MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CORSON CORNER BRIDGE
STATE ROUTES 43/151 OVER BLACK STREAM
ATHENS-HARTLAND, MAINE

Prepared by:
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Somerset County | Soils Report 2017-47
WIN 20815.00 | Bridge No. 2135

Fed No. STP-2081(500)
October 5, 2017

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Corson Corner Bridge which carries State Routes 43 and 151 over Black Stream between Athens and Hartland, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical design parameters, and construction recommendations for the replacement bridge substructures.

The existing structure was constructed in 1957. The existing structure is comprised of a single 21-foot span corrugated steel arch with concrete strip footings founded on soil. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the structure is “Structurally Deficient”, with a Sufficiency Rating of 68.8. The culvert is considered in poor condition and rated as a 4 with scattered holes and unzipping of plates resulting in fill material loss. The existing structure is considered “Scour Critical.” The westerly footing is undermined and both footings are exposed for most of their length. The bridge must be monitored during large storm events.

The proposed replacement structure will be a 24-foot span by 12-foot rise precast concrete box culvert. The concrete box culvert shall have 1-foot tall precast headwalls and toe walls extending one foot below calculated scour depth. The upstream and downstream ends of the culvert will be slope-tapered to match the 2H:1V (horizontal:vertical) sideslopes. The box culvert will be embedded approximately 3 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box shall be placed on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill bearing on compacted native soils.

The new Corson Corner Bridge will be constructed on nearly the same horizontal alignment as the existing bridge. The vertical profile of State Route 43/151 will increase approximately 22-inches over the buried structure to improve the vertical alignment and provide additional structural cover over the concrete box. Staged construction techniques will allow the maintenance of traffic during construction.

2.0 GEOLOGIC SETTING

The existing structure carries State Routes 43 and 151 over Black Stream between Athens and Hartland, Maine approximately 1000 feet east of the intersection of Route 43/151, Hall Farm Road, and Munns Flat Road as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Skowhegan Quadrangle, Maine, Open-file No. 86-38 (1986), indicates the surficial soils in the vicinity of the bridge project consist of glacial marine deposits with nearby contacts to glacial till. Glacial marine deposits accumulated on the ocean floor during the late-glacial marine submergence of inland river valleys. These soils are generally comprised of silt, clay, sand and minor amounts of gravel. The most common component is the clayey silt known as the

Presumpscot Formation, but sand is very abundant in some areas. The unit also may contain small areas of till, sand, and gravel that are not completely covered by the marine sediment. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones.

The Bedrock Geologic Map of the Skowhegan Quadrangle, Map Series GM-5 (1977), cites the bedrock at the project site as granitic rocks of the Hartland Pluton which ranges from hornblende-biotite granite to granodiorite with muscovite and garnet-bearing lithologies at contacts.

3.0 SUBSURFACE INVESTIGATION

Five test borings explored subsurface conditions at the site. Borings BB-AHBS-101, BB-AHBS-201, BB-AHBS-201A, and BB-AHBS-201B were drilled west of the existing structure. Boring BB-AHBS-202 was drilled east of the existing structure. The test boring locations are shown on Sheet 2 – Boring Location Plan. An interpretive subsurface profile across the site is shown on Sheet 3 – Interpretive Subsurface Profile.

The 100-series test borings were drilled on September 22 and 23, 2016. The 200-series test borings were drilled on May 3 and 4, 2017. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rigs were equipped with an automatic hammer to drive the split spoon. The hammers were calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in April 2016 and March 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.9288 to the 100-series raw field N-values and 0.873 to the 200-series raw field N-values. The hammer efficiency factors (0.9288 and 0.873) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored in test boring BB-AHBS-102 and BB-AHBS-201B using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core calculated. A geotechnical engineer logged the subsurface conditions encountered and selected the boring location and drilling methods, designated type and depth of sampling techniques, reviewed boring logs, and identified lab testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program and located by MaineDOT Region 3 survey.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of; two standard grain size analyses with natural water content, eight grain size analyses with hydrometer and natural moisture content, and four Atterberg limits tests. Soil test results are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of fill, reworked soils, relic topsoil, and glacial till. The fill unit and subsurface soils are underlain by igneous bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Fill, Reworked Soils, and Relic Topsoil

Encountered in the borings was a layer of fill, reworked soils, and relic topsoil. The combined thickness of these subunits is approximately 12.4 to 14.7 feet at the boring locations. The fill, reworked soils, and relic topsoil layers generally consisted of:

- Brown, moist, gravelly sand, some silt;
- Brown to grey, moist to wet, silty sand, little to trace gravel, trace clay, trace wood fiber;
- Brown to dark brown, damp to moist, sand, some to trace silt, some to trace gravel, trace clay, trace wood fiber;
- Brown to dark brown, moist to wet, silt, some sand, trace clay, trace gravel, trace wood fiber;
- Brown, wet, sandy silt, trace gravel, trace wood fiber;
- Grey, wet, fine to medium sand, little silt; and
- Olive, wet, silt, little clay, trace fine sand.

Corrected SPT N-values in the coarse-grained fill subunit ranged from 4 to 60 blows per foot (bpf), indicating the coarse-grained fill layer is very loose to very dense in consistency. Two grain size analyses resulted in material classifications of A-2-4 and A-4 under the AASHTO Soil Classification System and SM and SC-SM under the Unified Soil Classification System (USCS). The natural water content of the coarse-grained fill samples tested ranged from approximately 12 to 28 percent.

Corrected SPT N-values in the fine-grained fill, reworked soils, and relic topsoil subunits ranged from 3 to 12 blows per foot (bpf), indicating the fine-grained layers are soft to stiff in consistency. Five grain size analyses and three Atterberg limits tests of the fine-grained material resulted in material classification of A-4 under the AASHTO Soil Classification System and CL under the USCS. The natural water content of the samples tested ranged from approximately 33 to 90 percent. One grain size analysis and one Atterberg limits test of the coarse-grained relic topsoil layer resulted in the sample being classified as A-2-4 under the AASHTO Soil Classification System and SC-SM under USCS. The natural water content of the coarse-grained relic topsoil sample tested was approximately 38 percent.

Table 1 summarizes the results of Atterberg limits tests conducted on samples from the fill, reworked soils, and relic topsoil:

Boring/Sample No.	Subunit	Visual Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
BB-AHBS-101/4D	Fill	Silt, some sand	36.7	Non-plastic		
BB-AHBS-101/5DA	Relic Topsoil	Silt, some sand	32.6	Non-plastic		
BB-AHBS-201/4D	Fill	Silty sand	28.2	Non-plastic		
BB-AHBS-201/5D	Reworked soils	Silt, little clay	40.9	Non-plastic		

Table 1 – Summary of Atterberg Limits Tests

5.2 Glacial Till

A glacial till deposit was encountered beneath the fill, reworked soils, and relic topsoil in the borings. The thickness of the deposit encountered ranged from approximately 11.1 to 36.9 feet. The glacial till generally consisted of:

- Grey, wet, silty sand, some gravel, trace clay, cobbles;
- Brown, wet, silt, little clay, trace fine sand, cobbles;
- Brown, wet, silt, some to little sand, little to trace gravel, trace clay;
- Brown, wet, sandy silt, little sand, some gravel;
- Brown to brown-grey, wet, sand, some to trace gravel, little silt;
- Grey, wet, gravel, some sand, some silt;
- Cobbles; and
- Boulders.

Eleven SPT's failed to advance the minimum eighteen inches required to record an N-value without excessive blows from the hammer indicating the consistency of the deposit is predominately hard or very dense. One completed SPT in the upper region of the glacial till encountered resulted in a corrected N-value of 11 indicating this layer is medium dense. Three grain size analyses conducted samples of the glacial till resulted in the samples being classified as A-4 and A-2-4 under the AASHTO Soil Classification System and CL and SP-SM under the USCS. The moisture content of the tested samples ranged from approximately 16 to 23 percent.

5.3 Bedrock

Bedrock was encountered and cored in borings BB-AHBS-102 and BB-AHBS-201B. Table 2 summarizes the approximate depth to the initial bedrock core, elevations, and RQD's. The boring log for BB-AHBS-201B indicates that the rollercone was advanced 2.7 feet into bedrock 2.7 feet before coring the bedrock. The boring log for BB-AHBS-102 indicates the roller cone advanced 0.9 feet into bedrock before coring the bedrock.

Boring	Station	Offset (feet)	Approximate Depth to Initial Bedrock Core (feet)	Approximate Initial Core Elevation (feet)	RQD (%)
BB-AHBS-102	7+94	6.6 Rt	25.0	237.6	90
BB-AHBS-201B	7+37	7.2 Rt	43.0	220.8	62

Table 2 – Summary of Approximate Bedrock Depths and Elevations

The recovered bedrock is identified as white, fine to medium grained, granodiorite, fresh, hard, joints are low angle to steep, close to moderately close, tight to open, occasional slight to moderately weathered zones. The RQD of the bedrock cores was determined to range from 13 to 90 percent correlating to a rock mass quality of very poor to good. A detailed bedrock description and the RQD of each core run is provided on the boring logs in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.4 Groundwater

Groundwater measurements performed at the completion of the borings indicate that groundwater levels were between 7.3 and 12.6 feet bgs. Groundwater observations are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. Note that because water was introduced into the borehole during drilling operations, the groundwater measurements may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The Preliminary Design Report[1] (PDR) evaluated a traditional integral abutment bridge, buried arches placed on strip footings founded on piles, and a precast concrete box culvert as potential replacement bridge structures. Estimated construction costs, durability, and maintenance costs were examined to determine the overall structure cost of each alternative.

[1] Preliminary Design Report, Corson Corner Bridge #2135 over Black Stream, Athens-Harland, Maine, Dated Oct. 27, 2017,

This examination resulted in the precast concrete box culvert alternative being the preferred alternative because this alternative has the overall lowest cost.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design

The proposed replacement structure will consist of a 24-foot span precast concrete box culvert with a 12-foot rise and slope-tapered inlet and outlet walls. The box culvert will have 1 foot tall precast headwalls. To prevent undermining, the box culvert will have inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 2 feet into the streambed and 2 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. The riprap apron should be embedded 6 inches into the streambed and covered with the engineered streambed material to provide continuity of the natural streambed.

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be in accordance with MaineDOT Standard Specification 534 – Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures, and American Association of State Highway and Transportation Officials (AASHTO) Load Resistance and Factor Design Bridge Design Specifications, 7th Edition, 2014 with interim revisions through 2016 (LRFD).

The loading specified for the design of the box culvert shall be Modified HL-93 Strength I, which increases the HS-20 design truck wheel loads by a factor of 1.25. The precast concrete box culvert shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The design should use Soil Type 4 as presented in the MaineDOT BDG Section 3.6 to calculate earth loads and lateral earth pressures from the soil envelope. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf.

7.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will be included in the culvert design to retain crushed stone slope protection and prevent stones from dropping or eroding into the waterway. Nominal 1 foot by 1 foot concrete headwalls are recommended.

7.1.2 Precast Concrete Inlet and Outlet Walls

The precast concrete box culvert's outlet and inlet walls will be slope-tapered to match the 2H:1V sideslopes of the roadway embankment. The left and right outlet walls will share the same base slab. The sloped walls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles box culvert. Passive pressure resulting from the embedment of the box culvert and walls with engineered streambed, or any other, material shall not contribute to resisting forces.

Inlet and outlet walls that are fixed to the box culvert should be designed to resist movement using an at-rest earth pressure coefficient, K_o, of 0.47. Wingwall sections that are independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a, of 0.31 assuming a level backslope. Wingwall sections that are independent of the box culvert and have a backslope of 2H:1V should be designed using the Rankine active earth pressure coefficient of 0.46. See Appendix C – Calculations for supporting documentation.

7.1.3 Precast Concrete Toe Walls

Toe walls shall extend below the bottom slab connecting the left and right walls at the inlet and outlet of the box culvert to prevent undermining per MaineDOT BDG Section 8.3.1. The inlet and outlet toe walls should extend a minimum of 1 foot below the maximum depth of scour.

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill with a bearing elevation of approximately 247 feet. The subgrade soils at this elevation are expected to be medium dense in consistency. These soils are characterized as having adequate bearing resistance.

For a precast concrete box culvert with a base width of 26 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 8.2 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 5 ksf. Due to the large size of the concrete box culvert base, controlling deflection and not bearing resistance may govern the design. In no instance shall bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'c$. See Appendix C – Calculations for supporting calculations.

7.1.5 Modulus of Subgrade Reaction

Large span precast box culverts can be viewed similarly to a mat foundation where the volume of soil displaced by the foundation will result in a lower net applied stress. A common approach to the design of precast box culverts is to use beam on elastic foundation theory to compute the soil-structure interaction and deflections.

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction, k_s, equal to 72 pounds per cubic inch (pci). See Appendix C – Calculations for supporting calculations.

7.2 Settlement

The glacial till deposit encountered at the precast concrete box culvert bearing elevation is medium dense. These coarse-grained materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure.

The project calls for the vertical alignment of the new structure to increase approximately 22-inches. Loose sand and soft to stiff non-plastic silt was encountered above the precast concrete box culvert bearing elevation in the fill and reworked soils. These materials comprise, or underlie, the existing embankment. Elastic embankment settlement was computed using Rocscience Settle3D software assuming a 2-foot increase in embankment height and 2H:1V sideslopes. The resulting settlement is estimated to be on the order of 0.5 inch. See Appendix C – Calculations for supporting calculations.

Settlement of the box culvert and roadway approaches in response to the proposed raise in grade will occur relatively quickly. Construction loads on the embankment could also introduce elastic settlements and these settlements are anticipated to be small and will also occur relatively quickly. Post construction settlement of the embankment should be negligible with proper embankment construction. Post construction settlement of the precast concrete box culvert should be negligible with proper subgrade preparation.

7.3 Subgrade Preparation

The box culvert shall be placed on a 1-foot-thick layer of compacted Granular Borrow – Material for Underwater Backfill. The compacted Granular Borrow layer shall be placed on a subgrade consisting of compacted, undisturbed native soil. The soils encountered during the subsurface investigation at the elevation of the bedding layer generally consisted of medium dense glacial till, cobbles, and boulders. Any loose or soft soils encountered at the subgrade should be excavated in its entirety and replaced with compacted Granular Borrow – Material for Underwater Backfill.

7.4 Frost Protection

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Athens has a design freezing index (DFI) of approximately 1850 F-degree days. The anticipated coarse grained fill soils were assigned a water content of 20%. These components correlate to a frost depth of 6.2 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Madison, Maine has a DFI from the Modberg database of approximately 1847 F-degree days. Madison was selected because it lies near the same

20% was assumed. These components correlate to a frost depth of approximately 6.9 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 6.2 feet for frost protection. See Appendix C – Calculations for supporting calculations.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.5 Scour and Riprap

The box culvert shall be constructed with integral concrete headwalls and wingwalls to retain stone slopes and prevent stone slope protection from dropping or eroding into the waterway. Inlet and outlet toe walls shall be provided that extend a minimum of 1 foot below the maximum depth of scour. Inlet and outlet toe walls shall also be protected with riprap aprons.

Due to relatively high outlet velocities, the PDR recommends that slopes be armored with a 4-foot-thick layer of riprap conforming to MaineDOT Standard Specification 703.28 – Heavy Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill. The top of the riprap toe sections shall be constructed 1-foot below the streambed elevation. The riprap slopes shall be constructed no steeper than a maximum 2H:1V extending from the edge of the roadway down to the existing ground surface.

7.6 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore, seismic analysis is not required.

7.7 Construction Considerations

The box culvert will be bedded on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill conforming to Standard Specification 703.19. The soil envelope and backfill shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density. The precast concrete box culvert shall be installed in conformance with MaineDOT BDG Section 8 and MaineDOT Standard Specification Section 534.

The proposed box culvert will be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill, with a bottom of excavation elevation of approximately 565 feet. Based on the soils encountered in the borings, medium dense to very dense, coarse-grained soils, cobbles, and boulders will be present at this elevation.

The Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary construction traffic. Bidders and contractors should anticipate encountering boulders and cobbles in the excavations for the proposed box culvert. Any cobbles or boulders encountered at the bearing elevation shall be removed to a depth greater than that required to place the 1-foot-thick bedding layer of compacted Granular Borrow – Material for Underwater Backfill.

One 5-foot section of hollow-stem auger was abandoned in boring BB-AHBS-201A. The contractor should anticipate encountering this obstruction during construction.

Earthwork and excavations may result in the exposure of silt or other loose or soft soils. These soils may be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance or rutting occur, the Contractor shall remove and replace the materials with compacted granular borrow or crushed stone.

Soils may become saturated and water seepage may be encountered during construction and in excavations. There may be localized sloughing and instability in some excavations and cut slopes. The Contractor should control groundwater and surface water infiltration using temporary ditches, sump pumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application of the proposed replacement of Corson Corner Bridge in Athens-Hartland, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon limited subsurface investigations at discrete exploratory locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

It is recommended that the geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

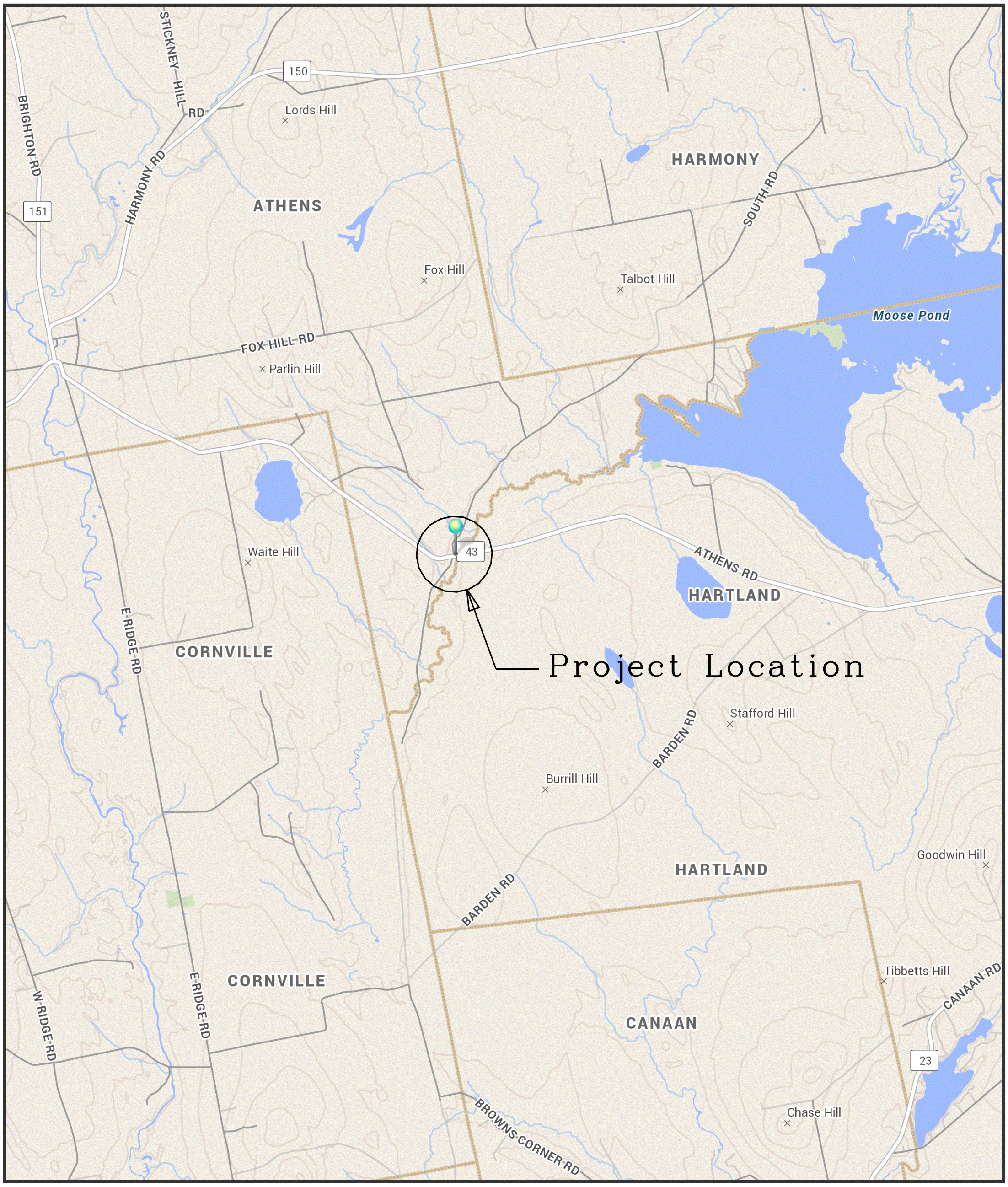
Sheets

Date:6/12/2017

Division: GEOTECH

Username: Terry.White

ATHENS-HARTLAND, MAINE



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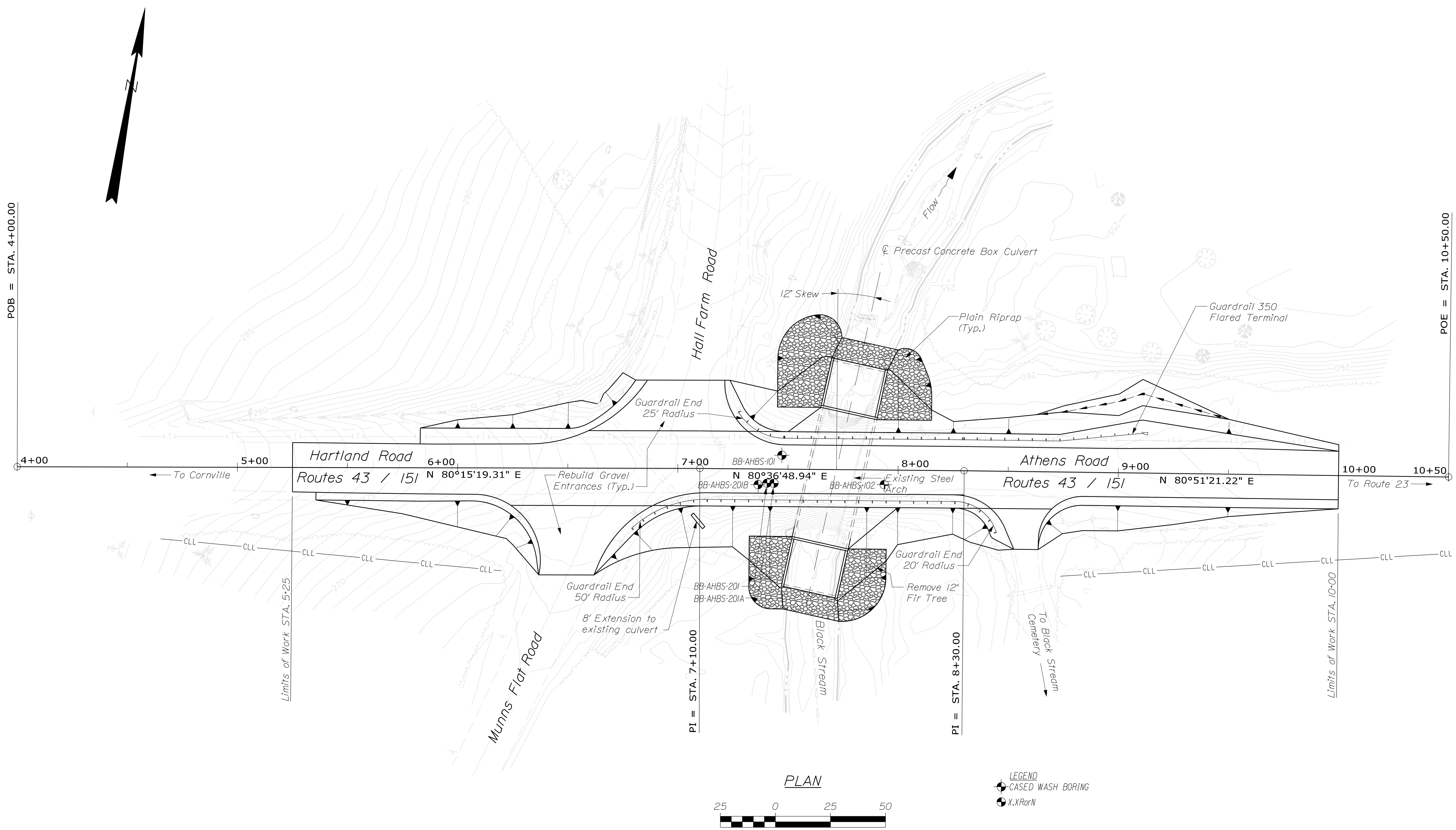
1 Miles

1 inch = 1.51 miles

Date: 6/12/2017
Time: 9:33:51 AM

SHEET NUMBER	CORSON CORNER BRIDGE BLACK STREAM	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	ATHENS-HARTLAND SOMERSET COUNTY	STP-2081(500)
OF 3	LOCATION MAP	WIN BRIDGE NO. 2135 020815.00 BRIDGE PLANS



PLAN

25 0 25 50
Scale in Feet

LEGEND
CASED WASH BORING
X.XRorN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2081(500)

WIN
020815.00

BRIDGE NO. 2135

BRIDGE PLANS

PROJ. MANAGER		BY	DATE
DESIGN-DETAILED	MARK GRAY	W ROHMAN	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	JUL 2017
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

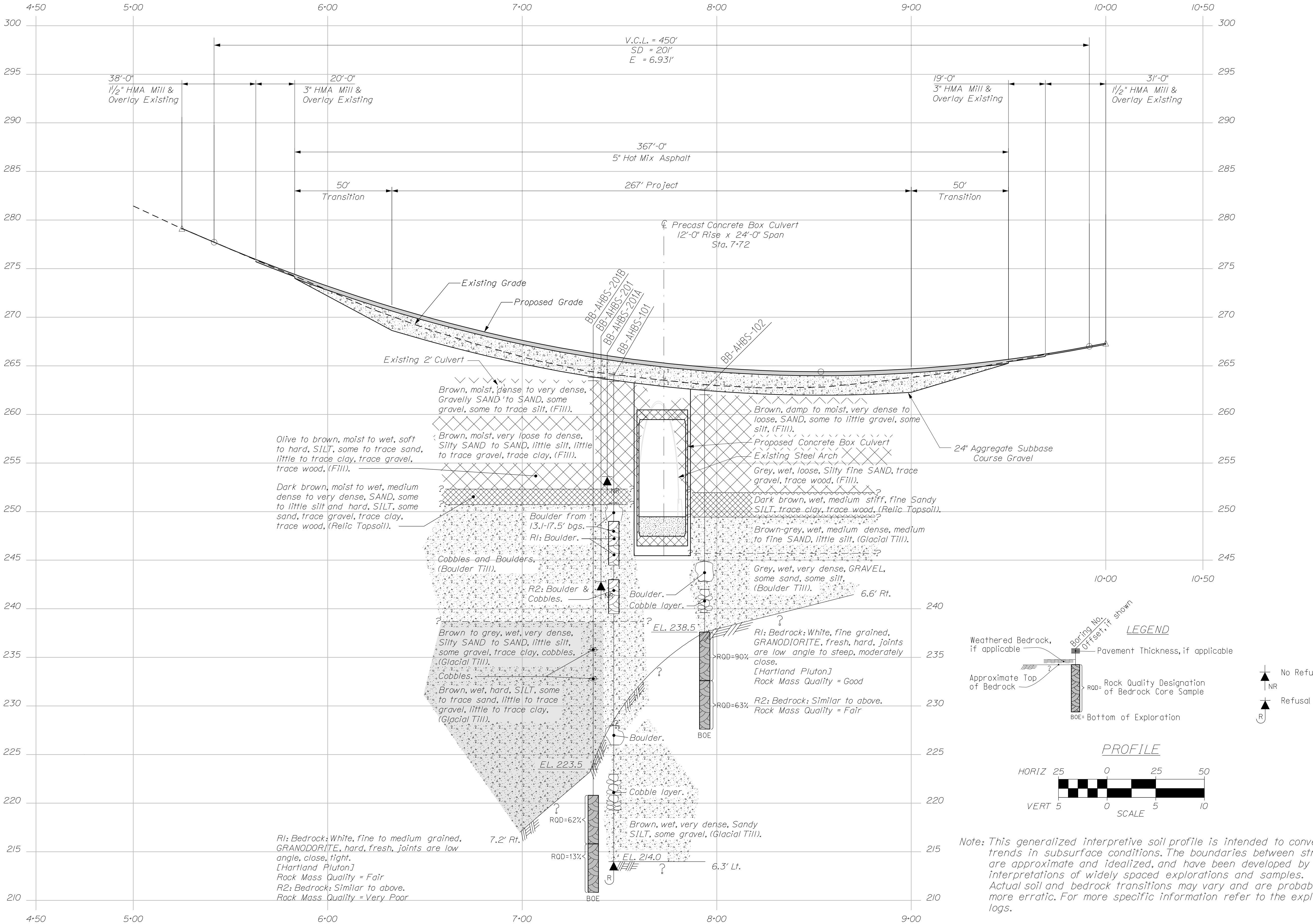
DATE

CORSON CORNER BRIDGE
BLACK STREAM
ATHENS-HARTLAND SOMERSET COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: Brandon.Slaven Date:11/3/2017



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2081(500)

WIN 020815.00

BRIDGE PLANS

BRIDGE NO. 2135

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

SIGNATURE

P.E. NUMBER

DATE

CORSON CORNER BRIDGE
BLACK STREAM
ATHENS-HARTLAND SOMERSET COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\OD\GEOTECH\MSTA\007_ISP1.dgn Division: GEOTECH Username: Brandon.Slaven Date:11/3/2017

Filename: ...\MSTA\008_BORING LOGS1.dgn   Division: GEOTECH   Username: Brandon.Slaven   Date:11/3/2017

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black
Location: Athens-Hartland, Maine
Boring No.: BB-AHBS-201B
WIN: 20815.00

Driller: S. W. Cole Explorations, LLC	Elevation (ft.) 263.8	Auger ID/OD: 5" Solid-Stem	
Operator: J. Lee	Datum: NAVD88	Sampler: Standard Split-Spoon	
Logged By: N. Strout	Rig Type: CME 850	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 05/03/2017 - 05/04/2017	Drilling Method: Cased Wash	Core Barrel: NQ-2"	
Boring Location: 7+36.5, 7.2 ft Rt.	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 9.3' (after completion)	
Hammer Efficiency Factor: 0.873	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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA		No soil samples taken. Soil strata similar to BB-AHBS-201 from 0-21 ft bgs. Advance by SSA to 25 ft bgs then placed HW casing to 25 feet.	
15								248.80	—15.00—	
25	1D	5/5	25.00 - 25.42	100-5"	- -		SPUN		Brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till). Cobble.	
30	2D	8/6	30.00 - 30.67	32/100-2"					Brown, wet, hard, SILT, some sand, little gravel, trace clay, (Glacial Till). Cobble.	G#271154 A-4, CL WC=16.2%
40	3D	3/1	40.00 - 40.25	100-3"	- -		OPEN	223.50	Similar to above. —40.30— Top of Bedrock at Elev. 223.5 ft. Advanced by rollercone through Bedrock from 40.3 to 43 ft bgs.	
43	R1	60/58	43.00 - 48.00	RQD = 62%			NQ2		R1:Bedrock: White, fine to medium grained, GRANODIORITE, fresh, hard, joints are low angle, close, tight. [Hartland Pluton] Rock Mass Quality = Fair. R1:Core Times (min:sec) 43.0-44.0 ft (3:55) 44.0-45.0 ft (4:00) 45.0-46.0 ft (4:49) 46.0-47.0 ft (4:55) 47.0-48.0 ft (2:52) 97% Recovery	
48	R2	60/49	48.00 - 53.00	RQD = 13%					R2:Bedrock: Similar to above except open fractures with slight to moderate weathering from 48.0 to 48.5 ft bgs. Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 48.0-49.0 ft (5:10) 49.0-50.0 ft (6:11) 50.0-51.0 ft (6:58) 51.0-52.0 ft (4:13) 52.0-53.0 ft (4:34) 82% Recovery	
53								210.80	—53.00— Bottom of Exploration at 53.00 feet below ground surface.	

Remarks:
Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.
Page 1 of 1
Boring No.: BB-AHBS-201B

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black
Location: Athens-Hartland, Maine
Boring No.: BB-AHBS-201
WIN: 20815.00

Driller: S. W. Cole Explorations, LLC	Elevation (ft.) 263.8	Auger ID/OD: 5" Solid-Stem
Operator: J. Lee	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: CME 850	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 05/03/2017	Drilling Method: Cased Wash	Core Barrel:
Boring Location: 7+40.5, 6.4 ft Rt.	Casing ID/OD: HW 4"/4.5"	Water Level*: 7.3' (after completion)
Hammer Efficiency Factor: 0.873	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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	263.47	4" of Pavement —0.33—	
1	1D	24/17	1.00 - 3.00	8/10/12/12	22	32			Brown, moist, dense, SAND, some gravel, trace silt, (Fill).	
3	2D	24/10	3.00 - 5.00	12/9/8/6	17	25			Similar to above, except medium dense.	
5	3D	24/12	5.00 - 7.00	3/2/1/2	3	4			Brown, moist, very loose, SAND, little gravel, little silt, (Fill).	G#271151 A-2-4, SM WC=12.0%
7	4D	24/10	7.00 - 9.00	3/2/1/2	3	4			Brown, wet, very loose, Silty SAND, little gravel, trace clay, (Fill).	G#271152 A-4, SC-SM WC=28.2% Non-Plastic
10	5D	24/16	10.00 - 12.00	1/1/1/1	2	3	33, 24, 31, 34, 47		Olive, wet, soft, SILT, little clay, trace fine sand, (Reworked Soils).	G#271153 A-4, CL WC=40.9% Non-Plastic
15	6D	24/2	15.00 - 17.00	7/6/7/11	13	19	52, 53, 51, OPEN	248.80	—15.00— Brown, wet, medium dense, SAND, little gravel, little silt (Boulder Till). Boulder from 17.5 to 19.8 ft bgs. Advanced by rollercone through boulder to 20 ft.	
20	MD	3/0	20.00 - 20.25	100-3"	- -			242.80	Rollercone bit damaged. Borehole abandoned. Relocate to BB-AHBS-201A. —21.00— Bottom of Exploration at 21.00 feet below ground surface.	

Remarks:
Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-AHBS-201

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black
Location: Athens-Hartland, Maine
Boring No.: BB-AHBS-201A
WIN: 20815.00

Driller: S. W. Cole Explorations, LLC	Elevation (ft.) 263.6	Auger ID/OD: Hollow-Stem 2.24"/4.5"
Operator: J. Lee	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: CME 850	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 05/03/2017	Drilling Method: Auger	Core Barrel:
Boring Location: 7+43.7, 6.4 ft Rt.	Casing ID/OD:	Water Level*:
Hammer Efficiency Factor: 0.873	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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							HSA		No soil samples taken. Soil on auger flights similar to BB-AHBS-201 from 0 to 10 ft bgs.	
10								253.60	HSA broke at 5 ft bgs and abandoned in borehole. Relocate to BB-AHBS-201B. —10.00— Bottom of Exploration at 10.00 feet below ground surface.	

Remarks:
Auto-hammer #362

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-AHBS-201A

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black
Location: Athens-Hartland, Maine
Boring No.: BB-AHBS-101
WIN: 20815.00

Driller: S. W. Cole Explorations, LLC	Elevation (ft.) 264.0	Auger ID/OD: 5" Solid Stem Auger
Operator: Scott Hollabaugh	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: Nathan Strout	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 9/23/2016	Drilling Method: Cased Wash	Core Barrel: NQ-2"
Boring Location: 7+47.2, 6.3 ft Lt.	Casing ID/OD: NW 3"/3.5"	Water Level*: 12.6' after completion
Hammer Efficiency Factor: 0.9288	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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	263.42	7" of Pavement —0.58—	
1	1D	24/17	1.00 - 3.00	16/19/20/16	39	60			Brown, moist, very dense, Gravelly SAND, some silt, (Fill).	
3	2D	24/15	3.00 - 5.00	6/10/10/7	20	31		259.50	2D(A) Similar to above, except dense. —4.50— 2D(B) Brown, moist, dense, Silty SAND, little gravel, (Fill).	
5	3D	24/16	5.00 - 7.00	5/5/3/2	8	12		258.00, 257.50	3D(A) Similar to above, except medium dense. 3D(B) Brown, moist, medium dense, fine to medium SAND, little silt, trace gravel, (Fill). —6.50— 3D(C) Brown, moist, stiff, SILT, some sand, trace clay, trace gravel, (Fill).	G#262840 A-4, CL WC=36.7% Non-Plastic
7	4D	24/14	7.00 - 9.00	2/3/3/4	6	9			4D Similar to above.	
10	5D	24/14	10.00 - 12.00	1/2/6/5	8	12	NW, PUSH	252.50	5D(A) Similar to above, except trace wood fiber. —11.50— 5D(B) Dark brown, moist, medium dense, SAND, some silt, trace gravel, trace clay, trace wood fiber, (Relic Topsoil).	G#262841 A-4, CL WC=32.6% Non-Plastic; G#262842 A-2-4, SC-SM WC=38.1%
12	6D	13/11	12.00 - 13.08	3/6/50-1"	--			250.90	6D(A) Dark brown, wet, SILT, some sand, trace gravel, trace clay, trace wood fiber. 6D(B) Grey, wet, fine to medium SAND, little silt. —13.10— Place HW casing to 13.1 ft bgs. Advance by roller cone to 15 ft bgs. GRANODIORITE Boulder from 13.1 to 17.5 ft bgs.	G#262843 A-4, CL WC=89.7%
15	R1	54/30	15.00 - 19.50				NQ2		(Boulder Till).	
20	MD	4/0	20.00 - 20.33	100-4"	--		NW		Boulder from 20.4 to 21.5 ft bgs.	
21	R2	42/6	21.00 - 24.50				NQ2		Cobbles from 23.2 to 24 ft bgs.	
25	7D	12/7	25.00 - 26.00	23/100	--		NW	239.50	—24.50— Place NW casing with spin cutting shoe. 7D(A) Grey, wet, very dense, Silty SAND, some gravel, trace clay, cobbles. 7D(B) Brown, wet, hard, SILT, little clay, trace fine sand, cobbles, (Glacial Till).	
30	8D	10/6	30.00 - 30.83	44/100-4"	--				Brown, wet, hard, SILT, little sand, trace clay, trace gravel.	G#262844 A-4, CL WC=22.5%
35	9D	5/2	35.00 - 35.42	100-5"	--			229.00	—35.00— Brown, wet, hard, Sandy SILT, some gravel, (Glacial Till). Boulder from 36 to 38 ft bgs.	
40	MD	5/0	40.00 - 40.42	100-5"	--				Cobbles from 41 to 44.7 ft bgs.	
45	10D	4/4	45.00 - 45.33	100-4"	--				Brown, wet, hard, Sandy SILT, some gravel, (Glacial Till).	
50	MD	0/0	50.00 - 50.00	50-0"	--			214.00	Spoon refusal. —50.00— Bottom of Exploration at 50.00 feet below ground surface.	

Remarks:
Auto-hammer #562

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Page 1 of 1
Boring No.: BB-AHBS-101

Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black
Location: Athens-Hartland, Maine
Boring No.: BB-AHBS-102
WIN: 20815.00

Driller: S. W. Cole Explorations, LLC	Elevation (ft.) 262.6	Auger ID/OD: 5" Solid Stem Auger
Operator: Scott Hollabaugh	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: Michael St. Pierre	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 9/22/2016	Drilling Method: Cased Wash	Core Barrel: NQ-2"
Boring Location: 7+93.8, 6.6 ft Rt.	Casing ID/OD: HW 4"/4.5"	Water Level*: 11.3' after completion
Hammer Efficiency Factor: 0.9288	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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/18	0.80 - 2.80	15/25/12/10	37	57	SSA	262.02, 261.30	7" of Pavement —0.58— 1D(A) Brown, damp, very dense, SAND, some gravel, some silt, (Fill). —1.30— 1D(B) Brown, damp to moist, very dense, SAND, some silt, little gravel, (Fill).	
2.8	2D	24/17	2.80 - 4.80	6/6/5/4	11	17			Similar to above.	
5	3D	24/12	5.00 - 7.00	3/2/1/2	3	5			Brown, damp to moist, loose, SAND, some gravel, some silt, (Fill).	
10	4D	24/10	10.00 - 12.00	3/2/1/2	3	5	14, 20, 34, 55, 42	252.10, 249.60	4D(A) Grey, wet, loose, Silty fine SAND, trace gravel, trace wood fiber. —10.50— 4D(B) Dark brown, wet, medium stiff, fine Sandy SILT, trace clay, trace wood fiber, (Relic Topsoil). —13.00—	
15	5D	24/14	15.00 - 17.00	5/3/4/3	7	11	21, 44, 193 OPEN	245.60	Brown-grey, wet, medium dense, medium to fine SAND, little silt, (Glacial Till). —17.00— Boulder from 17.7 to 19.8 ft bgs. Roller cone to 20 ft bgs and spin NW casing.	G#262845 A-2-4, SP-SM WC=21.4%
20	6D	7/3	20.00 - 20.58	19/50-1"	--		NW		Grey, wet, very dense, GRAVEL, some sand, some silt, (Boulder Till). Cobbles from 20.6 to 23 ft bgs. Roller cone to 25 ft bgs.	
25	R1	60/59	25.00 - 30.00	RQD = 90%			NQ2	238.50	—24.10— Top of bedrock at Elev. 238.5 ft. R1:Bedrock: White, fine-grained, GRANODIORITE, fresh, hard, joints are low angle to steep, moderately close. [HARTLAND PLUTON] Rock Mass Quality = Good. R1:Core Times (min:sec) 237.7-236.7' (2:38); 236.7-235.7' (2:34); 235.7-234.7' (3:15); 234.7-233.7' (3:19); 233.7-232.7' (3:03) 98% Recovery Core Blocked	
30	R2	60/ 29	30.00 - 35.00	RQD = 63%					R2:Bedrock: Similar to above. Rock Mass Quality = Fair. R2:Core Times (min:sec) 232.7-231.7 ft (2:06); 231.7-230.7 ft (1:55); 230.7-229.7 ft (2:23); 229.7-228.7 ft (2:22); 228.7-227.7 ft (2:36) 48% Recovery	
35								227.60	—35.00— Bottom of Exploration at 35.00 feet below ground surface.	

Remarks:
Auto-hammer #562

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-AHBS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2081(500)
WIN 020815.00
BRIDGE PLANS
BRIDGE NO. 2135

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

SIGNATURE
P.E. NUMBER
DATE

CORSON CORNER BRIDGE
BLACK STREAM
ATHENS-HARTLAND SOMERSET COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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))

Sample Container Labeling Requirements:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream **Location:** Athens-Hartland, Maine	**Boring No.:** BB-AHBS-101 **WIN:** 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	264.0	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/23/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+47.2, 6.3 ft Lt.	**Casing ID/OD:**	NW 3"/3.5"	**Water Level*:**	12.6' after completion
Hammer Efficiency Factor: 0.9288		**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	263.42		7" of Pavement 0.58	
	1D	24/17	1.00 - 3.00	16/19/20/16	39	60				Brown, moist, very dense, Gravelly SAND, some silt, (Fill).	
	2D	24/15	3.00 - 5.00	6/10/10/7	20	31				2D(A) Similar to above, except dense.	
								259.50		4.50	
5	3D	24/16	5.00 - 7.00	5/5/3/2	8	12				2D(B) Brown, moist, dense, Silty SAND, little gravel, (Fill). 3D(A) Similar to above, except medium dense.	
								258.00 257.50		6.00 3D(B) Brown, moist, medium dense, fine to medium SAND, little silt, trace gravel, (Fill).	
	4D	24/14	7.00 - 9.00	2/3/3/4	6	9				6.50 3D(C) Brown, moist, stiff, SILT, some sand, trace clay, trace gravel, (Fill). 4D Similar to above.	G#262840 A-4, CL WC=36.7% Non-PLastic
10	5D	24/14	10.00 - 12.00	1/2/6/5	8	12	NW			5D(A) Similar to above, except trace wood fiber.	G#262841 A-4, CL WC=32.6%
							PUSH	252.50		11.50 5D(B) Dark brown, moist, medium dense, SAND, some silt, trace gravel, trace clay, trace wood fiber, (Relic Topsoil).	Non-Plastic G#262842
	6D	13/11	12.00 - 13.08	3/6/50-1"	--					6D(A) Dark brown, wet, SILT, some sand, trace gravel, trace clay, trace wood fiber.	A-2-4, SC-SM WC=38.1%
							OPEN	250.90		6D(B) Grey, wet, fine to medium SAND, little silt. 13.10	G#262843 A-4, CL WC=89.7%
										Place HW casing to 13.1 ft bgs.	
15	R1	54/30	15.00 - 19.50				NQ2			Advance by roller cone to 15 ft bgs. GRANODIORITE Boulder from 13.1 to 17.5 ft bgs.	
										(Boulder Till).	
20	MD	4/0	20.00 - 20.33	100-4"	--		NW			Boulder from 20.4 to 21.5 ft bgs.	
	R2	42/6	21.00 - 24.50				NQ2				
										Cobbles from 23.2 to 24 ft bgs.	
25							NW	239.50		24.50	

Remarks:

Auto-hammer #562

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: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-101

WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	264.0	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/23/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+47.2, 6.3 ft Lt.	**Casing ID/OD:**	NW 3"/3.5"	**Water Level*:**	12.6' after completion
Hammer Efficiency Factor:	0.9288	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	12/7	25.00 - 26.00	23/100	--		NW			Place NW casing with spin cutting shoe. 7D(A) Grey, wet, very dense, Silty SAND, some gravel, trace clay, cobbles. 7D(B) Brown, wet, hard, SILT, little clay, trace fine sand, cobbles, (Glacial Till).	
30	8D	10/6	30.00 - 30.83	44/100-4"	--					Brown, wet, hard, SILT, little sand, trace clay, trace gravel.	G#262844 A-4, CL WC=22.5%
35	9D	5/2	35.00 - 35.42	100-5"	--			229.00		35.00 Brown, wet, hard, Sandy SILT, some gravel, (Glacial Till). Boulder from 36 to 38 ft bgs.	
40	MD	5/0	40.00 - 40.42	100-5"	--					Cobbles from 41 to 44.7 ft bgs.	
45	10D	4/4	45.00 - 45.33	100-4"	--					Brown, wet, hard, Sandy SILT, some gravel, (Glacial Till).	
50											

Remarks:

Auto-hammer #562

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-101

WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	264.0	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/23/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+47.2, 6.3 ft Lt.	**Casing ID/OD:**	NW 3"/3.5"	**Water Level*:**	12.6' after completion
Hammer Efficiency Factor: 0.9288		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid 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	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	MD	0/0	50.00 - 50.00	50-0"	--			214.00		Spoon refusal. 50.00	
										Bottom of Exploration at 50.00 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

Auto-hammer #562

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-102
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	262.6	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/22/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+93.8, 6.6 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	11.3' after completion
Hammer Efficiency Factor:	0.9288	**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
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
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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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/18	0.80 - 2.80	15/25/12/10	37	57	SSA	262.02		7" of Pavement —0.58	
								261.30		1D(A) Brown, damp, very dense, SAND, some gravel, some silt, (Fill). —1.30	
	2D	24/17	2.80 - 4.80	6/6/5/4	11	17				1D(B) Brown, damp to moist, very dense, SAND, some silt, little gravel, (Fill). Similar to above.	
5	3D	24/12	5.00 - 7.00	3/2/1/2	3	5				Brown, damp to moist, loose, SAND, some gravel, some silt, (Fill).	
10	4D	24/10	10.00 - 12.00	3/2/1/2	3	5	14	252.10		4D(A) Grey, wet, loose, Silty fine SAND, trace gravel, trace wood fiber. —10.50	
							20			4D(B) Dark brown, wet, medium stiff, fine Sandy SILT, trace clay, trace wood fiber, (Relic Topsoil).	
							34	249.60		—13.00	
							55				
							42				
15	5D	24/14	15.00 - 17.00	5/3/4/3	7	11	21			Brown-grey, wet, medium dense, medium to fine SAND, little silt, (Glacial Till).	G#262845 A-2-4, SP-SM WC=21.4%
							44	245.60		—17.00	
							193 OPEN			Boulder from 17.7 to 19.8 ft bgs. Roller cone to 20 ft bgs and spin NW casing.	
20	6D	7/3	20.00 - 20.58	19/50-1"	--		NW			Grey, wet, very dense, GRAVEL, some sand, some silt, (Boulder Till).	
										Cobbles from 20.6 to 23 ft bgs.	
										Roller cone to 25 ft bgs.	
								238.50		—24.10	
25										Top of bedrock at Elev. 238.5 ft.	

Remarks:

Auto-hammer #562

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-102
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	262.6	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/22/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+93.8, 6.6 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	11.3' after completion
Hammer Efficiency Factor:	0.9288	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
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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	R1	60/59	25.00 - 30.00	RQD = 90%			NQ2			R1:Bedrock: White, fine-grained, GRANODIORITE, fresh, hard, joints are low angle to steep, moderately close. [HARTLAND PLUTON] Rock Mass Quality = Good. R1:Core Times (min:sec) 237.7-236.7' (2:38); 236.7-235.7' (2:34); 235.7-234.7' (3:15); 234.7-233.7' (3:19); 233.7-232.7' (3:03) 98% Recovery Core Blocked	
30	R2	60/ 29	30.00 - 35.00	RQD = 63%						R2:Bedrock: Similar to above. Rock Mass Quality = Fair. R2:Core Times (min:sec) 232.7-231.7 ft (2:06); 231.7-230.7 ft (1:55); 230.7-229.7 ft (2:23); 229.7-228.7 ft (2:22); 228.7-227.7 ft (2:36) 48% Recovery	
35								227.60		35.00 **Bottom of Exploration at 35.00 feet below ground surface.**	
40											
45											
50											

Remarks:

Auto-hammer #562

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-201
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	263.8	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/03/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	7+40.5, 6.4 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	7.3' (after completion)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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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	263.5		4" of Pavement — 0.33	
	1D	24/17	1.00 - 3.00	8/10/12/12	22	32				Brown, moist, dense, SAND, some gravel, trace silt, (Fill).	
	2D	24/10	3.00 - 5.00	12/9/8/6	17	25				Similar to above, except medium dense.	
5	3D	24/12	5.00 - 7.00	3/2/1/2	3	4				Brown, moist, very loose, SAND, little gravel, little silt, (Fill).	G#271151 A-2-4, SM WC=12.0%
	4D	24/10	7.00 - 9.00	3/2/1/2	3	4				Brown, wet, very loose, Silty SAND, little gravel, trace clay, (Fill).	G#271152 A-4, SC-SM WC=28.2% Non-Plastic
10	5D	24/16	10.00 - 12.00	1/1/1/1	2	3	33			Olive, wet, soft, SILT, little clay, trace fine sand, (Reworked Soils).	G#271153 A-4, CL WC=40.9% Non-Plastic
							24				
							31				
							34				
							47				
15	6D	24/2	15.00 - 17.00	7/6/7/11	13	19	52	248.8		15.00 — Brown, wet, medium dense, SAND, little gravel, little silt (Boulder Till).	
							53				
							317 OPEN			Boulder from 17.5 to 19.8 ft bgs. Advanced by rollercone through boulder to 20 ft.	
20	MD	3/0	20.00 - 20.25	100-3"	- -			242.8		Rollercone bit damaged. Borehole abandoned. Relocate to BB-AHBS-201A. — 21.00	
										Bottom of Exploration at 21.00 feet below ground surface.	
25											

Remarks:

Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

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Boring No.: BB-AHBS-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-201A
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	263.6	**Auger ID/OD:**	Hollow-Stem 2.24"/4.5"
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/03/2017	**Drilling Method:**	Auger	**Core Barrel:**	
Boring Location:	7+43.7, 6.4 ft Rt.	**Casing ID/OD:**		**Water Level*:**	
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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							HSA			No soil samples taken. Soil on auger flights similar to BB-AHBS-201 from 0 to 10 ft bgs.	
5											
										HSA broke at 5 ft bgs and abandoned in borehole. Relocate to BB-AHBS-201B.	
10								253.6		10.00	
										Bottom of Exploration at 10.00 feet below ground surface.	
15											
20											
25											

Remarks:

Auto-hammer #362

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: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-201B
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	263.8	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/03/2017 - 05/04/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+36.5, 7.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.3' (after completion)
Hammer Efficiency Factor:	0.873	**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			No soil samples taken. Soil strata similar to BB-AHBS-201 from 0-21 ft bgs. Advance by SSA to 25 ft bgs then placed HW casing to 25 feet.		
5												
10												
15								248.8			15.00	
20												
25												

Remarks:

Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

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

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

Boring No.: BB-AHBS-201B

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream **Location:** Athens-Hartland, Maine	**Boring No.:** BB-AHBS-201B **WIN:** 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	263.8	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/03/2017 - 05/04/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+36.5, 7.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.3' (after completion)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOR/C = Weight of Rods or Casing
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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
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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	1D	5/5	25.00 - 25.42	100-5"	- -		SPUN			Brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
										Cobble.	
30	2D	8/6	30.00 - 30.67	32/100-2"						Brown, wet, hard, SILT, some sand, little gravel, trace clay, (Glacial Till). Cobble.	G#271154 A-4, CL WC=16.2%
35											
40	3D	3/1	40.00 - 40.25	100-3"	- -		OPEN	223.5		Similar to above. 40.30 Top of Bedrock at Elev. 223.5 ft. Advanced by rollercone through Bedrock from 40.3 to 43 ft bgs.	
	R1	60/58	43.00 - 48.00	RQD = 62%			NQ2			R1:Bedrock: White, fine to medium grained, GRANODIORITE, fresh, hard, joints are low angle, close, tight. [Hartland Pluton] Rock Mass Quality = Fair. R1:Core Times (min:sec) 43.0-44.0 ft (3:55) 44.0-45.0 ft (4:00) 45.0-46.0 ft (4:49) 46.0-47.0 ft (4:55) 47.0-48.0 ft (2:52) 97% Recovery	
45											
	R2	60/49	48.00 - 53.00	RQD = 13%						R2:Bedrock: Similar to above except open fractures with slight to moderate weathering from 48.0 to 48.5 ft bgs. Rock Mass Quality = Very Poor. R2:Core Times (min:sec)	
50											

Remarks:

Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

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: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream Location: Athens-Hartland, Maine	Boring No.: BB-AHBS-201B WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	263.8	Auger ID/OD:	5" Solid-Stem
Operator:	J. Lee	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	CME 850	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	05/03/2017 - 05/04/2017	Drilling Method:	Cased Wash	Core Barrel:	NQ-2"
Boring Location:	7+36.5, 7.2 ft Rt.	Casing ID/OD:	HW 4"/4.5" NW 3"/3.5"	Water Level*:	9.3' (after completion)
Hammer Efficiency Factor:	0.873	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										48.0-49.0 ft (5:10) 49.0-50.0 ft (6:11) 50.0-51.0 ft (6:58) 51.0-52.0 ft (4:13) 52.0-53.0 ft (4:34) 82% Recovery	
								210.8		53.00 **Bottom of Exploration at 53.00 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

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

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Athens-Hartland Work Number: 20815.00

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
BB-AHBS-201B, 2D	7+36.5	7.2 Rt.	30.0-30.67	271154	2	16.2			CL	A-4	IV
BB-AHBS-201, 3D	7+40.5	6.4 Rt.	5.0-7.0	271151	2	12.0			SM	A-2-4	II
BB-AHBS-201, 4D	7+40.5	6.4 Rt.	7.0-9.0	271152	2	28.2	-N	P-	SC-SM	A-4	III
BB-AHBS-201, 5D	7+40.5	6.4 Rt.	10.0-12.0	271153	2	40.9	-N	P-	CL	A-4	IV
BB-AHBS-101, 4D	7+47.2	6.3 Lt.	7.0-9.0	262840	1	36.7	-N	P-	CL	A-4	IV
BB-AHBS-101, 5D/A	7+47.2	6.3 Lt.	10.0-11.5	262841	1	32.6	-N	P-	CL	A-4	IV
BB-AHBS-101, 5D/B	7+47.2	6.3 Lt.	11.5-12.0	262842	1	38.1			SC-SM	A-2-4	II
BB-AHBS-101, 6D/A	7+47.2	6.3 Lt.	12.0-13.0	262843	1	89.7			CL	A-4	IV
BB-AHBS-101, 8D	7+47.2	6.3 Lt.	30.0-30.8	262844	1	22.5			CL	A-4	IV
BB-AHBS-102, 5D	7+93.8	6.6 Rt.	15.0-17.0	262845	1	21.4			SP-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.

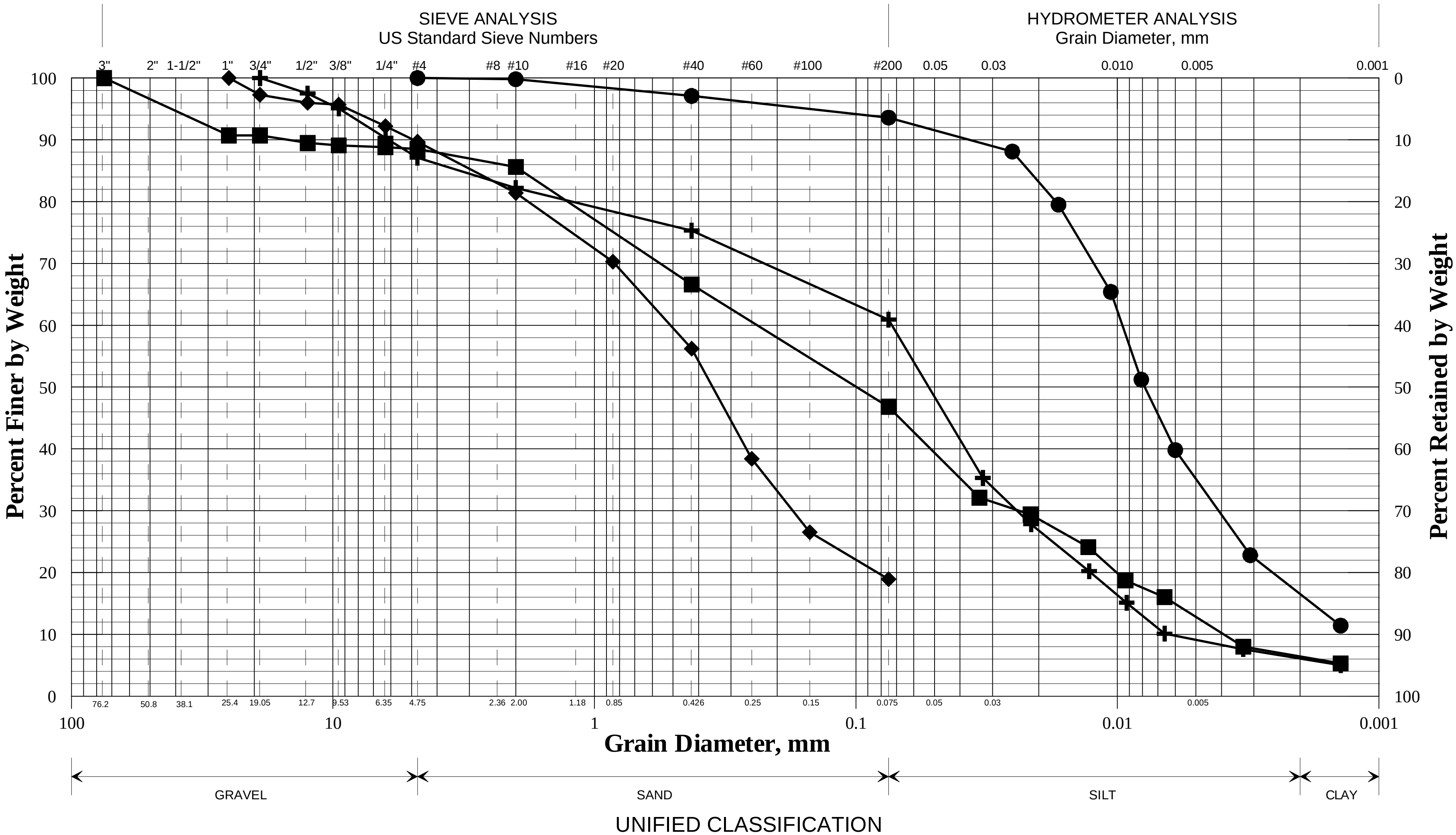
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



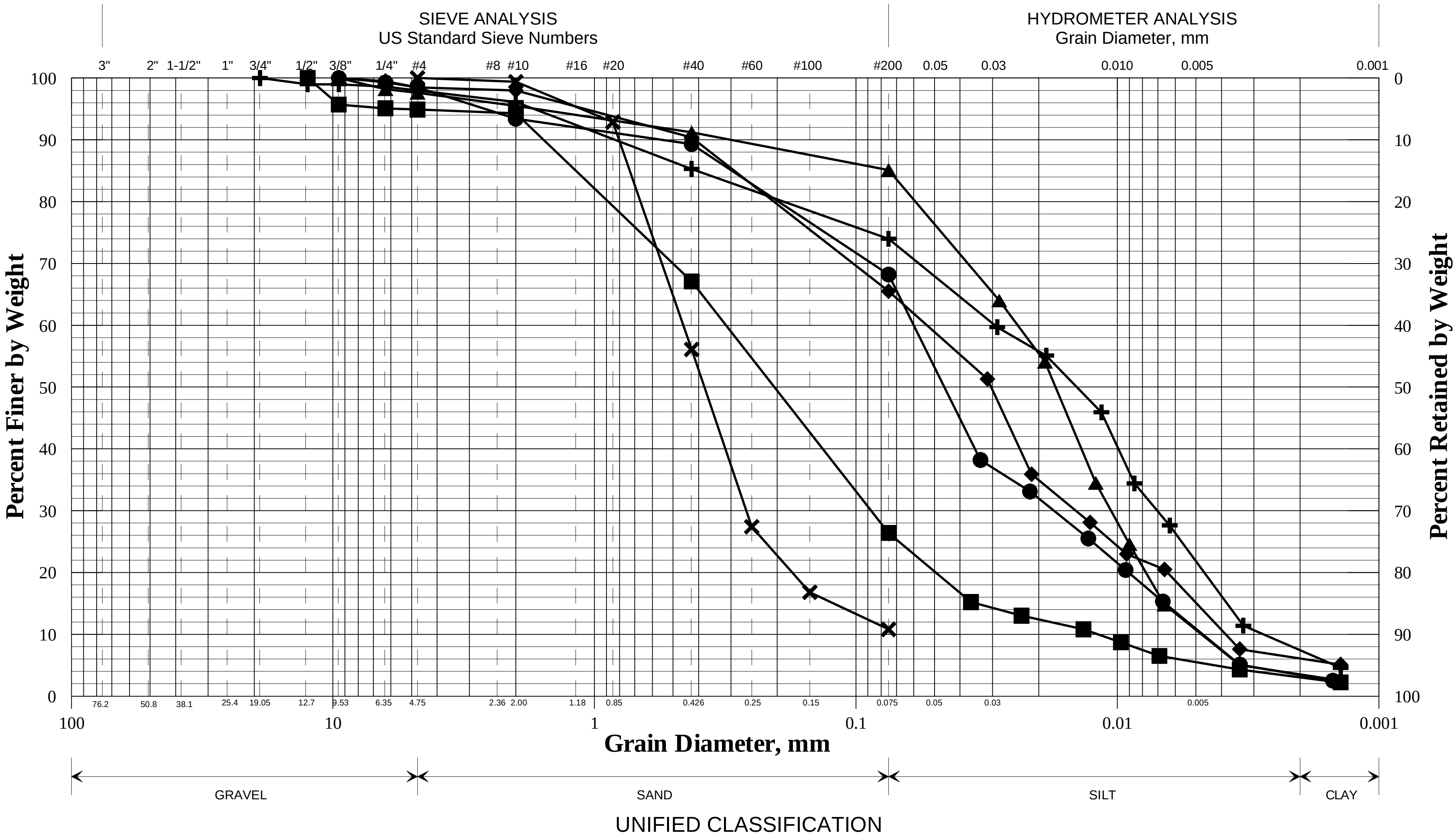
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-AHBS-201B/2D	7+36.5	7.2 RT	30.0-30.67	SILT, some sand, little gravel, trace clay.	16.2			
◆	BB-AHBS-201/3D	7+40.5	6.4 RT	5.0-7.0	SAND, little silt, little gravel.	12.0			
■	BB-AHBS-201/4D	7+40.5	6.4 RT	7.0-9.0	Silty SAND, little gravel, trace clay.	28.2			NP
●	BB-AHBS-201/5D	7+40.5	6.4 RT	10.0-12.0	SILT, little clay, trace sand.	40.9			NP
▲									
✖									

WIN
020815.00
Town
Athens, Hartland
Reported by/Date
WHITE, TERRY A 6/13/2017

SHEET 2

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-AHBS-101/4D	7+47.2	6.3 LT	7.0-9.0	SILT, some sand, trace clay, trace gravel.	36.7			NP
◆	BB-AHBS-101/5DA	7+47.2	6.3 LT	10.0-11.5	SILT, some sand, trace clay, trace gravel.	32.6			NP
■	BB-AHBS-101/5DB	7+47.2	6.3 LT	11.5-12.0	SAND, some silt, trace gravel, trace clay.	38.1			
●	BB-AHBS-101/6DA	7+47.2	6.3 LT	12.0-13.0	SILT, some sand, trace clay, trace gravel.	89.7			
▲	BB-AHBS-101/8D	7+47.2	6.3 LT	30.0-30.8	SILT, little sand, trace clay, trace gravel.	22.5			
✖	BB-AHBS-102/5D	7+93.8	6.6 RT	15.0-17.0	SAND, little silt.	21.4			

WIN
020815.00
Town
Athens, Hartland
Reported by/Date
WHITE, TERRY A 10/24/2016

SHEET 1

Appendix C

Calculations

Earth Pressure

Soil Parameters:
Assume existing material removed and replaced with material with properties similar to Soil Type 4, MaineDOT BDG Section 3.6.1.

Unit weight	$\gamma := 125 \cdot pcf$
Internal friction angle	$\phi := 32 \cdot deg$
Cohesion	$c := 0 \cdot psf$

1. Outlet walls fixed to box - At-Rest Earth Pressure - Rankine Theory

$$K_o := 1 - \sin(\phi)$$

Fang, Foundation Engineering Handbook 2nd ed. Pg. 224, Eq. 6.2

$$K_o = 0.47$$

Recommend: At-Rest Earth Pressure Coefficient, K_o= 0.47

2. Outlet walls free to rotate - Active Earth Pressure - Rankine Theory

The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

For cantilver walls with horizontal backslope:

$$K_{ar} := \tan\left(45 \cdot deg - \frac{\phi}{2}\right)^2$$

MaineDOT BDG Pg. 3-7

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

β = Angle of fill slope to the horizontal $\qquad \beta := 26.56 \cdot deg$

$$K_{ar\_slope} := \cos(\beta) \cdot \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

MaineDOT BDG Pg. 3-7

$$K_{ar\_slope} = 0.46$$

Pa is oriented at an angle of β to the vertical plane - See MaineDOT Bridge Design Guide Figure 3-3 attached.

Figure 3-2 Calculating β with Broken Backfill Surface

Rankine theory, as described in Section 3.6.5.2, may also be used for the design of yielding walls, for a simplified analysis (at the Structural Designer's option). The use of Rankine theory will result in a slightly more conservative design.

3.6.5.2 Rankine Theory

Rankine theory should be used for long-heeled cantilever walls. Refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of a long heeled cantilever wall. For simplicity (at the Structural Designer's option), Rankine theory may also be used to compute lateral earth pressures on any yielding wall listed in 3.6.5.1 Coulomb Theory, although its use will result in a slightly more conservative design.

For these cases, interface friction between the wall backface and the backfill is not considered. Rankine earth pressure is applied to a plane extending vertically from the heel of the wall base, as shown in Figure 3-3.

For a horizontal backfill surface where $\beta = 0°$, the value of the coefficient of active earth pressure (Rankine), K_a, may be taken as:

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$

where:

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

β= angle of backfill to the horizontal (degrees), as shown in Figure 3-3.

For a sloped backfill surface where $\beta > 0°$, the coefficient of active earth pressure (Rankine), K_a, may be taken as:

$$K_a = \cos\ \beta \cdot \frac{\cos\ \beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\ \beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

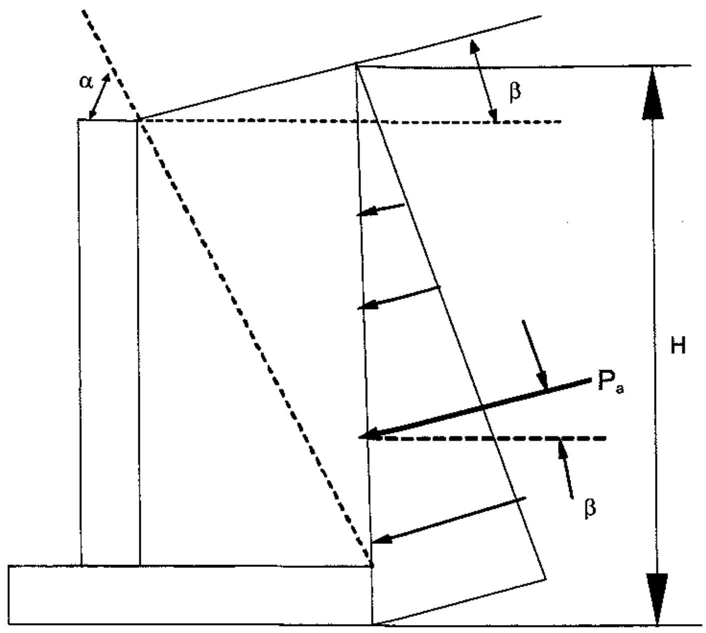


Figure 3-3 Rankine Theory

The resultant earth pressure force, P_a, is oriented at an angle, β, as shown if Figure 3-3. The resultant acts at a distance, H/3, from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient, Ka, may be determined using a β value adjusted per AASHTO LRFD Figures 3.11.5.8 -1 through 3, or substituted with β*, as shown in Figure 3-2.

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

α = angle (degrees) of back of wall to the horizontal as shown in Figure 3-1.

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

Bearing Resistance

Objective:
Estimate the factored bearing resistance for a box culvert bearing on soil at the Service Limit State and Strength Limit State.

Given:
1. Limited lab data
2. Soil engineering properties based on correlations to SPT N-values

Assumptions:
1. The box culvert's embedment into the streambed is conservatively assumed as 1 foot, which accounts for the possible scouring away of 1 foot of special fill.
2. The one foot thick layer of proposed Granular Borrow bedding material is ignored.
3. The proposed bearing elevation is approximately 246.0 feet.
4. Proposed finish roadway grade elevation is approximately 265.2 feet.
5. Proposed precast concrete box has a span of 24 feet and a base of 26 feet wide.
6. The subsurface conditions present at the proposed bearing elevation in the borings are representative of the conditions for the entire site. Use N-value of 11 bpf to represent the consistency of the soils encountered at the box bearing elevation for bearing resistance calculations, based on BB-AHBS-102;5D.
7. The bottom of the box culvert will be submerged for the structure's design life.

1. Estimate the factored bearing resistance at the Service Limit State:

The use of presumptive values may be used when sufficient knowledge of geological conditions at or near the structure site exists. AASHTO LRFD Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2*, *May 1983*, *Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Medium dense	4-8	5

The glacial till deposit is medium dense in consistency at the bearing elevation. Recommend 5 ksf to limit settlement to 1.0 inch for Service Limit State Loads

2. Estimate the factored bearing resistance at the Strength Limit State:

Assumed Foundation Width, Depth, and Water Surface

$$B := 26ft$$

$$D_f := 1.0 \cdot ft$$

$$D_w := 0 \cdot ft$$

$$\gamma_w := 62.4 \cdot pcf$$

Foundation soils:

$\gamma 1_d := 112 \cdot pcf$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 Average of Loose silty sand (102 pcf) and Dense silty sand (121 pcf)

$w_{sat} := .21$ — Moisture content of saturated sample BB-AHBS-102;5D

$\gamma 1_{sat} := \gamma 1_d \cdot (1 + w_{sat})$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit weight relationships

$\gamma 1_{sat} := 139 \cdot pcf$

$N_{design} := 11$

$\phi := 30 \cdot deg$ — Lambe and Whitman, Soil Mechanics, 1969 Figure 11.14 N vs. phi

Cohesion $c := 0$

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for strip footing

$s_\gamma := 1.0$ $s_c := 1.0$ Bowles 5th Ed., p. 220 Table 4-1

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for Gravelly SAND ϕ = 30 degrees.

$N_c := 30.13$ $N_q := 18.4$ $N_\gamma := 15.7$

Nominal Bearing Resistance per Terzaghi equation

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $q = 0.077 \cdot ksf$ — Das Principles of Foundation Engineering 7th Ed. p. 142: Eq. 3.16 Water table modification

$q_n := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$ — Bowles Foundation Analysis and Design 5th Ed. p. 220: Table 4-1 Bearing-capacity Equations

$q_n = 17 \cdot ksf$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.45$

$q_r := q_n \cdot \varphi_b$

$\boxed{q_r = 7.7 \cdot ksf}$ for $B = 26 \cdot ft$

Nominal Bearing Resistance for Strength Limit States

Reference: Munfakh, et al (2001) LRFD Article 10.6.3.1.2a

Total unit weight of the soil above the base slab/soil envelope

$\gamma_{above} := 125 \cdot pcf$

MainDOT Bridge Design Guide p. 3-3
Soil Type 4

Bearing Capacity Factors (Ref: LRFD Table 10.6.3.1.2a-1)

$N_c := 30.1$

$N_q := 18.4$

$N_\gamma := 22.4$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

assume:

$L := 62ft$

$s_\gamma := 1 - 0.4 \cdot \left(\frac{B}{L}\right)$

$s_q := 1 + \frac{B}{L} \cdot \tan(\phi)$

$s_\gamma = 0.832$ $s_q = 1.242$

Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

Assume groundwater, or stream elevation, will be above the invert of the structure for the entire design life.

Where the depth of water is less than the depth of the footing, all water coefficients are 0.5.

$C_{wq} := .5$ $C_{w\gamma} := 0.5$ $c_1 := 0$

Load Inclination factors

No knowledge of vertical and horizontal loads at this time. Use 1.0

$i_c := 1.0$ $i_\gamma := 1.0$ $i_q := 1.0$

Depth correction factors - only used when soils above the footing bearing elevation are as competent as the soils beneath the footing level. Otherwise 1.0

LRFD Table 10.6.3.1.2a-4

$$\frac{D_f}{B} = 0.038$$

Therefore : $d_q := 1.0$

Terms

$$N_{cm} := N_c \cdot s_c \cdot i_c$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$$

$N_{cm} = 30.1$ $\quad N_{\gamma m} = 18.643$ $\quad N_{qm} = 22.855$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c_1 \cdot N_{cm} + \gamma_{above} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma 1_{sat} \cdot \overrightarrow{\left(B \cdot N_{\gamma m}\right)} \cdot C_{w\gamma}\right]$$

$$q_n = 18.3 \cdot ksf$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

$$q_r := q_n \cdot \phi_b$$

$$\boxed{q_r = 8.2 \cdot ksf}$$

Recommend a factored bearing resistance of 8.2 ksf for footings 26 ft or greater on compacted granular fill.

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft^3	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

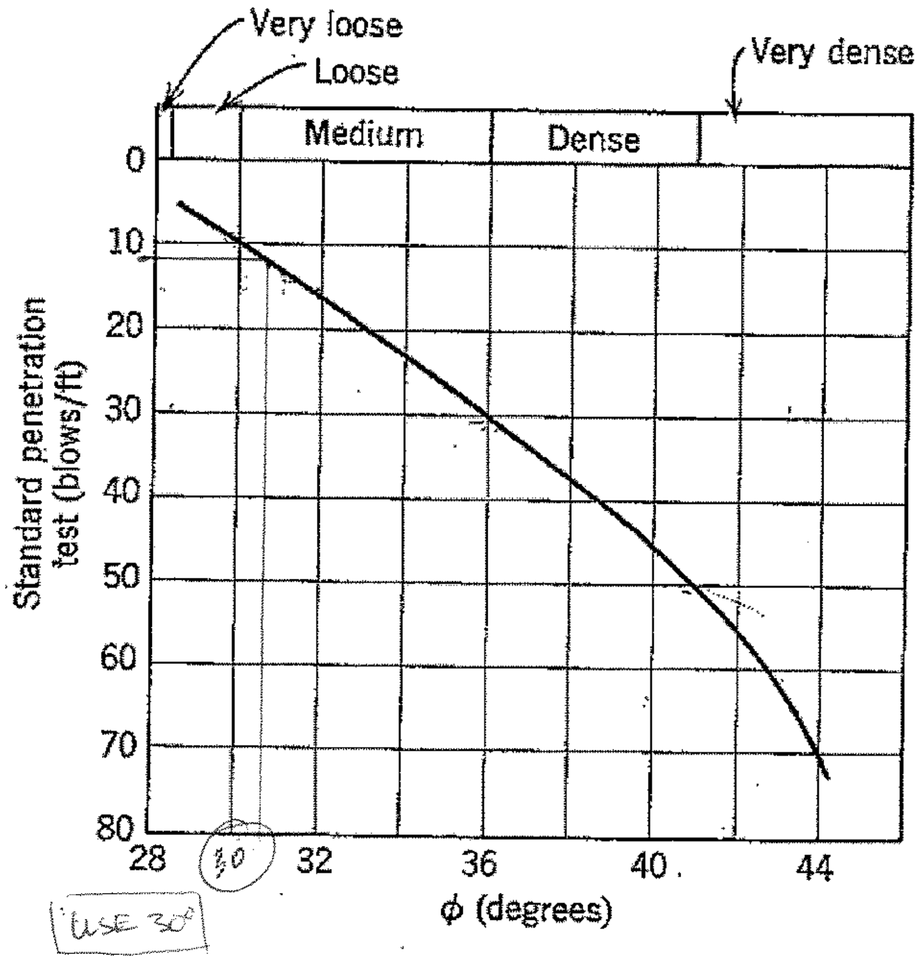


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Bowles Foundation Analysis and Design 5th ed.

The Terzaghi Bearing-Capacity Equation

One of the early sets of bearing-capacity equations was proposed by Terzaghi (1943) as shown in Table 4-1. These equations are similar to Eq. (k) derived in the previous section, but Terzaghi used shape factors noted when the limitations of the equation were discussed. Terzaghi's equations were produced from a slightly modified bearing-capacity theory devel-

TABLE 4-1
Bearing-capacity equations by the several authors indicated

Terzaghi (1943). See Table 4-2 for typical values and for $K_{p\gamma}$ values.

$$q_{\text{ult}} = cN_c s_c + \bar{q}N_q + 0.5\gamma B N_\gamma s_\gamma \qquad N_q = \frac{a^2}{a\cos^2(45+\phi/2)}$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

For:	strip	round	square
s_c =	1.0	1.3	1.3
s_γ =	1.0	0.6	0.8

Meyerhof (1963).* See Table 4-3 for shape, depth, and inclination factors.

Vertical load: $q_{\text{ult}} = cN_c s_c d_c + \bar{q}N_q s_q d_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma$

Inclined load: $q_{\text{ult}} = cN_c d_c i_c + \bar{q}N_q d_q i_q + 0.5\gamma B' N_\gamma d_\gamma i_\gamma$

$$N_q = e^{\pi\tan\phi}\tan^2\left(45 + \frac{\phi}{2}\right)$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

Hansen (1970).* See Table 4-5 for shape, depth, and other factors.

General:† $q_{\text{ult}} = cN_c s_c d_c i_c g_c b_c + \bar{q}N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$

when $\phi = 0$

use $q_{\text{ult}} = 5.14 s_u(1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 1.5(N_q - 1)\tan\phi$

Vesić (1973, 1975).* See Table 4-5 for shape, depth, and other factors.
Use Hansen's equations above.

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 2(N_q + 1)\tan\phi$

*These methods require a trial process to obtain design base dimensions since width B and length L are needed to compute shape, depth, and influence factors.

†See Sec. 4-6 when $i_i < 1$.

Bowles Foundation Analysis and Design 5th ed.

N_γ values shows the following:

ϕ	Terzaghi* (1943)	Bolton and Lau (1993)	Kumbhojkar (1993)	Table 4-2 (this text)
34°	36	43.5	32	36
48	780	638	650.7	780.1

*See Terzaghi (1943), Fig. 38 and page 128.

Fortunately the N_γ term does not make a significant contribution to the computed bearing capacity, so any of the values from Tables 4-2 or 4-4 can be used (or perhaps an average).

Bolton and Lau (1993) produced new N_q and N_γ values for strip and circular footings for both smooth and rough ground interfacings. Their N_q values for either smooth or rough strips are little different from the Hansen values for rough strips. The N_q values for circular footings range to more than two times the strip values. The N_γ values for rough footings compare well with the Vesić values in Table 4-4. Since the Table 4-4 values have shape s_i and depth d_i factors to be applied, it appears that these "new" values offer little advantage and are certainly more difficult to compute (see comparison with Terzaghi values in preceding table).

Meyerhof's Bearing-Capacity Equation

Meyerhof (1951, 1963) proposed a bearing-capacity equation similar to that of Terzaghi but included a shape factor s_q with the depth term N_q. He also included depth factors d_i and

TABLE 4-4

Bearing-capacity factors for the Meyerhof, Hansen, and Vesić bearing-capacity equations

Note that N_c and N_q are the same for all three methods; subscripts identify author for N_γ

ϕ	N_c	N_q	$N_{\gamma(H)}$	$N_{\gamma(M)}$	$N_{\gamma(V)}$	N_q/N_c	$2\tan\phi(1-\sin\phi)^2$
0	5.14*	1.0	0.0	0.0	0.0	0.195	0.000
5	6.49	1.6	0.1	0.1	0.4	0.242	0.146
10	8.34	2.5	0.4	0.4	1.2	0.296	0.241
15	10.97	3.9	1.2	1.1	2.6	0.359	0.294
20	14.83	6.4	2.9	2.9	5.4	0.431	0.315
25	20.71	10.7	6.8	6.8	10.9	0.514	0.311
26	22.25	11.8	7.9	8.0	12.5	0.533	0.308
28	25.79	14.7	10.9	11.2	16.7	0.570	0.299
30	30.13	18.4	15.1	15.7	22.4	0.610	0.289
32	35.47	23.2	20.8	22.0	30.2	0.653	0.276
34	42.14	29.4	28.7	31.1	41.0	0.698	0.262
36	50.55	37.7	40.0	44.4	56.2	0.746	0.247
38	61.31	48.9	56.1	64.0	77.9	0.797	0.231
40	75.25	64.1	79.4	93.6	109.3	0.852	0.214
45	133.73	134.7	200.5	262.3	271.3	1.007	0.172
50	266.50	318.5	567.4	871.7	761.3	1.195	0.131

* = $\pi + 2$ as limit when $\phi \rightarrow 0°$.

Slight differences in above table can be obtained using program BEARING.EXE on diskette depending on computer used and whether or not it has floating point.

AASHTO LRFD Bridge Design Specification, 7th ed. 2014

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

10.5.5.2.2—Spread Footings

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

C10.5.5.2.2

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

$$i_\gamma = \left[1 - \frac{H}{V + cBL\cot\phi_f}\right]^{(n+1)} \qquad (10.6.3.1.2a\text{-}8)$$

$$n = [(2 + L/B)/(1 + L/B)]\cos^2\theta + [(2 + B/L)/(1 + B/L)]\sin^2\theta \qquad (10.6.3.1.2a\text{-}9)$$

where:

B = footing width (ft)

L = footing length (ft)

H = unfactored horizontal load (kips)

V = unfactored vertical load (kips)

θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees)

It should further be noted that the resistance factors provided in Article 10.5.5.2.2 were derived for vertical loads. The applicability of these resistance factors to design of footings resisting inclined load combinations is not currently known. The combination of the resistance factors and the load inclination factors may be overly conservative for footings with an embedment of approximately D_f/B = 1 or deeper because the load inclination factors were derived for footings without embedment.

In practice, therefore, for footings with modest embedment, consideration may be given to omission of the load inclination factors.

Figure C10.6.3.1.2a-1 shows the convention for determining the θ angle in Eq. 10.6.3.1.2a-9.

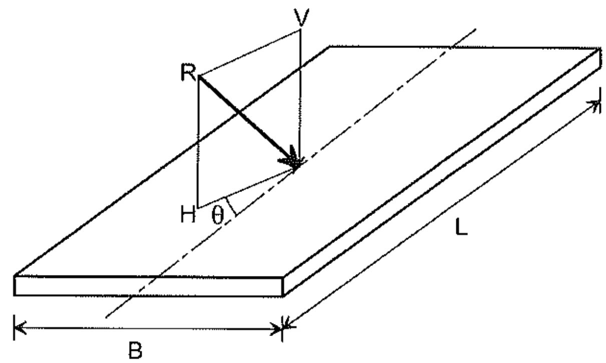


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Modulus of Subgrade Reaction

Objective:
Estimate the modulus of subgrade reaction for the box culvert design

Given:
1. Limited lab data

Assumptions:
1. The box culvert's embedment into the streambed is ignored.
2. The one foot thick layer of proposed Granular Borrow bedding material is ignored.
3. The proposed bearing elevation is approximately 566 feet.
4. Proposed finish roadway grade elevation is approximately 580.7 feet.
5. Proposed precast concrete box is 23 feet wide and 44 feet long (excluding slab connecting wingwalls).
6. The subsurface conditions present at the proposed bearing elevation in the borings are representative of the conditions for the entire site.
7. The bottom of the box culvert will be submerged for the structure's design life.

1. Estimate the subgrade modulus for the precast box culvert

Published values of subgrade modulus in submerged, medium dense, sand:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:
Range of modulus of subgrade reaction
Medium dense sand: k_s = 35 - 295 pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Range of modulus of subgrade reaction
Medium dense to very dense submerged coarse-grained soils: K_{v1}, 44 - 107 pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:
Typical subgrade reaction values for 0.3 m x 0.3 m plate
Saturated medium dense sand: $k_{0.3}$ (k_1) = 129-147 pci

Terzaghi Geotechnique, Vol. 5, No. 4, Table 1:
Values of vertical subgrade reaction for 1 ft x 1 ft plate on sand
Submerged sand, proposed: k_{s1} = 92 pci

Method 1: Many of the published ranges are wide or are unconservative for use in design. Use Terzaghi's recommended value, k_{s1} = 92 pci for a 1 ft x 1 ft plate and adjust to the dimensions of the box culvert base. (Width B - 26 ft, Length L = 62 ft)

Square to rectangle base adjustment:

$$k_{s1} := 92\text{pci} \qquad B := 26\text{ft} \qquad L := 62\text{ft}$$

$$k := \frac{k_{s1}\cdot\left[1 + 0.5\left(\frac{B}{L}\right)\right]}{1.5}$$

Das, Principles of Foundation Engineering 7th Ed. P. 311 Eqn. 6.44

$$k = 74.2\cdot\text{pci}$$

Method 2: Determine the modulus of subgrade reaction using the stress-strain modulus as proposed by Vesic using the method derived by Bowles that incorporates size effects.

Width of box culvert, B	$B = 26\,ft$	span and wall thickness
Depth of box, D	$D := 19.7ft$	approx. rise and cover
Length of box, L	$L := 62ft$	length neglects wingwall slab
Thickness of box, t	$t := 1ft$	assumed
Bearing Resistance at Service Limit, q	$q_{service} := 5ksf$	calculated

Modulus of Elasticity: Glacial Till between bearing elevation and bedrock is medium dense to very dense.
Bowles Table 2-8 provides:
loose sand range is 1450-3625 psi
dense sand range is 7250-11745 psi

Use average of high modulus of loose sand and low modulus of dense sand for medium dense sand.

$$E_s := 5437.5psi$$

Poisson's Ratio: medium dense gravelly sand
Bowles Table 2-7 provides:
sand and gravelly sand range is -0.1-1.00
commonly accepted range for design is 0.3-0.4

Use 0.35

$$\mu := 0.35$$

Adjust the Modulus of Elasticity for use in plane strain analysis:

$$E'_s := \frac{1 - \mu^2}{E_s}$$

Bowles Foundation Design and Analysis 5th Ed. P. 123 Eq. 2-65

$$E'_s := 1.614 \times 10^{-4} \cdot \frac{1}{psi}$$

At center of Box Culvert:

Determine the Steinbrenner influence factor, I_S:

Midpoint of box width, B':

$$B' := \frac{B}{2} \qquad B' = 13\,ft$$

Limit depth of influence to the depth to a hard layer (where E_s of lower layer is $10E_s$ of the upper layer) or 5B. Bearing Elev. = 245.5, Bedrock ranges from Elev. 238.5 to 223.5. Use average of Elev. 231.

$$H := 245.5ft - 231.0ft \qquad H = 14.5\,ft$$

Verify H to B ratio is less than 5:

$\frac{H}{B} = 0.558$ OK

$N := \frac{H}{B'}$ $N = 1.115$ Use 1.1

Placing the point of interest in the center of the box culvert will divide the mat into four subrectangles

$M := 4$ Number of contributing corners

Use Bowles Table 5-2 to determine I_1, I_2 with N=1.1 and M=4:

$I_1 := 0.131$ $I_2 := 0.12$

$$I_S := I_1 + \left(\frac{1 - 2\mu}{1 - \mu}\right) \cdot I_2$$

Steinbrenner Influence factor equation, Bowles pg. 306

$I_S = 0.186$

Determine the influence factor for footing at depth, I_F:

$D = 19.7\,\text{ft}$ $B = 26\,\text{ft}$ $L = 62\,\text{ft}$ $\mu = 0.35$

Depth ratio: $\frac{D}{B} = 0.758$

Length Ratio: $\frac{L}{B} = 2.385$

$I_F := 0.74$ Bowles, Fig. 5-7 pg. 303

$$k_s := \frac{1}{B' \cdot E'_s \cdot (M \cdot I_S) \cdot I_F}$$

Bowles Eq. 9-7 pg. 503

$k_s = 72 \cdot \text{pci}$

At corner of Box Culvert:

Determine the Steinbrenner influence factor, I_S:

box width, B:

$B = 26\,\text{ft}$

Depth influence factor:

$H = 14.5\,\text{ft}$ (same as midpoint)

Verify H to B ratio is less than 5:

$\frac{H}{B} = 0.558$ OK

$N := \frac{H}{B}$ $N = 0.558$ Use 0.6

Placing the point of interest in the corner of the box culvert results in one contributing corner and no subdivision.

$M := 1$ Number of contributing corners

Use Bowles Table 5-2 to determine I_1, I_2 with N=.3 and M=1:

$I_1 := 0.066$ $I_2 := 0.079$

$$I_S := I_1 + \left(\frac{1 - 2\mu}{1 - \mu}\right) \cdot I_2$$

Steinbrenner Influence factor equation, Bowles pg. 306

$I_S = 0.102$

Determine the influence factor for footing at depth, I_F: (same as midpoint)

$I_F := 0.78$

$$k_s := \frac{1}{B \cdot E'_s \cdot I_S \cdot I_F}$$ $k_s = 248 \cdot \text{pci}$

for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \tag{9-10}$$

r either horizontal or vertical members
for depth variation
terest below ground
give k_s the best fit (if load test or other data are available)

ation may be zero; at the ground surface A_s is zero for a lateral k_s
> 0. For footings and mats (plates in general), $A_s > 0$ and $B_s \cong 0$.
used with the proper interpretation of the bearing-capacity equa-
e d_i factors dropped) to give

$$q_{ult} = cN_c s_c + \gamma Z N_q s_q + 0.5\gamma B N_\gamma s_\gamma) \tag{9-10a}$$

$$s_c + 0.5\gamma B N_\gamma s_\gamma) \qquad \text{and} \qquad B_s Z^1 = C(\gamma N_q s_q) Z^1$$

to estimate k_s. In these equations the Terzaghi or Hansen bearing-
ed. The C factor is 40 for SI units and 12 for Fps, using the same
at a 0.0254-m and 1-in. settlement but with no SF, since this equa-
here there is concern that k_s does not increase without bound with
e $B_s Z$ term by one of two simple methods:

$$\text{Method 1: } B_s \tan^{-1}\frac{Z}{D}$$

$$\text{Method 2: } \frac{B_s}{D^n} Z^n = B_s' Z^n$$

lepth of interest, say, the length of a pile
h of interest
timate of the exponent

to estimate a value of k_s to determine the correct order of magnitude
btained using one of the approximations given here. Obviously if a
three times larger than the table range indicates, the computations
possible gross error. Note, however, if you use a reduced value of
r 12 mm) instead of 0.0254 m you may well exceed the table range.
putational error (or a poor assumption) is found then use judgment
table values are intended as guides. The reader should not use, say,
ven as a "good" estimate.
d in Fig. 9-9c (and used in your diskette program FADBEMLP as
stimated at some small value of, say, 6 to 25 mm, or from inspection
if a load test was done. It might also be estimated from a triaxial
ltimate" or at the maximum pressure from the stress-strain plot.
ϵ_{max} compute

$$X_{max} = \epsilon_{max}(1.5 \text{ to } 2B)$$

Bowles , Foundation Analysis and Design, 5th ed. p. 505

TABLE 9-1
Range of modulus of subgrade reaction k_s

Use values as guide and for comparison when using approximate equations

$$\frac{kN}{M^3} \rightarrow \frac{lb}{in^3} : \frac{224.8\ lb}{1\ kN} * \frac{1M^3}{61023.7 in^3} = .003684 \frac{kN}{M^3} = 1 \frac{lb}{in^3}$$

Soil	k_s, kN/m³	ks, lb/in^3
Loose sand	4800–16 000	18 - 59
Medium dense sand	9600–80 000	35 - 295
Dense sand	64 000–128 000	236 - 472
Clayey medium dense sand	32 000–80 000	118 - 295
Silty medium dense sand	24 000–48 000	88 - 177
Clayey soil:		
$q_a \le 200$ kPa	12 000–24 000	44 - 88
$200 < q_a \le 800$ kPa	24 000–48 000	88 - 177
$q_a > 800$ kPa	> 48 000	> 177

The 1.5 to 2B dimension is an approximation of the depth of significant stress-strain influence (Boussinesq theory) for the structural member. The structural member may be either a *footing* or a *pile*.

Example 9-5. Estimate the modulus of subgrade reaction k_s for the following design parameters:

$$B = 1.22 \text{ m} \qquad L = 1.83 \text{ m} \qquad D = 0.610 \text{ m}$$

$$q_a = 200 \text{ kPa (clayey sand approximately 10 m deep)}$$

$$E_s = 11.72 \text{ MPa (average in depth } 5B \text{ below base)}$$

Solution. Estimate Poisson's ratio $\mu = 0.30$ so that

$$E_s' = \frac{1-\mu^2}{E_s} = \frac{1-0.3^2}{11.72} = 0.077\,65 \text{ m}^2/\text{MN}$$

For **center:**

$$H/B' = 5B/(B/2) = 10 \text{ (taking } H = 5B \text{ as recommended in Chap. 5)}$$

$$L/B = 1.83/1.22 = 1.5$$

From these we may write

$$I_s = 0.584 + \frac{1-2(0.3)}{1-0.3}0.023 = \mathbf{0.597}$$

using Eq. (5-16) and Table 5-2 (or your program FFACTOR) for factors 0.584 and 0.023.

At $D/B = 0.61/1.22 = 0.5$, we obtain $I_F = 0.80$ from Fig. 5-7 (or when using FFACTOR for the I_s factors). Substitution into Eq. (9-7) with $B' = 1.22/2 = 0.61$, and $m = 4$ yields

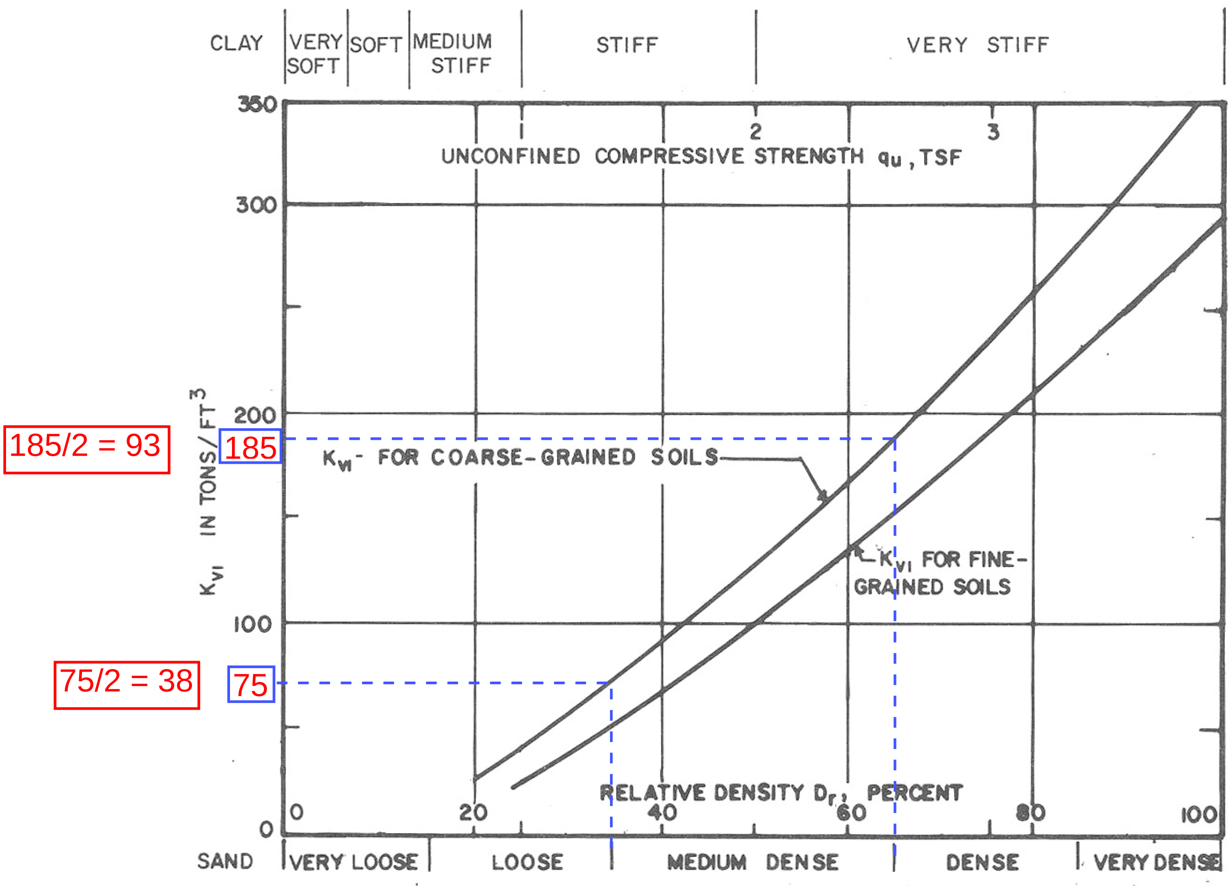
$$k_s = \frac{1}{0.61(0.077\,65)(4 \times 0.597)(0.8)} = \mathbf{11.05} \text{ MN/m}^3$$

You should note that k_s does not depend on the contact pressure of the base q_o.

For **corner:**

$$H/B' = 5B/B = 5(1.22)/1.22 = 5$$

[from Table 5-2 with $L/B = 1.5$ obtained for Eq. (5-16)]



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING

q = FOOTING UNIT LOAD IN tsf

B = FOOTING WIDTH

D = DEPTH OF FOOTING BELOW GROUND SURFACE

K_{VI} = MODULUS OF VERTICAL SUBGRADE REACTION

$$\frac{ton}{ft^3} \rightarrow \frac{lb}{in^3} = \frac{2000\ lb}{1\ ton} * \frac{1\ ft^3}{1728\ in^3} = 1.157\frac{ton}{ft^3} \rightarrow 1\frac{lb}{in^3}$$

93*1.15= 107 pci

38*1.15= 44 pci

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{4\ q\ B^2}{K_{VI}\ (B+1)^2}$$

FOR $B \geq 40$ FT:

$$\Delta H_i = \frac{2\ q\ B^2}{K_{VI}\ (B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{2\ q\ B^2}{K_{VI}\ (B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.

2. VALUES OF K_{VI} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{VI}/2$ IN COMPUTING SETTLEMENT

Figure 8-3: Modulus of Subgrade Reaction (NAVFAC, 1986a)

 Das, Priciples of Foundation Engineering 7th Ed.

Equation (6.44) indicates that the value of k for a very long foundation with a width B is approximately $0.67k_{(B \times B)}$.

The modulus of elasticity of granular soils increases with depth. Because the settlement of a foundation depends on the modulus of elasticity, the value of k increases with the depth of the foundation.

Table 6.2 provides typical ranges of values for the coefficient of subgrade reaction, $k_{0.3}(k_1)$, for sandy and clayey soils.

For long beams, Vesic (1961) proposed an equation for estimating subgrade reaction, namely,

$$k' = Bk = 0.65 \sqrt[12]{\frac{E_s B^4}{E_F I_F}} \frac{E_s}{1 - \mu_s^2}$$

or

$$k = 0.65 \sqrt[12]{\frac{E_s B^4}{E_F I_F}} \frac{E_s}{B(1 - \mu_s^2)} \tag{6.45}$$

where

E_s = modulus of elasticity of soil
B = foundation width
E_F = modulus of elasticity of foundation material
I_F = moment of inertia of the cross section of the foundation
μ_s = Poisson's ratio of soil

$$\frac{MN}{m^3} \rightarrow \frac{lb}{in^3}: \frac{224809\ lb}{1\ MN} * \frac{1\ m^3}{61024\ in^3} \rightarrow 3.684 \frac{lb}{in^3} = \frac{1\ MN}{M^3}$$

Table 6.2 Typical Subgrade Reaction Values, $k_{0.3}(k_1)$

Soil type	$k_{0.3}(k_1)$ MN/m³	pci
Dry or moist sand:		
Loose	8–25	29 - 92
Medium	25–125	92 - 461
Dense	125–375	461 - 1382
Saturated sand:		
Loose	10–15	37 - 55
Medium	35–40	129 - 147
Dense	130–150	478 - 553
Clay:		
Stiff	10–25	37 - 92
Very stiff	25–50	92 - 184
Hard	>50	> 184

Terzaghi, Geotechnique, Vol. 5 No. 4

the bending moments in piles which are acted upon by horizontal forces above the ground surface (Cummings, 1937) and of those in core-walls of earth- and rock-fill dams (Löfquist, 1951).

Attempts have also been made to apply the theories to the solution of bulkhead problems (Rifaat, 1935). Baumann (1935) used them for estimating the stresses in an anchored bulkhead which had failed. Quite recently Blum (1951) proposed a procedure for the design of anchored bulkheads by means of the theory of horizontal subgrade reaction. All these investigations and design procedures were based on the tacit assumption that K'_0 in equation (15) is identical with the coefficient of active earth pressure K_a. The error due to this assumption may be quite important.

EVALUATION OF COEFFICIENTS OF SUBGRADE REACTION

General procedure

The numerical values of the coefficients of subgrade reaction k_s and k_h required for the solution of engineering problems can either be estimated on the basis of published observational data or else they can be derived from the results of field tests to be performed on the subgrade of the proposed structure. For practical purposes, rough estimates of these values fully serve their purpose.

Vertical subgrade reaction

As a basis for estimating the coefficient of subgrade reaction k_s for beams and slabs, the value $\bar{k}_{s1}$ for a square plate with a width of 1 ft has been selected, because this value can, if necessary, be determined by averaging the results of several loading tests in the field, at the site of the structure.

If the subgrade consists of cohesionless or slightly cohesive sand, k_s can be estimated on the basis of the empirical values of $\bar{k}_{s1}$ given in Table 1. The density-category of the sand can be ascertained by means of a standard penetration test or other convenient means. The greatest error on the unsafe side results from using the proposed value 130 in the case of medium sand if its real value is equal to the lower limiting value of 60 tons/cu. ft.

Table 1.

Values of $\bar{k}_{s1}$ in tons/cu. ft for square plates, 1 ft × 1 ft, or beams 1 ft wide, resting on sand

Relative density of sand	Loose	Medium	Dense
Dry or moist sand, limiting values for $\bar{k}_{s1}$	20–60	60–300	300–1,000
Dry or moist sand, proposed values	40	130	500
Submerged sand, proposed values	25	80	300

In order to investigate the influence of such an error on the results of the computation of the bending moments in a beam, the maximum bending moment M_{max} in the beam shown in Fig. 1 was computed on the basis of both the assumed and the real value of $\bar{k}_{s1}$ for the supporting sand. The value of M_{max} for this beam is determined by equation (4). It was found that the moment computed by means of the proposed value exceeds the actual bending moment by not more than about 5%.

Once the value $\bar{k}_{s1}$ has been selected, the value of k_s to be used in the solution of a given

$$\frac{ton}{ft^3} \rightarrow \frac{lb}{in^3} = \frac{2000\ lb}{1\ ton} * \frac{1\ ft^3}{1728\ in^3} = 1.157\frac{ton}{ft^3} \rightarrow 1\frac{lb}{in^3}$$

80 ton/cu. ft * 1.15 = 92 pci

EVAL

problem can be cor headings. Experien sand is roughly equ (Fig. 3) or for a mat equation (8):

If applied to sp contact pressures su unit of area of the l porting concentrate half of the ultimat equation (9).

Va fo

Values Range Propos

For rec

* Highe

If the subgrade ately in simple pro basis of our presen numerical values o pressures which ar The latter is indep The proposed v medium sand, Tab of the loaded area normally consolida beams and rafts sh perfectly rigid.

The $\bar{k}_{s1}$ values f of the tests can be of such tests is too the test results can of the block shoul If the contact the value:

For $l = \infty$, $\bar{k}_{s1}$ = loaded subgrade l

The unit of k is kN/m^3. The value of the coefficient of subgrade reaction is not a constant for a given soil, but rather depends on several factors, such as the length L and width B of the foundation and also the depth of embedment of the foundation. A comprehensive study by Terzaghi (1955) of the parameters affecting the coefficient of subgrade reaction indicated that the value of the coefficient decreases with the width of the foundation. In the field, load tests can be carried out by means of square plates measuring 0.3 m $\times$ 0.3 m, and values of k can be calculated. The value of k can be related to large foundations measuring $B \times B$ in the following ways:

Foundations on Sandy Soils

For foundations on sandy soils,

$$k = k_{0.3}\left(\frac{B + 0.3}{2B}\right)^2 \tag{6.42}$$

where $k_{0.3}$ and k = coefficients of subgrade reaction of foundations measuring 0.3 m $\times$ 0.3 m and $B\,(\mathrm{m}) \times B\,(\mathrm{m})$, respectively (unit is kN/m^3).

Foundations on Clays

For foundations on clays,

$$k\,(\mathrm{kN/m^3}) = k_{0.3}(\mathrm{kN/m^3})\left[\frac{0.3\ (\mathrm{m})}{B(\mathrm{m})}\right] \tag{6.43}$$

The definitions of k and $k_{0.3}$ in Eq. (6.43) are the same as in Eq. (6.42).

For rectangular foundations having dimensions of $B \times L$ (for similar soil and q),

Method 1:

$$k = \frac{k_{(B\times B)}\left(1 + 0.5\dfrac{B}{L}\right)}{1.5} \tag{6.44}$$

where

k = coefficient of subgrade modulus of the rectangular foundation $(L \times B)$

$k_{(B\times B)}$ = coefficient of subgrade modulus of a square foundation having dimension of $B \times B$

Bowles, Foundation Analysis and Design 5th Ed. p. 125

TABLE 2-8

Value range* for the static stress-strain modulus E_s for selected soils (see also Table 5-6)

Field values depend on stress history, water content, density, and age of deposit

1 Mpa = 145 psi

Soil	E_s, MPa	Es, psi
Clay		
Very soft	2–15	
Soft	5–25	
Medium	15–50	
Hard	50–100	
Sandy	25–250	
Glacial till		
Loose	10–150	
Dense	150–720	
Very dense	500–1440	
Loess	15–60	
Sand		
Silty	5–20	725 - 2900
Loose	10–25	1450 - 3625
Dense	50–81	7250 - 11745
Sand and gravel		
Loose	50–150	7250 - 21750
Dense	100–200	14500 - 29000
Shale	150–5000	
Silt	2–20	

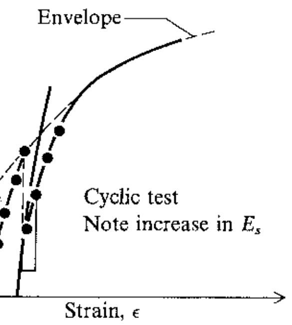
*Value range is too large to use an "average" value for design.

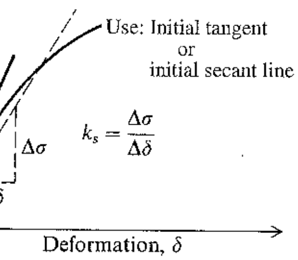
in situ, it is reasonable for confined compression tests to produce better "elastic" parameters. Although it is difficult to compare laboratory and field E_s values, there is some evidence that field values are often four to five times larger than laboratory values from the unconfined compression test. For this reason, current practice tends to try to obtain "field" values from in situ testing whenever possible. This topic will be taken up in more detail in the next chapter.

Table 2-8 gives a range of E_s values that might be obtained. Note that the range is very large, owing to the foregoing factors as well as those factors given on the table. With this wide range of values the reader should not try to use "averaged" values from this table for design.

If laboratory test plots similar to Fig. 2-43*a* are used, it is most common to use the initial tangent modulus to compute the stress-strain modulus E_s for the following reasons:

1. Soil is elastic only near the origin.
2. There is less divergence between all plots in this region.
3. The largest values are obtained—often three to five times larger than a tangent or secant modulus from another point along the curve.





lodulus of subgrade reaction.

(*c*)

ϵ_v defined using initial mass

(*d*)

his equation.
st described in Sec. 2-10 or
al strain (ϵ_2, ϵ_3) is taken as
$\epsilon_1 = \epsilon_v$, one can obtain the

$$\frac{\sigma_1}{G'} \qquad (e)$$

se σ_z for σ_1 in this equation.
ic strain $\epsilon_v = 0.0$; i.e., there
c strain is $\epsilon_v = \sigma_z/E_s = \epsilon_z$.
6*b*.

Bowles, Foundation Analysis and Design
5th ed. p. 123

TABLE 2-7
Values or value ranges for Poisson's ratio μ

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand, gravelly sand	−0.1–1.00
commonly used	0.3–0.4
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15
Steel	0.33

Another material property concept is the *bulk modulus* E_b, which is defined as the rat
hydrostatic stress to the volumetric strain ϵ_v and is given as

$$E_b = \frac{2}{3}G'\frac{1+\mu}{1-2\mu} = \frac{E_s}{3(1-2\mu)}$$

For an *elastic* material the shear modulus G' cannot be (−), so Eq. (*a*) sets the lower lim
$\mu > -1$. Equation (*f*) sets the upper limit at $\mu < 0.5$. It appears that the range of μ for
(that are not "elastic") is from about −0.1 to 1.00. Table 2-7 gives a range of values for s
materials. It is very common to use the following values for soils:

μ	Soil type
0.4–0.5	Most clay soils
0.45–0.50	Saturated clay soils
0.3–0.4	Cohesionless—medium and dense
0.2–0.35	Cohesionless—loose to medium

Although it is common to use $\mu = 0.5$ for saturated clay soils, the reader should be av
that this represents a condition of no volume change under the applied stress σ_z. Over t
however, volume change does occur as the pore fluid drains. Equation (*e*) defines the Poiss
ratio that develops initially ($\epsilon_v = 0$) and also later when $\epsilon_v > 0$. Since the strain is produ
from stress and Fig. 1-1 indicates a vertical variation, it necessarily follows that μ is str
dependent from Eq. (*e*).

A special case in geotechnical work is that of *plane strain*. This arises where strains o
parallel to two of the coordinate axes (say the *x* and *z*) but the strain is zero perpendic
to the *x*-*z* plane (along the *y* axis). If we set $\epsilon_y = 0$ in the set of equations for Hooke's
[(Eqs. (2-64)] and solve for the resulting values of E_s and μ, we obtain the following:

$$\boxed{E_s' = \frac{E_s}{1-\mu^2}} \qquad \mu' = \frac{\mu}{1-\mu} \qquad (2$$

Bowles Foundation Analysis and Design 5th ed. p. 304

TABLE 5-2

Values of I_1 and I_2 to compute the Steinbrenner influence factor I_s for use in Eq. (5-16a) for several $N = H/B'$ and $M = L/B$ ratios

N		M = 1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.2	I_1 =	0.009	0.008	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007	0.007
	I_2 =	0.041	0.042	0.042	0.042	0.042	0.042	0.043	0.043	0.043	0.043	0.043
0.4		0.033	0.032	0.031	0.030	0.029	0.028	0.028	0.027	0.027	0.027	0.027
		0.066	0.068	0.069	0.070	0.070	0.071	0.071	0.072	0.072	0.073	0.073
0.6		0.066	0.064	0.063	0.061	0.060	0.059	0.058	0.057	0.056	0.056	0.055
		0.079	0.081	0.083	0.085	0.087	0.088	0.089	0.090	0.091	0.091	0.092
0.8		0.104	0.102	0.100	0.098	0.096	0.095	0.093	0.092	0.091	0.090	0.089
		0.083	0.087	0.090	0.093	0.095	0.097	0.098	0.100	0.101	0.102	0.103
1.0		0.142	0.140	0.138	0.136	0.134	0.132	0.130	0.129	0.127	0.126	0.125
		0.083	0.088	0.091	0.095	0.098	0.100	0.102	0.104	0.106	0.108	0.109
1.5		0.224	0.224	0.224	0.223	0.222	0.220	0.219	0.217	0.216	0.214	0.213
		0.075	0.080	0.084	0.089	0.093	0.096	0.099	0.102	0.105	0.108	0.110
2.0		0.285	0.288	0.290	0.292	0.292	0.292	0.292	0.292	0.291	0.290	0.289
		0.064	0.069	0.074	0.078	0.083	0.086	0.090	0.094	0.097	0.100	0.102
3.0		0.363	0.372	0.379	0.384	0.389	0.393	0.396	0.398	0.400	0.401	0.402
		0.048	0.052	0.056	0.060	0.064	0.068	0.071	0.075	0.078	0.081	0.084
4.0		0.408	0.421	0.431	0.440	0.448	0.455	0.460	0.465	0.469	0.473	0.476
		0.037	0.041	0.044	0.048	0.051	0.054	0.057	0.060	0.063	0.066	0.069
5.0		0.437	0.452	0.465	0.477	0.487	0.496	0.503	0.510	0.516	0.522	0.526
		0.031	0.034	0.036	0.039	0.042	0.045	0.048	0.050	0.053	0.055	0.058
6.0		0.457	0.474	0.489	0.502	0.514	0.524	0.534	0.542	0.550	0.557	0.563
		0.026	0.028	0.031	0.033	0.036	0.038	0.040	0.043	0.045	0.047	0.050
7.0		0.471	0.490	0.506	0.520	0.533	0.545	0.556	0.566	0.575	0.583	0.590
		0.022	0.024	0.027	0.029	0.031	0.033	0.035	0.037	0.039	0.041	0.043
8.0		0.482	0.502	0.519	0.534	0.549	0.561	0.573	0.584	0.594	0.602	0.611
		0.020	0.022	0.023	0.025	0.027	0.029	0.031	0.033	0.035	0.036	0.038
9.0		0.491	0.511	0.529	0.545	0.560	0.574	0.587	0.598	0.609	0.618	0.627
		0.017	0.019	0.021	0.023	0.024	0.026	0.028	0.029	0.031	0.033	0.034
10.0		0.498	0.519	0.537	0.554	0.570	0.584	0.597	0.610	0.621	0.631	0.641
		0.016	0.017	0.019	0.020	0.022	0.023	0.025	0.027	0.028	0.030	0.031
20.0		0.529	0.553	0.575	0.595	0.614	0.631	0.647	0.662	0.677	0.690	0.702
		0.008	0.009	0.010	0.010	0.011	0.012	0.013	0.013	0.014	0.015	0.016
500.0		**0.560**	0.587	0.612	0.635	0.656	0.677	0.696	0.714	0.731	0.748	0.763
		0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001

Corner:

Midpoint

By interpolation:

I_1=0.131

I_2=0.122

TABLE 5-2

Values of I_1 and I_2 to compute the Steinbrenner influe[nce factor I_s for use] in Eq. (5-16a) for several $N = H/B'$ and $M = L/B$ rati[os]

N		M = 2.5	4.0	5.0	6.0	7.0	8.0	9.0
0.2	I_1 =	0.007	0.006	0.006	0.006	0.006	0.006	0.006
	I_2 =	0.043	0.044	0.044	0.044	0.044	0.044	0.044
0.4		0.026	0.024	0.024	0.024	0.024	0.024	0.024
		0.074	0.075	0.075	0.075	0.076	0.076	0.076
0.6		0.053	0.051	0.050	0.050	0.050	0.049	0.049
		0.094	0.097	0.097	0.098	0.098	0.098	0.098
0.8		0.086	0.082	0.081	0.080	0.080	0.080	0.079
		0.107	0.111	0.112	0.113	0.113	0.113	0.113
1.0		0.121	0.115	0.113	0.112	0.112	0.112	0.111
		0.114	0.120	0.122	0.123	0.123	0.124	0.124
1.5		0.207	0.197	0.194	0.192	0.191	0.190	0.190
		0.118	0.130	0.134	0.136	0.137	0.138	0.138
2.0		0.284	0.271	0.267	0.264	0.262	0.261	0.260
		0.114	0.131	0.136	0.139	0.141	0.143	0.144
3.0		0.402	0.392	0.386	0.382	0.378	0.376	0.374
		0.097	0.122	0.131	0.137	0.141	0.144	0.145
4.0		0.484	0.484	0.479	0.474	0.470	0.466	0.464
		0.082	0.110	0.121	0.129	0.135	0.139	0.142
5.0		0.553	0.554	0.552	0.548	0.543	0.540	0.536
		0.070	0.098	0.111	0.120	0.128	0.133	0.137
6.0		0.585	0.609	0.610	0.608	0.604	0.601	0.598
		0.060	0.087	0.101	0.111	0.120	0.126	0.131
7.0		0.618	0.653	0.658	0.658	0.656	0.653	0.650
		0.053	0.078	0.092	0.103	0.112	0.119	0.125
8.0		0.643	0.688	0.697	0.700	0.700	0.698	0.695
		0.047	0.071	0.084	0.095	0.104	0.112	0.118
9.0		0.663	0.716	0.730	0.736	0.737	0.736	0.735
		0.042	0.064	0.077	0.088	0.097	0.105	0.112
10.0		0.679	0.740	0.758	0.766	0.770	0.770	0.770
		0.038	0.059	0.071	0.082	0.091	0.099	0.106
20.0		0.756	0.856	0.896	0.925	0.945	0.959	0.969
		0.020	0.031	0.039	0.046	0.053	0.059	0.065
500.0		0.832	0.977	1.046	1.102	1.150	1.191	1.227
		0.001	0.001	0.002	0.002	0.002	0.003	0.003

Bowles, Foundation Design and Analysis
5th ed. p. 306

$$N = \frac{H}{B'}$$

$$B' = \frac{B}{2} \text{ for center; } = B \text{ for corner } I_i$$

$$L' = L/2 \text{ for center; } = L \text{ for corner } I_i$$

The influence factor I_F is from the Fox (1948*b*) equations, which suggest that the settlement is reduced when it is placed at some depth in the ground, depending on Poisson's ratio and L/B. Figure 5-7 can be used to approximate I_F. You can also use Table 5-2, which gives a select range of I_1 and I_2 values, to compute the composite Steinbrenner influence factor I_s as

$$\boxed{I_s = I_1 + \frac{1 - 2\mu}{1 - \mu} I_2} \tag{c}$$

Program FFACTOR (option 6) can be used to obtain both I_F and I_s directly; you have only to input appropriate base dimensions (actual L, B for I_F and B', L' for I_s) and Poisson's ratio μ.

Equation (5-16) can be written more compactly as follows:

$$\Delta H = q_o B' \frac{1 - \mu^2}{E_s} m I_s I_F \tag{5-16a}$$

where I_s is defined in Eq. (*c*) and m = number of corners contributing to settlement ΔH. At the footing center $m = 4$; at a side $m = 2$, and at a corner $m = 1$. Not all the rectangles have to have the same L'/B' ratio, but for any footing, use a constant depth H.

This equation is strictly applicable to *flexible bases* on the half-space. The half-space may consist of either cohesionless materials of any water content or unsaturated cohesive soils. The soils may be either inorganic or organic; however, if organic, the amount of organic material should be very small, because both E_s and μ are markedly affected by high organic content. Also, in organic soils the foregoing equation has limited applicability since secondary compression or "creep" is usually the predominating settlement component.

In practice, most foundations are flexible. Even very thick ones deflect when loaded by the superstructure loads. Some theory indicates that if the base is rigid the settlement will be uniform (but may tilt), and the settlement factor I_s will be about 7 percent less than computed by Eq. (*c*). On this basis if your base is "rigid" you should reduce the I_s factor by about 7 percent (that is, $I_{sr} = 0.931 I_s$).

Equation (5-16*a*) is very widely used to compute immediate settlements. These estimates, however, have not agreed well with measured settlements. After analyzing a number of cases, the author concluded that the equation is adequate but the method of using it was incorrect. The equation should be used [see Bowles (1987)] as follows:

1. Make your best estimate of base contact pressure q_o.
2. For round bases, convert to an equivalent square.
3. Determine the point where the settlement is to be computed and divide the base (as in the Newmark stress method) so the point is at the corner or common corner of one or up to 4 contributing rectangles (see Fig. 5-7).

TABLE 5-3
Comparison of computed versus measured settlement for a number of cases provided by the references cited.

Reference	H, ft	B, ft	L/B	D/B	N or q_c	E_s, ksf	μ	Δp, ksf	I_s	I_f	Settlement, in. Computed	Settlement, in. Measured
D'Appolonia et al. (1968)	$4B$	12.5	1.6	0.5	25*	1,200	0.33	3.4	0.589	0.75	0.33	0.3–0.4
Schmertmann (1970)												
Case 1	$5B$	8.5	8.8	0.78	40	310	0.4	3.74	0.805	0.87	1.45	1.53
Case 2	$5B$	9.8	4.2	1	120	620	0.3	3.34	0.774	0.75	0.67	0.8–0.9
Case 5	$5B$	62	1.0	0	65	350	0.45	1.56	0.50	1.0	2.64	2.48
Case 6	B	87	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

where E_s, E_f = modulus of soil and footing, respectively, in consistent units
B, I_f = footing width and its moment of inertia based on cross section (not plan) in consistent units

One can obtain k_s from k'_s as

$$k_s = \frac{k'_s}{B}$$

Since the twelfth root of any value $\times$ 0.65 will be close to 1, for all practical purposes the Vesić equation reduces to

$$k_s = \frac{E_s}{B(1-\mu^2)} \tag{9-6a}$$

One may rearrange Eq. (5-16*a*) and, using $E'_s = (1-\mu^2)/E_s$ as in Eqs. (5-18) and (5-19) and $m = 1$, obtain

$$\Delta H = \Delta q B E'_s I_s I_F$$

Bowles Foundation Analysis and Design 5th ed. p. 503

and, since k_s is defined as $\Delta q/\Delta H$, obtain

$$\boxed{k_s = \frac{\Delta q}{\Delta H} = \frac{1}{BE'_s I_s I_F} \qquad (9\text{-}7)}$$

but carefully note the definition of E'_s. Now one can correctly incorporate the size effects that are a major concern—particularly for the mat foundations of the next chapter. As for Eqs. (5-18) and (5-19), we can write a k_s ratio from Eq. (9-7) as follows:

$$\frac{k_{s1}}{k_{s2}} = \frac{B_2 E'_{s2} I_{s2} I_{F2}}{B_1 E'_{s1} I_{s1} I_{F1}} \tag{9-8}$$

Equation (9-8) should be used instead of Eqs. (9-3) through (9-5), and Eq. (9-7) is at least as theoretically founded as Eq. (9-6). Carefully note in using these equations that their basis is in the settlement equation [Eq. (5-16*a*)] of Chap. 5, and use B, I_s, and I_F as defined there.

Equations (9-7) and (9-8) show a direct relationship between k_s and E_s. Since one does not often have values of E_s, other approximations are useful and often quite satisfactory if the computed deflection (directly dependent on k_s) can be tolerated for any reasonable value. It has been found that bending moments and the computed soil pressure are not very sensitive to what is used for k_s because the structural member stiffness is usually 10 or more times as great as the soil stiffness as defined by k_s. Recognizing this, the author has suggested the following for approximating k_s from the allowable bearing capacity q_a furnished by the geotechnical consultant:

$$\begin{aligned} \text{SI: } k_s &= 40(\text{SF})q_a \qquad \text{kN/m}^3 \\ \text{Fps: } k_s &= 12(\text{SF})q_a \qquad \text{k/ft}^3 \end{aligned} \tag{9-9}$$

where q_a is furnished in ksf or kPa. This equation is based on $q_a = q_{\text{ult}}/\text{SF}$ and the ultimate soil pressure is at a settlement ΔH = 0.0254 m or 1 in. (1/12 ft) and k_s is $q_{\text{ult}}/\Delta H$. For ΔH = 6, 12, 20 mm, etc., the factor 40 (or 12) can be adjusted to 160 (or 48), 83 (or 24), 50 (or 16), etc.; 40 is reasonably conservative but smaller assumed displacements can always be used.

Embankment Settlement

Settle3D Analysis Information
20815 Athens-Hartland Corson Corner Br

Project Settings

Document Name	Embankment1_Borehole.s3z
Project Title	20815 Athens-Hartland Corson Corner Br
Analysis	Embankment Settlement
Author	Brandon Slaven
Company	MaineDOT
Date Created	9/28/2017, 10:14:18 AM
Stress Computation Method	Boussinesq
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.271689
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.271689
Loading Stress [ksf]	0.056266	0.27
Effective Stress [ksf]	0.0676036	4.04641
Total Stress [ksf]	0.0676036	6.71089
Total Strain	7.18555e-005	0.00134164
Pore Water Pressure [ksf]	0	2.66448
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.0731275	4.04416
Over-consolidation Ratio	1	1
Void Ratio	0	0
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0.799408

Embankments

1. Embankment: "Fill"

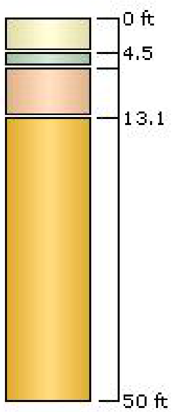
Label	Fill
Center Line	(0, 0) to (0, 54.815)
Number of Layers	1
Near End Angle	90 degrees
Far End Angle	90 degrees
Base Width	42

Layer	Stage	Left Bench Width (ft)	Left Angle (deg)	Height (ft)	Unit Weight (kips/ft^3)	Right Angle (deg)	Right Bench Width (ft)
1	Stage 1	0	26.6	2	0.135	26.6	0

Soil Layers

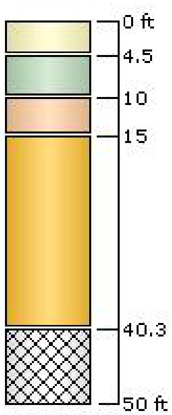
BB-AHBS-101: (-6.3, 30)

Layer #	Type	Thickness [ft]	Depth [ft]
1	Dense to Very Dense, FILL	4.5	0
2	Very Loose to Dense, SAND	2	4.5
3	Soft to Stiff, SILT	6.6	6.5
4	Dense or Hard, Glacial Till	36.9	13.1



BB-AHBS-201: (6.4, 23.3)

Layer #	Type	Thickness [ft]	Depth [ft]
1	Dense to Very Dense, FILL	4.5	0
2	Very Loose to Dense, SAND	5.5	4.5
3	Soft to Stiff, SILT	5	10
4	Dense or Hard, Glacial Till	25.3	15



Soil Properties

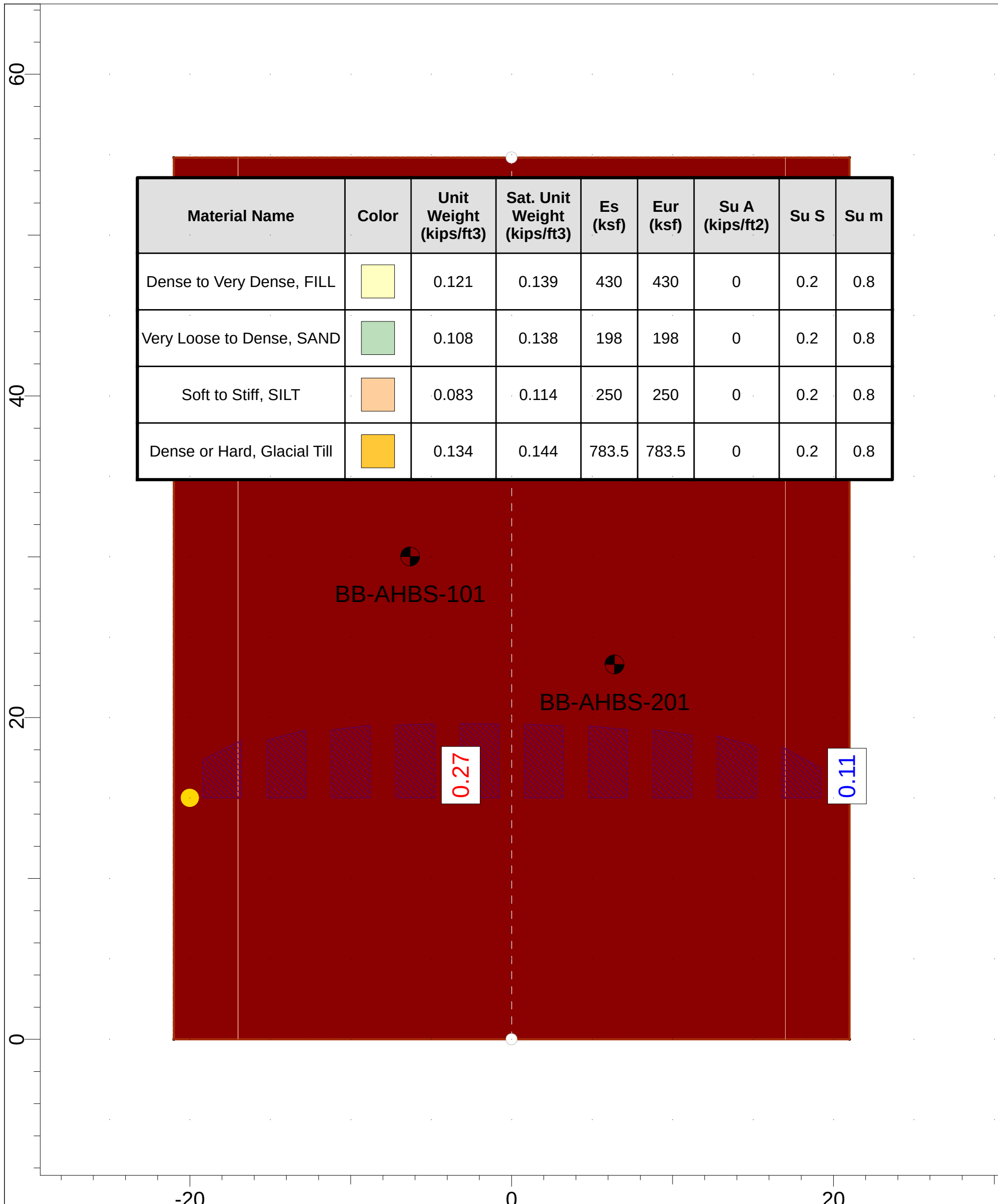
Property	Dense to Very Dense, FILL	Very Loose to Dense, SAND	Soft to Stiff, SILT	Dense or Hard, Glacial Till
Color				
Unit Weight [kips/ft^3]	0.121	0.108	0.083	0.134
Saturated Unit Weight [kips/ft^3]	0.139	0.138	0.114	0.144
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	430	198	250	783.5
Esur [ksf]	430	198	250	783.5
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

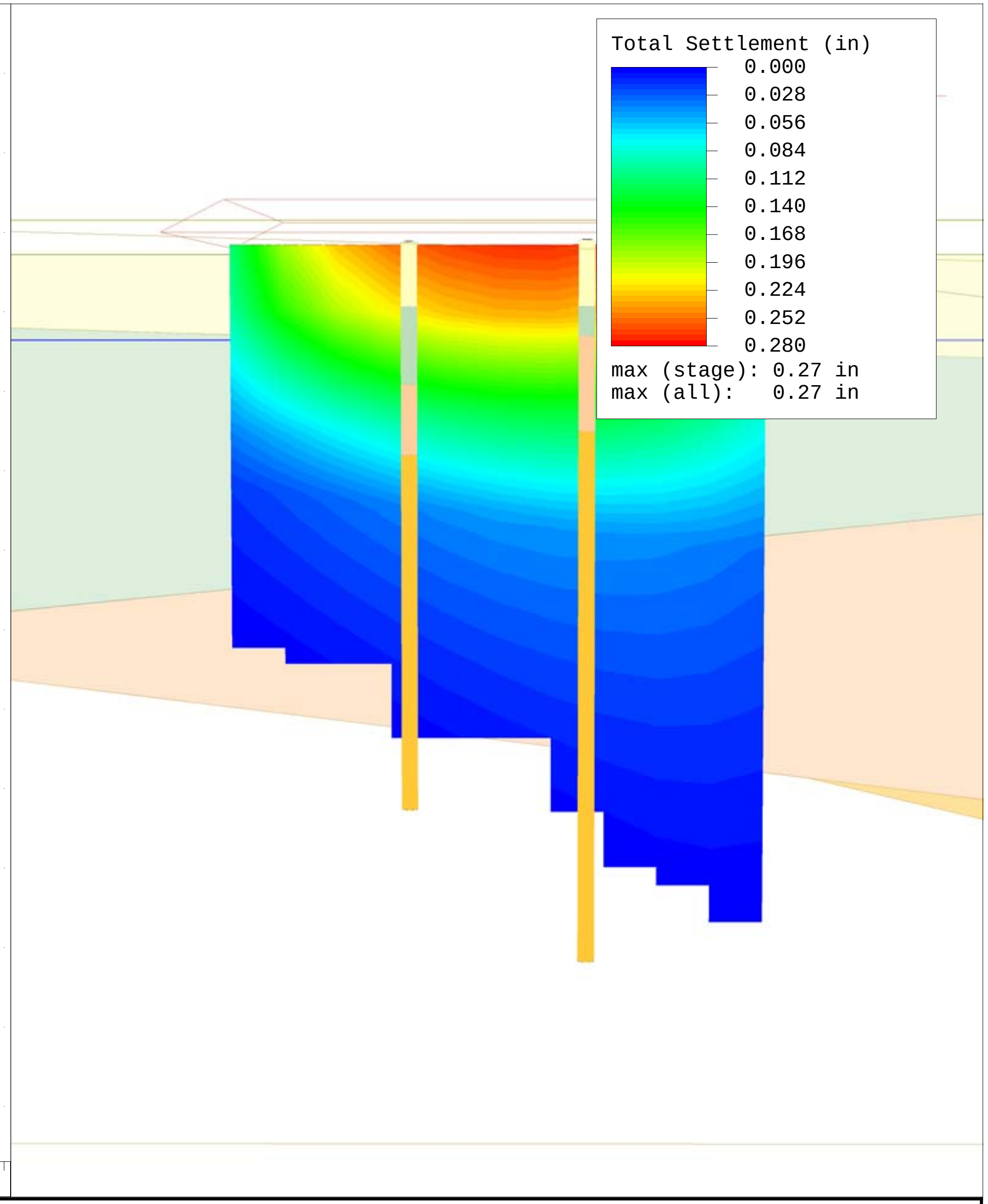
Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

Piezometric Line Entities

ID	Depth (ft)
1	7.3 ft

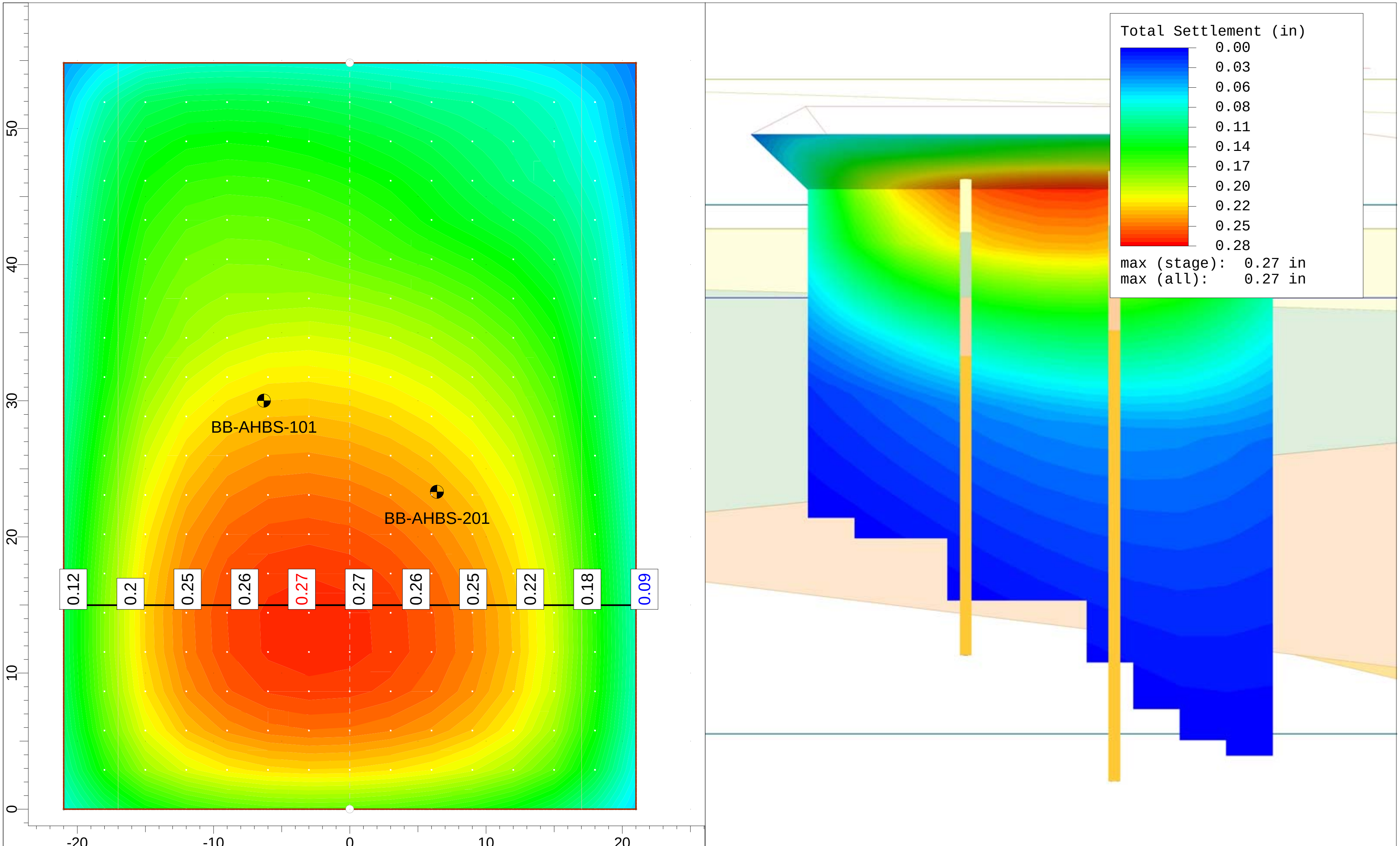


Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Su A (kips/ft2)	Su S	Su m
Dense to Very Dense, FILL		0.121	0.139	430	430	0	0.2	0.8
Very Loose to Dense, SAND		0.108	0.138	198	198	0	0.2	0.8
Soft to Stiff, SILT		0.083	0.114	250	250	0	0.2	0.8
Dense or Hard, Glacial Till		0.134	0.144	783.5	783.5	0	0.2	0.8



Project	20815 Athens-Hartland Corson Corner Br		
Analysis Description	Embankment Settlement		
Drawn By	Brandon Slaven	Company	MaineDOT
Date	9/28/2017, 10:14:18 AM	File Name	Embankment1_Borehole.s3z

SETTLE3D 3.020



Project	20815 Athens-Hartland Corson Corner Br		
Analysis Description	Embankment Settlement		
Drawn By	Brandon Slaven	*Company*	MaineDOT
Date	9/28/2017, 10:14:18 AM	*File Name*	Embankment1_Borehole.s3z

SETTLE3D 3.020

Development of embankment soil model for Settle 3D

OBJECTIVE
Estimate soil parameters for Settle 3D analyses.

Given:
1) Boring logs and lab data

Assumptions:
1) See attached boring logs (BB-AHBS-101 and BB-AHBS-201) for layer delineation developing a 'composite' soil profile based on soil layering and properties encountered.
2) Assume groundwater table is at highest point measured in the borings (7.3' bgs)

Soil Model

Soil Layer Embankment (Gravel Borrow)

Assume material used to construct 2 foot increase in grade is similar to gravel borrow.

Soil Total Unit Weight

MaineDOT BDG Table 3-3
Soil Type 5

$\gamma_e := 135\text{pcf}$

Assume the total (moist) unit weight of Soil Type 4 considers placement at the material's optimum moisture content. Assume optimum moisture content of gravel borrow occurs at 8 percent.

Optimum moisture content

$w_{e\_opt} := .08$

Dry unit weight

$$\gamma_{e\_dry} := \frac{\gamma_e}{1 + w_{e\_opt}}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{e\_dry} = 125 \cdot \text{pcf}$

Saturated Moisture Content

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 - Natural Moisture Content in a saturated state

$w_{e\_sat} := .23$

Saturated Unit Weight

$$\gamma_{e\_saturated} := \gamma_{e\_dry} \cdot \left(1 + w_{e\_sat}\right)$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{e\_saturated} = 154 \cdot \text{pcf}$

Soil Layer No. 1 (Dense to Very Dense, Granular Fill)

Dry Unit Weight

$\gamma_{1dry} := 121\text{pcf}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Dry unit weight, Dense silty sand

Saturated Moisture Content

Natural water content at saturated state:
Dense angular silty sand = 15%

$w_{1sat} := .15$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

Saturated Unit Weight

$\gamma_{1saturated} := \gamma_{1dry} \cdot (1 + w_{1sat})$

$\gamma_{1saturated} = 139 \cdot \text{pcf}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

Design N-value

$N_{60\_1} := 25$

BB-AHBS-201;2D

Young's Modulus

$E_{s\_1} := 600 \cdot (N_{60\_1} + 6) + 2000$

$E_{s\_1} := 20600\text{kPa} = 430 \cdot \text{ksf}$

Bowles Foundation Analysis and Design 5th Ed. p. 316: Table 5-6 for Gravelly SAND with N > 15

Soil Layer No. 2 (Very loose to Dense, Silty SAND)

Soil Dry Unit Weight

Dry Unit Weight of Sands:
Loose uniform sand = 92 pcf
Dense uniform sand = 115 pcf
Loose silty sand = 102 pcf
Dense silty sand = 121 pcf
Average dry unit weight for sands: 108 pcf

$\gamma_{2dry} := 108\text{pcf}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Dry unit weights for sands

Saturated Moisture Content

$w_{2sat} := .28$

Moisture content of sample BB-AHBS-201, Assumed below the water table (Saturated) at time of sampling

Saturated Unit Weight

$\gamma_{2saturated} := \gamma_{2dry} \cdot (1 + w_{2sat})$

$\gamma_{2saturated} = 138 \cdot \text{pcf}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

Design N-value

$N_{60\_2} := 4$

BB-AHBS-201;3D, 4D

Young's Modulus

$E_{s\_2} := 500(N_{60\_2} + 15)$

$E_{s\_2} := 9500\text{kPa} = 198 \cdot \text{ksf}$

Bowles Foundation Analysis and Design 5th Ed. p. 316: Table 5-6 for normally consolidated sand

Soil Layer No. 3 (soft to stiff, SILT)

Specific Gravity

$G_s := 2.61$

Lab test results from samples
BB-AHBS-101; 4D, 5DA

Unit Weight of Water

$\gamma_w := 62.4\text{pcf}$

Saturated Moisture Content

$w_{3sat} := .37$

Average of moisture content of samples
BB-AHBS-101;4D, 5DA, 5DB and BB-AHBS-201;5D

Saturation

$S := 1$

Assume 100% saturation for soils below water table

Dry Unit Weight

$$\gamma_{3dry} := \frac{G_s \cdot \gamma_w}{1 + w_{3sat} \cdot G_s}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{3dry} = 83 \cdot \text{pcf}$

Saturated Unit Weight

$$\gamma_{3saturated} := \gamma_{3dry} \cdot \left(1 + w_{3sat}\right)$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{3saturated} = 114 \cdot \text{pcf}$

Design Shear Strength

$S_{u\_3} := 500\text{psf} = 23.94 \cdot \text{kPa}$

Young's Modulus

$E_{s\_3} := 500 \cdot S_{u\_3}$

Bowles Foundation Analysis and Design 5th Ed. p. 316: Table 5-6 for silt with PI < 30

$E_{s\_3} = 250 \cdot \text{ksf}$

Soil Layer No. 4 (Dense or Hard, Glacial Till)

Dry Unit Weight

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Dry unit weight, Glacial Till

$\gamma_{4dry} := 134pcf$

Saturated Moisture Content

Natural water content at saturated state:
Dense angular silty sand = 19%

Average of moisture content results from samples BB-AHBS-101;8D, BB-AHBS-201; 2D

$w_{1sat} := .19$

Saturated Unit Weight

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{1saturated} := \gamma_{1dry} \cdot (1 + w_{1sat})$

$\gamma_{1saturated} = 144 \cdot pcf$

Young's Modulus

Loose Sand 1450-3625 psi
Dense Sand 7250-11745 psi

Bowles, Foundation Analysis and Design 5th Ed. p. 125: Table 2-8

Average upper limit of loose sand value and lower limit of dense sand value for medium dense sand.

$$E_{s\_4} := \frac{3625psi + 7250psi}{2}$$

$E_{s\_4} = 5437.5\, psi$

$E_{s\_4} = 783 \cdot ksf$

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream

Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-101

WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	264.0	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/23/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+47.2, 6.3 ft Lt.	**Casing ID/OD:**	NW 3"/3.5"	**Water Level*:**	12.6' after completion
Hammer Efficiency Factor:	0.9288	**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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	263.42	7" of Pavement ——0.58	
	1D	24/17	1.00 - 3.00	16/19/20/16	39	60			Brown, moist, very dense, Gravelly SAND, some silt, (Fill).	
	2D	24/15	3.00 - 5.00	6/10/10/7	20	31			2D(A) Similar to above, except dense. Layer 1	
5								259.50	——4.50 2D(B) Brown, moist, dense, Silty SAND, little gravel, (Fill). 3D(A) Similar to above, except medium dense. Layer 2	
	3D	24/16	5.00 - 7.00	5/5/3/2	8	12		258.00	——6.00	
								257.50	3D(B) Brown, moist, medium dense, fine to medium SAND, little silt, trace gravel, (Fill). ——6.50	
	4D	24/14	7.00 - 9.00	2/3/3/4	6	9			3D(C) Brown, moist, stiff, SILT, some sand, trace clay, trace gravel, (Fill). 4D Similar to above.	G#262840 A-4, CL WC=36.7% Non-PLastic
10	5D	24/14	10.00 - 12.00	1/2/6/5	8	12	NW		5D(A) Similar to above, except trace wood fiber.	G#262841 A-4, CL WC=32.6% Non-Plastic
							PUSH	252.50	——11.50 5D(B) Dark brown, moist, medium dense, SAND, some silt, trace gravel, trace clay, trace wood fiber, (Relic Topsoil).	G#262842 A-2-4, SC-SM WC=38.1%
	6D	13/11	12.00 - 13.08	3/6/50-1"	--				6D(A) Dark brown, wet, SILT, some sand, trace gravel, trace clay, trace wood fiber.	G#262843 A-4, CL WC=89.7%
							OPEN	250.90	6D(B) Grey, wet, fine to medium SAND, little silt. Layer 3 ——13.10 Place HW casing to 13.1 ft bgs. Advance by roller cone to 15 ft bgs. GRANODIORITE Boulder from 13.1 to 17.5 ft bgs.	
15	R1	54/30	15.00 - 19.50				NQ2		(Boulder Till). Layer 4	
20	MD	4/0	20.00 - 20.33	100-4"	--		NW		Boulder from 20.4 to 21.5 ft bgs.	
	R2	42/6	21.00 - 24.50				NQ2		Cobbles from 23.2 to 24 ft bgs.	
25							NW	239.50	——24.50	

Remarks:

Auto-hammer #562

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: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-101
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	264.0	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/23/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+47.2, 6.3 ft Lt.	**Casing ID/OD:**	NW 3"/3.5"	**Water Level*:**	12.6' after completion
Hammer Efficiency Factor: 0.9288		**Hammer Type:**	Automatic ☒ Hydraulic ☐	Rope & Cathead ☐	

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	12/7	25.00 - 26.00	23/100	--		NW			Place NW casing with spin cutting shoe. 7D(A) Grey, wet, very dense, Silty SAND, some gravel, trace clay, cobbles. 7D(B) Brown, wet, hard, SILT, little clay, trace fine sand, cobbles, (Glacial Till).	
30	8D	10/6	30.00 - 30.83	44/100-4"	--					Brown, wet, hard, SILT, little sand, trace clay, trace gravel.	G#262844 A-4, CL WC=22.5%
										Layer 4 Continue	
35	9D	5/2	35.00 - 35.42	100-5"	--			229.00		35.00 Brown, wet, hard, Sandy SILT, some gravel, (Glacial Till). Boulder from 36 to 38 ft bgs.	
40	MD	5/0	40.00 - 40.42	100-5"	--					Cobbles from 41 to 44.7 ft bgs.	
45	10D	4/4	45.00 - 45.33	100-4"	--					Brown, wet, hard, Sandy SILT, some gravel, (Glacial Till).	
50											

Remarks:

Auto-hammer #562

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: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-201
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	263.8	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/03/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	7+40.5, 6.4 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	7.3' (after completion)
Hammer Efficiency Factor:	0.873	**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	263.47		4" of Pavement 0.33	
	1D	24/17	1.00 - 3.00	8/10/12/12	22	32				Brown, moist, dense, SAND, some gravel, trace silt, (Fill).	
	2D	24/10	3.00 - 5.00	12/9/8/6	17	25				Similar to above. Layer 1	
5	3D	24/12	5.00 - 7.00	3/2/1/2	3	4				Brown, moist, very loose, SAND, little gravel, little silt, (Fill).	G#271151 A-2-4, SM WC=12.0%
	4D	24/10	7.00 - 9.00	3/2/1/2	3	4				Brown, wet, very loose, Silty SAND, little gravel, trace clay, (Fill). Layer 2	G#271152 A-4, SC-SM WC=28.2% Non-Plastic
10	5D	24/16	10.00 - 12.00	1/1/1/1	2	3	33			Olive, wet, soft, SILT, little clay, trace fine sand, (Reworked Soils).	G#271153 A-4, CL WC=40.9% Non-Plastic
							24				
							31			Layer 3	
							34				
							47			15.00	
15	6D	24/2	15.00 - 17.00	7/6/7/11	13	19	52	248.80		Brown, wet, medium dense, SAND, little gravel, little silt (Boulder Till).	
							53				
							317 OPEN			Boulder from 17.5 to 19.8 ft bgs. Advanced by rollercone through boulder to 20 ft.	
										Layer 4	
20	MD	3/0	20.00 - 20.25	100-3"	- -			242.80		Rollercone bit damaged. Borehole abandoned. Relocate to BB-AHBS-201A. 21.00	
										Bottom of Exploration at 21.00 feet below ground surface.	
25											

Remarks:

Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

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: Corson Corner Bridge #2135 carries Routes 43/151 over Black Stream
Location: Athens-Hartland, Maine

Boring No.: BB-AHBS-201B
WIN: 20815.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	263.8	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/03/2017 - 05/04/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+36.5, 7.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.3' (after completion)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	1D	5/5	25.00 - 25.42	100-5"	- -		SPUN			Brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
										Cobble.	
30	2D	8/6	30.00 - 30.67	32/100-2"						Brown, wet, hard, SILT, some sand, little gravel, trace clay, (Glacial Till). Cobble.	G#271154 A-4, CL WC=16.2%
										Layer 4 Continued	
35											
40	3D	3/1	40.00 - 40.25	100-3"	- -		OPEN	223.50		Similar to above. 40.30 Top of Bedrock at Elev. 223.5 ft. Advanced by rollercone through Bedrock from 40.3 to 43 ft bgs.	
	R1	60/58	43.00 - 48.00	RQD = 62%			NQ2			R1:Bedrock: White, fine to medium grained, GRANODIORITE, fresh, hard, joints are low angle, close, tight. [Hartland Pluton] Rock Mass Quality = Fair. R1:Core Times (min:sec) 43.0-44.0 ft (3:55) 44.0-45.0 ft (4:00) 45.0-46.0 ft (4:49) 46.0-47.0 ft (4:55) 47.0-48.0 ft (2:52) 97% Recovery	
45											
	R2	60/49	48.00 - 53.00	RQD = 13%						R2:Bedrock: Similar to above except open fractures with slight to moderate weathering from 48.0 to 48.5 ft bgs. Rock Mass Quality = Very Poor. R2:Core Times (min:sec)	
50											

Remarks:

Auto-hammer #362
HW casing driven using 140# auto-hammer with 30" drop.

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.

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
262840	**BB-AHBS-101/4D**	**GEOTECHNICAL (DISTURBED)**	**9/23/2016**	**10/7/2016**

Sample Type: **GEOTECHNICAL** Location: Station: **7+47.2** Offset, ft: **6.3 LT** Dbfg, ft: **7.0-9.0**

WIN/Town **020815.00 - ATHENS, HARTLAND** Sampler: **CONSULTANT SW COLE**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**99.0**
⅜ in. [9.5 mm]	**99.0**
¼ in. [6.3 mm]	**98.6**
No. 4 [4.75 mm]	**98.0**
No. 10 [2.00 mm]	**96.1**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**85.3**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**74.0**
[0.0288 mm]	**59.7**
[0.0187 mm]	**55.1**
[0.0115 mm]	**45.9**
[0.0086 mm]	**34.4**
[0.0063 mm]	**27.6**
[0.0033 mm]	**11.4**
[0.0014 mm]	**4.6**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	**NP**
Specific Gravity, Corrected to 20°C (T 100)	**2.61**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**36.7**

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/19/2016**

Paper Copy: Lab File; Project File; Geotech File

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
262841	BB-AHBS-101/5DA	GEOTECHNICAL (DISTURBED)	9/23/2016	10/7/2016

Sample Type: **GEOTECHNICAL** Location: Station: **7+47.2** Offset, ft: **6.3** **LT** Dbfg, ft: **10.0-11.5**

WIN/Town **020815.00 - ATHENS, HARTLAND** Sampler: **CONSULTANT SW COLE**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	100.0
¼ in. [6.3 mm]	99.5
No. 4 [4.75 mm]	98.5
No. 10 [2.00 mm]	98.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	90.4
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	65.5
[0.0314 mm]	51.3
[0.0213 mm]	35.9
[0.0127 mm]	28.1
[0.0092 mm]	23.0
[0.0066 mm]	20.5
[0.0034 mm]	7.6
[0.0014 mm]	5.1

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.61
Loss on Ignition, % (T 267)	
Water Content (T 265), %	32.6

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/20/2016**

Paper Copy: Lab File; Project File; Geotech File

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d	
			lb/ft^3	**kN/m^3**
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

TABLE 5-6
Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$\boxed{E_s = 500(N + 15)}$ $= 7000\sqrt{N}$ $= 6000N$ — — — ‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ *$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$ $E_{s(\mathrm{OCR})} \approx E_{s,\mathrm{nc}}\sqrt{\mathrm{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $\boxed{= 600(N + 6) \quad N \le 15}$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $\quad$ \$$E'_s = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E'_s = 4q_c + 5000$ where $E'_s = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

4. It is not easy to determine if a cohesionless deposit is overconsolidated or what the OCR might be. Cementation may be less difficult to discover, particularly if during drilling or excavation sand "lumps" are present. Carefully done consolidation tests will aid in obtaining the OCR of cohesive deposits as noted in Chap. 2.

In general, with an OCR > 1 you should carefully ascertain the site conditions that will prevail at the time settlement becomes the design concern. This evaluation is, of course, true for any site, but particularly so if OCR > 1.

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

5-9.1 Effects on Settlements

A major problem in foundation design is to proportion the footings and/or contact pressure so that settlements between adjacent footings are nearly equal. Figure 5-9 illustrates the problem

TABLE 5-6
Equations for stress-strain modulus E_s by several test methods (*continued*)

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil		
Use the undrained shear strength s_u in units of s_u		
Clay and silt	$I_P > 30$ or *organic*	$E_s = (100 \text{ to } 500)s_u$
Silty or sandy clay	$I_P < 30$ or *stiff*	$\boxed{E_s = (500 \text{ to } 1500)s_u}$ Again, $E_{s,\mathrm{OCR}} \approx E_{s\,\mathrm{nc}}\sqrt{\mathrm{OCR}}$ Use smaller s_u-coefficient for highly plastic clay.

Of general application in clays is

$$E_s = Ks_u \qquad \text{(units of } s_u) \tag{a}$$

where K is defined as

$$K = 4200 - 142.54I_P + 1.73I_P^2 - 0.0071I_P^3 \tag{b}$$

and I_P = plasticity index in **percent.** Use $20\% \le I_P \le 100\%$ and round K to the nearest multiple of 10.

Another equation of general application is

$$E_s = 9400 - 8900I_P + 11\,600I_c - 8800S \qquad \text{(kPa)}$$
$$I_P, I_c, S = \text{previously defined above and/or in Chap. 2} \tag{c}$$

*Vesić (1970).

†Author's equation from plot of D'Appolonia et al. (1970).

‡USSR (may not be standard blow count N).

¶Japanese Design Standards (lower value for structures).

§Senneset et al. (1988)

General sources: First European Conference on Standard Penetration Testing (1974), vol. 2.1, pp. 150–151; *CGJ*, November 1983, pp. 726–737; *Use of In Situ Tests in Geotechnical Engineering*, ASCE (1986), p. 1173; Mitchell and Gardner (1975); *Penetration Testing (Second European Conference)* (1982), vol. 1, p. 160; 11th ICSMFE (1985), vol. 2, pp. 462, 765; vol. 4, p. 2185; *International Symposium on Penetration Testing* (1988), 2 vols.

Notes:

1. For q_c generally use (2.5 to 3)q_c for normally consolidated sand and about 4 to 6 q_c for overconsolidated sand.
2. Can use Eqs. (*a*) and (*b*) above for all clay. They are particularly applicable for OCR > 1. Probably should use both Eqs. (*a*) and (*c*), and if results differ significantly either use an average or compute another E_s using a different equation.
3. For sands try to use more than one equation or else use one of the equations and compare the computed E_s to published table (see Table 2-8) values.
4. For silts use any of the above equations, but if the equations are given for sand use smaller coefficients.
5. For sand, using $E_s = 250$ or $500(N + 15)$ may give a modulus that is too small (but conservative). Suggest when you use equations of this form you compute E_s by one or more additional equations and average the results.
6. Note: Using $\sqrt{\mathrm{OCR}}$ is the same as $(\mathrm{OCR})^{1/2}$, so that exponent $n = 0.5$. You can use other values for the exponent from about 0.3 to 0.5. However, since all the equations for E_s are approximations the use of $n = 0.5$ is sufficiently accurate unless you have good-quality field or laboratory test values.

Bowles, Foundation Analysis and Design 5th Ed. p. 125

TABLE 2-8

Value range* for the static stress-strain modulus E_s for selected soils (see also Table 5-6)

Field values depend on stress history, water content, density, and age of deposit

1 Mpa = 145 psi

Soil	E_s, MPa	Es, psi
Clay		
Very soft	2–15	
Soft	5–25	
Medium	15–50	
Hard	50–100	
Sandy	25–250	
Glacial till		
Loose	10–150	
Dense	150–720	
Very dense	500–1440	
Loess	15–60	
Sand		
Silty	5–20	725 - 2900
Loose	10–25	1450 - 3625
Dense	50–81	7250 - 11745
Sand and gravel		
Loose	50–150	7250 - 21750
Dense	100–200	14500 - 29000
Shale	150–5000	
Silt	2–20	

*Value range is too large to use an "average" value for design.

in situ, it is reasonable for confined compression tests to produce better "elastic" parameters. Although it is difficult to compare laboratory and field E_s values, there is some evidence that field values are often four to five times larger than laboratory values from the unconfined compression test. For this reason, current practice tends to try to obtain "field" values from in situ testing whenever possible. This topic will be taken up in more detail in the next chapter.

Table 2-8 gives a range of E_s values that might be obtained. Note that the range is very large, owing to the foregoing factors as well as those factors given on the table. With this wide range of values the reader should not try to use "averaged" values from this table for design.

If laboratory test plots similar to Fig. 2-43a are used, it is most common to use the initial tangent modulus to compute the stress-strain modulus E_s for the following reasons:

1. Soil is elastic only near the origin.
2. There is less divergence between all plots in this region.
3. The largest values are obtained—often three to five times larger than a tangent or secant modulus from another point along the curve.

Frost

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG Section 5.2.1.

From Design Freezing Index Map: **Athens, Maine**
DFI = 1850 degree-days.
Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 1800 $d1 := 74.5$

For DFI = 1900 $d2 := 76.6$

$$d := in \cdot \left(\frac{d2 - d1}{10} \cdot 2 + d1 \right)$$

Depth of Frost Penetration $d = 75 \cdot in$ $d = 6.2 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Madison lies along the same Maine Design Freezing Index contour - use Madison data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Madison, Maine
Air Design Freezing Index = 1847 F-days
N-Factor = 0.70
Surface Design Freezing Index = 1293 F-days
Mean Annual Temperature = 42.4 deg F
Design Length of Freezing Season = 136 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Asphalt	5.0	.1	144.0	28	28	.9	.9	0
1-Coarse	67.3	20.0	125.0	28	34	3.8	1.9	3,600

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic f

Total Depth of Frost Penetration = 5.63 ft = 67.6 in.

Recommendation: 6.2 feet for design of foundations constructed on coarse grained soils

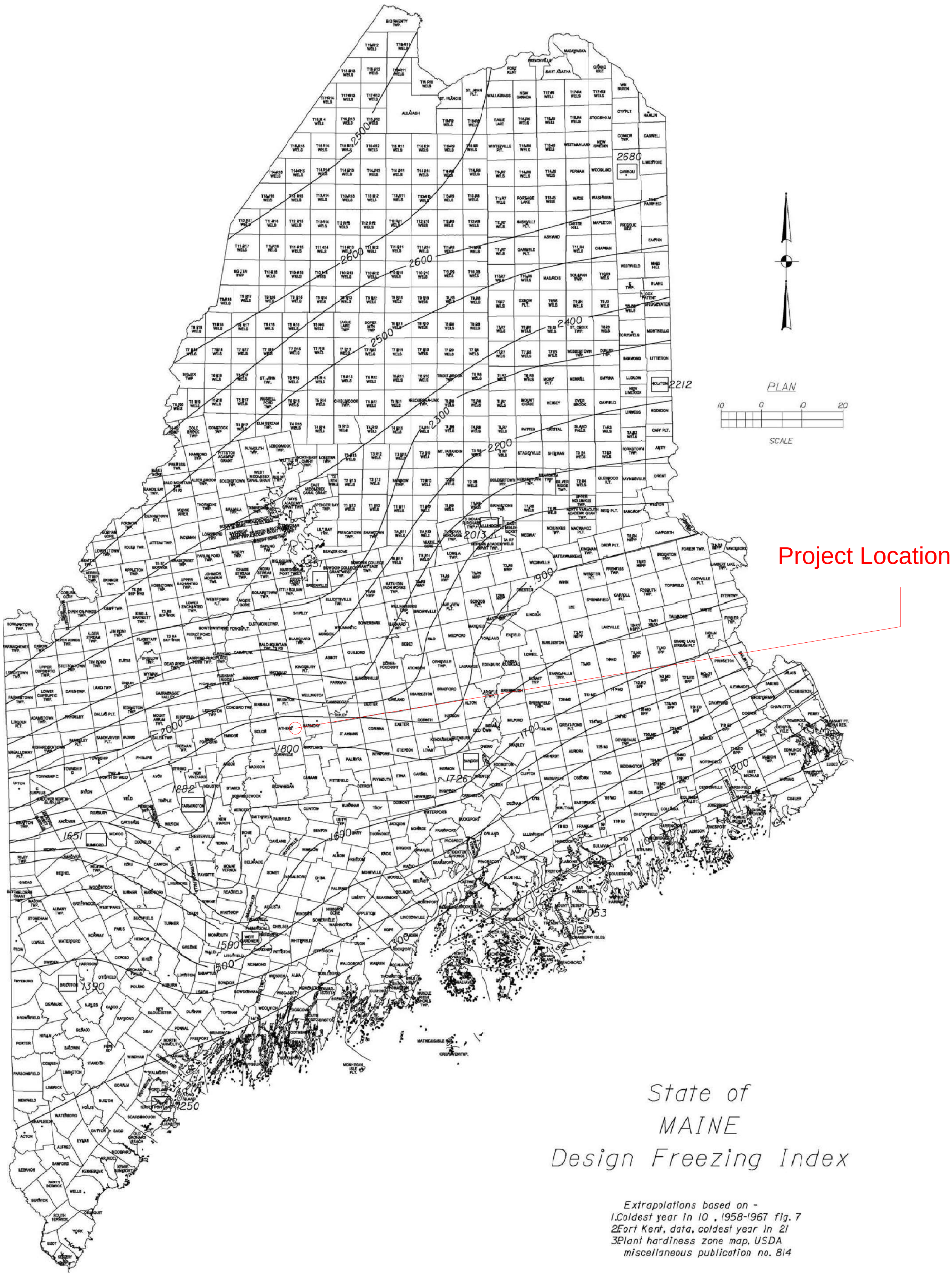


Figure 5-1 Maine Design Freezing Index Map

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0