MAINE DEPARTMENT OF TRANSPORTATION
HIGHWAY PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

LARGE CULVERT #1013530
FOREST AVENUE
ORONO, MAINE

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STATE OF MAINE
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Penobscot County
WIN 27194.00

Soils Report 2026-42
June 24, 2026

Additional review of this report provided by S.W.Cole, see stamped memo dated June 2026.

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Appendices

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical design and construction recommendations for the replacement of existing twin culverts (#1013530) which carries Forest Avenue over Great Brook in Orono, Maine. The existing structure consists of dual 36-inch diameter, approximately 58-foot-long corrugated metal pipe (CMP) culverts. No original construction plans for the existing structure were available for review. The CMPs are in poor condition and need replacement both from an infrastructure and environmental standpoint. The culvert is located approximately 0.05 of a mile west of Essex Street as shown in the attached Location Map. Forest Avenue is a Highway Corridor Priority 4 road.

The proposed replacement structure will be an approximately 8-foot span by 6-foot rise by 72-foot-long precast concrete box culvert. The box culvert shall have 2-foot deep toe walls at the inlet and outlet. The invert of the proposed culvert is approximately 8.9 feet below the existing road grade at the roadway centerline. The box culvert invert will be embedded 2 feet into the streambed. The roadway embankment slopes at the proposed culvert inlet and outlet shall be no steeper than 1.75H:1V to protect against erosion. The proposed roadway profile includes a grade raise of approximately 2 feet along the centerline and up to 4 feet in areas of roadway widening.

The new box culvert will be located on horizontal and vertical alignments that will approximately match existing alignments. The roadway will be closed during construction to facilitate removal of the existing culvert and installation of the proposed box culvert, and traffic will be detoured onto state and local roads.

2.0 SUBSURFACE INVESTIGATION

One (1) boring (HB-ORO-101) and one (1) probe (HB-ORO-102) were drilled for this project on October 23, 2023 by the MaineDOT drill crew using a trailer mounted drill rig. Boring HB-ORO-101 was drilled behind the existing culvert at the northwest corner. Probe HB-ORO-102 was drilled behind the existing culvert at the southwest corner. Boring and probe were terminated at depths of 27 feet and 20 feet, respectively. Exploration locations are shown on the attached Boring Location Plan. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on the attached Boring Logs.

Boring HB-ORO-101 was drilled using solid stem auger, cased wash boring, and open hole techniques. Soil samples were 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 MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in November 2022. All N-values discussed in this report are corrected values (N_{60}) computed by applying an average energy transfer factor of 0.906 to the raw field N-values. This hammer efficiency factor (0.906) and both the raw field N-value and corrected N-value (N60) are shown on the attached Boring Logs. Probe HB-ORO-102 was drilled using solid stem auger techniques.

No soil samples were obtained in the probe. The boring and probe were not advanced to refusal, and bedrock was not cored.

The MaineDOT Geotechnical Team member selected the boring and probe locations, drilling methods, designated type and depth of sampling, reviewed field logs for accuracy and identified field and laboratory testing requirements. An experienced Northeast Transportation Training and Certification Program (NETTCP) certified subsurface inspector logged the subsurface conditions encountered. The boring and probes were located in the field by taping to surveyed site features after completion of the drilling program.

3.0 LABORATORY TESTING

A laboratory testing program was conducted to assist in soil classification, evaluation of engineering properties of the soils and geologic assessment of the project site. Laboratory testing consisted of three (3) standard grain size analyses with natural water content, two (2) standard grain size analyses with hydrometer and natural water content, and two (2) Atterberg Limits tests. The results of the laboratory testing program are discussed in the following section and are shown in the attached Boring Logs, Laboratory Testing Summary Sheet, Grain Size Distribution Curve Sheet, and Atterberg Limits Plots.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test boring and probe generally consisted of fill, underlain by Presumpscot Formation, underlain by glacial till. An interpretive subsurface profile depicting the generalized soil stratigraphy at the boring location is shown on the attached Interpretive Subsurface Profile.

Boring HB-ORO-101 and probe HB-ORO-102 were drilled to depths of approximately 27.0 feet and 20.0 feet below ground surface (bgs), respectively, without encountering a refusal surface.

The table below summarizes the field and laboratory information obtained in boring HB-ORO-101:

Approx. Depth BGS[1] (feet)	Soil Description	AASHTO[2] Classification	USCS[3]	WC%[4]
0.0 – 9.5	Fill: Brown, moist, fine to coarse sand, some gravel, little silt. Brown, wet, gravelly fine to coarse sand, little silt, organics.	A-1-b A-1-b	SM SW-SM	6.3 18.2
9.5 – 24.5	Presumpscot Formation: Grey, wet, silt, some clay, trace fine sand, roots. Grey, wet, clayey silt, trace fine sand.	A-6 or A-4	CL	34.3 to 37.1

24.5 – >27.0	Glacial Till: Grey brown, gravelly fine to coarse sand, little silt, weathered rock.	A-1-a	SM	10.0

[1]BGS = below ground surface
[2]AASHTO = American Association of State Highway and Transportation Officials
[3]USCS = Unified Soil Classification System
[4]WC% = Water content in percent

Two (2) N_{60}-values obtained in the fill were 24 blows per foot (bpf) and 26 bpf, indicating that the fill is medium dense. Three (3) sets of vane shear tests (VSTs) were completed in the Presumpscot Formation using Geonor rectangular vanes. Where completed, these tests measured undisturbed undrained shear strengths ranging from 446 psf to 1138 psf, indicating a soft to stiff consistency. The remolded shear strengths ranged from about 67 psf to 268 psf. Based on the ratio of undrained to remolded shear strength from the vane shear test, the Presumpscot Formation was determined to have sensitivities ranging from approximately 3.2 to 6.7 and are classified as moderately sensitive to sensitive (Bjerrum, 1954). One (1) N_{60}-value obtained in the glacial till was 54 bpf, indicating that the till is very dense.

The following table summarizes the results of Atterberg Limits tests done on two (2) samples of the silt and clayey silt:

Boring No. and Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
HB-ORO-101 3D	37.1	36	24	12	1.09
HB-ORO-101 4D	34.3	31	21	10	1.33

The plasticity indices of the samples indicate that the silt and clayey silt have medium plasticity (Burmister, 1949). The natural water content is greater than the liquid limit resulting in liquidity indices greater than 1. Based on Bowles, 2016, the index tests indicate that these samples are on the verge of being a viscous liquid if disturbed. Soils with a liquidity index greater than 1 have a high potential to remold and lose strength and structure when disturbed. Liquidity index values greater than or equal to 1 are also indicative of soils that are unconsolidated and are commonly referred to as "quick."

Groundwater was recorded at a depth 3.0 feet bgs in boring HB-ORO-101. Groundwater levels can be expected to fluctuate subject to seasonal variations, local soil conditions, topography, precipitation, and construction activity.

5.0 FOUNDATION ALTERNATIVE

A precast concrete box culvert was the only culvert replacement alternative considered for this project. A precast concrete box culvert satisfies the purpose and need of this project because of the structure’s durability, ease and speed of construction, and economic advantages.

6.0 GEOTECHNICAL DESIGN AND CONSTRUCTION RECOMMENDATIONS

The following sections discuss geotechnical recommendations for the design and construction of the proposed culvert. Geotechnical engineering evaluations were made in accordance with 2024 AASHTO LRFD Bridge Design Specifications 10th Edition (LRFD) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through June 2018 (BGD).

6.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of an 8-foot span by 6-foot rise by 72-foot-long precast concrete box culvert. To prevent undermining, the box culvert will have 2-foot inlet and outlet toe walls (cutoff walls) embedded at or below the bottom of riprap aprons and culvert support materials. 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.

Due to the cohesive material encountered at the bearing level, the proposed structure shall be bedded on a 2-foot thick mat of compacted Culvert Bedding Stone (Pay Item 203.55) that is reinforced with geogrid and underlain by stabilization/reinforcement geotextile. The stabilization/reinforcement geotextile should be hand-deployed on the prepared soil subgrade prior to installing the culvert bedding stone. The culvert bedding stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type C Underdrain Backfill material. The Culvert Bedding Stone shall be placed in maximum 6-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor.

The geogrid reinforcement shall meet the requirements of Special Provision 620, attached. The geotextile shall meet Class 1 Stabilization/Reinforcement Geotextile requirements in accordance with MaineDOT Standard Specification 722.01. Adjoining sections of the stabilization geotextile should be overlapped by a minimum of 1 foot.

The soil envelope and backfill for the box culvert 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. To limit future settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

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 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 Load Resistance and Factor Design Bridge Design Specifications, 10th Edition 2024 (LRFD).

The loading specified for the design of the box shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased 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 earth pressures from the soil envelope. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf.

6.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will not be included in the culvert design.

6.1.2 Precast Concrete Box Inlet and Outlet Walls

The precast concrete box culvert's base, ceiling, and full height outlet and inlet walls will extend the full length of the culvert. The left and right outlet walls are integral to the precast culvert and will share the same precast base slab. The full height inlet and outlet walls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The inlet and outlet walls shall be designed to resist lateral earth pressures, vehicular loads and forces resulting from creep, temperature and shrinkage deformations of the concrete box culvert. The inlet and outlet walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) of 2.0 feet per LRFD Article 3.11.6.4. 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.

The inlet and outlet walls are fixed to the box culvert for the entire length, and should be designed to resist movement using an at-rest earth pressure coefficient, Ko, of 0.47.

6.1.3 Precast Concrete Inlet and outlet Toe Walls

Toe walls (cutoff 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 and should bear on natural Silt/Clayey Silt soils at/below the bottom of the Culvert Bedding Stone and riprap apron materials.

6.1.4 Bearing Resistance

To provide a stable subgrade, it is recommended that the precast concrete box culvert be underlain by a 2-foot-thick mat of compacted Culvert Bedding Stone, reinforced with geogrid and underlain by stabilization/reinforcement geotextile placed on the native soil subgrade.

The factored bearing resistance for the precast concrete box culvert bearing on compacted granular bedding material placed on native soils at the service and strength limit states are presented in the table below. Supporting calculations in accordance with LRFD are provided in the attached Calculations.

Limit State	Resistance Factor φ_b	AASHTO LRFD Reference	Factored Bearing Resistance (ksf)
Service	1.0	Article 10.5.5.1	3.0
Strength	0.5	Table 10.5.5.2.2-1	1.5

6.1.5 Modulus of Subgrade Reaction

A modulus of subgrade reaction (k_s) equal to 25 pounds per cubic inch shall be used for the structural design of the box culvert's base slab. Calculations are included in the attached Calculations.

6.2 Settlement

The proposed box culvert will bear on a deposit of soft to stiff Presumpscot Formation. Based on the proposed construction and grade raise, the soil deposits will undergo immediate and consolidation settlement in response to a net increase of vertical overburden pressure. The change in effective stress includes an additional 250 psf beneath the road from the grade raise fill resulting in a net stress increase of 58 psf below the proposed 11-foot wide proposed culvert. A combined elastic and consolidation settlement on the order of 1.6-inch is estimated within the first year of construction. An additional 1.9-inches of long-term settlement (consolidation and secondary) is anticipated over the remaining 50-year service life.

6.3 Frost Protection

Foundations placed on the fill or native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Orono has a design freezing index (DFI) of approximately 1750 F-degree days. A water content of 10 percent was used for fine-grained soils. These components correlate to a frost depth of 5.3 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 5.3 feet for frost protection. However, there are no requirements for embedment of precast concrete boxes at stream crossings in the BDG. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

6.4 Scour and Riprap

Where required, slopes shall be armored with riprap conforming to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile conforming to MaineDOT Standard Specification 722.03 – 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 toe of the riprap sections shall be constructed 1-foot below the streambed elevation. The riprap slopes shall be constructed no steeper than 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

6.5 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.

7.0 CONSTRUCTION CONSIDERATIONS

Excavations for culvert foundations are anticipated to extend approximately 11.0 feet below existing grades. Sheet-pile-supported and/or open cut excavation techniques are anticipated to be suitable for this project. If temporary sheet piling is used, on the order of 1 inch of additional settlement would be anticipated due disturbance of the underlying Silt/Clayey Silt when sheet piles are removed. Therefore, the removal of temporary sheeting should occur as soon as practical following culvert construction.

Damming and diversion and/or temporary dewatering are anticipated to be necessary to control groundwater and/or river inflow in excavations. Depending on permitting and water levels at the time of construction, we anticipate that it would be possible to dam the stream with sandbags and an impermeable membrane, and temporarily divert the flow through a pipe so the contractor can construct foundations in the dry. It may also be necessary to employ localized pumping from sumps to maintain dewatering. It is anticipated that inflow of surface water or runoff to excavations can be handled by open pumping from sumps installed at the bottoms of excavations. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping. Dewatering discharge should be managed in accordance with the contractor's Stormwater Prevention Plan and MaineDOT Best Management Practices. Regardless of the method of excavation, all excavations and earth support systems shall meet all applicable OSHA regulations.

The soils encountered in the boring at the box subgrade elevation generally consisted of stiff Silt/Clayey Silt becoming soft with depth. Any unsuitable soils (i.e. disturbed or remolded silt and clay, organic material) encountered at the subgrade elevation should be excavated to expose competent, firm material and replaced with compacted granular borrow.

The saturated Silt/Clayey Silt at the box culvert bearing elevation may easily become disturbed by construction activities.

The following items are recommended to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to limit disturbance and rutting of the Silt/Clayey Silt and to allow the bearing pad construction in the dry. Cofferdams may be required to divert flow away from the new culvert location during construction;
- The contractor shall not operate heavy equipment over the excavated subgrade to minimize subgrade disturbance;
- Limit vibration-induced disturbance to limit the risk of excavation bottom heave;

- Use of a smooth-edged bucket and careful grade control are recommended to avoid over excavation and/or disturbance of the subgrade; and
- Hand-deploy the stabilization/reinforcement geotextile on the prepared soil subgrade prior to installing the Culvert Bedding Stone.

The Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary construction traffic. All loose and unsuitable material encountered at the bearing elevation shall be removed and replaced with compacted Granular Borrow – Material for Underwater Backfill. If disturbance and rutting occur, the Contractor shall remove and replace the disturbed materials with compacted Granular Borrow – Material for Underwater Backfill.

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.

Using the excavated native soils as backfill around the culvert shall not be permitted. The native soils may only be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703.

The Contractor will have to excavate the existing subbase and subgrade fill soils in the vicinity of the culvert. These materials should not be used to re-base the roadway. Excavated subbase sand and gravel may be used as fill below roadway subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Highway Program for specific application to the proposed replacement of an existing large culvert (#1013530) under Forest Avenue in Orono, 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 a limited subsurface investigation at discrete exploratory location 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.

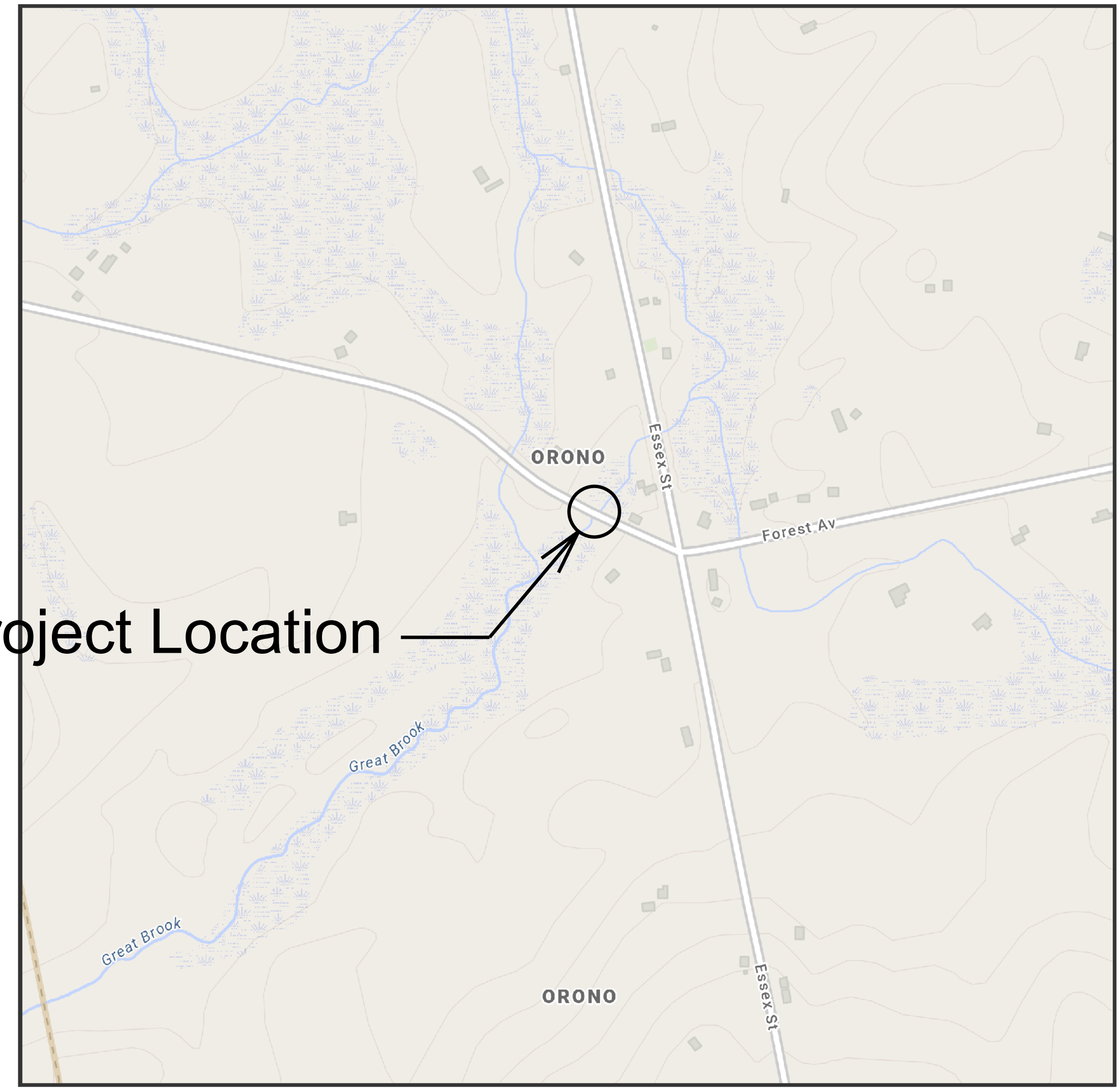
It is recommended that a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

Sheets

Date:5/1/2026

Username: Terry.White

ORONO, MAINE



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0.1 Miles

1 inch =0.14 miles

Date: 5/1/2026
Time: 8:42:34 AM

SHEET NUMBER	ORONO FOREST AVENUE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
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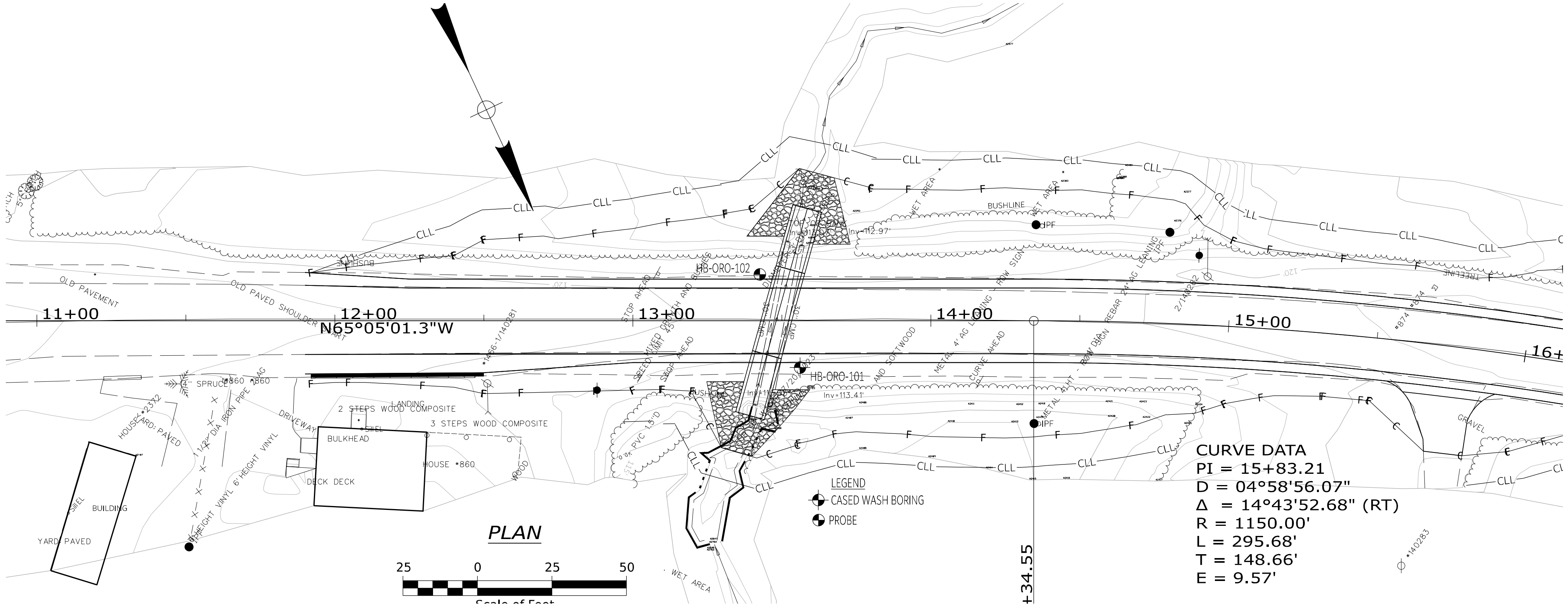
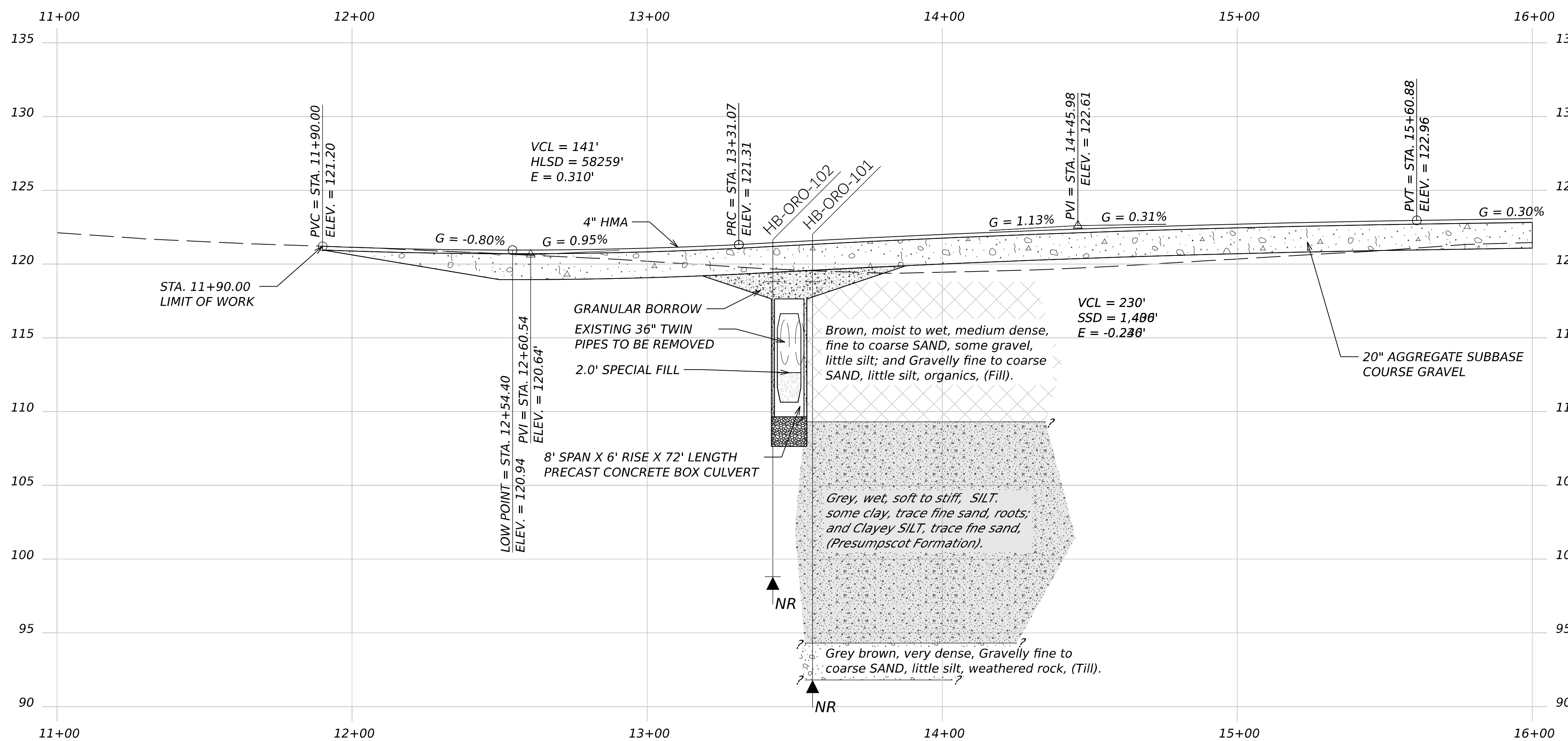
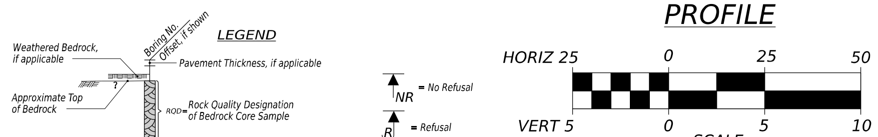
OF 2	LOCATION MAP	WIN 27194.00 HIGHWAY PLANS







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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

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

Project: Large Culvert Replacement on Orono Road
Location: Orono, Maine
Boring No.: HB-ORO-101
WIN: 27194.00

Driller:	MaineDOT	Elevation (ft.)	118.8	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Andrle	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/23/2023; 08:00-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	13+56.1, 15.7 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	3.0 ft bgs.
Hammer Efficiency Factor: 0.906		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	24/19	0.00 - 2.00	4/8/8/10	16	24	SSA		Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#379685 A-1-b, SM WC=6.3%
5									Cobble from 5.0-5.0 ft bgs.	
	2D	24/9	5.50 - 7.50	5/8/9/9	17	26	40		Brown, wet, medium dense, Gravelly fine to coarse SAND, little silt with organics, (Fill).	G#379686 A-1-b, SW-SM WC=18.2%
							19			
							33			
							30			
							24	109.3	9.5 Grey, wet, medium stiff to stiff, SILT, some clay, trace fine sand with roots, (Presumpscot Formation).	
10	3D	24/24	10.00 - 12.00	2/2/1/2	3	5	OPEN HOLE			G#379687 A-6, CL WC=37.1% LL=36 PL=24 PI=12
	V1		12.63 - 13.00	Su=1138/268 psf					55x110 mm vane raw torque readings: V1: 25.5/6.0 ft-lbs.	
	V2		13.63 - 14.00	Su=714/156 psf					V2: 16.0/3.5 ft-lbs.	
15	4D V3	24/24	15.00 - 17.00 15.63 - 16.00	WOH/WOH/WOH/WOH Su=446/67 psf	---			103.8	15.0 Grey, wet, soft, Clayey SILT, trace fine sand, (Presumpscot Formation). 55x110 mm vane raw torque readings: V3: 10.0/1.5 ft-lbs.	G#379688 A-4, CL WC=34.3% LL=31 PL=21 PI=10
	V4		16.63 - 17.00	Su=491/89 psf					V4: 11.0/2.0 ft-lbs.	
20	5D V5	24/20	20.00 - 22.00 20.63 - 21.00	WOR/WOH/WOR/WOH Su=446/112 psf	---				Grey, wet, soft to medium stiff, Clayey SILT, trace fine sand, (Presumpscot Formation). 55x110 mm vane raw torque readings: V5: 10.0/2.5 ft-lbs.	
	V6		21.63 - 22.00	Su=714/223 psf					V6: 16.0/5.0 ft-lbs.	
25								94.3	24.5 Grey brown, very dense, Gravelly fine to coarse SAND, little silt, (Till). Possible weathered rock in tip of spoon.	
	6D	24/18	25.00 - 27.00	15/15/21/25	36	54				G#379689 A-1-a, SM WC=10.0%
								91.8	27.0 Bottom of Exploration at 27.0 feet below ground surface. NO REFUSAL	
30										

Remarks:

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.

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Boring No.: HB-ORO-101

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

Project: Large Culvert Replacement on Orono Road
Location: Orono, Maine
Boring No.: HB-ORO-102
WIN: 27194.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	118.8	Auger ID/OD:	5" Dia.
Operator:	Daggett/Andrle	Datum:	NAVD88	Sampler:	N/A
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	10/23/2023-10/23/2023	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	13+42.6, 15.5 ft Lt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions:
D = Split Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA		Probe, no material samples taken.	
5									
10									
15									
20							98.8	20.0 Bottom of Exploration at 20.0 feet below ground surface. NO REFUSAL	
25									

Remarks:

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.: HB-ORO-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
27194.00
WIN 27194.00
HIGHWAY PLANS

		BY	DATE
PROJ. MANAGER	L. ROWE		
DESIGN-DETAILED			
CHECKED-REVIEWED			
DESIGN2-DETAILED2	YT LEE	T.WHITE	MAY 2026
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

ORONO
FOREST AVENUE
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE WITH BORING LOGS

SHEET NUMBER
2
OF 2

Username: Yueh-Ti.Lee
Date: 6/24/2026

Filename: C:\Users\Yueh-Ti.Lee\State of Maine\MaineDOT Work - 27194_00\GEOTECH\BLP&ISP wBL1.dgn

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_{60}-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_{60}-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):

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

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

Rock Quality Based on RQD

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

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

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

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

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: Large Culvert Replacement on Orono Road
Location: Orono, Maine

Boring No.: HB-ORO-101
WIN: 27194.00

Driller:	MaineDOT	**Elevation (ft.)**	118.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/23/2023; 08:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	13+56.1, 15.7 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	3.0 ft bgs.
Hammer Efficiency Factor: 0.906		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/19	0.00 - 2.00	4/8/8/10	16	24	SSA			Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#379685 A-1-b, SM WC=6.3%
5	2D	24/9	5.50 - 7.50	5/8/9/9	17	26	40			Cobble from 5.0-5.0 ft bgs. Brown, wet, medium dense, Gravelly fine to coarse SAND, little silt with organics, (Fill).	G#379686 A-1-b, SW-SM WC=18.2%
							19				
							33				
							30				
							24	109.3		9.5	
10	3D	24/24	10.00 - 12.00	2/2/1/2	3	5	OPEN HOLE			Grey, wet, medium stiff to stiff, SILT, some clay, trace fine sand with roots, (Presumpscot Formation).	G#379687 A-6, CL WC=37.1% LL=36 PL=24 PI=12
	V1		12.63 - 13.00	Su=1138/268 psf						55x110 mm vane raw torque readings: V1: 25.5/6.0 ft-lbs.	
	V2		13.63 - 14.00	Su=714/156 psf						V2: 16.0/3.5 ft-lbs.	
15	4D	24/24	15.00 - 17.00	WOH/WOH/WOH/ WOH	---			103.8		15.0 Grey, wet, soft, Clayey SILT, trace fine sand, (Presumpscot Formation). 55x110 mm vane raw torque readings: V3: 10.0/1.5 ft-lbs. V4: 11.0/2.0 ft-lbs.	G#379688 A-4, CL WC=34.3% LL=31 PL=21 PI=10
	V3		15.63 - 16.00	Su=446/67 psf							
	V4		16.63 - 17.00	Su=491/89 psf							
20	5D	24/20	20.00 - 22.00	WOR/WOH/WOR/ WOH	---					Grey, wet, soft to medium stiff, Clayey SILT, trace fine sand, (Presumpscot Formation). 55x110 mm vane raw torque readings: V5: 10.0/2.5 ft-lbs. V6: 16.0/5.0 ft-lbs.	
	V5		20.63 - 21.00	Su=446/112 psf							
	V6		21.63 - 22.00	Su=714/223 psf							
25								94.3		24.5	

Remarks:

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: Large Culvert Replacement on Orono Road Location: Orono, Maine	Boring No.: HB-ORO-101 WIN: 27194.00

Driller:	MaineDOT	**Elevation (ft.)**	118.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/23/2023; 08:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	13+56.1, 15.7 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	3.0 ft bgs.
Hammer Efficiency Factor:	0.906	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/18	25.00 - 27.00	15/15/21/25	36	54				Grey brown, very dense, Gravelly fine to coarse SAND, little silt, (Till). Possible weathered rock in tip of spoon.	G#379689 A-1-a, SM WC=10.0%
								91.8		27.0	
										Bottom of Exploration at 27.0 feet below ground surface. NO REFUSAL	
30											
35											
40											
45											
50											

Remarks:

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: Large Culvert Replacement on Orono Road
Location: Orono, Maine

Boring No.: HB-ORO-102
WIN: 27194.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	118.8	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	10/23/2023-10/23/2023	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	13+42.6, 15.5 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample Information									
0						SSA			Probe, no material samples taken.	
5										
10										
15										
20							98.8		20.0 **Bottom of Exploration at 20.0 feet below ground surface.** NO REFUSAL	
25										

Remarks:

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): Orono Work Number: 27194.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
HB-ORO-101, 1D	13+56.1	15.7 Rt.	0.0-2.0	379685	1	6.3			SM	A-1-b	II
HB-ORO-101, 2D	13+56.1	15.7 Rt.	5.5-7.5	379686	1	18.2			SW-SM	A-1-b	0
HB-ORO-101, 3D	13+56.1	15.7 Rt.	10.0-12.0	379687	1	37.1	36	12	CL	A-6	III
HB-ORO-101, 4D	13+56.1	15.7 Rt.	15.0-17.0	379688	1	34.3	31	10	CL	A-4	IV
HB-ORO-101, 6D	13+56.1	15.7 Rt.	25.0-27.0	379689	1	10.0			SM	A-1-a	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.

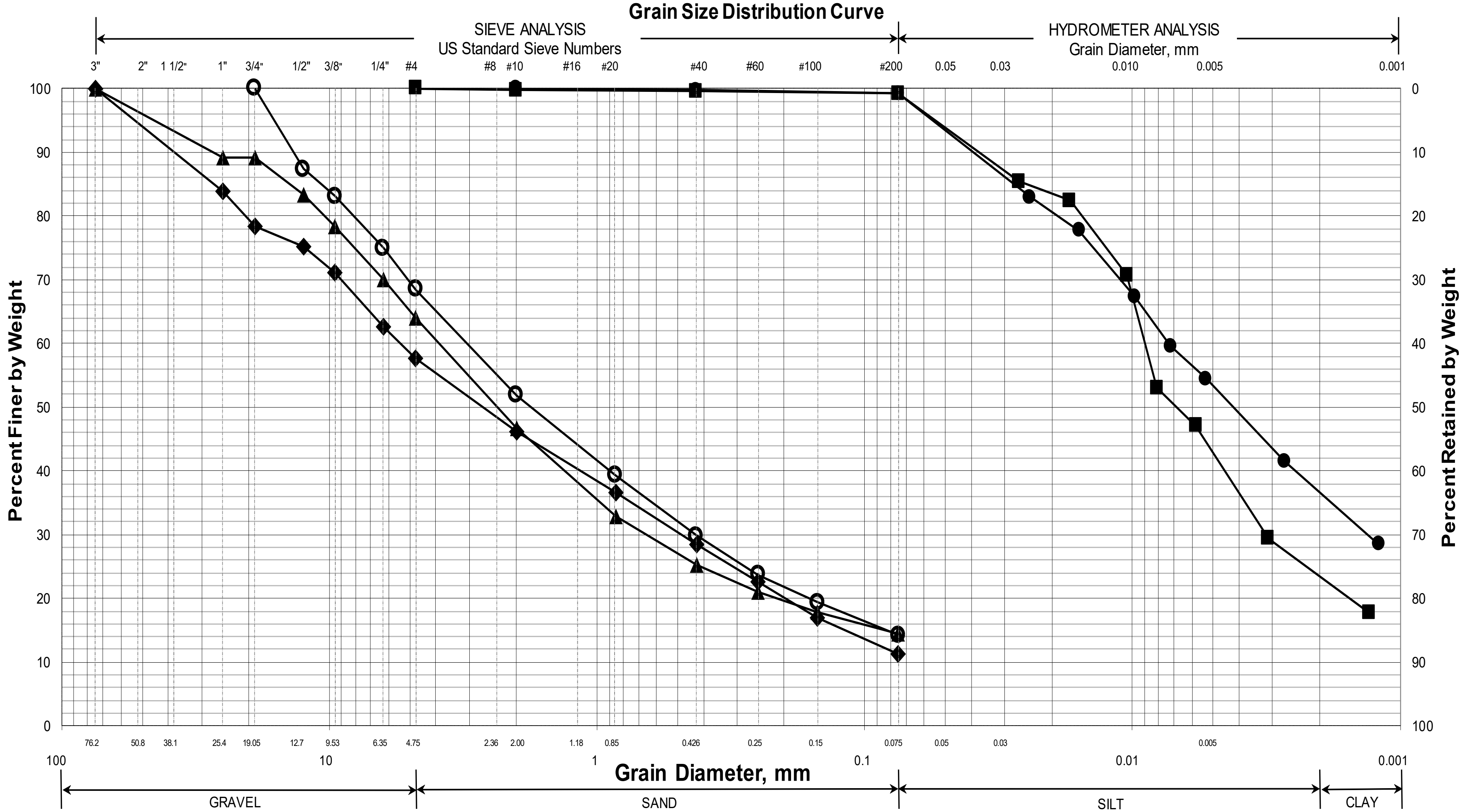
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	HB-ORO-101/1D	13+56.1	15.7 RT	0.0-2.0	SAND, some gravel, little silt.	6.3			
◆	HB-ORO-101/2D	13+56.1	15.7 RT	5.5-7.5	Gravelly SAND, little silt.	18.2			
■	HB-ORO-101/3D	13+56.1	15.7 RT	10.0-12.0	SILT, some clay, trace sand.	38.1	36	24	12
●	HB-ORO-101/4D	13+56.1	15.7 RT	15.0-17.0	Clayey SILT, trace sand.	34.3	31	21	10
▲	HB-ORO-101/6D	13+56.1	15.7 RT	25.0-27.0	Gravelly SAND, little silt.	10.0			
×									

WIN	
027194.00	
Town	
Orono	
Reported by/Date	
WHITE, TERRY A	4/30/2026

SHEET 1

12 Desert Rd, Freeport MaineDOT TESTING LABORATORIES 219 Hogan Rd, Bangor

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
379685	**HB-ORO-101/1D**	**GEOTECHNICAL (DISTURBED)**	**10/23/2023**	**1/23/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **13+56.1** Offset, ft: **15.7 RT** Dbfg, ft: **0.0-2.0**

WIN/Town **027194.00 - ORONO** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

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]	**87.4**
⅜ in. [9.5 mm]	**83.1**
¼ in. [6.3 mm]	**75.0**
No. 4 [4.75 mm]	**68.5**
No. 10 [2.00 mm]	**51.9**
No. 20 [0.850 mm]	**39.3**
No. 40 [0.425 mm]	**29.8**
No. 60 [0.250 mm]	**23.7**
No. 100 [0.150 mm]	**19.4**
No. 200 [0.075 mm]	**14.2**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**6.3**

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: **1/26/2024**

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
379686	**HB-ORO-101/2D**	**GEOTECHNICAL (DISTURBED)**	**10/23/2023**	**1/23/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **13+56.1** Offset, ft: **15.7 RT** Dbfg, ft: **5.5-7.5**

WIN/Town **027194.00 - ORONO** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**83.9**
¾ in. [19.0 mm]	**78.3**
½ in. [12.5 mm]	**75.2**
⅜ in. [9.5 mm]	**71.1**
¼ in. [6.3 mm]	**62.7**
No. 4 [4.75 mm]	**57.6**
No. 10 [2.00 mm]	**46.2**
No. 20 [0.850 mm]	**36.6**
No. 40 [0.425 mm]	**28.5**
No. 60 [0.250 mm]	**22.6**
No. 100 [0.150 mm]	**16.9**
No. 200 [0.075 mm]	**11.3**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**18.2**

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: **1/25/2024**

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
379687	HB-ORO-101/3D	GEOTECHNICAL (DISTURBED)	10/23/2023	1/23/2024

Sample Type: **GEOTECHNICAL** Location: Station: **13+56.1** Offset, ft: **15.7 RT** Dbfg, ft: **10.0-12.0**

WIN/Town **027194.00 - ORONO** Sampler: **BRUCE WILDER**

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]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	99.8
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.2
[0.0266 mm]	85.4
[0.0171 mm]	82.4
[0.0105 mm]	70.7
[0.0081 mm]	53.0
[0.0058 mm]	47.1
[0.0031 mm]	29.4
[0.0013 mm]	17.7

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	36
Plastic Limit (T 90), %	24
Plasticity Index (T 90), %	12
Specific Gravity, Corrected to 20°C (T 100)	2.66
Loss on Ignition, % (T 267)	
Water Content (T 265), %	38.1

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

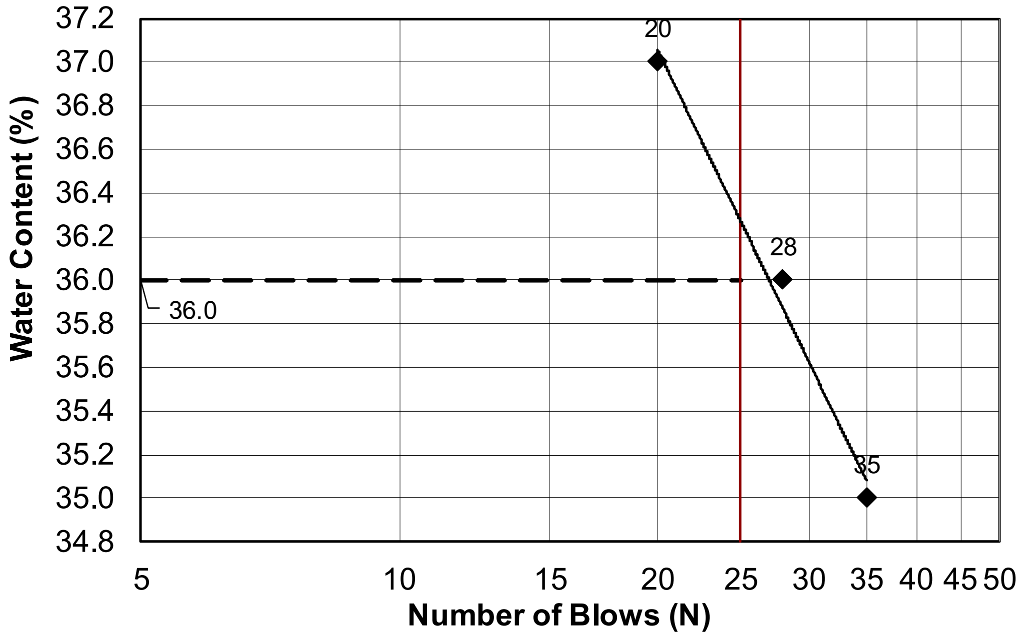
Comments:

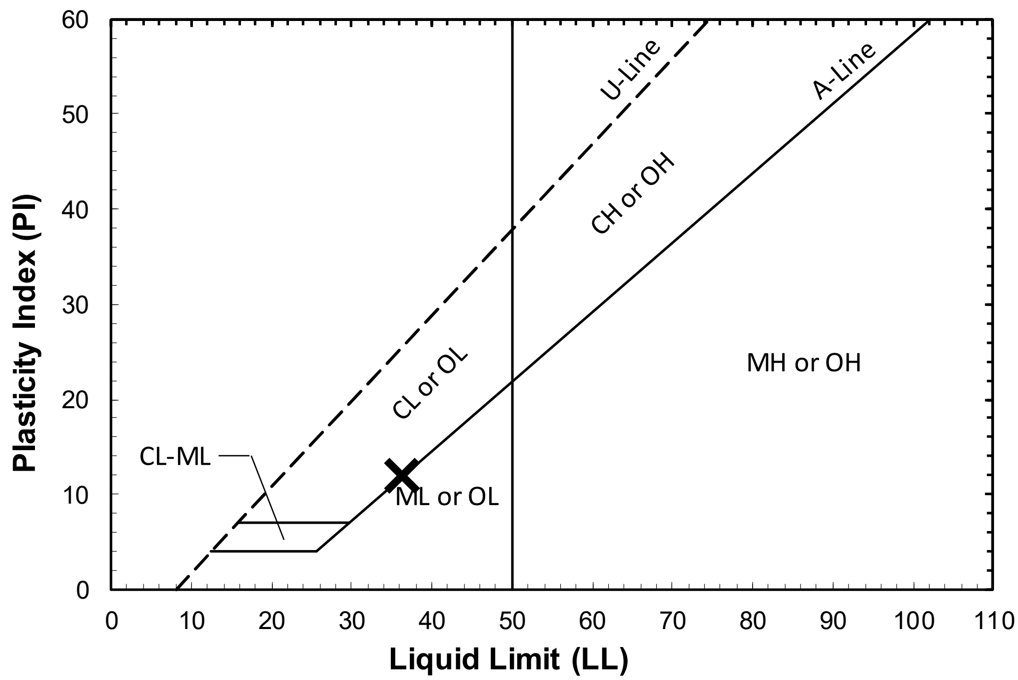
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/2/2024**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Orono	Reference No.	379687
WIN	027194.00	Water Content, %	38.1
Sampled	10/23/2023	Liquid Limit @ 25 blows (T 89), %	36
Boring No./Sample No.	HB-ORO-101/3D	Plastic Limit (T 90), %	24
Station	13+56.1	Plasticity Index (T 90), %	12
Depth	10.0-12.0	Tested By	BBURR





12 Desert Rd, Freeport MaineDOT TESTING LABORATORIES 219 Hogan Rd, Bangor

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
379688	HB-ORO-101/4D	GEOTECHNICAL (DISTURBED)	10/23/2023	1/23/2024

Sample Type: **GEOTECHNICAL** Location: Station: **13+56.1** Offset, ft: **15.7 RT** Dbfg, ft: **15.0-17.0**

WIN/Town **027194.00 - ORONO** Sampler: **BRUCE WILDER**

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]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.8
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.2
[0.0241 mm]	82.9
[0.0158 mm]	77.7
[0.0098 mm]	67.4
[0.0072 mm]	59.6
[0.0053 mm]	54.4
[0.0027 mm]	41.5
[0.0012 mm]	28.5
[mm]	

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	31
Plastic Limit (T 90), %	21
Plasticity Index (T 90), %	10
Specific Gravity, Corrected to 20°C (T 100)	2.81
Loss on Ignition, % (T 267)	
Water Content (T 265), %	34.3

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

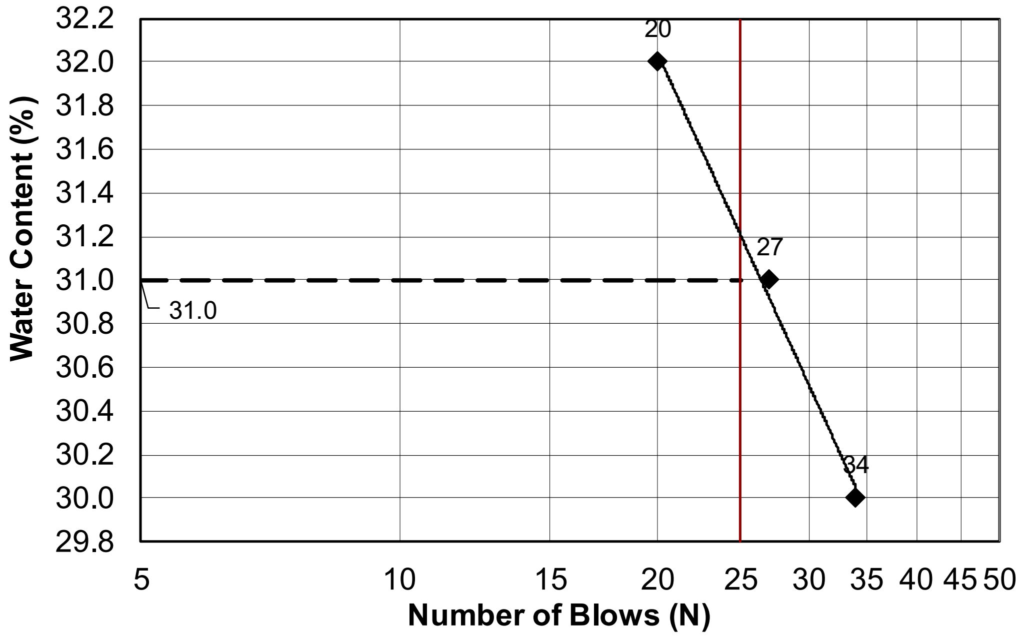
Comments:

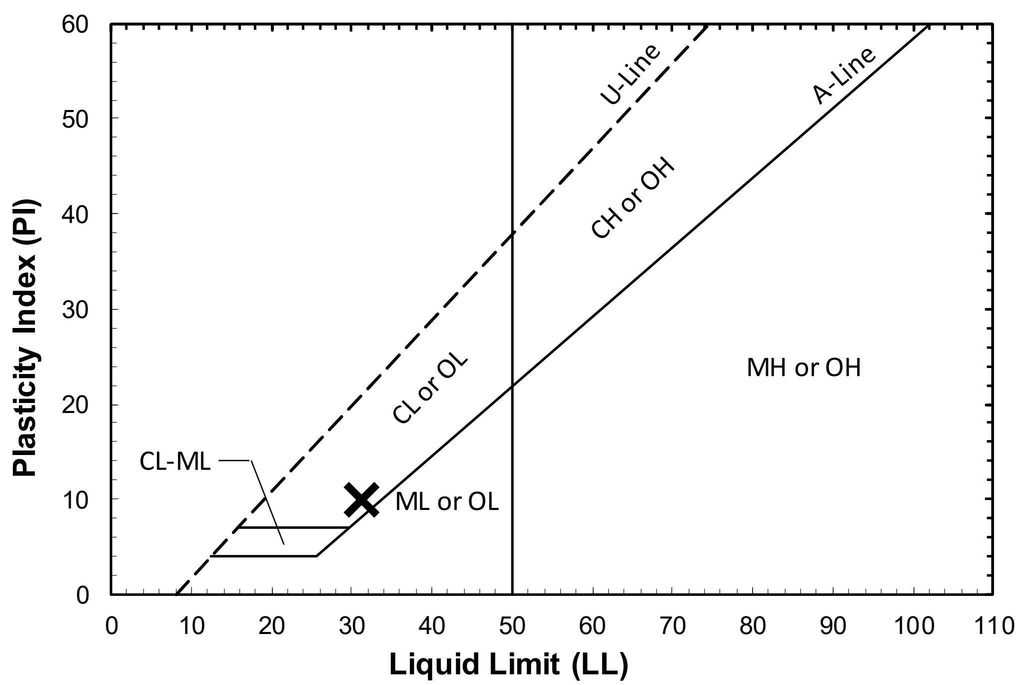
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/2/2024**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Orono	Reference No.	379688
WIN	027194.00	Water Content, %	34.3
Sampled	10/23/2023	Liquid Limit @ 25 blows (T 89), %	31
Boring No./Sample No.	HB-ORO-101/4D	Plastic Limit (T 90), %	21
Station	13+56.1	Plasticity Index (T 90), %	10
Depth	15.0-17.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
379689	**HB-ORO-101/6D**	**GEOTECHNICAL (DISTURBED)**	**10/23/2023**	**1/23/2023**

Sample Type: **GEOTECHNICAL** Location: Station: **13+56.1** Offset, ft: **15.7 RT** Dbfg, ft: **25.0-27.0**

WIN/Town **027194.00 - ORONO** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**89.1**
¾ in. [19.0 mm]	**89.1**
½ in. [12.5 mm]	**83.4**
⅜ in. [9.5 mm]	**78.4**
¼ in. [6.3 mm]	**70.1**
No. 4 [4.75 mm]	**64.0**
No. 10 [2.00 mm]	**46.6**
No. 20 [0.850 mm]	**32.9**
No. 40 [0.425 mm]	**25.3**
No. 60 [0.250 mm]	**21.0**
No. 100 [0.150 mm]	**17.8**
No. 200 [0.075 mm]	**14.5**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**10.0**

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: **1/25/2024**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

Liquidity Index

Liquidity Index

$$LI := \frac{WC - PL}{LL - PL}$$

Das, Principles of Engineering, 7th Edition, Equation 4.16

HB-ORO-101, 3D

$WC := 37.1$

$LL := 36$

$PL := 24$

$$LI := \frac{WC - PL}{LL - PL} = 1.09$$

HB-ORO-101, 4D

$WC := 34.3$

$LL := 31$

$PL := 21$

$$LI := \frac{WC - PL}{LL - PL} = 1.33$$

Sensitivity

HB-ORO-101/V1

$Su := 1138psf$

$Su_{re} := 268psf$

$$\frac{Su}{Su_{re}} = 4.25$$

HB-ORO-101/V2

$Su := 714psf$

$Su_{re} := 156psf$

$$\frac{Su}{Su_{re}} = 4.58$$

HB-ORO-101/V3

$Su := 446psf$

$Su_{re} := 67psf$

$$\frac{Su}{Su_{re}} = 6.66$$

HB-ORO-101/V4

$Su := 491psf$

$Su_{re} := 89psf$

$$\frac{Su}{Su_{re}} = 5.52$$

HB-ORO-101/V5

$Su := 446psf$

$Su_{re} := 112psf$

$$\frac{Su}{Su_{re}} = 3.98$$

HB-ORO-101/V6

$Su := 714psf$

$Su_{re} := 223psf$

$$\frac{Su}{Su_{re}} = 3.2$$

Sensitivities range from 3.2 to 6.7, ranging from moderately sensitive to sensitive

Fang, Foundation Engineering Handbook 3.13.3

ratio is smaller than in the soil remolded for the Atterberg limit tests. If the soil is located below the groundwater table (GWT) where it is saturated, one would therefore expect that smaller void ratios would have less water space and the w_N value would be smaller. From this we might deduce the following:

If w_N is close to w_L,	soil is normally consolidated.
If w_N is close to w_P,	soil is some- to heavily overconsolidated.
If w_N is intermediate,	soil is somewhat overconsolidated.
If w_N is greater than w_L,	soil is on verge of being a viscous liquid.

Although the foregoing gives a qualitative indication of overconsolidation, other methods must be used if a quantitative value of OCR is required.

We note that w_N can be larger than w_L, which simply indicates the in situ water content is above the liquid limit. Since the soil is existing in this state, it would seem that overburden pressure and interparticle cementation are providing stability (unless visual inspection indicates a liquid mass). It should be evident, however, that the slightest remolding disturbance has the potential to convert this type of deposit into a viscous fluid. Conversion may be localized, as for pile driving, or involve a large area. The larger w_N is with respect to w_L, the greater the potential for problems. The *liquidity index* has been proposed as a means of quantifying this problem and is defined as

$$I_L = \frac{w_N - w_P}{w_L - w_P} = \frac{w_N - w_P}{I_P} \tag{2-14}$$

where, by inspection, values of $I_L \geq 1$ are indicative of a liquefaction or "quick" potential. Another computed index that is sometimes used is the *relative consistency*,[2] defined as

$$I_C = \frac{w_L - w_N}{I_P} \tag{2-14a}$$

Here it is evident that if the natural water content $w_N \leq w_L$, the relative consistency is $I_C \geq 0$; and if $w_N > w_L$, the relative consistency or consistency index $I_C < 0$.

Where site evidence indicates that the soil may be stable even where $w_N \geq w_L$, other testing may be necessary. For example (and typical of highly conflicting site results reported in geotechnical literature) Ladd and Foott (1974) and Koutsoftas (1980) both noted near-surface marine deposits underlying marsh areas that exhibited large OCRs in the upper zones with w_N near or even exceeding w_L. This is, of course, contradictory to the previously given general statements that if w_N is close to w_L the soil is "normally consolidated" or is about to become a "viscous liquid."

Grain Size

The grain size distribution test is used for soil classification and has value in designing soil filters. A soil filter is used to allow drainage of pore water under a hydraulic gradient with

[2]This is the definition given by ASTM D 653, but it is more commonly termed the *consistency index,* particularly outside the United States.

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

Unit weight $\gamma := 125 \cdot \text{pcf}$

Internal friction angle $\phi := 32 \cdot \text{deg}$

Cohesion $c := 0 \cdot \text{psf}$

Outlet Walls Fixed to Box
At-Rest Earth Pressure - Rankine Theory

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

$\boxed{K_o = 0.47}$

Fang, Foundation Engineering Handbook 2nd ed. Pg. 224, Eq. 6.2 Formula for normally consolidated soils.

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:

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

$\boxed{K_{ar} = 0.31}$

For a sloped 2H:1V backfill

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

$$K_{ar\_slope} := \cos(\beta) \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}} \qquad \boxed{K_{ar\_slope} = 0.46}$$

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

6.1 AT-REST LATERAL PRESSURES

At-rest pressures exist in level ground, and develop under long-term conditions as the soil is deposited and acted upon by changes in the loading environment as caused by erosion, glaciers, and physicochemical processes. At-rest pressures rigorously only apply for walls that are placed into the ground with a minimum of disturbance and that remain unmoved during loading, or for unmoving, frictionless walls with a backfill placed with a minimum of compactive effort. In practice such conditions are rarely achieved. However, at-rest pressures are still useful in design as either a baseline against which other pressure states can be judged or as an assumed conservative choice for the design loading.

At-rest effective lateral pressures are often assumed to follow a linear distribution (Fig. 6.2), with the effective lateral pressure σ'_x taken as a simple multiple of the vertical effective pressure σ'_z:

$$\sigma'_x = K_0(\sigma'_z) \tag{6.1}$$

In homogeneous, dry soil with a constant K_0 and unit weight, both the vertical and lateral pressures are linearly distributed. With the presence of a water table, the at-rest pressure distribution exhibits a break in slope at the water table, reflecting the use of submerged unit weights to determine vertical effective stresses (Fig. 6.2).

Our early concepts of the parameter K_0 were formed on the basis of normally consolidated soils. Jaky (1944) proposed a relationship between K_0 and the drained friction angle ϕ' for normally consolidated soils:

$$\boxed{K_0 = 1 - \sin\phi'} \tag{6.2}$$

Numerous studies have confirmed the general validity of this empirical equation (Brooker and Ireland, 1965; Mayne and Kulhawy, 1982). However, results from laboratory experiments and in-situ tests have shown that the K_0 value also varies as a function of overconsolidation ratio (OCR) and stress history. For the case of a soil that has been subjected to one or more cycles of unloading, Schmidt (1966) proposed that K_0 can be determined as a function of its value in the normally consolidated state using the relationship

$$K_{0u} = K_{0nc}(\text{OCR})^{\alpha} \tag{6.3}$$

in which K_{0u} is the coefficient for unloading, K_{0nc} is the coefficient for the normally consolidated soil, and α is a dimensionless coefficient. Experimental data have confirmed this relationship, and Mayne and Kulhawy (1982) showed that, for most soils, α can be taken as $\sin\phi'$.

Soils that are overconsolidated and are in the process of being reloaded pose a difficulty in that Equation 6.3 does not apply. For this condition, a more complex equation is needed as well as a full knowledge of the stress history of the soil (Mayne and Kulhawy, 1982). For practical purposes, it may be enough to know that the K_0 during reloading falls about halfway between that for unloading and normally consolidated conditions. Also, K_0 might be directly determined through in-situ testing methods.

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

	Coefficient of Lateral Earth Pressure			
Soil type	*OCR = 1*	*OCR = 2*[a]	*OCR = 5*[a]	*OCR = 10*[a]
Loose sand	0.45	0.65	1.10	1.50
Medium sand	0.40	0.60	1.05	1.55
Dense sand	0.35	0.55	1.00	1.50
Silt	0.50	0.70	1.10	1.60
Lean clay, CL	0.60	0.80	1.20	1.65
Highly plastic clay, CH	0.65	0.80	1.10	1.40

[a] Unloading cycle.

Table 6.1 presents typical values for K_0 for a subset of soils. For other conditions, K_0 values can be determined directly from Equations 6.2 and 6.3, and/or using in-situ testing techniques.

Because the K_0 value in a given soil often varies with depth, and the soil types themselves may change with depth, the at-rest lateral pressure distribution is typically not linear as shown in Figure 6.2. Self-boring pressuremeter tests in clays with overconsolidated profiles induced by desiccation have demonstrated that the K_0 under such conditions decreases with depth in the soil deposit and reaches a steady state where the desiccation effects are no longer present (Clough and Denby, 1980).

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

Most walls move, either by global shifting or by local deformations. These movements cause adjustments to occur in the earth loads and the pressure distributions. Conventional means for assessing the effects of system movements are to set them into the context of extreme conditions. These are referred to as the active and passive earth pressure loadings.

6.2.1 Active Pressure

Assuming that a gravity wall with no friction on its face is translated away from a soil mass that is initially at the at-rest condition, then the soil mass adjacent to the wall will pass into a failure state as shown in Figure 6.3. At this stage, the

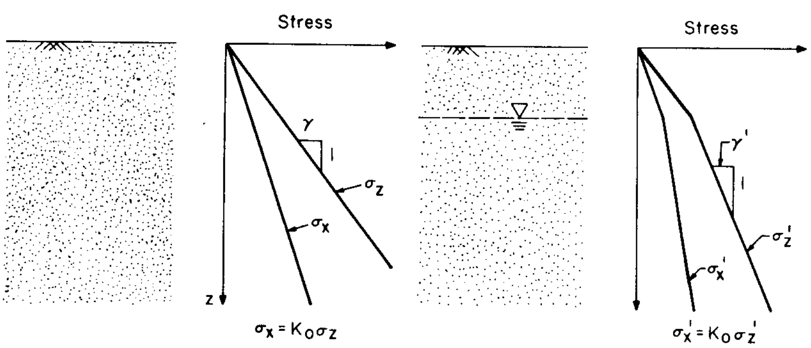


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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:

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

where:

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

$\beta =$ 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:

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

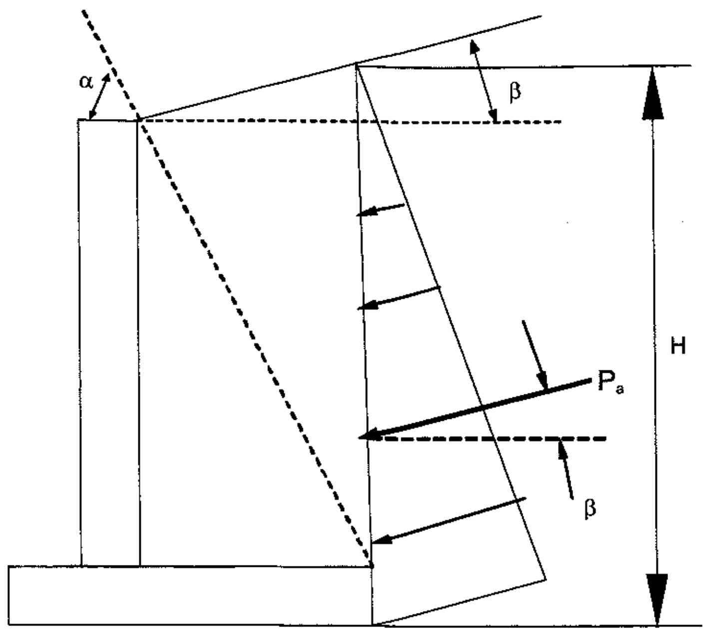


Figure 3-3 Rankine Theory

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 <u>AASHTO LRFD</u> 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

Bearing Resistance - Existing Soils:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - Box Culvert on Silt/Clayey Silt

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 10th Edition 2024
Table C10.6.2.5.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Silt/Clayey Silt (CL)

Based on Shear Strength, soils are medium stiff to stiff near the bearing elevation

Density In Place: medium stiff to stiff

Bearing Resistance: Ordinary Range (ksf) 2 to 6

Recommended Value of Use: $q_{nom} := 3 \cdot ksf$

Resistance factor at the **service limit state** = 1.0 (LRFD Article 10.5.5.1) $\phi_{service\_bc} := 1.0$

$q_{factored\_service\_bc} := q_{nom} \cdot \phi_{service\_bc}$ $q_{factored\_service\_bc} = 3 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - Box Culvert on Silt/Clayey Silt

Reference: AASHTO LRFD Bridge Design Specifications 10th Edition 2024 - Article 10.6.3.1

Assumptions:

1. The box will be founded at ~ Elev 110.97 feet

   Bottom of Construction will be 3 feet below box invert $D_{footing} := 3.0 \cdot ft$

2. Assumed parameters for fill soils:

   Saturated unit weight: $\gamma_s := 135 \cdot pcf$

   Internal friction angle: $\phi_{ns} := 32 \cdot deg$

   Undrained shear strength: $c_{ns} := 0 \cdot psf$

3. Box Culvert parameters

   Width of box culvert, B $B_{box} := 11 \cdot ft$

   Length of box culvert, L $L_{box} := 72 \cdot ft$

Foundation soils:

Foundation soils properties based on HB-ORO-101

$\gamma1_d := 75 \cdot pcf$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Soft Clay

$w_{sat} := 35.7\%$ — HB-ORO-101 3D, 4D mean Natural Water Content

$\gamma1_{sat} := \gamma1_d \cdot (1 + w_{sat})$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit weight relationships

$\gamma1_{sat} = 101.775 \cdot pcf$

$\phi := 0 \cdot deg$

$c := 658psf$ — Average of 6 vane tests in HB-ORO-101

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

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

$N_c := 5.14 \qquad N_q := 1 \qquad N_\gamma := 0$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$$s_c := 1 + \left(\frac{B_{box}}{5L_{box}}\right)$$

$s_\gamma := 1$

$s_q := 1$

$s_c = 1.0306 \qquad s_\gamma = 1 \qquad s_q = 1$

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.

Depth the water table: $D_w := 0 \cdot ft \qquad C_{wq} := 0.5 \qquad C_{w\gamma} := 0.5$

Load Inclination factors

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

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

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-3

$$\frac{D_{footing}}{B_{box}} = 0.27$$

Therefore : $d_q := 1.0$

Terms

$N_{cm} := N_c \cdot s_c \cdot i_c$ LRFD Eq. 10.6.3.1.2a-2

$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$ LRFD Eq. 10.6.3.1.2a-3

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ LRFD Eq. 10.6.3.1.2a-4

$N_{cm} = 5.2971 \qquad N_{\gamma m} = 0 \qquad N_{qm} = 1$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c \cdot N_{cm} + \gamma_s \cdot D_{footing} \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma 1_{sat} \cdot \overrightarrow{\left(B_{box} \cdot N_{\gamma m}\right)} \cdot C_{w\gamma}\right]$$

$q_n = 3.7 \cdot ksf$

Factored Bearing Resistance

$\phi_b := 0.5$

$q_r := q_n \cdot \phi_b$

$\boxed{q_r = 1.8 \cdot ksf}$

Recommend a limiting factored bearing resistance of 1.5 ksf for the Strength Limit State.

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.

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/ft³	kN/m³
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 C10.6.2.5.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Bearing Resistance (ksf) Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel–sand mixture, boulder–gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.5.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

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.

The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (Barker et al., 1991). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in Table 10.5.5.2.2-1 is provided in Allen (2005).

The resistance factors for plate load tests and passive resistance were based on engineering judgment and past ASD practice.

10.5.5.2.3—Driven Piles

Resistance factors shall be selected from Table 10.5.5.2.3-1 based on the method used for determining the driving criterion necessary to achieve the required nominal pile bearing resistance.

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the design resistance factors selected should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site shall be defined as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions.

C10.5.5.2.3

Where nominal pile bearing resistance is determined by static load test, dynamic testing, wave equation, or dynamic formulas, the uncertainty in the nominal resistance is strictly due to the reliability of the resistance determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each production pile is field-verified based on compliance with a driving criterion developed using a dynamic method (see Articles 10.7.3.8.2, 10.7.3.8.3, 10.7.3.8.4, or 10.7.3.8.5). The actual penetration depth where the pile is stopped using the driving criterion (e.g., a blow count measured during pile driving) will likely not be the same as the estimated depth from the static analysis. Hence, the reliability of the nominal pile bearing resistance is dependent on the reliability of the method used to verify the nominal resistance during pile installation (see Allen, Consideration should be given to the relative change in the computed nominal resistance based on effective versus gross footing dimensions for the size of footings typically used for bridges. Judgment should be used in deciding whether the use of gross footing dimensions for computing nominal bearing resistance at the strength limit state would result in a conservative design.

10.6.3.1.2—Theoretical Estimation

10.6.3.1.2a—Basic Formulation

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. In cases where the cohesive soils may soften and lose strength with time, the bearing resistance of these soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

Except as noted below, the nominal bearing resistance of a soil layer, in ksf, should be taken as:

$$\boxed{q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a\text{-}1)}$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \quad (10.6.3.1.2a\text{-}4)$$

where:

c = cohesion, taken as undrained shear strength (ksf)

N_c = cohesion term (undrained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)

C10.6.3.1.2a

The bearing resistance formulation provided in Eqs. 10.6.3.1.2a-1 through 10.6.3.1.2a-4 is the complete formulation as described in the Munfakh et al. (2001). However, in practice, not all of the factors included in these equations have been routinely used.

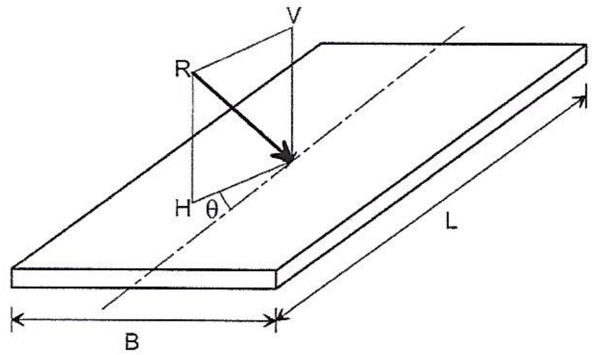


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

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Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

$$d_q = 1 + 2\tan\phi_f(1-\sin\phi_f)^2 \arctan\left(\frac{D_f}{B}\right) \qquad (10.6.3.1.2a\text{-}10)$$

where:

d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation(dim)
ϕ_f = angle of internal friction of soil (degrees)
D_f = footing embedment depth (ft)
B = footing width (ft)

Arctan (D_f/B) is in radians.

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0. The depth correction factor, d_q, shall not exceed 1.4.

Eq. 10.6.3.1.2a-10 has been verified to cover a range of friction angle, ϕ_f, of 32 degrees to 42 degrees, and a range of D_f/B of 1 to 8. Depth correction factor values beyond this range have not been verified at this time.

10.6.3.1.2b—Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \qquad (10.6.3.1.2b\text{-}1)$$

$$\phi^* = \tan^{-1}(0.67\tan\phi_f) \qquad (10.6.3.1.2b\text{-}2)$$

where:

c^* = reduced effective stress soil cohesion for punching shear (ksf)
ϕ^* = reduced effective stress soil friction angle for punching shear (degrees)

C10.6.3.1.2b

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the footing. Local shear failure is a transitional condition between general and punching shear failure. Punching shear failure is characterized by vertical shear around the perimeter of the footing and is accompanied by a vertical movement of the footing and compression of the soil immediately below the footing but does not affect the soil outside the loaded area. Punching shear failure occurs in loose or compressible soils, in weak soils under slow (drained) loading, and in dense sands for deep footings subjected to high loads.

Modulus of Subgrade Reaction

Objective:

Estimate the modulus of subgrade reaction for the box culvert base slab design.

Given:

1. Limited lab data, SPT N-values, and in-situ vane shear test results.

Assumptions:

1. The proposed bearing elevation of base slab is approximately 110 feet.
2. Proposed finished roadway grade is approximately 121.5.
3. Proposed precast concrete box is 11 feet wide and approximately 72 feet long.
4. The subsurface conditions present at the proposed bearing elevation is soft to stiff glaciomarine Silt/Clayey Silt, with Su=446-1138 psf, Su(average)=658 psf.
5. The bottom of the box culvert will be submerged for the structure's design life.
6. $q_u/2=s_u$ q_u= 658x2 psf = 1316 psf = 0.59 TSF

Published values of subgrade modulus

Published values of subgrade modulus in Silt/Clayey Silt:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:
Range of modulus of subgrade reaction 44 to 88 pci
Subgrade of clayey soil q_u< 200kPa = 4200psf, lower limit: k_s = 44 pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Fine grained soils qu=0.59 TSF, use lowest value presented on Fine Grained Curve
K_{V1}, 23 pci / 2 = 11.5 pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:
Typical subgrade reaction values for 0.3 m x 0.3 m plate
No value for medium stiff clay, use stiff clay, 37 - 92 pci: $k_{0.3}$ (k_1) = 50 pci

Adjust Published values for dimensions of base slab

Published range for medium stiff, silty clay subgrade is 11.5 - 50 pci

Assume a subgrade modulus of 40.2 pci, average of 11.5, 44, and 50 pci.

Value of k_{s1} = 35.2 pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base (Width B = 11 ft, Length L = 72 ft).

Square to rectangle base adjustment:

$k_{s1} := 35.2\text{pci} \quad B := 11\text{ft} \quad L := 72\text{ft}$

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

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

$k = 25.3 \cdot \text{pci}$

Recommend a subgrade modulus of 25 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
o 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

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

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

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 e 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

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

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

44 pci

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

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

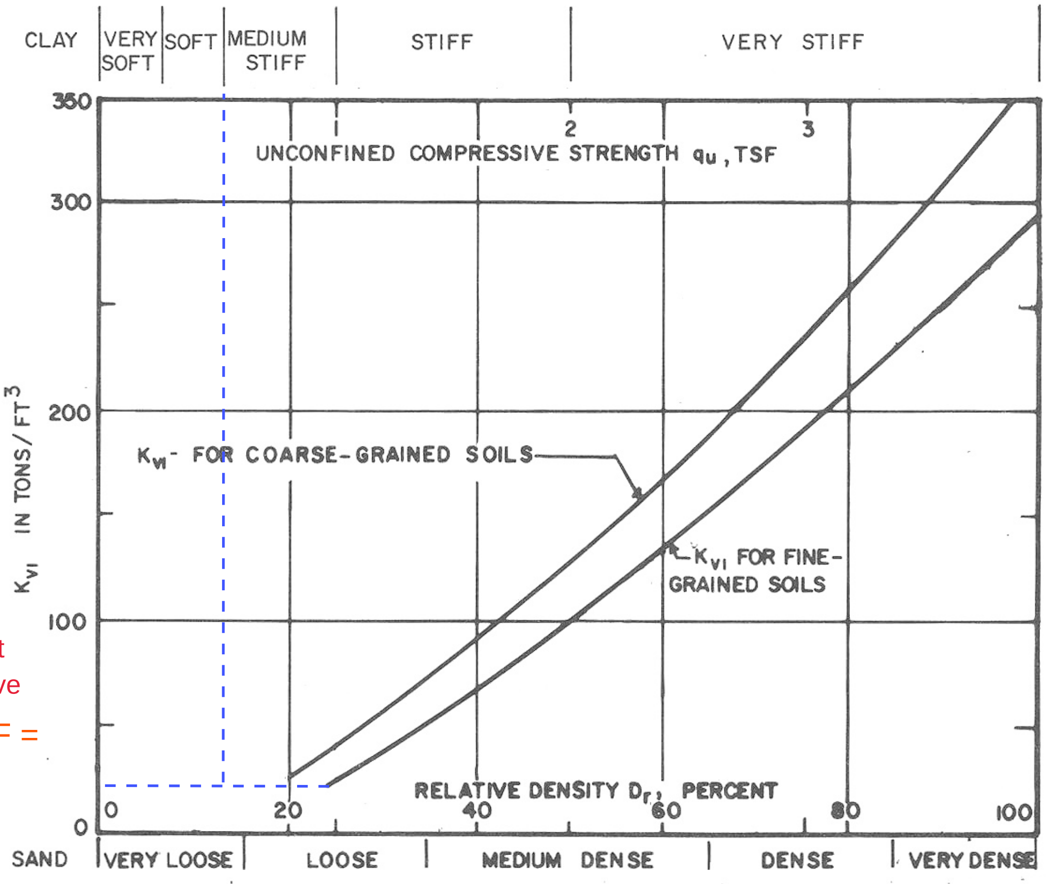
using Eq. (5-16) and Table 5-2 (or your program FFACTOR) for factors 0.584 and 0.023.
At $D/B = 0.61/1.22 = 0.5$, we obtain $I_F = 0.80$ from Fig. 5-7 (or when using FFACTOR for the I_s factors). Substitution into Eq. (9-7) with $B' = 1.22/2 = 0.61$, and $m = 4$ yields

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

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)]



Select lowest value on curve

20 TCF = 23 PCI

Reduce per Note 2

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

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

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

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:

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

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 $B \times B$

Settlement

Objective:
1) To estimate soil parameters for Settle 3D analysis

Given:
1) Boring Log HB-ORO-101 and lab test data.

Assumptions:
1) Groundwater is at El. 115.8.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 is used to construct the proposed raise roadway grade (approximately 2 feet).
3) Unless otherwise noted, HB-ORO-101 will be used to determine strata elevations and consistencies.

References:
1) Das, B. M. (2014). Principles of geotechnical engineering (7th ed.)
2) Bowles, J. E. (1996). Foundations analysis and design (5th ed.)
3) Cox, C., & Mayne, P. W. Constitutive model input parameters for numerical analyses of geotechnical problems: An in-situ testing case study
4) Andrews, D. W. (1986). The engineering aspects of the Presumpscot formation.

Surcharge Load

Proposed raise roadway grade = 2 feet

$H_{fill} := 2ft$

$\gamma_{fill} := 125pcf$ BDG Table 3-3, Soil Type 4, Granular Borrow

$\sigma_{z\_induced} := \gamma_{fill} \cdot H_{fill}$

$\boxed{\sigma_{z\_induced} = 0.25 \cdot ksf}$

Existing Ground Elevation = El. 118.8 ft

Soil Layer 1 (Elev. 118.8 - 109.3) Fill: Granular Borrow, with drainage system

$N_{60\_1} := 24$

N60 of HB-ORO-101, 1D = 24
HB-ORO-101, 2D = 26, Conservatively use N60 = 24

$E_{s1} := \frac{500(N_{60\_1} + 15)}{50} ksf$

Bowles Table 5-6, Equation for stress-strain modulus E_s for Sand (normally consolidated)

$\boxed{E_{s1} = 390 \cdot ksf}$

$E_{ur1} := 4 \cdot E_{s1}$

Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur1} = 1560 \cdot ksf}$

$\boxed{\gamma_{dry1} := 125pcf}$

BDG Table 3-3, Soil Type 4, Granular Borrow

$w_{sat1} := 12.3\%$

HB-ORO-101, 1D, 2D mean Natural Water Content

$\gamma_{sat1} := \gamma_{dry1} \cdot (1 + w_{sat1})$

$\boxed{\gamma_{sat1} = 140 \cdot pcf}$

Soil Layer 2 (Elev. 109.3 - 94.3) Persumpscot Formation: soft to stiff SILT/ Clayey SILT

$N_{60\_2} := 5$

Shear Strength = 658 psf (Average of 6 vane tests), assume N60 of Persumpscot Formation = 5 (N60 for medium stiff fine-grained soils = 5-8)

$E_{s2} := \frac{300(N_{60\_2} + 6)}{50} ksf$

Bowles Table 5-6, Equation for stress-strain modulus E_s for Silt, Sandy Silt, or Clayey Silt

$\boxed{E_{s2} = 66 \cdot ksf}$

$E_{ur2} := 4 \cdot E_{s2}$

Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur2} = 264 \cdot ksf}$

$\boxed{\gamma_{dry2} := 75 \cdot pcf}$

Das, Table 3.2: Dry Unit Weights, Soft Clay

$w_{sat2} := 35.7\%$

HB-ORO-101, 3D, 4D mean Natural Water Content

$\gamma_{sat2} := \gamma_{dry2} \cdot (1 + w_{sat2})$

$\boxed{\gamma_{sat2} = 102 \cdot pcf}$

$\boxed{C_{c2} := 0.4}$ Andrews, Table III

$\boxed{C_{r2} := 0.04}$ Assume 10% of Cc

$\boxed{OCR_2 := 1.0\ ksf}$ Conservatively assume normally consolidated

$\boxed{e_2 := 1.4}$ Das, Table 3.2: Void Ratio, Soft Clay = 0.9-1.4. Conservatively choose upper bound

$C_{v2} := 0.05 \frac{ft^2}{day}$ Andrews, pg. 11, Cv range from 0.05-0.15 square feet per day. Choose lower bound

$\boxed{C_{v2} = 18.262 \cdot \frac{ft^2}{yr}}$

Soil Layer 3 (Elev. 94.3 - 91.8) Till: very dense Gravelly SAND

$N_{60\_3} := 54$ N60 of HB-ORO-101, 6D

$E_{s3} := \frac{250(N_{60\_3} + 15)}{50} ksf$ Bowles Table 5-6, Equation for stress-strain modulus E_s for Sand (saturated)

$\boxed{E_{s3} = 345 \cdot ksf}$

$E_{ur3} := 4 \cdot E_{s3}$ Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur3} = 1380 \cdot ksf}$

$\boxed{\gamma_{dry3} := 134pcf}$ Das, Table 3.2: Dry Unit Weights, Glacial till

$w_{sat3} := 10.0\%$ HB-ORO-101, 6D Natural Water Content

$\gamma_{sat3} := \gamma_{dry1} \cdot (1 + w_{sat1})$

$\boxed{\gamma_{sat3} = 140 \cdot pcf}$

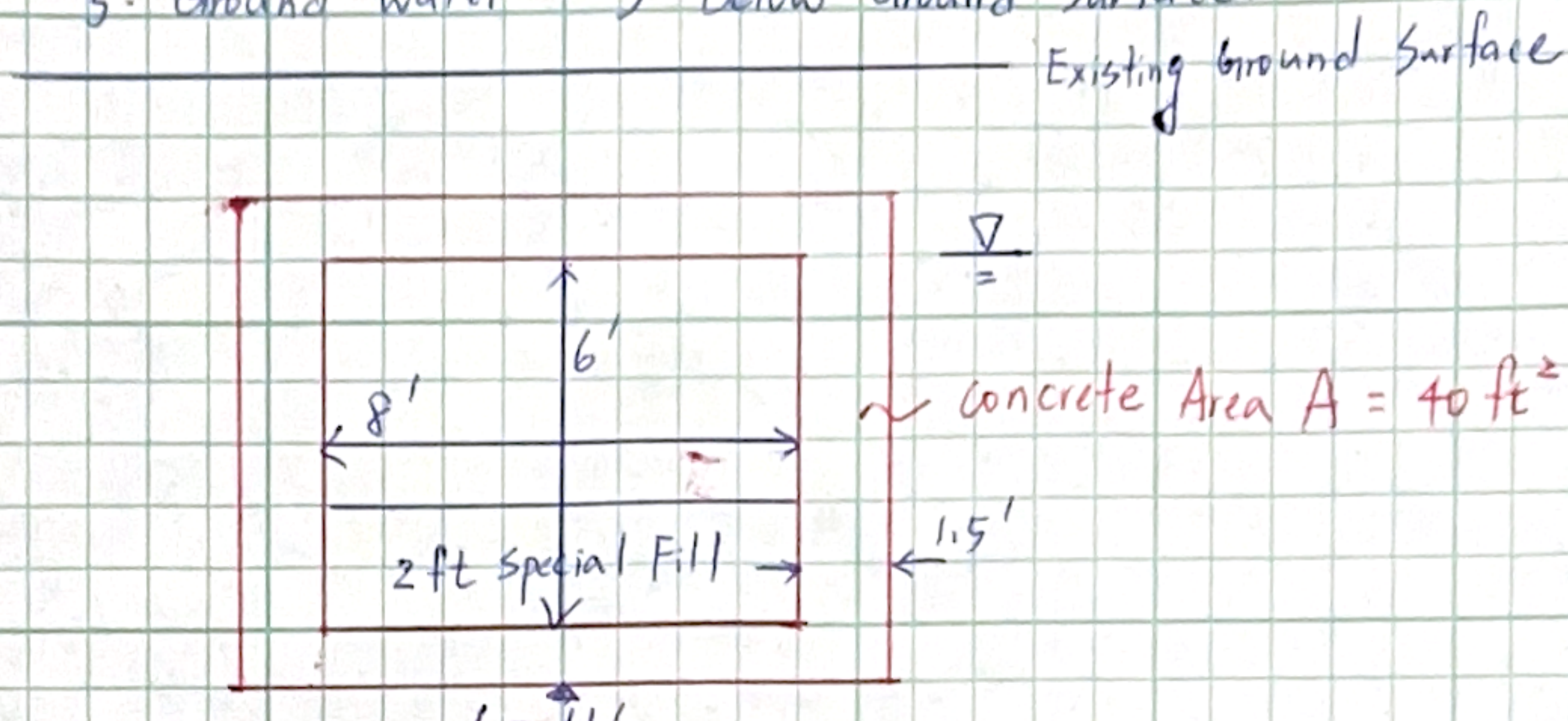
Assumptions : 1. Fill Unit Weight = γ_{fill} = 125 pcf , γ_{sat_fill} = 135 pcf

2. Special Fill Unit Weight : $\gamma_{sat\_special\_fill}$ = 135 pcf

3. Concrete Unit Weight = γ_{conc} = 150 pcf

4. Ground water Unit Weight = 62.4 pcf

5. Ground water = 3' Below Ground Surface.



Existing Effective Stress :

$$\sigma'_{v\,existing} = 125\,pcf \times 3\,ft + (135\,pcf - 62.4\,pcf) \times 7ft = 883.2\,psf$$

Proposed Effective Stress :

$$\sigma'_{v\,proposed} = 125\,pcf \times 2ft + \frac{150\,pcf \times 40\,ft^2}{11\,ft} + (135\,pcf - 62.4\,pcf) \times 2\,ft$$
$$= 940.7\,psf$$

Change in Effective Stress :

$$\Delta\sigma'_v = 940.7\,psf - 883.2\,psf = 57.5\,psf = 0.058\,ksf$$ #

Settle3D Analysis Information
27194 Orono Large Culvert

Project Settings

Document Name	27194 Orono Settlement.s3z
Project Title	27194 Orono Large Culvert
Analysis	Immediate and consolidation settlement
Author	Yueh-Ti Lee
Company	MaineDOT
Date Created	6/24/2026
Comments	
Delta q = 0.25 ksf surcharge, and 0.058 ksf at proposed culvert base	
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Use average properties to calculate layered stresses	

Stage Settings

Stage #	Name	Time [years]
1	Stage 1	0
2	Stage 2	1
3	Stage 3	50

Results

Time taken to compute: 0.0604452 seconds

Stage: Stage 1 = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.660465
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.660465
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.161245	0.289427
Effective Stress [ksf]	0.25	1.7054
Total Stress [ksf]	0.25	3.36425
Total Strain	0.000468826	0.0042125
Pore Water Pressure [ksf]	0	1.65885
Excess Pore Water Pressure [ksf]	0	0.289427
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.25375	1.69764
Over-consolidation Ratio	1	1
Void Ratio	0	1.39375
Permeability [ft/y]	0	0.0828206
Coefficient of Consolidation [ft^2/y]	0	18.262
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.34108

Stage: Stage 2 = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.58785
Consolidation Settlement [in]	-0.00140463	0.927389
Immediate Settlement [in]	0	0.660465
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.161245	0.289427
Effective Stress [ksf]	0.25	1.67639
Total Stress [ksf]	0.25	3.36425
Total Strain	0.000468826	0.0219709
Pore Water Pressure [ksf]	0	1.68786
Excess Pore Water Pressure [ksf]	0	0.19026
Degree of Consolidation [%]	0	46.3392
Pre-consolidation Stress [ksf]	0.25375	1.69764
Over-consolidation Ratio	1	1.01708
Void Ratio	0	1.39396
Permeability [ft/y]	0	0.0828206
Coefficient of Consolidation [ft^2/y]	0	18.262
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	35.2335
Undrained Shear Strength	0	0.339911

Stage: Stage 3 = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.50412
Consolidation Settlement [in]	0	2.00131
Immediate Settlement [in]	0	0.660465
Secondary Settlement [in]	0	0.842346
Loading Stress [ksf]	0.161245	0.289427
Effective Stress [ksf]	0.25	1.86664
Total Stress [ksf]	0.25	3.36425
Total Strain	0.000468826	0.0334829
Pore Water Pressure [ksf]	0	1.4976
Excess Pore Water Pressure [ksf]	0	3.52457e-007
Degree of Consolidation [%]	0	99.9999
Pre-consolidation Stress [ksf]	0.25375	1.85938
Over-consolidation Ratio	1	1
Void Ratio	0	1.36575
Permeability [ft/y]	0	0.0828206
Coefficient of Consolidation [ft^2/y]	0	18.262
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	99.9999
Undrained Shear Strength	0	0.347299

Loads

1. Rectangular Load

Length	72 ft
Width	11 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	792 ft^2
Load	0.058 ksf
Depth	9 ft
Installation Stage	Stage 1 = 0 y

Coordinates

X [ft]	Y [ft]
-28.128	10.036
43.872	10.036
43.872	21.036
-28.128	21.036

2. Rectangular Load

Length	28 ft
Width	150 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4200 ft^2
Load	0.25 ksf
Depth	0 ft
Installation Stage	Stage 1 = 0 y

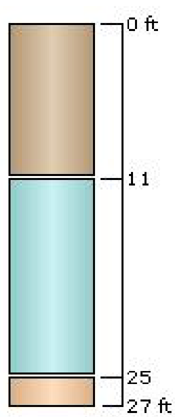
Coordinates

X [ft]	Y [ft]
-6.128	-59.464
21.872	-59.464
21.872	90.536
-6.128	90.536

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Fill	11	0	No
2	Presumpscot: Silt/ Clayey Silt	14	11	No
3	Till	2	25	No



Soil Properties

Property	Fill	Presumpscot: Silt/ Clayey Silt	Till
Color			
Unit Weight [kips/ft^3]	0.125	0.075	0.134
Saturated Unit Weight [kips/ft^3]	0.14	0.102	0.14
Immediate Settlement	Enabled	Enabled	Enabled
Es [ksf]	390	66	345
Esur [ksf]	1560	264	1380
Primary Consolidation	Disabled	Enabled	Disabled
Material Type		Non-Linear	
Cc		0.4	
Cr		0.04	
e0		1.4	
OCR	1	1	1
Cv [ft^2/y]		18.262	
B-bar		1	
Secondary Consolidation	Disabled	Mesri	Disabled
Ca/Cc		0.04	
Undrained Su A [kips/ft2]	0	0	0
Undrained Su S	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8
Piezo Line ID	1	1	1

Groundwater

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

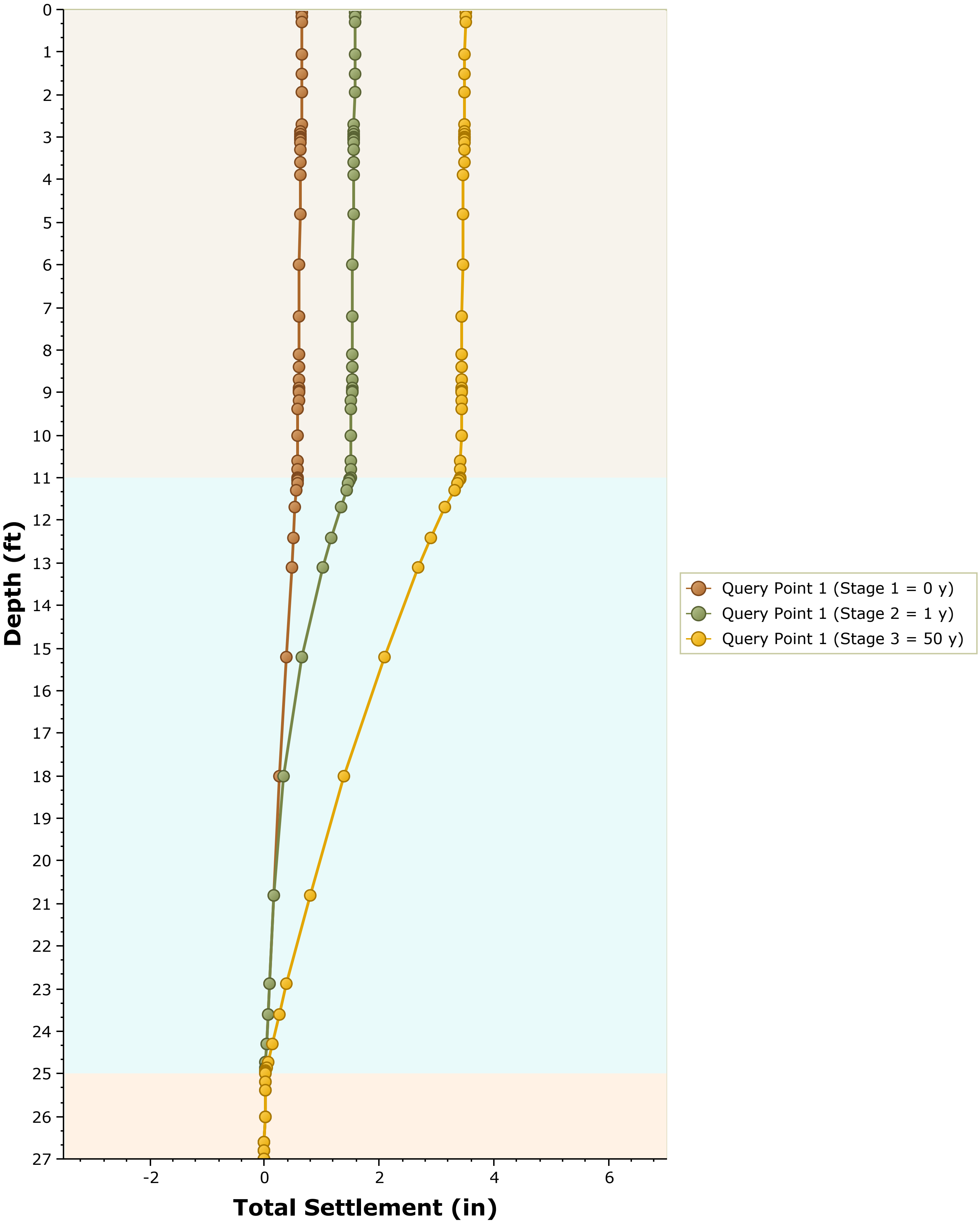
Piezometric Line Entities

ID	Depth (ft)
1	3 ft

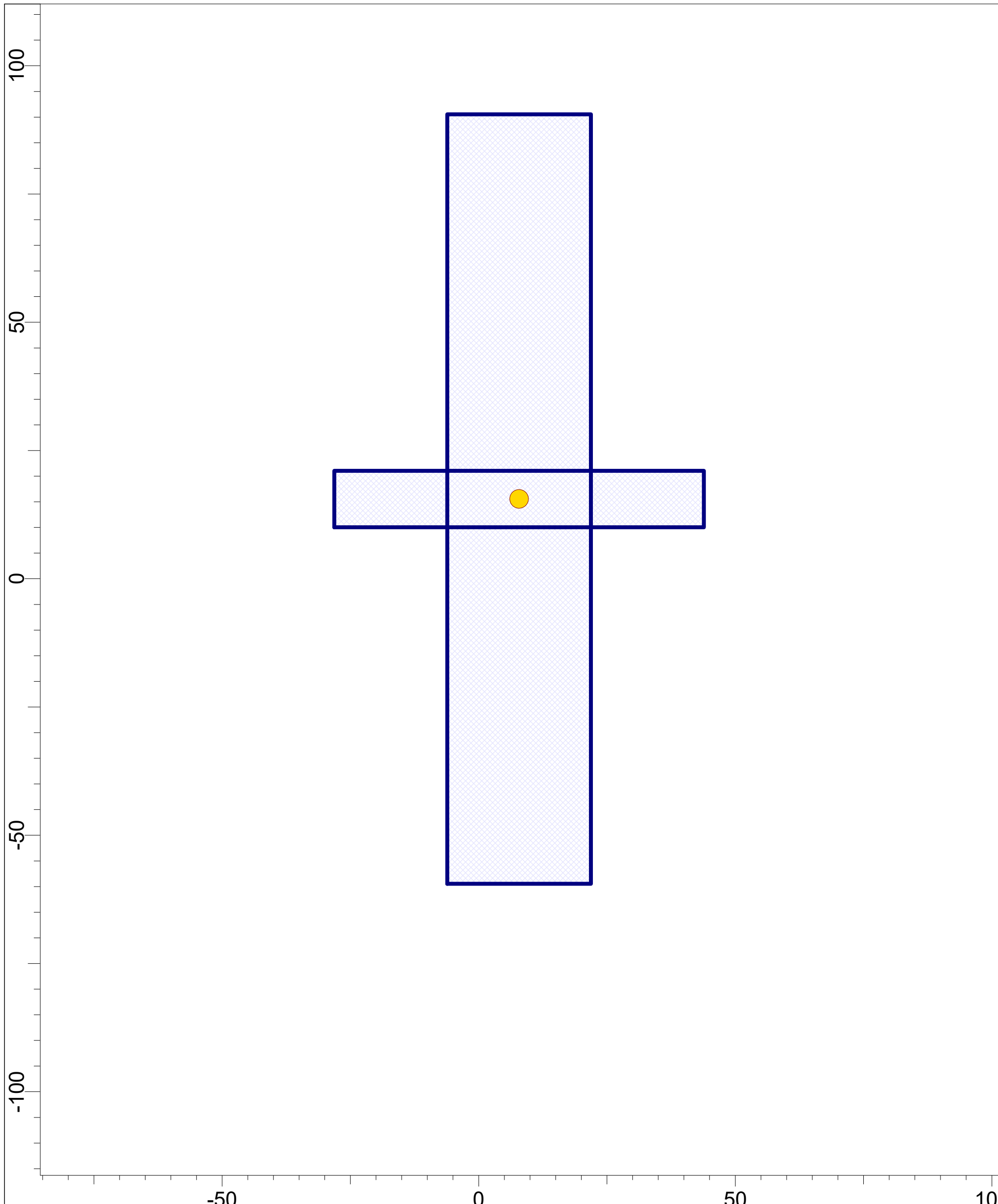
Query Points

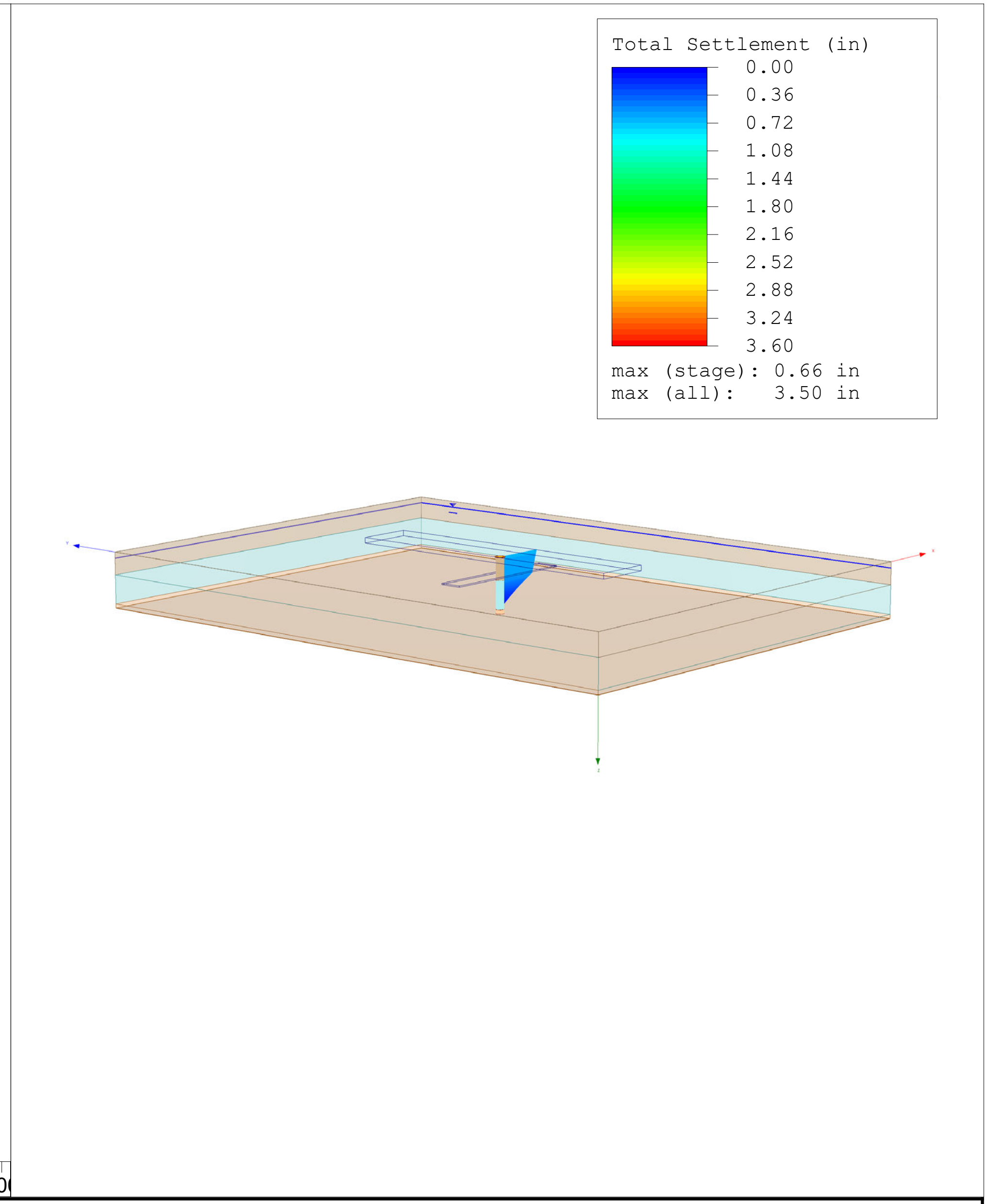
Point #	(X,Y) Location	Number of Divisions
1	7.872, 15.536	Auto: 59

Total Settlement vs. Depth

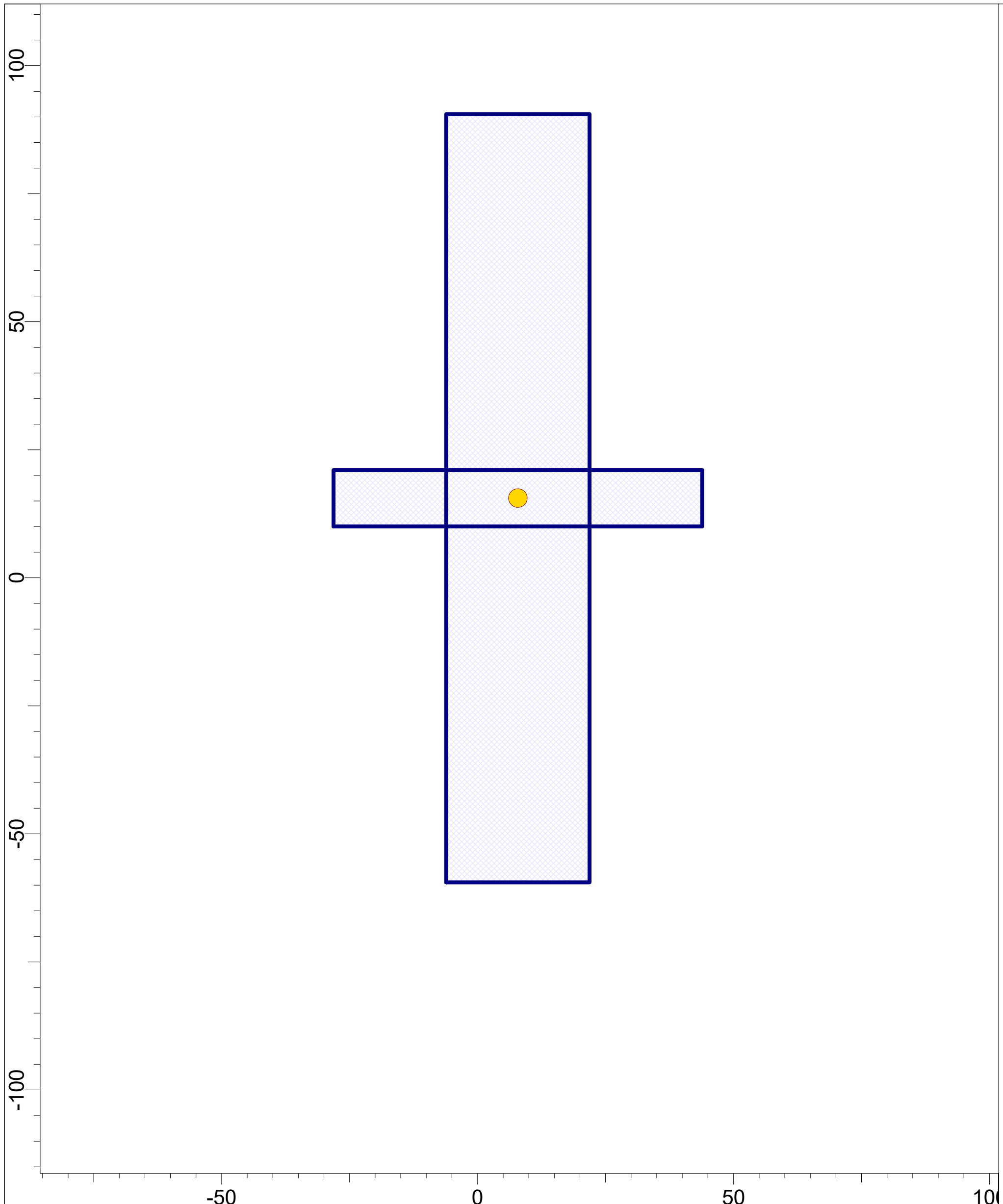


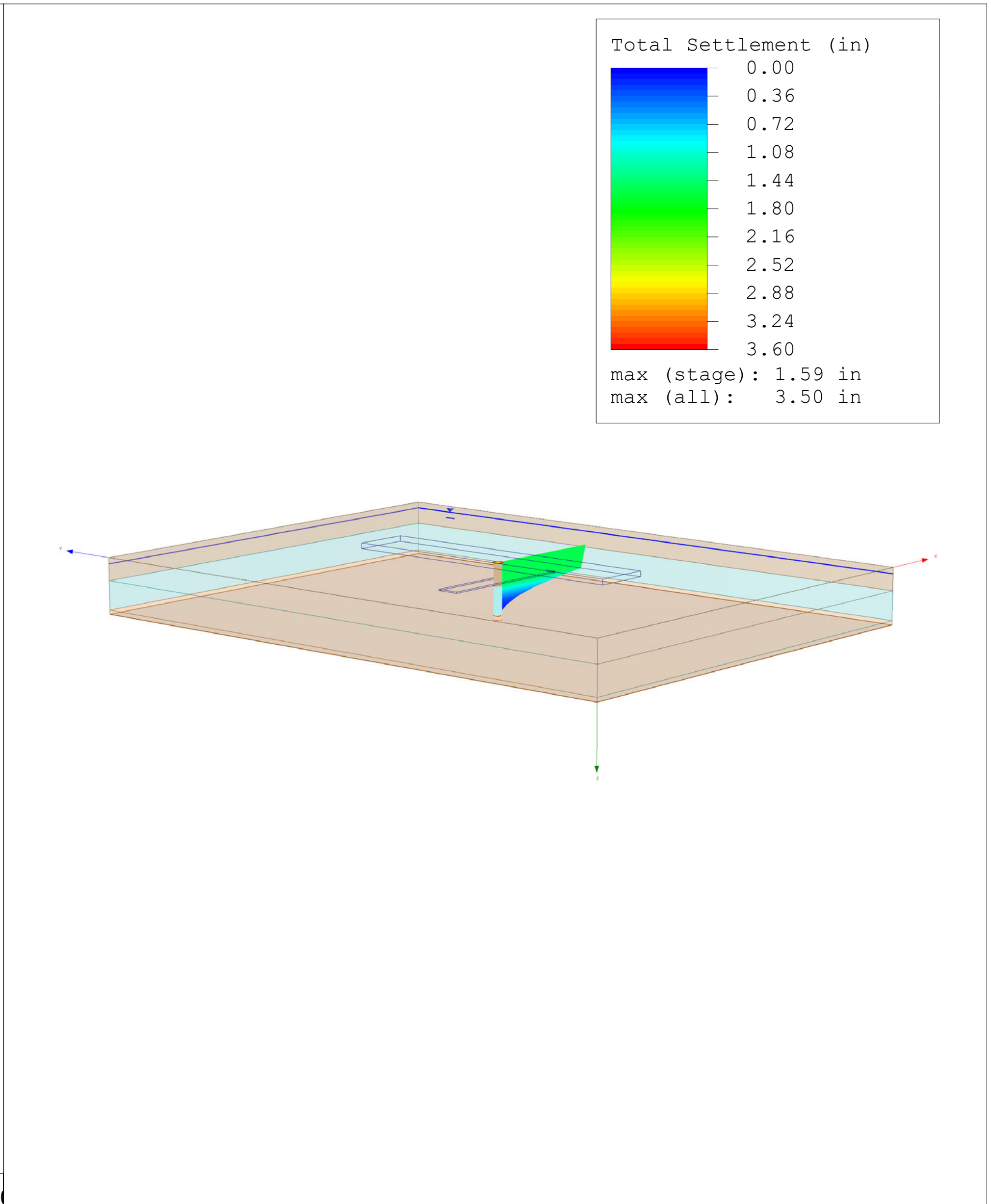
Reference Stage: None





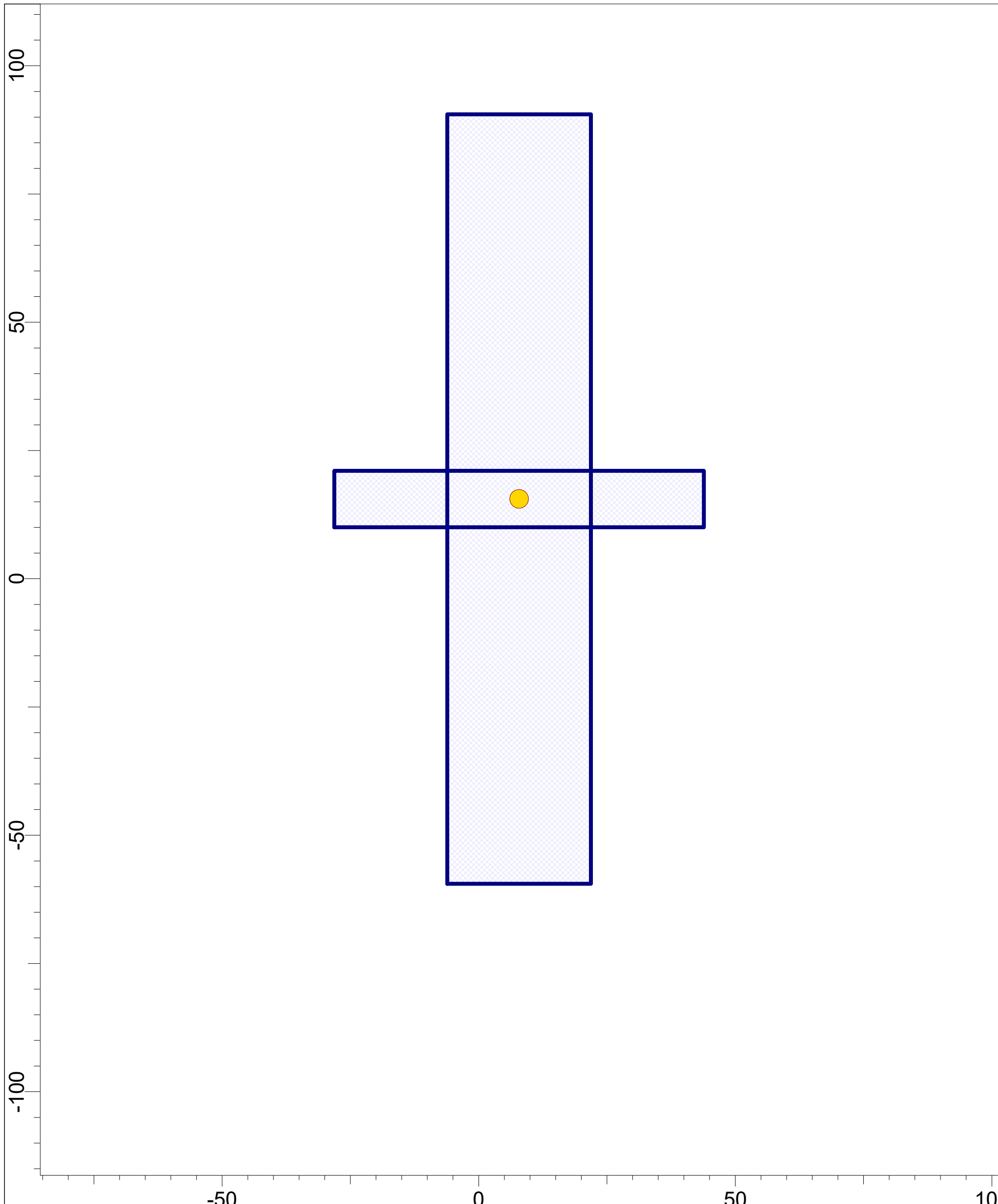
SETTLE3D 3.015	Project	27194 Orono Large Culvert	
	Analysis Description	Immediate and consolidation settlement	
	Drawn By	Yueh-Ti Lee	Company: MaineDOT
	Date	6/24/2026	File Name: 27194 Orono Settlement.s3z

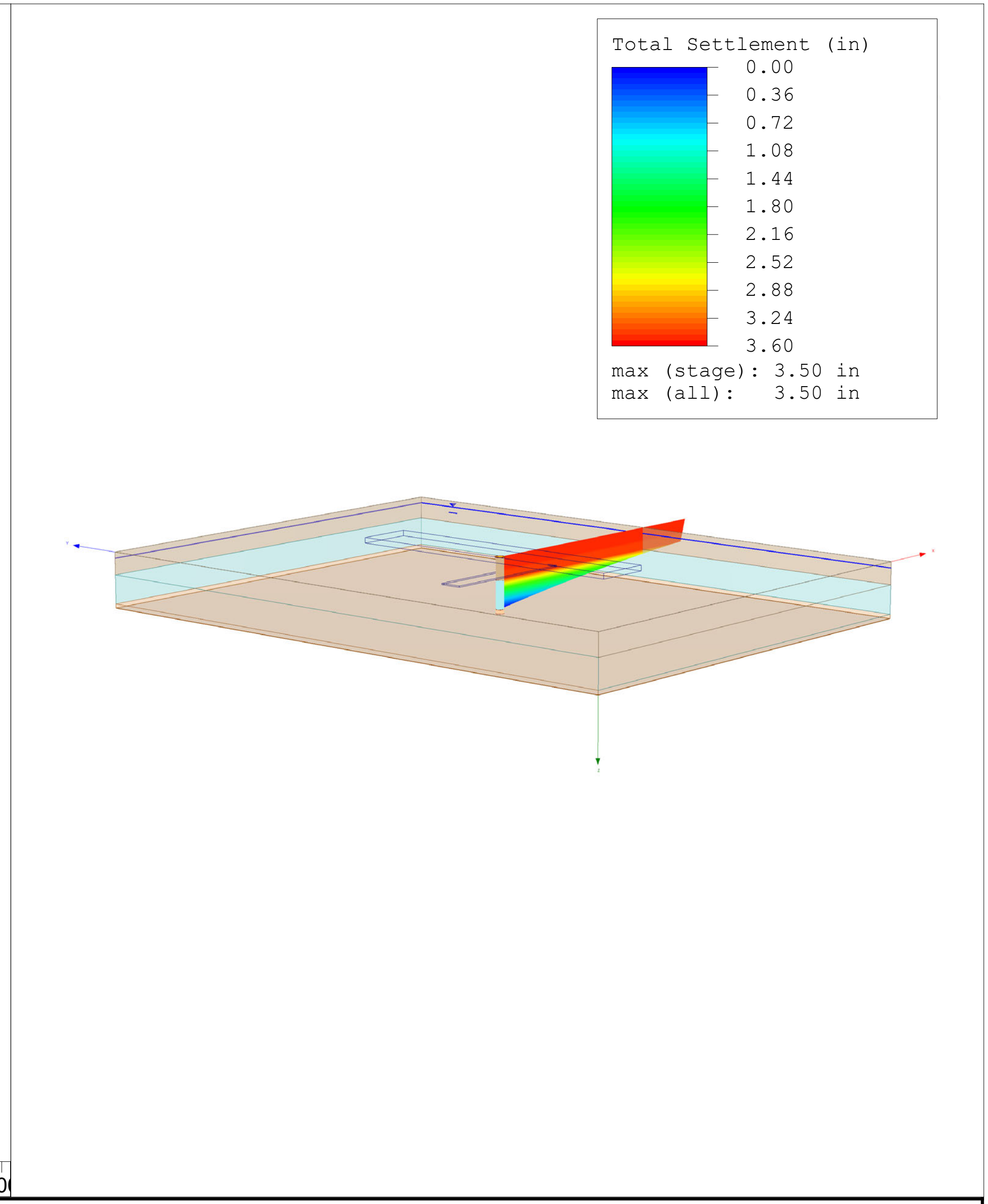




Project	27194 Orono Large Culvert		
Analysis Description	Immediate and consolidation settlement		
Drawn By	Yueh-Ti Lee	*Company*	MaineDOT
Date	6/24/2026	*File Name*	27194 Orono Settlement.s3z

SETTLE3D 3.015





Project	27194 Orono Large Culvert		
Analysis Description	Immediate and consolidation settlement		
Drawn By	Yueh-Ti Lee	Company	MaineDOT
Date	6/24/2026	File Name	27194 Orono Settlement.s3z

SETTLE3D 3.015

Bowles, Foundation Analysis and Design, 5th Ed.

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)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — — $‡E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ $*E_s = (1 + D_r^2)q_c$
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)	$†E_s = 40\,000 + 1050N$ $E_{s(\text{OCR})} \approx E_{s,\text{nc}}\sqrt{\text{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$ If $q_c < 2500$ kPa use $2500 < q_c < 5000$ use where $E_s' = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	$E_s = (1 \text{ to } 2)q_c$ $^{\$}E_s' = 2.5q_c$ $E_s' = 4q_c + 5000$
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

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

over-consolidated. The higher "past pressures", P_c, may be a result of soil unloading by erosion or water table change or may actually be caused by physio-chemical changes in the soil such as cementation or secondary compression. A soil that is in-equilibrium with existing stresses is normally consolidated.

If an over-consolidated soil is loaded, say with a large building or highway embankment, settlement will be less than if the soil was normally consolidated. Significant settlement will take place only after the past pressure is exceeded. This fact can be used to advantage by geotechnical engineers in the design of foundations on the Presumpscot Formation deposit.

Compression Index, C_c

The most commonly determined characteristic is the compression index, C_c. C_c is the slope of the portion of a void ratio versus log effective vertical pressure plot at pressures greater than the past pressures. C_c values for the Presumpscot Formation are given in Table III, Compression Index.

TABLE III

Compression Index

Location	C_c
Wells	0.38
Portland	0.4 to 0.6
Portland	0.33
Augusta	0.4
Kennebunk	0.56
Bath	0.34

The coefficient of compression varies with orientation within the soil mass. The vertical coefficient is measured by conventional testing methods. The horizontal coefficient takes special devices. The co-efficients are also a function of testing procedures.

Vertical coefficients for the Presumpscot Formation in the Portland area are reported in the 0.05 to 0.15 square feet per day range. One report gives the ratio of horizontal to vertical coefficient of 1.2 to 1.5.

With information on the coefficient of consolidation in hand, the settlement rate can be predicted <u>if</u> the drainage characteristics of the deposit, such as sand layer spacings and overlying soil permeability, are known.

<u>Over-Consolidation Ratio, OCR</u>

The Presumpscot Formation is an over-consolidated deposit. The upper crust has been significantly over-consolidated due, probably to the combined forces of dessication, drying and wetting in the presence of certain salts (Bowles, 1979), and chemical bonding. The soft, deeper deposit is also slightly over-consolidated as the result of secondary compression.

The amount of over-consolidation can be expressed as the Over-Consolidation Ratio, OCR:

$$OCR = \frac{\text{Apparent past vertical pressure, Pc}}{\text{Existing vertical pressure, P}}$$

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: **Orono, Maine**
DFI = 1750 degree-days.
Case 1 - fine grained soils W=6.3% (HB-ORO-101 1D).

For DFI = 1700
at w=10%

$d_1 := 62.2\text{in}$

For DFI = 1800
at w=10%

$d_2 := 64.0\text{in}$

Depth of Frost Penetration

$$d := \frac{d_1 + d_2}{2} = 5.3 \cdot \text{ft}$$

5.2 General

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

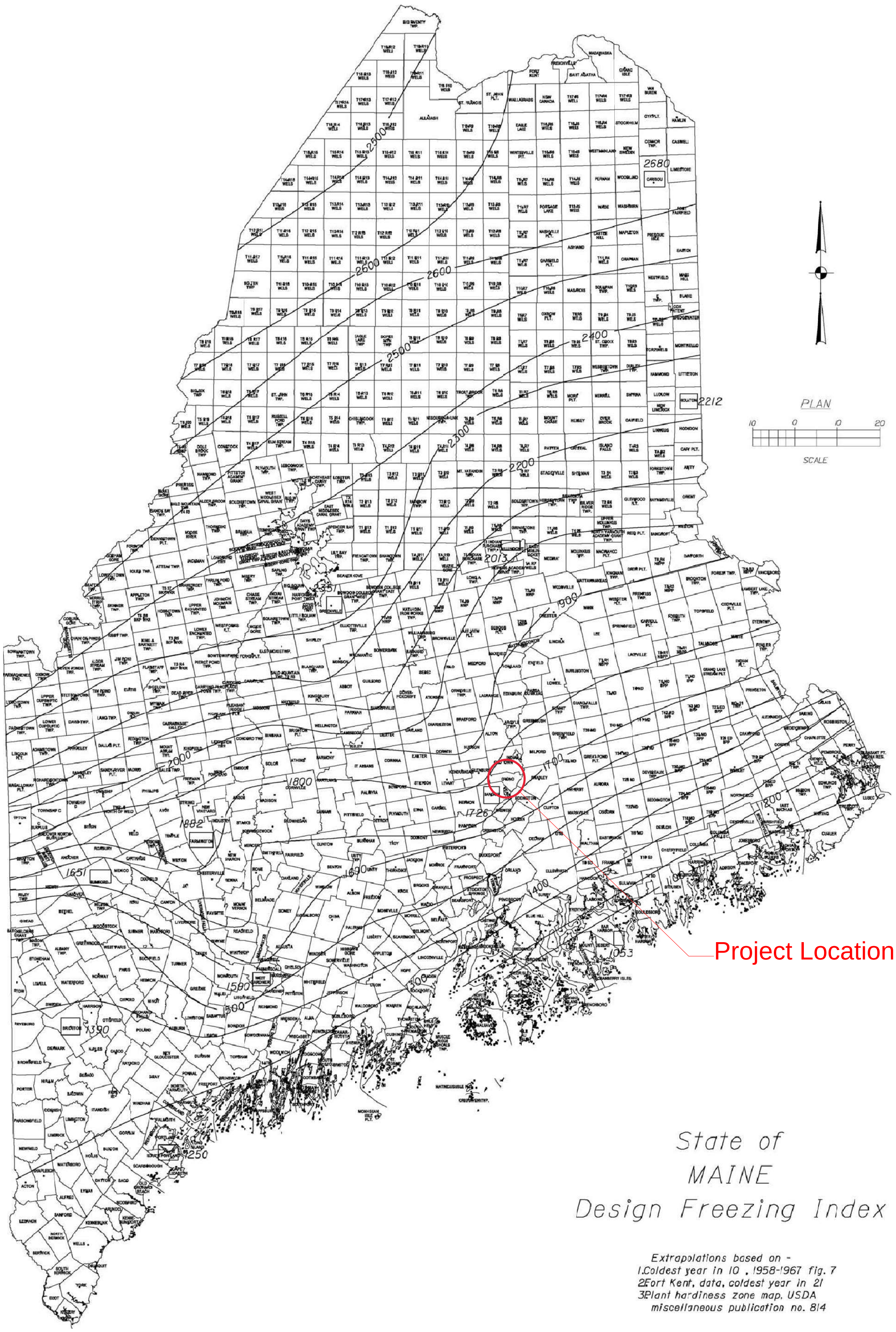


Figure 5-1 Maine Design Freezing Index Map

<u>Appendix D</u>

Special Provision 620 – Geotextile (Reinforcement Geogrid)

SPECIAL PROVISION
SECTION 620 – GEOTEXTILES
(Reinforcement Geogrid)

Amend Standard Specification 620 – GEOTEXTILES to include the following:

620.01 Description This work shall consist of furnishing and installing Reinforcement Geogrid within the Culvert Bedding Stone in accordance with these specifications and in reasonably close conformity with the lines, grades, and dimensions shown on the plans or as directed by the Resident.

620.02 Material Reinforcement Geogrid shall consist of a regular network of integrally connected, polymeric tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil, aggregate or other material. The Reinforcement Geogrid structure shall be dimensionally stable to retain its geometry under construction stresses and shall have high resistance to damage during construction, ultraviolet degradation, and all forms of chemical and biological degradation encountered in the soil being reinforced.

The Reinforcement Geogrid shall meet or exceed the Minimum Average Roll Values (MARV) of the properties in Table 1. Acceptable manufacturers for Reinforcement Geogrids must be approved by the Resident.

Table 1 - Physical Property Requirements
(Biaxial Reinforcement Geogrid)

Reinforcement Geogrid Mechanical Property	Test Method	Minimum Average Roll Value (MARV)[1]
Tensile strength at 5% Strain MD or XD	ASTM D 6637	1,200 lb/ft
Rib Junction Strength	GRI-GG2	1,000 lb/ft in both directions
Aperture Openings		Between 0.75 and 3 inches
Percent Open Area		50 to 80%

[1] Values are minimum average roll values determined in accordance with ASTM D 4759

A biaxial Reinforcement Geogrid shall be used in this application.

620.03 Placement Reinforcement Geogrid shall be installed, in accordance with the manufacturer's recommendations, unless otherwise modified by this Special Provision. The Reinforcement Geogrid shall be placed within the layers of Crushed Stone Bedding at the proper elevation and alignment as shown on the Plans or as directed by the Resident.

1. The Reinforcement Geogrid shall be placed in continuous longitudinal strips. Splicing along the length will not be allowed. Reinforcement Geogrid shall be oriented such that the roll length runs either parallel or perpendicular to the construction centerline. The Contractor shall verify correct orientation of the Reinforcement Geogrid.

2. Reinforcement Geogrid may be temporarily secured in-place with staples, pins, sand bags or backfill as required by fill properties, fill placement procedures, or weather conditions, or as directed by the Resident.

3. Coverage of less than 100 percent shall not be allowed.

4. The Reinforcement Geogrid shall be lightly anchored and pulled taut to reduce any slack as directed by the Resident.

5. Fill shall not be dumped directly onto the Reinforcement Geogrid. It shall be dumped at the edge of the Reinforcement Geogrid or on a previous course of fill with a minimum compacted depth of 8 inches.

6. The Reinforcement Geogrid shall be covered with fill materials within 7 days of placement to protect against unnecessary exposure.

7. Fill may then be pushed onto the Reinforcement Geogrid using a track mounted bulldozer. At no time shall construction equipment be allowed directly onto the Reinforcement Geogrid. Track mounted equipment shall be allowed on previous courses of fill with a minimum compacted depth of 8 inches. Smooth drum roller compaction equipment shall be allowed on previous courses of fill with a minimum compacted depth of 8 inches and spread fill with a minimum depth of 12 inches, loose measure. At no time shall rubber tired or sheeps-foot rollers be allowed onto the reinforced fill. Turning of vehicles should be kept to a minimum to prevent tracks from displacing the fill and damaging the Reinforcement Geogrid. Sudden breaking and sharp turning shall be avoided. Equipment speeds over 10 MPH shall not be allowed.

8. Placement, spreading, and compaction of soil on top of the Reinforcement Geogrid shall advance from one end of the Reinforcement Geogrid and move towards the other. Care shall be taken to minimize the development of wrinkles and to ensure that the Reinforcement Geogrid doesn't move from its position during fill placement. A spotter shall observe all fill placement operations to ensure the Reinforcement Geogrid does not slip, achieves the minimum coverage specified on the Plans, and is not damaged by the work.

9. Fill shall be compacted as specified in (1) the Standard Specifications or (2) to at least 90 percent of the maximum dry density determined in accordance with AASHTO T-180, whichever is greater. Density testing shall be made at a minimum frequency of one (1) test per lift or as otherwise specified in the Standard Specifications. Care shall be taken not to drive test apparatus through the Reinforcement Geogrid tensile elements.

10. All rutting formed during construction shall be filled with new Culvert Bedding Stone. In no case shall rutting be filled by blading down

<u>620.04 Overlap</u> Adjacent rolls of Reinforcement Geogrid shall be overlapped a minimum of 1 foot.

<u>620.05 Seams</u> Seams along adjacent lengths of Reinforcement Geogrid shall be tied together with hog rings or cable ties every 3 to 6 feet.

<u>620.06 Certification</u> Prior to construction the Contractor shall submit to the Resident the Manufacturer's certification that the Reinforcement Geogrid supplied has been evaluated in full compliance with this Specification and is fit for long-term, critical soil reinforcement applications.

The Contractor's submittal package shall include, but not be limited to, actual tests for tension/creep, durability/aging, construction damage, and quality control tensile testing.

620.08 Shipment, Storage, Protection, and Repair of Fabric The Contractor shall check the Reinforcement Geogrid upon delivery to ensure that the proper material has been received. Each Reinforcement Geogrid roll shall be shipped in a protective bag and clearly marked with roll number, lot number, geogrid style and principle strength direction. During all periods of shipment and storage, the Reinforcement Geogrid shall be protected from temperatures greater than 140°F and all deleterious materials that might otherwise become affixed to the Reinforcement Geogrid and effect its performance. The manufacturer's recommendations shall be followed with regard to protection from direct sunlight. The Reinforcement Geogrid shall be stored off the ground in a clean, dry environment out of the pathway of construction equipment.

Any Reinforcement Geogrid damage shall be repaired or replaced in accordance with the manufacturer's recommendations. The Contractor shall replace any Reinforcement Geogrid damaged during installation at no additional cost to the Department.

620.09 Method of Measurement Reinforcement Geogrid will be measured by the number of Square Yards of surface area installed. Overlaps for connections, splices, patches, and repairs of damaged Reinforcement Geogrid, etc. are incidental to this Pay Item.

620.10 Basis of Payment Reinforcement Geogrid placement will be paid for per Square Yard in-place which shall be full compensation for all off-loading, inspection, storage, labor, materials, equipment, tools and any incidentals to complete the installation.

Payment will be made under:

Pay Item	Pay Unit
620.65 Reinforcement Geogrid	Square Yard