MaineDOT Soils Report 2020-27

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GEOTECHNICAL DESIGN REPORT - ROADWAY

ROUTE 26 IMPROVEMENTS
MAINEDOT WIN 018767.00
WOODSTOCK, MAINE

by Haley & Aldrich, Inc.
Portland, Maine

for Maine Department of Transportation
Augusta, Maine

File No. 130458-002
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Attention: Kate Maguire, P.E.
Senior Geotechnical Engineer

Subject: Geotechnical Design Report - Roadway
Route 26 Improvements
MaineDOT WIN 018767.00
Woodstock, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report entitled, "Geotechnical Design Report - Roadway, Route 26 Improvements, MaineDOT WIN 018767.00, Woodstock, Maine," prepared in accordance with our proposal, dated 18 July 2017 and executed by your Bradford Foley on 7 August 2017.

This Geotechnical Design Report (GDR) presents the results of preliminary design phase subsurface explorations and geotechnical evaluations, and provides geotechnical design recommendations that are specific to the roadway, slopes and retaining wall that are planned for the project. The work was completed by Haley & Aldrich, Inc. (Haley & Aldrich) in support of the Maine Department of Transportation's (MaineDOT's) development of the Plan Impacts Complete (PIC) package. Please note that geotechnical evaluations and design recommendations for the large diameter and/or box culverts that are planned for the project were previously provided in the report entitled "Geotechnical Design Report – Culverts," dated 20 August 2020 (Culvert Report).

It is our understanding that this GDR may be included as a reference document in the package of information (i.e., Contract Documents; CDs) that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the CDs and that the information contained in the CDs takes precedence over the information provided in this GDR.

Horizontal Coordinate System and Elevation Datum

Plan locations of test borings were determined in the field by MaineDOT using GPS survey equipment and were provided to Haley & Aldrich as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 West Zone. As-drilled test boring locations were related to station and offset distance/direction relative to the baseline stationing by MaineDOT. The baseline stationing for the project extends from approximately Sta. 11+00 (east; project beginning) to Sta. 147+50 (west; project end).

The project elevation datum and elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

Background

U.S. Route 26 (Route 26) carries traffic northbound and southbound through the Town of Woodstock, Maine, as shown in Figure 1, Project Locus.

In general, the project site is located along an approximate 2.6-mile long segment of Route 26 in Woodstock, Maine. This section of roadway was originally constructed in the 1930s or 1940s and is situated between two segments of roadway that were more recently reconstructed by MaineDOT. It is our understanding that MaineDOT will be rehabilitating the 2.6-mile long section of roadway, which includes the following key elements:

- Replacement of an existing, skewed (relative to the roadway) 4 ft x 3 ft box culvert with a 48-in. diameter, 84-ft long reinforced concrete pipe (RCP) culvert at Sta. 13+42 (approximate).
- Replacement of existing twin, skewed (relative to the roadway) 36-in. diameter corrugated metal pipe (CMP) culverts with a 6 ft x 10 ft x 126-ft long precast concrete box culvert at Sta. 24+93 (approximate).
- Replacement of an existing, skewed (relative to the roadway) 36-in. diameter RCP culvert with 60-in. diameter, 88-ft long RCP culvert at Sta. 123+85 (approximate).
- Modification to the existing horizontal and/or vertical profile and widening of the roadway between Sta. 52+00 and Sta. 61+50 (approximate). Along this portion of the alignment, the roadway is situated on an embankment that extends up to 25 ft (approximate) above the St. Lawrence and Atlantic Railroad (railroad), which is located near the toe of the embankment and immediately south of the roadway. In addition, steep mountainous terrain exists immediately northeast of the roadway and an existing 225-ft long (approximate) soil nail wall is present.
- Replacement of the existing soil nail retaining wall located approximately between Sta. 58+50 and Sta. 60+75 with a new retaining wall offset to the northeast in the same general area as the existing wall (to accommodate the new realigned/widened roadway).

As noted above, this GDR is specific to modifications to the roadway (i.e., fourth and fifth bullet items shown above). Geotechnical information and recommendations for the first three bullets were included in a separate report.

Multiple roadway rehabilitation options were under consideration between Sta. 52+00 and Sta. 61+50. Based on our review of drawings provided by MaineDOT on 3 May 2019, it is our understanding that the vertical profile will be raised by up to 2 ft and the horizontal alignment will be shifted slightly northeast, (away from the railroad) into the existing slope and existing retaining wall.

Geologic Setting

The Surficial Geology of the Bryant Pond Quadrangle (2008) indicate mapped surficial deposits along the project roadway alignment consist primarily of glacial till deposits typically shown at higher elevations. Ice contact deposits are also mapped within the project site, typically at lower elevations, consisting of sand and gravel glacial stream sediment and finer grained glacial lake sediments. Alluvial deposits are also mapped at lower elevations typically adjacent to existing streams, consisting primarily of fine-grained sediment. In low-lying areas minor wetland deposits are mapped, consisting of near surface silt and organic sediment. Regionally, within the glacial valleys several eskers are mapped consisting of sand and gravel glacial stream deposits. Refer to Figure 2 for the surficial geology map.

The Bedrock Geology of the Bryant Pond Quadrangle (1965) inidicate bedrock within the site project alignment and surrounding area is mapped as Songo granodiorite. Regionally, the Songo granodiorite comprises an igneous pluton. Pegmatites and quartz monzonite are commonly mapped along the contact fringes of the pluton. Songo granodiorite is Middle to Upper Devonian in age. Refer to Figure 3 for the bedrock geology map.

Subsurface Exploration Program

PREVIOUS EXPLORATIONS BY OTHERS

Previous explorations were conducted at the site by MaineDOT in support of construction of the existing soil nail wall. Three test borings, designated HB-WOOD-101, HB-WOOD-101A, and HB-WOOD-102, were drilled on 8 December 2011 using solid stem augers. The test boring locations are shown on Figures 4 and 5, and test boring logs included in Appendix A.

PRELIMINARY DESIGN PHASE EXPLORATIONS BY HALEY & ALDRICH

Haley & Aldrich completed a preliminary design phase subsurface exploration program at the site in August and September 2017. A total of 16 test borings, designated HB-WOOD-201 through BB-WOOD-216, and eight test probes, designated P1 through P8, were drilled along the proposed alignment. Only ten of the test borings (HB-WOOD-205 through HB-WOOD-214) and the test probes that were drilled for the roadway are included in this GDR and are discussed herein. The remaining six test borings were drilled for the proposed drainage improvements and were included in the Culvert Report.

The test boring locations were laid out in the field by Haley & Aldrich prior to the start of drilling. "As-drilled" exploration locations and ground surface elevations at test boring locations were determined in the field by MaineDOT upon the completion of drilling using GPS survey equipment. The coordinate location and station/offset information provided by MaineDOT for each test boring/probe is shown on Table I. Please note that only station/offset information is provided on the test borings logs included in Appendix A. The plan locations of the test borings/probes are shown on Figures 4 and 5, Site and Subsurface Exploration Location Plan.

A Haley & Aldrich geotechnical engineer monitored the drilling, logged and conducted visual inspection/classification of the soil and rock samples collected, prepared test boring logs documenting the conditions encountered, and confirmed that all drilling and sampling was performed in accordance with MaineDOT requirements.

The test borings were drilled by New England Boring Contractors of Hermon, Maine using a truck-mounted Mobile Drill B-59 drill rig along the roadway and a track-mounted Bombardier Mobile Drill B-53 drill rig along the toe of the embankment adjacent to the railroad siding. The test probes were drilled using a trailer-mounted Mobile B-47 drill rig. Test borings were advanced into or through the naturally-deposited overburden soils to depths ranging from approximately 10 to 40 ft below existing ground surface (BGS) using either 3-in. (NW-size) or 4-in. (HW-size) outside diameter (OD) steel casing. Please note that the upper portion of each test boring was drilled with a solid-stem auger prior to setting the steel casing, as shown on the test borings logs in Appendix A. Once the casing was set, the holes were advanced using cased-wash drilling methods. Test probes were advanced to “refusal” using solid stem augers. No soil or rock sampling was conducted during advancement of the test probes.

Soil samples were generally collected continuously through the fill soils and at standard, 5-ft intervals thereafter, by driving a 1-3/8-in. inside diameter (ID) split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. Drilling and sampling were performed in accordance with MaineDOT specifications. The drill rigs were equipped with automatic hammers calibrated annually per MaineDOT requirements (see Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). Calculated hammer efficiencies of 0.869 and 0.75 were used for the calibrated automatic hammer system for the truck-mounted Mobile Drill B-59 drill rig and the track-mounted Bombardier Mobile Drill B-53 drill rig, respectively.

The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value (N-uncorrected) is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-values (N_{60}) shown on the test boring logs are equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.869 or 0.75) divided by 0.6 (i.e., 60 percent calculated hammer efficiency). Both the raw blow count data and the corrected N-values are shown on the boring logs.

A photoionization detector (PID) was used in the field to screen for the presence of volatile organic compounds (VOCs) in the recovered overburden soil samples. No elevated PID readings were detected in the test borings except one reading of 11.0 ppm immediately below the bituminous concrete in HB-WOOD-213. PID readings are provided as part of the individual soil sample descriptions shown on the test boring logs provided in Appendix A.

Each test boring, except HB-WOOD-207, was advanced approximately 4.8 to 5.0 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel.

Bedrock and soil samples were classified in the field using the 'Field Guide for Description of Rock Core', the 'Soil Description Field Guide' and the Unified Soil Classification System (USCS) flow charts for classifications of soils described through texture and grain size of a soil. The field forms used for classification of bedrock and soil samples can be found in Appendix A. All soil and bedrock samples were collected and preserved in glass jars and wooden boxes, respectively, and are available for review upon request. The soil and bedrock samples (i.e., the samples that were not submitted for laboratory testing) are currently being stored at the Haley & Aldrich laboratory facility in Portland, Maine.

Generalized Subsurface Conditions

The subsurface conditions encountered in the test borings generally consisted of the following geologic units presented in order of increasing depth below ground surface: man-placed fill soils overlying naturally-deposited glaciofluvial deposits, glacial lacustrine deposits, glacial till and bedrock.

Refer to Table II for a summary of the soil units and thicknesses encountered in each test boring. Refer to Table III for a summary of the refusal depths/elevations of the test probes. Detailed soil and bedrock descriptions are provided on the Haley & Aldrich test boring logs included in Appendix A. Working cross sections at Sta. 57+00, Sta. 59+00, Sta. 59+50, Sta. 60+00 and Sta. 60+50 are included in Appendix C. Please note that the soil descriptions provided on the test boring logs and summarized below do not represent actual field conditions other than at the specific test boring locations. Actual conditions may vary from those described and shown herein and may not become apparent until construction begins.

The generalized subsurface conditions present at the site are presented below.

Soil Unit	Approximate Range in Encountered Thickness (ft)	Generalized Description
Bituminous Concrete	0.3 to 1.0	surficial layer of bituminous concrete (asphalt pavement) *(encountered in roadway test borings)*
Fill	1.3 to 11.1	loose to very dense, fine to coarse SAND with variable amounts of silt and gravel; very loose to very dense fine SAND with variable amounts of silt, medium to coarse sand and gravel; loose to medium dense GRAVEL with variable amounts of silt and sand *(encountered in each test boring)*
Glaciofluvial Deposit	2.7 to 6.0	loose to medium dense fine SAND with variable amounts of silt, medium to coarse sand and gravel; loose fine to coarse SAND with trace fine gravel (*encountered in test borings HB-WOOD-206 and HB-WOOD-214*)
Glacial Lacustrine Deposit	9.5	loose fine SAND with little to no silt (*encountered in test boring HB-WOOD-214*)
Glacial Till	0.9 to 24.3	very dense fine to coarse SAND with variable amounts of silt and gravel; medium dense to very dense fine SAND with variable amounts of silt, medium to coarse sand and gravel; medium dense to very dense Gravelly SAND with trace silt; dense GRAVEL with little fine sand and trace medium sand; deposit is very loosely to well bonded with frequent coarse gravel, cobbles and boulders; recovered soil samples regularly contained crushed coarse gravel

		(encountered in each test boring)
Bedrock	top of bedrock surface encountered at depths ranging from 2.9 to 33.6 ft BGS (El. 678.2 to El. 722.7) in all test borings except HB-WOOD-207. Refusal was encountered at depths ranging from 1 to 11.9 ft BGS (El. 720.3 to El. 733.4) in test probes. Bedrock generally sloped downwards from northeast to southwest, towards Bryant Pond. Bedrock encountered at the site consisted of very hard GRANODIORITE. GRANOFELS was encountered in HB-WOOD-208 only.	

BEDROCK CONDITIONS

As stated previously, approximately 5 ft of bedrock was sampled in each test boring except HB-WOOD-207. The sampled and recovered bedrock generally consisted of the following:

- Very hard, fresh to slightly weathered, fine to coarse grained GRANODIORITE of the Songo Granodiorite Formation. Joints dip at low to steep angles and are very close to widely spaced. Joints are tight to open with occasional silty clay infilling. Joint surfaces are weathered, discolored, planar to undulating, and smooth to rough.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for bedrock encountered at the site ranged from 67 to 100 percent indicating good to excellent rock quality.

Photographs of the sampled bedrock are provided for reference in Appendix A.

GROUNDWATER CONDITIONS

Observation wells were not installed in any of the completed boreholes. As a result, long term static water levels at test boring locations were not determined. Water levels were measured in the boreholes after completion of drilling and sampling and are noted on the boring logs in Appendix A. Where observed, the water levels ranged from approximately 6 to 10 ft below ground surface (BGS) (El. 709.3 to El. 728.0) in borings in the roadway and approximately 3 to 18 ft BGS (El. 693.8 to El. 709.6) in borings at the toe of slope southwest of the roadway. Please note that water levels measured in the completed boreholes are likely impacted by drilling means/methods and may not be representative of actual static water levels at the site.

In general, groundwater levels can be expected to fluctuate, subject to test boring drilling means/methods, seasonal variation, local soil conditions, topography and precipitation. Groundwater levels encountered during construction may differ from those measured in the test borings.

Laboratory Testing Program

A preliminary laboratory testing program was undertaken on soil samples collected during the field investigation to assist in soil classification/identification. In general, laboratory testing was performed on disturbed soil samples collected during SPT sampling. All laboratory soil testing was performed by the State of Maine, Department of Transportation Laboratory in Bangor, Maine. Geotechnical laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures.

The preliminary phase testing program included 18 grain size analyses with natural water content (sieve only, no hydrometer). A summary of laboratory test results is provided below.

Laboratory Test	ASTM Test Designation	Soil Unit	No. of Tests Completed	Range in Test Results
Grain Size	ASTM D 422 (Sieve Only)	Fill	16	AASHTO Classification: A-1-a, A-1-b, A-2-4, A-4 USCS Classification: SM, SW-SM Percent Passing No. 200 Sieve: 9% to 45% (average 19%)
		Glacial Till	2	AASHTO Classification: A-2-4, A-1-b USCS Classification: SM Percent Passing No. 200 Sieve: 15% to 24% (average 19%)

Notes:
1. Refer to the Key to Soil and Rock Descriptions and Terms in Appendix A for USCS definitions.

Laboratory test results are provided in Appendix B and individual test results have been included on the test boring reports in Appendix A.

Geotechnical Design Recommendations

Technical evaluations used as the basis for development of geotechnical design recommendations were coordinated with MaineDOT. Engineering calculations that support the recommendations outlined in this section are provided for reference in Appendix D.

SEISMIC SITE CLASS

Due to the nature and thickness of the overburden soils encountered in the test borings, we recommend the site in the vicinity of the retaining wall be considered "Site Class C." Based on the site location and the assignment of "Site Class C," the "USGS Seismic Design Maps Web Services" provided the recommended AASHTO response spectra for a 7 percent probability of exceedance in 75 years as summarized below.

Parameter	Design Value
Mapped Peak Ground Acceleration, PGA =	0.091
Mapped Short (0.2-second) Period, S_S =	0.183
Mapped Long (1.0-second) Period, S_1 =	0.049
Site Factor, Fpga =	1.2
Site Factor, Fa =	1.2
Site Factor, Fv =	1.7
Acceleration Coefficient, As (0 second period) =	0.109
Horizontal Response Spectral Acceleration, S_{DS} (0.2 second period) =	0.220
Horizontal Response Spectral Acceleration, S_{D1} (1 second period) =	0.084
Seismic Performance Zone =	1

We recommend the new retaining wall be design using the above seismic parameters.

ANTICIPATED SUBGRADE CONDITIONS NEW PAVEMENT SECTION

The following table summarizes the anticipated pavement section subgrade conditions at each test boring performed in the roadway:

Test Boring No.	Station	Anticipated deposit at bottom of subgrade	USCS Symbol	Description of soil at subgrade	Percent passing No. 200 sieve at bottom of subgrade[1]
HB-WOOD-205	53+07.2	Fill	SM	Dense fine to coarse SAND, little silt and gravel	19
HB-WOOD-207	54+93.5	Fill	SM	Very dense, fine to coarse SAND, some gravel, little silt	12
HB-WOOD-209	56+87.6	Fill	SM	Very dense, Gravelly fine to medium SAND, little silt, trace coarse sand	15
HB-WOOD-211	58+81.0	Fill	SM	Dense, fine to medium SAND, some gravel, little silt, trace coarse sand	15
HB-WOOD-213	60+77.4	Fill	SW	Very dense, Gravelly fine to coarse SAND, trace silt	9

[1] Percent passing No. 200 sieve data based on laboratory test data.

EMBANKMENT SETTLEMENT

Minimal (less than 2 ft) raises-in-grade are planned along the roadway alignment. Given the proposed raises-in-grade, and the nature and consistency of the fill and naturally-deposited soils that are present at the site, we anticipate that the resulting magnitude of settlement will be minimal, will result from elastic compression of in-situ soils, and will likely occur rapidly during construction, soon after new embankment fills are placed and prior to final paving. We anticipate post-construction settlement to be negligible (i.e., < ½ in.). Refer to Appendix D for calculations. Based on subsurface conditions at the site, we do not anticipate that long-term, consolidation settlements of the in-situ soils will occur as a result of the planned-raises-in-grade.

EMBANKMENT STABILITY

Computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 8.0 by Rocscience Inc. to evaluate global stability of the embankments northeast and southwest of the road. Evaluations were performed for both existing and proposed conditions at Sta. 57+00 and Sta. 60+00. These locations were selected to be representative of a steep “downhill” slope condition (Sta. 57+00) and a tall “uphill” wall condition (Sta. 60+00). A live load surcharge of 250 psf was included in our calculations within the roadway limits to model traffic loading. Both static and pseudo-static slope stability evaluations were conducted.

Soil and rock material and strength properties used in the global stability evaluations were based on the results of laboratory testing and our experience, and are summarized below. Based on observations at the site, a layer of riprap is present on the existing “downhill” slope. Based on historic drawings, this riprap layer is anticipated to be on the order of 4 ft thick. The riprap was included in the global stability evaluations.

The following physical and strength properties of the soil strata at the site were used to evaluate static and pseudo-static global stability.

	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
RipRap	140	45	0
Existing Fill	125	34	0
Glaciofluvial Deposits	120	32	0
Glacial Lacustrine Deposits	115	30	0
Glacial Till	130	38	0

<u>Sta. 57+00 Downslope (southwest of Route 26)</u> – The minimum calculated static factor of safety of both the existing and proposed slope is 1.2. The minimum calculated factor of safety under pseudo-static earthquake loading from our evaluations for both the existing and proposed slope is 1.1, using a horizontal coefficient of 0.055 (i.e., one half of the peak ground acceleration coefficient, As). Values ranging from As/3 to As/2 are recommended in literature (Melo and Sharma, 2004); the reduction from As is due to soil slope flexibility and the fact that the peak ground acceleration during an earthquake lasts only for a very short period of time.

<u>Sta. 60+00 Downslope (southwest of Route 26)</u> – The minimum calculated static factor of safety of the existing slope is 1.4 and of the proposed slope is 1.5. The minimum calculated factor of safety under pseudo-static earthquake loading from our evaluations for both the existing and proposed slope is 1.3.

<u>Sta. 60+00 Upslope (northeast of Route 26)</u> – The minimum calculated static factor of safety of the existing slope is 1.4 and of the proposed slope is 1.1 (for the case of a retaining wall, but not accounting

for any wall reinforcement). The minimum calculated factor of safety under pseudo-static earthquake loading from our evaluations is 1.3 for the existing slope and 1.0 for the proposed slope (for the case of a retaining wall, but not accounting for any wall reinforcement).

The minimum factor of safety required for static stability evaluations is 1.3 where the slope does not support or contain a structural element, based on the requirements of LRFD Article 11.6.2.3. The minimum factor of safety required for pseudo-static stability evaluations is 1.0 based on the requirements of LRFD.

At Sta. 57+00, the factors of safety of both the existing and proposed embankment are less than required. The change in geometry for the proposed condition does not impact the minimum factor of safety. At Sta. 60+00, for the existing slope, the factors of safety for both the static case and under pseudostatic earthquake loading exceed the minimum required. However, the factors of safety for the proposed uphill slope are less than required. The proposed soil nails (if a soil nail wall type is selected for the new retaining wall) or other wall reinforcement will need to be designed to increase the factors of safety to meet the minimum required. Refer to Appendix D for calculations.

EXISTING SOIL NAIL WALL

An existing soil nail wall is present on the northeast side of the road between approx. Sta. 58+50 and Sta. 60+75 (see Figures 4 and 5). The soil nail wall was designed by Earthwork Engineering, Inc. under contract with Thomas Drilling and Blasting. The wall was constructed in 2013 by Thomas Drilling and Blasting based on the soil nail wall design submittal prepared by Earthwork Engineering, Inc. Based on the soil nail wall design submittal (included in Appendix E), the wall consists of the following features:

- One row of nails located 2 ft from the top of the retaining wall.
- A second row of nails at El. 737.5 where the wall is more than approximately 5 ft tall.
- Horizontal nail spacing of 5 ft.
- Nails consist of 40/20 hollow bars installed at a 15 degree downward angle from the horizontal.
- Drawings indicate that nails were installed 5 ft into bedrock. The soil nail wall design calculations analyzed 15 to 20-ft long nails. Documentation confirming soil nail lengths was not available at the time this report was prepared.

We understand that MaineDOT has reviewed their records and do not have construction logs or as-built information for the soil nail wall. Haley & Aldrich contacted Thomas Drilling & Blasting to request that they provide any as-built information they had in their files. Thomas Drilling & Blasting reviewed their records but did not find any as-built information for this wall.

Working cross sections showing the approximate location and length of the existing soil nails based on the design submittal, are included in Appendix C. As shown on these sections, the face of the existing soil nail wall and portions of the existing nails will need to be cut and/or removed to construct the relocated roadway section and new retaining wall. We anticipate that the actual, “as-constructed” location, length and inclination of the soil nails will differ from the details and geometry shown in the design submittal and the sketches included in Appendix C.

DRAINAGE

Provisions to provide permanent drainage and to control both subsurface and surface water are recommended to ensure a drained condition is present both behind the new retaining wall and below the new pavement section in this area. Therefore we recommend the following: 1) use of underdrains to collect water within the roadway section; 2) use of a drainage swale at the top of the soil nail wall to collect runoff from the slope above and divert it away from the wall and roadway.; and 3) use of a prefabricated vertical drainage board installed between the excavated soil and shotcrete facing.

FINAL DESIGN PHASE SUBSURFACE EXPLORATIONS AND EVALUATIONS

No subsurface explorations have been completed outside the limits and northeast (uphill) of the existing roadway where the proposed retaining wall is planned to be constructed.

We strongly recommend that additional test borings be completed between Sta. 52+00 and Sta. 61+50 to characterize and quantify cut materials and determine design requirements for temporary and permanent cut slopes and/or retaining walls. Additional explorations will require access to steep/mountainous terrain and abutting properties. Depending on ability to access the site, geophysical investigations may also be warranted in this area in addition to test borings.

Construction Considerations

The primary purpose of this section is to comment on geotechnical aspects of proposed construction. This section is written primarily for the individuals having responsibility for preparation of geotechnical related plans and specifications as well as personnel appointed to monitor construction activities.

Prospective contractors should evaluate the potential for construction problems on the basis of their own knowledge and experience in the Woodstock, Maine area, and on the basis of similar projects in other localities, taking into account their proposed construction methods, procedures, equipment and personnel. Please note that the construction considerations provided below relate to the subject project only.

EXCAVATION

Excavation will be required to shift the road to the northeast between approximately Sta. 52+00 and Sta. 61+50 and create a stable slope and/or construct a new retaining wall. Based on the proposed and existing grades shown on cross sections provided on 3 May 2019, excavation depths up to 15 ft BGS will be required. The subsurface conditions in the vicinity of this excavation are not known. We anticipate glacial till will be encountered at or near the ground surface, but it is not known whether bedrock is present within the excavation depth/limits. If rock is anticipated or confirmed by supplemental test borings, rock excavation using mechanical and/or controlled blasting will be required to construct the new retaining wall. We recommend that the Contractor be made responsible for the design, stability and safety of all excavations in accordance with local, state and federal regulations.

DEWATERING

Limited information on groundwater is available at this time, particularly to the northeast of the roadway where the significant excavations will be located. Given that the excavation will be into the side of a slope that continues farther up to the northeast, we anticipate some dewatering will be required. Furthermore, we anticipate that construction dewatering could likely be accomplished using of localized sumps/pumps and discharging to a nearby stormwater system. We recommend the Contractor be made responsible for controlling all infiltration from groundwater and surface runoff to permit excavation, fill placement and construction in the dry.

Excavation and control of water should be done by methods that prevent disturbance to roadway subgrade soils. Sumps and pumps should be designed with proper filters to control the loss of fine grained soil.

Dewatering and discharge of dewatering effluent should be performed in accordance with all applicable local, state and federal regulations. Dewatering discharge should be recharged on site if possible. If on-site recharge is not feasible, dewatering discharge will likely need to be directed to the local storm drain system. Sedimentation tanks, filtration systems, and/or other treatment methods may be required for legal disposal of the effluent into the storm drain system.

CONSTRUCTION AT EXISTING SOIL NAIL WALL

The existing soil nail wall between approx. Sta. 58+50 and Sta. 60+75 is located within the area that will be excavated to shift the roadway to the northeast. As shown on the working sections in Appendix C and described above, the face of the existing soil nail wall and portions of the existing nails will need to be cut and/or removed to construct the relocated roadway section. If a soil nail wall is selected to support the excavation northeast of the roadway, the soil nail wall design and construction will need to account for the presence of the existing soil nail elements.

REUSE OF EXCAVATED ON-SITE SOILS

Based on the current roadway realignment approach, the overall project is expected to be exporting excavated materials. Therefore, reuse of excavated soils generated from construction activities may not be required or applicable.

Based on the test borings drilled at the site we anticipate that the excavated material will consist primarily of in-situ fill (in existing roadway section) and potentially glacial soils (in the vicinity of the cut area between Sta. 52+00 and Sta. 61+50). As described above, the conditions in the cut area are unknown.

Based on gradation testing, the fines content (percent passing the No. 200 sieve) of the existing fill ranged from 9 to 45 percent, with an average of 19 percent. The fines content of the limited number of glacial till samples tested ranged from 15 to 24 percent, with an average of 19 percent. It may be possible to reuse portions of both of these materials as embankment fill. However reuse will be dependent on weather, moisture content, and nature of fines (i.e., clay vs silt). It is likely that the zones with higher fines content (greater than 20 percent) will be difficult to place and compact in wet weather.

Limitations

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

Use of this report by any person or entity, including by MaineDOT, for a purpose other than relative to the subject project is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the report is adequate for such other use. Use of this report by any other person or entity for such other purpose without written authorization by Haley & Aldrich shall be at such person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

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

It is our understanding that this report may be included as a reference document in the documents that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the documents and that the information contained in the documents takes precedence over the information provided in this report.

Closure

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

Sincerely yours,
HALEY & ALDRICH, INC.

Kevin A. Russ, P.E.
Geotechnical Engineer

Erin A. Force, P.E.
Project Manager

STATE OF MAINE
ERIN A. FORCE
No. 12207
LICENSED
PROFESSIONAL ENGINEER

Wayne A. Chadbourne, P.E.
Senior Associate

Enclosures:

- Table I – Roadway Subsurface Exploration Location Data
- Table II – Roadway Subsurface Exploration Subsurface Data – Test Borings
- Table III – Roadway Subsurface Exploration Subsurface Data – Test Probes
- Figure 1 – Project Locus
- Figure 2 – Surficial Geology Map
- Figure 3 – Bedrock Geology Map
- Figure 4 – Site and Subsurface Exploration Location Plan, Sta. 52+00 to Sta. 59+00
- Figure 5 – Site and Subsurface Exploration Location Plan, Sta. 59+00 to Sta. 66+00
- Appendix A – Test Boring Logs
- Appendix B – Laboratory Test Results
- Appendix C – Working Cross Sections
- Appendix D – Geotechnical Calculations
- Appendix E – Existing Soil Nail Wall Design Submittal and Drawings

G:\PROJECTS\130458 - Woodstock\Deliverables\Roadway Report\2020-0820-HAI-18767 Woodstock Roadway GT Design Report-F MEDOT COMMENTS ADDRESSED.docx

References

Guidotti, Charles V., Geologic Map of the Bryant Pond Quadrangle, Maine, Quadrangle Mapping Series No. 3, Maine Geological Survey, Augusta, Maine, 1965.

Thompson, Woodrow B., Bryant Pond Quadrangle, Maine, Open File Report No. 08-80, Maine Geological Survey, Augusta, Maine, 2008.

TABLES

TABLE I
Roadway Subsurface Exploration Location Data
Route 26 Highway Rehabilitation
MaineDOT WIN 018767.00
Woodstock, Maine

Haley & Aldrich, Inc. File No.: 130458-002

Test Boring No.[1]	Ground Surface Elevation[3,4]	Station[5]	Offset Distance & Direction[5]	Coordinates[2]	
				Northing	Easting
HB-WOOD-205	719.0	53+07.2	2.5 RT	564,447.95	912,294.04
HB-WOOD-206	710.8	53+05.2	47.4 LT	564,414.82	912,256.64
HB-WOOD-207	726.8	54+93.5	16.2 LT	564,580.09	912,161.82
HB-WOOD-208	711.8	54+92.6	50.8 LT	564,557.11	912,135.91
HB-WOOD-209	733.6	56+87.6	1.9 LT	564,732.73	912,042.19
HB-WOOD-210	712.5	56+94.9	51.8 LT	564,702.86	912,001.51
HB-WOOD-211	736.4	58+81.0	15.8 LT	564,859.92	911,895.92
HB-WOOD-212	712.6	58+86.5	61.8 LT	564,831.30	911,859.39
HB-WOOD-213	735.6	60+77.4	3.0 RT	565,013.21	911,771.62
HB-WOOD-214	712.1	60+77.3	80.9 LT	564,955.95	911,710.33
P1	721.3	53+60.4	12.1 LT	564,479.97	912,249.20
P2	724.6	54+24.2	0.6 LT	564,536.70	912,217.75
P3	730.2	55+66.2	1.6 LT	564,644.28	912,125.29
P4	731.7	56+30.8	17.8 LT	564,680.89	912,069.86
P5	734.6	57+52.2	14.8 LT	564,769.37	911,987.49
P6	735.8	58+23.0	0.3 LT	564,829.78	911,947.77
P7	736.3	59+48.2	0.2 RT	564,918.77	911,859.76
P8	736.2	60+18.5	14.8 LT	564,958.17	911,799.51

Notes:
1 Test boring and test probe locations are shown on Figures 4 and 5, Site and Subsurface Exploration Location Plans.
2 As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet and reference NAD83, Maine 2000 West Zone coordinate system.
3 Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.
4 Elevations are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
5 Station and offset information determined by MaineDOT and provided to Haley & Aldrich.

	Individual	Date
Prepared By:	KAR	12/12/2017
Checked By:	EAF	5/2/2019
Reviewed By:	WAC	5/10/2019

TABLE II
Roadway Subsurface Exploration Subsurface Data - Test Borings
Route 26 Highway Rehabilitation
MaineDOT WIN 018767.00
Woodstock, Maine

Haley & Aldrich, Inc. File No.: 130458-002

Test Boring No.[1]	Ground Surface Elevation[2,3]	Approximate Strata Thickness[4] (ft)					Approximate Top of Bedrock Depth (ft)[4]	Approximate Elevation of Top of Bedrock[2,3,4]	Approximate Bottom of Exploration Depth (ft)	Approximate Elevation of Bottom of Exploration[2,3]
		Bituminous Concrete	Fill	Glaciofluvial Deposit	Glacial Lacustrine Deposit	Glacial Till				
HB-WOOD-205	719.0	0.7	8.3	NE	NE	5.8	14.8	704.2	20.0	699.0
HB-WOOD-206	710.8	NE	1.3	2.7	NE	7.3	11.3	699.5	17.3	693.5
HB-WOOD-207	726.8	1.0	8.6	NE	NE	> 21.4	NE	NE	31.0	695.8
HB-WOOD-208	711.8	NE	9.3	NE	NE	24.3	33.6	678.2	40.0	671.8
HB-WOOD-209	733.6	0.3	6.7	NE	NE	12.1	19.1	714.5	24.1	709.5
HB-WOOD-210	712.5	NE	3.5	NE	NE	14.5	18.0	694.5	23.3	689.2
HB-WOOD-211	736.4	0.6	11.1	NE	NE	2.0	13.7	722.7	19.8	716.6
HB-WOOD-212	712.6	NE	2.0	NE	NE	0.9	2.9	709.7	10.0	702.6
HB-WOOD-213	735.6	0.5	4.2	NE	NE	8.4	13.1	722.5	19.5	716.1
HB-WOOD-214	712.1	NE	4.0	6.0	9.5	8.6	28.1	684.0	33.5	678.6

Notes:
[1] Test boring and test probe locations are shown on Figures 4 and 5, Site and Subsurface Exploration Location Plans.
[2] Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.
[3] Elevations are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[4] "NE" indicates stratum was not encountered in test boring.

	Individual	Date
Prepared By:	KAR	12/12/2017
Checked By:	EAF	5/2/2019
Reviewed By:	WAC	5/10/2019

TABLE III
Roadway Subsurface Exploration Subsurface Data - Test Probes
Route 26 Highway Rehabilitation
MaineDOT WIN 018767.00
Woodstock, Maine

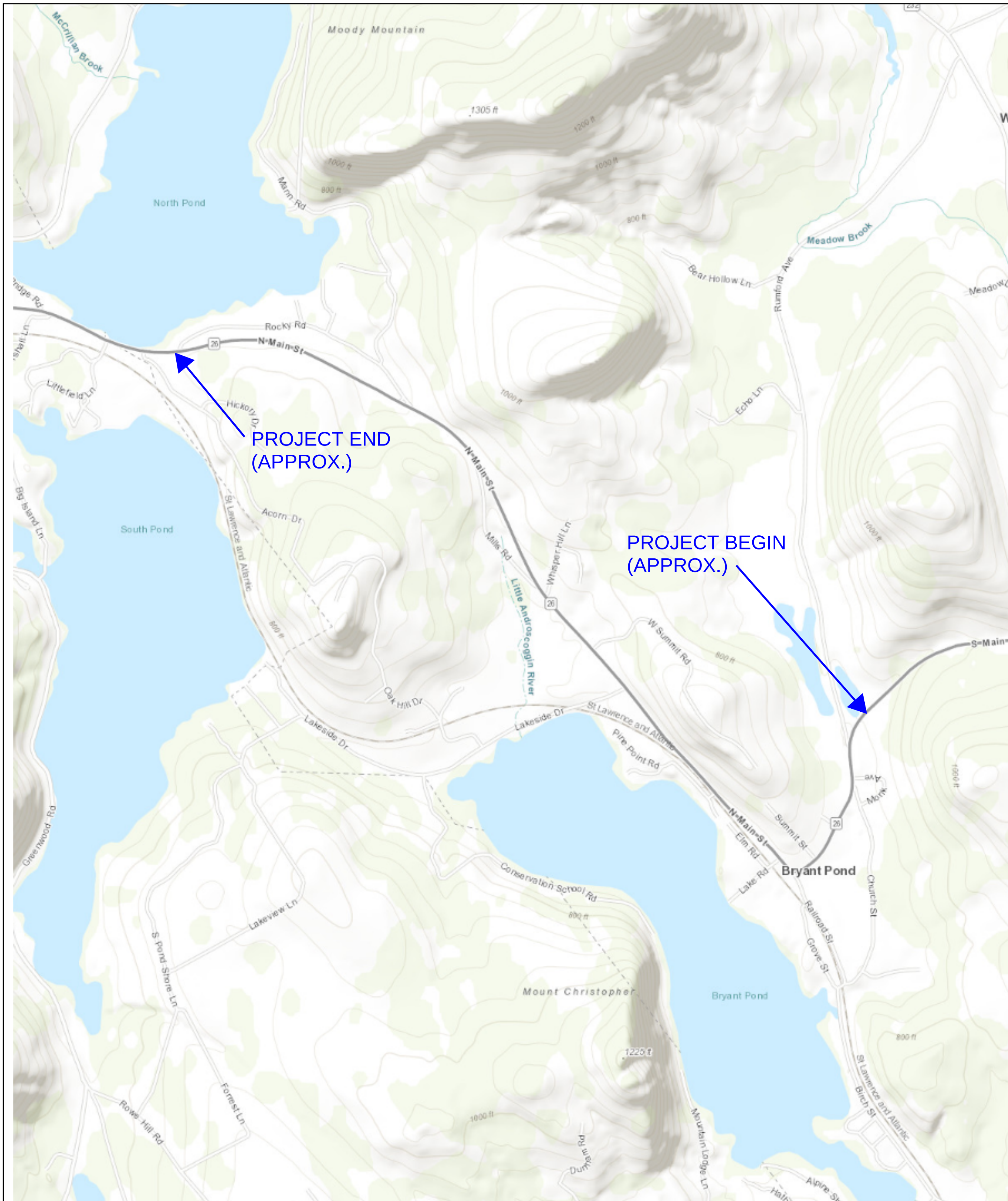
Haley & Aldrich, Inc. File No.: 130458-002

Test Probe No. [1]	Ground Surface Elevation [2,3]	Refusal Depth [4] (ft)	Refusal Elevation[2,3,4]	Comments and Observations[5]
P1	721.3	1.0	720.3	granular soil observed to 1.0 ft
P2	724.6	3.9	720.7	granular soil observed to 3.9 ft
P3	730.2	1.5	728.7	granular soil observed to 1.5 ft
P4	731.7	9.7	722.0	granular soil observed to 9.7 ft
P5	734.6	6.4	728.2	granular soil observed to 6.4 ft
P6	735.8	6.6	729.2	granular soil observed to 6.6 ft
P7	736.3	2.9	733.4	granular soil observed to 2.9 ft
P8	736.2	11.9	724.3	granular soil observed to 11.9 ft

Notes:
[1] Test boring and test probe locations are shown on Figures 4 and 5, Site and Subsurface Exploration Location Plans.
[2] Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.
[3] Elevations are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[4] Practicable test probe refusal depth shown is approximate as judged by Haley & Aldrich field representative.
[5] Material type based on observation of auger cuttings.

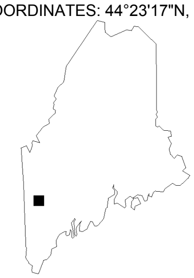
	Individual	Date
Prepared By:	KAR	12/12/2017
Checked By:	EAF	5/2/2019
Reviewed By:	WAC	5/10/2019

FIGURES



MAP SOURCE: ESRI

SITE COORDINATES: 44°23'17"N, 70°39'37"W



HALEY ALDRICH

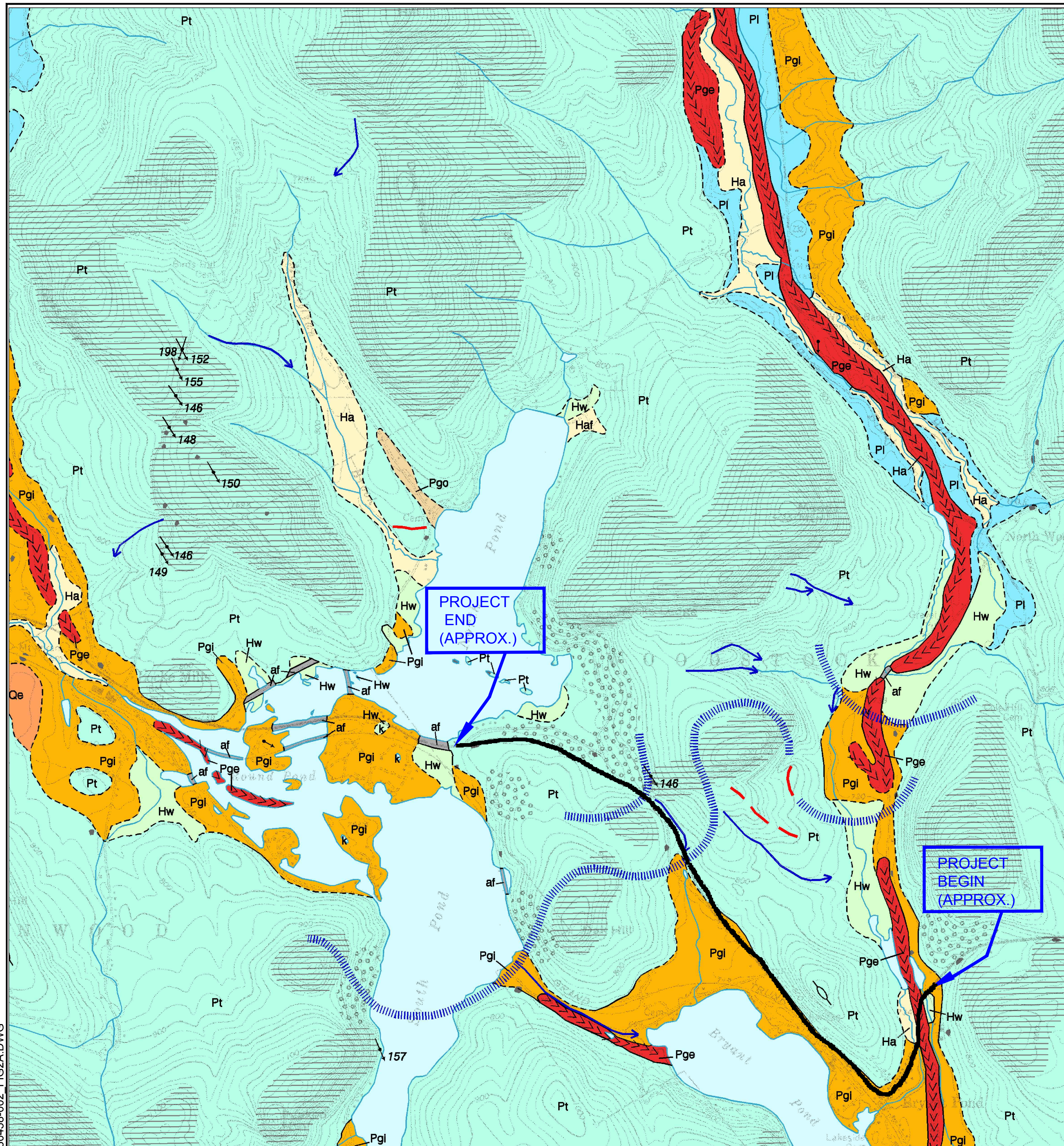
ROUTE 26 HIGHWAY REHABILITATION
MAINEDOT WIN 018767.00
WOODSTOCK, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
AUGUST 2020

FIGURE 1

130458-002_1_LOCUS.PDF



LEGEND

- **Ha** — **Stream alluvium** - Sand, silt, gravel, and organic sediment. Deposited on flood plains of streams. Unit includes some wetland areas, and may also include low terraces that are not flooded often.
- **Haf** — **Alluvial fan** - Gravel and sand deposited near the mouth of a small brook that drains into North Pond.
- **Qst** — **Stream terraces** - Extensive sand and gravel terraces occur in the Androscoggin River valley. Formed by postglacial erosion and deposition along the river, and derived in part from reworking of glacial lake sediments.
- **Hw** — **Wetland deposits** - Peat, muck, silt, and clay. Deposited in poorly drained areas on valley floors. Unit may grade into or include areas of stream alluvium.
- **Qe** — **Eolian deposits** - Windblown sand derived from glacial-lake sediments in the Androscoggin River basin. Includes longitudinal dunes oriented parallel to the prevailing wind direction when the dunes formed. Smaller unmapped areas of eolian sand may occur elsewhere in the quadrangle.
- **Pgo** — **Outwash deposits** - Gravel deposited by a glacial meltwater stream in a small valley west of North Pond.
- **Plbe** — **Glacial Lake Bethel deposits** - Sand, gravel, silt, and clay deposited in a glacial lake that occupied the Androscoggin River basin in the southwestern part of the quadrangle. Includes deltaic and fine-grained lake-bottom sediments. The lake level was controlled by one or more spillways at ~690-700 ft elevation in the adjacent Bethel quadrangle.
- **Plh** — **Glacial Lake Hanover deposits** - Sand, gravel, silt, and clay deposited in a glacial lake in the Androscoggin Valley in the northern part of the quadrangle. Includes deltaic and fine-grained lake-bottom sediments, which in many places have been partly eroded by postglacial streams. Glacial Lake Hanover probably was dammed by till deposits that temporarily blocked the narrow portion of the valley between Rumford Point and Rumford Center. Elevations of Plh deposits suggest a lake level as high as 780 ft.
- **Pl** — **Glacial-lake deposits (undifferentiated)** - Sand and gravel deposited in small ice-dammed lakes in the north sloping Barkers Brook and Otter Brook valleys. Meltwater ponded in the Barkers Brook valley spilled south through a gap at an elevation of about 730 ft between North Woodstock and Bryant Pond. The lake in the Otter Brook valley drained south through a spillway at about 750 ft, east of Walkers Mtn.
- **Pgi** — **Ice-contact deposits** - Miscellaneous sand and gravel deposits formed in contact with remnants of glacial ice. Includes glacial-stream sediments and subaqueous fans built into glacial lakes.
- **Pge** — **Esker deposits** - Sand and gravel deposited by meltwater streams in subglacial tunnels that developed in valleys.
- **Pt** — **Till** - Loose to very compact, poorly sorted, massive to weakly stratified mixture of sand, silt, and gravel-size rock debris deposited by glacial ice. Locally includes lenses of water-laid sand and gravel.
- **Bedrock outcrops/thin-drift areas** - Ruled pattern indicates areas where outcrops are common and/or surficial sediments are generally less than 10 ft thick (mapped partly from air photos). Dots show individual outcrops.
- **af** — **Artificial fill** - Earth, rock, and/or man-made fill along roads and railroads.
- **Contact** - Boundary between map units. Dashed where approximately located.
- **Scarp** - Scarp separating adjacent terrace levels in alluvial deposits along the Androscoggin River.
- **Glacially streamlined hill** - Symbol shows trend of long axis, which is parallel to former glacial ice-flow direction.
- 135 — **Glacial striation locality** - Arrow shows ice-flow direction inferred from striations on bedrock. Dot marks point of observation. Number is azimuth (in degrees) of flow direction.
- **End moraine** - Red line indicates the axis of a till ridge which is inferred to have been deposited at the margin of the last ice sheet in late-glacial time. A prominent cross-valley moraine and associated meltwater channels occur in the gap east of Walkers Mtn.
- **Dip of cross-bedding** - Arrow shows average dip direction of cross-bedding in sand and gravel deposits formed in a glacial stream or lake. This usually is the direction of meltwater flow (in stream deposits) or the direction in which a delta or subaqueous fan was building into a lake. Dot marks point of observation.
- **Meltwater channel** - Channel eroded by glacial meltwater stream or outflow from glacial lake. Arrow shows inferred direction of water flow.
- >>>>> **Crest of esker** - Shows trend of esker ridge. Chevrons point in direction of glacial meltwater flow.
- k — **Kettle** - "k" indicates a depression left by the melting of a stagnant glacial ice mass.
- **Ice-margin position** - Shows an approximate position of the glacier margin during ice retreat, based on meltwater deposits, moraines, and/or positions of meltwater channels.
- **Area of large boulders** - Area of glacial till where there are many large boulders, typically 3-5 ft or larger, scattered over the ground surface. These areas have been mapped only where observed, and they are likely to occur elsewhere in the till-covered uplands.
- **Sand dune** - Arrow shows inferred wind direction, based on long axis of longitudinal dune ridge.

NOTES

1. BASE MAP SOURCE: THOMPSON, WOODROW B., (2008). BRYANT POND QUADRANGLE, MAINE, OPEN FILE REPORT NO. 08-80, MAINE GEOLOGICAL SURVEY, AUGUSTA, MAINE.

0 1425 2850
APPROX. SCALE IN FEET

N

HALEY ALDRICH

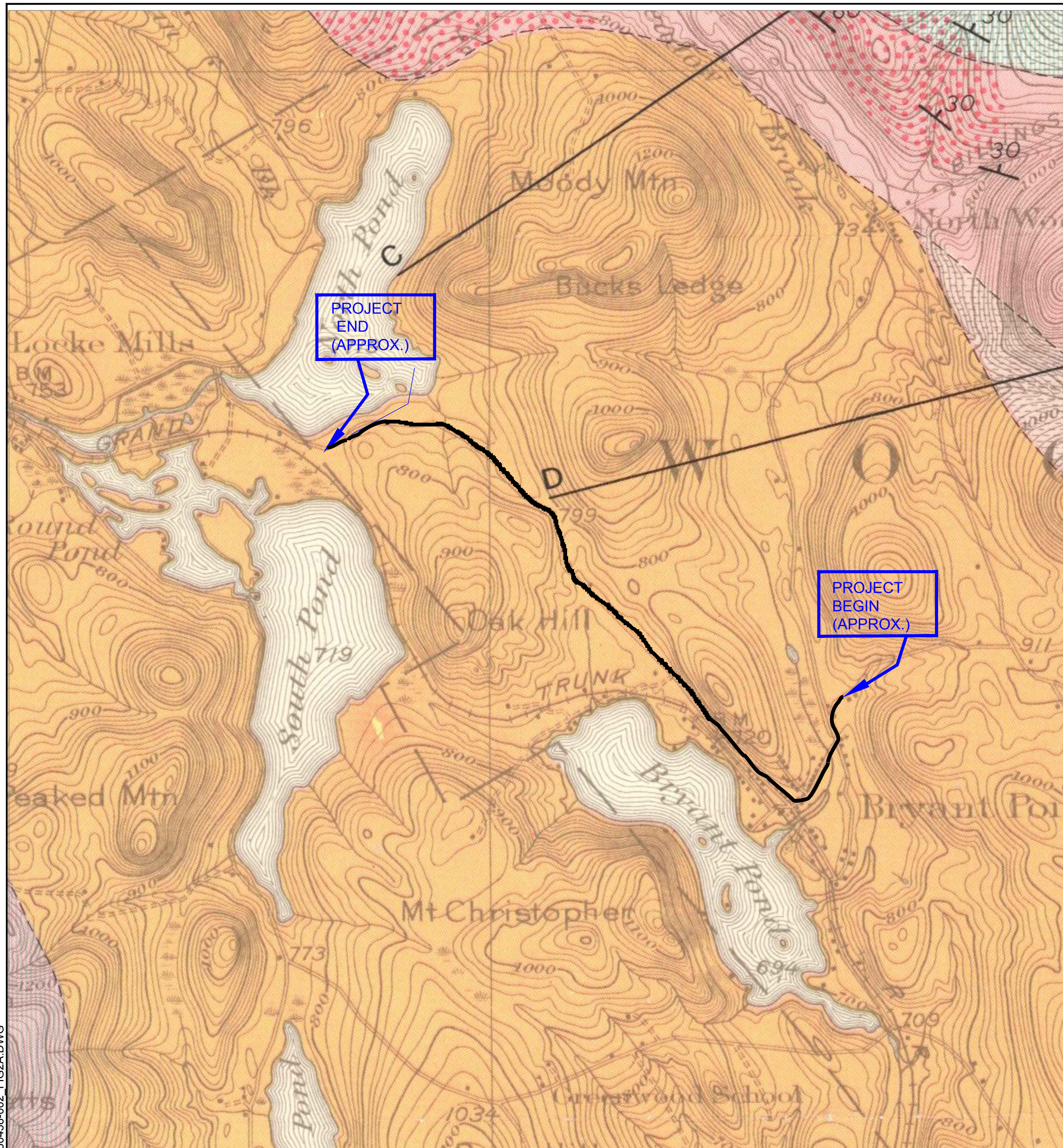
ROUTE 26 HIGHWAY REHABILITATION
MAINEDOT WIN 018767.00
WOODSTOCK, MAINE

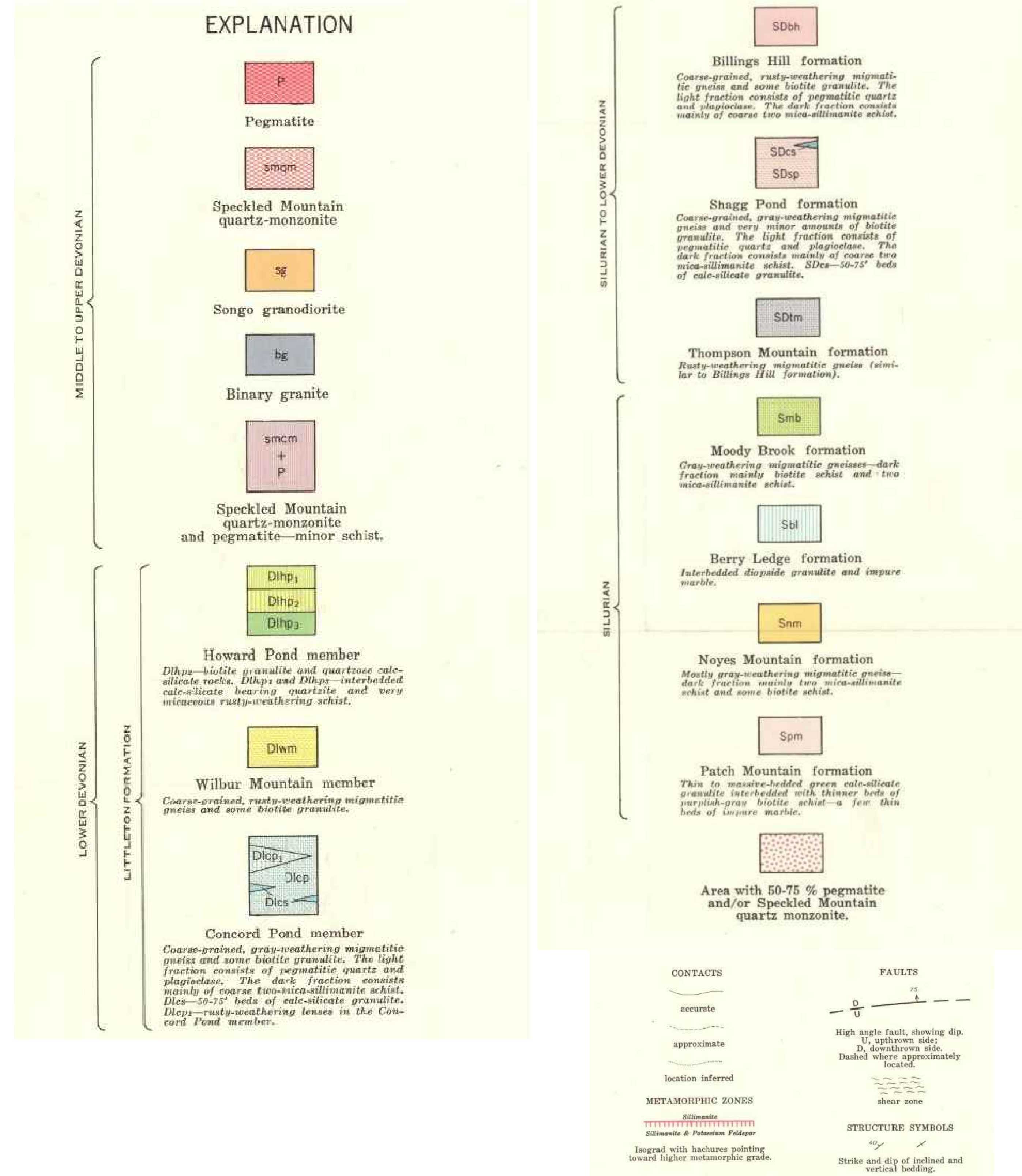
SURFICIAL GEOLOGY MAP

SCALE: AS SHOWN
AUGUST 2020

FIGURE 2

MLS Printed: 5/14/2019 1:40 PM Sheet: LAYOUT5-HA-FIG-B-L-H
G:\PROJECTS\130458 - WOODSTOCK\130458-002_FIG2A.DWG





NOTES

1. BASE MAP SOURCE: CHARLES V. GUIDOTTI, (1965). GEOLOGIC MAP OF THE BRYANT POND QUADRANGLE, MAINE, QUADRANGLE MAPPING SERIES NO. 3, MAINE GEOLOGICAL SURVEY, AUGUSTA, MAINE.

0 1850 3700
APPROX. SCALE IN FEET

N

HALEY ALDRICH

ROUTE 26 HIGHWAY REHABILITATION
MAINEDOT WIN 018767.00
WOODSTOCK, MAINE

BEDROCK GEOLOGY MAP

SCALE: AS SHOWN
AUGUST 2020

FIGURE 3

MLS
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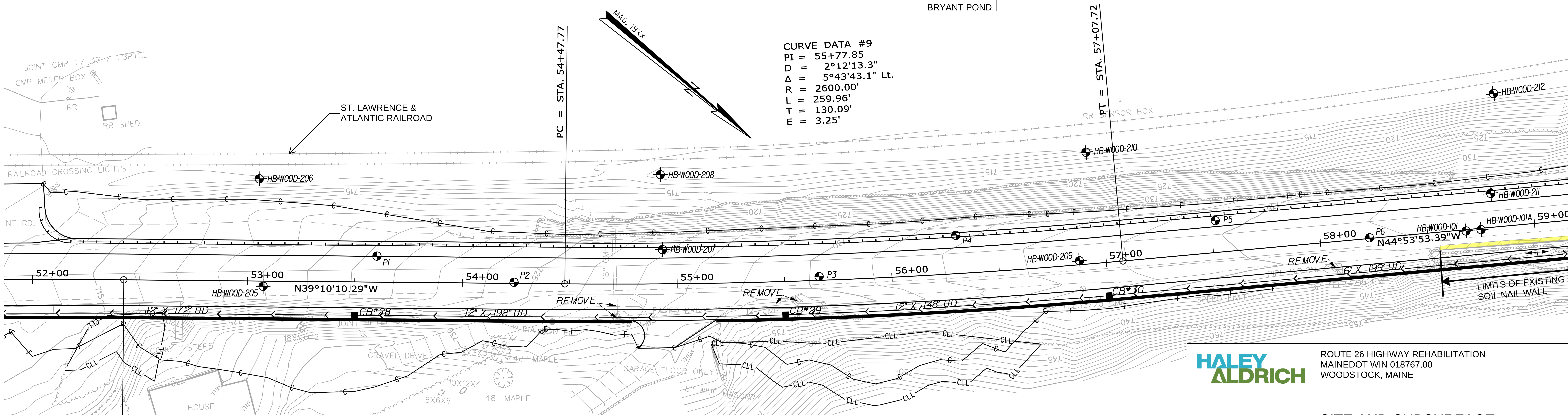
Printed: 5/14/2019 5:34 PM Sheet: LAYOUTS-HA-FIG-3-B-L



LEGEND

HB-WOOD-101 DESIGNATION AND LOCATION OF TEST BORING DRILLED BY MAINE DEPARTMENT OF TRANSPORTATION ON 8 DECEMBER 2011, USING SOLID STEM AUGERS.

HB-WOOD-206 DESIGNATION AND LOCATION OF TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS FROM 24 AUGUST THROUGH 1 SEPTEMBER 2018 USING CASED WASH DRILLING METHODS.

P2 DESIGNATION AND LOCATION OF SOLID STEM AUGER PROBE DRILLED BY NEW ENGLAND BORING CONTRACTORS FROM X TO X 2017.

NOTES

1. PLAN AND PROFILE PROVIDED BY MAINE DEPARTMENT OF TRANSPORTATION ON 8 SEPTEMBER 2019.
2. AS-DRILLED LOCATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINE DEPARTMENT OF TRANSPORTATION USING GPS SURVEY EQUIPMENT.
3. HB-WOOD-100 SERIES TEST BORINGS SHOWN WERE MONITORED BY MAINE DEPARTMENT OF TRANSPORTATION.
4. HB-WOOD-200 TEST BORINGS AND PROBES SHOWN WERE MONITORED BY HALEY & ALDRICH, INC.
5. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).

HALEY ALDRICH

ROUTE 26 HIGHWAY REHABILITATION
MAINEDOT WIN 018767.00
WOODSTOCK, MAINE

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN STA. 52+00 TO 59+00

SCALE: AS SHOWN
AUGUST 2020

FIGURE 4

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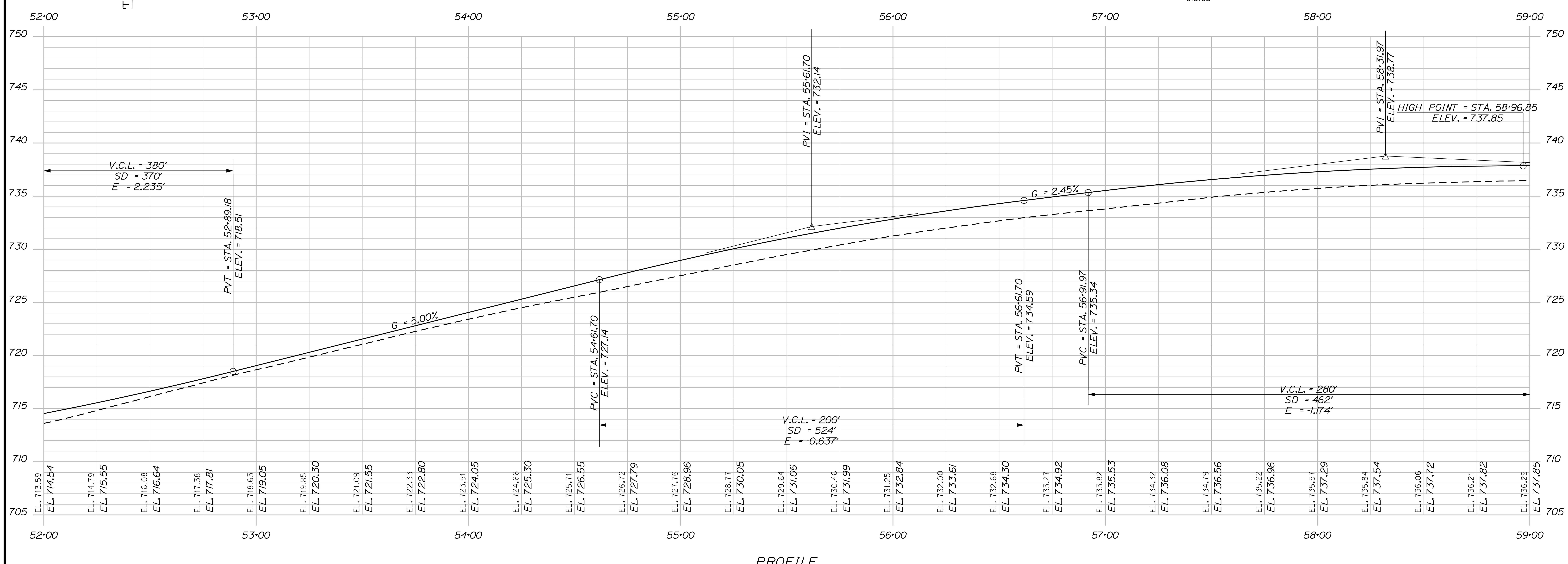
eforce



Filename: ...\MSTA\001_BLP&ISP 52 1.dgn
Division: GEOTECH
Username: Terry.White
Date:9/8/2017

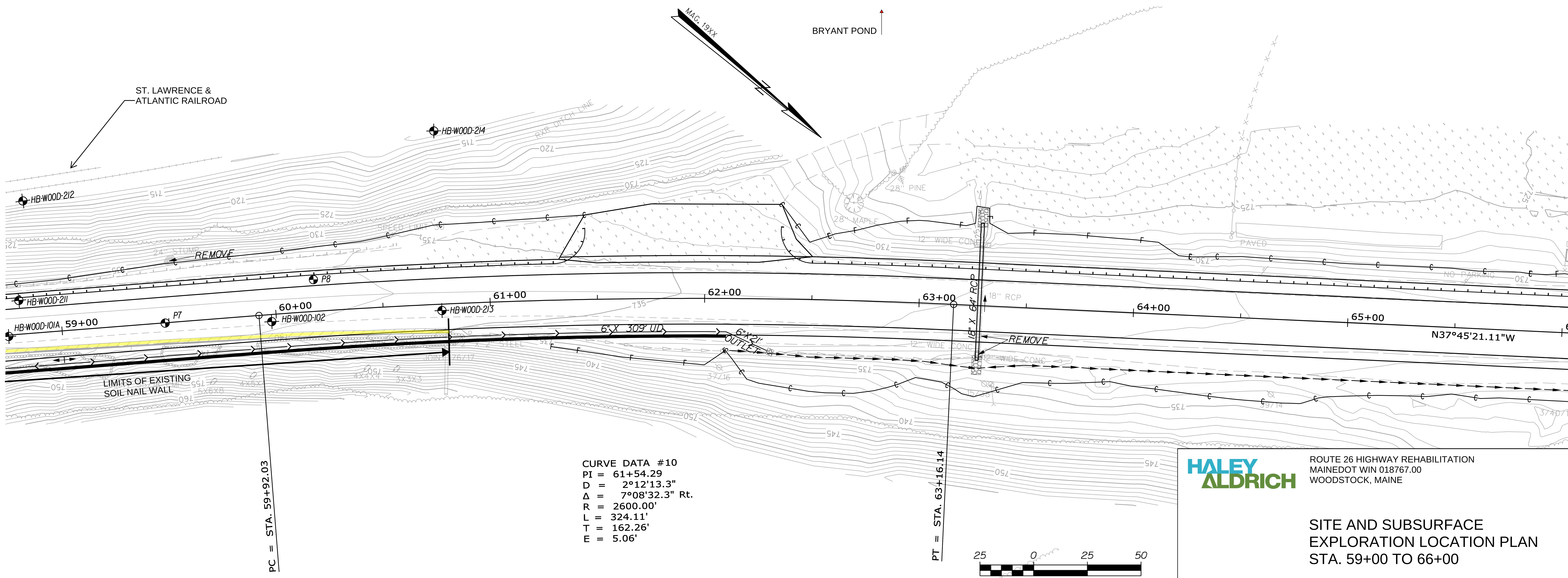
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
NHPP-1876(700)
WIN 018767.00
HIGHWAY PLANS

PROJ. MANAGER | DESIGN-DETAILED | CHECKED-REVIEWED | DESIGN2-DETAILED2 B.STEINERT | DESIGN3-DETAILED3 | REVISIONS 1 | REVISIONS 2 | REVISIONS 3 | REVISIONS 4 | FIELD CHANGES
BY T. WHITE | DATE SEP 2017 | SIGNATURE | P.E. NUMBER | DATE

WOODSTOCK
ROUTE 26

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
4
OF 5



CURVE DATA #10
PI = 61+54.29
D = 2°12'13.3"
Δ = 7°08'32.3" Rt.
R = 2600.00'
L = 324.11'
T = 162.26'
E = 5.06'

25 0 25 50
Scale in Feet

NOTES:
1. REFER TO FIGURE 2 FOR NOTES AND LEGEND.

HALEY ALDRICH

ROUTE 26 HIGHWAY REHABILITATION
MAINEDOT WIN 018767.00
WOODSTOCK, MAINE

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN
STA. 59+00 TO 66+00

SCALE: AS SHOWN
AUGUST 2020

FIGURE 5

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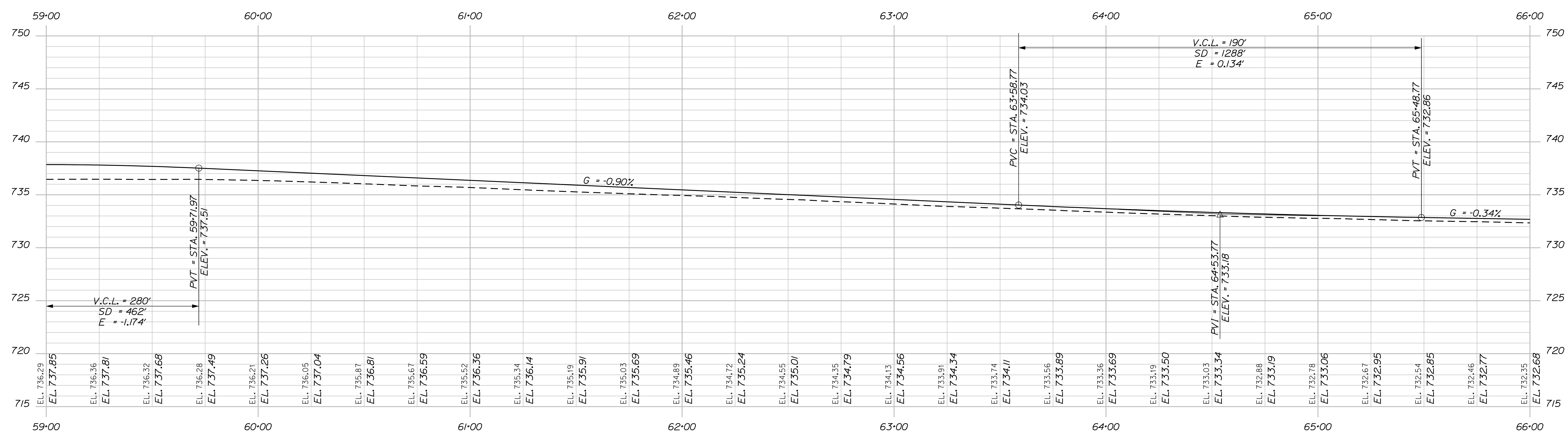
eforce



PROFILE

Date:9/8/2017
Username: Terry.White
Division: GEOTECH
Filename: ...\msta\001_BLP&ISP 62 2.dgn

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
NHPP-1876(700)
WIN 018767.00
HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED	B.STENERT	SEP 2017
DESIGN2-DETAILED2	T.WHITE	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

WOODSTOCK
ROUTE 26

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
5
OF 5

APPENDIX A

Test Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations (in this order, if applicable):

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
-tightness (tight, open, or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

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

Sample Container Labeling Requirements:

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

Previous Explorations by Others

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Bryant Pond Retaining Wall Route 232			**Boring No.:**	HB-WOOD-101
Location:	Woodstock, Maine			**WIN:**	18767.00
Driller:	MaineDOT	**Elevation (ft.)**	736.2	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/8/11-12/8/11	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	58+67.9, 0.7 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	735.70		6" PAVEMENT. 0.50	
	1D	12/10	1.00 - 2.00	10/50	---					Brown, moist, very dense, gravelly, fine to coarse SAND, trace silt.	
								734.20		2.00 Augered through possible weathered rock from 2.0-3.8 ft bgs.	
								732.40		3.80 **Bottom of Exploration at 3.80 feet below ground surface.** REFUSAL	
5											
10											
15											
20											
25											

Remarks:

Old Project WIN 19168.00.
Boring HB-WOOD-101 AND HB-WOOD-101A are on same field boring log sheet.

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: Bryant Pond Retaining Wall Route 232
Location: Woodstock, Maine

Boring No.: HB-WOOD-101A
WIN: 18767.00

Driller:	MaineDOT	**Elevation (ft.)**	736.2	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/8/11-12/8/11	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	58+75, 0.6 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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.
0							SSA	735.70		6" PAVEMENT. 0.50	
										No description given.	
								733.70		2.50	
										Bottom of Exploration at 2.50 feet below ground surface. REFUSAL	
5											
10											
15											
20											
25											

Remarks:

Old Project WIN 19168.00.
Boring HB-WOOD-101 AND HB-WOOD-101A are on same field boring log sheet.

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: Bryant Pond Retaining Wall Route 232
Location: Woodstock, Maine

Boring No.: HB-WOOD-102
WIN: 18767.00

Driller:	MaineDOT	**Elevation (ft.)**	736.1	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/8/11-12/8/11	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	59+97.8, 3.3 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	735.60		6" PAVEMENT. 0.50	
	1D	24/19	1.00 - 3.00	15/20/13/11	33	46				Brown, damp, dense, fine to coarse SAND, some garvel, trace silt.	
5								731.20		4.90 **Bottom of Exploration at 4.90 feet below ground surface.** REFUSAL	
10											
15											
20											
25											

Remarks:

Old Project WIN 19168.00.

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.

Preliminary Design Phase Explorations by Haley & Aldrich

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Route 26 Highway Rehabilitation Location: Woodstock, Maine	Boring No.: HB-WOOD-205 WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	719.0	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30;HW+NW-300#/
Date Start/Finish:	8-25-17/8-25-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 53+07.2, 2.5 ft RT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	9.7 ft
Hammer Efficiency Factor:	0.869	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = 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							SSA	718.3		-BITUMINOUS CONCRETE- 0.7	
	1D	24/16	1.0 - 3.0	25/23/16/15	39	56				Brown, dry, very dense, fine to coarse SAND, some gravel, little silt and gravel -FILL-(SM) PID=0.0 ppm	G#300102 A-1-b, SM WC=4.8%
								716.6		2.4	
	2D	24/19	3.0 - 5.0	13/18/27/26	45	65				Light brown, dry, very dense, Silty fine to medium SAND, trace coarse sand and gravel -FILL-(SM) PID=0.0 ppm	G#300103 A-4, SM WC=9.6%
5	3D	24/22	5.0 - 7.0	15/19/16/21	35	51				Light brown, dry, very dense, fine to medium SAND, some silt, trace coarse sand and gravel -FILL-(SM) PID=0.0 ppm	G#300104 A-2-4, SM WC=8.0%
	4D	24/14	7.0 - 9.0	12/14/13/10	27	39	277			Light brown, moist, medium dense, fine to medium SAND, some silt and gravel, trace coarse sand -FILL-(SM) PID=0.0 ppm	G#300105 A-2-4, SM WC=6.9%
							45				
							24	710.0		9.0	
10	5D	22/13	10.0 - 11.8	36/40/42/68(4.0)	82	119	113			Red-brown, moist, very dense, fine to coarse SAND, little fine gravel, trace silt, well graded, well bonded -GLACIAL TILL-(SW) PID=0.0 ppm	
							228	707.2		11.8	
							169	706.0		Note: Washed through boulder from 11.8 to 13 ft. -BOULDER- 13.0	
							100(5") 100(1")			Note: Probable glacial till based on drill action and observation of wash water. -GLACIAL TILL-	
							20	704.2		14.8	
15	R1	58/43	15.2 - 20.0	RQD = 74%			NQ Core			Top of Bedrock at El.704.2 R1: Light grey, fine to medium grained, GRANODIORITE. Very hard, fresh. Single joint at 16.6 ft moderately dipping, moderately close, tight, rough, undulating, oxidized joint surface. Rock Quality=Fair Recovery=74% -SONGO GRANODIORITE FORMATION- R1 Core Times (min:sec): 15.2-16.2' (2:32); 16.2-17.2' (4:11) 17.2-18.2' (3:24); 18.2-19.2' (4:17); 19.2-20.0' (4:08)	
20								699.0		20.0 **Bottom of Exploration at 20.0 feet below ground surface.**	
25											

Remarks:

PID = photoionization detector

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: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-206
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	710.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	SS-140#/30;NW-300#/18
Date Start/Finish:	8-29-17/8-29-17	**Drilling Method:**	SSA/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 53+05.2, 47.4 ft LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	Not observed
Hammer Efficiency Factor:	0.750	**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	24/10	0.0 - 2.0	1/2/5/6	7	9	SSA			Dark brown, dry, loose, fine to coarse SAND, little fine gravel, trace coarse gravel, well graded -FILL-(SW) PID=0.0 ppm	
								709.5		1.3	
	2D	24/15	2.0 - 4.0	22/13/13/16	26	33				Note: Cobble encountered between 2 and 2.5 ft. Light brown, dry, medium dense to dense, fine SAND, little gravel, trace silt, medium to coarse sand, poorly-graded, depositional layering with <1/16-in. thick silt laminae -GLACIOFLUVIAL DEPOSIT-(SP)	
								706.8		4.0	
5	3D	24/10	4.0 - 6.0	20/35/33/33	68	85	NW Spin			Light brown, dry, very dense, fine SAND, some medium sand, little coarse sand and gravel, trace silt, loosely bonded -GLACIAL TILL-(SP) PID=0.0 ppm	
10	4D	16/11	10.0 - 11.3	22/25/50(4.0)	75	94				Light brown, moist, very dense, fine SAND, little medium sand, trace silt and coarse gravel, poorly-graded, loosely bonded -GLACIAL TILL-(SP) PID=0.0 ppm	
								699.5		11.3	
	R1	60/60	12.3 - 17.3	RQD = 100%			NQ Core			Top of Bedrock at El.699.5 R1: Grey, medium to coarse grained GRANODIORITE. Very hard, fresh. Joints dipping steeply, close to moderately close, tight to open, slightly weathered joint surfaces. Rock Quality=Excellent Recovery=100% -SONGO GRANODIORITE FORMATION- R1 Core Times (min:sec): 12.3-13.3' (2:33); 13.3-14.3' (2:01); 14.3-15.3' (2:02); 15.3-16.3' (2:03); 16.3-17.3' (1:53)	
15											
								693.5		17.3	
										Bottom of Exploration at 17.3 feet below ground surface.	
20											
25											

Remarks:

PID = photoionization detector

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

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Boring No.: HB-WOOD-206

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Route 26 Highway Rehabilitation **Location:** Woodstock, Maine	**Boring No.:** HB-WOOD-207 **WIN:** 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	726.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; NW-300#/18
Date Start/Finish:	8-24-17/8-24-17	**Drilling Method:**	SSA/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 54+93.5, 16.2 ft LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	Dry
Hammer Efficiency Factor: 0.869		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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							SSA			-BITUMINOUS CONCRETE-	
	1D	24/14	1.0 - 3.0	12/14/27/41	41	59		725.8		1.0 Brown, dry, very dense, fine to coarse SAND, some gravel, little silt, crushed rock from 2.5 to 3 ft in spoon (granodiorite) -FILL-(SM) PID=0.0 ppm	G#300106 A-1-b, SM WC=3.3%
	2D/A	24/16	3.0 - 5.0	12/29/8/5	37	54				2D (3.0-4.0 ft bgs) Brown, dry, very dense, fine to medium SAND, some gravel, little silt, trace coarse sand -FILL-(SM) PID=0.0 ppm	G#300107 A-1-b, SM WC=3.8%
								722.8		4.0	G#300109 A-2-4, SM WC=11.4%
5	3D	24/19	5.0 - 7.0	2/1/1/1	2	3	9			2D/A (4.0-5.0 ft bgs) Light brown, dry, medium dense, fine to medium SAND, some silt, trace coarse sand and gravel -FILL-(SM) PID=0.0 ppm Samples 2D/A and 3D combined.	
							12				
	4D	24/9	7.0 - 9.0	1/2/1/2	3	4	17			Light brown to dark brown, dry, very loose, fine to medium SAND, some silt, trace coarse sand and gravel -FILL-(SM) PID=0.0 ppm Samples 2D/A and 3D combined.	G#300110 A-2-4, SM WC=15.4%
							25				
							25(6") NQ Core	717.2		Red-brown, dry, very loose, fine to medium SAND, little silt, trace coarse sand and gravel -FILL-(SM) PID=0.0 ppm	
10											
										9.6 Note: Cored through boulder from 9.6 to 12.6 ft. -BOULDER-	
							Open	714.2		12.6 Note: Drill wash water contents indicate silty granular soils from 12.6 to 15 ft.	
15	5D	24/17	15.0 - 17.0	20/31/32/75	63	91				Olive-brown, moist, very dense, fine to coarse SAND, little silt, trace gravel, well graded, two 3-in. crushed gravel pieces from 16.5 to 17 ft -GLACIAL TILL-(SW-SM) PID=0.0 ppm	
								708.8 708.2		18.0 Note: Advanced rollerbit through cobble from 18 to 18.6 ft. -COBBLE- 18.6	
20	6D	24/16	20.0 - 22.0	24/20/32/34	52	75				Olive-brown, moist, very dense, fine to medium SAND, little silt, trace coarse sand and fine gravel, poorly-graded, two 1-in. and one 3-in. gravel pieces from 21 to 22 ft -GLACIAL TILL-(SP-SM) PID=0.0 ppm	
25											

Remarks:

PID = photoionization detector

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-207
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	726.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; NW-300#/18
Date Start/Finish:	8-24-17/8-24-17	**Drilling Method:**	SSA/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 54+93.5, 16.2 ft LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	Dry
Hammer Efficiency Factor:	0.869	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							Open NQ Core	701.9		Note: Advanced core barrel through cobbles and boulders with intermittent soil (glacial till) from 25.5 to 31 ft. -BOULDER- (24.9)	
								700.7		Note: Drill action indicates glacial till from 26.1 to 28.6 ft. -GLACIAL TILL- (26.1)	
								698.2		-COBBLE- (28.6)	
								697.3		Note: Drill action indicates glacial till from 29.5 to 31 ft. Borehole collapsed at 31 ft, terminated boring. -GLACIAL TILL- (29.5)	
30											
								695.8		(31.0) **Bottom of Exploration at 31.0 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

PID = photoionization detector

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-208
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	711.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; NW-300#/18
Date Start/Finish:	8-30-17/8-30-17	**Drilling Method:**	SSA/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 54+92.6, 50.8 ft LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	14.7 ft
Hammer Efficiency Factor: 0.750		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
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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	24/5	0.0 - 2.0	1/2/3/5	5	6	SSA	710.4		Dark brown, dry, loose, coarse GRAVEL, little fine sand and fine gravel, trace silt and medium to coarse sand, poorly-graded, rock in spoon tip -FILL-(GP) PID=0.0 ppm (1.4)	
	2D	24/9	2.0 - 4.0	5/6/7/7	13	16				Light brown, dry, medium dense, fine to coarse SAND, trace fine gravel, well graded -FILL-(SW) PID=0.0 ppm	
	3D	24/10	4.0 - 6.0	5/7/8/8	15	19				Brown, moist, medium dense, medium SAND, little coarse sand, trace fine sand and coarse gravel -FILL-(SP)	
5	4D	24/3	6.0 - 8.0	10/9/13/12	22	28	NW Spun			Light brown, moist, medium dense, coarse GRAVEL, little coarse sand and fine gravel, trace fine and medium sand, poorly-graded -FILL-(GP) PID=0.0 ppm Note: Rock stuck in spoon tip. Note: Strata change at 9.3 ft based on drill advance rate and rotation pressure gauge.	
10	5D	24/16	10.0 - 12.0	20/25/40/32	65	81		702.5		(9.3) Olive-brown, moist, very dense, fine to coarse SAND, little coarse sand and fine gravel, trace silt and coarse gravel, well graded, loosely bonded, two 3-in. gravel pieces -GLACIAL TILL-(SW) PID=0.0 ppm	
	R1							699.1		(12.7)	
	R1		13.0 - 16.0				NQ Core			Note: Cored through boulder from 13 to 15.3 ft. -BOULDER-	
15	6D	20/16	15.5 - 17.2	8/39/84/62(4.0)	126	158	NW Spin	696.5		(15.3) Olive-grey to olive-brown, moist, very dense, fine to coarse SAND, little silt, well graded -GLACIAL TILL-(SW) PID=0.0 ppm Note: Top 3 in. of sample contained soil, remaining bottom 13 in. contained pulverized rock. Top 6 in. of sample likely impacted by core barrel resulting in low blow counts.	
							RC	694.5		(17.3) -BOULDER-	
20	R2		19.0 - 24.0				NQ Core	692.1		(19.7) Note: Cored from 19 to 24 ft through boulder, cobbles and coarse gravel. Recovered the following: two 4-in. dia. stones; two 3-in. dia. stones, one 2-in. dia. stone, and two 1-in. dia. stones. -GLACIAL TILL/COBBLES-	
25							NW Spin	687.6 / 687.1		(24.2) -COBBLE-	

Remarks:

PID = photoionization detector

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

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Boring No.: HB-WOOD-208

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-208
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	711.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; NW-300#/18
Date Start/Finish:	8-30-17/8-30-17	**Drilling Method:**	SSA/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 54+92.6, 50.8 ft LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	14.7 ft
Hammer Efficiency Factor:	0.750	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/9	25.0 - 27.0	14/12/15/13	27	34	NW Spun			24.7 Olive-brown, moist, medium dense, fine SAND, little silt and coarse sand, trace medium sand and fine gravel, poorly-graded, loosely bonded, contains two 1-in. gravel pieces -GLACIAL TILL-(SP-SM) PID=0.0 ppm	
								684.6		27.2 -BOULDER-	
								682.9		28.9 -GLACIAL TILL-	
30								682.3		29.5 -COBBLE-	
	8D	12/5	31.0 - 32.0	17/109				680.8		31.0 Olive-brown, moist, very dense, Gravelly SAND, trace silt, well graded, moderately bonded -GLACIAL TILL-(SW) PID=0.0 ppm Note: Rock in spoon tip.	
								680.2		31.6 -COBBLE-	
							Open	679.3		32.5 -GLACIAL TILL-	
35	R3	60/57	35.0 - 40.0	RQD = 82%			NQ Core	678.2		33.6 Top of Bedrock at El.678.2 R3: Light grey to light green, medium grained, GRANOFELS. Very hard, fresh. Joints dipping at low to moderate angles, very close to moderately close, tight or healed joints, planar, smooth, joints are calcite healed or slightly oxidized and tight. Rock Quality=Good Recovery=95% -SONGO GRANODIORITE FORMATION- R3 Core Times (min:sec): 35.0-36.0' (1:22); 36.0-37.0' (1:41); 37.0-38.0' (1:56); 38.0-39.0' (1:54); 39.0-40.0' (2:02)	
40								671.8		40.0 **Bottom of Exploration at 40.0 feet below ground surface.**	
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-209
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	733.6	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW+NW-300#/
Date Start/Finish:	8-28-17/8-28-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 56+87.6, 1.9 ft LT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	6.0 ft
Hammer Efficiency Factor:	0.869	**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
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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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	733.3	-BITUMINOUS CONCRETE- 0.3	
	1D	20/7	1.0 - 2.7	22/35/51/50(2.0)	86	125			Brown, dry, very dense, Gravelly fine to medium SAND, little silt, trace coarse sand -FILL-(SM) PID=0.0 ppm Note: Coarse gravel piece from 2.7 to 3 ft.	G#300111 A-1-b, SM WC=1.6%
	2D	24/11	3.0 - 5.0	1/2/3/39	5	7			Red-brown, dry, loose, fine to medium SAND, little silt and gravel, trace coarse sand -FILL-(SM) PID=0.0 ppm	G#300112 A-2-4, SM WC=12.0%
5	3D	24/14	5.5 - 7.5	22/26/25/14	51	74	HW Spin		Note: Coarse gravel piece from 5 to 5.3 ft. Brown, dry, very dense, Gravelly fine to medium SAND, little silt, trace coarse sand -FILL-(SM) PID=0.0 ppm	G#300113 A-1-b, SM WC=3.9%
								726.6	7.0	
	4D	24/14	7.5 - 9.5	20/25/28/45	53	77			Olive-brown to grey-brown, moist, very dense, fine to medium SAND, some silt, little gravel, trace coarse sand, loosely bonded -GLACIAL TILL-(SM) PID=0.0 ppm	G#300114 A-2-4, SM WC=10.1%
10										
									Note: Drill wash returns fine to coarse sand.	
								720.5	13.1	
	R1	8/8	13.5 - 14.2				NQ Core		Note: Cored through cobble from 13.5 to 14 ft. -COBBLE-	
							30	719.6	14.0	
15	5D	16/8	15.0 - 16.3	56/72/62(4.0)	134	194	113		Olive-brown to grey-brown, moist, very dense, fine to medium SAND, trace coarse sand, silt and fine gravel, well graded, well bonded -GLACIAL TILL-(SW) PID=0.0 ppm	
							350			
							427			
							NW Spin			
								714.5	19.1	
20	R2	60/58	19.1 - 24.1	RQD = 83%			NQ Core		Top of Bedrock at El.714.5 R2: Pink-grey to grey, medium to coarse grained, GRANODIORITE. Very hard, fresh to slightly weathered from 22.6 to 23.1. Single joint from 22.6 to 23.1 ft dipping steeply, moderately close, open (0.25 in.), silty clay infilling 0.25-in. thick with fine to coarse sand, planar, rough. Rock Quality=Good Recovery=97% -SONGO GRANODIORITE FORMATION- R2 Core Times (min:sec): 19.1-20.1' (1:45); 20.1-21.1' (1:14); 21.1-22.1' (1:18); 22.1-23.1' (1:16); 23.1-24.1' (1:32)	
								709.5	24.1	
25										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-209
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	733.6	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW+NW-300#/
Date Start/Finish:	8-28-17/8-28-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 56+87.6, 1.9 ft LT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	6.0 ft
Hammer Efficiency Factor:	0.869	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N-uncorrected = Raw Field SPT N-value
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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
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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										**Bottom of Exploration at 24.1 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-210
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	712.5	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	SS-140#/30;HW+NW-300#/
Date Start/Finish:	8-31-17/8-31-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 56+94.9, 51.8 ft LT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	10.6 ft
Hammer Efficiency Factor:	0.750	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/1	0.0 - 2.0	1/1/1/1	2	3	SSA			Dark brown, moist, very loose, fine SAND, little silt, trace medium sand, poorly-graded -FILL-(SP-SM) PID=0.0 ppm Note: Very low recovery, soil in jar is from auger flights.	
	2D	24/16	2.0 - 4.0	WOH/1/17/38	18	23	3	709.5		Dark brown, moist, very loose, fine SAND, little silt, trace medium sand, poorly-graded -FILL-(SM) PID=0.0 ppm — 3.0	
							103	709.0		Dark brown to brown with depth, medium dense, fine SAND, some medium sand, trace silt -FILL-(SP-SM)- — 3.5	
	3D	24/7	4.0 - 6.0	17/32/37/34	69	86	114			Olive-brown to brown, moist, dense, Gravelly SAND, trace silt, well graded, loosely bonded -GLACIAL TILL-(SW) PID=0.0 ppm	
5							HW Spun			Grey, dry, very dense, Gravelly SAND, trace silt, well graded, loosely bonded, recovery primarily contains rock fragments (granodiorite) -GLACIAL TILL-(SW) PID=0.0 ppm	
								704.5		Note: Frequent cobbles from 4 to 8 ft based on rotation pressure gauge and rate of advancement of casing. — 8.0	
										Note: Drill head advanced rapidly from 8 to 10 ft.	
10	4D	24/16	10.0 - 12.0	32/44/43/42	87	109	NW Spun			Olive-grey to grey-brown, moist, very dense, fine to coarse SAND, little gravel, trace silt, well graded, moderately bonded, contains two 1-in. gravel pieces -GLACIAL TILL-(SW) PID=0.0 ppm	
	5D	2/2	13.0 - 13.2	50(2.0)				699.3		Grey, moist, very dense, fine to coarse sand, trace silt and fine gravel, well graded -GLACIAL TILL-(SW) PID=0.0 ppm — 13.2	
	R1	44/16	14.3 - 18.0				NQ Core	698.0		Note: R1 recovery consists of 2-in. boulder pieces, one 7-in. cobble and two 3.5-in. cobbles. -BOULDER- — 14.5	
15										-GLACIAL TILL/COBBLES-	
	6D	1/0	18.0 - 18.1	50(1.0)			NQ Core	694.5		— 18.0 Top of Bedrock at El.694.5	
	R2	60/48	18.3 - 23.3	RQD = 67%						R2: Grey medium to coarse grained, GRANODIORITE. Very hard, fresh. Joints indiscernable. Rock Quality=Fair Recovery=80% -SONGO GRANODIOIRITE FORMATION- R2 Core Times (min:sec): 18.3-19.3' (1:24); 19.3-20.3' (1:30); 20.3-21.3' (1:16); 21.3-22.3' (1:53); 22.3-23.3' (1:24)	
20											
								689.2		— 23.3 **Bottom of Exploration at 23.3 feet below ground surface.**	
25											

Remarks:

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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.

Boring No.: HB-WOOD-210

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Route 26 Highway Rehabilitation **Location:** Woodstock, Maine	**Boring No.:** HB-WOOD-211 **WIN:** 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	736.4	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW+NW-300#/
Date Start/Finish:	8-24-17/8-24-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 58+81, 15.8 ft LT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	8.6 ft
Hammer Efficiency Factor:	0.869	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = 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							SSA	735.8		-BITUMINOUS CONCRETE- 0.6	
	1D	24/16	1.2 - 3.2	15/13/14/26	27	39				Brown, dry, medium dense, fine to medium SAND, some gravel, little silt, trace coarse sand -FILL-(SM) PID=0.0 ppm	G#300115 A-1-b, SM WC=6.6%
	2D	15.6/10	3.5 - 4.8	14/39/50(2.0)	89+	129				Brown, dry, very dense, Gravelly fine to medium SAND, little silt, trace coarse sand -FILL-(SM) PID=0.0 Note: 3 in. in spoon tip is pulverzied rock. 4.4	G#300116 A-1-b, SM WC=2.7%
							33	732.0			
5							NQ Core	730.7		Note: Drive HW casing to 5 ft, advance rollerbit to 5.5 ft, cored to 7.5 ft, recovered 2 in. of rock. -BOULDER- 5.7	
	3D	24/18	7.5 - 9.5	33/39/37/56	76	110	25			Brown-grey, dry, very dense, fine to medium SAND, some silt, little gravel, trace coarse sand, multiple 2 to 3 in. diameter gravel pieces -FILL-(SM) PID=0.0 ppm	G#300117 A-2-4, SM WC=7.5%
							17				
							15				
10							19				
							28	724.7		11.7	
							32	723.8		Note: Advanced rollerbit through cobble to 12.6 ft. -COBBLE- 12.6	
							25	722.7		Note: Coarse gravel piece from 13 to 13.3 ft. -GLACIAL TILL- 13.7	
							50(6")			Top of Bedrock at El.722.7	
15	R1	58/58	15.0 - 19.8	RQD = 100%			NQ Core			R1: Grey, medium grained, GRANODIORITE. Very hard, fresh. Solid core stem. Rock Quality=Excellent Recovery=100% -SONGO GRANODIORITE FORMATION- R1 Core Times (min:sec): 15.0-16.0' (5:23); 16.0-17.0' (3:42); 17.0-18.0' (2:56); 18.0-19.0' (2:52); 19.0-19.8' (2:58)	
								716.6		19.8	
20										**Bottom of Exploration at 19.8 feet below ground surface.**	
25											

Remarks:

PID = photoionization detector

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: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-212
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	712.6	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Bombardier	**Hammer Wt./Fall:**	SS-140#/30;NW-300#/18
Date Start/Finish:	8-31-17/8-31-17	**Drilling Method:**	SSA/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 58+86.5, 61.8 ft LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	3.0 ft
Hammer Efficiency Factor:	0.750	**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							SSA			-RAILROAD BALLAST-	
	1D	17/17	1.5 - 2.9	3/11/50(3.0)				711.1		1.5	
								710.6		Light brown, dry, very loose, fine SAND, trace medium sand, coarse sand and fine gravel, poorly-graded -FILL-(SP) PID=0.0 ppm	
										2.0	
								709.7		Grey, dry, very dense, fine to coarse SAND, little gravel, well graded, rock fragments -GLACIAL TILL-(SW)	
										2.9	
5	R1	60/57	5.0 - 10.0	RQD = 83%			NQ Core			Top of Bedrock at El.709.7 R1: Grey, medium grained, GRANODIORITE. Very hard, fresh. Joints dipping at low angles, single vertical joint from 6 to 6.5 ft, very close, open, rough, undulating, slightly weathered joint surfaces. Rock Quality=Good Recovery=95% -SONGO GRANODIORITE FORMATION- R1 Core Times (min:sec): 5.0-6.0' (2:03); 6.0-7.0' (1:35); 7.0-8.0' (1:53); 8.0-9.0' (2:05); 9.0-10.0' (1:46)	
10								702.6		10.0 **Bottom of Exploration at 10.0 feet below ground surface.**	
15											
20											
25											

Remarks:

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Boring No.: HB-WOOD-212

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 26 Highway Rehabilitation
Location: Woodstock, Maine

Boring No.: HB-WOOD-213
WIN: 18767.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	735.6	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-59 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW+NW-300#/
Date Start/Finish:	8-29-17/8-29-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 60+77.4, 3.0 ft RT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	7.6 ft
Hammer Efficiency Factor: 0.869		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = 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							SSA	735.1		-BITUMINOUS CONCRETE- 0.5	
	1D	24/17	1.0 - 3.0	29/22/19/21	41	59				Brown, dry, very dense, Gravelly fine to coarse SAND, trace silt, well graded, contains asphalt pieces -FILL-(SW) PID=11.0 ppm	G#300118 A-1-a, SW-SM WC=2.3%
	2D	24/14	3.0 - 5.0	20/25/21/23	46	67		730.9		Brown, dry, very dense, Gravelly fine to medium SAND, little silt, trace coarse sand -FILL-(SM) PID=0.0 ppm 4.7	G#300119 A-1-b, SM WC=3.1%
5	3D	24/15	5.0 - 7.0	17/16/15/54	31	45	HW Spin	728.6		Brown to olive-brown, dry, dense, fine to medium SAND, little silt and gravel, trace coarse sand, loosely bonded -GLACIAL TILL-(SM) PID=0.0 ppm 7.0	G#300120 A-1-b, SM WC=5.2%
								727.9		-COBBLES- 7.7 Coarse gravel from 8.0 to 8.3 ft. Note: Spun HW casing to 10 ft. Drill wash water contains sand, chips of granodiorite within glacial till.	
10	4D	24/18	10.0 - 12.0	17/52/64/28	116	168		722.5		Olive-brown, black and white, moist, very dense, fine to coarse SAND, trace silt and gravel, well graded, contains 9 in. of pulverized rock (granodiorite) -GLACIAL TILL-(SW-SM) PID=0.0 ppm 13.1	
										Top of Bedrock at El.722.5	
15	R1	60/60	14.5 - 19.5	RQD = 100%			NQ Core	716.1		R1: Grey, medium grained, GRANODIORITE. Very hard, fresh, solid core stem. Rock Quality=Excellent Recovery=100% -SONGO GRANODIORITE FORMATION- R1 Core Times (min:sec): 14.5-15.5' (1:31); 15.5-16.5' (1:33) 16.5-17.5' (1:05); 17.5-18.5' (1:17); 18.5-19.5' (1:31) 19.5	
20										**Bottom of Exploration at 19.5 feet below ground surface.**	
25											

Remarks:

PID = photoionization detector

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:	Route 26 Highway Rehabilitation	**Boring No.:**	HB-WOOD-214		
Location:	Woodstock, Maine	**WIN:**	18767.00		
Driller:	New England Boring Contractors	**Elevation (ft.)**	712.1	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	SS-140#/30;HW+NW-300#/
Date Start/Finish:	9-1-17/9-1-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 60+77.3, 80.9 ft LT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	18.3 ft
Hammer Efficiency Factor:	0.750	**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	24/15	0.0 - 2.0	WOH/2/2/4	4	5	Push			Dark brown, dry, loose, fine SAND, little silt, medium sand and coarse sand, poorly-graded, reworked native soil -FILL-(SP) PID=0.0 ppm 1.0	
								711.1		Brown, dry, loose, fine to coarse SAND, trace fine gravel, well graded, reworked native soil -FILL-(SW) PID=0.0 ppm 2.0	
	2D	24/14	2.0 - 4.0	4/3/4/3	7	9		710.1		Light brown, dry, loose, fine to medium SAND, trace coarse sand, poorly-graded, becomes coarser with depth, reworked native soil -FILL-(SP) PID=0.0 ppm 4.0	
	3D	24/14	4.0 - 6.0	2/3/4/5	7	9	13	708.1		Light brown to brown, moist, loose, fine to coarse SAND, trace fine gravel, well graded, oxidation observed -GLACIOFLUVIAL DEPOSIT-(SW) PID=0.0 ppm	
5							30				
							28				
							31				
							25				
							28			10.0	
10	4D	24/8	10.0 - 12.0	2/3/4/4	7	9	11	702.1		Light brown, wet, loose, fine SAND, poorly-graded, depositional layering apparent -GLACIAL LACUSTRINE DEPOSIT-(SP) PID=0.0 ppm	
							17				
							20				
							33				
							40				
15	5D	24/10	15.0 - 17.0	3/3/3/3	6	8	8			Grey-brown to light-brown, wet, loose, fine SAND, little silt, poorly-graded -GLACIAL LACUSTRINE DEPOSIT-(SP-SM)	
							6				
							8				
							8				
							24	692.6		Note: Drill action indicates gravel encountered at 19.5 ft. 19.5	
20	6D	24/5	20.0 - 22.0	28/17/17/14	34	43	18			Grey, wet, dense, fine to coarse GRAVEL, little fine sand, trace medium sand, poorly-graded -GLACIAL TILL-(GP) PID=0.0 ppm	
							23				
							27				
							25				
							33				
25											

Remarks:

PID = photoionization detector

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Route 26 Highway Rehabilitation			**Boring No.:**	HB-WOOD-214
Location:	Woodstock, Maine			**WIN:**	18767.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	712.1	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	K. Russ	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	SS-140#/30;HW+NW-300#/
Date Start/Finish:	9-1-17/9-1-17	**Drilling Method:**	SSA/HW/NW Drive	**Core Barrel:**	NQ 2.0 in.
Boring Location:	STA 60+77.3, 80.9 ft LT	**Casing ID/OD:**	HW-4.0 in. ID/NW-3.0 in. ID	**Water Level*:**	18.3 ft
Hammer Efficiency Factor:	0.750	**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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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/5	25.0 - 27.0	8/6/5/5	11	14	17			Grey, wet, medium dense, Gravelly SAND, well graded, loosely bonded -GLACIAL TILL-(SW) PID=0.0 ppm	
							20				
								684.0		28.1	
	R1	60/50	28.5 - 33.5	RQD = 70%			NQ Core			Top of Bedrock at El.684.0 R1: Grey, fine to medium grained, GRANODIORITE. Very hard, fresh. Joints dipping moderately, close to moderately close, tight to open, rough, planar, occasional oxidized joint surfaces. Rock Quality=Fair Recovery=83% -SONGO GRANODIORITE FORMATION- R1 Core Times (min:sec): 28.5-29.5' (1:57); 29.5-30.5' (1:46); 30.5-31.5' (1:53); 31.5-32.5' (2:15); 32.5-33.5' (2:09)	
30											
								678.6		33.5	
										Bottom of Exploration at 33.5 feet below ground surface.	
35											
40											
45											
50											

Remarks:

PID = photoionization detector

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Top Row: HB-WOOD-208, Run No. R1, 13.0 ft (left) to 15.3 ft (middle), Run No. R2, 19.0 ft (middle) to 24.0 ft (right); **Top Middle Row**: HB-WOOD-208, Run No. R3, 35.0 ft (left) to 40.0 ft (right); **Bottom Middle Row**: HB-WOOD-210, Run No. R1, 14.3 ft (left) to 18.0 ft (right); **Bottom Row**: HB-WOOD-210, Run No. R2, 18.3 ft (left) to 23.3 ft (right)

Top Row: HB-WOOD-212, Run No. R1, 5.0 ft (left) to 10.0 ft (right); **Top Middle Row:** HB-WOOD-214, Run No. R1, 28.5 ft (left) to 33.5 ft (right)

Top Row: See Woodstock Culvert Report; **Top Middle Row**: See Woodstock Culvert Report; **Bottom Middle Row**: HB-WOOD-211, Run No. R1, 15.0 ft (left) to 19.8 ft (right); **Bottom Row**: HB-WOOD-207, Run No. R1, 9.6 ft (left) to 12.6 ft (middle); HB-WOOD-207, Run No. R2, 25.5 ft (middle) to 31.0 ft (right)

Top Row: HB-WOOD-205, Run No. R1, 15.2 ft (left) to 20.0 ft (middle); HB-WOOD-209, Run No. R1, 13.5 ft (middle) to 14.0 ft (right); **Top Middle Row**: HB-WOOD-209, Run No. R2, 19.2 ft (left) to 24.2 ft (right); **Bottom Middle Row**: HB-WOOD-213, Run No. R1, 14.5 ft (left) to 19.5 ft (right); **Bottom Row**: HB-WOOD-206, Run No. R1, 12.3 ft (left) to 17.3 ft (right)

APPENDIX B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Woodstock   Work Number: 18767.00

SEE CULVERT REPORT FOR REMAINING LABORATORY ANALYSIS RESULTS

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
HB-WOOD-201, 2D	13+15.7	9.9 Rt.	2.5-4.5	300176	1	4.4			SP-SM	A-1-a	0
HB-WOOD-201, 3D	13+15.7	9.9 Rt.	4.5-6.5	300177	1	13.5			SM	A-1-b	II
HB-WOOD-201, 5D	13+15.7	9.9 Rt.	8.5-10.5	300178	1	66.8			SM	A-2-4	II
HB-WOOD-201, 6D	13+15.7	9.9 Rt.	10.5-12.5	300179	1	36.2			SM	A-4	III
HB-WOOD-201, 7D	13+15.7	9.9 Rt.	12.5-14.5	300180	2	30.6			SM	A-2-4	II
HB-WOOD-201, 8D	13+15.7	9.9 Rt.	15.0-17.0	300181	2	21.3			SM	A-2-4	II
HB-WOOD-201, 9D	13+15.7	9.9 Rt.	20.0-22.0	300182	2	19.3			SM	A-2-4	II
HB-WOOD-202, 1D	13+81.2	12.4 Lt.	0.0-2.0	300183	3	4.3			SW-SM	A-1-b	0
HB-WOOD-202, 2D	13+81.2	12.4 Lt.	2.0-4.0	300184	3	2.8			GW-GM	A-1-a	0
HB-WOOD-202, 3D/4D	13+81.2	12.4 Lt.	4.0-6.5	300185	3	12.3			SM	A-2-4	II
HB-WOOD-202, 4D/A	13+81.2	12.4 Lt.	6.5-8.0	300186	3	37.9			SM	A-2-4	II
HB-WOOD-202, 5D	13+81.2	12.4 Lt.	10.0-12.0	300187	4	33.9			SM	A-4	III
HB-WOOD-202, 7D	13+81.2	12.4 Lt.	15.0-17.0	300188	4	20.3			SM	A-2-4	II
HB-WOOD-202, 8D	13+81.2	12.4 Lt.	20.0-22.0	300189	4	19.6			CL	A-4	IV
HB-WOOD-203, 2D	24+62.9	18.7 Rt.	2.5-4.5	300190	5	4.3			SW-SM	A-1-b	0
HB-WOOD-203, 3D	24+62.9	18.7 Rt.	4.5-6.5	300191	5	13.1			SW-SM	A-1-b	0
HB-WOOD-203, 4D	24+62.9	18.7 Rt.	6.5-8.5	300192	5	59.3			SP-SM	A-3	0
HB-WOOD-203, 5D	24+62.9	18.7 Rt.	8.5-9.5	300193	5	56.2			SM	A-2-4	II
HB-WOOD-203, 5D/A	24+62.9	18.7 Rt.	9.5-10.5	300194	5	25.5			SP-SM	A-3	0
HB-WOOD-203, 6D	24+62.9	18.7 Rt.	15.0-17.0	300195	5	21.6			SM	A-2-4	II
HB-WOOD-204, 1D	25+20.9	13.9 Lt.	1.5-3.5	300196	6	4.5			SM	A-1-b	II
HB-WOOD-204, 2D	25+20.9	13.9 Lt.	3.5-5.5	300197	6	6.6			SM	A-1-b	II
HB-WOOD-204, 4D	25+20.9	13.9 Lt.	7.5-9.0	300198	6	166			SP-SM	A-2-4	0
HB-WOOD-204, 4D/A-5D	25+20.9	13.9 Lt.	9.0-11.5	300199	6	42.0			SM	A-2-4	II
HB-WOOD-204, 6D	25+20.9	13.9 Lt.	15.0-17.0	300200	6	25.1			SP-SM	A-3	0
HB-WOOD-204, 7D	25+20.9	13.9 Lt.	20.0-22.0	300101	6	23.7			SM	A-2-4	II
HB-WOOD-205, 1D	53+07.2	2.5 Rt.	1.0-3.0	300102	7	4.8			SM	A-1-b	II
HB-WOOD-205, 2D	53+07.2	2.5 Rt.	3.0-5.0	300103	7	9.6			SM	A-4	III
HB-WOOD-205, 3D	53+07.2	2.5 Rt.	5.0-7.0	300104	7	8.0			SM	A-2-4	II
HB-WOOD-205, 4D	53+07.2	2.5 Rt.	7.0-9.0	300105	7	6.9			SM	A-2-4	II
HB-WOOD-207, 1D	54+93.5	16.2 Lt.	1.0-3.0	300106	8	3.3			SM	A-1-b	II
HB-WOOD-207, 2D	54+93.5	16.2 Lt.	3.0-4.0	300107	8	3.8			SM	A-1-b	II
HB-WOOD-207, 2D/A-3D	54+93.5	16.2 Lt.	4.0-7.0	300109	8	11.4			SM	A-2-4	II
HB-WOOD-207, 4D	54+93.5	16.2 Lt.	7.0-9.0	300110	8	15.4			SM	A-2-4	II
HB-WOOD-209, 1D	56+87.6	1.9 Lt.	1.0-2.7	300111	9	1.6			SM	A-1-b	II
HB-WOOD-209, 2D	56+87.6	1.9 Lt.	3.0-5.0	300112	9	12.0			SM	A-2-4	II
HB-WOOD-209, 3D	56+87.6	1.9 Lt.	5.5-7.5	300113	9	3.9			SM	A-1-b	II
HB-WOOD-209, 4D	56+87.6	1.9 Lt.	7.5-9.5	300114	9	10.1			SM	A-2-4	II
HB-WOOD-211, 1D	58+81	15.8 Lt.	1.2-3.2	300115	10	6.6			SM	A-1-b	II
HB-WOOD-211, 2D	58+81	15.8 Lt.	3.5-4.8	300116	10	2.7			SM	A-1-b	II
HB-WOOD-211, 3D	58+81	15.8 Lt.	7.5-9.5	300117	10	7.5			SM	A-2-4	II

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98   NP = Non Plastic

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

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Laboratory Testing Summary Sheet

Town(s): Woodstock Work Number: 18767.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	AASHTO	Frost
HB-WOOD-213, 1D	60+77.4	3.0 Lt.	1.0-3.0	300118	10	2.3			SW-SM	A-1-a	0
HB-WOOD-213, 2D	60+77.4	3.0 Lt.	3.0-5.0	300119	10	3.1			SM	A-1-b	II
HB-WOOD-213, 3D	60+77.4	3.0 Lt.	5.0-7.0	300120	10	5.2			SM	A-1-b	II
HB-WOOD-215, 1D	123+85.2	15.4 Rt.	0.0-2.0	300121	11	7.8			SM	A-1-b	II
HB-WOOD-215, 3D/A	123+85.2	15.4 Rt.	4.6-6.0	300122	11	45.1			SM	A-2-4	II
HB-WOOD-215, 4D	123+85.2	15.4 Rt.	8.5-10.5	300123	11	24.2			SM	A-2-4	II
HB-WOOD-216, 1D	123+88.6	14.6 Lt.	0.0-2.0	300124	12	7.5			SM	A-1-b	II
HB-WOOD-216, 3D	123+88.6	14.6 Lt.	4.0-6.0	300125	12	34.7			SP-SM	A-2-4	0
HB-WOOD-216, 4D/A	123+88.6	14.6 Lt.	6.5-8.0	302626	12	28.1			SP-SM	A-3	0
HB-WOOD-216, 5D	123+88.6	14.6 Lt.	8.0-10.0	302627	12	21.9			SM	A-2-4	II
HB-WOOD-216, 6D	123+88.6	14.6 Lt.	10.0-12.0	302628	12	23.7			SM	A-2-4	II

SEE CULVERT REPORT FOR REMAINING LABORATORY ANALYSIS RESULTS

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	Loss of Ignition, % (T 267)					
HB-WOOD-201, 5D	13+15.7	9.9 Rt.	8.5-10.5	300178	1				8.8		
HB-WOOD-201, 6D	13+15.7	9.9 Rt.	10.5-12.5	300179	1				3.0		
HB-WOOD-201, 7D	13+15.7	9.9 Rt.	12.5-14.5	300180	2				1.8		
HB-WOOD-201, 8D	13+15.7	9.9 Rt.	15.0-17.0	300181	2				0.4		
HB-WOOD-202, 4D/A	13+81.2	12.4 Lt.	6.5-8.0	300186	3				9.4		
HB-WOOD-202, 5D	13+81.2	12.4 Lt.	10.0-12.0	300187	4				3.8		
HB-WOOD-203, 4D	24+62.9	18.7 Rt.	6.5-8.5	300192	5				8.1		
HB-WOOD-203, 5D	24+62.9	18.7 Rt.	8.5-9.5	300193	5				6.4		
HB-WOOD-203, 5D/A	24+62.9	18.7 Rt.	9.5-10.5	300194	5				1.5		
HB-WOOD-204, 4D	25+20.9	13.9 Lt.	7.5-9.0	300198	6				54.5		
HB-WOOD-204, 4D/A-5D	25+20.9	13.9 Lt.	9.0-11.5	300199	6				6.3		
HB-WOOD-215, 3D/A	123+85.2	15.4 Rt.	4.6-6.0	300122	11				23.3		
HB-WOOD-216, 3D	123+88.6	14.6 Lt.	4.0-6.0	300125	12	Ignition Test not run.					

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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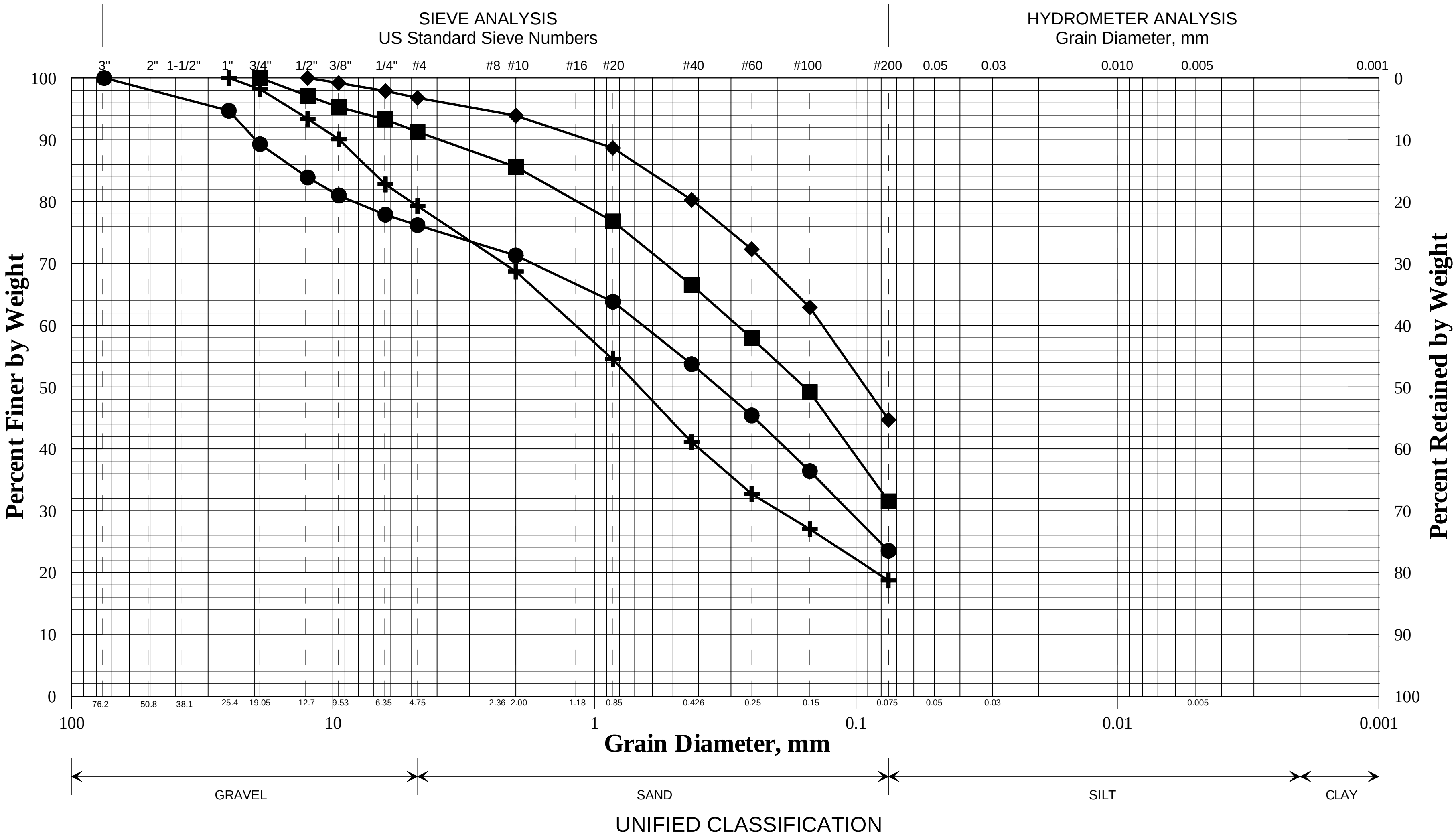
PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	HB-WOOD-205/1D	53+07.2	2.5 RT	1.0-3.0	SAND, some gravel, little silt.	4.8			
◆	HB-WOOD-205/2D	53+07.2	2.5 RT	3.0-5.0	Silty SAND, trace gravel.	9.6			
■	HB-WOOD-205/3D	53+07.2	2.5 RT	5.0-7.0	SAND, some silt, trace gravel.	8.0			
●	HB-WOOD-205/4D	53+07.2	2.5 RT	7.0-9.0	SAND, some gravel, some silt.	6.9			
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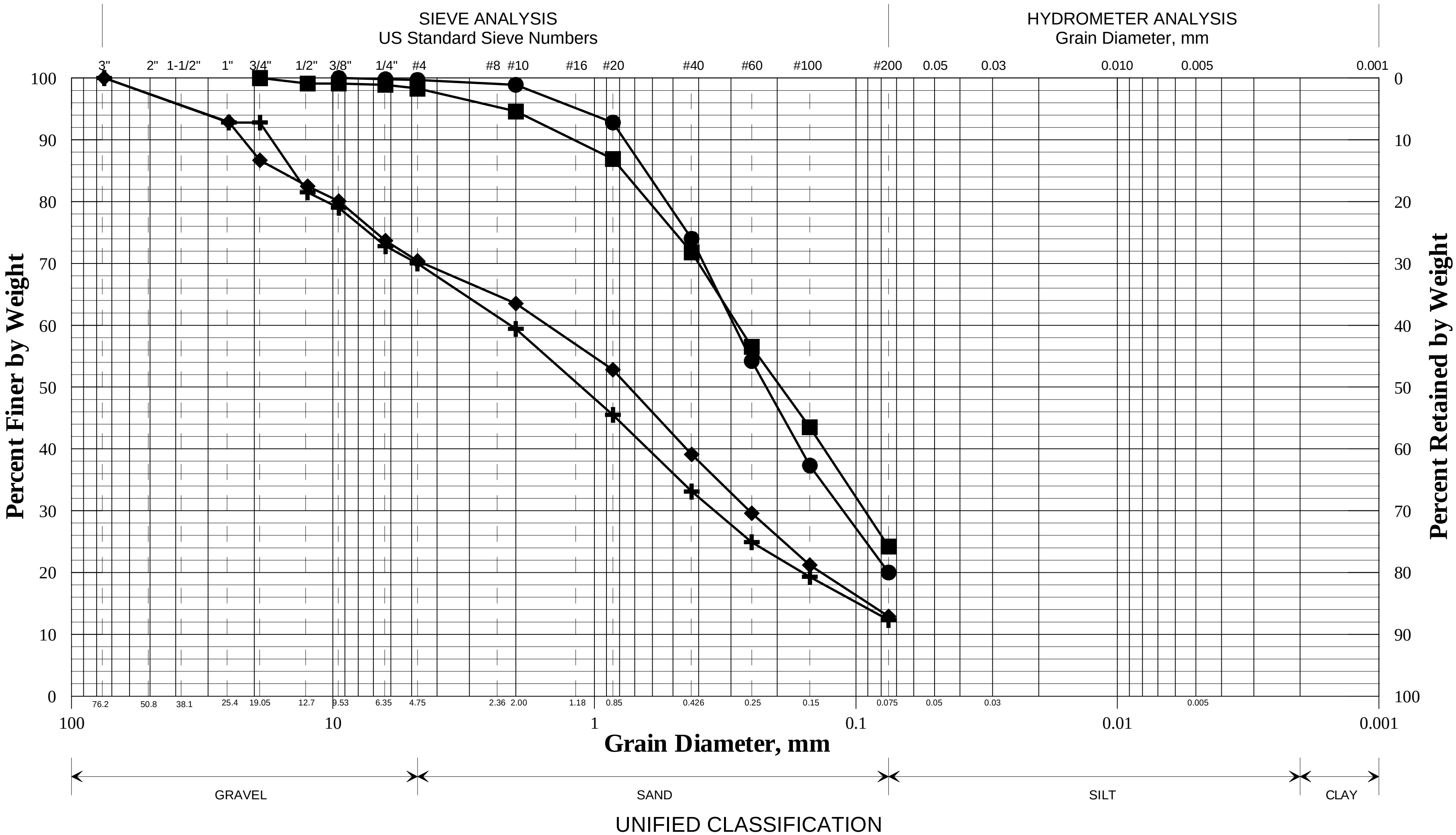
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	HB-WOOD-207/1D	54+93.5	16.2 LT	1.0-3.0	SAND, some gravel, little silt.	3.3			
◆	HB*-WOOD-207/2D	54+93.5	16.2 LT	3.0-4.0	SAND, some gravel, little silt.	3.8			
■	HB-WOOD-207/2DA+3D	54+93.5	16.2 LT	4.0-7.0	SAND, some silt, trace gravel.	11.4			
●	HB-WOOD-207/4D	54+93.5	16.2 LT	7.0-9.0	SAND, little silt, trace gravel.	15.4			
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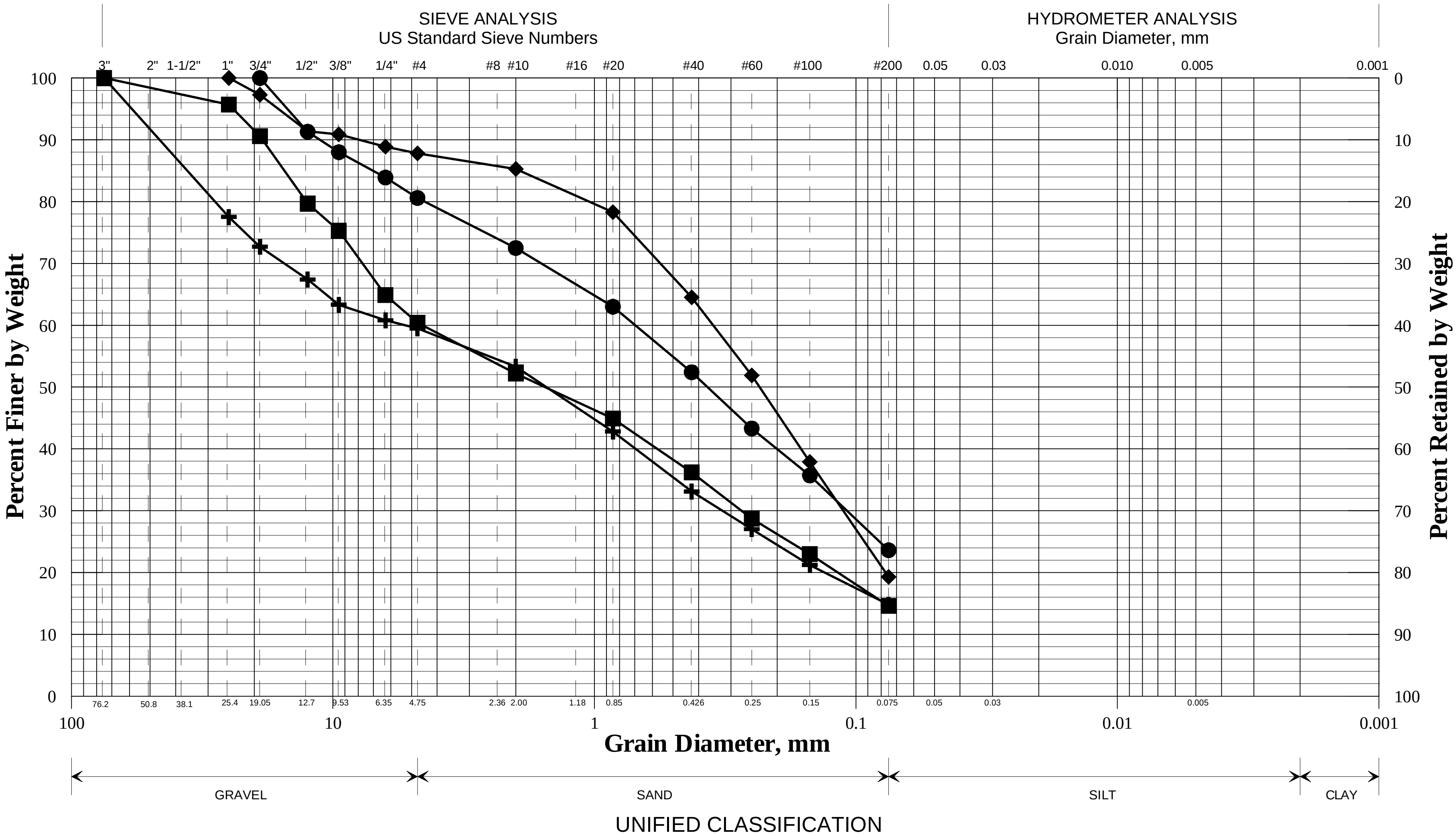
WHITE, TERRY A 10/4/2017

SHEET 8

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	HB-WOOD-209/1D	56+87.6	1.9 LT	1.0-2.7	Gravelly SAND, little silt.	1.6			
◆	HB-WOOD-209/2D	56+87.6	1.9 LT	3.0-5.0	SAND, little silt, little gravel.	12.0			
■	HB-WOOD-209/3D	56+87.6	1.9 LT	5.5-7.5	Gravelly SAND, little silt.	3.9			
●	HB-WOOD-209/4D	56+87.6	1.9 LT	7.5-9.5	SAND, some silt, little gravel.	10.1			
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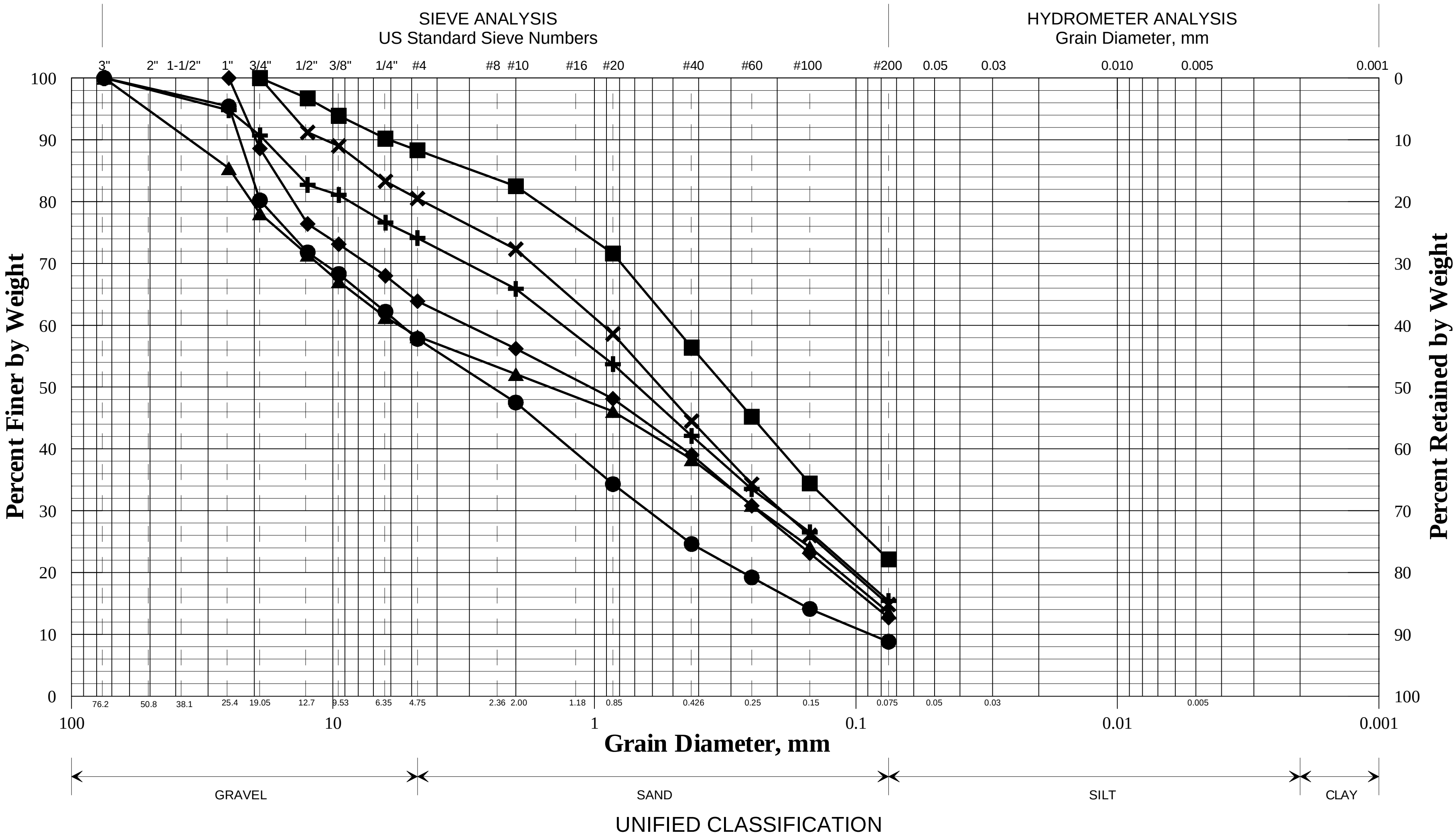
Reported by/Date
WHITE, TERRY A 10/4/2017

SHEET 9

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	HB-WOOD-211/1D	58+81	15.8 LT	1.2-3.2	SAND, some gravel, little silt.	6.6			
◆	HB-WOOD-211/2D	58+81	15.8 LT	3.5-4.8	Gravelly SAND, little silt.	2.7			
■	HB-WOOD-211/3D	58+81	15.8 LT	7.5-9.5	SAND, some silt, little gravel.	7.5			
●	HB-WOOD-213/1D	60+77.4	3.0 LT	1.0-3.0	Gravelly SAND, trace silt.	2.3			
▲	HB-WOOD-213/2D	60+77.4	3.0 LT	3.0-5.0	Gravelly SAND, little silt.	3.1			
✖	HB-WOOD-213/3D	60+77.4	3.0 LT	5.0-7.0	SAND, little gravel, little silt.	5.2			

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Woodstock	
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WHITE, TERRY A	10/4/2017

APPENDIX C

Working Cross Sections

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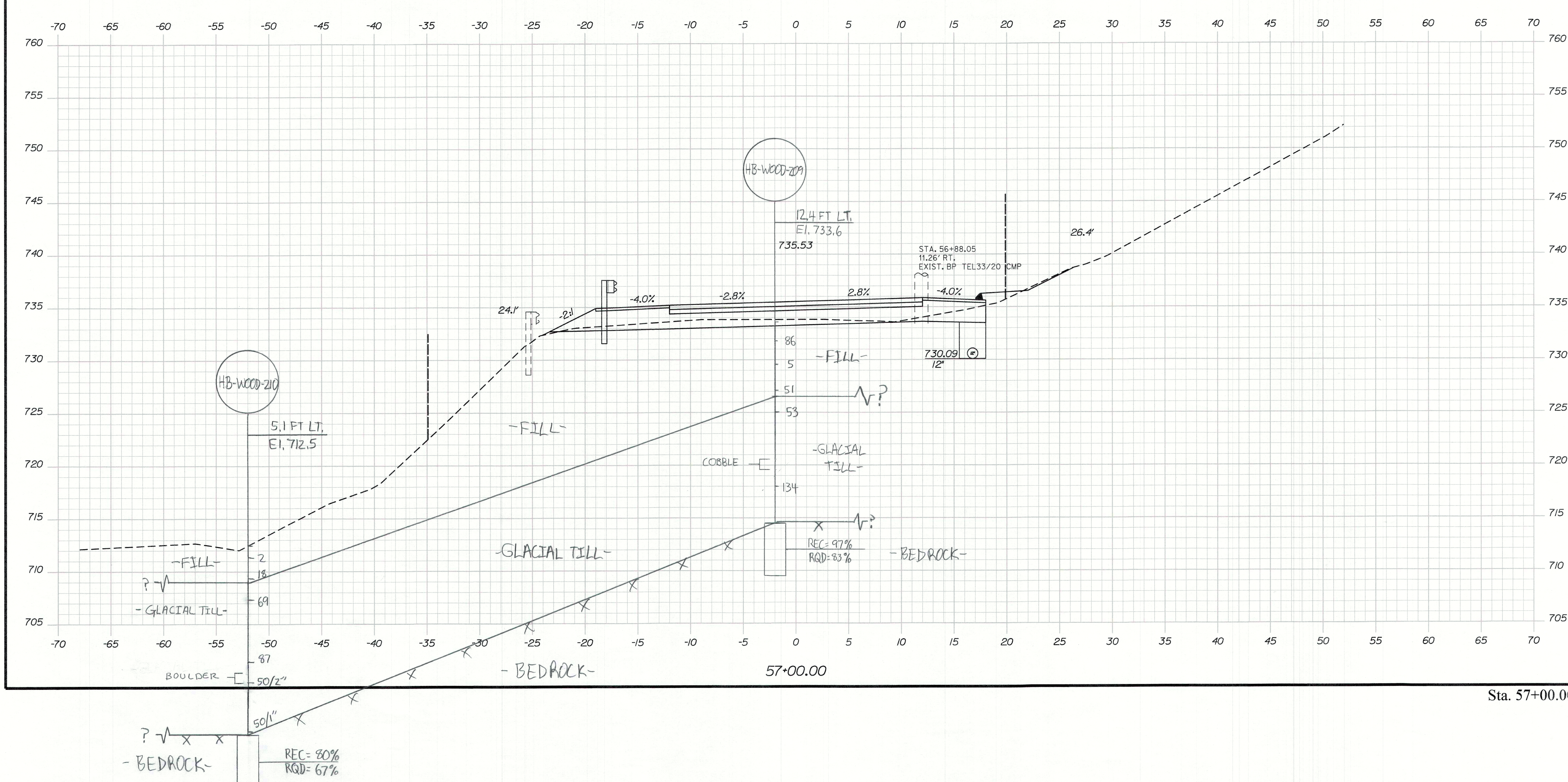

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

NHPP-1876(700)

WIN
WIN 018767.00

HIGHWAY PLANS

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CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	$designer2$	$detailer2$	$projdate2$
DESIGN3-DETAILED3	$designer3$	$detailer3$	$projdate3$
REVISIONS 1	-----		$revdate1$
REVISIONS 2			$revdate2$
REVISIONS 3	$revision3$		$revdate3$
REVISIONS 4	$revision4$		$revdate4$
FIELD CHANGES	$fieldchange$		$fielddate$

SIGNATURE
P.E. NUMBER
DATE

WOODSTOCK
ROUTE 26

CROSS SECTIONS

SHEET NUMBER
95
OF _

Sta. 57+00.00 to Sta. 57+00.00

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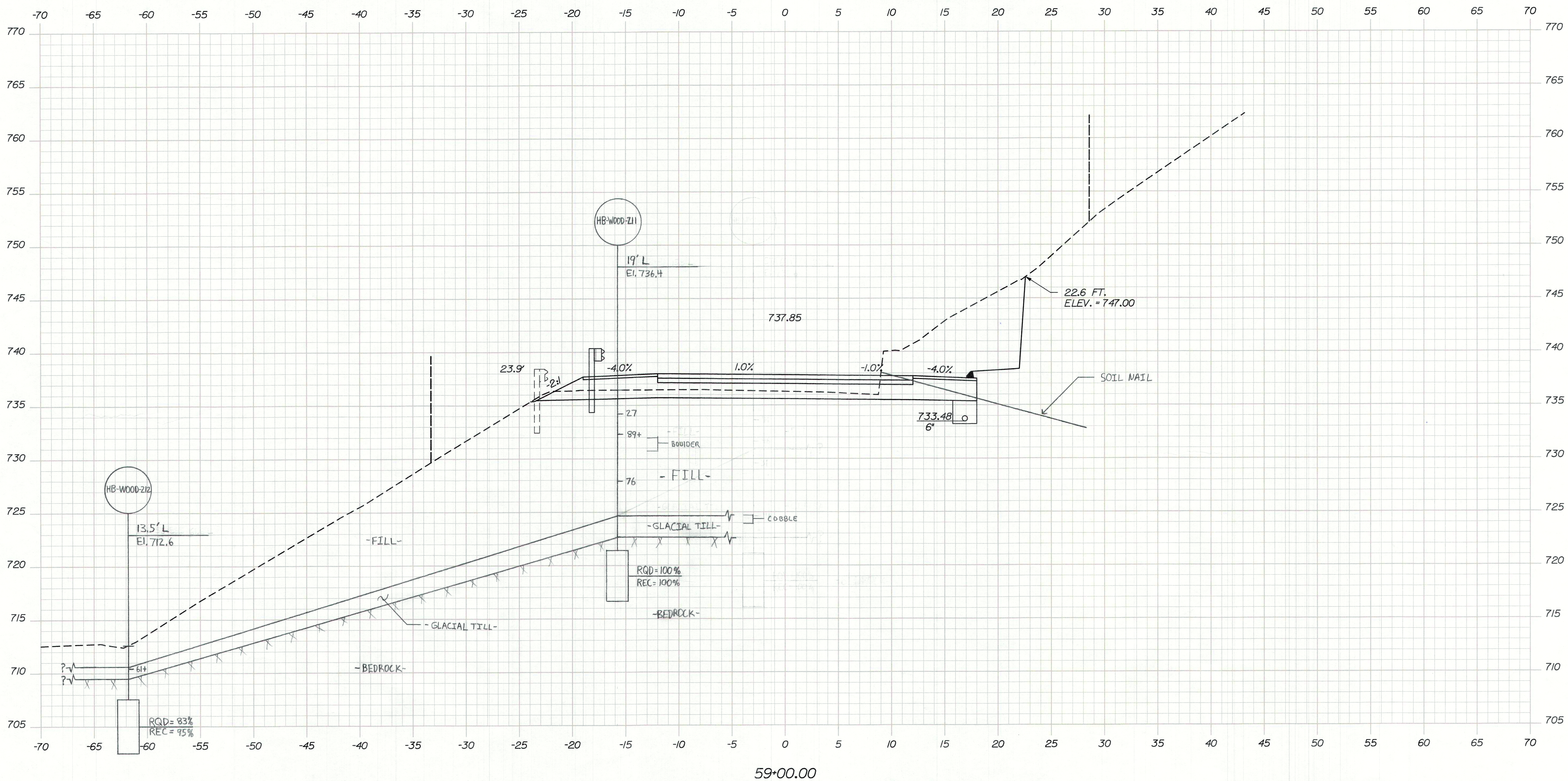

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

NHPP-1876(700)

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HIGHWAY PLANS

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CHECKED-REVIEWED			
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SIGNATURE

P.E. NUMBER

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WOODSTOCK
ROUTE 26

CROSS SECTIONS

SHEET NUMBER
98
OF _

Sta. 59+00.00 to Sta. 59+00.00

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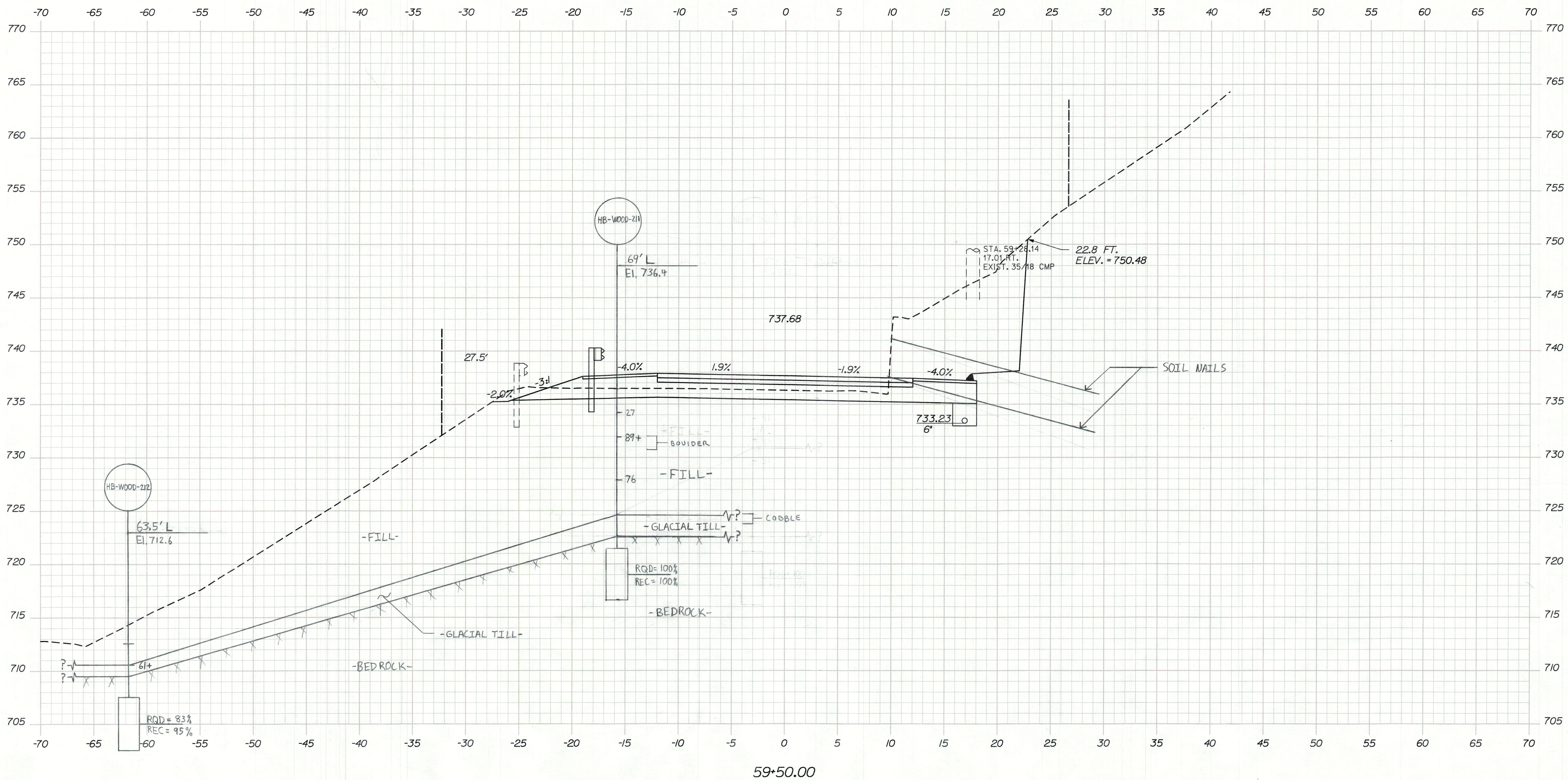

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

NHPP-1876(700)

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HIGHWAY PLANS

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DESIGN3-DETAILED3	$designer3$	$detailer3$	$projdate3$
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REVISIONS 3	$revision3$		$revdate3$
REVISIONS 4	$revision4$		$revdate4$
FIELD CHANGES	$fieldchange$		$fielddate$

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DATE

WOODSTOCK
ROUTE 26

CROSS SECTIONS

SHEET NUMBER
99
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Sta. 59+50.00 to Sta. 59+50.00

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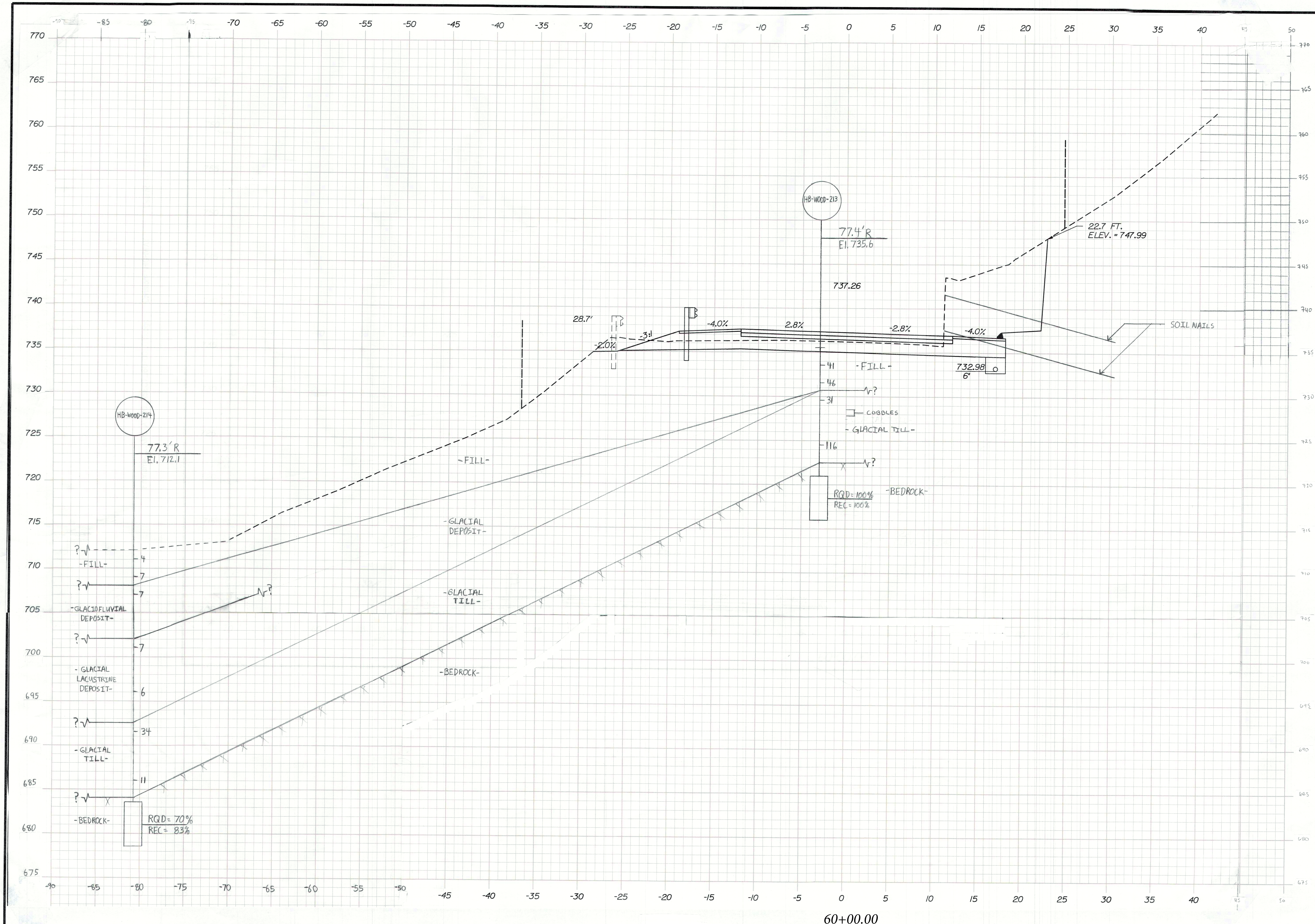

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

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HIGHWAY PLANS

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CHECKED-REVIEWED	$designer2$	$detailer2$	$projdate2$
DESIGN2-DETAILED2	$designer2$	$detailer2$	$projdate2$
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REVISIONS 1			$revdate1$
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REVISIONS 3	$revision3$		$revdate3$
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FIELD CHANGES	$fieldchange$		$fielddate$

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WOODSTOCK
ROUTE 26

CROSS SECTIONS

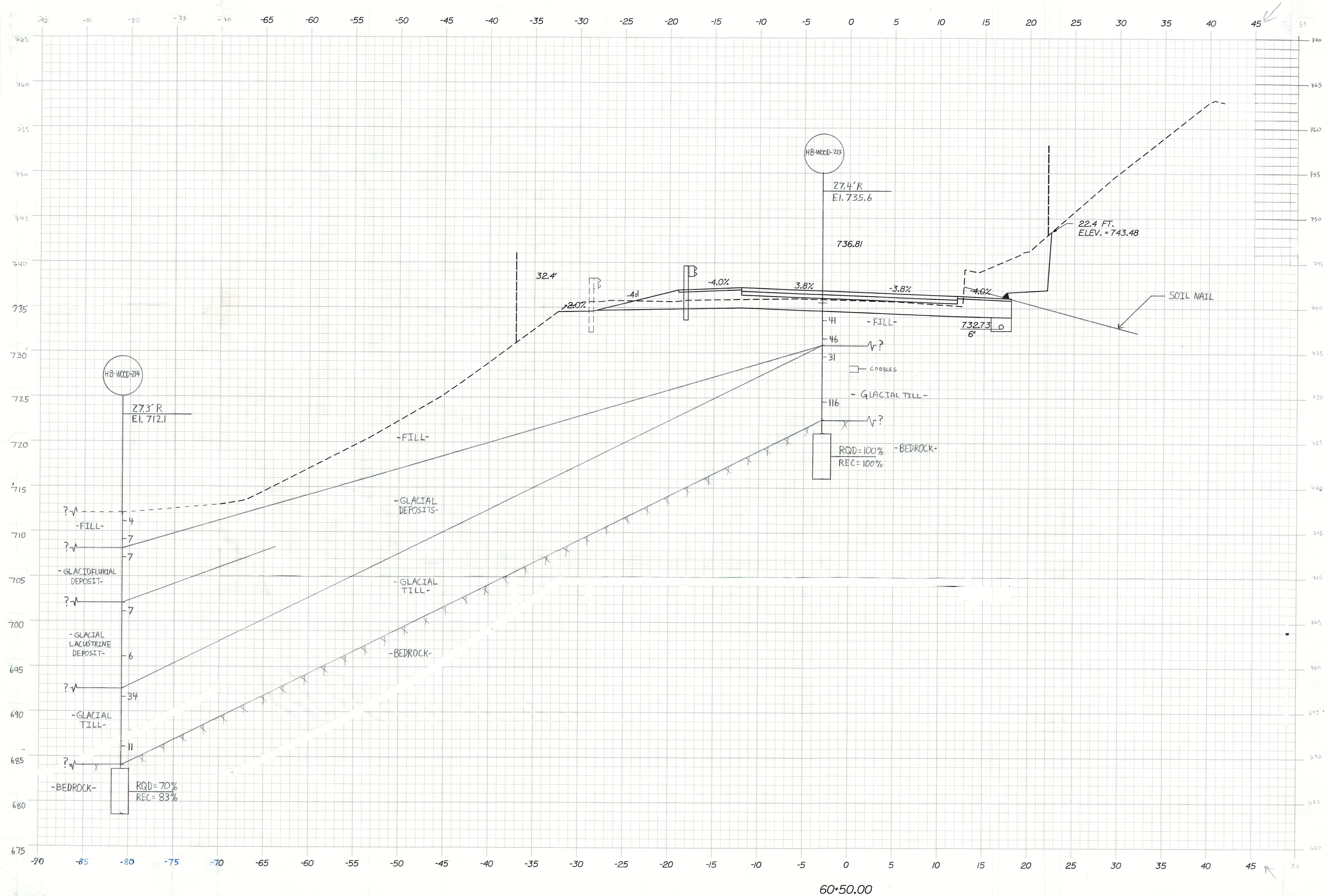
SHEET NUMBER

100

OF _

Sta. 60+50.00 to Sta. 60+50.00

Filename: ...\MSTA100_XSECT_60+00_015.dgn Division: HIGHWAY Username: Clark.E.Sulloway Date:2/14/2019



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

NHPP-1876(700)

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WIN 018767.00

HIGHWAY PLANS

	BY	DATE
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FIELD CHANGES	$fieldchange$	$fielddate$

SIGNATURE
P.E. NUMBER
DATE

WOODSTOCK
ROUTE 26

CROSS SECTIONS

SHEET NUMBER
101
OF _

Sta. 60+00.00 to Sta. 60+00.00

APPENDIX D

Geotechnical Calculations

Seismic Site Class

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using SPT N-values from test borings drilled near the proposed retaining wall in the vicinity of Sta. 58+50 to Sta. 60+75.

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at four nearby test borings (HB-WOOD 211 through HB-WOOD-214), recommend a **Seismic Site Class C**.

REFERENCES

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (2012 Interim Revisions).
2. AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017.
3. International Building Code 2009.
4. ASCE/SEI 7-05 Minimum Design Loads For Buildings and Other Structures.
5. International Building Code 2012.
6. ASCE/SEI 7-10 Minimum Design Loads For Buildings and Other Structures.

AVAILABLE INFORMATION

1. Boring logs dated 24, 29, and 31 August 2017 and 1 September 2017 drilled by New England Boring Contractors, Inc. (monitored by Haley & Aldrich, Inc.).
2. Elevations reference the North American Vertical Datum of 1988 (NAVD 88).

ASSUMPTIONS

1. Where SPT N-value was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N-values for the extended profile were then assumed based on the available information.
2. WOH/WOR = SPT N-value of 1.

PROCEDURE

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

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

$$\overline{V_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of *i* th soil (ft/s).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

PROCEDURE

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

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th soil layer not to exceed 100 ft (blows/ft).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

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

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th cohesionless soil layer (blows/ft).
d_i = thickness of *i* th cohesionless soil layer (ft).
m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and m.

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

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

where

s_{ui} = undrained shear strength of *i* th cohesive soil layer (psf), not to exceed 5000 psf
d_i = thickness of *i* th cohesive soil layer (ft).
k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and k.

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

SITE CLASS DEFINITIONS

(Table from AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (with 2012 Interim Revisions)).

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with 2500 ft/sec < $\bar{v}_s < 5000$ ft/sec
C	Very dense soil and soil rock with 1200 ft/sec < $\bar{v}_s$ < 2500 ft/sec, or with either $\bar{N}$ > 50 blows/ft or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1200 ft/sec, or with either 15 blows/ft < $\bar{N}$ < 50 blows/ft or 1.0 ksf < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/sec, or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40%, and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific ground motion response evaluations, such as: • Peats or highly organic clays (H > 10 ft of peat or highly organic clay, where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H > 120 ft)

Exceptions:

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

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

	CALCULATIONS	File No.	130458-002
		Sheet	4 of 8
Client	Maine Department of Transportation	Date	13-May-19
Project	Route 26 Improvements, MaineDOT WIN 018767.00	Computed by	KAR
Subject	Seismic Site Class Evaluation	Checked by	EAF

CALCULATIONS - METHOD B

Exploration ID: HB-WOOD-211
Ground Surface El.: 736.4

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	2.2	734.2	SAND (Fill)	3.4	27	0.126
2D	4.2	732.2	SAND (Fill)	2.8	89	0.031
3D	8.5	727.9	SAND (Fill)	7.5	76	0.099
R1	13.7	722.7	BEDROCK	86.3	100	0.863
			Totals =	100.0		1.119

N-bar (blows/ft) = 89.4

Site Class = C

HALEY ALDRICH	CALCULATIONS	File No.	130458-002
		Sheet	5 of 8
Client	Maine Department of Transportation	Date	13-May-19
Project	Route 26 Improvements, MaineDOT WIN 018767.00	Computed by	KAR
Subject	Seismic Site Class Evaluation	Checked by	EAF

CALCULATIONS - METHOD B

Exploration ID: HB-WOOD-212
Ground Surface El.: 712.6

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	2.2	712.6	FILL/GLACIAL TILL	2.9	61	0.048
R1	2.9	709.7	BEDROCK	97.1	100	0.971
			Totals =	100.0		1.019

N-bar (blows/ft) = 98.2
Site Class = C

HALEY ALDRICH	CALCULATIONS	File No.	130458-002
		Sheet	6 of 8
Client	Maine Department of Transportation	Date	13-May-19
Project	Route 26 Improvements, MaineDOT WIN 018767.00	Computed by	KAR
Subject	Seismic Site Class Evaluation	Checked by	EAF

CALCULATIONS - METHOD B

Exploration ID: HB-WOOD-213
Ground Surface El.: 735.6

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	2.0	733.6	SAND (Fill)	3.0	41	0.073
2D	4.0	731.6	SAND (Fill)	1.7	46	0.037
3D	6.0	729.6	SAND (Glacial Till)	3.8	31	0.123
4D	11.0	724.6	SAND (Glacial Till)	4.6	116	0.040
R1	13.1	722.5	BEDROCK	86.9	100	0.869
			Totals =	100.0		1.141

N-bar (blows/ft) = 87.6
Site Class = C

HALEY ALDRICH	CALCULATIONS	File No.	130458-002
		Sheet	7 of 8
Client	Maine Department of Transportation	Date	13-May-19
Project	Route 26 Improvements, MaineDOT WIN 018767.00	Computed by	KAR
Subject	Seismic Site Class Evaluation	Checked by	

CALCULATIONS - METHOD B

Exploration ID: HB-WOOD-214
Ground Surface El.: 712.1

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1.0	711.1	SAND (Fill)	2.0	4	0.500
2D	3.0	709.1	SAND (Fill)	2.0	7	0.286
3D	5.0	707.1	AND (Glacialfluvia	6.0	7	0.857
4D	11.0	701.1	ND (Glacial Lacustr	3.5	7	0.500
5D	16.0	696.1	ND (Glacial Lacustr	6.0	6	1.000
6D	21.0	691.1	RAVEL (Glacial Til	4.0	34	0.118
7D	26.0	686.1	SAND (Glacial Till)	4.6	11	0.418
R1	28.1	684	BEDROCK	71.9	100	0.719
			Totals =	100.0		4.398

N-bar (blows/ft) = 22.7

Site Class = D

HALEY ALDRICH	CALCULATIONS	File No.	130458-002
		Sheet	8 of 8
Client	Maine Department of Transportation	Date	13-May-19
Project	Route 26 Improvements, MaineDOT WIN 018767.00	Computed by	KAR
Subject	Seismic Site Class Evaluation	Checked by	

RESULTS SUMMARY

Boring Number	Site Class
HB-WOOD-211	C
HB-WOOD-212	C
HB-WOOD-213	C
HB-WOOD-214	D

CONCLUSIONS & RECOMMENDATIONS

Based on the above results, recommend a Seismic Site Class C.

Embankment Settlement

HALEY ALDRICH	CALCULATIONS	File No.	130458-002
		Sheet	1 of 2
Client	Maine Department of Transportation	Date	13-May-19
Project	Route 26 Improvements, Woodstock, Maine	Computed by	EAF
Subject	Embankment Settlement	Checked by	KAR

PROBLEM STATEMENT & OBJECTIVE

Estimate the elastic settlement for the proposed roadway raise-in-grade.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017.

AVAILABLE INFORMATION

1. Draft plan set titled, "Woodstock, Oxford County, Route 26, Federal Aid Project No. NHPP-1876(700)" by MaineDOT.
2. Boring logs HB-WOOD-207, HB-WOOD-209, HB-WOOD-211, and HB-WOOD-213.

ASSUMPTIONS

1. Width of additional fill embankment at Station 57+50 is about 36 ft.
2. Length of additional fill embankment is about 700 ft (Sta. 54+00 to Sta. 61+00).
3. Greatest fill height at Station 57+50 is about 2 ft.
4. New embankment fill unit weight is 125 pcf.

LOADING

1. Additional pressure is 2 ft * 125 pcf = 250 psf = 0.250 ksf.

ITERATIONS

1. Sta. 57+50 (width = 36 ft, length = 700 ft, pressure = 0.250 ksf)

CALCULATIONS

See page 2.

CONCLUSIONS

Elastic settlements will be less than 1/2 in. from the new fill placement.

CALCULATION:

$$S_e = \frac{q_0(1-v^2)\sqrt{A'}}{144\, E_s \beta_z}$$ Equation 10.6.2.4.2-1

where

q_0 = applied vertical stress (ksf)

A' = effective area of footing (ft^2) *** use full area

E_s = Young's Modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of Es are not available from the results of insitu or laboratory tests (ksi)

β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)

v = Poisson's Ratio, taken as specified in Article 10.4.6.3 if direct measurements of v are not available from the results of insitu or laboratory tests (dim)

S_e = elastic settlement (ft)

FIXED SERVICE LIMIT STATE PARAMETERS

E_s (ksi)	5	ksi from Table C10.4.6.3-1 (med. dense sand)
v	0.3	Table C10.4.6.3-1
L (ft)	700	
β_z	1.41	Table 10.6.2.4.2-1
qo	0.25	ksf

VARIABLE SERVICE LIMIT STATE PARAMETERS

B (ft)	A (ft^2)	S_e (in.)
36	25200	0.43

Table 10.6.2.4.2-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)

Soil Type	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, v (dim)
Clay:		
Soft sensitive	0.347–2.08	0.4–0.5 (undrained)
Medium stiff to stiff	2.08–6.94	
Very stiff	6.94–13.89	
Loess	2.08–8.33	0.1–0.3
Silt	0.278–2.78	0.3–0.35
Fine Sand:		
Loose	1.11–1.67	0.25
Medium dense	1.67–2.78	
Dense	2.78–4.17	
Sand:		
Loose	1.39–4.17	0.20–0.36
Medium dense	4.17–6.94	
Dense	6.94–11.11	0.30–0.40
Gravel:		
Loose	4.17–11.11	0.20–0.35
Medium dense	11.11–13.89	
Dense	13.89–27.78	0.30–0.40
Estimating E_s from $SPT\ N$ Value		
Soil Type		E_s (ksi)
Silts, sandy silts, slightly cohesive mixtures		$0.056\ N1_{60}$
Clean fine to medium sands and slightly silty sands		$0.097\ N1_{60}$
Coarse sands and sands with little gravel		$0.139\ N1_{60}$
Sandy gravel and gravels		$0.167\ N1_{60}$
Estimating E_s from q_c (static cone resistance)		
Sandy soils		$0.028 q_c$

Embankment Stability

HALEY ALDRICH	CALCULATIONS	File No.:	130458-002
		Sheet:	1 of 1
Client:	Maine Department of Transportation	Date:	7-May-2019
Project:	Route 26 Improvements, MaineDOT WIN 018767.00	Computed by:	KAR
Subject:	Global Stability	Checked by:	EAF

PROBLEM STATEMENT AND OBJECTIVE

Calculate the global stability minimum factor of safety for the existing and proposed embankments

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017.
2. Slide version 8.0 by RocScience.
3. Maine DOT Bridge Design Guide, 2003, with 2014 updates.

AVAILABLE INFORMATION

1. Plan set titled, "Woodstock, Oxford County, Route 26, Federal Aid Project No. NHPP-1876(700)" by MaineDOT

ASSUMPTIONS

1. Water level will be modeled at the top of glacial till.
2. Seismic cases will have a seismic force of As/2 (0.109g/2) = 0.055 g based on the seismic site class calculations .
3. A 250 psf traffic surcharge will be modeled.

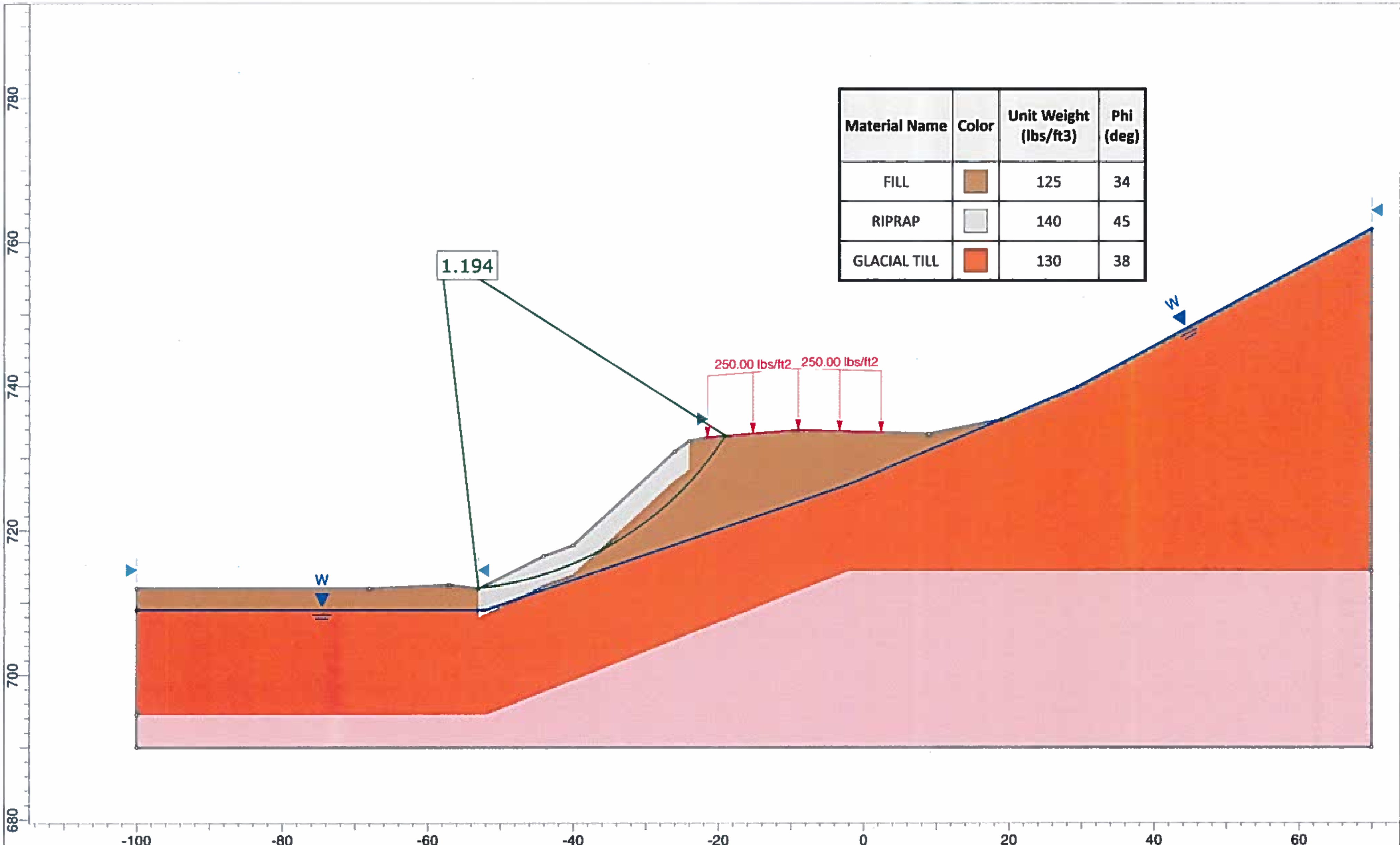
SOIL PROPERTIES

Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
RipRap	140	45	0
Existing Fill	125	34	0
Glaciofluvial Deposits	120	32	0
Glacial Lacustrine Deposits	115	30	0
Glacial Till	130	38	0

RESULTS AND CONCLUSIONS

Location	Factor of Safety		
	Static	Seismic	
Sta. 57+00 Existing Downslope	1.2	1.1	
Sta. 57+00 Proposed Downslope	1.2	1.1	
Sta. 60+00 Existing Downslope	1.4	1.3	
Sta. 60+00 Proposed Downslope	1.5	1.3	
Station 60+00 Existing Upslope	1.4	1.3	* Soil nails or other reinforcement not included.
Station 60+00 Proposed Upslope	1.1	1.0	* Soil nails or other reinforcement not included.

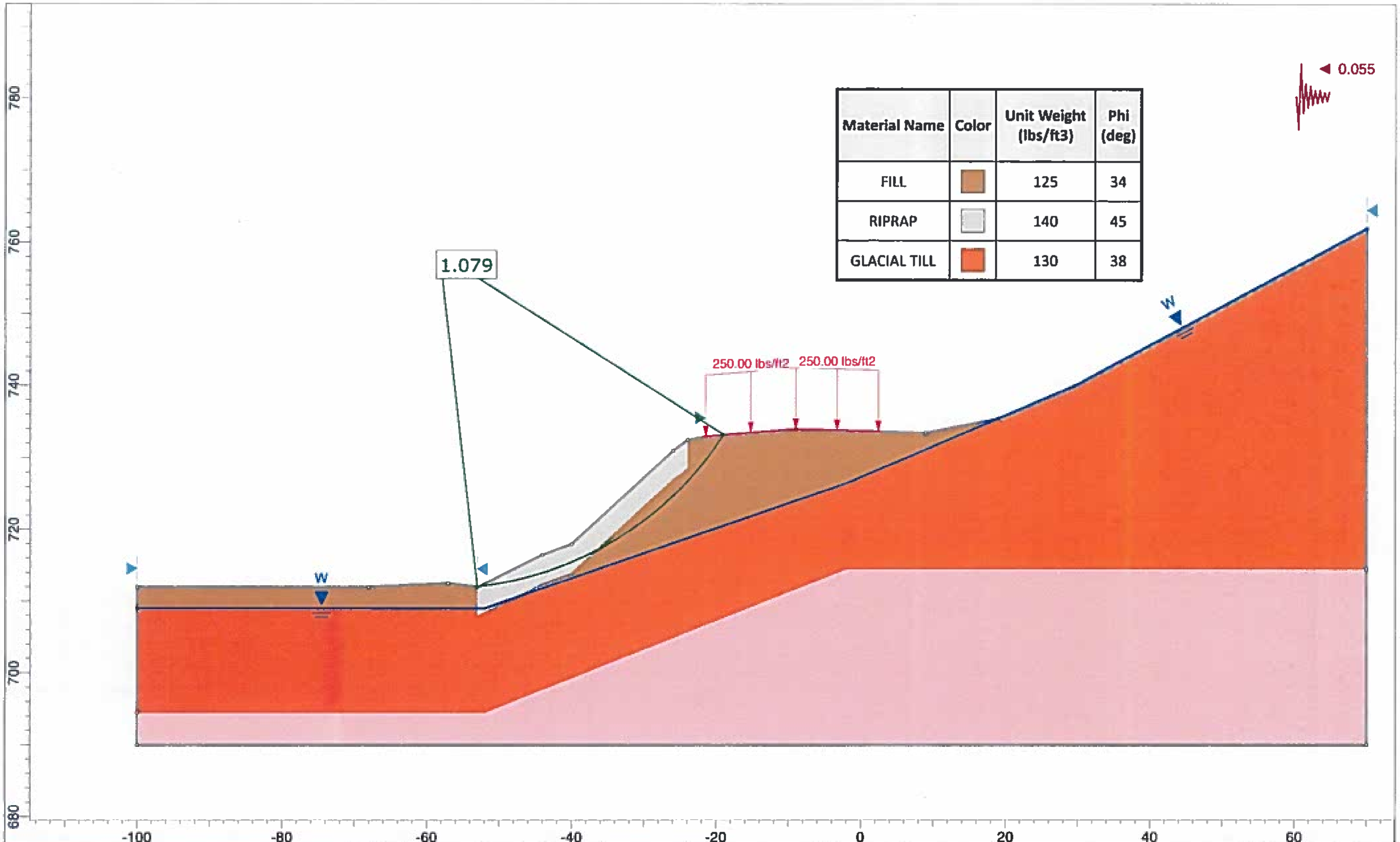
1. Both the simplified Bishop and Spencer methods were used. The lower factor of safety from the two methods is reported above and shown in the following Slide output.
2. Based on AASHTO LRFD Section 11.6.2.3, an acceptable resistance factor for where the slope does not contain or support a structure is 0.75 (F.S. = 1/0.75 = 1.3).
3. Based on Maine DOT Bridge Design Guide Section 5.9.4, a minimum seismic factor of safety of 1.0 is acceptable for slope stability.



rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine		
Analysis Description	STA. 57+00 Original Ground Surface		
Drawn By Kevin A. Russ, P.E.	*Scale* 1:220	*Company*	Haley & Aldrich, Inc.
Date 5/7/19		*File Name*	2019-0508-Slide Sta 57+00 Existing-F.slim

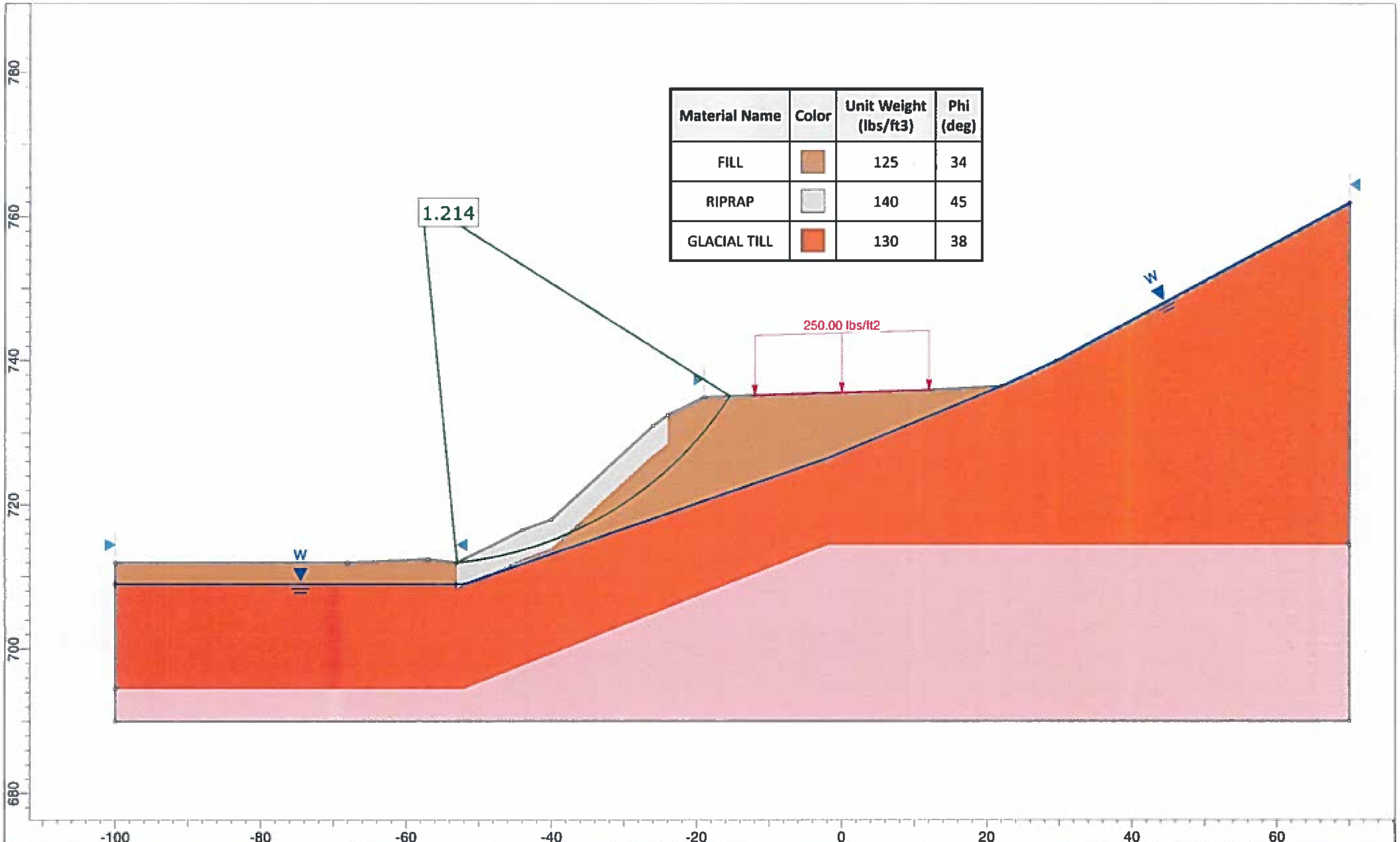


Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine
Analysis Description	STA. 57+00 Original Ground Surface
Drawn By	Kevin A. Russ, P.E.
Scale	1:220
Company	Haley & Aldrich, Inc.
Date	5/7/19
File Name	2019-0508-Slide Sta 57+00 Existing-F.slim

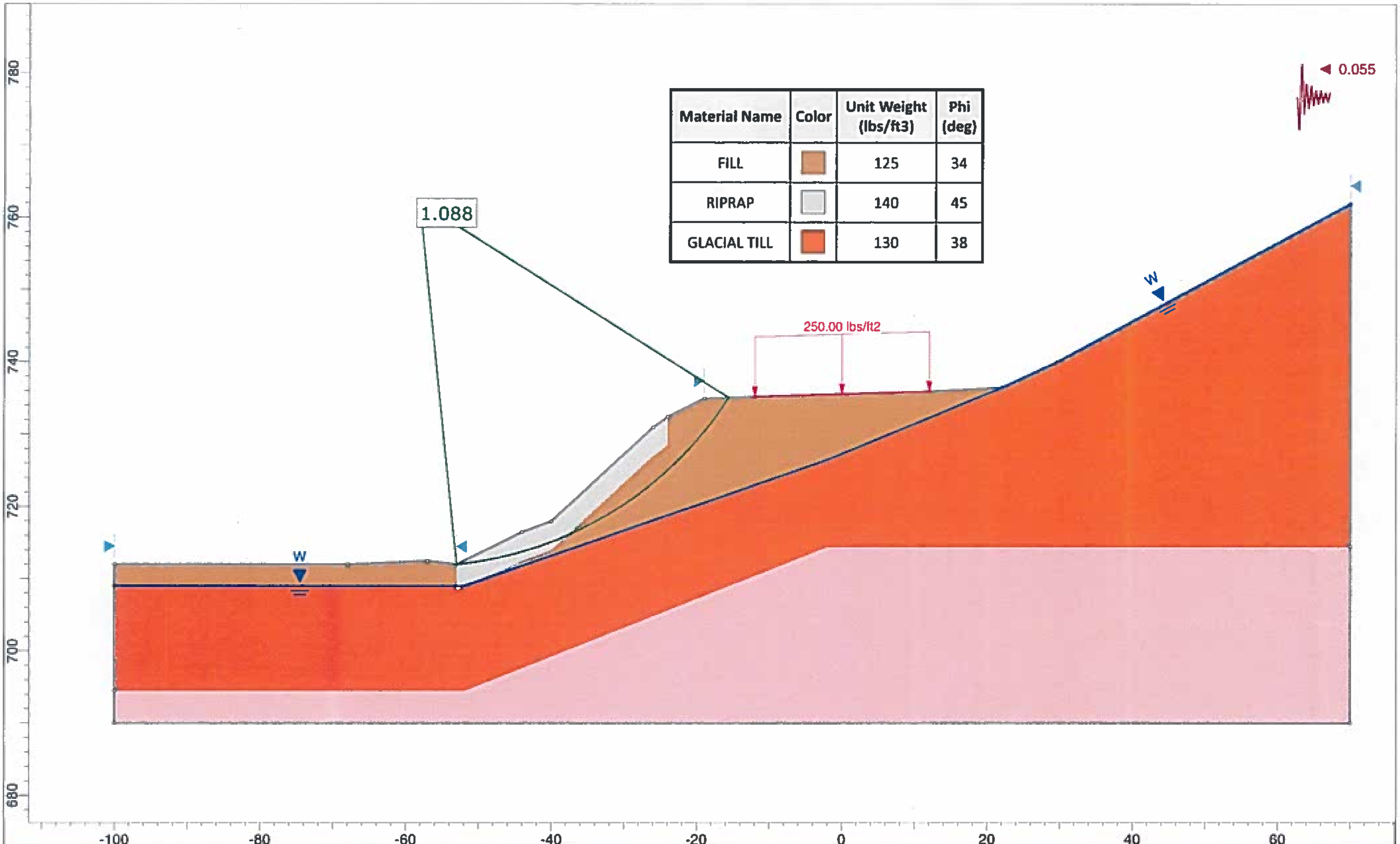


Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine		
Analysis Description	STA. 57+00 Proposed Ground Surface		
Drawn By	Kevin A. Russ, P.E.	*Scale* 1:219	*Company* Haley & Aldrich, Inc.
Date			*File Name* 2019-0508-Slide Sta 57+00 Proposed-F.slmd



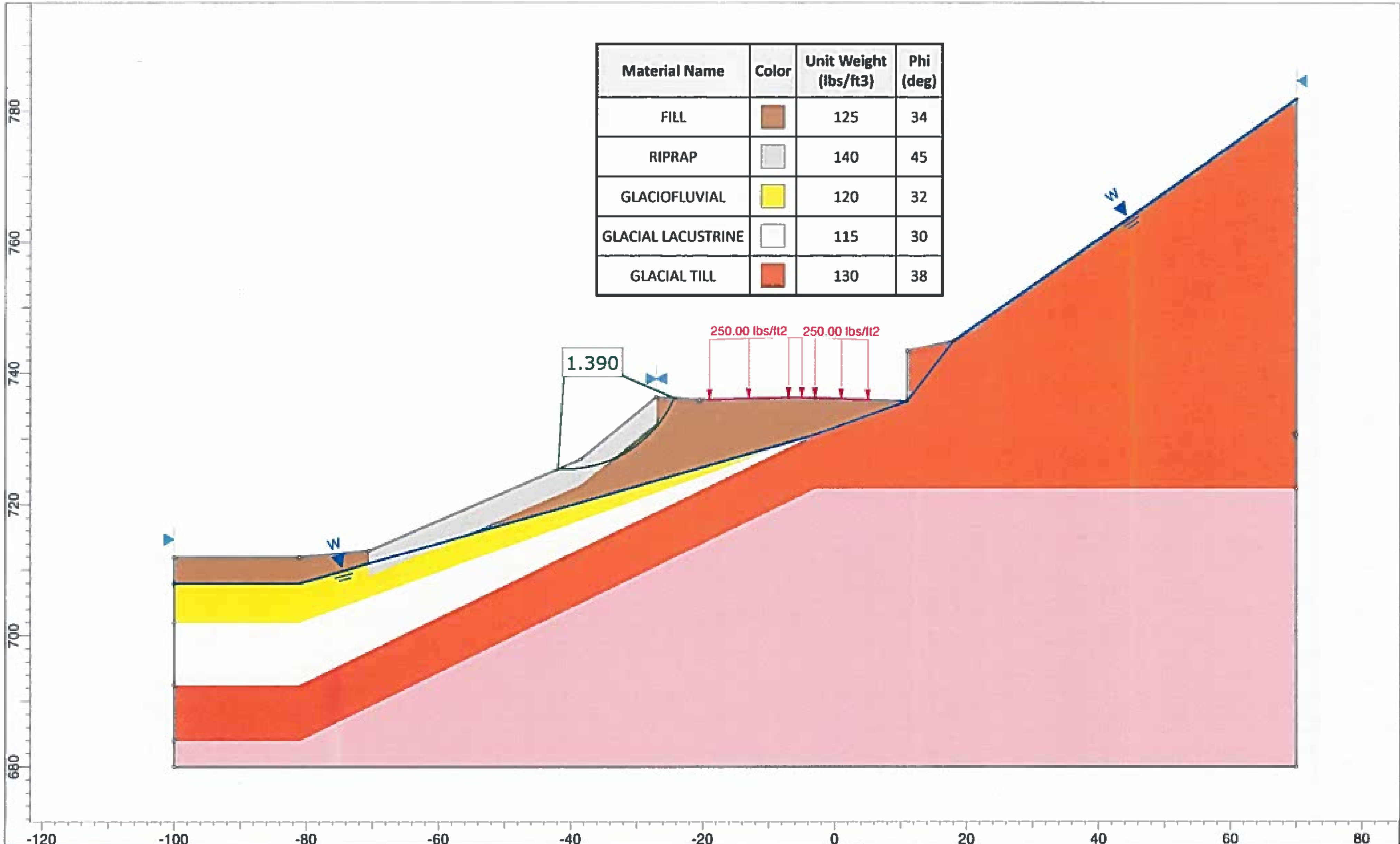
Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine
Analysis Description	STA. 57+00 Proposed Ground Surface
Drawn By	Kevin A. Russ, P.E.
Scale	1:219
Company	Haley & Aldrich, Inc.
Date	
File Name	2019-0508-Slide Sta 57+00 Proposed-F.slmd

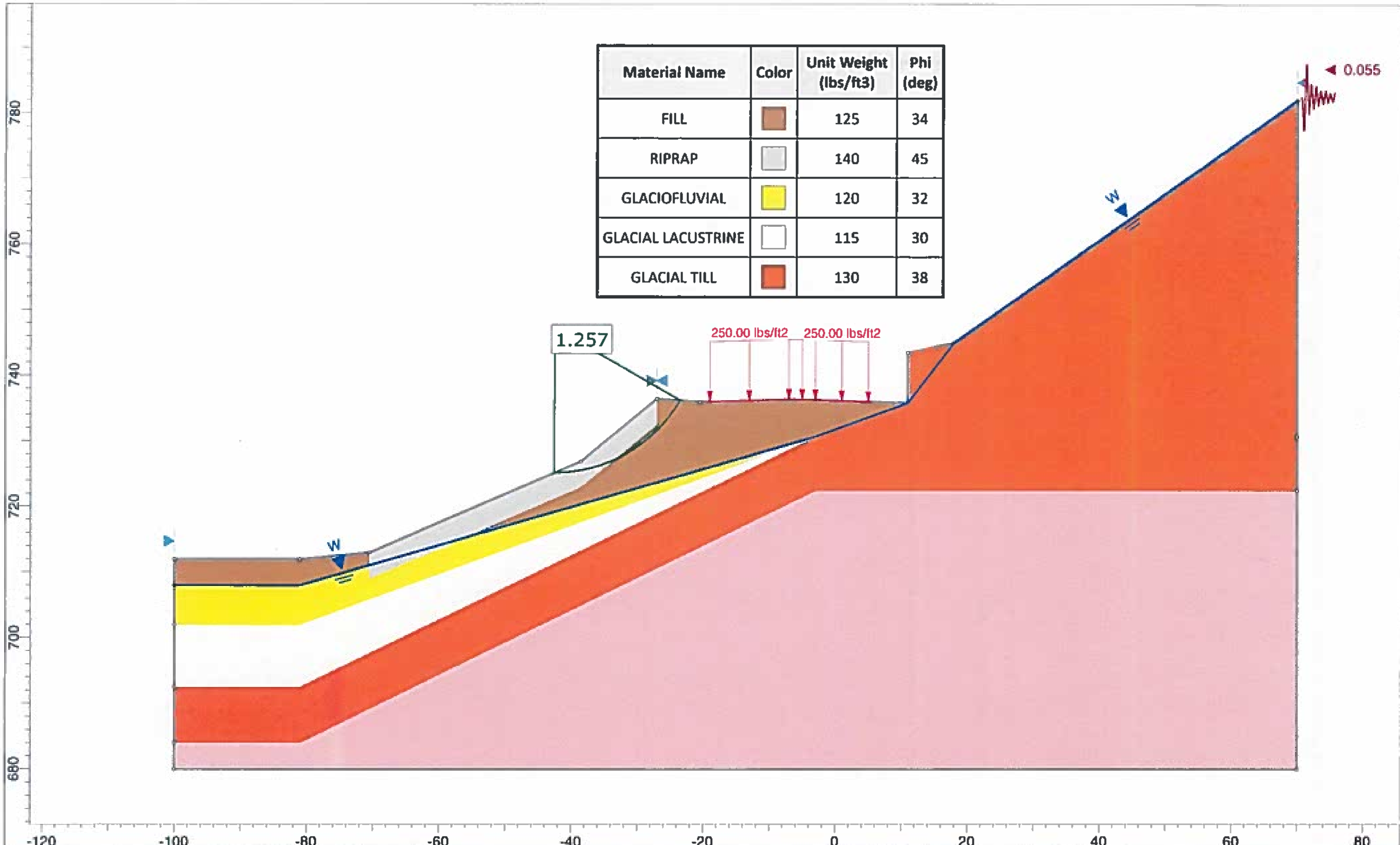
Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38



rocscience

SLIDEINTERPRET 6.018

Project	Route 26 Improvements, Woodstock, Maine
Analysis Description	STA. 60+00 Existing Ground Surface
Drawn By	Kevin A. Russ, P.E.
Scale	1:242
Company	Haley & Aldrich, Inc.
Date	5-7-2019
File Name	2019-0508-HAI-Sta 60+00 Existing-F.slmd

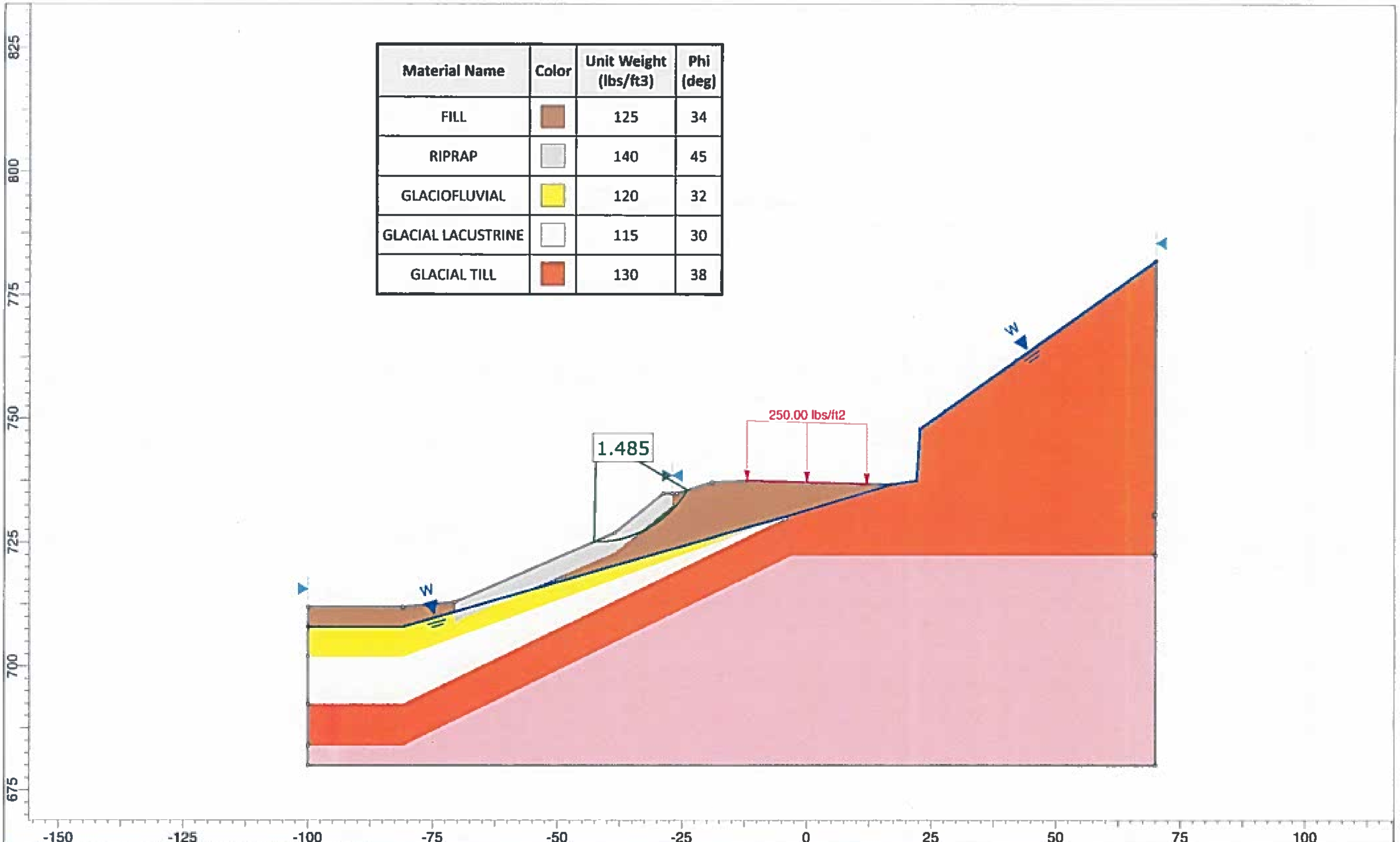


Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine
Analysis Description	STA. 60+00 Existing Ground Surface
Drawn By	Kevin A. Russ, P.E.
Scale	1:242
Company	Haley & Aldrich, Inc.
Date	5-7-2019
File Name	2019-0508-HAI-Sta 60+00 Existing-F.slmd

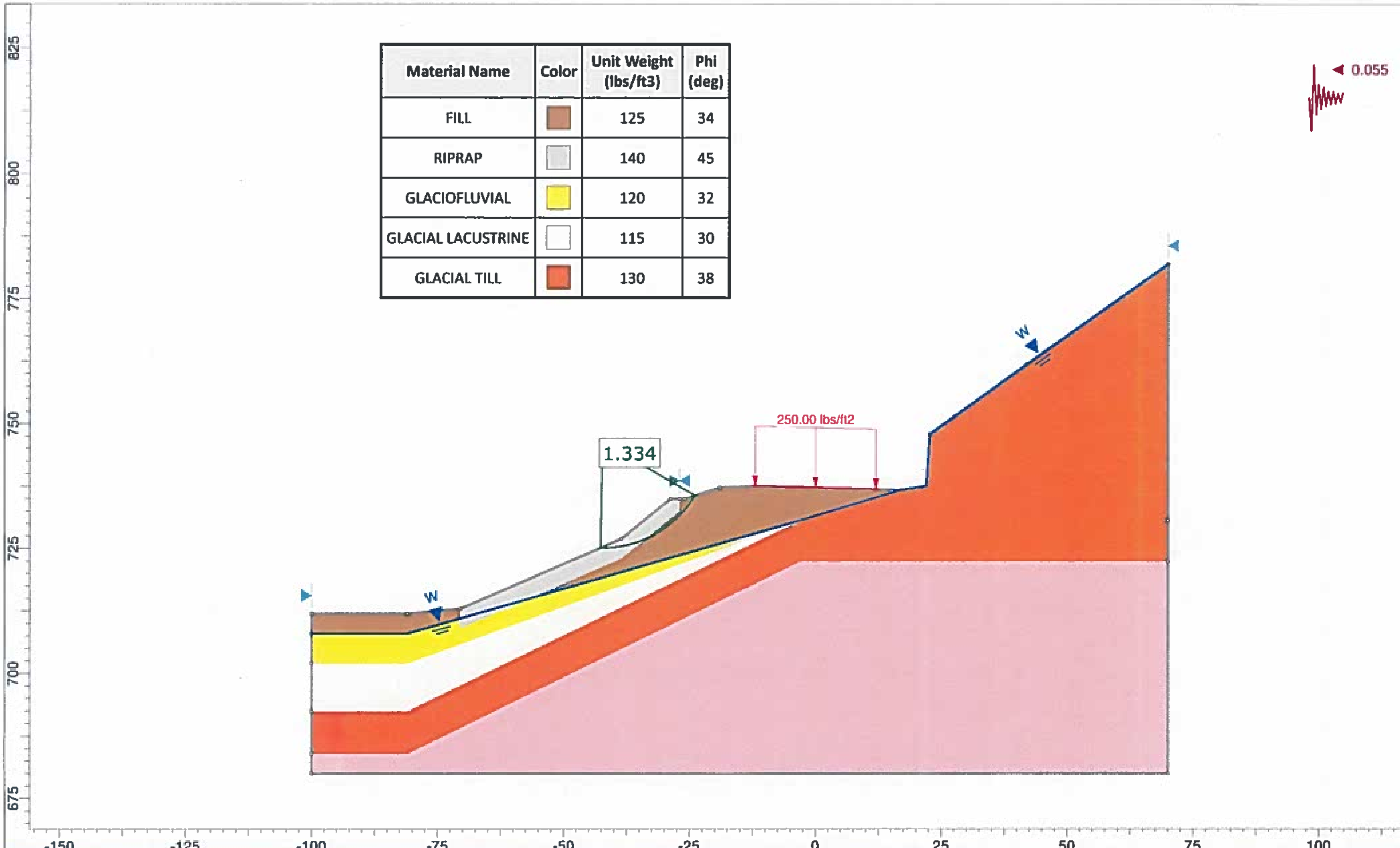


Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine
Analysis Description	STA. 60+00 Proposed Ground Surface
Drawn By	Kevin A. Russ, P.E.
Scale	1:319
Company	Haley & Aldrich, Inc.
Date	5/7/19
File Name	2019-0507-HAI-Sta 60+00 Proposed-F.slmd

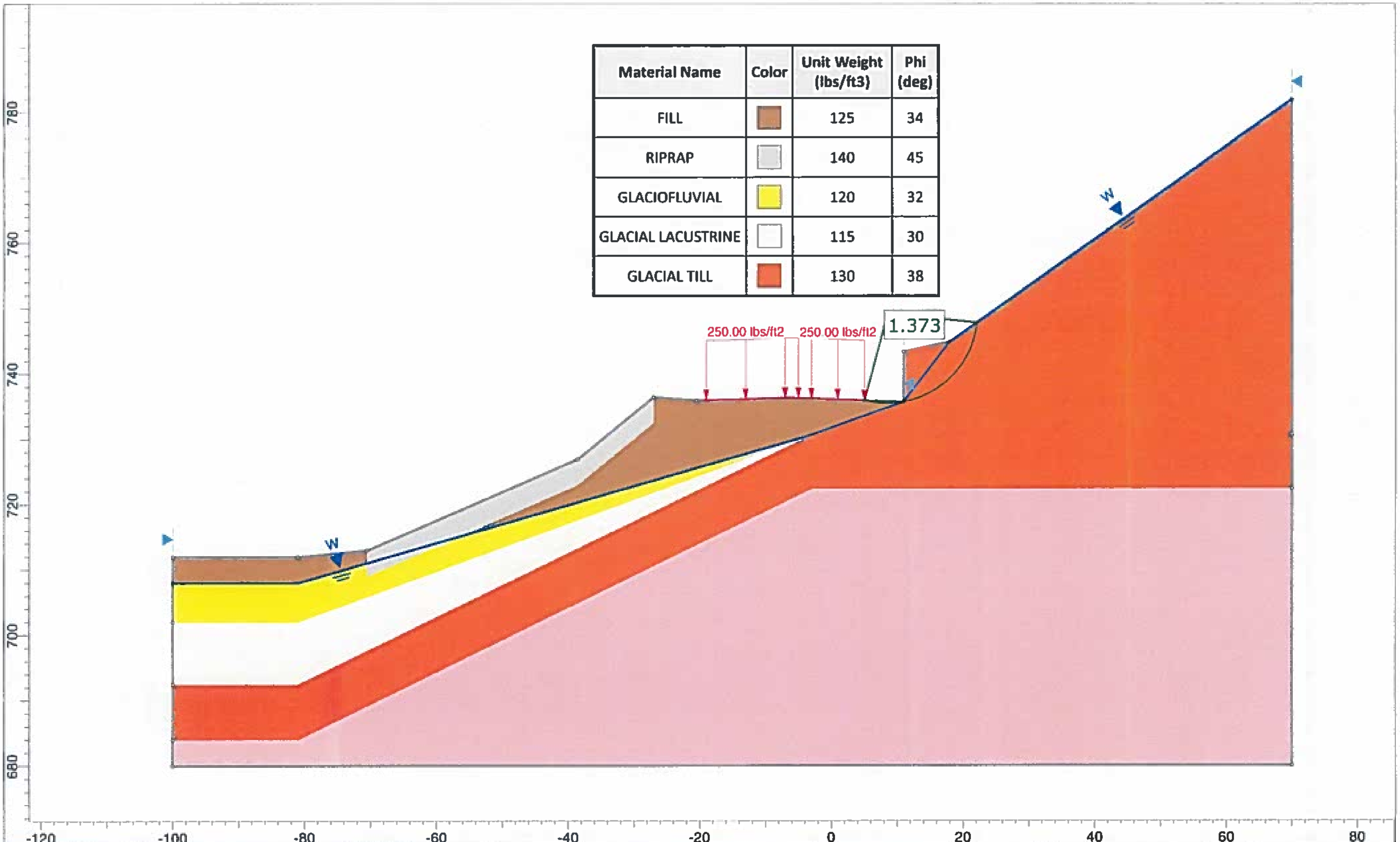


Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine		
Analysis Description	STA. 60+00 Proposed Ground Surface		
Drawn By	Kevin A. Russ, P.E.	*Scale* 1:319	*Company* Haley & Aldrich, Inc.
Date	5/7/19	*File Name*	2019-0507-HAI-Sta 60+00 Proposed-F.slmd

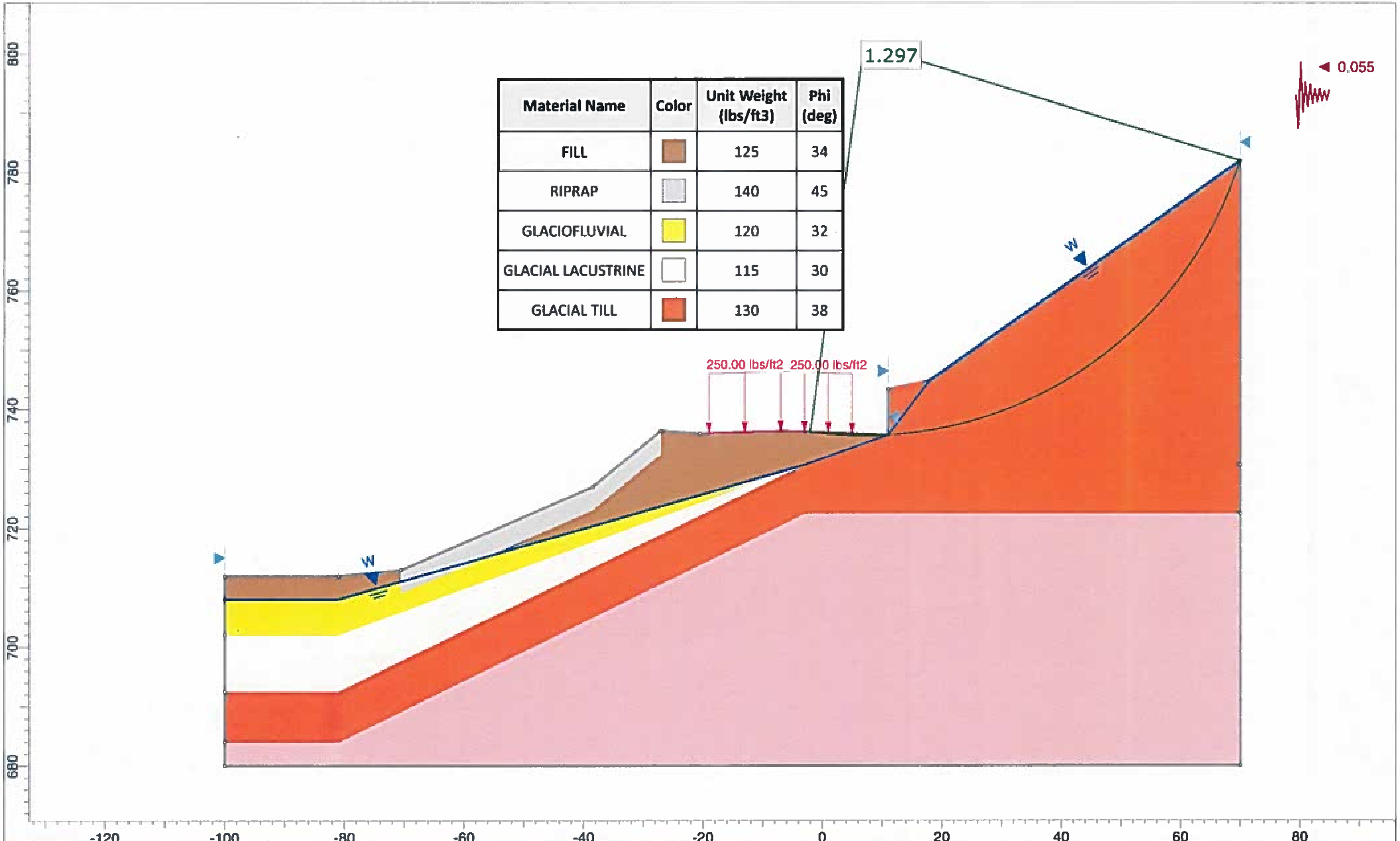


Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine		
Analysis Description	STA. 60+00 Existing Ground Surface		
Drawn By Kevin A. Russ, P.E.	*Scale* 1:242	*Company*	Haley & Aldrich, Inc.
Date 5/7/19		*File Name*	2019-0508-HAI-Sta 60+00 Existing-F.slmd



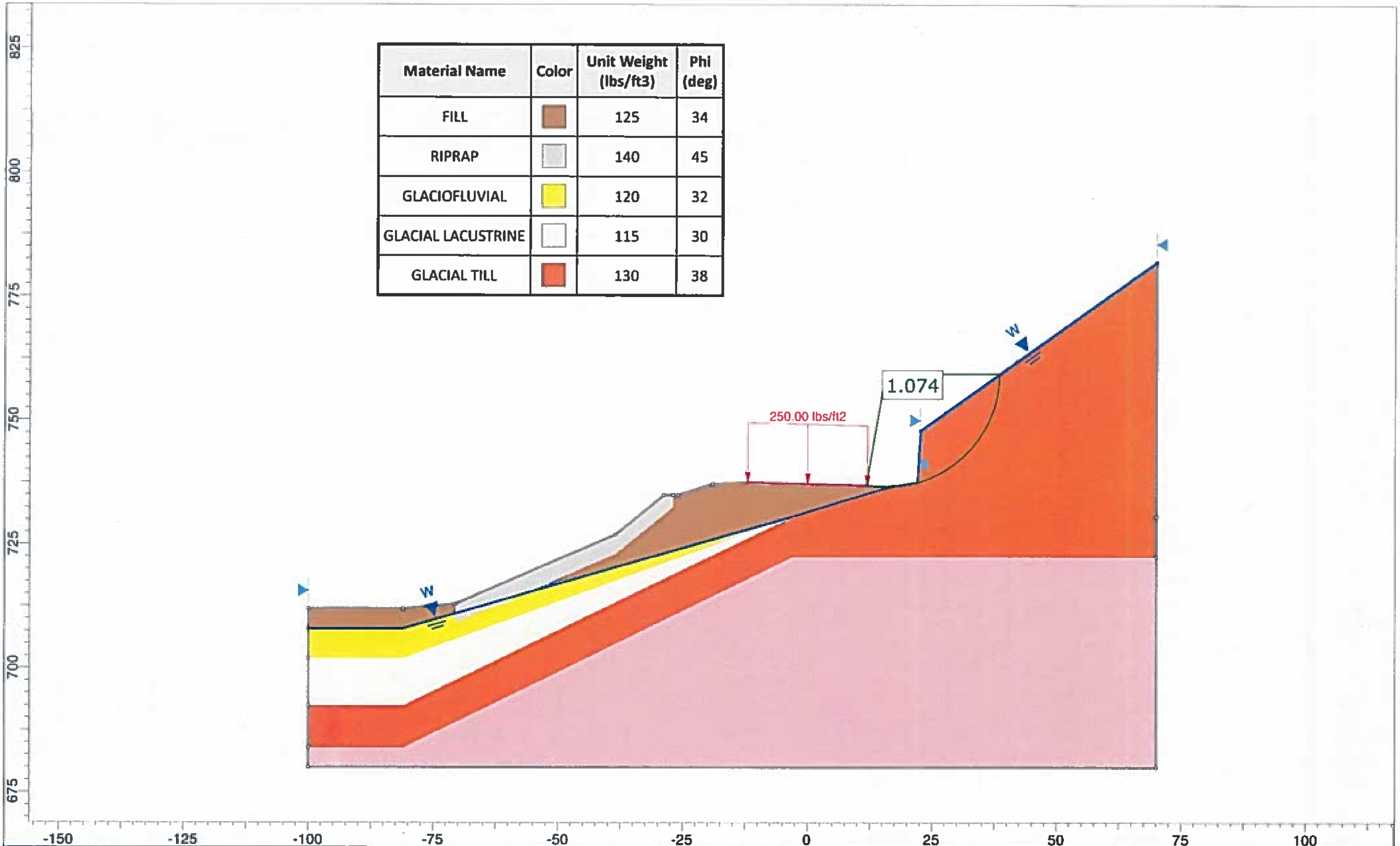
Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine		
Analysis Description	STA. 60+00 Existing Ground Surface		
Drawn By Kevin A. Russ, P.E.	*Scale* 1:266	*Company*	Haley & Aldrich, Inc.
Date	5-7-2019	*File Name*	2019-0508-HAI-Sta 60+00 Existing-F.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38



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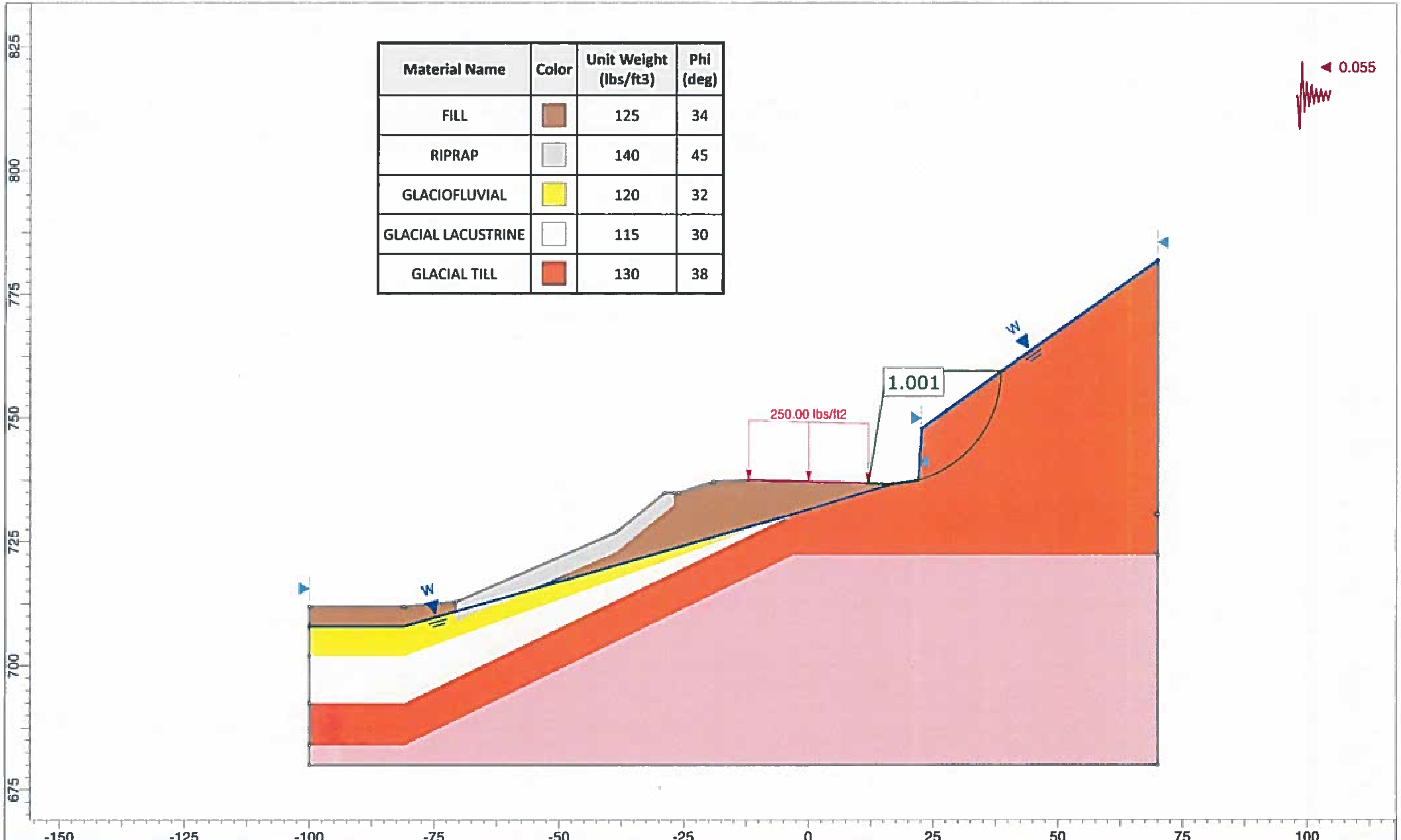
SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine
Analysis Description	STA. 60+00 Proposed Ground Surface
Drawn By	Kevin A. Russ, P.E.
Scale	1:319
Company	Haley & Aldrich, Inc.
Date	5/7/19
File Name	2019-0507-HAI-Sta 60+00 Proposed-F.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Phi (deg)
FILL		125	34
RIPRAP		140	45
GLACIOFLUVIAL		120	32
GLACIAL LACUSTRINE		115	30
GLACIAL TILL		130	38

rocscience

SLIDEINTERPRET 8.018

Project	Route 26 Improvements, Woodstock, Maine		
Analysis Description	STA. 60+00 Proposed Ground Surface		
Drawn By	Kevin A. Russ, P.E.	*Scale* 1:319	*Company* Haley & Aldrich, Inc.
Date	5/7/19	*File Name*	2019-0507-HAI-Sta 60+00 Proposed-F.slmd

APPENDIX E

Existing Soil Nail Wall Design Submittal and Drawings

Design Calculations for Permanent Soil Nails

Retaining Wall Repairs
Maine State Route 26
Woodstock, Maine
Maine DOT Project No. 19168

Prepared For:
Thomas Drilling and Blasting Corp.
P.O. Box 200
Spofford, NH
(603) 363-4706

Prepared By:
Earthwork Engineering, Inc.
175 Ridge Road
Hollis, NH 03049
(603) 465-9500

STATE OF MAINE
PAUL A.
DETERLING
No. 5508
LICENSED
PROFESSIONAL ENGINEER

Paul A Deterling

Paul A. Deterling, P.E.
Maine P.E. No.5508
January 2, 2013

Permanent Soil Nail Design

Retaining Wall Repair
Maine State Route 26
Woodstock, Maine

1.0 Design Procedure and Assumptions

The retaining wall repair will consist of permanent soil nails installed through the existing retaining wall and a concrete facing cast against the wall. Design of the concrete facing is to be done by others. The height of support for the soil nails varies from 2 to 8 feet. The walls will be analyzed to resist lateral pressures due to soil loading. The analysis of the soil nail wall will be done using the SNAILZ computer program developed by the California Department of Transportation. The program uses a bi-linear wedge analysis for failure planes exiting at the toe of the wall and tri-linear wedge analysis for failure planes developing below the wall. The program uses force equilibrium based on mobilized ϕ and c. The results of this computer analysis will provide the global factor of safety.

The analysis will be done using ultimate stress design so that a factor of safety of greater than 1.30 is provided for the final condition. In addition, the wall will be analyzed with seismic loading to confirm a minimum factor of safety of 1.10 is maintained. The design peak ground acceleration for seismic loading will be 0.1g. The overall design of the wall will be done in accordance with methods presented in FHWA Geotechnical Design Circular No. 7 *"Soil Nail Walls"*.

2.0 Design Parameters and Variables

The soil conditions at the retaining wall location are taken from borings HB-Wood-101 and -102. The borings were advanced 3.8 and 4.9 feet to refusal, respectively. The soils above refusal consisted of a dense to very dense fine to coarse sand with some gravel and trace silt (glacial till). No rock cores were taken of the bedrock. Based on the "Bedrock Geologic Map of Maine" (1985) the bedrock in the area of Bryant Pond is a tonalite, which is a general description for quartz diorite. The soil slope behind the existing retaining wall is described as a till drumlin in the geotechnical report. The slope has exposed rock in many locations which could be boulders or bedrock.

For design it will be assumed that the soil slope behind the retaining wall is glacial till with bedrock at a depth of 12 feet below grade. This profile will be extended up the slope. It is noted that based on the soil conditions assumed for design, the stability of the existing 35 degree slope would be marginal. Therefore it is expected that the bedrock actually daylights up the slope rather than paralleling the slope angle. The unit weight and friction angle for the dense glacial till are estimated based on Table 3-3 from *"Foundation Analysis and Design"* by Bowles (copy on page A1).

o Glacial Till — Friction Angle = $\phi_{till} := 39 \cdot deg$

Unit Weight = $\gamma_{till} := 130 \cdot pcf$

3.0 Design of Soil Nails

The height of support will vary with a maximum height of 8 feet. The area at the top of the wall slopes up at approximately a 35 degree angle (1.4H:1V). No surcharge will be applied to this sloped area. The design of the soil nail wall will be done using the SNAILZ computer program developed by the California Department of Transportation.

Ultimate values for soil bond stress will be determined for the glacial till. In addition, ultimate values for the nail punching and tendon yield stress will be calculated for input. Finally the wall height and nail diameter, length and spacing will be input and the program will then determine the factor of safety at equilibrium. The overall design will be done in accordance with recommendations given in FHWA Geotechnical Engineering Circular No. 7 *"Soil Nail Walls"*.

a.) Input Parameters

The ultimate soil bond stress, punching load and bar yield strength will be determined.

- **Ultimate Bond Stress for Glacial Till and Bedrock**

  The soil conditions at the site consist of a dense to very dense glacial till deposit over bedrock. Based on recommended values given in Table 5.2 of FHWA Geotechnical Design Circular No. 7 *"Soil Nail Walls"* (page A2 of calculations) an ultimate bond stress for the glacial till will be determined, assuming Type A grouting. The assumed design bond stresses will be verified from field testing on some of the soil nails.

  For a medium to very dense glacial till the recommended range of ultimate bond stress is 14 to 28 psi. For design a bond stress of 25 psi will be used for the dense to very dense till layer. This value will be verified in the field with verification and proof testing

  Ultimate Bond Stress for Till Layer = $\sigma_{till} := 25 \cdot psi$

  Below the glacial till layer is bedrock, assumed to be a quartz diorite. This is generally a hard intrusive igneous rock. The ultimate bond stress for granite and basalt, both igneous rocks, will be used for design. The range of ultimate bond stress for granite and basalt ranges from 200 to 609 psi. For design a value of 150 psi will be used.

  Ultimate Bond Stress for Bedrock = $\sigma_{rock} := 150 \cdot psi$

- **Ultimate Punching Load**

  The anchor plates for the soil nails will be 12"x12" and will bear against the existing concrete retaining wall which is at least 12 inches thick. The ultimate punching resistance of the plate bearing against the concrete will be determined for input into the computer program. The ultimate punching resistance will be determined in accordance with guidelines given in Section 5.6.4 of FHWA Geotechnical Engineering Circular No. 7 *"Soil Nail Walls"*.

Dimension of Bearing Plate = $L_{BP} := 12 \cdot in$

Thickness of Concrete Wall = $h := 12 \cdot in$

Strength of Concrete = $f_c := 3000 \cdot psi$

Diameter of Conical Failure Surface = $D_c := L_{BP} + h$

$$D_c = 24 \cdot in$$

Ultimate Punching Load = $V_F := 0.58 \cdot \sqrt{f_c \cdot 1000 \cdot ksi} \cdot \pi \cdot \left(\frac{D_c \cdot h}{144}\right)$ (Eq. 5.56)

$$V_F = 200 \cdot kips$$

Ultimate Punching Shear Capacity = $R_{FP} := V_F$

$$R_{FP} = 200 \cdot kips$$

- **<u>Ultimate Yield Stress of Tendon</u>**

The soil nail tendons will be 40/20 hollow bars with properties given on page A3. The area of the tendon will be reduced by 11.2% to account for corrosion based on mildly corrosive conditions for a 60 year design life (see page A3a).

Tendon Diameter = $d_t := 1.57 \cdot in$ (40/20 Hollow Bar)

Area of Tendon = $A_t := 1.13 \cdot in^2$

Reduced Area of Tendon for Corrosion = $A_{tr} := 88.8\% \cdot (A_t)$

$$A_{tr} = 1.00 \cdot in^2$$

Equivalent Solid Diameter = $d_e := \sqrt{\frac{4A_{tr}}{\pi}}$

$$d_e = 1.13 \cdot in$$

Yield Strength = $P_y := 95.6 \cdot kips$

Yield Stress of Tendon = $f_y := \frac{P_y}{A_t}$

$$f_y = 85 \cdot ksi$$

Bar Tensile Capacity = $R_T := A_{tr} \cdot f_y$ (Eq. 5.38)

$R_T = 85 \cdot kip$

b.) Design of Soil Nail Walls

The height of support for the soil nail wall varies from 2 to 8 feet. Two design cases will be analyzed for final construction wall heights of 5 with one row of nails and 8 feet with two rows of nails. The design cases will be analyzed for the permanent condition and the permanent condition with earth quake loading. The required minimum factor of safety for the permanent condition will be 1.30 and for the seismic condition will be 1.10.

The top nail will be located 2 feet from the top of the existing retaining wall and the nails will be installed at a maximum spacing of 5 feet horizontally. The soil nails will be installed at a 15 degree angle. The nail tendons will be 40/20 Hollow Bars with a 3-inch diameter cross cut drill bit. During drilling this bit will cut a larger hole than the bit size and in addition the grout will permeate beyond the limits of the drilled hole. This is discussed in the FHWA paper "Hollow-Core Soil Nails, State of the Practice" (April 2006) and is shown in figure 2 (see page A4). For the given soil conditions a grout body diameter of 4 inches would be used for design however since it is anticipated that the nails will be drilled into bedrock a nail diameter of 3 inches will be used.

General Design Parameters

Nail Shaft Diameter = $d_s := 3 \cdot in$

Vert. Distance to Top Level = $L_1 := 2 \cdot ft$

Vert. Distance Between Levels = $L_v := 3 \cdot ft$

Horiz. Distance Between Nails = $L_h := 5 \cdot ft$

Design Case I - Wall Height Up To 5 Feet with One Level of Nails

The design case will be analyzed for the section of wall with heights of 2 to 5 feet. The analysis will be done for the permanent wall configuration and the permanent wall with seismic loading.

Analysis A - H = 5' (Permanent Wall Configuration)

The design height of support is 5 feet with one level of nails installed. The minimum required factor of safety for this long term condition is 1.30.

Levels Of Nails = $N := 1$

Length of Nails = $L_{nail} := 15 \cdot ft$

Computer Results

Factor of Safety = 1.32 $\geq$ 1.30 (Output on Page B1 to B5)

Nail Loads:

Stress in Top Nail = $f_t := 64.569 \cdot ksi$ (Page B4)

Top Nail Load = $T_1 := A_{tr} \cdot f_t$

$$T_1 = 64.8 \cdot kip$$

Maximum Average Nail Force = $T_{avgs} := 0.000 \cdot kip$ (Page B5)

Average Nail Load = $T_{avg} := \text{mean}(T_1)$

$$T_{avg} = 64.8 \cdot kip$$

Maximum Nail Load = $T_{max} := \max(T_1)$

$$T_{max} = 64.8 \cdot kip$$

Maximum Nail Tensile Force = $T_{maxs} := \left(\frac{T_{avgs}}{T_{avg}}\right) \cdot T_{max}$ (Eq. 6.13)

$$T_{maxs} = 0 \cdot kip$$

Design Nail Head Tensile Force = $T_A := T_{maxs} \cdot \left[0.6 + 0.057 \cdot \left(\frac{L_v}{ft} - 3\right)\right]$ (Eq. 6.15)

$$T_A = 0.0 \cdot kip$$

Analysis B - H = 5' (Permanent Wall with Seismic Loading)

The design height of support is 5 feet with one level of nails installed. This is the permanent condition with seismic loading. The peak ground acceleration will be 0.1g for this analysis. The minimum required factor of safety is 1.10 with seismic loading.

Levels Of Nails = $N := 1$

Length of Nails = $L_{nail} := 15 \cdot ft$

Computer Results

Factor of Safety = 1.12 $\geq$ 1.10 (Output on Page C1 to C5)

Nail Loads:

Stress in Top Nail = $f_t := 76.113 \cdot ksi$ (Page C4)

Top Nail Load = $T_1 := A_{tr} \cdot f_t$

$$T_1 = 76.4 \cdot kip$$

Maximum Average Nail Force = $T_{avgs} := 19.113 \cdot kip$ (Page C5)

Average Nail Load = $T_{avg} := \text{mean}(T_1)$

$$T_{avg} = 76.4 \cdot kip$$

Maximum Nail Load = $T_{max} := \max(T_1)$

$$T_{max} = 76.4 \cdot kip$$

Maximum Nail Tensile Force = $T_{maxs} := \left(\frac{T_{avgs}}{T_{avg}}\right) \cdot T_{max}$ (Eq. 6.13)

$$T_{maxs} = 19.1 \cdot kip$$

Design Nail Head Tensile Force = $T_B := T_{maxs} \cdot \left[0.6 + 0.057 \cdot \left(\frac{L_v}{ft} - 3\right)\right]$ (Eq. 6.15)

$$T_B = 11.5 \cdot kip$$

Design Case 2 - Wall Height Up To 8 Feet with Two Levels of Nails

The design case will be analyzed for the section of wall with heights of 6 to 8 feet. The analysis will be done for the permanent wall configuration and the permanent wall with seismic loading.

Analysis C - H = 8' (Permanent Wall Configuration)

The design height of support is 8 feet with one level of nails installed. The minimum required factor of safety for this long term condition is 1.30.

Levels Of Nails = $N := 2$

Length of Nails = $L_{top} := 20 \cdot ft$

$$L_{bottom} := 15 \cdot ft$$

Computer Results

Factor of Safety = 1.41 $\geq$ 1.30 (Output on Page D1 to D5)

Nail Loads:

Stress in Top Nail = $f_t := 60.219 \cdot ksi$ (Page D4)

Top Nail Load = $T_1 := A_{tr} \cdot f_t$

$T_1 = 60.4 \cdot kip$

Stress in Bottom Nail = $f_t := 60.219 \cdot ksi$ (Page D4)

Bottom Nail Load = $T_2 := A_{tr} \cdot f_t$

$T_2 = 60.4 \cdot kip$

Maximum Average Nail Force = $T_{avgs} := 0.000 \cdot kip$ (Page D5)

Average Nail Load = $T_{avg} := \text{mean}(T_1, T_2)$

$T_{avg} = 60.4 \cdot kip$

Maximum Nail Load = $T_{max} := \max(T_1, T_2)$

$T_{max} = 60.4 \cdot kip$

Maximum Nail Tensile Force = $T_{maxs} := \left(\frac{T_{avgs}}{T_{avg}}\right) \cdot T_{max}$ (Eq. 6.13)

$T_{maxs} = 0 \cdot kip$

Design Nail Head Tensile Force = $T_C := T_{maxs} \cdot \left[0.6 + 0.057 \cdot \left(\frac{L_v}{ft} - 3\right)\right]$ (Eq. 6.15)

$T_C = 0.0 \cdot kip$

Analysis D - H = 8' (Permanent Wall with Seismic Loading)

The design height of support is 8 feet with one level of nails installed. This is the permanent condition with seismic loading. The peak ground acceleration will be 0.1g for this analysis. The minimum required factor of safety is 1.10 with seismic loading.

Levels Of Nails = $N := 2$

Length of Nails = $L_{top} := 20 \cdot ft$

$L_{bottom} := 15 \cdot ft$

Computer Results

Factor of Safety = 1.18 $\geq$ 1.10 (Output on Page E1 to E5)

Nail Loads:

Stress in Top Nail = $f_t := 72.053 \cdot ksi$ (Page E4)

Top Nail Load = $T_1 := A_{tr} \cdot f_t$

$T_1 = 72.3 \cdot kip$

Stress in Bottom Nail = $f_t := 72.053 \cdot ksi$ (Page E4)

Bottom Nail Load = $T_2 := A_{tr} \cdot f_t$

$T_2 = 72.3 \cdot kip$

Maximum Average Nail Force = $T_{avgs} := 21.777 \cdot kip$ (Page E5)

Average Nail Load = $T_{avg} := \text{mean}(T_1, T_2)$

$T_{avg} = 72.3 \cdot kip$

Maximum Nail Load = $T_{max} := \max(T_1, T_2)$

$T_{max} = 72.3 \cdot kip$

Maximum Nail Tensile Force = $T_{maxs} := \left(\frac{T_{avgs}}{T_{avg}}\right) \cdot T_{max}$ (Eq. 6.13)

$T_{maxs} = 21.8 \cdot kip$

Design Nail Head Tensile Force = $T_D := T_{maxs} \cdot \left[0.6 + 0.057 \cdot \left(\frac{L_v}{ft} - 3\right)\right]$ (Eq. 6.15)

$T_D = 13.1 \cdot kip$

Design Nail Head Tensile Force for Wall

Design Nail Head Tensile Force = $T_o := \max(T_A, T_B, T_C, T_D)$

$T_o = 13.1 \cdot kips$

4.0 Design of Soil Nail Connection at Existing Wall

The soil nails will be drilled through the existing concrete retaining wall. A steel plate will be used to connect the soil nails to the existing wall. The plate will have headed studs attached to support the concrete facing to be cast against the wall. The concrete facing is to be designed by others.

4.a) Design of Bearing Plate

The bearing plate thickness will be determined based on the maximum nail load determined above. The bearing plate will be designed as a cantilever beam with the load decreasing linearly to the free edge.

Yield Strength of Plate = $F_y := 36 \cdot \text{ksi}$

Side Dimension of Plate = $L_{BP} = 12 \cdot \text{in}$

Design Nail Head Tensile Force = $T_o = 13.1 \cdot \text{kip}$

Average Bearing Pressure = $P_b := \dfrac{T_o}{L_{BP}^{\,2}}$

$P_b = 91 \cdot \text{psi}$

Allowable Bearing Stress = $F_b := 0.70 \cdot (0.85 \cdot f_c)$ (ACI 318, Sect. 10.15)

$F_b = 1785 \cdot \text{psi}$ > $P_b = 91 \cdot \text{psi}$ *OK*

Bending in Plate = $M_b := \left[\dfrac{\dfrac{T_o}{2} \cdot \left(\dfrac{L_{BP}}{2} \right)}{3} \right]$

$M_b = 1.1 \cdot \text{kft}$

Required Section Modulus = $S_x := \dfrac{M_b}{75\% \cdot (F_y)}$

$S_x = 0.48 \cdot \text{in}^3$

Required Plate Thickness = $t_{min} := \sqrt{\dfrac{6 \cdot S_x}{L_{BP}}}$

$t_{min} = 0.49 \cdot \text{in}$

Use 12"x12"x1/2" Bearing Plates for Soil Nails

4.b) Bearing Plate Punching Shear Capacity

The anchor plates for the soil nails will be 12"x12". The ultimate punching resistance of the plate bearing against the concrete wall will be determined based on procedures given in Section 5.6.4.2 of the FHWA Manual. The punching load distribution will be outward from the edge of the anchor plate at 45 degrees.

Dimension of Bearing Plate = $L_{BP} = 12 \cdot in$

Effective Diameter of Conical Failure Surface = $D_c := L_{BP} + h$

$D_c = 24 \cdot in$

Effective Depth of Failure Surface = $h_c := h$

Ultimate Punching Shear Force = $V_F := 0.58 \cdot \sqrt{f_c \cdot psi \cdot 48} \cdot \pi \cdot D_c \cdot h_c$

$V_F = 199.1 \cdot kip$

Factor of Safety Against Punching Shear Failure = $FS_{FP} := \frac{V_F}{T_o}$

$FS_{FP} = 15.24$ $\geq$ 1.50 *OK*

4.c) Headed-Stud Punching Shear Capacity

The ultimate punching resistance of the headed-studs embedded into the permanent concrete facing will be determined based on procedures given in Section 5.6.4.2 of the FHWA Manual. The punching load distribution will be outward from the edge of the headed studs at 45 degrees. The studs will be 1/2" diameter with a 1" diameter head. The dimensions for the studs are taken from Table A.6 from Appendix A of the FHWA Design Manual (see page A5). Four studs will be attached to each bearing plate.

Distance Between Studs = $S_{HS} := 9 \cdot in$

Length of Studs = $L_s := 4.125 \cdot in - 0.125 \cdot in$ (1/2 x 4 1/8 Headed Stud)

$L_s = 4.0000 \cdot in$

Thickness of Stud Head = $t_H := 0.31 \cdot in$

Thickness of Bearing Plate = $t_P := .625 \cdot in$ (See design below)

Effective Depth of Failure Surface = $h_c := L_s - t_H + t_P$

$h_c = 4.3 \cdot in$

Effective Diameter of Conical Failure Surface = $D_c := S_{HS} + h_c$

$D_c = 13 \cdot in$

Ultimate Punching Shear Force = $V_F := 0.58 \cdot \sqrt{f_c \cdot psi \cdot 48} \cdot \pi \cdot D_c \cdot h_c$

$V_F = 39.7 \cdot kip$

Factor of Safety Against Stud Punching Failure = $FS_{HS} := \dfrac{V_F}{T_o}$

$FS_{HS} = 3.04$ $\geq$ 1.50 *OK*

4.d) Headed-Stud Tensile Capacity

The nail head capacity against tensile failure of the headed-stud connectors to the permanent facing will be analyzed based on procedures given in Section 5.6.5 of the FHWA Manual.

Number of Headed-Studs per Nail = $N_H := 4$

Headed-Stud Shaft Diameter = $D_S := 0.50 \cdot in$

Cross Sectional Area of Headed-Stud Shaft = $A_{SH} := \dfrac{\pi \cdot D_S^2}{4}$

$A_{SH} = 0.2 \cdot in^2$

Headed-Stud Head Diameter = $D_H := 1.0 \cdot in$

Cross Sectional Area of Headed-Stud Head = $A_{HH} := \dfrac{\pi \cdot D_H^2}{4}$

$A_{HH} = 0.79 \cdot in^2$

Yield Strength of Headed-Stud = $f_y := 92 \cdot ksi$ (A325 Stud, AISC Table I-C)

Stud Head to Shaft - Area Ratio = $R_A := \dfrac{A_{HH}}{A_{SH}}$

$R_A = 4.00$ $\geq$ 2.50 *OK* (Eq. 5.62)

Head Thickness to Diameter Ratio = $R_T := \dfrac{t_H}{D_H - D_S}$

$R_T = 0.62$ $\geq$ 0.50 *OK* (Eq. 5.63)

Tensile Capacity of Headed-Studs = $R_{HT} := N_H \cdot A_{SH} \cdot f_y$

$R_{HT} = 72 \cdot kip$

Factor of Safety for Tensile Failure = $FS_{HT} := \dfrac{R_{HT}}{T_o}$

$FS_{HT} = 5.53$ $\geq$ 2.00 *OK*

END OF CALCULATIONS

Table 3-3. Empirical values for ϕ, D_r, and unit weight of granular soils based on the standard penetration number with corrections for depth and for fine saturated sands

Description		Very loose		Loose		Medium		Dense		Very dense	
Relative density D_r*	0		0.15		0.35		0.65		0.85		1.00
Standard penetration no. N			4		10		30		50		
Approx. angle of internal friction ϕ°†	25°–30°		27–32°		30–35°		35–40°		38–43°		
Approx. range of moist unit weight, (γ) pcf (kN/m^3)		70–100‡ (11–16)		90–115 (14–18)		110–130 (17–20)		110–140 (17–22)		130–150 (20–23)	

* USBR [Gibbs and Holtz (1957)].

† After Meyerhof (1956). $\phi = 25 + 25D_r$ with more than 5 percent fines and $\phi = 30 + 25D_r$ with less than 5 percent fines. Use larger values for granular material with 5 percent or less fine sand and silt.

‡ It should be noted that excavated material or material dumped from a truck will weigh 70 to 90 pcf. Material must be quite dense and hard to weigh much over 130 pcf. Values of 105 to 115 pcf for nonsaturated soils are common.

Table 3-4. Empirical values for q_u* and consistency of cohesive soils based on the standard penetration number

Consistency		Very soft		Soft		Medium		Stiff		Very stiff		Hard
q_u, ksf (2cu)	0		0.5		1.0		2.0		4.0		8.0	
N, standard penetration resistance	0		2		4		8		16		32	
$\gamma_{(sat)}$, pcf (kN/m^3)		100–120 (16–19)				110–130 (17–20)		120–140 (19–22)				

* These values should be used as a guide only. Local cohesive samples should be tested, and the relationship between N and the unconfined compressive strength q_u established as $q_u = KN$.

Table 5-2. Summary of Typical $\sigma_{bond\ nominal\ strength}$ Values (Grout-to-Ground Bond) for Preliminary Micropile Design that have been used in Practice.
(Taken from FHWA -SA-97-070 "Micropile Design & Construction Guidelines")

Soil/Rock Description	**Typical Range of Grout-to-Ground Bond Nominal Strengths (psi)**			
	Type A	**Type B**	**Type C**	**Type D**
Silt & Clay (some sand) (soft, medium plastic)	5 - 10	5 - 14	7 - 17	7 - 21
Silt & Clay (some sand) (stiff, dense to very dense)	7 - 17	10 - 28	14 - 28	14 - 28
Sand (some silt) (fine, loose-medium dense	10 - 21	10 - 28	14 - 28	14 - 35
Sand (some silt, gravel) (fine-coarse, med.-very dense)	14 - 31	17 - 52	21 - 52	21 - 56
Gravel (some sand) (medium-very dense)	14 - 38	17 - 52	21 - 52	21 - 56
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	14 - 28	14 - 45	17 - 45	17 - 49
Soft Shales (fresh-moderate fracturing, little to no weathering)	30 - 80	N/A	N/A	N/A
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	75 - 200	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	150 - 300	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	75 - 250	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	200 - 609	N/A	N/A	N/A

Type A - Gravity grout only
Type B - Pressure grouted through the casing during withdrawal
Type C - Primary grout placed under gravity head, then one phase of secondary "global" pressure grouting
Type D - Primary grout placed under gravity head, then one or more phases of secondary "global" pressure grouting

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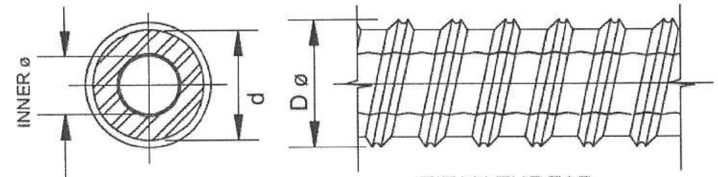


TITAN THREAD

Left Hand (shown), Sizes 30/16-52/26
Right Hand, Sizes 73/53 - 103/78-51

Rod Size D Ø / INNER Ø (mm)	Thread	Area (in²)	Area (mm²)	Ultimate G.U.T.S. (kips)	Ultimate G.U.T.S. (kN)	Yield (kips)	Yield (kN)	Max. Test (kips)	Max. Test (kN)	Outside Diameter Effective d Ø (in)	Effective d Ø (mm)	Nominal D Ø (in)	Nominal D Ø (mm)	Weight (lbs./lf.)	Weight (kg/m)
30/16	L.H. THREAD	0.53	340	55.1	245	42.7	190	41.6	185	1.02	26	1.18	30	1.8	2.7
30/14	L.H. THREAD	0.58	375	61.8	275	49.5	220	48.3	215	1.03	26	1.18	30	1.9	2.9
30/11	L.H. THREAD	0.64	415	72.0	320	58.5	260	57	255	1.03	26	1.18	30	2.2	3.3
40/20	L.H. THREAD	1.13	730	121.4	540	95.6	425	94.4	420	1.42	36	1.57	40	3.8	5.6
40/16	L.H. THREAD	1.40	900	148.4	660	118.1	525	116.9	520	1.42	36	1.57	40	4.8	7.2
52/26	L.H. THREAD	1.94	1250	208.0	925	164.2	730	163.0	725	1.92	49	2.05	52	6.7	9.9
73/56	R.H. THREAD	2.11	1360	232.7	1035	186.6	830	185.5	825	2.76	70	2.87	73	7.3	10.8
73/53	R.H. THREAD	2.50	1615	260.9	1160	218.1	970	208.7	928	2.76	70	2.87	73	8.9	13.2
73/45	R.H. THREAD	3.50	2260	356.4	1585	285.6	1270	284.5	1265	2.76	70	2.87	73	12.0	17.8
73/35	R.H. THREAD	4.20	2710	419.4	1865	321.6	1430	320.4	1425	2.76	70	2.87	73	14.2	21.2
103/78	R.H. THREAD	4.87	3140	510.5	2270	404.8	1800	403.6	1795	3.94	100	4.06	103	17.0	25.3
103/51	R.H. THREAD	8.80	5680	823.0	3660	600.4	2670	599.3	2665	3.94	100	4.06	103	30.0	44.6
127/103	R.H. THREAD	5.39	3475	521.7	2320	456.5	2030	417.4	1856	4.84	123	5.00	127	19.2	28.6

Imperial values converted from metric values; July 2010

Note:
Subject to change without notice.

Shear Force
Allowable shear force is determined by the formula:

$$Q_{allow} = \frac{\text{Yield} * A}{1.75 * \sqrt{3}}$$

A: Area

Certified to
ISO 9001

TITAN 127/111:
Allowable bending moment = 23.9 kNm (737.5 lbsft)

A3a

ISCHEBECK® TITAN

Table 1: Sacrificial loss of steel on Titan hollow bars

			60 years			120 years		
Bar size	Cross section	Ground aggressivity	Diameter loss (mm)	Reduced area (mm²)	% loss	Diameter loss (mm)	Reduced area (mm²)	% loss
		Non	0.9	342	10.5	1.5	318	17.0
30/16	338mm²	Mild	1.5	318	17.0	2.5	278	27.0
		Aggressive	2.9	263	31.0	4.9	190	50.0
		Non	0.9	349	9.5	1.5	325	15.5
30/14	385mm²	Mild	1.5	325	15.5	2.5	287	25.4
		Aggressive	2.9	287	25.4	4.9	202	47.5
		Non	0.9	408	8.5	1.5	384	14.0
30/11	446mm²	Mild	1.5	384	14.0	2.5	346	22.5
		Aggressive	2.9	331	26.0	4.9	261	41.5
		Non	0.9	715	6.8	1.5	681	11.2
40/20	767mm²	Mild	1.5	681	11.2	2.5	626	18.4
		Aggressive	2.9	626	18.4	4.9	500	34.8
		Non	0.9	828	5.8	1.5	794	9.7
40/16	879mm²	Mild	1.5	794	9.7	2.5	739	16.0
		Aggressive	2.9	718	18.3	4.9	613	30.3
		Non	0.9	1271	5.0	1.5	1226	8.3
52/26	1337mm²	Mild	1.5	1226	8.3	2.5	1153	14.0
		Aggressive	2.9	1124	16.0	4.9	983	26.5
		Non	0.9	1533	6.0	1.5	1469	10.0
73/53	1631mm²	Mild	1.5	1469	9.9	2.5	1415	13.0
		Aggressive	2.9	1320	19.0	4.9	1112	32.0
		Non	0.9	2998	4.7	1.5	2904	7.7
103/78	3146mm²	Mild	1.5	2904	7.7	2.5	2750	12.6
		Aggressive	2.9	2688	14.6	4.9	2385	24.2
		Non	0.9	6145	2.3	1.5	6049	3.8
103/51	6290mm²	Mild	1.5	6049	3.8	2.5	5890	6.4
		Aggressive	2.9	5890	6.4	4.9	5516	12.3
		Non	0.9	10263	1.8	1.5	10141	2.9
130/60	10446mm²	Mild	1.5	10141	2.9	2.5	9940	4.8
		Aggressive	2.9	9940	4.8	4.9	9464	9.4

A4

core nails. The magnitude of the increase in bond and the stiffness characteristics should be studied further based on side-by-side comparison of nails installed using the hollow-core construction technique and the conventional "drill-and-grout" technique in a given geomaterial.

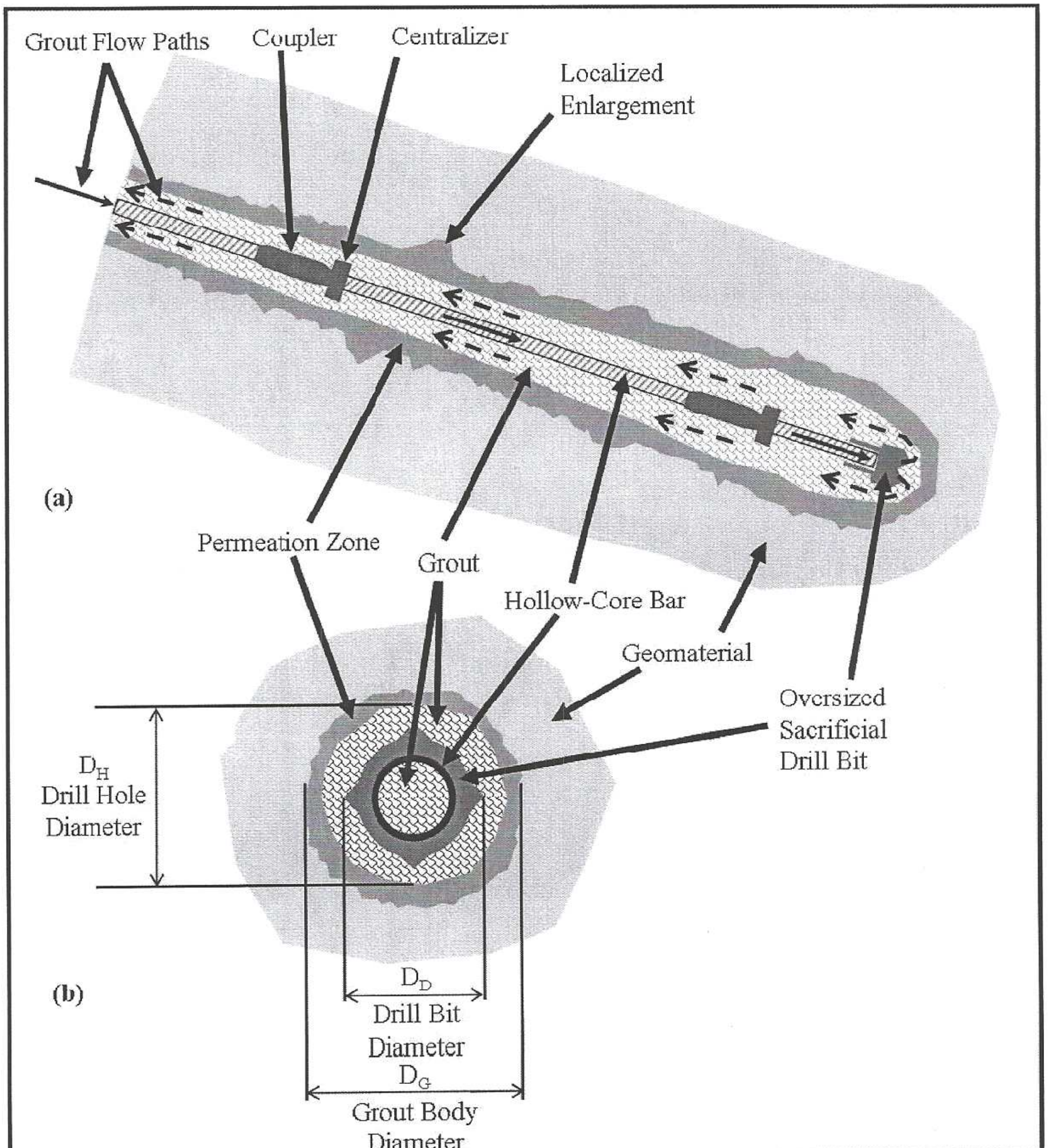


Figure 2. (a) Schematic of hollow-core soil nail installation and grout paths, (b) Schematic of a cross-section of hollow-core soil nail and the grout body.

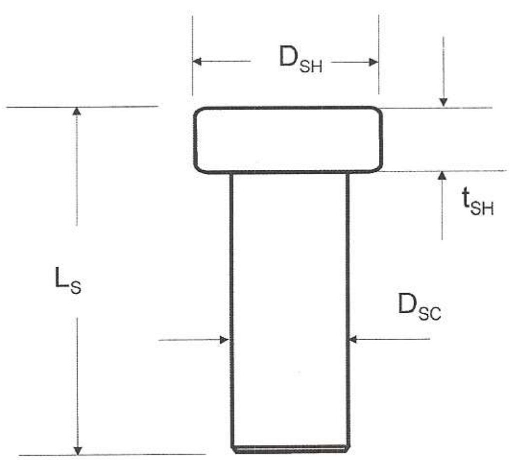
TABLE A.6
HEADED-STUD DIMENSIONS

Headed-Stud Size	Nominal Length		Head Diameter		Shaft Diameter		Head Thickness		Head Area/ Shaft Area	Head Thickness/ (Head Diameter-Shaft Diameter)
	L_s		D_H		D_S		t_H			
	mm	in.	mm	in.	mm	in.	in.	mm	in.	
1/4 x 4 1/8	105	4.125	12.7	0.5	6.4	0.25	4.7	0.19	4.0	0.75
3/8 x 4 1/8	105	4.125	19.1	0.75	9.7	0.38	7.1	0.28	4.0	0.75
3/8 x 6 1/8	156	6.125	19.1	0.75	9.7	0.38	7.1	0.28	4.0	0.75
1/2 x 4 1/8	105	4.125	25.4	1	12.7	0.5	7.9	0.31	4.0	0.62
1/2 x 5 5/16	135	5.3125	25.4	1	12.7	0.5	7.9	0.31	4.0	0.62
1/2 x 6 1/8	156	6.125	25.4	1	12.7	0.5	7.9	0.31	4.0	0.62
5/8 x 6 9/16	162	7.875	31.8	1.3	15.9	0.625	7.9	0.31	4.0	0.50
3/4 x 3 11/16	89	15.5	31.8	1.3	19.1	0.750	9.5	0.38	2.8	0.75
3/4 x 4 3/16	106	4.1875	31.8	1.25	19.1	0.75	9.5	0.38	2.8	0.75
3/4 x 5 3/16	132	5.1875	31.8	1.25	19.1	0.75	9.5	0.38	2.8	0.75
3/4 x 6 3/16	157	6.1875	31.8	1.25	19.1	0.75	9.5	0.38	2.8	0.75
7/8 x 4 3/16	102	4	34.9	1.4	22.2	0.875	9.5	0.38	2.5	0.75
7/8 x 5 3/16	127	5	34.9	1.4	22.2	0.875	9.5	0.38	2.5	0.75
7/8 x 6 3/16	152	6	34.9	1.4	22.2	0.875	9.5	0.38	2.5	0.75

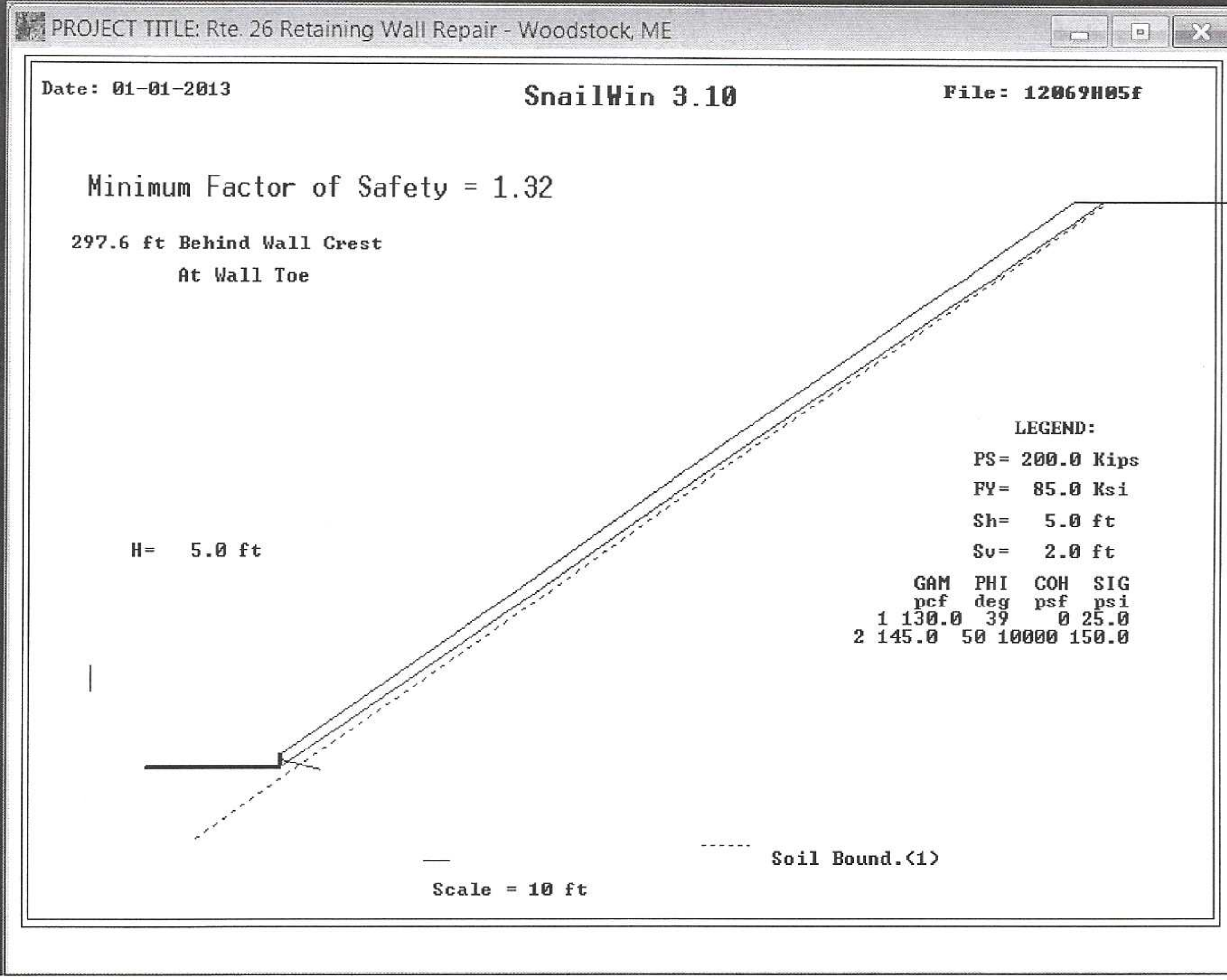
Source: Byrne et al. (1998).

Nominal length indicated is before welding.

- For $\mathbf{D_S} \le 1/2$", L_S is approximately 1/8" shorter after welding.
- For $\mathbf{D_S} > 5/8$", L_S is approximately 3/16" shorter after welding.



B1


PROJECT TITLE: Rte. 26 Retaining Wall Repair - Woodstock, ME
Date: 01-01-2013
SnailWin 3.10
File: 12069H05f
Minimum Factor of Safety = 1.32
297.6 ft Behind Wall Crest
At Wall Toe
H= 5.0 ft
LEGEND:
PS= 200.0 Kips
FY= 85.0 Ksi
Sh= 5.0 ft
Sv= 2.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 130.0 39 0 25.0
2 145.0 50 10000 150.0
Soil Bound.(1)
Scale = 10 ft

B2


```
**************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*          ENGINEERING SERVICE CENTER             *
*     DIVISION OF MATERIALS AND FOUNDATIONS       *
*   Office of Roadway Geotechnical Engineering    *
*    Date: 01-01-2013          Time: 13:33:23     *
**************************************************
```

Project Identification - Rte. 26 Retaining Wall Repair - Woodstock, ME

--------- WALL GEOMETRY ---------

		Angle (Deg)	Length (Feet)
Vertical Wall Height	=	5.0 ft	
Wall Batter	=	4.8 degree	
First Slope from Wallcrest.	=	35.0	350.0
Second Slope from 1st slope.	=	0.0	100.0
Third Slope from 2nd slope.	=	0.0	0.0
Fourth Slope from 3rd slope.	=	0.0	0.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

There is NO SURCHARGE imposed on the system.

--------- OPTION #1 ---------

Ultimate Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	130.0	39.0	0.0	25.0	0.0	0.0	0.0	0.0
2	145.0	50.0	10000.0	150.0	0.0	-4.0	50.0	31.0

* Ultimate bond Stress values also depend on BSF (Bond Stress Factor.)

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 290.0 to 310.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 1
Horizontal Spacing	= 5.0 ft
Yield Stress of Reinforcement	= 85.0 ksi
Diameter of Grouted Hole	= 3.0 in
Punching Shear	= 200.0 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	15.0	15.0	2.0	1.13	0.65

B4

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.361	292.0	35.2	357.2	89.9	0.0

Reinf. Stress at Level 1 = 62.436 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.340	294.0	35.0	358.8	89.9	0.0

Reinf. Stress at Level 1 = 63.435 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.326	296.0	34.8	360.5	89.9	0.0

Reinf. Stress at Level 1 = 64.114 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.316	298.0	34.6	362.1	89.9	0.0

Reinf. Stress at Level 1 = 64.569 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	8.075	300.0	7.8	151.4	51.0	238.3

Reinf. Stress at Level 1 = 10.106 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6	8.016	302.0	7.8	152.4	50.8	238.9

Reinf. Stress at Level 1 = 10.156 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 7	7.913	304.0	7.7	153.4	50.6	239.6

Reinf. Stress at Level 1 = 10.264 Ksi (Pullout controls...)

MINIMUM DISTANCE LOWER FAILURE UPPER FAILURE

B5

SAFETY FACTOR	BEHIND WALL TOE (ft)	PLANE ANGLE (deg)	PLANE LENGTH (ft)	PLANE ANGLE (deg)	PLANE LENGTH (ft)

NODE 8

7.861	306.0	7.7	154.4	50.4	240.2

Reinf. Stress at Level 1 = 10.308 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 9

7.806	308.0	7.6	155.4	50.3	240.8

Reinf. Stress at Level 1 = 10.356 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

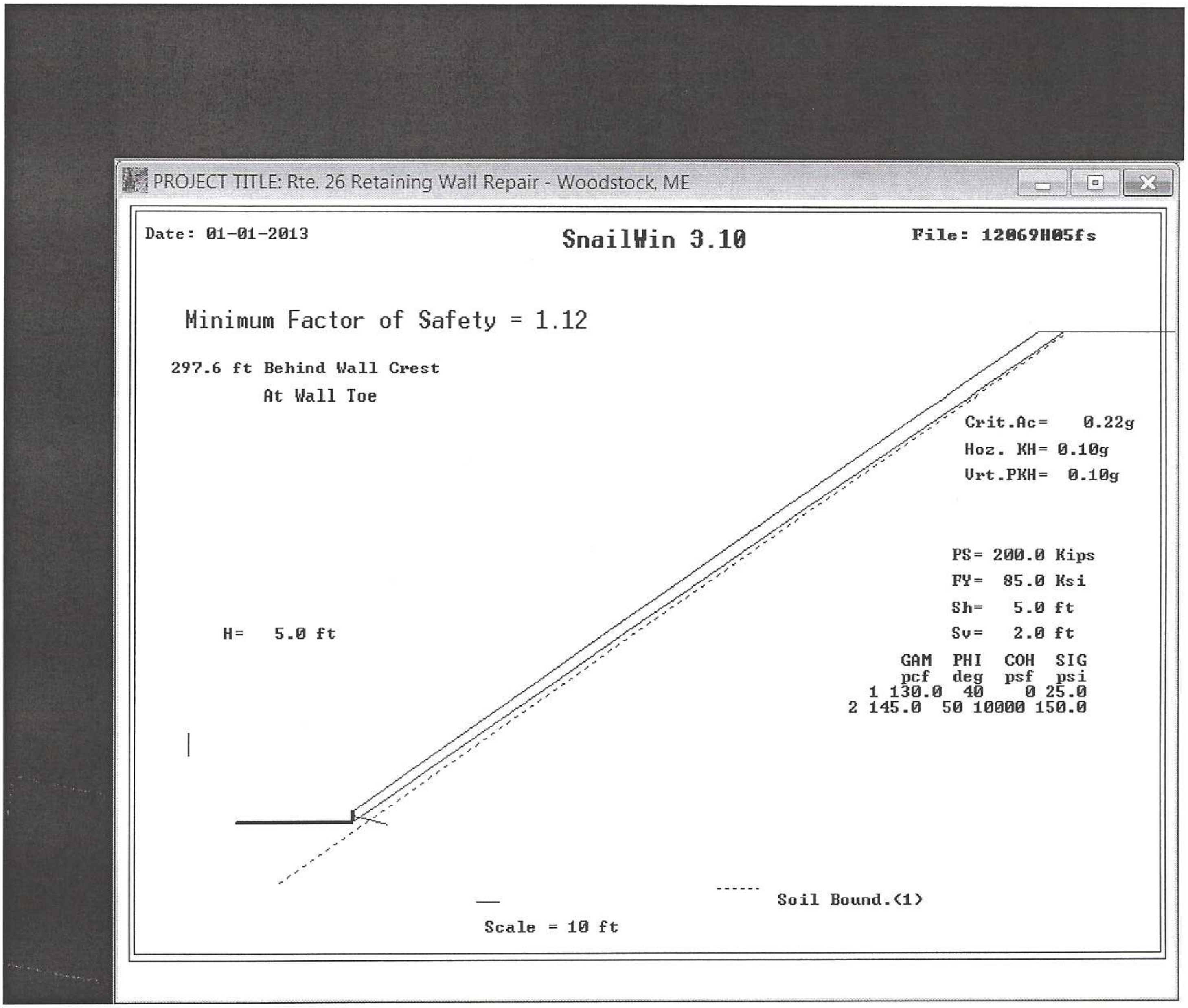
NODE10

7.751	310.0	7.6	156.4	50.1	241.5

Reinf. Stress at Level 1 = 10.405 Ksi (Pullout controls...)

```
**********************************************************************
*                  For Factor of Safety = 1.0                        *
*          Maximum Average Reinforcement Working Force:              *
*                      0.000 Kips/level                              *
**********************************************************************
```

C1



PROJECT TITLE: Rte. 26 Retaining Wall Repair - Woodstock, ME
Date: 01-01-2013
SnailWin 3.10
File: 12069H05fs
Minimum Factor of Safety = 1.12
297.6 ft Behind Wall Crest
At Wall Toe
Crit.Ac= 0.22g
Hoz. KH= 0.10g
Vrt.PKH= 0.10g
PS= 200.0 Kips
FY= 85.0 Ksi
Sh= 5.0 ft
Sv= 2.0 ft
H= 5.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 130.0 40 0 25.0
2 145.0 50 10000 150.0
Soil Bound.(1)
Scale = 10 ft

C2


```
**************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*          ENGINEERING SERVICE CENTER             *
*     DIVISION OF MATERIALS AND FOUNDATIONS       *
*   Office of Roadway Geotechnical Engineering    *
*    Date: 01-01-2013          Time: 13:04:19     *
**************************************************
```

Project Identification - Rte. 26 Retaining Wall Repair - Woodstock, ME

--------- WALL GEOMETRY ---------

		Angle (Deg)	Length (Feet)
Vertical Wall Height	=	5.0 ft	
Wall Batter	=	4.8 degree	
First Slope from Wallcrest.	=	35.0	350.0
Second Slope from 1st slope.	=	0.0	100.0
Third Slope from 2nd slope.	=	0.0	0.0
Fourth Slope from 3rd slope.	=	0.0	0.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

There is NO SURCHARGE imposed on the system.

--------- OPTION #1 ---------

Ultimate Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	130.0	40.0	0.0	25.0	0.0	0.0	0.0	0.0
2	145.0	50.0	10000.0	150.0	0.0	-4.0	50.0	31.0

* Ultimate bond Stress values also depend on BSF (Bond Stress Factor.)

C3

--------- EARTHQUAKE ACCELERATION ---------

Horizontal Earthquake Coefficient = 0.10 (a/g)
Vertical Earthquake Coefficient = 1.00

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 290.0 to 310.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels = 1
Horizontal Spacing = 5.0 ft
Yield Stress of Reinforcement = 85.0 ksi
Diameter of Grouted Hole = 3.0 in
Punching Shear = 200.0 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	15.0	15.0	2.0	1.13	0.65

c4

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.169	292.0	35.2	357.2	89.9	0.0

Reinf. Stress at Level 1 = 72.684 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.145	294.0	35.0	358.8	89.9	0.0

Reinf. Stress at Level 1 = 74.224 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.129	296.0	34.8	360.5	89.9	0.0

Reinf. Stress at Level 1 = 75.321 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.117	298.0	34.6	362.1	89.9	0.0

Reinf. Stress at Level 1 = 76.113 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	7.439	300.0	7.8	151.4	51.0	238.3

Reinf. Stress at Level 1 = 10.969 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6	7.366	302.0	7.8	152.4	50.8	238.9

Reinf. Stress at Level 1 = 11.053 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 7	7.296	304.0	7.7	153.4	50.6	239.6

Reinf. Stress at Level 1 = 11.132 Ksi (Pullout controls...)

MINIMUM DISTANCE LOWER FAILURE UPPER FAILURE

	SAFETY FACTOR	BEHIND WALL TOE (ft)	PLANE ANGLE (deg)	PLANE LENGTH (ft)	PLANE ANGLE (deg)	PLANE LENGTH (ft)
NODE 8	7.230	306.0	7.7	154.4	50.4	240.2

Reinf. Stress at Level 1 = 11.208 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 9	7.138	308.0	7.6	155.4	50.3	240.8

Reinf. Stress at Level 1 = 11.326 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE10	7.084	310.0	7.6	156.4	50.1	241.5

Reinf. Stress at Level 1 = 11.386 Ksi (Pullout controls...)

```
**********************************************************************
*                  For Factor of Safety = 1.0                        *
*          Maximum Average Reinforcement Working Force:              *
*                     19.113 Kips/level                              *
**********************************************************************
```

D1

PROJECT TITLE: Rte. 26 Retaining Wall Repair - Woodstock, ME

Date: 01-02-2013

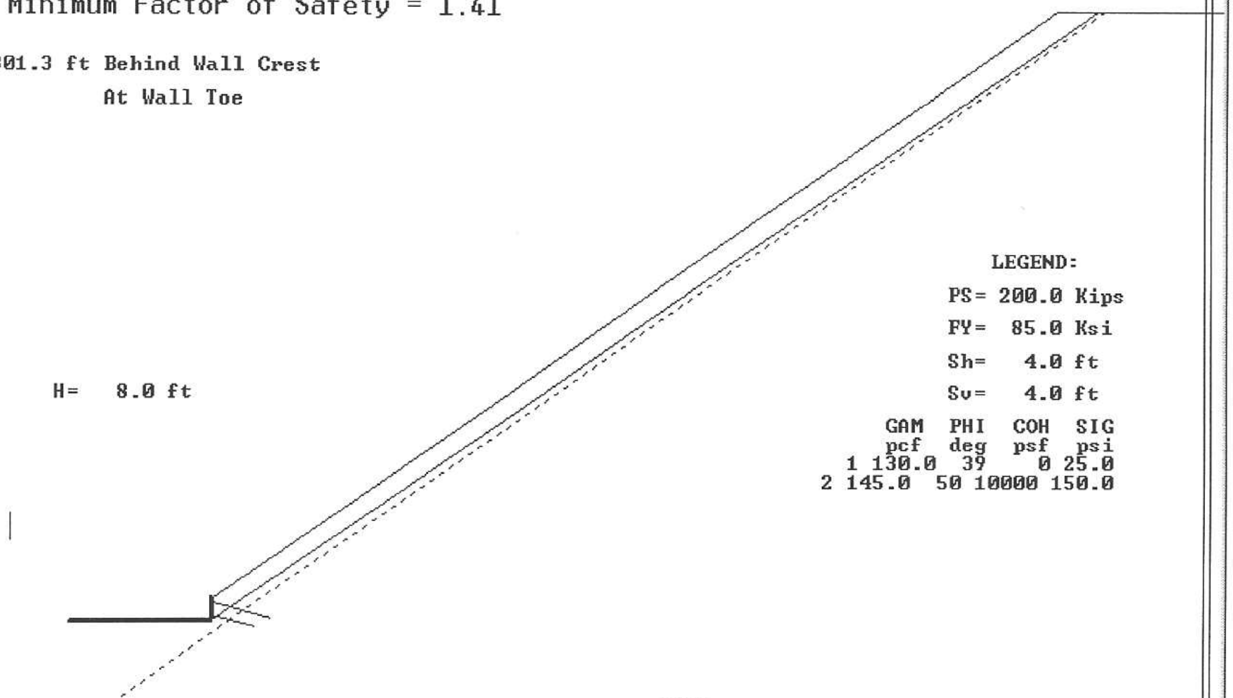
SnailWin 3.10

File: 12069H08f

Minimum Factor of Safety = 1.41

301.3 ft Behind Wall Crest

At Wall Toe



H= 8.0 ft

LEGEND:

PS= 200.0 Kips

FY= 85.0 Ksi

Sh= 4.0 ft

Sv= 4.0 ft

	GAM	PHI	COH	SIG
	pcf	deg	psf	psi
1	130.0	39	0	25.0
2	145.0	50	10000	150.0

------ Soil Bound.(1)

Scale = 10 ft

D2


```
**************************************************
*     CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*           ENGINEERING SERVICE CENTER             *
*      DIVISION OF MATERIALS AND FOUNDATIONS       *
*    Office of Roadway Geotechnical Engineering    *
*     Date: 01-02-2013          Time: 10:50:08     *
**************************************************
```

Project Identification - Rte. 26 Retaining Wall Repair - Woodstock, ME

--------- WALL GEOMETRY ---------

```
 Vertical Wall Height           =   8.0 ft
 Wall Batter                    =   4.8 degree
                                  Angle   Length
                                  (Deg)   (Feet)
 First Slope from Wallcrest.   =  35.0    350.0
 Second Slope from 1st slope. =   0.0    100.0
 Third Slope from 2nd slope.  =   0.0      0.0
 Fourth Slope from 3rd slope. =   0.0      0.0
 Fifth Slope from 3rd slope.  =   0.0      0.0
 Sixth Slope from 3rd slope.  =   0.0      0.0
 Seventh Slope Angle.         =   0.0
```

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

There is NO SURCHARGE imposed on the system.

--------- OPTION #1 ---------

Ultimate Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	130.0	39.0	0.0	25.0	0.0	0.0	0.0	0.0
2	145.0	50.0	10000.0	150.0	0.0	-4.0	50.0	31.0

* Ultimate bond Stress values also depend on BSF (Bond Stress Factor.)

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 290.0 to 310.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 2
Horizontal Spacing	= 4.0 ft
Yield Stress of Reinforcement	= 85.0 ksi
Diameter of Grouted Hole	= 3.0 in
Punching Shear	= 200.0 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	20.0	15.0	2.0	1.13	0.65
2	15.0	15.0	4.0	1.13	0.65

D4


```
          MINIMUM     DISTANCE     LOWER FAILURE    UPPER FAILURE
          SAFETY       BEHIND          PLANE            PLANE
          FACTOR      WALL TOE     ANGLE  LENGTH    ANGLE  LENGTH
                        (ft)       (deg)   (ft)     (deg)   (ft)

   Toe    1.511        292.0        35.6  358.9      89.9    0.0

  Reinf. Stress at Level  1 =  56.243 Ksi (Yield Stress controls.)
                          2 =  56.243 Ksi (Yield Stress controls.)

          MINIMUM     DISTANCE     LOWER FAILURE    UPPER FAILURE
          SAFETY       BEHIND          PLANE            PLANE
          FACTOR      WALL TOE     ANGLE  LENGTH    ANGLE  LENGTH
                        (ft)       (deg)   (ft)     (deg)   (ft)

NODE 2
     1.479       294.0        35.4  360.6      89.9    0.0

  Reinf. Stress at Level  1 =  57.461 Ksi (Yield Stress controls.)
                          2 =  57.461 Ksi (Yield Stress controls.)

          MINIMUM     DISTANCE     LOWER FAILURE    UPPER FAILURE
          SAFETY       BEHIND          PLANE            PLANE
          FACTOR      WALL TOE     ANGLE  LENGTH    ANGLE  LENGTH
                        (ft)       (deg)   (ft)     (deg)   (ft)

NODE 3
     1.455       296.0        35.2  362.2      89.9    0.0

  Reinf. Stress at Level  1 =  58.417 Ksi (Yield Stress controls.)
                          2 =  58.417 Ksi (Yield Stress controls.)

          MINIMUM     DISTANCE     LOWER FAILURE    UPPER FAILURE
          SAFETY       BEHIND          PLANE            PLANE
          FACTOR      WALL TOE     ANGLE  LENGTH    ANGLE  LENGTH
                        (ft)       (deg)   (ft)     (deg)   (ft)

NODE 4
     1.437       298.0        35.0  363.8      89.9    0.0

  Reinf. Stress at Level  1 =  59.169 Ksi (Yield Stress controls.)
                          2 =  59.169 Ksi (Yield Stress controls.)

          MINIMUM     DISTANCE     LOWER FAILURE    UPPER FAILURE
          SAFETY       BEHIND          PLANE            PLANE
          FACTOR      WALL TOE     ANGLE  LENGTH    ANGLE  LENGTH
                        (ft)       (deg)   (ft)     (deg)   (ft)

NODE 5
     1.422       300.0        34.8  365.5      89.9    0.0

  Reinf. Stress at Level  1 =  59.759 Ksi (Yield Stress controls.)
                          2 =  59.759 Ksi (Yield Stress controls.)

          MINIMUM     DISTANCE     LOWER FAILURE    UPPER FAILURE
          SAFETY       BEHIND          PLANE            PLANE
          FACTOR      WALL TOE     ANGLE  LENGTH    ANGLE  LENGTH
                        (ft)       (deg)   (ft)     (deg)   (ft)

NODE 6
     1.412       302.0        34.7  367.1      89.9    0.0

  Reinf. Stress at Level  1 =  60.219 Ksi (Yield Stress controls.)
                          2 =  60.219 Ksi (Yield Stress controls.)

          MINIMUM     DISTANCE     LOWER FAILURE    UPPER FAILURE
          SAFETY       BEHIND          PLANE            PLANE
          FACTOR      WALL TOE     ANGLE  LENGTH    ANGLE  LENGTH
                        (ft)       (deg)   (ft)     (deg)   (ft)
```

```
NODE 7
    7.798       304.0         7.8  153.4     51.0  241.7

  Reinf. Stress at Level  1 =   6.838 Ksi (Pullout controls...)
                          2 =  10.900 Ksi (Yield Stress controls.)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND         PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE 8
    7.712       306.0         7.8  154.4     50.8  242.3

  Reinf. Stress at Level  1 =   6.866 Ksi (Pullout controls...)
                          2 =  11.022 Ksi (Yield Stress controls.)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND         PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE 9
    7.637       308.0         7.7  155.4     50.7  242.9

  Reinf. Stress at Level  1 =   6.883 Ksi (Pullout controls...)
                          2 =  11.129 Ksi (Yield Stress controls.)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND         PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE10
    7.583       310.0         7.7  156.4     50.5  243.6

  Reinf. Stress at Level  1 =   6.883 Ksi (Pullout controls...)
                          2 =  11.210 Ksi (Yield Stress controls.)


     *********************************************************************
     *                 For Factor of Safety = 1.0                        *
     *         Maximum Average Reinforcement Working Force:              *
     *                    0.000 Kips/level                               *
     *********************************************************************
```

E1

PROJECT TITLE: Rte. 26 Retaining Wall Repair - Woodstock, ME

Date: 01-02-2013

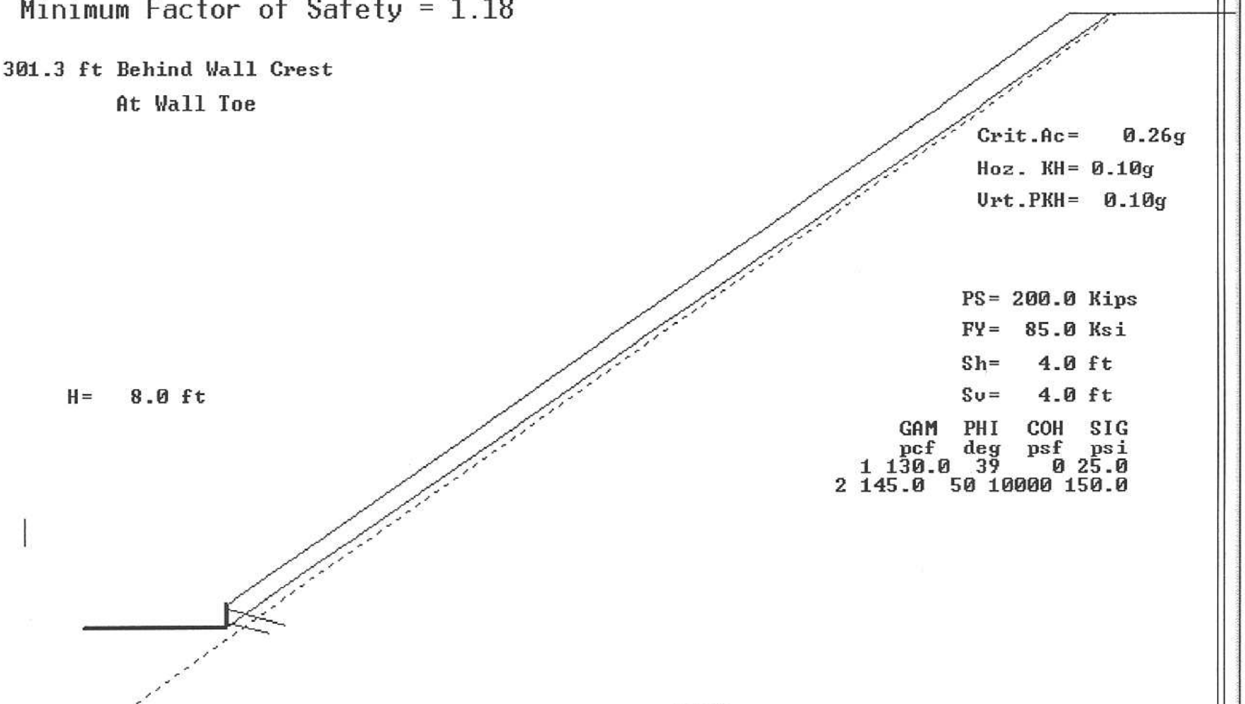
SnailWin 3.10

File: 12069H08fs

Minimum Factor of Safety = 1.18

301.3 ft Behind Wall Crest

At Wall Toe



Crit.Ac= 0.26g

Hoz. KH= 0.10g

Vrt.PKH= 0.10g

PS= 200.0 Kips

FY= 85.0 Ksi

Sh= 4.0 ft

Sv= 4.0 ft

H= 8.0 ft

	GAM pcf	PHI deg	COH psf	SIG psi
1	130.0	39	0	25.0
2	145.0	50	10000	150.0

------ Soil Bound.(1)

Scale = 10 ft

EZ


```
*************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION     *
*          ENGINEERING SERVICE CENTER            *
*     DIVISION OF MATERIALS AND FOUNDATIONS      *
*   Office of Roadway Geotechnical Engineering   *
*    Date: 01-02-2013          Time: 10:51:13    *
*************************************************
```

Project Identification - Rte. 26 Retaining Wall Repair - Woodstock, ME

--------- WALL GEOMETRY ---------

		Angle (Deg)	Length (Feet)
Vertical Wall Height	=	8.0 ft	
Wall Batter	=	4.8 degree	
First Slope from Wallcrest.	=	35.0	350.0
Second Slope from 1st slope.	=	0.0	100.0
Third Slope from 2nd slope.	=	0.0	0.0
Fourth Slope from 3rd slope.	=	0.0	0.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

There is NO SURCHARGE imposed on the system.

--------- OPTION #1 ---------

Ultimate Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	130.0	39.0	0.0	25.0	0.0	0.0	0.0	0.0
2	145.0	50.0	10000.0	150.0	0.0	-4.0	50.0	31.0

* Ultimate bond Stress values also depend on BSF (Bond Stress Factor.)

--------- EARTHQUAKE ACCELERATION ---------

Horizontal Earthquake Coefficient = 0.10 (a/g)
Vertical Earthquake Coefficient = 1.00

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 290.0 to 310.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 2
Horizontal Spacing	= 4.0 ft
Yield Stress of Reinforcement	= 85.0 ksi
Diameter of Grouted Hole	= 3.0 in
Punching Shear	= 200.0 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	20.0	15.0	2.0	1.13	0.65
2	15.0	15.0	4.0	1.13	0.65

E4

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.291	292.0	35.6	358.9	89.9	0.0

Reinf. Stress at Level 1 = 65.841 Ksi (Yield Stress controls.)
2 = 65.841 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.257	294.0	35.4	360.6	89.9	0.0

Reinf. Stress at Level 1 = 67.635 Ksi (Yield Stress controls.)
2 = 67.635 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.230	296.0	35.2	362.2	89.9	0.0

Reinf. Stress at Level 1 = 69.090 Ksi (Yield Stress controls.)
2 = 69.090 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.209	298.0	35.0	363.8	89.9	0.0

Reinf. Stress at Level 1 = 70.279 Ksi (Yield Stress controls.)
2 = 70.279 Ksi (Yield Stress controls.)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	1.193	300.0	34.8	365.5	89.9	0.0

Reinf. Stress at Level 1 = 71.253 Ksi (Yield Stress controls.)
2 = 71.253 Ksi (Yield Stress controls.)

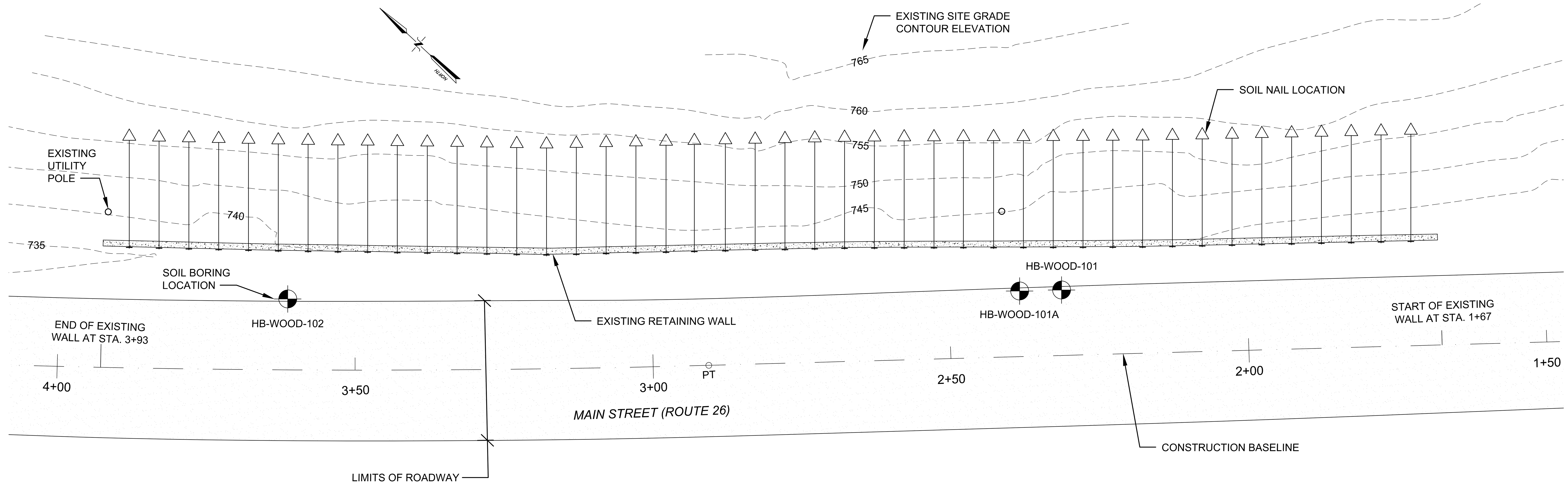
	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6	1.180	302.0	34.7	367.1	89.9	0.0

Reinf. Stress at Level 1 = 72.053 Ksi (Yield Stress controls.)
2 = 72.053 Ksi (Yield Stress controls.)

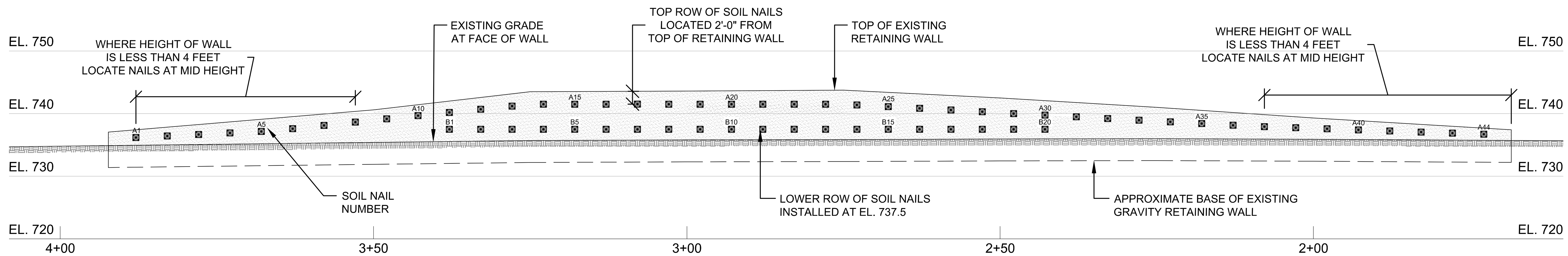
	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

E5

```
NODE 7
    7.093        304.0        7.8  153.4      51.0  241.7

  Reinf. Stress at Level  1 =   7.518 Ksi (Pullout controls...)
                          2 =  11.983 Ksi (Yield Stress controls.)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND         PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE 8
    7.032        306.0        7.8  154.4      50.8  242.3

  Reinf. Stress at Level  1 =   7.529 Ksi (Pullout controls...)
                          2 =  12.087 Ksi (Yield Stress controls.)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND         PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE 9
    6.974        308.0        7.7  155.4      50.7  242.9

  Reinf. Stress at Level  1 =   7.538 Ksi (Pullout controls...)
                          2 =  12.188 Ksi (Yield Stress controls.)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND         PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE10
    6.952        310.0        7.7  156.4      50.5  243.6

  Reinf. Stress at Level  1 =   7.508 Ksi (Pullout controls...)
                          2 =  12.227 Ksi (Yield Stress controls.)


     **********************************************************************
     *                 For Factor of Safety = 1.0                         *
     *          Maximum Average Reinforcement Working Force:              *
     *                      21.777 Kips/level                             *
     **********************************************************************
```



PLAN OF EXISTING RETAINING WALL
SCALE: 1"= 10'–0"



ELEVATION OF RETAINING WALL WITH SOIL NAILS
SCALE: 1"= 10'–0"

GENERAL NOTES

THESE PLANS DETAIL THE PERMANENT SOIL NAILS TO BE INSTALLED AS PART OF THE REPAIR WORK TO THE RETAINING WALL IN ALONG ROUTE 26 IN WOODSTOCK, ME.

SOIL NAIL INSTALLATION PROCEDURE

AFTER VERIFYING FIELD CONDITIONS ARE CONSISTENT WITH THOSE SHOWN ON THESE DRAWINGS, LAYOUT SOIL NAIL LOCATIONS ON FACE OF EXISTING WALL. SOIL NAIL LOCATIONS MAY BE ADJUSTED IN THE FIELD TO AVOID CRACKED SECTIONS OF THE EXISTING RETAINING WALL. AFTER SOIL NAIL LOCATIONS ARE FINALIZED NOTIFY DESIGN ENGINEER OF ANY REQUIRED ADJUSTMENTS REVIEW PRIOR TO PROCEEDING. ONCE LOCATIONS ARE FINALIZED DRILL CORE HOLES THROUGH EXISTING CONCRETE RETAINING WALL AT THE SOIL NAIL LOCATIONS.

DRILL PRODUCTION NAILS AT THE LOCATION, ANGLE AND LENGTH DETAILED ON THE WALL ELEVATION AND SECTIONS. SOIL NAILS WILL BE DRILLED USING HOLLOW BAR TENDONS WITH GROUT PUMPED THROUGH THE CENTERAL CORE OF THE TENDON DURING THE DRILLING OPERATION. DRILL SOIL NAIL 5 FEET INTO ROCK USING THE 40/20 HOLLOW BAR WITH 3–INCH DIA. CUTTING BIT. CUTTING BIT TO BE CAPABLE OF DRILLING THROUGH DENSE GLACIAL TILL, WEATHERED ROCK AND INTO SOUND BEDROCK. NAILS SHALL BE DRILLED A MINIMUM OF FIVE FEET INTO SOUND BEDROCK. IF BEDROCK IS NOT ENCOUNTERED WITHIN 20 FEET FROM THE FACE OF THE RETAINING WALL NOTIFY THE DESIGN ENGINEER BEFORE PROCEEDING.

AFTER THE PRODUCTION NAILS HAVE BEEN INSTALLED AND SET FOR AT LEAST 24 HOURS THE NUTS SHALL THEN BE TIGHTENED WITH A TORQUE WRENCH BY APPLYING A TORQUE OF 200 FT.–LBS. AFTER COMPLETION OF SOIL NAILS A PERMANENT CONCRETE FACING IS TO BE CAST AGAINST THE EXISTING RETAINING WALL BY OTHERS.

MATERIAL SCHEDULE

ITEM	MATERIAL	GRADE
SOIL NAIL TENDON	40/20 HOLLOW BAR	Fy=85 ksi
BEARING PLATE	12"x 12"x ½"	ASTM A36 (Fy=36 ksi)
NAIL GROUT	W/C RATIO ≈ 0.50	fc=3000 psi

NOTES:
1. SEE SHEET 2 OF 2 FOR DETAIL OF BEARING PLATE.
2. MATERIALS OF EQUIVALENT OR GREATER STRENGTH MAY BE SUBSTITUTED UPON APPROVAL BY THE DESIGN ENGINEER.

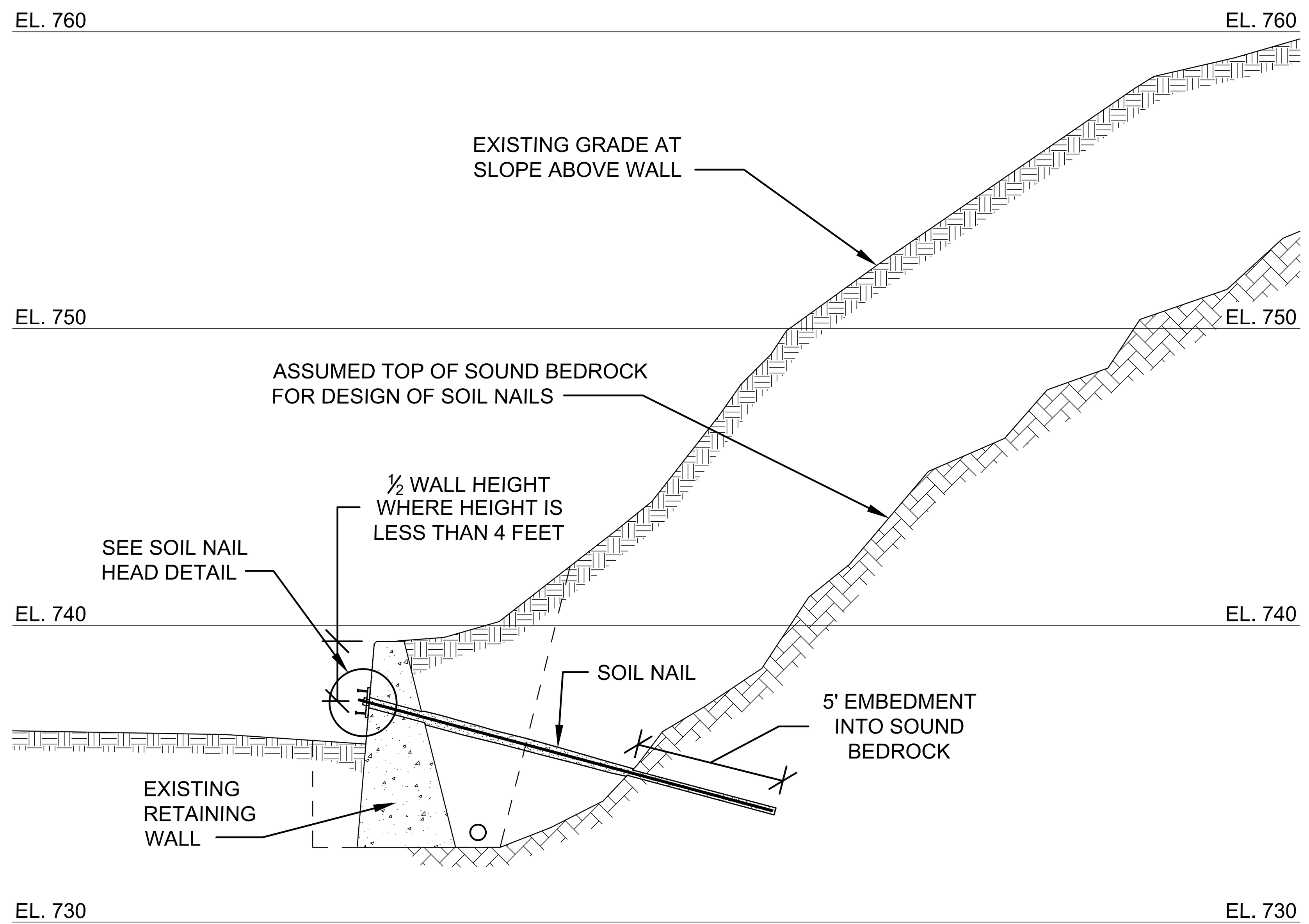
DESIGN AND DRAWING PREPARED BASED IN PART ON UNVERIFIED INFORMATION PROVIDED BY OTHERS. IF ACTUAL SITE AND/OR SOIL CONDITIONS VARY FROM THOSE SHOWN ON THIS DRAWING NOTIFY THE DESIGN ENGINEER TO REVIEW PRIOR TO THE START OF CONSTRUCTION.

NO.	DATE	REVISIONS

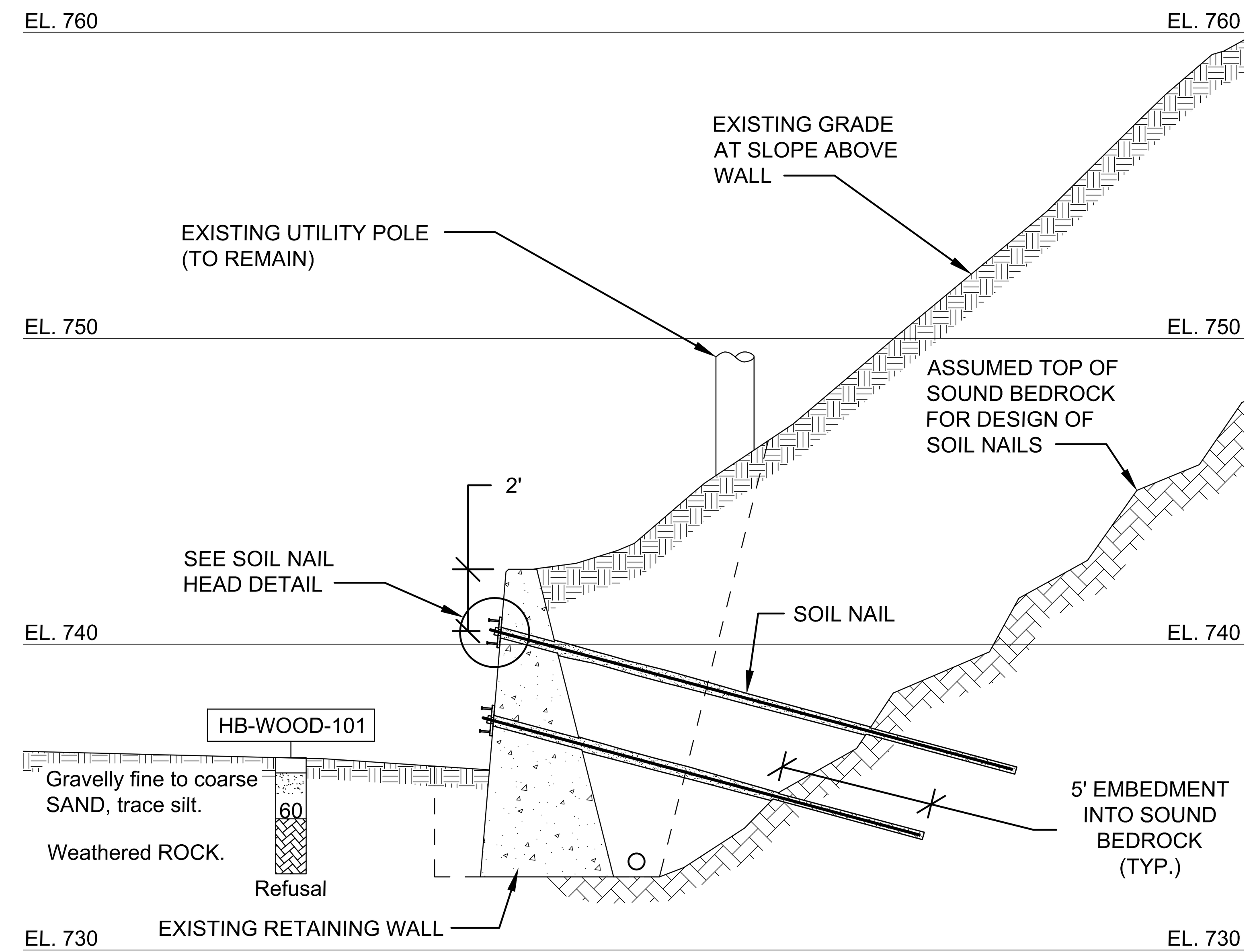
STATE OF MAINE
PAUL A. DETERLING
No. 5508
REGISTERED PROFESSIONAL ENGINEER

DESIGN ENGINEER	CONTRACTOR	PROJECT	DRAWING TITLE
EARTHWORK ENGINEERING, INC. 175 Ridge Road - Hollis, NH 03049 Tel. (603) 465-9500 - Fax (603) 465-9650	THOMAS DRILLING AND BLASTING Route 9 - Spofford, NH 03462 Tel. (603) 363-4707	RETAINING WALL REPAIR ROUTE 26 (MAIN STREET) WOODSTOCK, MAINE	PERMANENT SOIL NAIL SYSTEM SITE PLAN, WALL ELEVATION GENERAL NOTES AND SCHEDULE

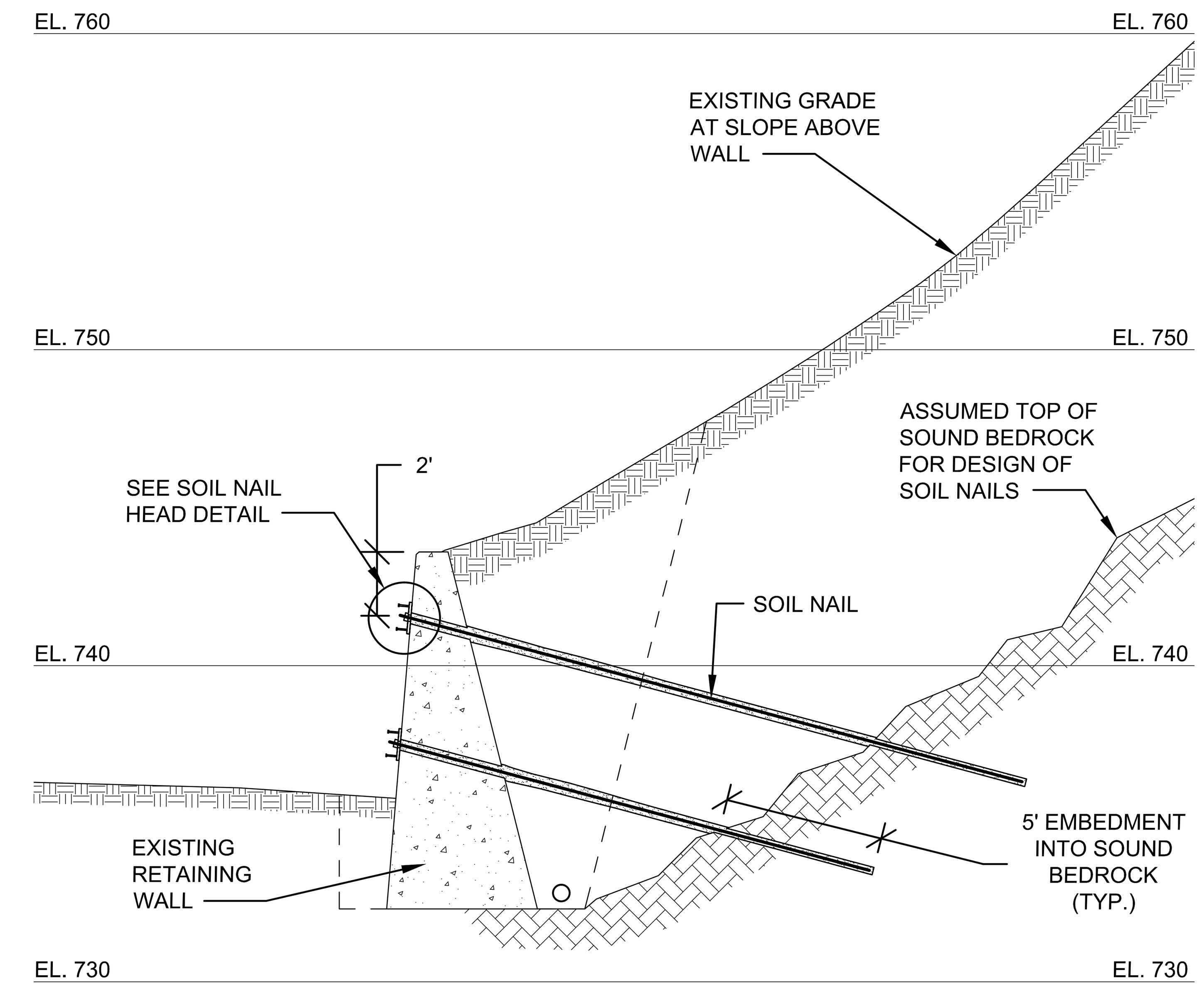
DRWG BY: PAD
DATE: 1/2/13
PROJECT: 12069
SHEET: 1 of 2



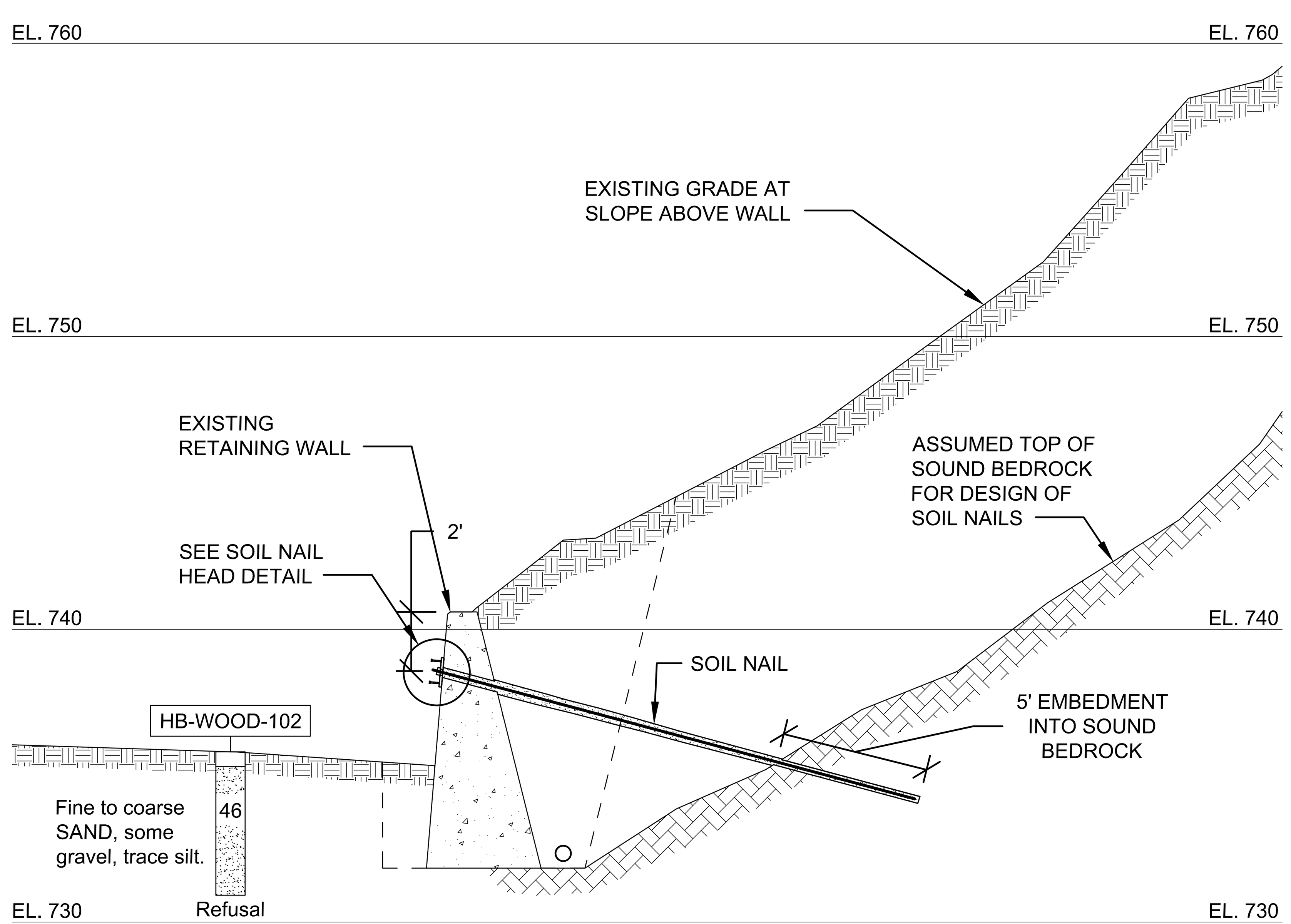
WALL SECTION AT STATION 2+00
SCALE: ¼"= 1'–0"



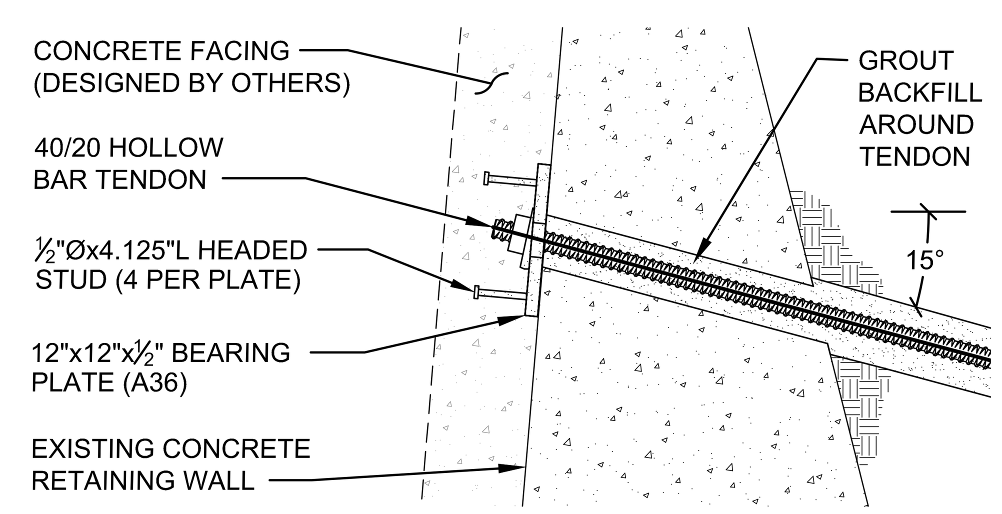
WALL SECTION AT STATION 2+50
SCALE: ¼"= 1'–0"



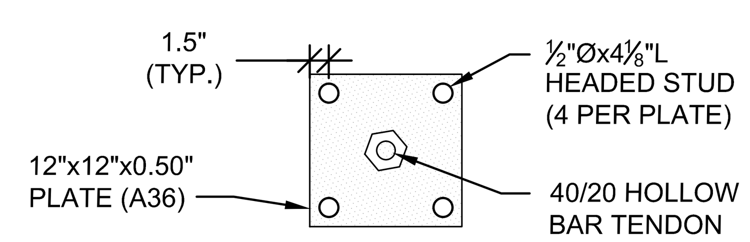
WALL SECTION AT STATION 3+00
SCALE: ¼"= 1'–0"



WALL SECTION AT STATION 3+50
SCALE: ¼"= 1'–0"



SOIL NAIL HEAD DETAIL
SCALE: 1"= 1'–0"



SOIL NAIL PLATE DETAIL
SCALE: 1"= 1'–0"

DESIGN AND DRAWING PREPARED BASED IN PART ON UNVERIFIED INFORMATION PROVIDED BY OTHERS. IF ACTUAL SITE AND SOIL CONDITIONS VARY FROM THOSE SHOWN NOTIFY THE DESIGN ENGINEER TO REVIEW PRIOR TO THE START OF CONSTRUCTION.

NO.	DATE	REVISIONS

STATE OF MAINE · PAUL A. DETERLING · No. 5506 · REGISTERED · PROFESSIONAL ENGINEER

DESIGN ENGINEER	CONTRACTOR	PROJECT	DRAWING TITLE	
EARTHWORK ENGINEERING, INC. 175 Ridge Road - Hollis, NH 03049 Tel. (603) 465-9500 - Fax (603) 465-9650	THOMAS DRILLING AND BLASTING Route 9 - Spofford, NH 03462 Tel. (603) 363-4707 - Fax (603) 363-4249	RETAINING WALL REPAIR ROUTE 26 - WOODSTOCK, MAINE MAINE D.O.T. PROJECT NO. 19168	PERMANENT SOIL NAIL SYSTEM SITE PLAN, WALL ELEVATION GENERAL NOTES AND SCHEDULE	DRWG BY: PAD DATE: 1/2/13 PROJECT: 12069 SHEET: 2 of 2