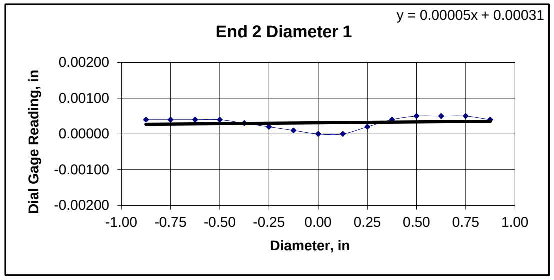
0° = 0.0005 90° = 0.0007

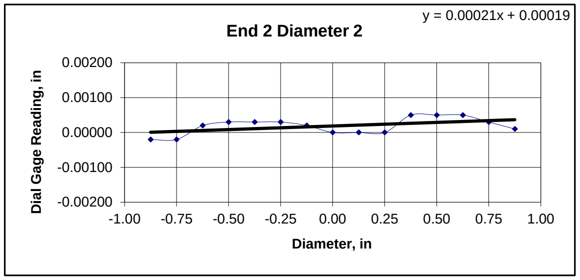
Maximum difference must be < 0.0020 in. Difference = ± 0.00040

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
- Slope of Best Fit Line 0.00001
- Angle of Best Fit Line: 0.00057

End 2:
- Slope of Best Fit Line 0.00005
- Angle of Best Fit Line: 0.00286

Maximum Angular Difference: 0.00229

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:
- Slope of Best Fit Line 0.00016
- Angle of Best Fit Line: 0.00917

End 2:
- Slope of Best Fit Line 0.00021
- Angle of Best Fit Line: 0.01203

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES

Spherically Seated

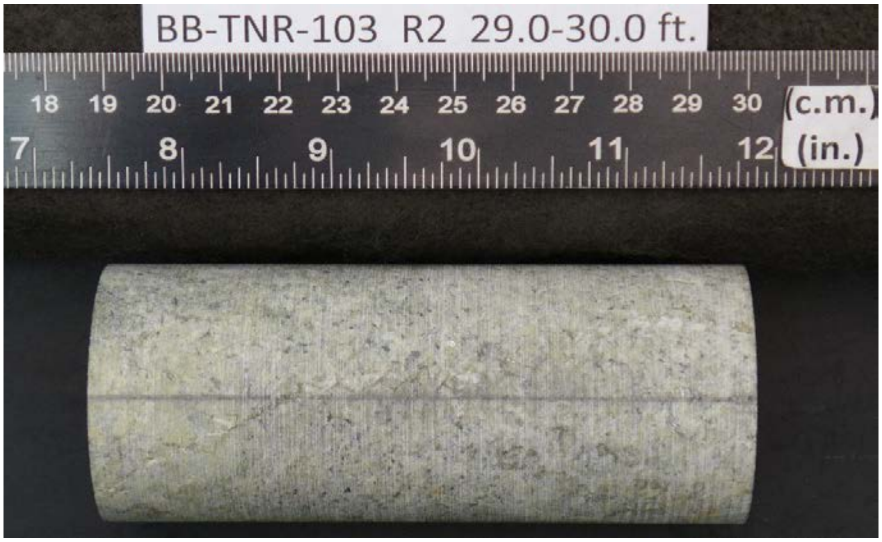
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00070	1.990	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES
END 2					
Diameter 1, in	0.00050	1.990	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00070	1.990	0.00035	0.020	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-103
Sample ID:	R2
Depth, ft:	29.0-30.0

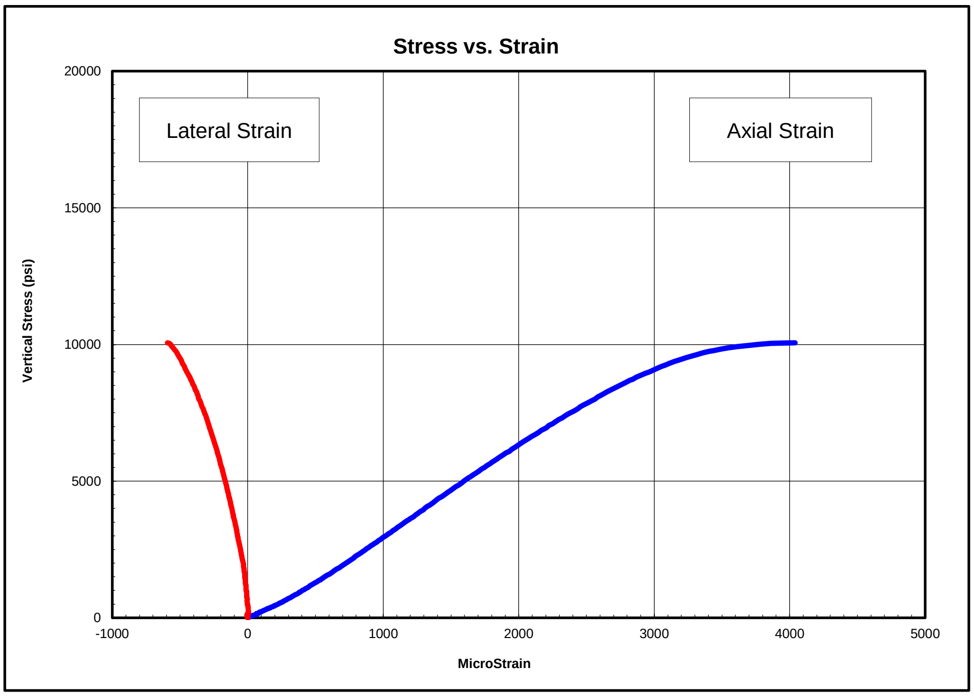


After cutting and grinding

After break

Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-104
Sample ID:	R1
Depth, ft:	37.5-38.3
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: 10,060 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1000-3700	3,330,000	0.12
3700-6400	3,380,000	0.18
6400-9100	2,780,000	0.22

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:	Nobis Engineering, Inc.	Test Date:	1/11/2017
Project Name:	N. Parish Rd Bridge	Tested By:	daa/rlc
Project Location:	Turner, ME	Checked By:	jsc
GTX #:	305852		
Boring ID:	BB-TNR-104		
Sample ID:	R1		
Depth:	37.5-38.3 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:	4.48	4.48	4.48		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	599.71				
Bulk Density, lb/ft^3	164			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3			**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	-0.00050	-0.00050	-0.00050	-0.00050	-0.00050	-0.00030	-0.00010	0.00000	0.00010	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00020
Diameter 2, in (rotated 90°)	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00010	0.00000	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00020	-0.00020

Difference between max and min readings, in:

0° = 0.00060 90° = 0.00030

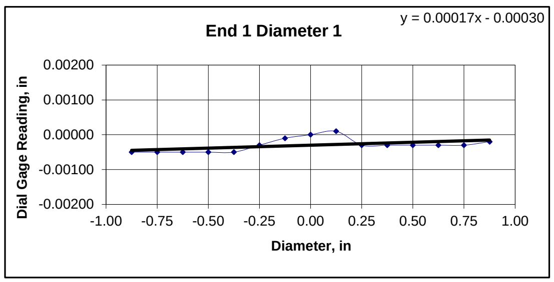
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	0.00000	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00010	0.00020	0.00020	0.00020	0.00020
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000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

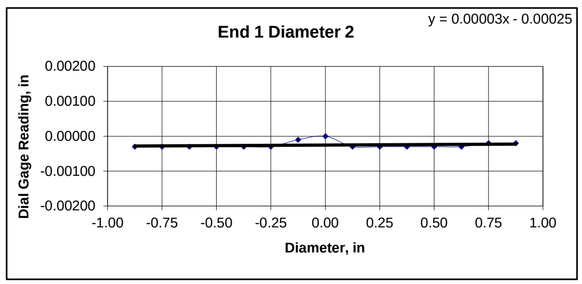
Difference between max and min readings, in:

0° = 0.0004 90° = 0.0002

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00030

Flatness Tolerance Met? YES





DIAMETER 1

End 1:

Slope of Best Fit Line	0.00017
Angle of Best Fit Line:	0.00974

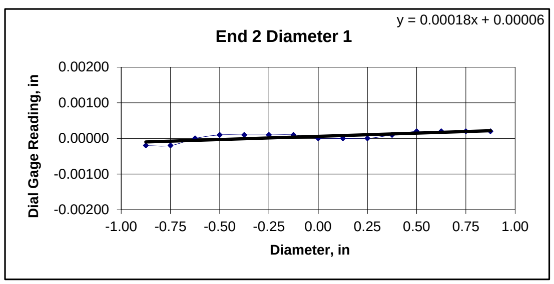
End 2:

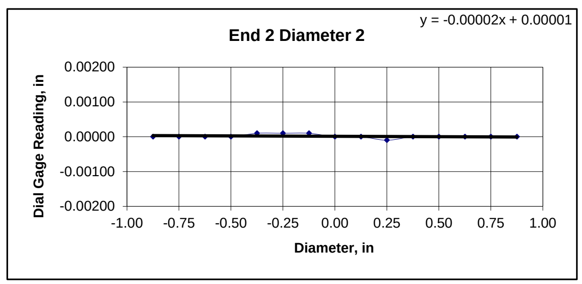
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01031

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? YES

Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00003
Angle of Best Fit Line:	0.00172

End 2:

Slope of Best Fit Line	-0.00002
Angle of Best Fit Line:	-0.00115

Maximum Angular Difference: 0.00286

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	0.00060	1.990	0.00030	0.017	YES
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES

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

Perpendicularity Tolerance Met? YES

Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-104
Sample ID:	R1
Depth, ft:	37.5-38.3

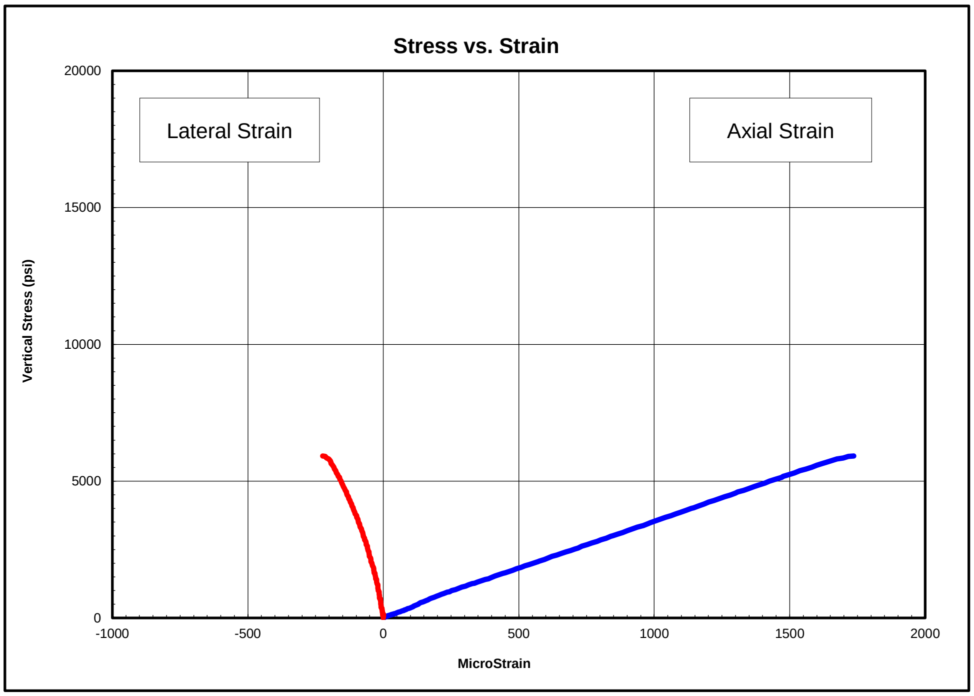
After cutting and grinding

After break

GeoTesting EXPRESS

Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-104
Sample ID:	R2
Depth, ft:	43.7-44.4
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 5,920 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
600-2200	3,400,000	0.08
2200-3700	3,430,000	0.12
3700-5300	3,450,000	0.16

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:	Nobis Engineering, Inc.	Test Date:	1/12/2017
Project Name:	N. Parish Rd Bridge	Tested By:	daa/rlc
Project Location:	Turner, ME	Checked By:	jsc
GTX #:	305852		
Boring ID:	BB-TNR-104		
Sample ID:	R2		
Depth:	43.7-44.4 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:	4.57	4.57	4.57		
Specimen Diameter, in:	1.98	1.99	1.99		
Specimen Mass, g:	633.43				
Bulk Density, lb/ft^3	170	**Minimum Diameter Tolerence Met?**		YES	
Length to Diameter Ratio:	2.3	**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	-0.00040	-0.00040	-0.00040	-0.00040	-0.00050	-0.00040	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	0.00000	0.00000
Diameter 2, in (rotated 90^o)	0.00010	-0.00020	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	0.00000	0.00000	0.00000	0.00010	0.00010	-0.00020	-0.00020	-0.00020

Difference between max and min readings, in:

0° = 0.00050 90° = 0.00040

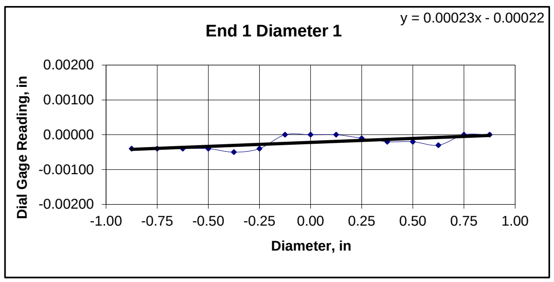
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	-0.00020	-0.00020	-0.00020	0.00010	0.00010
Diameter 2, in (rotated 90^o)	-0.00030	-0.00030	-0.00030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

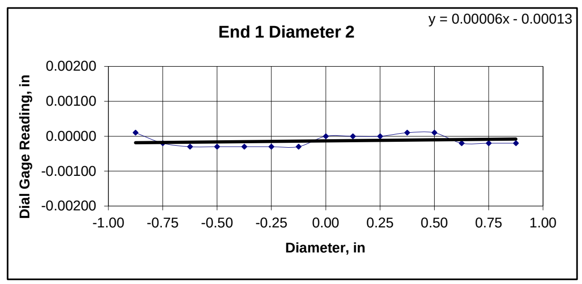
Difference between max and min readings, in:

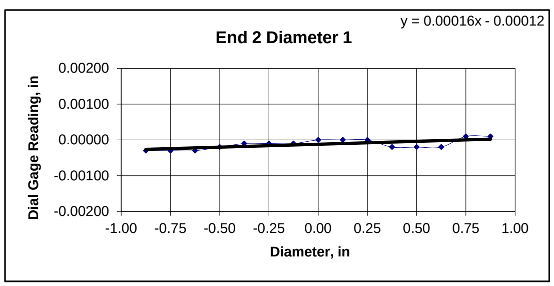
0° = 0.0004 90° = 0.0003

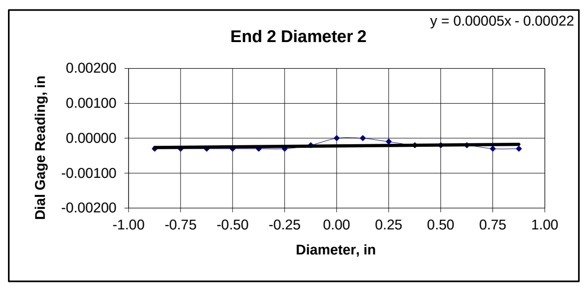
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00025

Flatness Tolerance Met? YES









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00023
	Angle of Best Fit Line:	0.01318
End 2:		
	Slope of Best Fit Line	0.00016
	Angle of Best Fit Line:	0.00917
Maximum Angular Difference:		0.00401

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00006
	Angle of Best Fit Line:	0.00344
End 2:		
	Slope of Best Fit Line	0.00005
	Angle of Best Fit Line:	0.00286
Maximum Angular Difference:		0.00057

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	1.985	0.00025	0.014	YES
Diameter 2, in (rotated 90^o)	0.00040	1.985	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00040	1.985	0.00020	0.012	YES
Diameter 2, in (rotated 90^o)	0.00030	1.985	0.00015	0.009	YES

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

Perpendicularity Tolerance Met? YES

GeoTesting EXPRESS

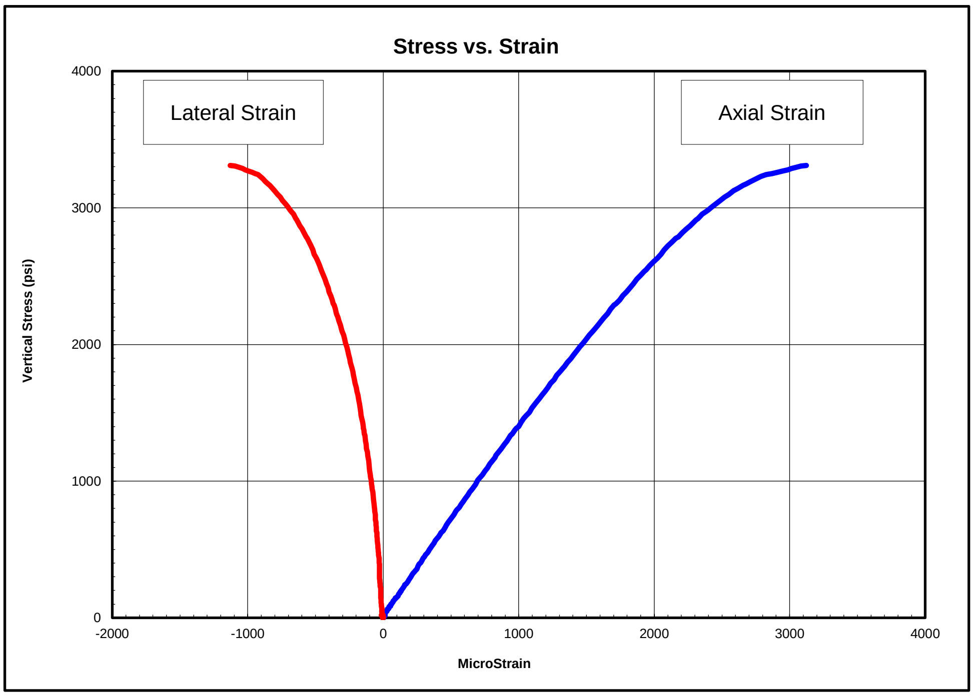
Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-104
Sample ID:	R2
Depth, ft:	43.7-44.4

After cutting and grinding

After break

Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-105
Sample ID:	R1
Depth, ft:	38.0-38.8
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 3,314 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
300-1200	1,410,000	0.14
1200-2100	1,270,000	0.26
2100-3000	1,050,000	0.45

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:	Nobis Engineering, Inc.	Test Date:	1/12/2017
Project Name:	N. Parish Rd Bridge	Tested By:	daa/rlc
Project Location:	Turner, ME	Checked By:	jsc
GTX #:	305852		
Boring ID:	BB-TNR-105		
Sample ID:	R1		
Depth:	38.0-38.8 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:	4.63	4.63	4.63		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	618.91				
Bulk Density, lb/ft^3	163			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3			**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	0.00030	0.00030	0.00030	0.00040	0.00000	0.00000	0.00000	0.00000	0.00000	0.00020	0.00020	0.00020	0.00030	0.00040	0.00040
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00010	0.00010	0.00020	0.00000	0.00000	0.00000	0.00000	0.00010	0.00020	-0.00010	-0.00010	-0.00010

Difference between max and min readings, in:

0° = 0.00040 90° = 0.00040

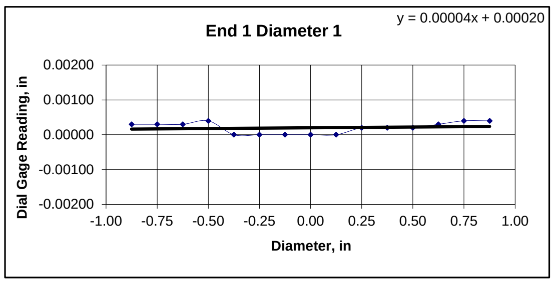
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010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

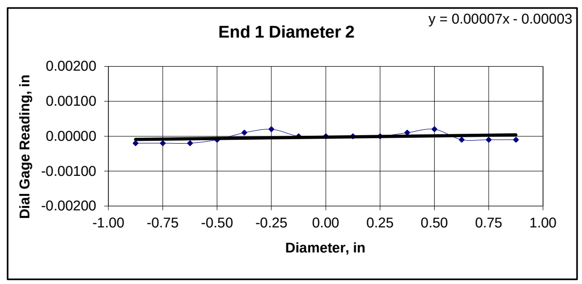
Difference between max and min readings, in:

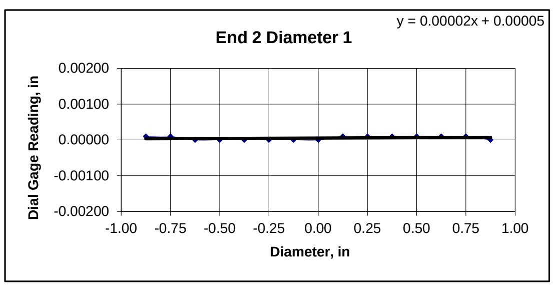
0° = 0.0001 90° = 0.0001

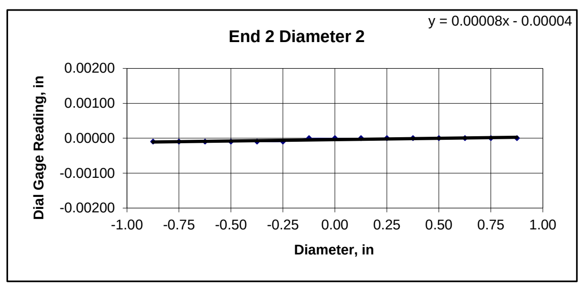
Maximum difference must be < 0.0020 in. Difference = ± 0.00020

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00004
Angle of Best Fit Line:	0.00229

End 2:

Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115

Maximum Angular Difference: 0.00115

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00007
Angle of Best Fit Line:	0.00401

End 2:

Slope of Best Fit Line	0.00008
Angle of Best Fit Line:	0.00458

Maximum Angular Difference: 0.00057

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	0.00040	1.990	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00040	1.990	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00010	1.990	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Nobis Engineering, Inc.
Project Name:	N. Parish Rd Bridge
Project Location:	Turner, ME
GTX #:	305852
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-TNR-105
Sample ID:	R1
Depth, ft:	38.0-38.8

After cutting and grinding

After break

APPENDIX E - Calculations

APPENDIX E.1 – Loads Provided by HNTB

Brien Waterman

From: Joshua Olund <jolund@HNTB.com>
Sent: Thursday, August 10, 2017 9:08 AM
To: Kurt Jelinek; Brien Waterman
Cc: Laura Krusinski; Joe Stilwell; Timothy Cote
Subject: WIN 18749 - N Parish Bridge Preliminary Pile Demands

Kurt and Brien –

As discussed on our call Monday afternoon, I'm sending preliminary pile demands for your use in running WEAP analyses and to begin developing pile forces in L-Pile. Keep in mind these are preliminary demands since we are still developing the superstructure and substructure designs, but they should be representative of where we're heading.

- Pile Factored Axial Demand = 450 kip (assuming 5 piles)
- Pile Head Movement = 0.96 inch (assuming 100°F erection temperature)

If you have questions on these preliminary demands, please let me know. As our design progresses and items become more substantially complete, I'll be sure to update the axial demands.

Thanks.

Josh Olund, P.E., PhD
Senior Bridge Engineer
Tel (207) 228-0893 **Cell** (207) 712-7028

HNTB CORPORATION
340 County Road Suite 6C, Westbrook, ME 04092 | www.hntb.com

APPENDIX E.2 – Lateral Pile Evaluation

Project No.: 92270.02
Project Title: North Parish Bridge - Turner, Maine
Calculated by: BTW **Date:** 11.2.2017
Checked by: RAC **Date:** 11.6.2017

Preliminary Lateral Pile Evaluation

Objective: Perform lateral pile analysis for proposed integral abutments.

Approach: 1. Use Lpile 2016 to perform preliminary lateral pile evaluation.

References:
1. Borings BB-TNR-103 and 105.
2. AASHTO LRFD Bridge Design Specifications, 7th Edition.
3. Computer Program L-Pile User's Manual, by Ensoft, Inc.
4. Computer Program Group User's Manual by Ensoft, Inc.
6. MaineDOT Bridge Design Manual, 2003, revised through 2014
7. Preliminary Cross Sections, MaineDOT, June 2017
8. Email from HNTB, sent on August 10, 2017.
9. 98% Plans for WIN 18749.00, dated October 17, 2017.

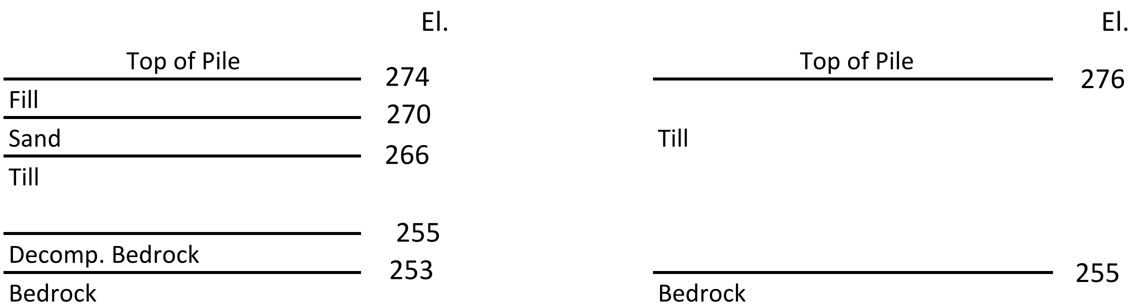
Given:
1. Pile loads and thermal deflection (see Reference No. 8).
2. Proposed abutment is integral with HP 14x102 piles.
3. Based on Reference No. 9, the top of piles at Abutment 1 (North) are about El. 274 and top of piles at Abutment 2 (South) are at about El. 276.

Assumptions: 1. Piles terminate on or within bedrock corresponding to approximate 21-foot pile lengths (not including pile cap embedment).

Solution:

General Soil Profile (N.T.S)

South Abutment (BB-TNR-103) **North Abutment (BB-TNR-105)**



Note: Based on boring BB-TNR-108, top of bedrock is at approximate El. 255

Soil Properties and Profile

Depths listed below were taken from bottom of pile caps. Soil unit weight (γ_{moist}) and soil friction angle (ϕ) were estimated based on SPT-N Values. The soil modulus parameter (k) was determined using Table 3.7 from Reference No. 4. Uniaxial Compressive Strength was taken from lab data.

Nobis
Engineering a Sustainable Future

Project No.: 92270.02
Project Title: North Parish Bridge - Turner, Maine
Calculated by: BTW **Date:** 11.2.2017
Checked by: RAC **Date:** 11.6.2017

South Abutment (Boring BB-TNR-103)

Soil Layer	Elevation (ft)	Depth (ft)	γ_{moist} (pcf)	ϕ (°)	C (psf)	k (pci)	U.C.S. (ksi)	ε_{50} or k_rm (dims)
Fill	274 to 270	0 to 4	125	32	-	90	-	-
Sand	270 to 266	4 to 8	125	32	-	60	-	-
Till	266 to 255	8 to 19	135	-	4000	-	-	0.004
Decomposed Bedrock	255 to 253	19 to 21	140	37	-	125	-	-
Bedrock	<253	>21	150	-	-	-	4.6	-

Notes: 1. Assume groundwater at a depth of 4 feet below top of pile (i.e. El. 270).

North Abutment (Boring BB-TNR-105 and BB-TNR-108)

Soil Layer	Elevation (ft)	Depth (ft)	γ_{moist} (pcf)	ϕ (°)	C (psf)	k (pci)	U.C.S. (ksi)	ε_{50} or k_rm (dims)
Till	276 to 255	0 to 21	135	-	4000	-	-	0.004
Bedrock	<255	>21	150	-	-	-	4.6	-

Notes: 1. Assume groundwater at a depth of 6 feet below top of pile (i.e. El. 270).

Consistency of Clay	ϵ_{50}
Soft	0.020
Medium	0.010
Stiff	0.005

Table 3.5 Values of ε_{50} for Clays

Average Undrained Shear Strength (kPa)	ϵ_{50}
50-100	0.007
100-200	0.005
300-400	0.004

Table 3.6 Values of ε_{50} for Stiff Clays

Clay Type	Avg. Undrained Shear Strength (c_u)	Static	Cyclic
Soft	1.74 – 3.47 psi 250 – 500 psf 12 – 24 kPa	30 pci 8,140 kPa/m	
Medium	3.47 – 6.94 psi 500 – 1000 psf 24 – 48 kPa	100 pci 27,150 kPa/m	
Stiff	6.94 – 13.9 psi 1000 – 2000 psf 48 – 96 kPa	500 pci 136,000 kPa/m	200 pci 54,300 kPa/m
Very Stiff	13.9 – 27.8 psi 2000 – 4000 psf 96 – 192 kPa	1000 pci 271,000 kPa/m	400 pci 108,500 kPa/m
Hard	27.8 – 55.6 psi 4000 – 8000 psf 192 – 383 kPa	2000 pci 543,000 kPa/m	800 pci 217,000 kPa/m

Table 3.8 Soil Modulus Parameter k for Clays

Relative Density	Loose	Medium	Dense
Submerged Sand	20 lb/in^3 5,430 kN/m^3	60 lb/in^3 16,300 kN/m^3	125 lb/in^3 33,900 kN/m^3
Sand Above WT	25 lb/in^3 6,790 kN/m^3	90 lb/in^3 24,430 kN/m^3	225 lb/in^3 61,000 kN/m^3

Table 3.7 Soil-Modulus Parameter k for Sands

Nobis
Engineering a Sustainable Future

Project No.: 92270.02
Project Title: North Parish Bridge - Turner, Maine
Calculated by: BTW **Date:** 11.2.2017
Checked by: RAC **Date:** 11.6.2017

Pile Properties

Pile:

HP 14x102

Pile Type:
Yield Stress: 50 ksi
Elastic modulus: 29000 ksi

Based on Reference No. 9, pile spacing is greater than 5B and piles are oriented along weak axis (see illustration below).

Loads and Deflection

Per HNTB's emails sent August 10 and 23, 2017:

Pile Factored Axial Demand = 450 kip (assuming 5 piles)
Pile Head Movement = 0.96 inch (assuming 100^{o}F erection temperature)
Pile Head Moment = 2,300 kip-in

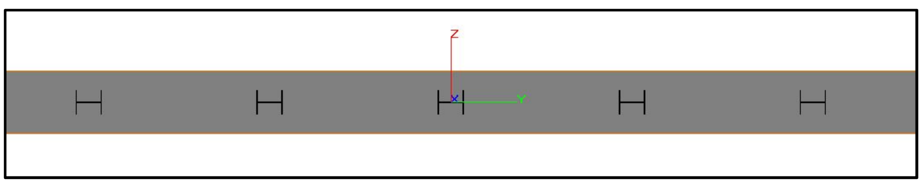


Illustration of pile cap with piles

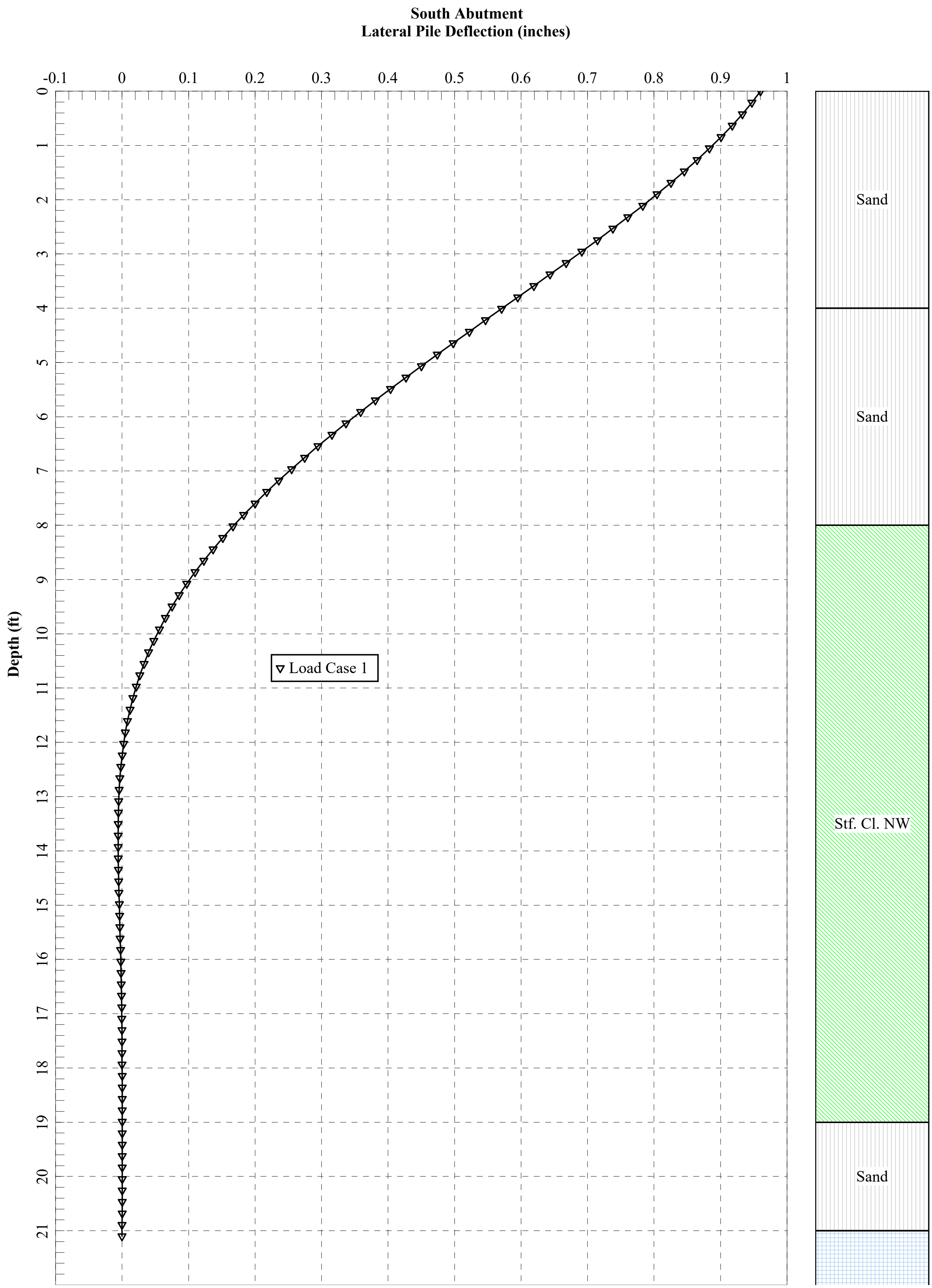
Load Case	Pile-Head Loading Condition	Condition (1) for Loading Type	Condition (2) for Loading Type	Axial Load (p-delta) (lbs)
1	(1) Displ. [inch or meter] and (2) Moment [in-lb or kN-m]	0.96	-2300000	450000

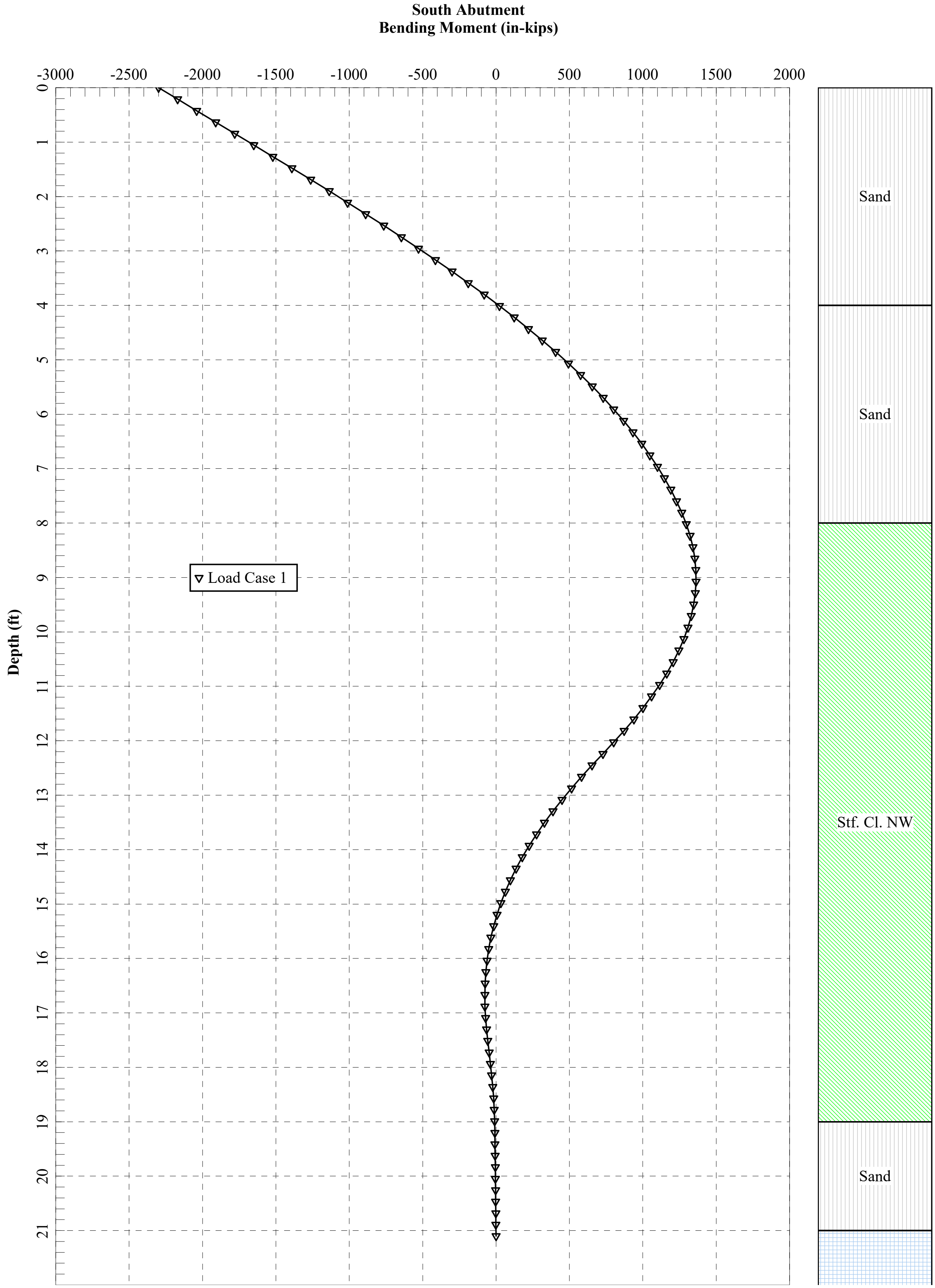
Pile Loading

Results

Please see attached graphs for lateral deflection, moment, and shear.

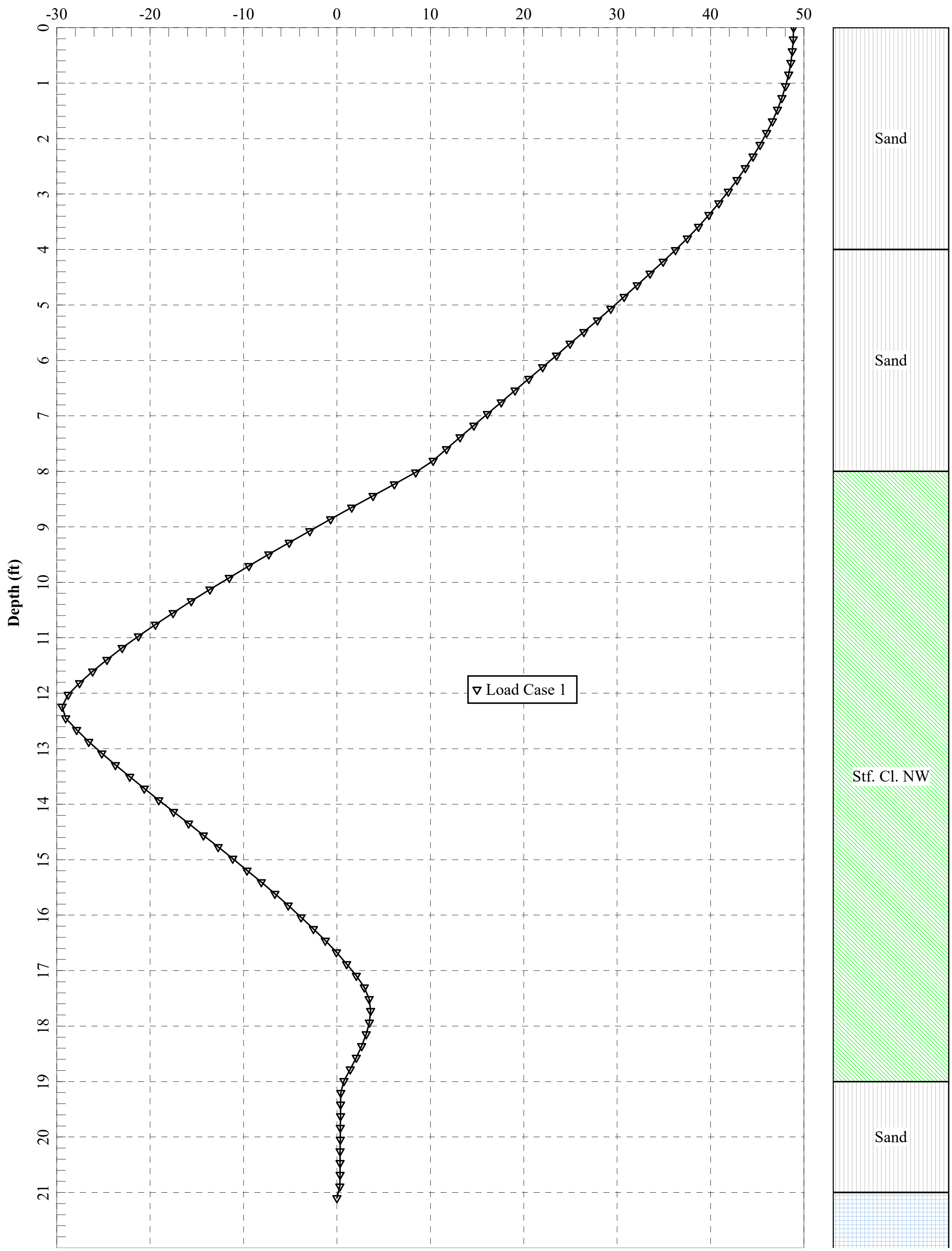
Abutment No. 1 - South Abutment


South Abutment
Lateral Pile Deflection (inches)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
Load Case 1
Sand
Sand
Stf. Cl. NW
Sand



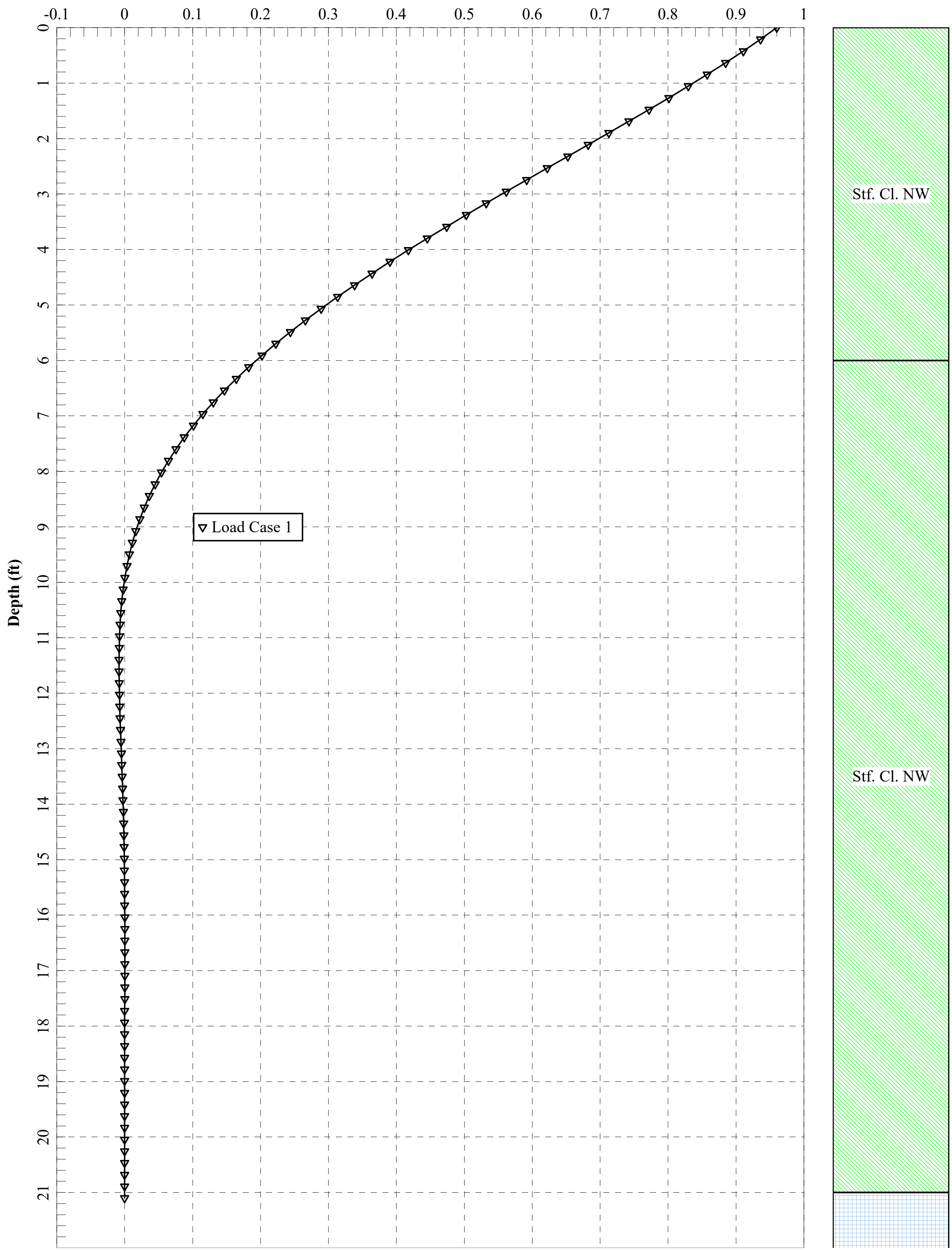
South Abutment
Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
Load Case 1
Sand
Sand
Stf. Cl. NW
Sand

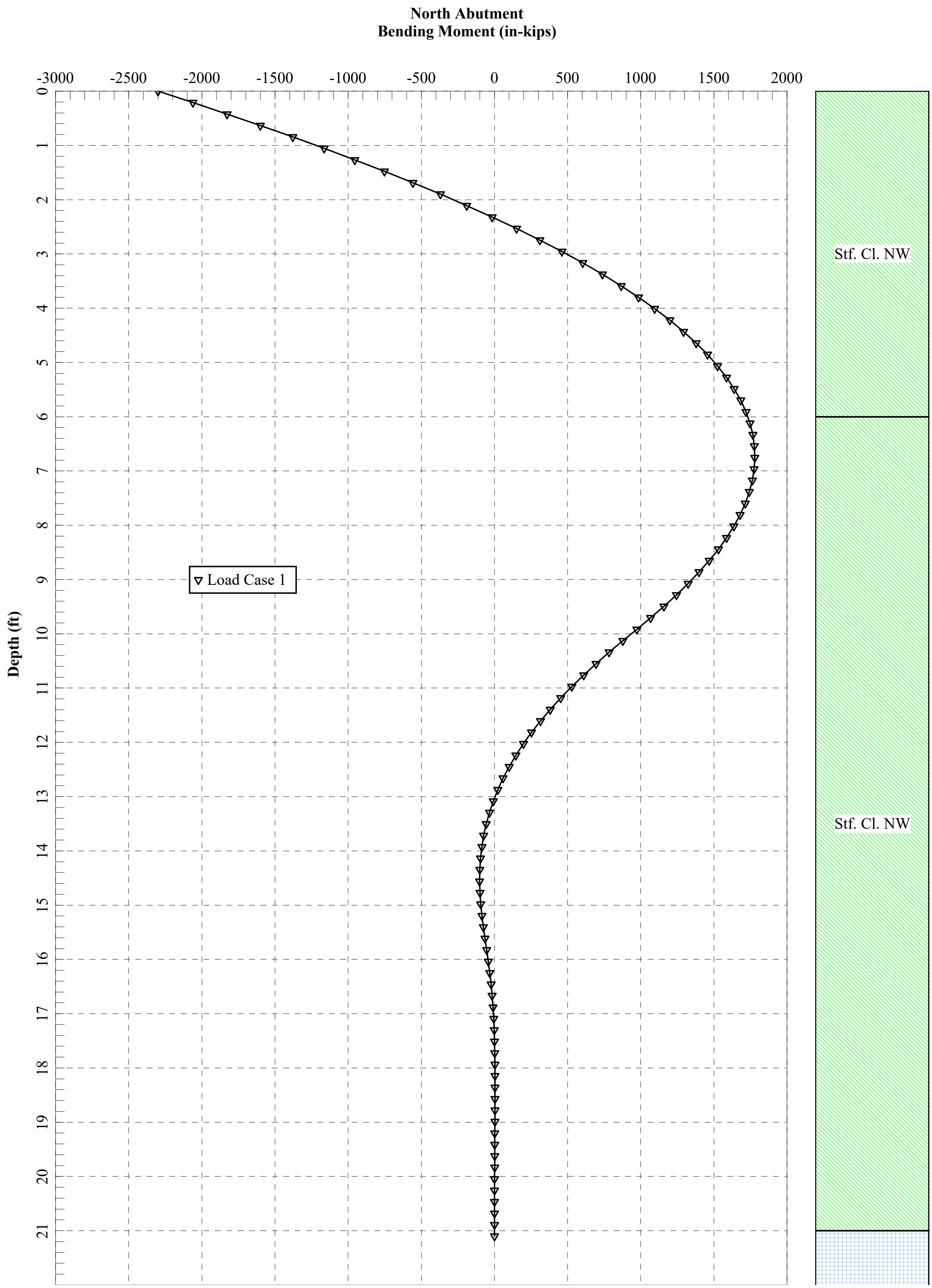
South Abutment
Shear Force (kips)



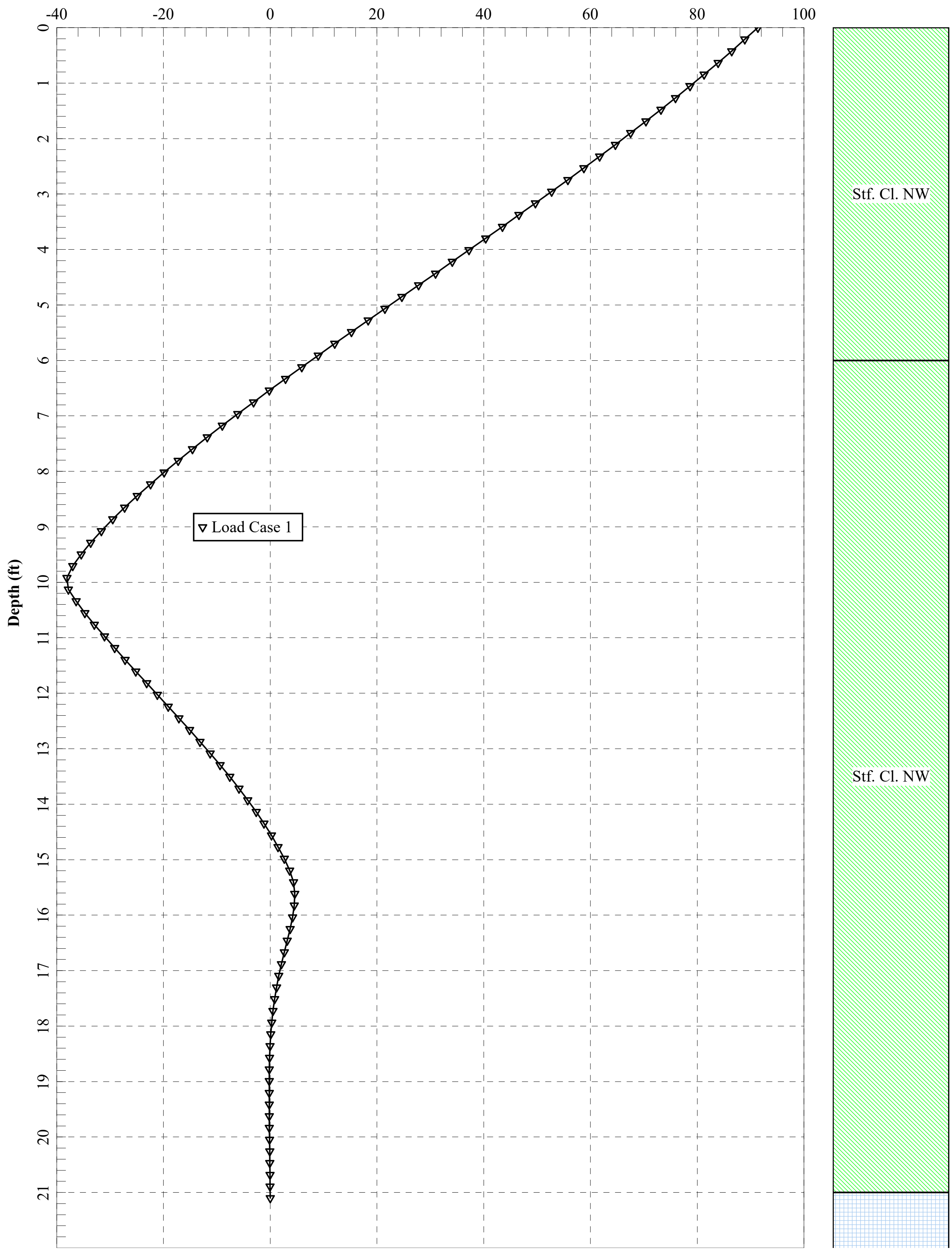
Abutment No. 2 - North Abutment

North Abutment
Lateral Pile Deflection (inches)




North Abutment
Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
Load Case 1
Stf. Cl. NW
Stf. Cl. NW

North Abutment
Shear Force (kips)



APPENDIX E.3 – Lateral Earth Pressures

Earth Pressure Coefficients

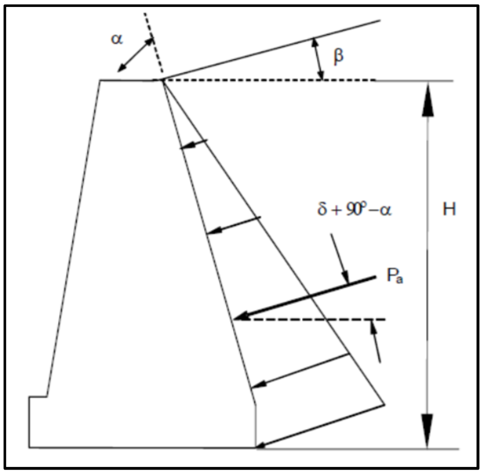
Objective: Calculate passive earth pressure coefficients for proposed integral abutments.

Approach: Use Coulomb's method and Rankin's method to calculate passive earth pressure coefficients in accordance with MaineDOT Bridge Design Guide.

References:
1. MaineDOT Bridge Design Guide
2. Foundation Analysis and Design, Bowles, 5th Edition

Assumptions: 1. Retained soil is granular with an effective friction angle of 32°

Coulomb Passive Earth Pressure (from MaineDOT Bridge Design Guide)



3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

Values of the coefficient of passive lateral earth pressure, K_p, may be taken from Figures 3.11.5.4-1 and 2 in AASHTO LRFD or using Coulomb theory, as shown below:

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

α = angle (degrees) of back of wall to the horizontal

ϕ = angle of internal soil friction (degrees), taken from Table 3-3

δ = friction angle between fill and wall (degrees), taken from Table 3-3 for soil against concrete

β = angle of backfill to horizontal (degrees)

α	β	ϕ	δ
90.0	0	32	20

K_{p_coul} = 6.89

Passive Earth Pressure (from Bowles 5th Edition, Section 11-5, pg. 602)

$$K_{p\_rank} = \frac{\cos(\alpha) + [\cos(\alpha)^2 - \cos(\phi)^2]^{0.5}}{\cos(\alpha) - [\cos(\alpha)^2 - \cos(\phi)^2]^{0.5}}$$

Slope angle of backfill soil from horizontal: α = 0°

K_{p_rank} = 3.25

Soil angle of internal friction: ϕ = 32°

APPENDIX E.4 Global Slope Stability

North Parish Bridge (#2619) over Nezinscot River
Longitudinal Direction @ STA 8+50 To 12+00
Turner, Maine

Discipline: Geotechnical

Title: Global Stability Analysis for Proposed Bridge in Longitudinal Direction at Proposed Abutments

WIN #: 18749.00

Design Basis/Assumptions:

Purpose: Perform global stability analyses at proposed bridge abutments.

Assumptions:

1) Engineering properties for existing fill and the native soils encountered in borings BB-TNR-101 through BB-TNR-107 estimated based on standard penetration test (SPT) data and engineering judgment based on visual classification.
2) Proposed Fill: Unit Weight = 135 pcf, Angle of Internal Friction = 36°.
3) RipRap: Unit Weight = 145 pcf, Angle of Internal Friction = 40°.
4) Groundwater conditions the same or similar to those observed in the subsurface explorations by Nobis. High and low groundwater modeled at approximately EL. 279.36 feet and EL. 269.99, respectively, based on Reference 1.
5) Surcharge loading of 250 psf.

Approach:

Use the computer slope stability analysis software Slide 6.0, by RocScience, Inc., to perform global stability analyses based at the proposed bridge slopes located between @ STA 8+50 & 12+00, provided by HNTB (Reference 1). HP 14x104 piles, spaced 10 feet apart, were modeled. Based on Reference 3, the shear strength of the piles is 605 kips/pile.

References:

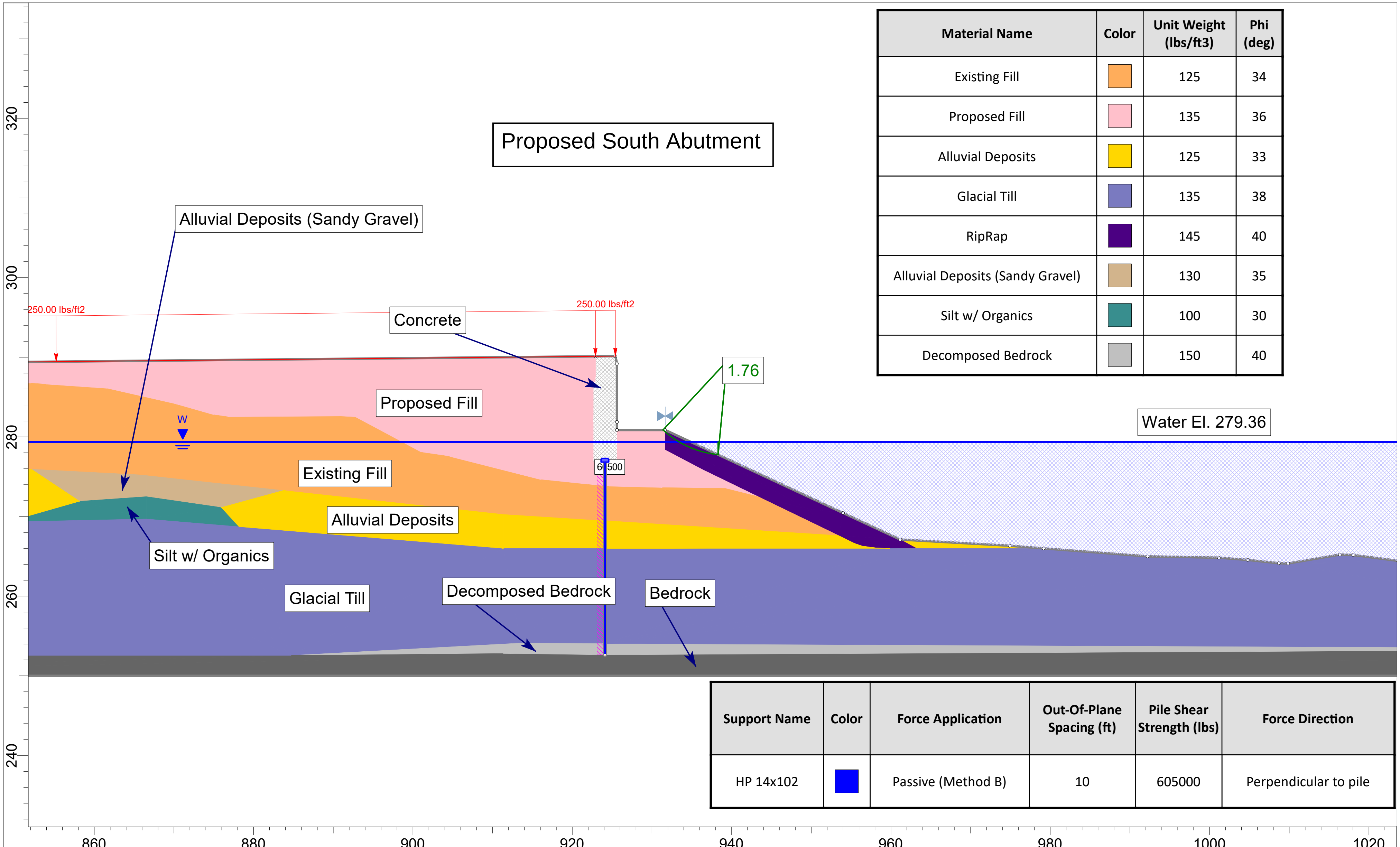
1) 98% Design drawings provided by HNTB, sent on October 16, 2017.
2) SPT data and field classifications by Nobis at borings performed nearest to proposed slopes (BB-TNR-101 through BB-TNR-107).
3) Pile shear strength of 605 kips/pile, provided via email by HNTB on October 20, 2017.

Remarks/Conclusions:

The minimum factor of safety against global slope failure is 1.53. Additionally, if the piles are ignored, then the global slope stability factor of safety is at least 1.3.

Originated By: Kamil Kocia, Staff Engineer   Date: 10/29/2017

Approved By: Brien Waterman   Date: 10/30/2017

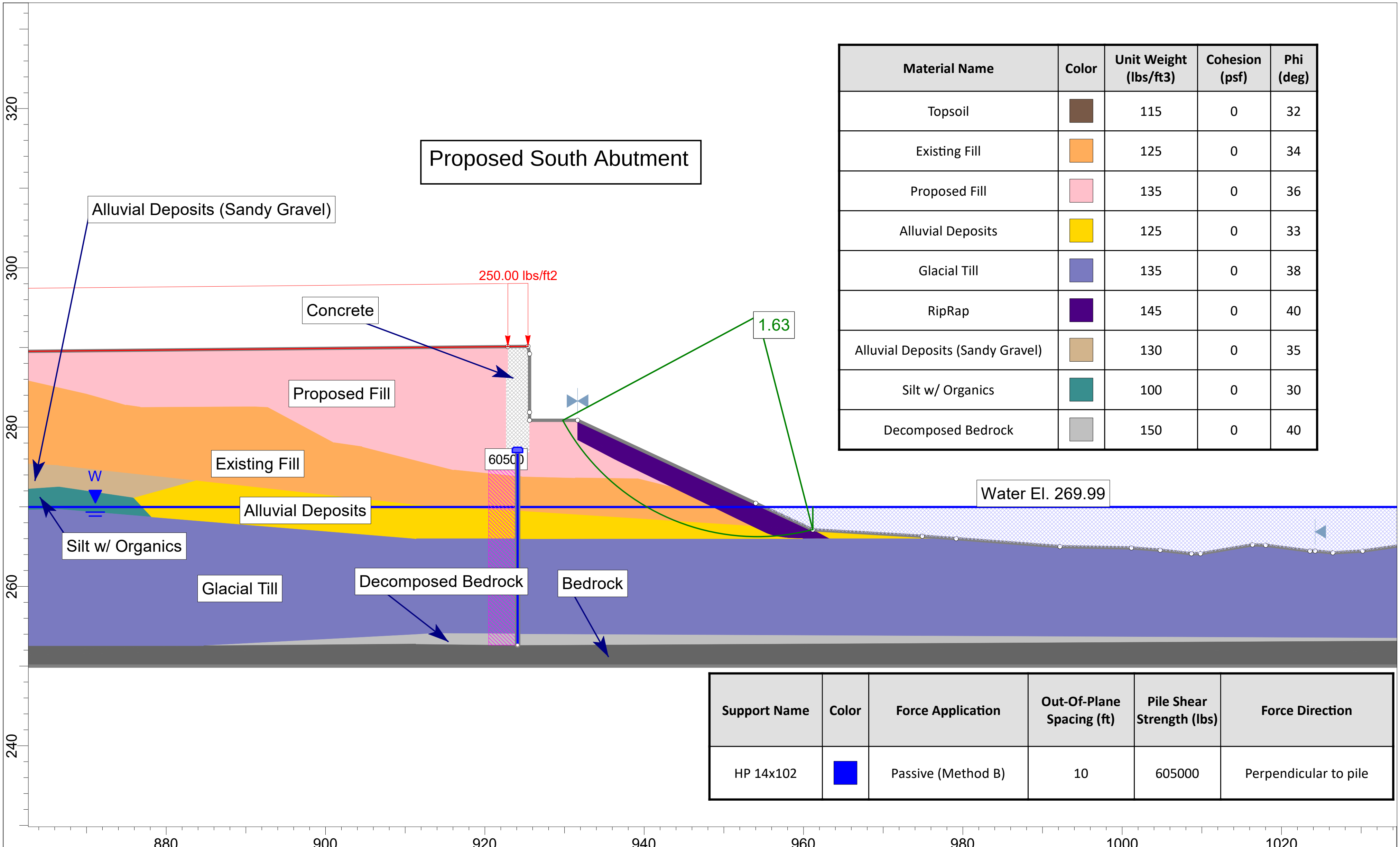


Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
Existing Fill		125	34
Proposed Fill		135	36
Alluvial Deposits		125	33
Glacial Till		135	38
RipRap		145	40
Alluvial Deposits (Sandy Gravel)		130	35
Silt w/ Organics		100	30
Decomposed Bedrock		150	40

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
HP 14x102		Passive (Method B)	10	605000	Perpendicular to pile

Project	North Parish Bridge (#2619) Over Nezinscot River
Analysis Description	Global Slope Stability Analysis Longitudinal High Groundwater
Drawn By	KAK Check: BTW
Scale	1:200
Company	Nobis Engineering
Date	10/29/2017, 11:36:12 AM
File Name	Longitudinal L to R High Groundwater.slim

SLIDEINTERPRET 6.039



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Alluvial Deposits		125	0	33
Glacial Till		135	0	38
RipRap		145	0	40
Alluvial Deposits (Sandy Gravel)		130	0	35
Silt w/ Organics		100	0	30
Decomposed Bedrock		150	0	40

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
HP 14x102		Passive (Method B)	10	605000	Perpendicular to pile

Project	North Parish Bridge (#2619) Over Nezinscot River
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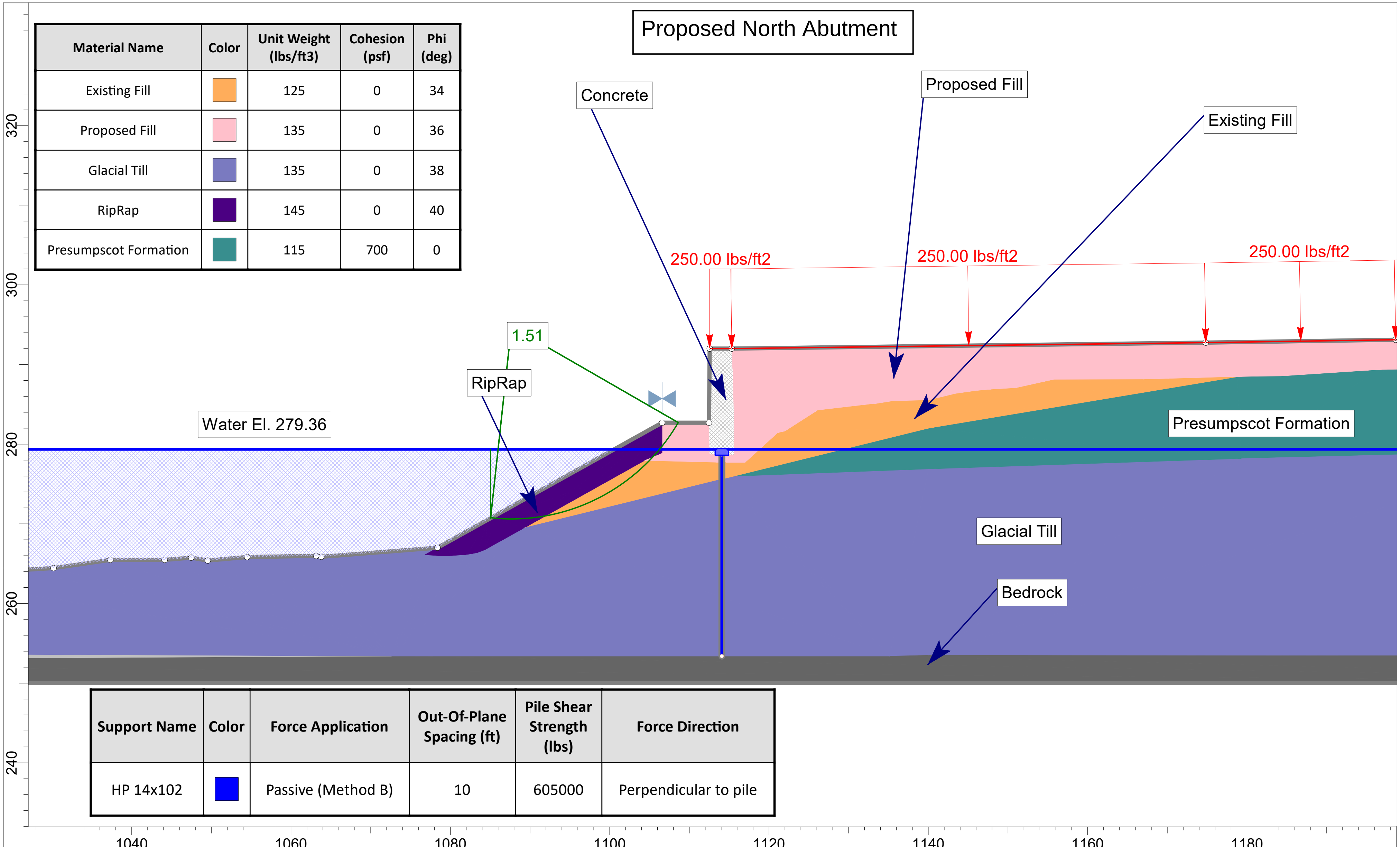


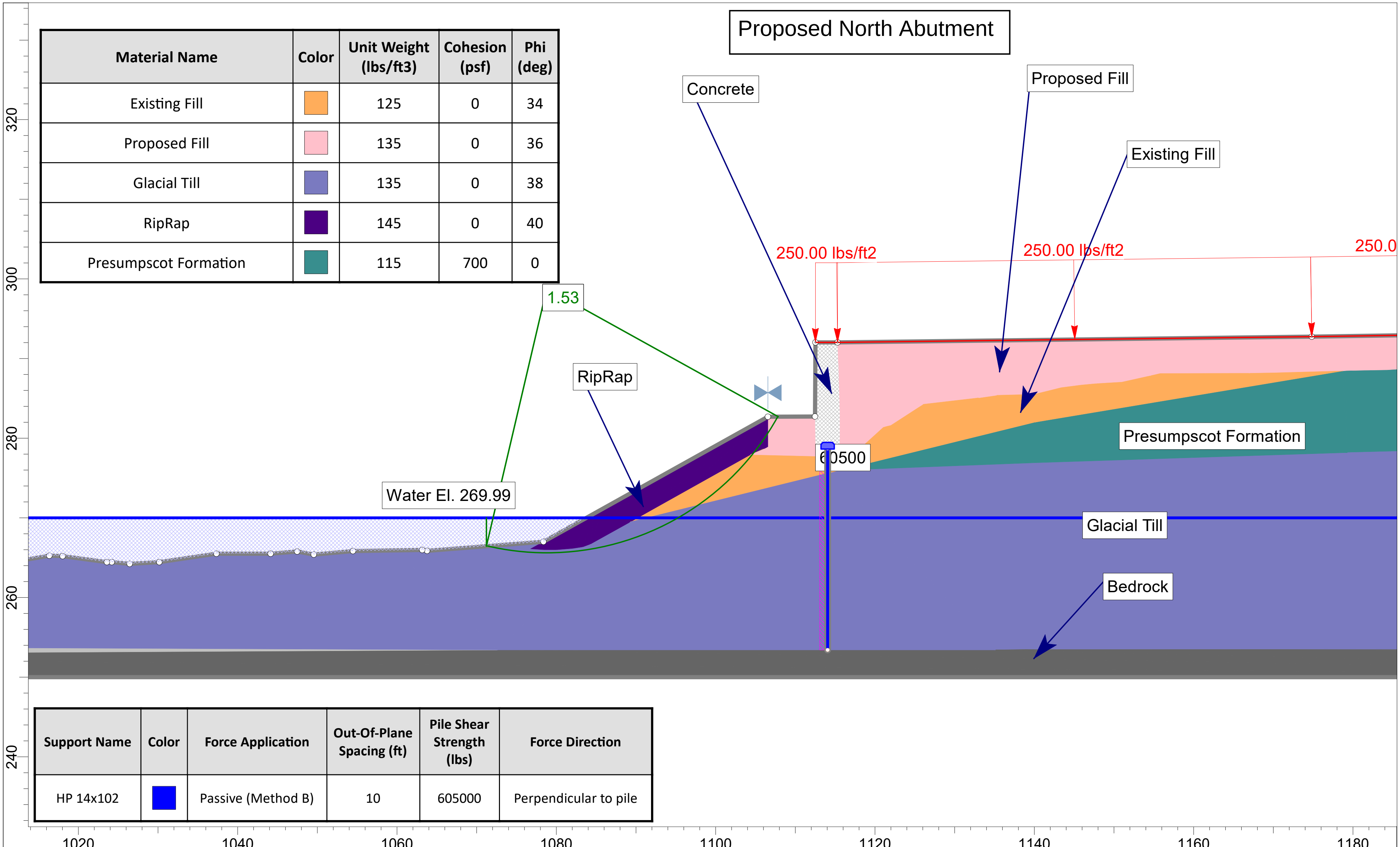
Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Existing Fill		125	0	34
Proposed Fill		135	0	36
Glacial Till		135	0	38
RipRap		145	0	40
Presumpscot Formation		115	700	0

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
HP 14x102		Passive (Method B)	10	605000	Perpendicular to pile

SLIDEINTERPRET 6.039

Project	North Parish Bridge (#2619) Over Nezinscot River
Analysis Description	Global Slope Stability Analysis Longitudinal High Groundwater
Drawn By	KAK Check: BTW
Scale	1:200
Company	Nobis Engineering
Date	10/29/2017, 11:36:12 AM
File Name	Longitudinal R to L High Groundwater.slim

Proposed North Abutment

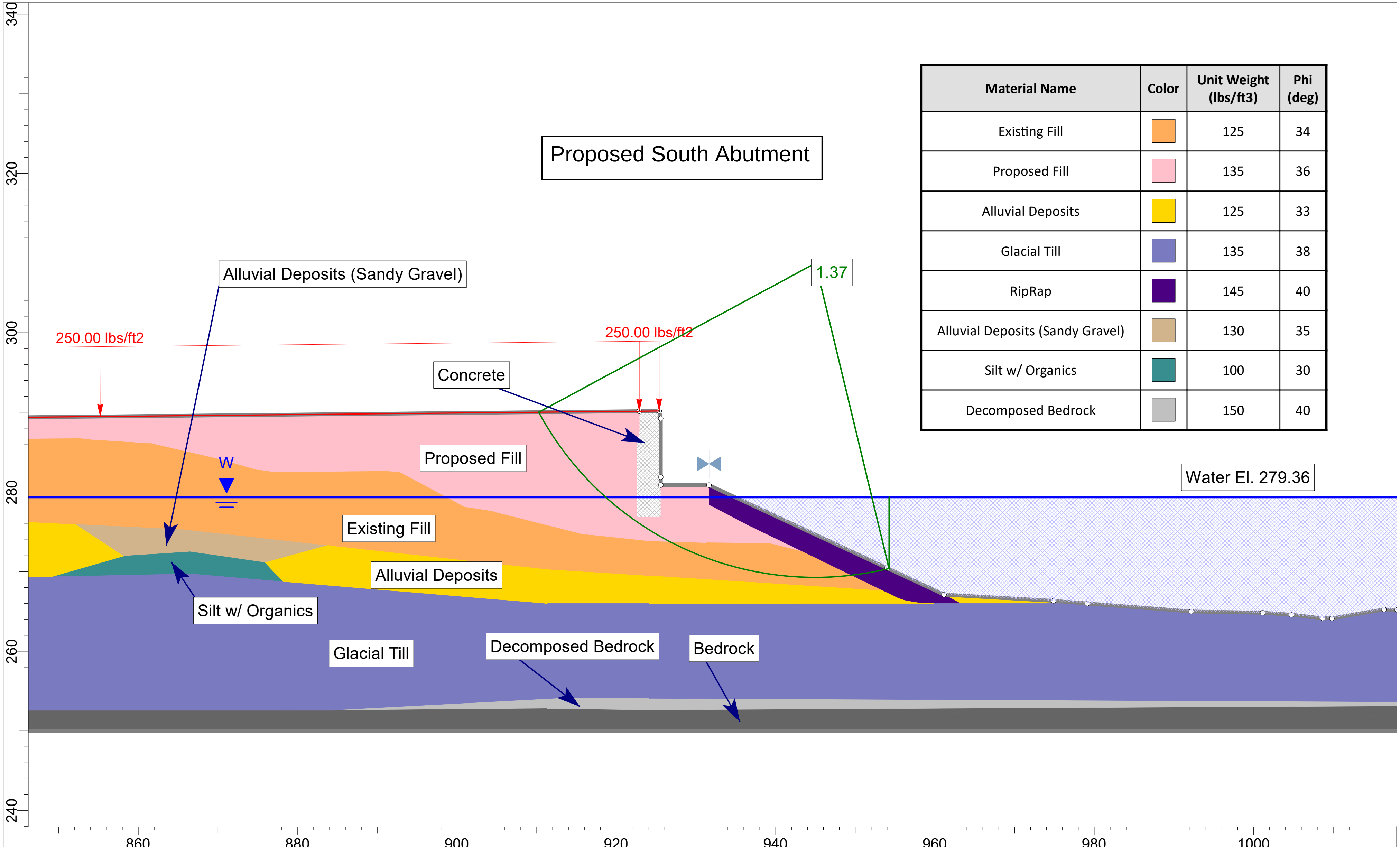


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Existing Fill		125	0	34
Proposed Fill		135	0	36
Glacial Till		135	0	38
RipRap		145	0	40
Presumpscot Formation		115	700	0

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
HP 14x102		Passive (Method B)	10	605000	Perpendicular to pile

Project	North Parish Bridge (#2619) Over Nezinscot River
Analysis Description	Global Slope Stability Analysis Longitudinal Low Groundwater
Drawn By	KAK Check: BTW
Scale	1:200
Company	Nobis Engineering
Date	10/29/2017, 11:36:12 AM
File Name	Longitudinal R to L Low Groundwater.slim

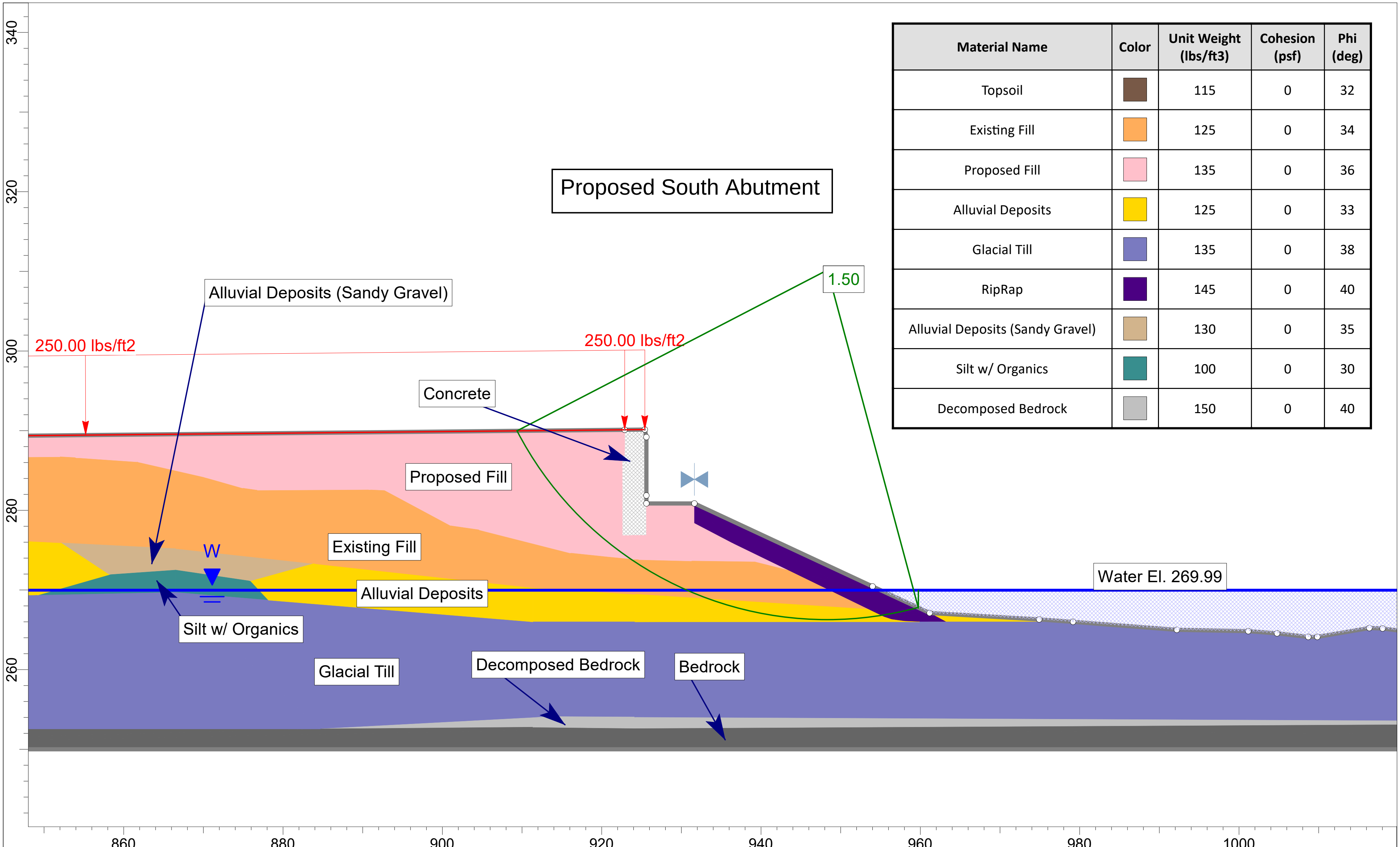
SLIDEINTERPRET 6.039



Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
Existing Fill		125	34
Proposed Fill		135	36
Alluvial Deposits		125	33
Glacial Till		135	38
RipRap		145	40
Alluvial Deposits (Sandy Gravel)		130	35
Silt w/ Organics		100	30
Decomposed Bedrock		150	40

Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis Longitudinal High Groundwater		
Drawn By	KAK Check: BTW	*Scale* 1:200	*Company* Nobis Engineering
Date	10/29/2017, 11:36:12 AM	*File Name*	Longitudinal L to R High Groundwater.slim

SLIDEINTERPRET 6.039

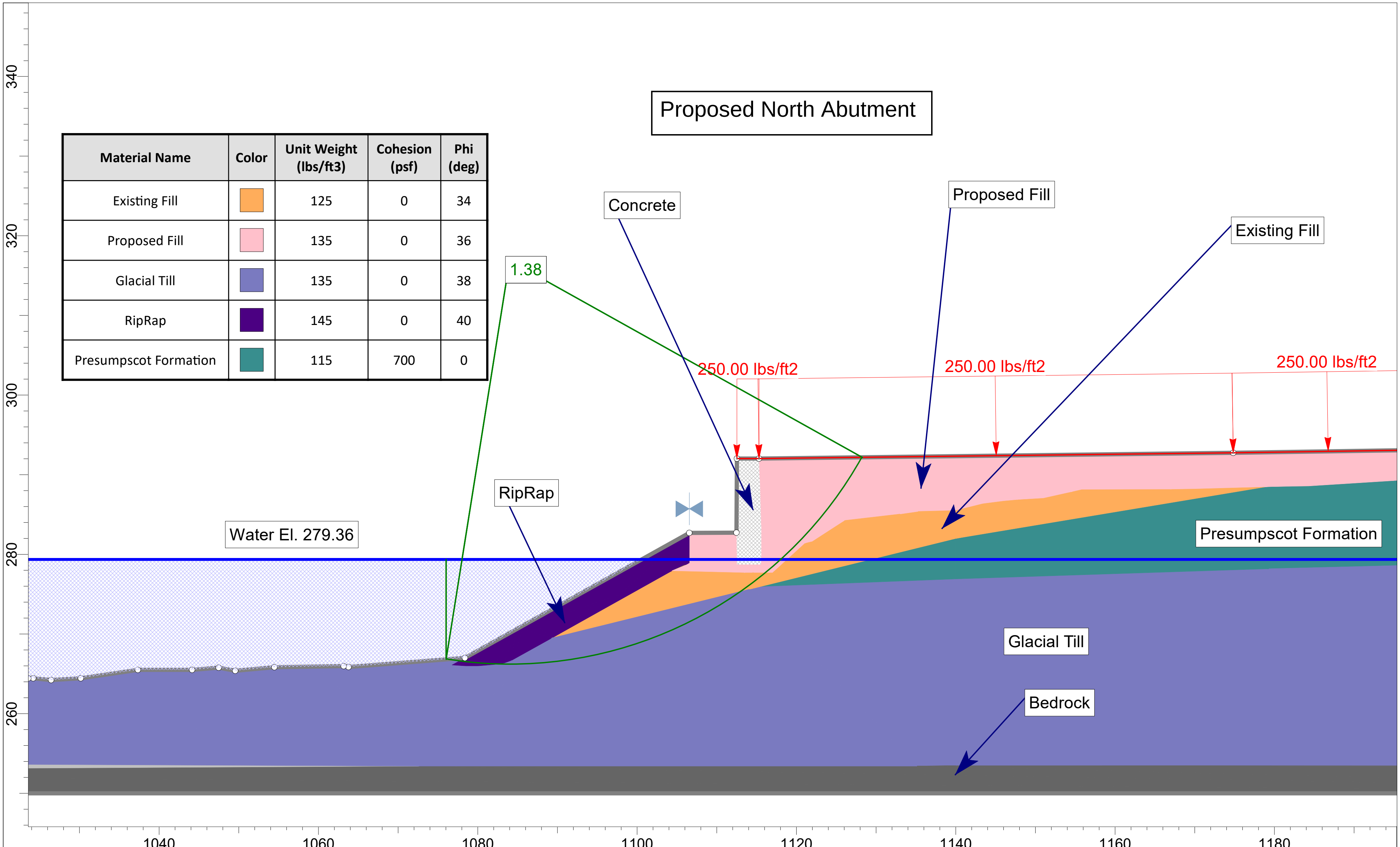


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Alluvial Deposits		125	0	33
Glacial Till		135	0	38
RipRap		145	0	40
Alluvial Deposits (Sandy Gravel)		130	0	35
Silt w/ Organics		100	0	30
Decomposed Bedrock		150	0	40

rocscience

SLIDEINTERPRET 6.039

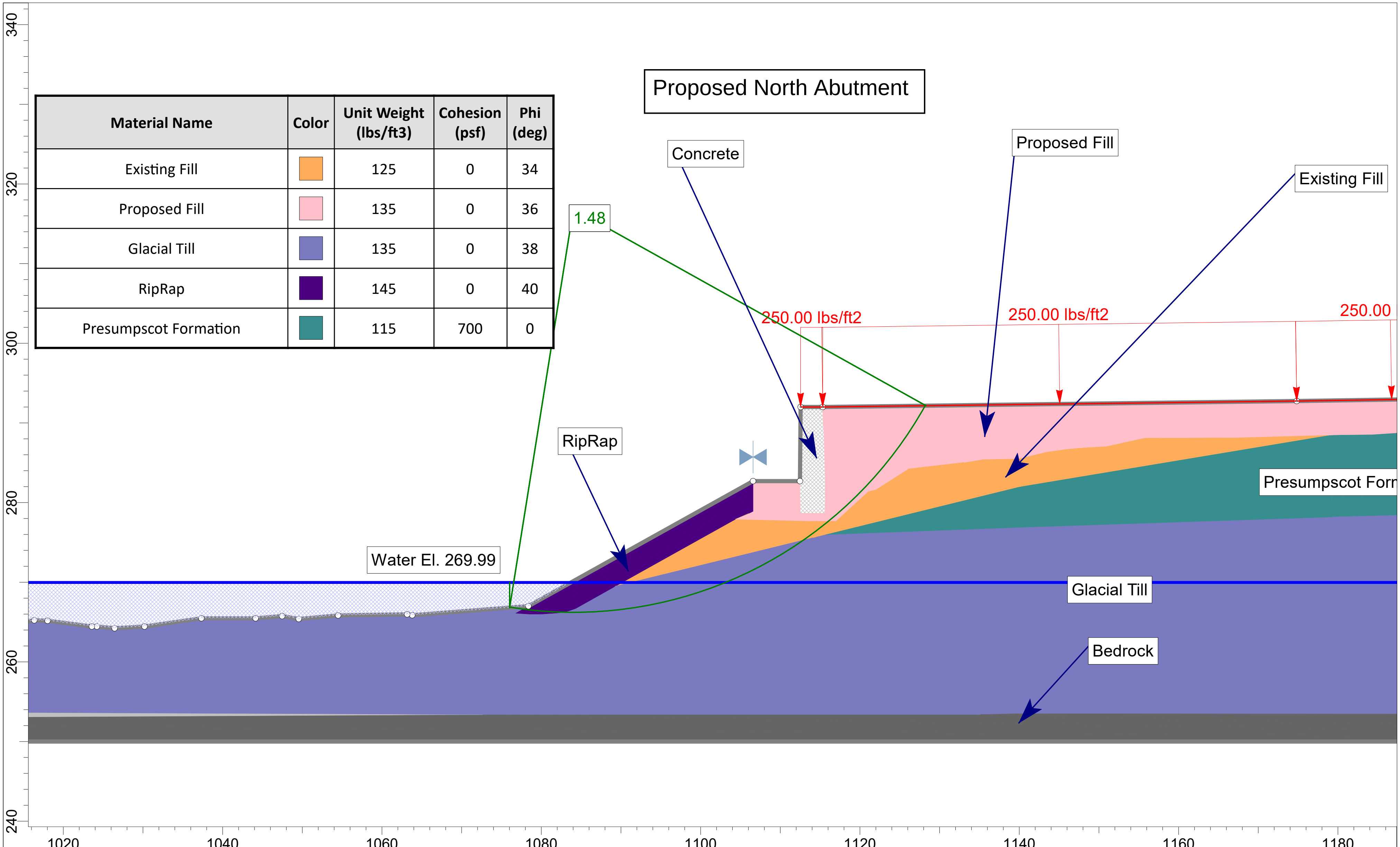
Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis Longitudinal Low Groundwater		
Drawn By KAK Check: BTW	*Scale* 1:200	*Company*	Nobis Engineering
Date 10/29/2017, 11:36:12 AM		*File Name*	Longitudinal L to R Low Groundwater.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Existing Fill		125	0	34
Proposed Fill		135	0	36
Glacial Till		135	0	38
RipRap		145	0	40
Presumpscot Formation		115	700	0

Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis Longitudinal High Groundwater		
Drawn By	KAK Check: BTW	*Scale* 1:200	*Company* Nobis Engineering
Date	10/29/2017, 11:36:12 AM	*File Name*	Longitudinal R to L High Groundwater.slim

SLIDEINTERPRET 6.039



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Existing Fill		125	0	34
Proposed Fill		135	0	36
Glacial Till		135	0	38
RipRap		145	0	40
Presumpscot Formation		115	700	0

Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis Longitudinal Low Groundwater		
Drawn By KAK Check: BTW	*Scale* 1:200	*Company*	Nobis Engineering
Date 10/29/2017, 11:36:12 AM		*File Name*	Longitudinal R to L Low Groundwater.slim

SLIDEINTERPRET 6.039

North Parish Bridge (#2619) over Nezinscot River
Embankment Slopes @ STA 9+00 & 11+25
Turner, Maine

Discipline: Geotechnical

Title: Global Stability Analysis for Proposed Embankments @ STA 9+00 & 11+25

WIN #: 18749.00

Design Basis/Assumptions:

Purpose: Perform global stability analyses at proposed embankments located @ STA 9+00 & 11+25.

Assumptions:

1) Engineering properties for existing fill and the native soils encountered in borings BB-TNR-103 and BB-TNR-105 estimated based on standard penetration test (SPT) data and engineering judgment based on visual classification.
2) Proposed Fill: Unit Weight = 135 pcf, Angle of Internal Friction = 36°.
3) RipRap: Unit Weight = 145 pcf, Angle of Internal Friction = 40°.
4) Groundwater conditions the same or similar to those observed in the subsurface explorations by Nobis. High and low groundwater modeled at approximately EL. 279.36 feet and EL. 269.99, respectively, based on Reference 1.
5) Surcharge loading of 250 psf.

Approach:

Use the computer slope stability analysis software Slide 6.0, by RocScience, Inc., to perform global stability analyses based at the proposed embankment @ STA 9+00 & 11+25, provided by HNTB (Reference 1).

References:

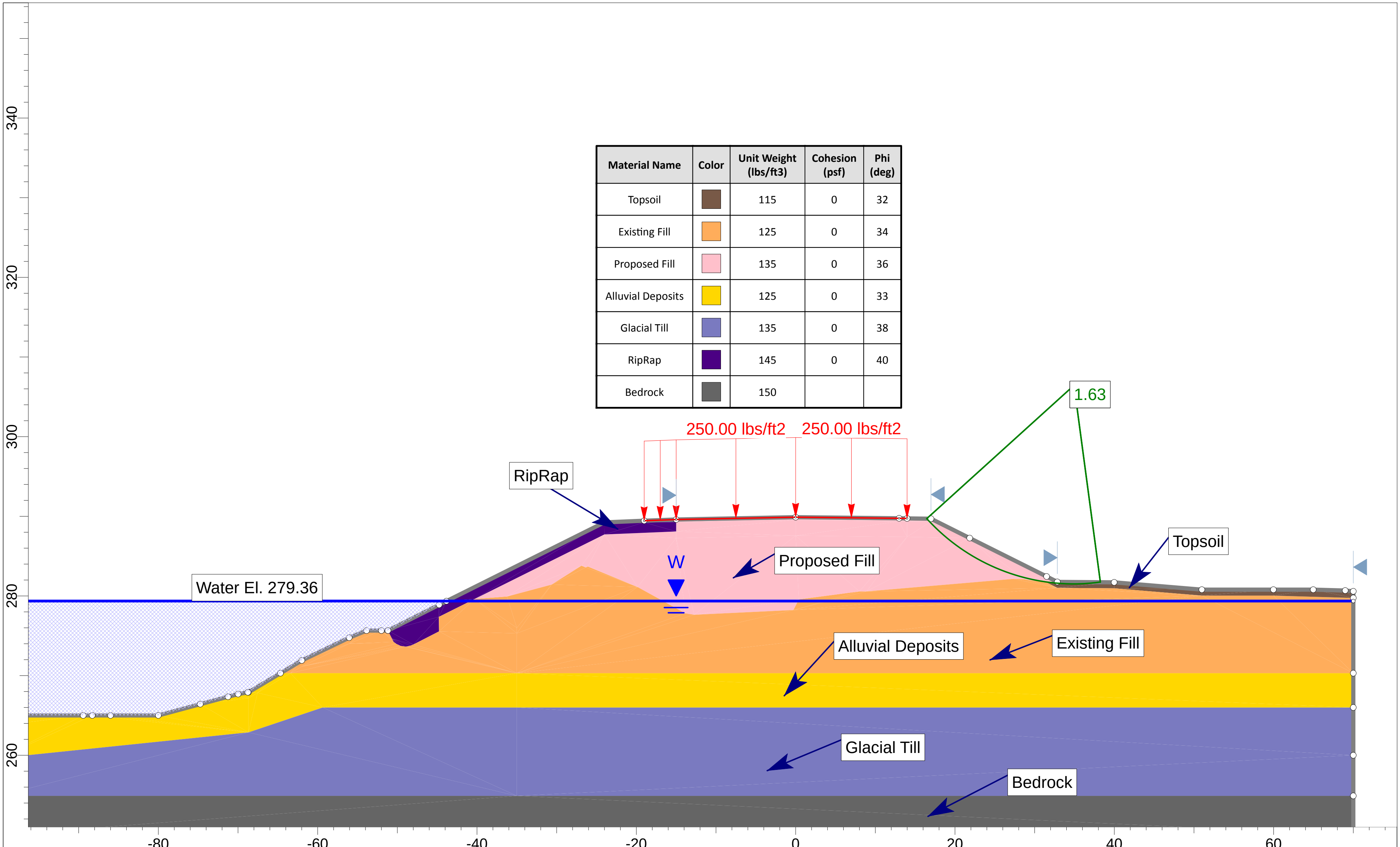
1) 98% Design drawings provided by HNTB, sent on October 16, 2017.
2) SPT data and field classifications by Nobis at borings performed nearest to proposed embankments (BB-TNR-103 and BB-TNR-105).

Remarks/Conclusions:

Based on the Slide analysis, the minimum factor of safety against global slope failure is 1.39 and 1.50 under high groundwater level conditions @ STA 9+00 and 11+25 respectively.

Originated By: Kamil Kocia, Staff Engineer Date: 10/9/2017

Approved By: : Brien Waterman, PE Date: 10/17/2017



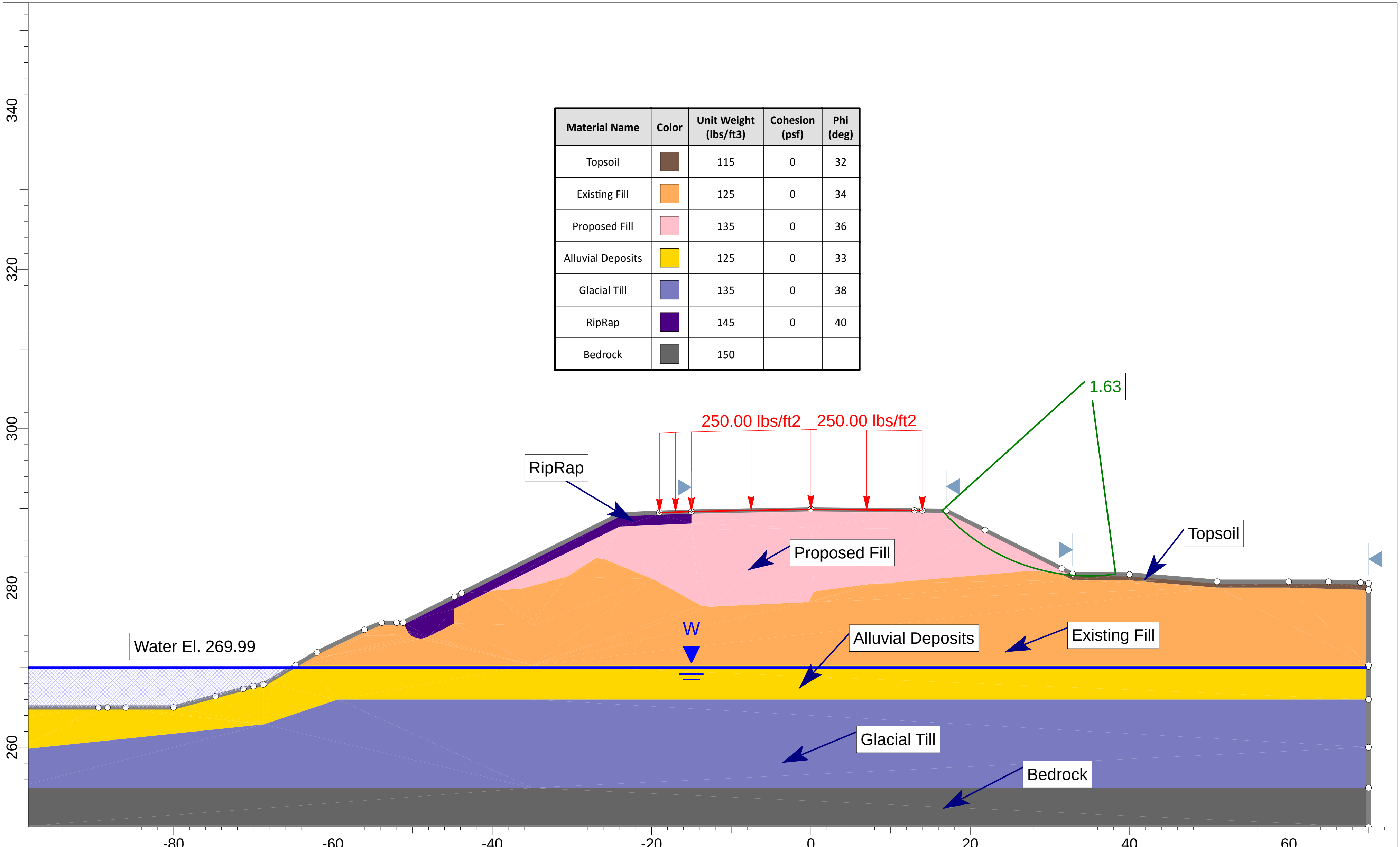
Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Alluvial Deposits		125	0	33
Glacial Till		135	0	38
RipRap		145	0	40
Bedrock		150		

Nobis

Engineering a Sustainable Future

SLIDEINTERPRET 6.020

Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis STA 9+00 L to R High Groundwater		
Drawn By KAK Check: BTW	*Scale* 1:200	*Company*	Nobis Engineering
Date	10/2/2017, 9:36:12 AM	*File Name*	9+00 L to R High Groundwater.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Alluvial Deposits		125	0	33
Glacial Till		135	0	38
RipRap		145	0	40
Bedrock		150		

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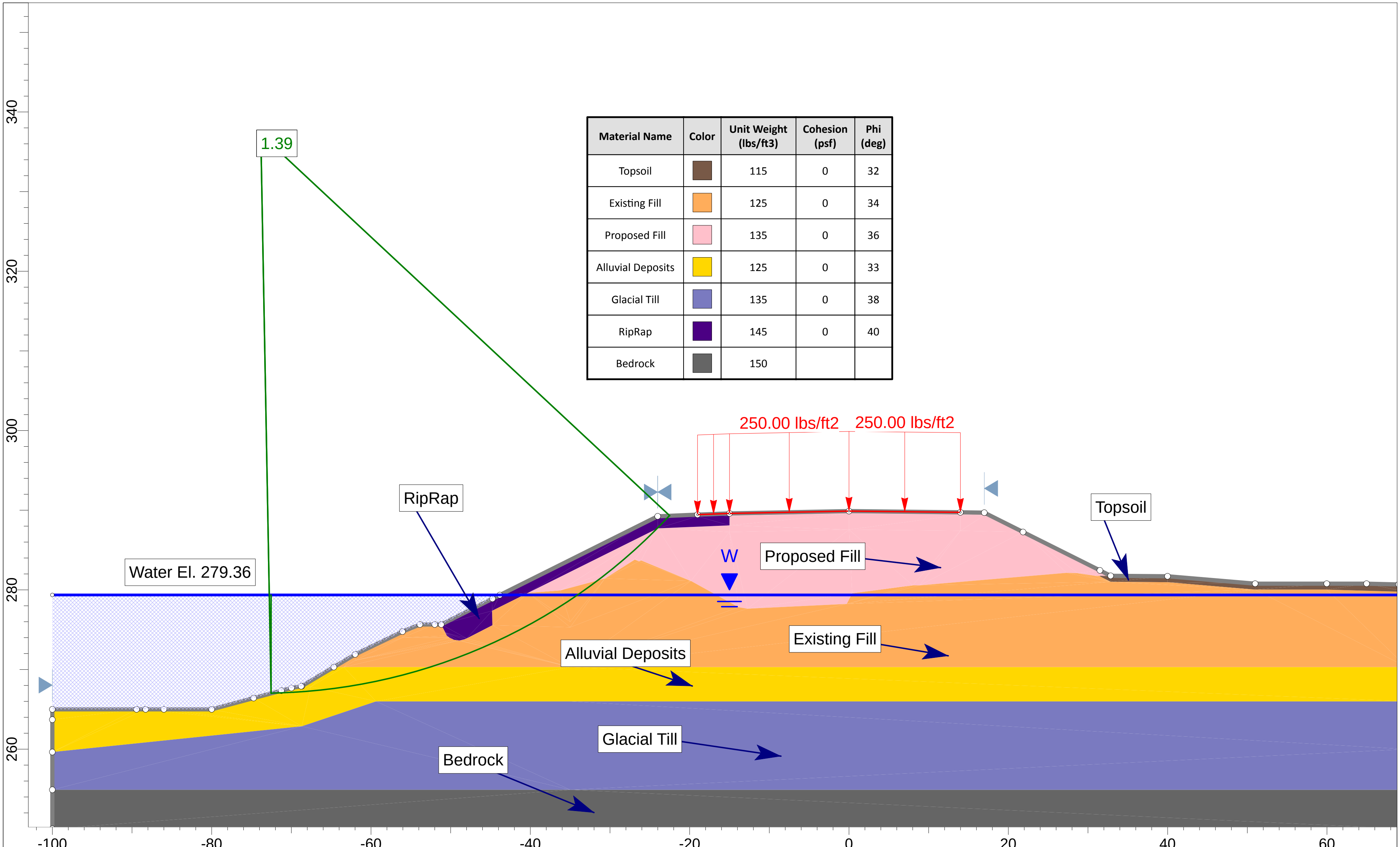
Engineering a Sustainable Future

SLIDEINTERPRET 6.020

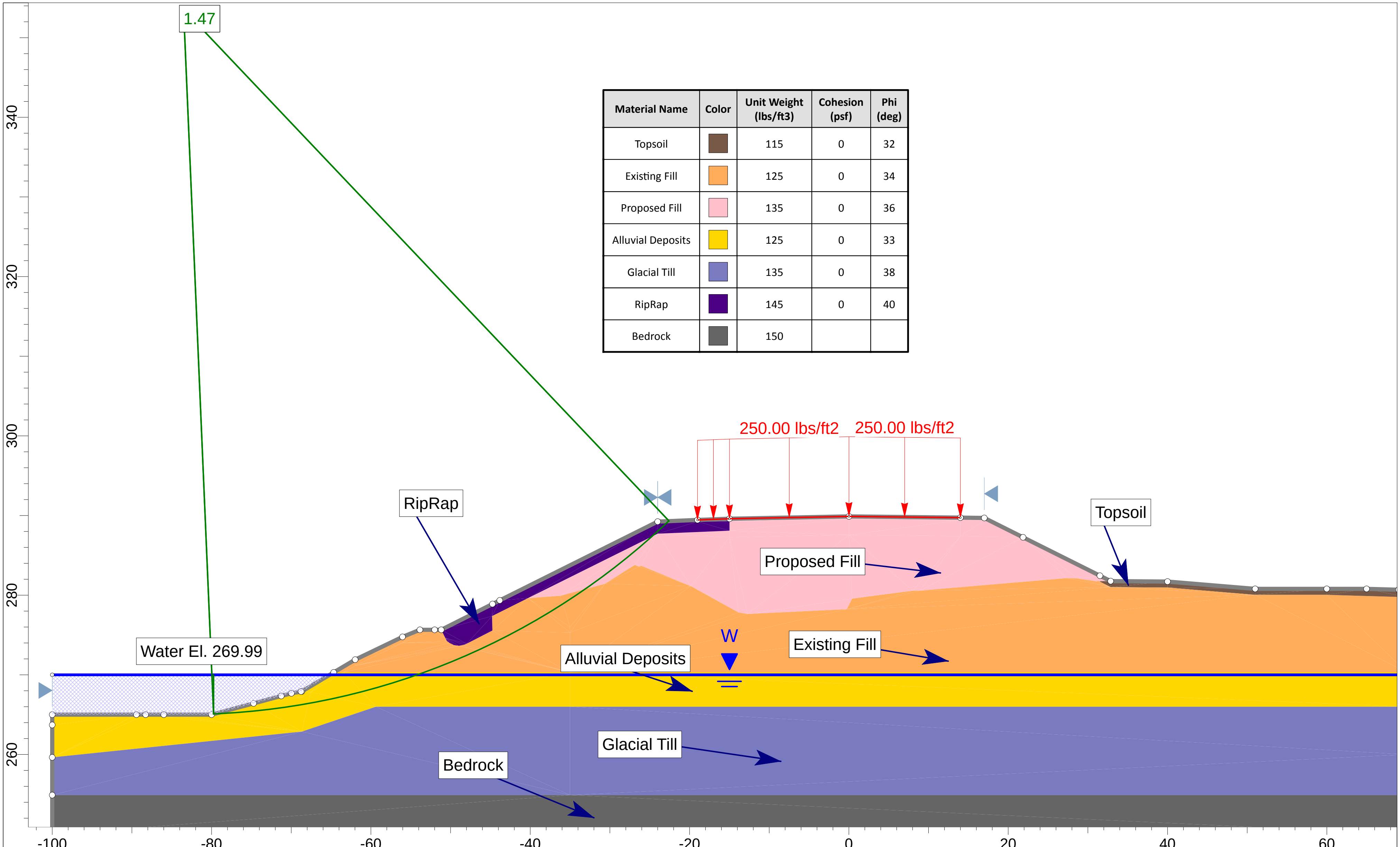
Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis STA 9+00 L to R Low Groundwater		
Drawn By KAK Check: BTW	*Scale* 1:200	*Company*	Nobis Engineering
Date	10/2/2017, 9:36:12 AM	*File Name*	9+00 L to R Low Groundwater.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Alluvial Deposits		125	0	33
Glacial Till		135	0	38
RipRap		145	0	40
Bedrock		150		

Nobis
Engineering a Sustainable Future
SLIDEINTERPRET 6.020

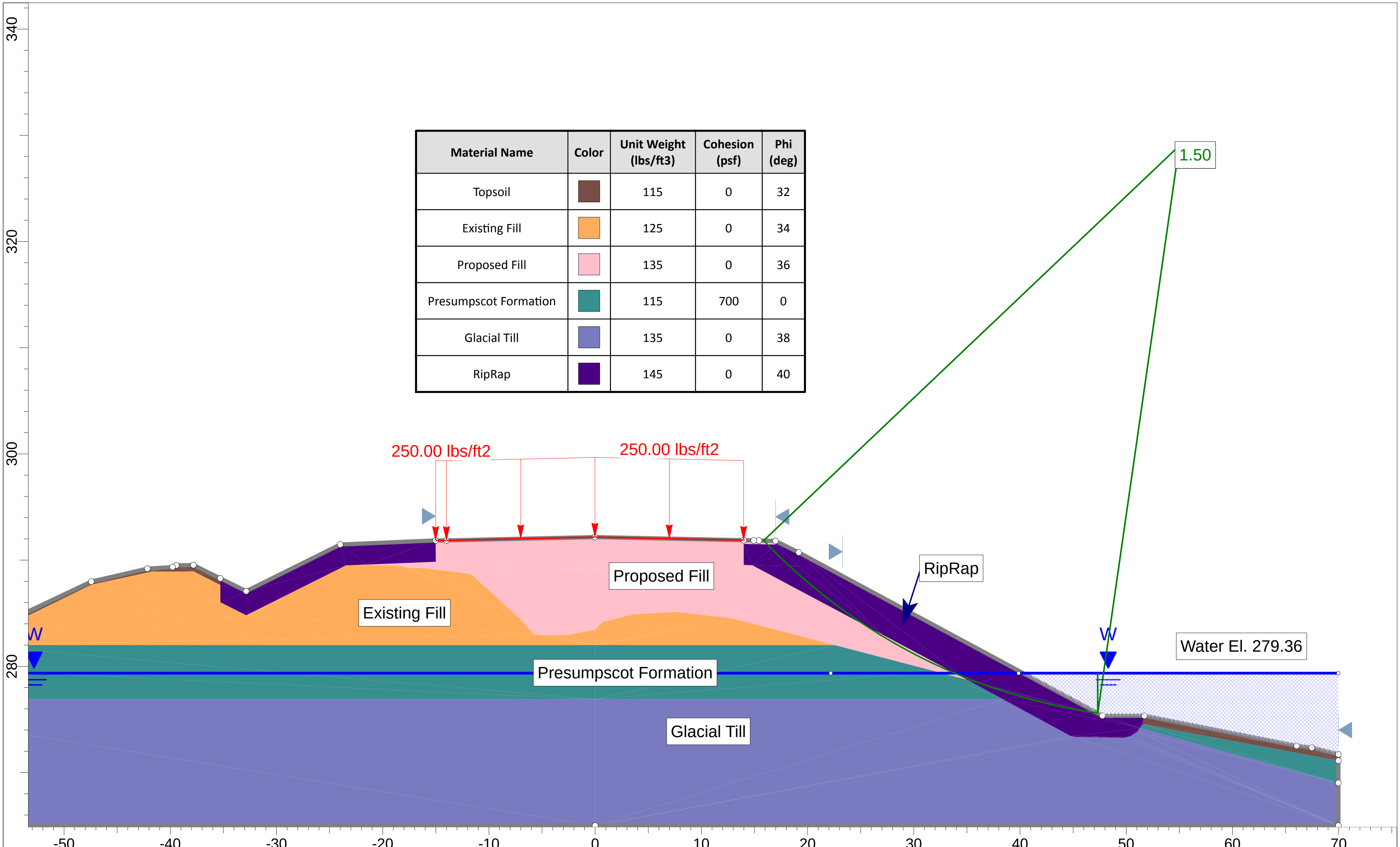
Project	North Parish Bridge (#2619) Over Nezinscot River
Analysis Description	Global Slope Stability Analysis STA 9+00 R to L High Groundwater
Drawn By	KAK Check: BTW
Scale	1:200
Company	Nobis Engineering
Date	10/2/2017, 9:36:12 AM
File Name	9+00 R to L High Groundwater.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Alluvial Deposits		125	0	33
Glacial Till		135	0	38
RipRap		145	0	40
Bedrock		150		

Nobis
Engineering a Sustainable Future
SLIDEINTERPRET 6.020

Project North Parish Bridge (#2619) Over Nezinscot River			
Analysis Description Global Slope Stability Analysis STA 9+00 R to L Low Groundwater			
Drawn By KAK Check: BTW	*Scale* 1:200	*Company* Nobis Engineering	
Date 10/2/2017, 9:36:12 AM		*File Name* 9+00 R to L Low Groundwater.slim	

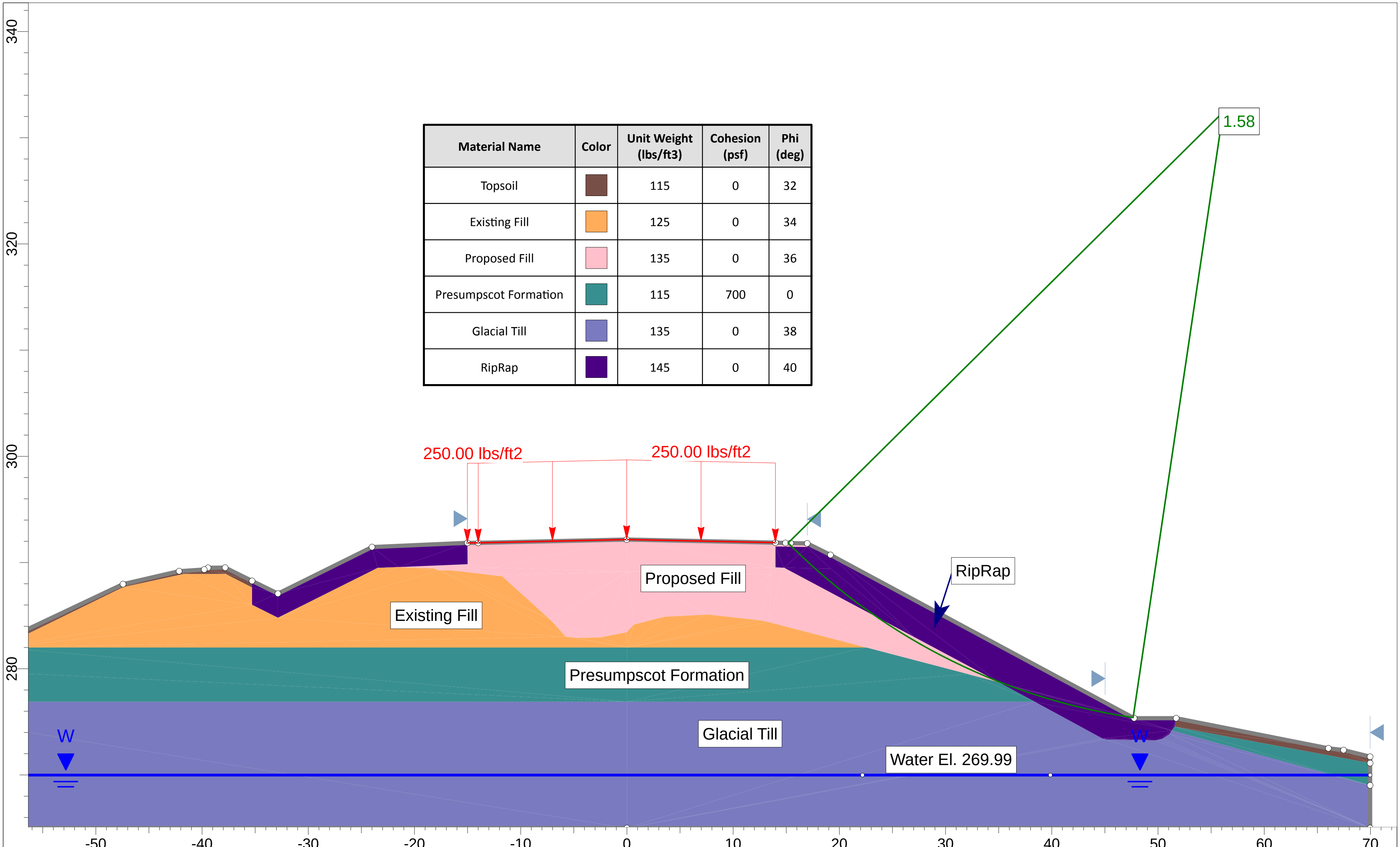


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Presumpscot Formation		115	700	0
Glacial Till		135	0	38
RipRap		145	0	40

Engineering a Sustainable Future

SLIDEINTERPRET 6.020

Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis STA 11+25 L to R High Groundwater		
Drawn By KAK Check: BTW	*Scale* 1:150	*Company*	Nobis Engineering
Date	10/2/2017, 11:27:03 AM	*File Name*	11+25 L to R High Groundwater.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Topsoil		115	0	32
Existing Fill		125	0	34
Proposed Fill		135	0	36
Presumpscot Formation		115	700	0
Glacial Till		135	0	38
RipRap		145	0	40

Nobis

Engineering a Sustainable Future

SLIDEINTERPRET 6.020

Project	North Parish Bridge (#2619) Over Nezinscot River		
Analysis Description	Global Slope Stability Analysis STA 11+25 L to R Low Groundwater		
Drawn By KAK Check: BTW	*Scale* 1:150	*Company*	Nobis Engineering
Date 10/2/2017, 11:27:03 AM		*File Name*	11+25 L to R Low Groundwater.slim

APPENDIX E.5 – Settlement Estimate – Proposed North Abutment

Consolidation Settlement Analysis of North Parish Road Bridge (#2619)

Purpose: Evaluate settlement at proposed embankment / roadway section located at STA 11+25. Estimate settlement induced by proposed raises in grades.

Assumptions: 1. The soil layers beneath the site are horizontal and continuous.
2. Based on borings BB-TNR-105, -106, -107, the clay/silt layer (Presumpscot Formation clay) is above the groundwater table and as a result is unsaturated. We assumed the silt/clay will be above the groundwater table during proposed fill placement.

References: 1. An Introduction to Geotechnical Engineering, Holtz & Kovacs, 1981.
2. 98% Design Construction Drawings for WIN 18749.00 Turner North Parish road over Nezinscot River, located in Turner, Maine.
3. Settle^{3D} User Manual, provided in the RocScience Settle^{3D} program.
4. The Engineering Aspects of The Presumpscot Formation, by David W. Andrews, P.E. at Morrison Geotechnical Engineering, dated 1987.
5. Preliminary Geotechnical Design Basis Letter, prepared by Nobis Engineering, dated February 22, 2017.

Solution:

Consolidation Analysis

Based on the 100-series boring, completed at the northern side of the bridge (i.e. BB-TNR-105, -106, -107), approximately 5 to 10 feet of clay was encountered. Additionally, based on the borings, the clay was encountered above the groundwater table.

Consolidation analysis for the proposed embankment/roadway section was performed using Settle^{3D} software developed by Rocscience, Inc. A plan and 3D view of the model is attached. Settle^{3D} was used to model the proposed site geometry considering the influence of the structural fill loads and existing soil strata. Settle^{3D} was used to perform a settlement analysis while considering the influence of the existing and proposed soil and loading conditions in a simplified 3D model. Based on the 98% plans provided by MaineDOT (Reference 2), the raises in grades in the building area generally vary between 1 and 12 feet along the south and north sections of the roadway. We evaluated a 10-feet raise in grades on the north side of the bridge, where Presumpscot formation clay was present.

Subsurface Conditions

Generalized soil profile within the embankment footprint on the north side of North Parish Road consists of:

1. Existing Fill (silty/gravel sand-like), 1 to 5 foot thick, over
2. Presumpscot Formation Clay and/or Silt, 5 to 10 feet thick, over
3. Glacial Till, 20 to 22 feet thick.

The existing fill will be replaced with compacted structural fill. The generalized subsurface profile evaluated in the model consisted of:

1. Structural Fill – 10 feet thick (modeled by load), overlying

2. Existing Fill – 1-foot thick, overlying
3. Presumpscot Formation Clay and/or Silt – 8 feet thick, overlying
4. Glacial Till – 20 feet in thickness

Material Properties

Material properties including recompression ratio (RR), virgin compression ratio (CR), overconsolidation ratio (OCR), preconsolidation stress (σ'_p), and estimated coefficient of consolidation (c_v) can be found on Reference 4. The unit weights were estimated based on sample descriptions, lab data, and SPT-N values, found on Reference 5. The elastic constants (i.e. Poisson's ratio (Es) and Young's modulus(ν)) were estimated based on sample descriptions, SPT-N values, and tabular data presented in Reference 3.

Elastic Constants

Elastic constants were estimated for the Existing Fill and Glacial Till to estimate immediate settlement. The elastic constants for the Structural Fill and Glacial Till are based on a consistency of loose and medium dense, respectively. The following elastic constants (based on Reference 3 and Reference 4) were used in our analysis.

Existing Fill:	Es = 150 ksf ν = 0.30
Glacial Till:	Es = 2,000 ksf ν = 0.35

Compression Indices

Compression indices were obtained from in-situ and non-in-situ information from instrumented sties in Reference 4. The following compression indices (based on Reference 3 and 4) were used for the Clay / Silt and Silt / Clay formation in our analysis.

RR = 0.02 (for Presumpscot Formation)

CR = 0.20 (for Presumpscot Formation)

Overconsolidation Ratio

Based on Reference 3, the Presumpscot Formation is an overconsolidated deposit with an upper crust that has been significantly overconsolidated. We assumed an overconsolidation (OCR) of 3 for our evaluation.

Coefficient of Consolidation

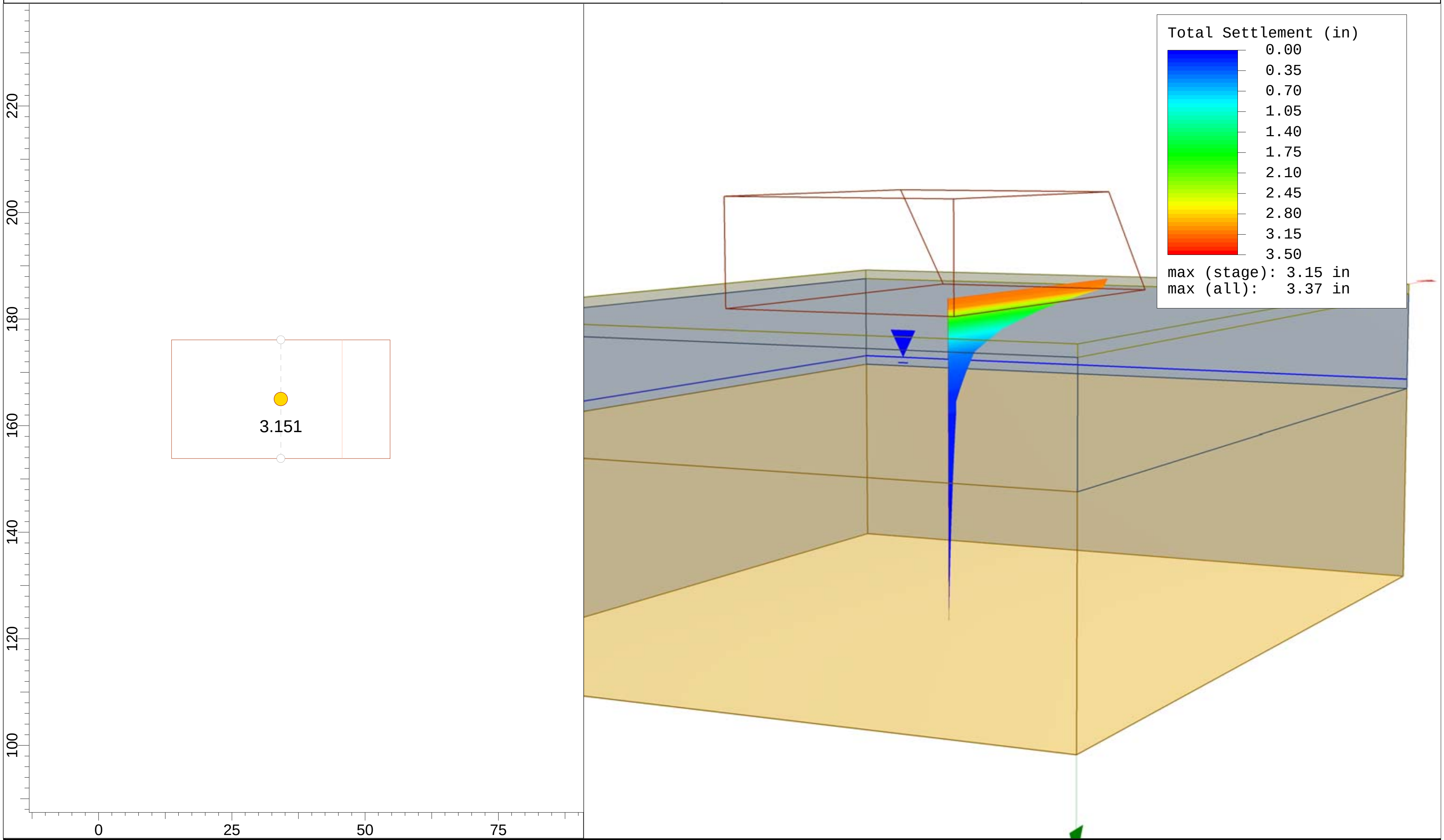
The coefficient of consolidation is used to perform time-settlement analysis and determine post-construction settlements. The coefficient of consolidation was estimated based on References 3 and 4. Based on Reference 4, the coefficient of consolidation for Presumpscot Formation deposits ranges from approximately 18 to 55 ft^2/year. We assumed a coefficient of consolidation of 35 ft^2/year for our evaluation.

Results

Based on our evaluation of the loads induced by the 10 feet raise in grades, total settlement is on the order of 3 inches. Additionally, the clay was encountered above the groundwater table and is not completely saturated, therefore, we anticipate the settlement will occur during placement of the fill.

Stage 2 = 0 d

Data Type: Total Settlement



Project	North Parish Road over Nezinscot River
Analysis Description	Settlement Calculation
Drawn By	KK checker: BTW
Scale	$ModelScale
Company	Nobis Engineering Inc.
Date	11/1/2017
File Name	BB-TNR-105 through 107 - Surcharge.s3z

Engineering a Sustainable Future

SETTLE3D 2.019

Data Type:

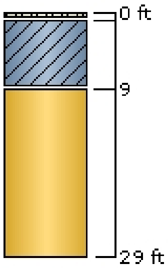
Settle3D Analysis Information

North Parish Road over Nezinscot River

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Existing Fill	1	0	Yes
2	Presumpscot Formation (Clay/Silt)	8	1	Yes
3	Glacial Till	20	9	No

Ground Surface Drained: Yes



Soil Properties

Property	Existing Fill	Presumpscot Formation (Clay/Silt)	Glacial Till
Color			
Unit Weight [kips/ft^3]	0.125	0.11	0.13
Saturated Unit Weight [kips/ft^3]	0.125	0.11	13
Immediate Settlement	Enabled	Disabled	Enabled
Es [ksf]	150		2000
Esur [ksf]	150		2000
Primary Consolidation	Disabled	Enabled	Disabled
Material Type		Non-Linear	
Cce		0.2	
Cre		0.02	
OCR	1	3	1
Cv [ft^2/y]		35	
B-bar		1	
Secondary Consolidation	Disabled	Standard	Disabled
Cae		0.008	
Caer		0.008	

Engineering a Sustainable Future

SETTLE3D 2.019

Project North Parish Road over Nezinscot River

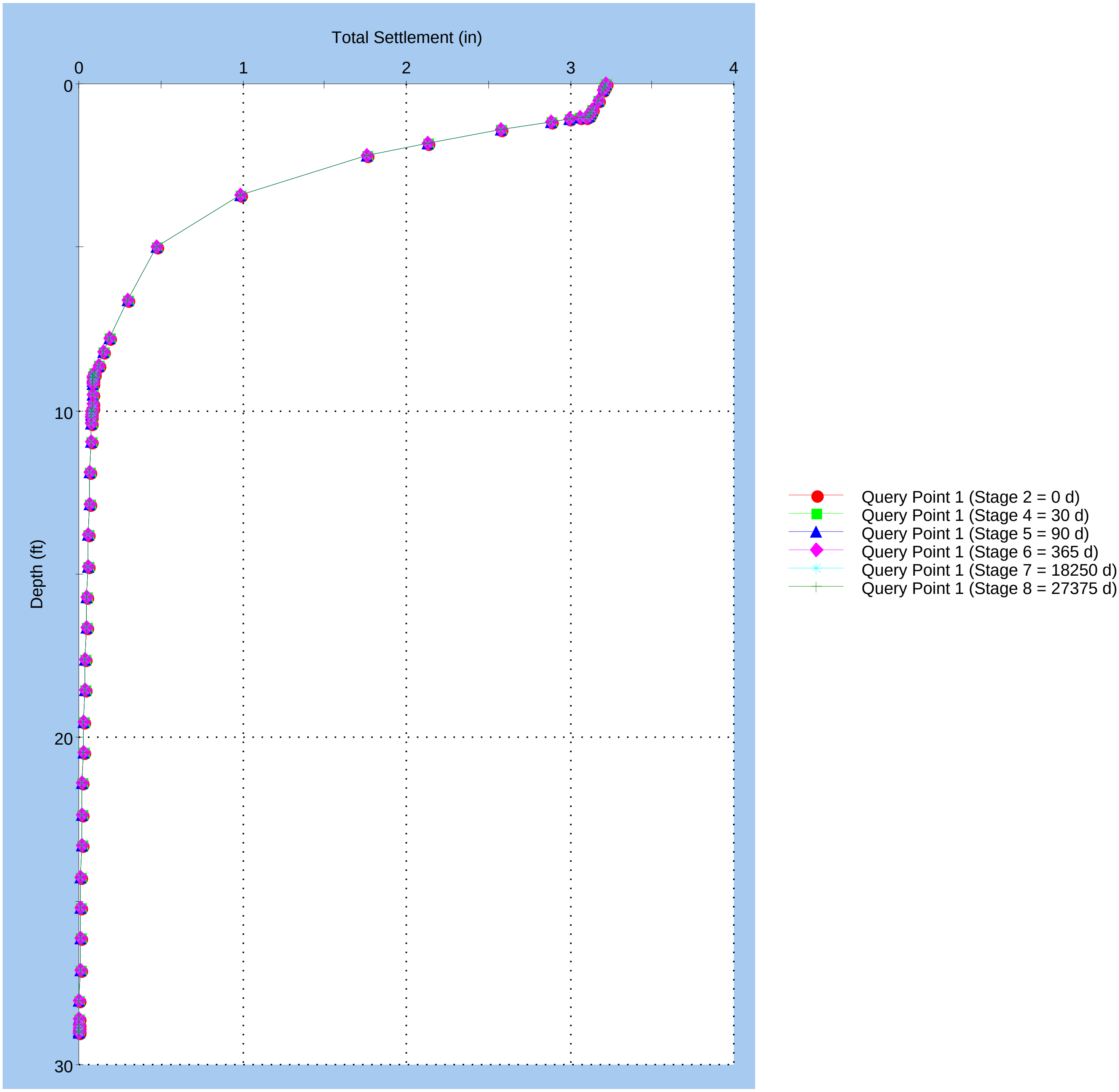
Analysis Description Settlement Calculation

Drawn By KK checker: BTW | *Scale* $ModelScale | *Company* Nobis Engineering Inc.

Date 11/1/2017 | *File Name* BB-TNR-105 through 107 - Surcharge.s3z

Data Type:

Total Settlement vs. Depth



Reference Stage: None

Nobis Engineering a Sustainable Future	*Project* North Parish Road over Nezinscot River		
	Analysis Description Settlement Calculation		
	Drawn By KK checker: BTW	*Scale* $ModelScale	*Company* Nobis Engineering Inc.
SETTLE3D 2.019	*Date* 11/1/2017		*File Name* BB-TNR-105 through 107 - Surcharge.s3z

APPENDIX E.6 – Seismic Calculations

Preliminary seismic site classification for the North Parish Road Bridge over the Nezinscot River

Objective: Evaluate seismic site classification for the above mentioned project site.

References:

1) AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014, with 2016 Interim Revisions.
2) Borings Observed by Nobis Engineering in December, 2016.
3) MaineDOT Bridge Design Manual, 2003, with revisions through March, 2014.

Solution:

AASHTO Section 3.10 was used to determine the seismic site classification for the North Parish Road Bridge project site, as follows:

Step 1: Check for the three categories of Site Class F as described in Table 3.10.3.1-1 - Site Classification Definitions, as follows:

1. Peats or highly organic clays (H>10 ft of peat or highly organic clay where H = thickness of soil).
2. Very high plasticity clays (H>25 ft with PI >75).
3. Very thick soft/medium stiff clays (H > 120 ft).

Soil conditions for Site Class F were not shown on the Nobis Boring Logs.

Step 2: Check for existence of a soft layer with total thickness >10 ft, where soft layer is defined by su<0.5 ksf, w>40%, and PI>20. If these criteria are met, classify site as Site Class E.

Soil conditions for Site Class E were not shown on the Nobis Boring Logs.

Step 3: Categorize the site using one of three methods (i.e. A, B, or C).

Method B (N-bar Method) was used to determine the average Standard Penetration Test (SPT) blow count (blows/ft) for the upper 100 ft of the soil profile using the 2016 borings. See attached Table 3.10.3.1-1. The 2016 borings were performed in general accordance with ASTM D1586. The samples were obtained using a 1-3/8" diameter sampler driven with a 140-lb safety hammer dropping a distance of 30 inches.

Conclusion:

Excluding test boring BB-TNR-104, which was cored through the bridge deck to mudline approximately 23.5 feet below deck surface; the average Standard Penetration Resistance, N-bar, for the site ranges from 13 to 34 (average value 26) based on the Nobis boring logs. The seismic site classification for the project site is **Site Class D.** The calculated values for the borings are summarized in the following table. See attached tables presenting the value of SPT-N vs. depth for the respective borings.

Sample Calculation: Consider Boring BB-TNR-101

Determine N-bar: Use the average N value of each layer, N_i, for each layer provided in the table toward the end of this report.

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \overline{N} = \frac{5' + 5.3' + 4.7' + 5' + 5' + 5' + 70'}{\frac{5'}{25} + \frac{5.3'}{9} + \frac{4.7'}{6} + \frac{5'}{6} + \frac{5'}{26} + \frac{5'}{17} + \frac{70'}{100}} = 28\ bpf$$

Determine the site class for this boring using the Site Class Definitions, attached (Table 3.10.3.1-1, Reference 1)

An N-bar of 28 bpf does not meet the criteria for Site Class C. Boring BB-TNR-101 is classified as **Site Class D** as its standard penetration resistance of 28 bpf is less than 50 bpf. Below, summary tables provide general information for all of the borings considered in this analysis.

Table: Summary of Seismic Site Classification from Borings Performed by Nobis Engineering

Boring No.	N-bar (bpf)	Site Class	Comments
BB-TNR-101	**28**	Site Class D	Boring terminated at 30.4 feet bgs
BB-TNR-102	**24**	Site Class D	Boring terminated at 31.8 feet bgs
BB-TNR-103	**31**	Site Class D	Boring terminated at 34 feet bgs
BB-TNR-104	**89**	Site Class C	Boring terminated at 24 feet bgs
BB-TNR-105	**25**	Site Class D	Boring terminated at 45 feet bgs
BB-TNR-106	**34**	Site Class D	Boring terminated at 30 feet bgs
BB-TNR-107	**13**	Site Class E	Boring terminated at 32 feet bgs

Note: The SPT-N values were corrected to N_{60} values assuming the automatic hammer used delivered 63% energy.

Data from Boring BB-TNR-101 (performed December, 2016)

Layer	Depth Range Start [ft]	Depth Range End [ft]	Thickness (d_i) [ft]	N_i blows/ft	d_i/N_i	Comments
1	0	5	5	25	0.200	Fill
2	5	10.3	5.3	9	0.589	
3	10.3	15	4.7	6	0.783	Sand/Silt
4	15	20	5	6	0.833	
5	20	25	5	26	0.192	Glacial Till
6	25	30	5	17	0.294	
7	30	100	70	100	0.700	Glacial Till & Bedrock
SUM			**100**		**3.592**	

N-bar	**28**	**Site Class D - Stiff Soil**

Data from Boring BB-TNR-102 (performed December, 2016)

Layer	Depth Range Start [ft]	Depth Range End [ft]	Thickness (d_i) [ft]	N_i blows/ft	d_i/N_i	Comments
1	0	5	5	4	1.250	Fill
2	5	10.3	5.3	7	0.757	
3	10.3	13	2.7	8	0.338	Sandy Gravel
4	13	15.8	2.8	4	0.700	Silt with Organics (N-bar not representative, use 4)
5	15.8	25	9.2	25	0.368	Glacial Till
6	25	30	5	56	0.089	
7	30	100	70	100	0.700	Glacial Till & Bedrock
SUM			**100**		**4.202**	

N-bar	**24**	**Site Class D - Stiff Soil**

Data from Boring BB-TNR-103 (performed December, 2016)

Layer	Depth Range Start [ft]	Depth Range End [ft]	Thickness (d_i) [ft]	N_i blows/ft	d_i/N_i	Comments
1	0	5.7	5.7	3	1.900	Fill
2	5.7	10	4.3	32	0.134	Sand & Silt
3	10	15	5	18	0.278	Glacial Till
4	15	20	5	35	0.143	
5	20	24	4	81	0.049	Decomposed Bedrock
6	24	100	76	100	0.760	Bedrock
SUM			**100**		**3.264**	

N-bar	**31**	**Site Class D - Stiff Soil**

Data from Boring BB-TNR-104 (performed December, 2016)

Layer	Depth Range Start [ft]	Depth Range End [ft]	Thickness (d_i) [ft]	N_i blows/ft	d_i/N_i	Comments
1	0	4.5	4.5	27	0.167	Borehole cored through bridge deck to mudline (Glacial Till) approximately 23.5 feet below deck surface
2	4.5	9.5	5	100	0.050	
3	9.5	10.6	1.1	69	0.016	
4	10.6	14	3.4	100	0.034	Decomposed Bedrock
5	14	100	86	100	0.860	Bedrock
SUM			**100**		**1.127**	

N-bar	**89**	**Site Class C - Very Dense Soil & Soft Rock**

Data from Boring BB-TNR-105 (performed December, 2016)

Layer	Depth Range Start [ft]	Depth Range End [ft]	Thickness (d_i) [ft]	N_i blows/ft	d_i/N_i	Comments
1	0	5	5	3	1.667	Fill
2	5	7	2	7	0.286	Clay
3	7	10.1	3.1	5	0.620	
4	10.1	15	4.9	59	0.083	Glacial Till
5	15	20	5	9	0.556	
6	20	25	5	53	0.094	
7	25	30	5	90	0.056	
8	30	33.5	3.5	72	0.049	
9	33.5	100	66.5	100	0.665	Bedrock
SUM			**100**		**4.074**	

N-bar	**25**	**Site Class D - Stiff Soil**

Data from Boring BB-TNR-106 (performed December, 2016)

Layer	Depth Range Start [ft]	Depth Range End [ft]	Thickness (d_i) [ft]	N_i blows/ft	d_i/N_i	Comments
1	0	1	1	12	0.083	Topsoil/Subsoil
2	1	10.8	9.8	6	1.633	Clay
3	10.8	15	4.2	81	0.052	Glacial Till
4	15	20	5	26	0.192	
5	20	27.5	7.5	34	0.221	
6	27.5	100	72.5	100	0.725	Glacial Till & Bedrock
SUM			**100**		**2.906**	

N-bar	**34**	**Site Class D - Stiff Soil**

Data from Boring BB-TNR-107 (performed December, 2016)

Layer	Depth Range		Thickness (d_i)	N_i	d_i/N_i	Comments
	Start [ft]	End [ft]	[ft]	blows/ft		
1	0	1	1	4	0.250	Topsoil/Subsoil
2	1	4	3	4	0.750	Clay
3	4	6	2	1	4.000	
4	6	10.8	4.8	4	1.200	
5	10.8	15	4.2	18	0.233	Glacial Till
6	15	20	5	14	0.357	
7	20	25	5	60	0.083	
8	25	30	5	100	0.050	
9	30	100	70	100	0.700	Glacial Till & Bedrock
SUM			**100**		**7.624**	

N-bar	**13**

Site Class E

Design Maps Summary Report

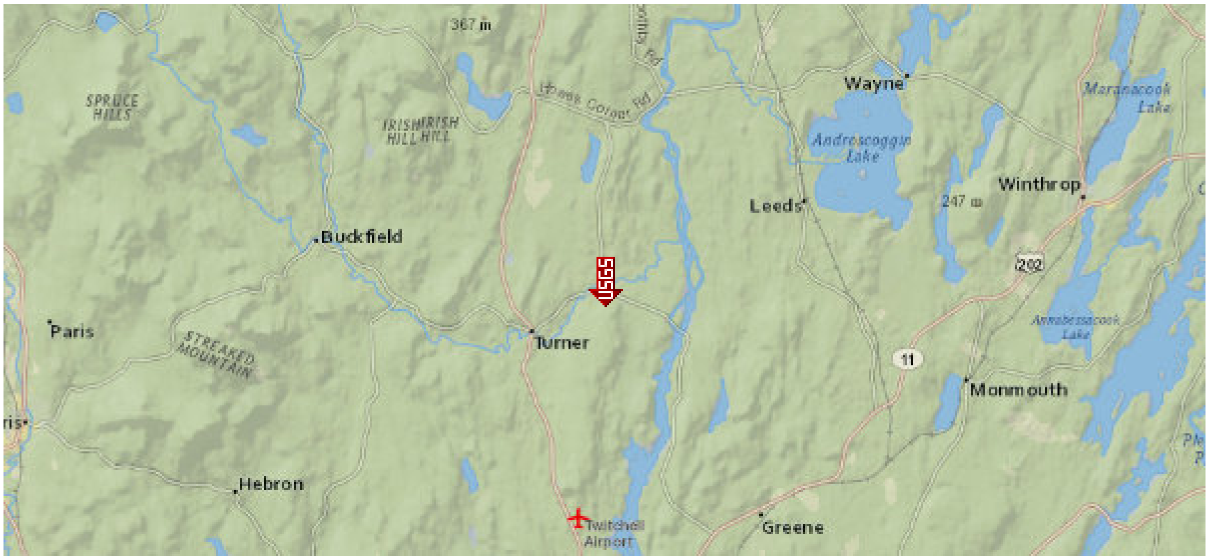
User–Specified Input

Report Title North Parish Road over Nezinscot River
Fri January 20, 2017 20:59:33 UTC

Building Code Reference Document 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design
(which utilizes USGS hazard data available in 2002)

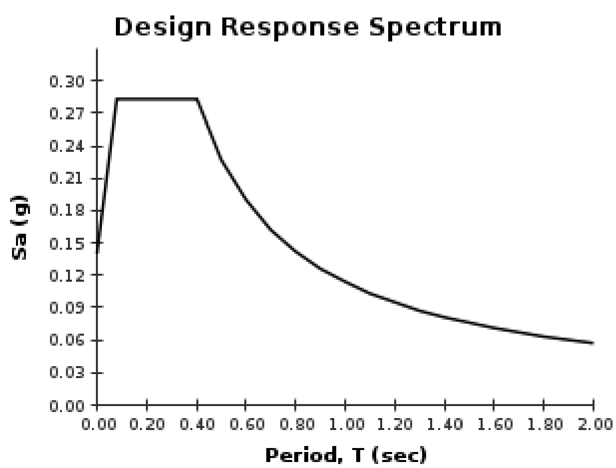
Site Coordinates 44.27507°N, 70.21949°W

Site Soil Classification Site Class D – "Stiff Soil"



USGS–Provided Output

PGA =	0.088 g	**A_s =**	0.140 g
S_S =	0.177 g	**S_{DS} =**	0.283 g
S_1 =	0.047 g	**S_{D1} =**	0.114 g



Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (44.27507°N, 70.21949°W)

Site Class D – "Stiff Soil"

Article 3.4.1 — Design Spectra Based on General Procedure

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

From Figure 3.4.1-2 [1]	PGA = 0.088 g
From Figure 3.4.1-3 [2]	S_S = 0.177 g
From Figure 3.4.1-4 [3]	S_1 = 0.047 g

Article 3.4.2.1 — Site Class Definitions

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

Table 3.4.2.1–1 Site Class Definitions

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

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

Article 3.4.2.3 — Site Coefficients

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

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

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

For Site Class = D and PGA = 0.088 g, F_{PGA} = 1.600

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

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	S_S ≤ 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	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.177 g, F_a = 1.600

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

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

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

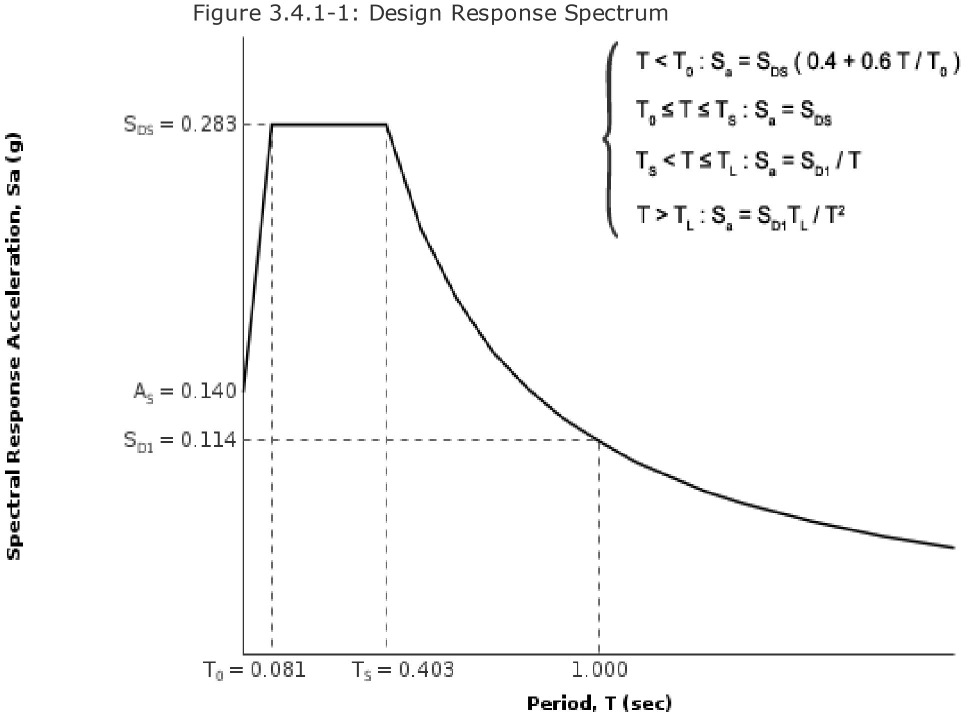
For Site Class = D and S_1 = 0.047 g, F_v = 2.400

Equation (3.4.1-1): $A_S = F_{PGA}\ PGA = 1.600 \times 0.088 = 0.140$ g

Equation (3.4.1-2): $S_{DS} = F_a\ S_S = 1.600 \times 0.177 = 0.283$ g

Equation (3.4.1-3): $S_{D1} = F_v\ S_1 = 2.400 \times 0.047 = 0.114$ g

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

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

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

For S_{D1} = 0.114 g, Seismic Design Category = A

Seismic Design Category ≡ “the design category in accordance with Table 3.5-1” = A

References

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

APPENDIX E.7 – Preliminary Driveability Calculation

Project No.:	92270.02		
Project Title:	North Parish Bridge - Turner, Maine		
Calculated by:	BTW	**Date:**	11.6.2017
Checked by:	RAC	**Date:**	11.6.2017

Preliminary Driveability Evaluation - Revision 1

Objective: Evaluate preliminary driveability of proposed HP14x102 piles for integral abutments.

Approach: 1. Use GRLWEAP to evaluate preliminary driveability.

References: 1. Borings BB-TNR-103 and 105.
2. Preliminary Cross Sections, MaineDOT, June 2017

Assumptions: 1. Piles terminate on or within bedrock corresponding to approximate 21-foot pile lengths.

Solution:

For piles in compression or tension

$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ where;

$f_y =$ 50 ksi

$\phi_{da} =$ 1

$\sigma_{dr} =$ 45 ksi

Required nominal geotechnical driving resistance

R_{dr} = Pile Factored Axial Demand / fdyn

Pile Factored Axial Demand = 450 kips

$\phi_{dyn} =$ 0.65

$R_{dr} =$ 692 kips

General Soil Profile (N.T.S)

South Abutment (BB-TNR-103)

Layer	El.
Top of Pile	274
Fill	270
Sand	266
Till	255
Decomp. Bedrock	253
Bedrock	

North Abutment (BB-TNR-105)

Layer	El.
Top of Pile	276
Till	255
Bedrock	

Note: Based on boring BB-TNR-108, top of bedrock is at approximate El. 255

Nobis
Engineering a Sustainable Future

Project No.:	92270.02		
Project Title:	North Parish Bridge - Turner, Maine		
Calculated by:	BTW	**Date:**	11.6.2017
Checked by:	RAC	**Date:**	11.6.2017

South Abutment - Pile Size = HP14 x 102

N.Parish Bridge-Turner-PRELIM-South Abut

Hammer Information

Select from following list [3/21/2012-2003]: ID: 41

ID	Name	Type	Ram Wt	Energy/Power
40	DELMAG D 19-32	OED	4.000	42.440
41	DELMAG D 19-42	OED	4.000	43.240
42	DELMAG D200-42	OED	44.090	492.044

Hammer parameters

Efficiency	0.8			
Pressure	1520	psi	Fixed	100 %
Stroke	10.81	ft	Variable	

Pile material: Concrete / Steel (selected) / Timber

Cushion Information

	Hammer	Pile	
Area	227.	0.	in^2
Elastic Modulus	530.	0.	ksi
Thickness	2.	0.	in
C.O.R.	0.8	0.	
Stiffness	0.	0.	kips/in
Helmet Weight	1.9		kips

Pile Information

Length	21.	ft	Auto	Segments
Penetration	21.	ft	Auto.	S-Length
Section Area	30.	in^2	Auto.	S-St. Wt
Elast Modulus	29000.	ksi	0	Splices
Spec Weight	493.356	lb/ft^3		
Toe Area	207.42	in^2	Pile Type:	
Perimeter	4.802	ft	H Pile	
Pile Size	14.78	in		

Ultimate Capacities (up to 10)
kips

1	450.0	6	692.0
2	500.0	7	750.0
3	550.0	8	800.0
4	600.0	9	850.0
5	650.0	10	900.0

Incr. 0 Action >>

Soil Parameters — 2nd Toe - No

Quake

Shaft	0.1	in	Const
Toe	0.04	in	

Damping

Shaft	0.05	s/ft	Const
Toe	0.15	s/ft	Smith

Shaft Resistance

Percentage 15 %

100%

Dist. Shape Num 0.0

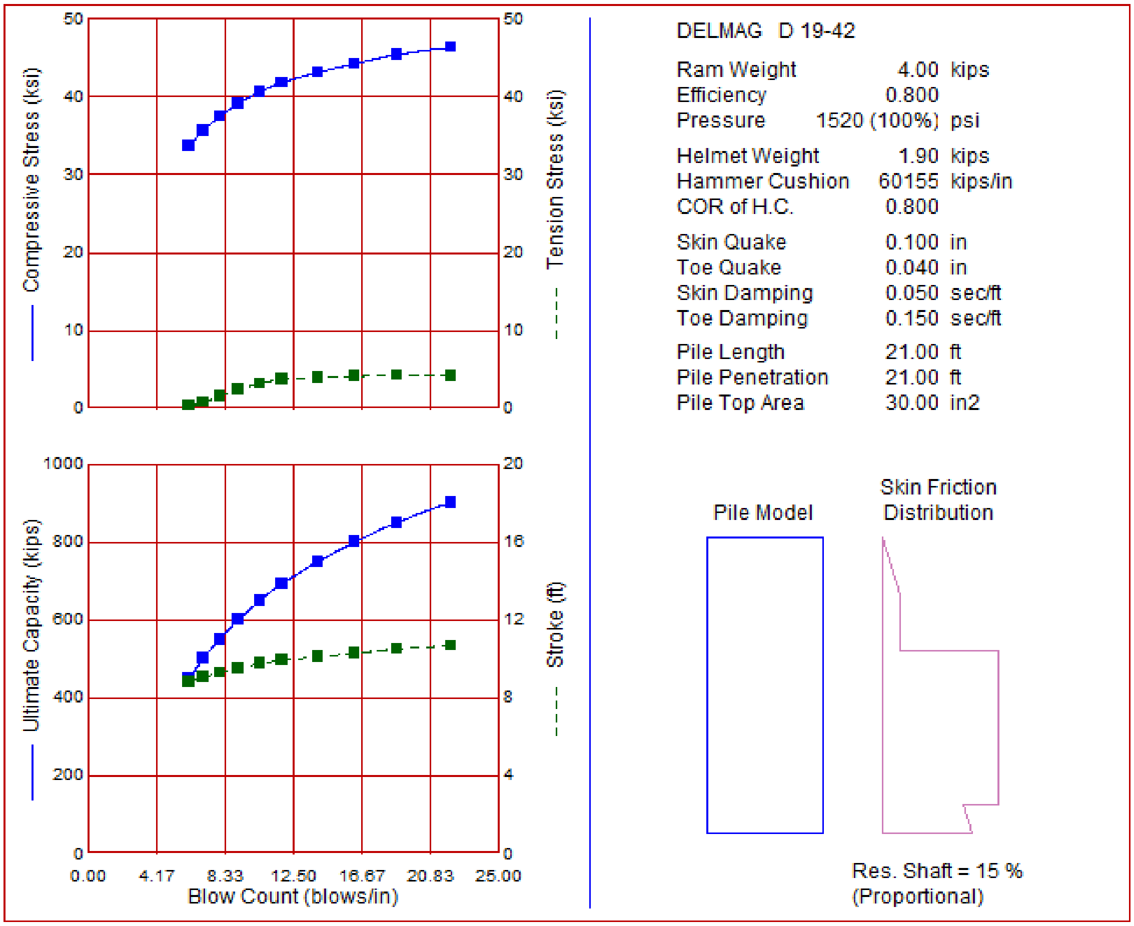
Residual Stress Analysis: No

Engineering a Sustainable Future

Project No.:	92270.02
Project Title:	North Parish Bridge - Turner, Maine
Calculated by:	BTW
Date:	11.6.2017
Checked by:	RAC
Date:	11.6.2017

South Abutment - Pile Size = HP14 x 102

Nobis Engineering, Inc. — 06-Nov-2017
N.Parish Bridge-Turner-PRELIM-South Abut — GRLWEAP Version 2010



Nobis Engineering, Inc. — 06-Nov-2017
N.Parish Bridge-Turner-PRELIM-South Abut — GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
450.0	33.73	0.36	6.1	8.83	15.61
500.0	35.63	0.78	7.0	9.05	15.79
550.0	37.44	1.53	8.0	9.29	15.97
600.0	39.12	2.44	9.1	9.52	16.36
650.0	40.68	3.18	10.5	9.76	16.83
692.0	41.82	3.72	11.8	9.92	17.19
750.0	43.14	3.99	14.0	10.10	17.56
800.0	44.23	4.12	16.2	10.29	17.95
850.0	45.40	4.29	18.8	10.51	18.41
900.0	46.34	4.19	22.1	10.68	18.77

Nobis
Engineering a Sustainable Future

Project No.: 92270.02
Project Title: North Parish Bridge - Turner, Maine
Calculated by: BTW **Date:** 11.6.2017
Checked by: RAC **Date:** 11.6.2017

North Abutment - Pile Size = HP14 x 102

N.Parish Bridge-Turner-PRELIM-North Abut

Hammer Information
Select from following list [3/21/2012-2003]: ID: 41

ID	Name	Type	Ram Wt	Energy/Power
40	DELMAG D 19-32	OED	4.0000	42.440
41	DELMAG D 19-42	OED	4.0000	43.240
42	DELMAG D200-42	OED	44.0900	492.044

Hammer parameters

Efficiency	0.8		
Pressure	1520	psi	Fixed 100 %
Stroke	10.81	ft	Variable

Pile material
○ Concrete ◉ Steel ○ Timber

Cushion Information

	Hammer	Pile	
Area	227.	0.	in^2
Elastic Modulus	530.	0.	ksi
Thickness	2.	0.	in
C.O.R.	0.8	0.	
Stiffness	0.	0.	kips/in
Helmet Weight	1.9		kips

Pile Information

Length	21.	ft	Auto	Segments
Penetration	21.	ft	Auto.	S-Length
Section Area	30.	in^2	Auto.	S-St. Wt
Elast Modulus	29000.	ksi	0	Splices
Spec Weight	493.356	lb/ft^3		
Toe Area	207.42	in^2	Pile Type:	
Perimeter	4.802	ft	H Pile	
Pile Size	14.78	in		

Ultimate Capacities (up to 10)
kips

1	450.0	6	692.0
2	500.0	7	750.0
3	550.0	8	800.0
4	600.0	9	850.0
5	650.0	10	900.0

Incr. 0 Action >>

Soil Parameters
2nd Toe - No

Quake

Shaft	0.1	in	Const
Toe	0.04	in	

Damping

Shaft	0.05	s/ft	Const
Toe	0.15	s/ft	Smith

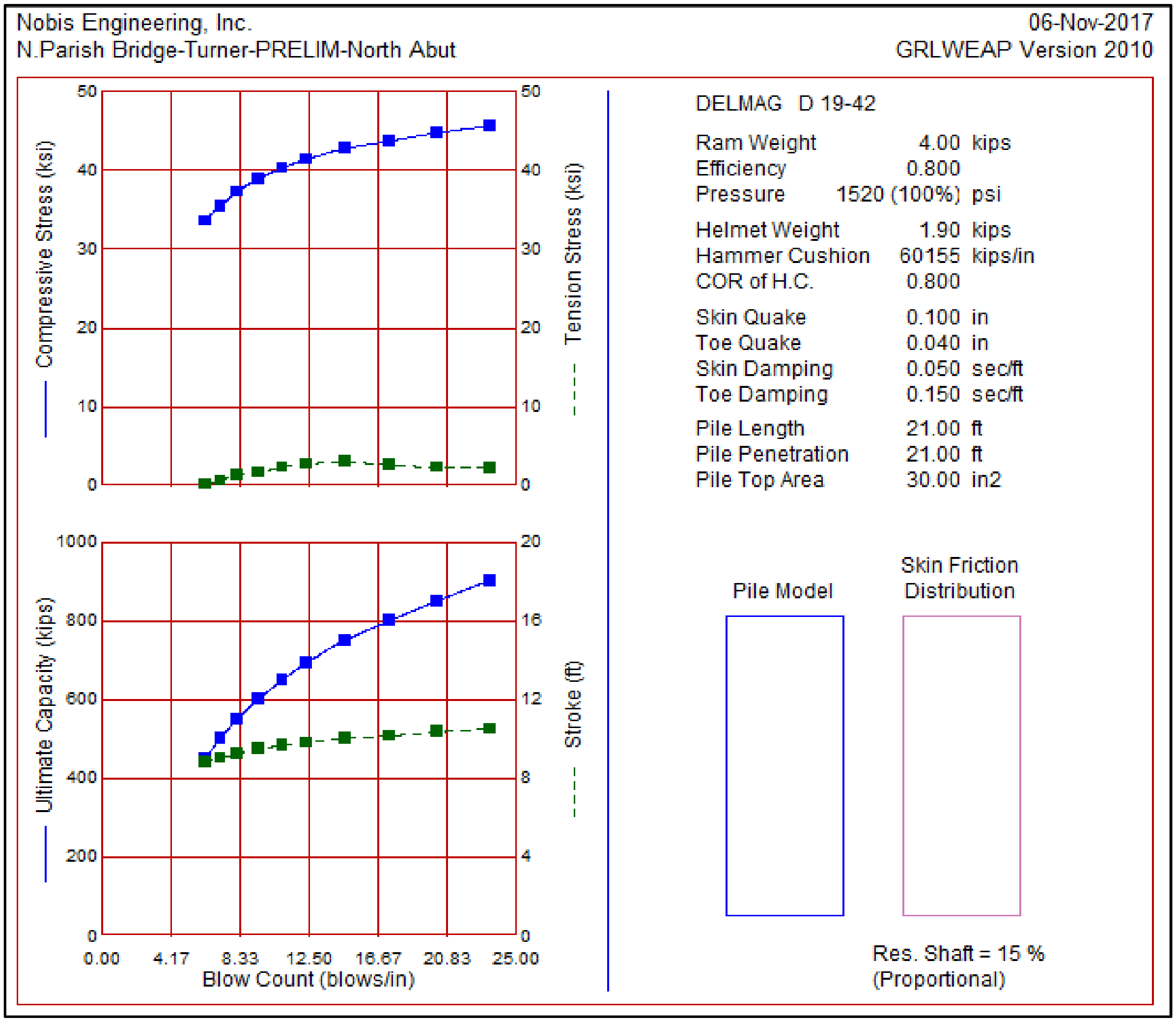
Shaft Resistance
Percentage 15 %
100%
Dist. Shape Num 0.0

Residual Stress Analysis: No

Nobis
Engineering a Sustainable Future

Project No.: 92270.02
Project Title: North Parish Bridge - Turner, Maine
Calculated by: BTW **Date:** 11.6.2017
Checked by: RAC **Date:** 11.6.2017

North Abutment - Pile Size = HP14 x 102



Nobis Engineering, Inc.
N.Parish Bridge-Turner-PRELIM-North Abut
06-Nov-2017
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
450.0	33.56	0.18	6.2	8.81	15.35
500.0	35.48	0.63	7.1	9.03	15.50
550.0	37.28	1.37	8.2	9.26	15.70
600.0	38.88	1.69	9.4	9.49	16.13
650.0	40.31	2.34	10.9	9.67	16.50
692.0	41.38	2.73	12.3	9.82	16.79
750.0	42.73	3.04	14.6	10.03	17.21
800.0	43.70	2.63	17.3	10.16	17.45
850.0	44.74	2.33	20.2	10.35	17.84
900.0	45.61	2.28	23.4	10.51	18.17