MAINE DEPARTMENT OF TRANSPORTATION
HIGHWAY PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Construction of

PADDY HILL ROAD BRIDGE
PADDY HILL ROAD
MEDFORD, MAINE

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Piscataquis County
WIN 27222.00

Soils Report 2026-23
Bridge No. 6789

April 27, 2026

Additional review of this report provided by GZA, see stamped memo dated May 2026.

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Appendices

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the replacement of an existing cross culvert (#120362) which carries Paddy Hill Road over an unnamed stream in Medford, Maine. A subsurface investigation has been completed at the site to evaluate subsurface conditions and to develop geotechnical design and construction recommendations for the replacement structure. This report presents the subsurface information obtained during the subsurface investigation and soil laboratory testing programs and provides geotechnical design parameters and design and construction recommendations for the culvert replacement.

The existing structure consists of an approximately 48-inch diameter, 50-foot-long corrugated metal pipe (CMP) culvert. An approximately 30-inch, 50-foot-long CMP overflow was added adjacent to the main culvert following a past washout. 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. Paddy Hill Road is a Highway Corridor Priority 4 road.

The proposed replacement structure will be an approximately 12-foot span by 7-foot rise by 98-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 8.2 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 4 feet along the centerline and up to 8 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 GEOLOGIC SETTING

The existing culvert carries an unnamed stream under Paddy Hill Road in Medford and is located approximately 0.03 miles south of Hathorn Road as shown on Sheet 1 – Location Map.

According to the Maine Geological Survey (MGS) map titled Surficial Geology of the Boyd Lake Quadrangle, Maine, Open File 81-5 (1981) the surficial soils at the site consist of Presumpscot Formation and Till. Presumpscot Formation consists of sand, silt, and clay. Till consists of sand, silt, clay, and stones.

According to the map titled Bedrock Geologic Map of Maine (1985) published by the MGS, the bedrock in the vicinity of the site consists of interbedded pelite, limestone, and dolostone of the Sangerville Formation.

3.0 SUBSURFACE INVESTIGATION

One (1) boring (HB-MED-101) and one (1) probe (HB-MED-102) were drilled for this project on February 20, 2024 by S.W. Cole using a track-mounted drill rig. Boring HB-MED-101 was drilled behind the existing culvert at the northwest corner. Probe HB-MED-102 was drilled behind the existing culvert at the southeast corner. Boring and probe terminated at depths ranging from 22.0 to 25.1 feet. Exploration locations are shown on Sheet 2 – Boring Location Plan. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on the Boring Logs in Appendix A.

Boring HB-MED-101 was drilled using hollow stem auger drilling 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 S.W. Cole drill rig is equipped with an automatic hammer to drive the split spoon. All N-values discussed in this report are corrected values (N_{60}) computed by applying an average energy transfer factor of 1.087 to the raw field N-values. This hammer efficiency factor (1.087) and both the raw field N-value and corrected N-value (N60) are shown on the Boring Logs in Appendix A. Field vane shear testing was attempted in Boring HB-MED-101 at depths of 15 feet and 20 feet. However, both tests failed to obtain measurable shear strength. Probe HB-MED-102 was drilled using hollow stem auger and hydraulic push 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. A NorthEast Transportation Training and Certification (NETTCP) certified Subsurface Investigator logged the subsurface conditions encountered. The boring and probe were located in the field by taping to surveyed site features after completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test boring to assist in soil classification, evaluation of engineering properties of the soils and geologic assessment of the project site. Laboratory testing consisted of two (2) standard grain size analyses with natural water content, three (3) standard grain size analyses with hydrometer and natural water content, and three (3) Atterberg Limits tests. The results of the laboratory testing program are discussed in the following section and are included in Appendix B – Laboratory Test Results. Laboratory test information is also shown on the Boring Logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test boring and probe generally consisted of fill, underlain by Presumpscot Formation. An interpretive subsurface profile depicting the generalized soil stratigraphy at the boring location is shown on Sheet 3 – Interpretive Subsurface Profile.

Boring HB-MED-101 and probe HB-MED-102 were drilled to depths of approximately 22.0 feet and 25.1 feet below ground surface (bgs), respectively, without encountering a refusal surface. The thickness of the Presumpscot Formation was not fully explored.

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

Approx. Depth BGS[1] (feet)	Soil Description	AASHTO[2] Classification	USCS[3]	WC%[4]
0.0 – 4.5	Fill: Brown, fine to coarse sand, some gravel, trace silt.	A-1-b	SW-SM	5.6
4.5 – >22.0	Presumpscot Formation: Grey, wet, fine sandy silt, trace gravel, trace organics. Grey, wet, silty clay, trace fine sand. Grey, wet, clayey silt, trace fine sand.	A-4 or A-6	CL	21.9 to 30.6

[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

One (1) N_{60}-value obtained in the fill was 92 blows per foot (bpf), indicating that the fill is very dense. Five (5) N_{60}-values obtained in the Presumpscot Formation ranged from 7 to 25 bpf, indicating that the Presumpscot Formation is medium stiff to very stiff in consistency. In-situ vane shear tests were conducted with Geonor rectangular vanes in the Presumpscot Formation. A 55 x 110 mm vane was used. Two (2) vane shear tests were attempted within the Presumpscot Formation and were unable to be advanced. Typically cohesive soils with a shear strength less than 1,000 psf allow advancement. Therefore, the clay is anticipated to be stiff to very stiff.

The following table summarizes the results of Atterberg Limits tests done on three (3) samples of the silty clay and clayey silt:

Boring No. and Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
HB-MED-101 3D	23.6	31	20	11	0.33
HB-MED-101 4D	22.6	27	19	8	0.45
HB-MED-101 6D	21.9	28	20	8	0.24

The plasticity indices of the samples indicate that the silty clay and clayey silt have low to medium plasticity (Burmister, 1949). The natural water content is generally within a few percent of the plastic limit. Based on Bowles, 2016, the index tests indicate that these samples are overconsolidated. The test boring and probe were not advanced to sufficient depths to conclusively determine whether the Presumpscot Formation becomes softer below the explored depths.

Therefore, the presence of softer soils at greater depths should be considered in the design and evaluation.

Groundwater was not recorded in boring and probe. However, sample 2D from boring HB-MED-101 at a depth of 5 to 7 feet bgs was described as wet, which may indicate the presence of groundwater or perched water conditions at that depth. Groundwater levels can be expected to fluctuate subject to seasonal variations, local soil conditions, topography, precipitation, and construction activity.

6.0 FOUNDATION ALTERNATIVE

A precast concrete box culvert was the only bridge 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.

7.0 GEOTECHNICAL DESIGN AND CONSTRUCTION RECOMMENDATIONS

The following sections discuss geotechnical recommendations for the design and construction of the proposed culvert.

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 12-foot span by 7-foot rise by 98-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, stabilization/reinforcement geotextile wrapped crushed stone mat (Culvert Bedding Stone; Pay Item 203.55). The stabilization/reinforcement geotextile should be hand-deployed on the prepared soil subgrade prior to installing the stone mat. The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type C Underdrain Backfill material (i.e., Culvert Bedding Stone). 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 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.

7.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will not be included in the culvert design.

7.1.2 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.

7.1.3 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/Clay soils at/below the bottom of the Culvert Bedding Stone and riprap apron materials.

7.1.4 Bearing Resistance

To provide a stable subgrade, it is recommended that the precast concrete box culvert be underlain by a 2-foot-thick layer of Culvert Bedding Stone wrapped in 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 Appendix C – Calculations.

Limit State	Resistance Factor φ_b	AASHTO LRFD Reference	Factored Bearing Resistance (ksf)
Service	1.0	Article 10.5.5.1	4.0
Strength	0.5	Table 10.5.5.2.2-1	5.0

7.1.5 Modulus of Subgrade Reaction

A modulus of subgrade reaction (k_s) equal to 45 pounds per cubic inch may be used for the structural design of the box culvert's base slab. Calculations are included in Appendix C – Calculations.

7.2 Settlement

The proposed box culvert will bear on a deposit of very stiff Presumpscot Formation. The full thickness of the Presumpscot Formation is unknown and may be underlain by a softer material. In the settlement calculation, an additional 20-foot-thick layer of lightly overconsolidated Presumpscot Formation was considered below the bottom of the boring. 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 500 psf beneath the road from the new fill and a reduction of 220 psf below the 15-foot wide proposed culvert. A combined elastic and consolidation settlement on the order of 1.0-inch is estimated within the first year of construction. An additional 2.0-inches of long-term settlement (consolidation and secondary) is anticipated over the remaining 50-year service life.

7.3 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, Medford has a design freezing index (DFI) of approximately 1950 F-degree days. A water content of 10 % was used for fine-grained soils. These components correlate to a frost depth of 5.6 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 5.6 feet for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.4 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 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.

7.5 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.

8.0 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/Clay 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 river 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 very stiff Silt/Clay. Any unsuitable soils (i.e. low strength silts and clays, loose sands, and organic material) encountered at the subgrade elevation should be excavated to expose competent, firm material and replaced with compacted granular borrow.

The saturated Silt/Clay 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/ Clay 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.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Highway Program for specific application to the proposed replacement of an existing cross culvert (#120362) under Paddy Hill Road in Medford, 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

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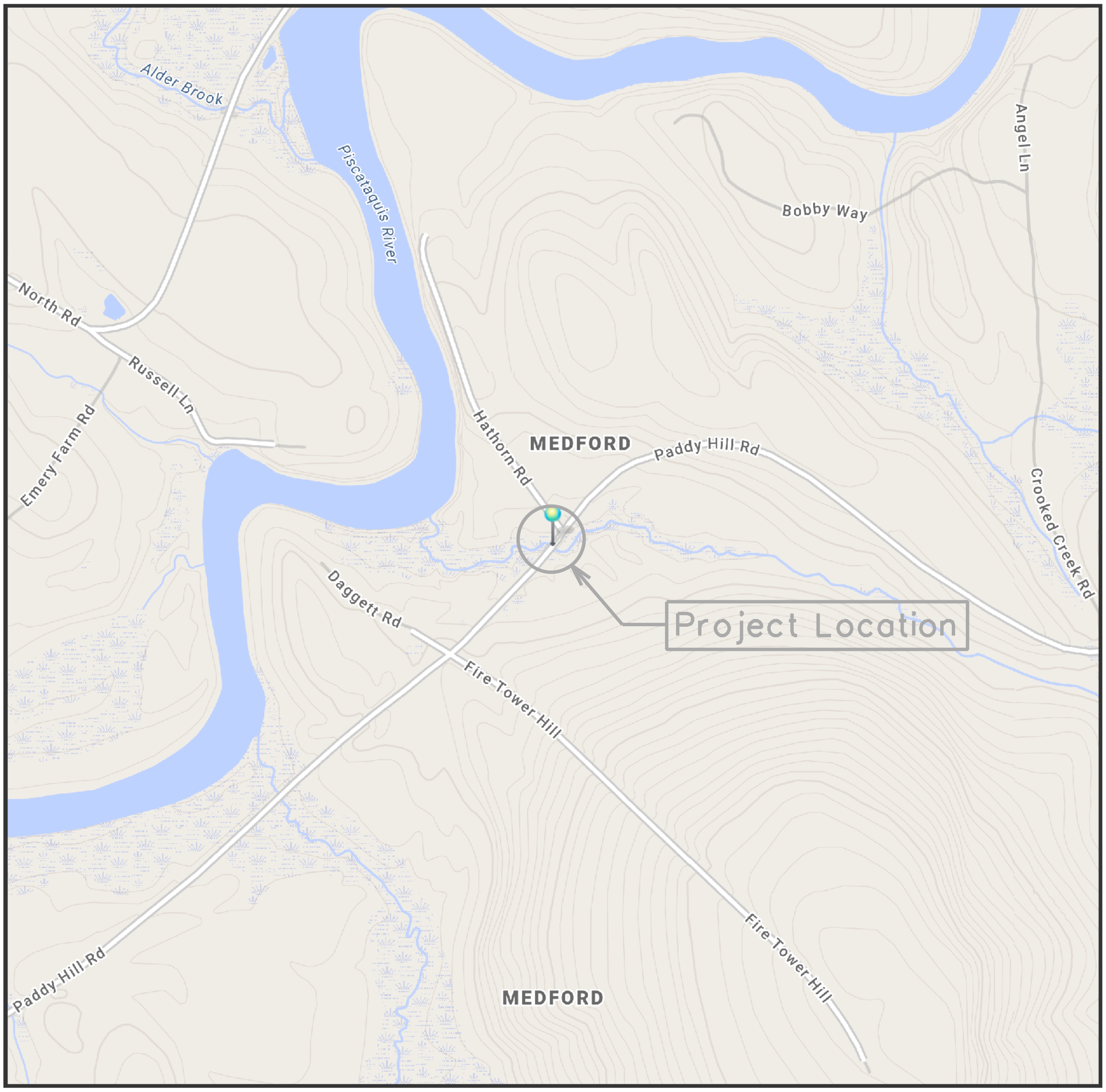
Username: Terry.White

MEDFORD, MAINE



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0.25 Miles

1 inch =0.28 miles

Date: 2/19/2026
Time: 7:16:12 AM

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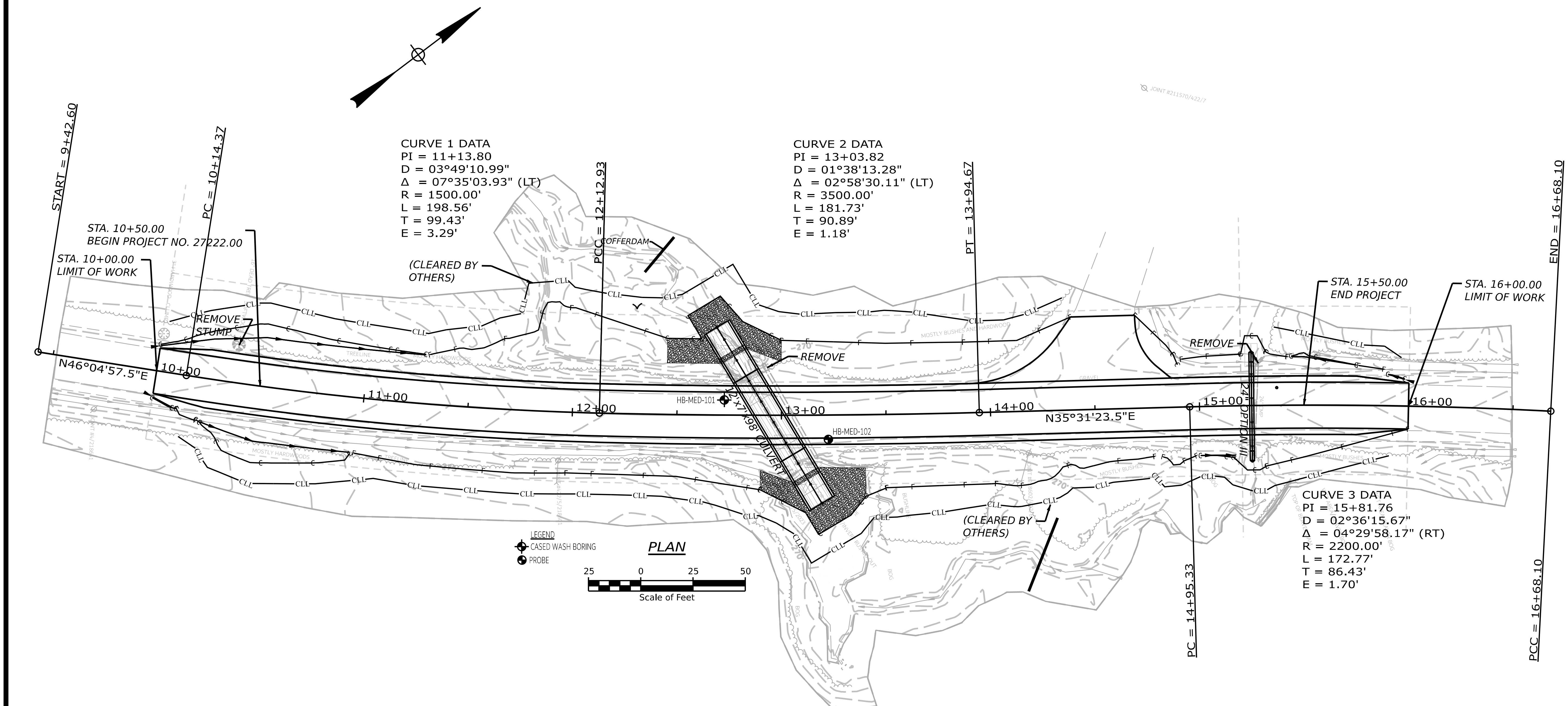
OF 3	LOCATION MAP	WIN 27222.00 HIGHWAY PLANS



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2722200

WIN
27222.00

HIGHWAY PLANS

PROJ. MANAGER	LAURIE ROWE		
DESIGN-DETAILED			
CHECKED-REVIEWED			
DESIGN2-DETAILED2	T LEE	T WHITE	APR 2026
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MEDFORD
PADDY HILL ROAD
BORING LOCATION PLAN

SHEET NUMBER

2

OF 3

Username: Yueh-Ti.Lee Date: 4/23/2026

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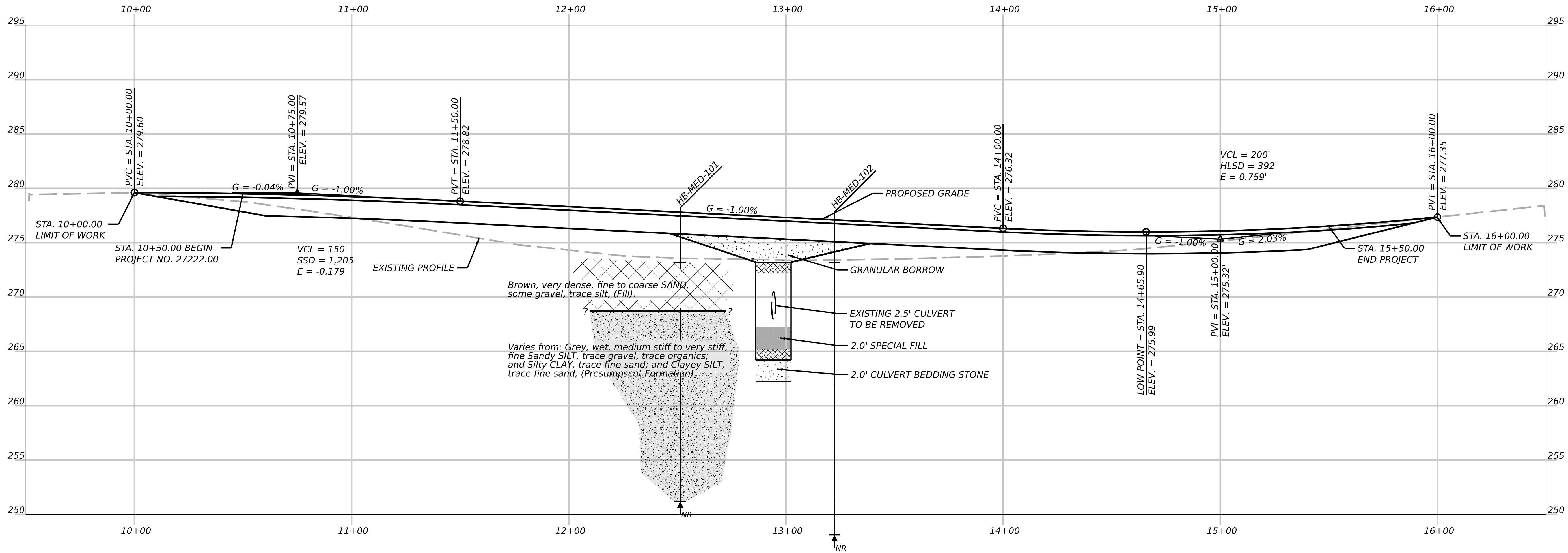

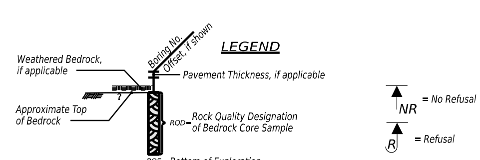

PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

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.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2722200

WIN
27222.00

HIGHWAY PLANS

SIGNATURE

P.E. NUMBER

DATE

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

MEDFORD
PATTY HILL ROAD
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

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Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

RQD (%) = sum of the lengths of intact pieces of core* > 4 inches / 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: Patty Hill Road Large Culvert
Location: Medford, Maine

Boring No.: HB-MED-101
WIN: 27222.00

Driller:	S.W. Cole	**Elevation (ft.)**	273.2	**Auger ID/OD:**	2.25/6.25"
Operator:	Ryan/Dillon	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B.Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/20/2024; 08:30-10:30	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	12+72.6, 7.2 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	1.087	**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							HSA				
	1D	24/17	2.00 - 4.00	38/33/18/15	51	92				Brown, (frozen, very dense), fine to coarse SAND, some gravel, trace silt, (Fill). Frozen soil was encountered within the sample frome 2.0 to 3.0 ft bgs.	G#379720 A-1-b, SW-SM WC=5.6%
								268.7		4.5	
5	2D	24/15	5.00 - 7.00	1/2/2/2	4	7				Grey, wet, medium stiff, fine Sandy SILT, trace gravel, trace organics, (Presumpscot Formation).	G#379721 A-4, CL WC=30.6%
								264.7		8.5	
10	3D	24/20	10.00 - 12.00	3/4/5/6	9	16				Grey, wet, very stiff, Silty CLAY, trace fine sand, (Presumpscot Formation).	G#379722 A-6, CL WC=23.6% LL=31 PL=20 PI=11
15	4D	24/22	15.00 - 17.00	3/4/6/6	10	18				Grey, wet, very stiff, Clayey SILT, trace fine sand, (Presumpscot Formation).	G#379723 A-4, CL WC=22.6% LL=27 PL=19 PI=8
	MV		15.00 - 15.20	Would not push						Failed 55x110 mm vane attempt.	
	5D	24/21	17.00 - 19.00	5/7/7/8	14	25				Grey, wet, very stiff, Clayey SILT, trace fine sand, (Presumpscot Formation).	
20	MV	24/24	20.00 - 20.20	Would not push	9	16				Failed 55x110 mm vane attempt.	G#379724 A-4, CL WC=21.9% LL=28 PL=20 PI=8
	6D		20.00 - 22.00	3/4/5/5						Grey, wet, very stiff, Clayey SILT, trace fine sand, (Presumpscot Formation).	
								251.2		22.0 **Bottom of Exploration at 22.0 feet below ground surface.** NO REFUSAL	
25											

Remarks:

Auto Hammer #362

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Patty Hill Road Large Culvert
Location: Medford, Maine

Boring No.: HB-MED-102
WIN: 27222.00

Drilling Contractor:	S.W. Cole	**Elevation (ft.)**	273.2	**Auger ID/OD:**	2.25/6.25"
Operator:	Ryan/Dillon	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B.Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	2/20/2024-2/20/2024	**Drilling Method:**	Hollow Stem Auger/ Hyd. Push	**Core Barrel:**	N/A
Boring Location:	13+22.4, 11.6 ft Rt.	**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.
0						HSA			Probe, no material samples taken.	
5										
10										
15						HP			Switched from HSA to hydraulic push of drill rods at 15.0 ft bgs. Hydraulic Push drill rods to 25.1 ft bgs.	
20										
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.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Patty Hill Road Large Culvert
Location: Medford, Maine

Boring No.: HB-MED-102
WIN: 27222.00

Drilling Contractor:	S.W. Cole	**Elevation (ft.)**	273.2	**Auger ID/OD:**	2.25/6.25"
Operator:	Ryan/Dillon	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B.Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	2/20/2024-2/20/2024	**Drilling Method:**	Hollow Stem Auger/ Hyd. Push	**Core Barrel:**	N/A
Boring Location:	13+22.4, 11.6 ft Rt.	**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.
25							248.1		25.1	
									Bottom of Exploration at 25.1 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.

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Medford Work Number: 27222.00

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-MED-101, 1D	12+72.6	7.2 Lt.	2.0-4.0	379720	1	5.6			SW-SM	A-1-b	0
HB-MED-101, 2D	12+72.6	7.2 Lt.	5.0-7.0	379721	1	30.6			CL	A-4	IV
HB-MED-101, 3D	12+72.6	7.2 Lt.	10.0-12.0	379722	1	23.6	31	11	CL	A-6	IV
HB-MED-101, 4D	12+72.6	7.2 Lt.	15.0-17.0	379723	1	22.6	27	8	CL	A-4	IV
HB-MED-101, 6D	12+72.6	7.2 Lt.	20.0-22.0	379724	1	21.9	28	8	CL	A-4	IV

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.

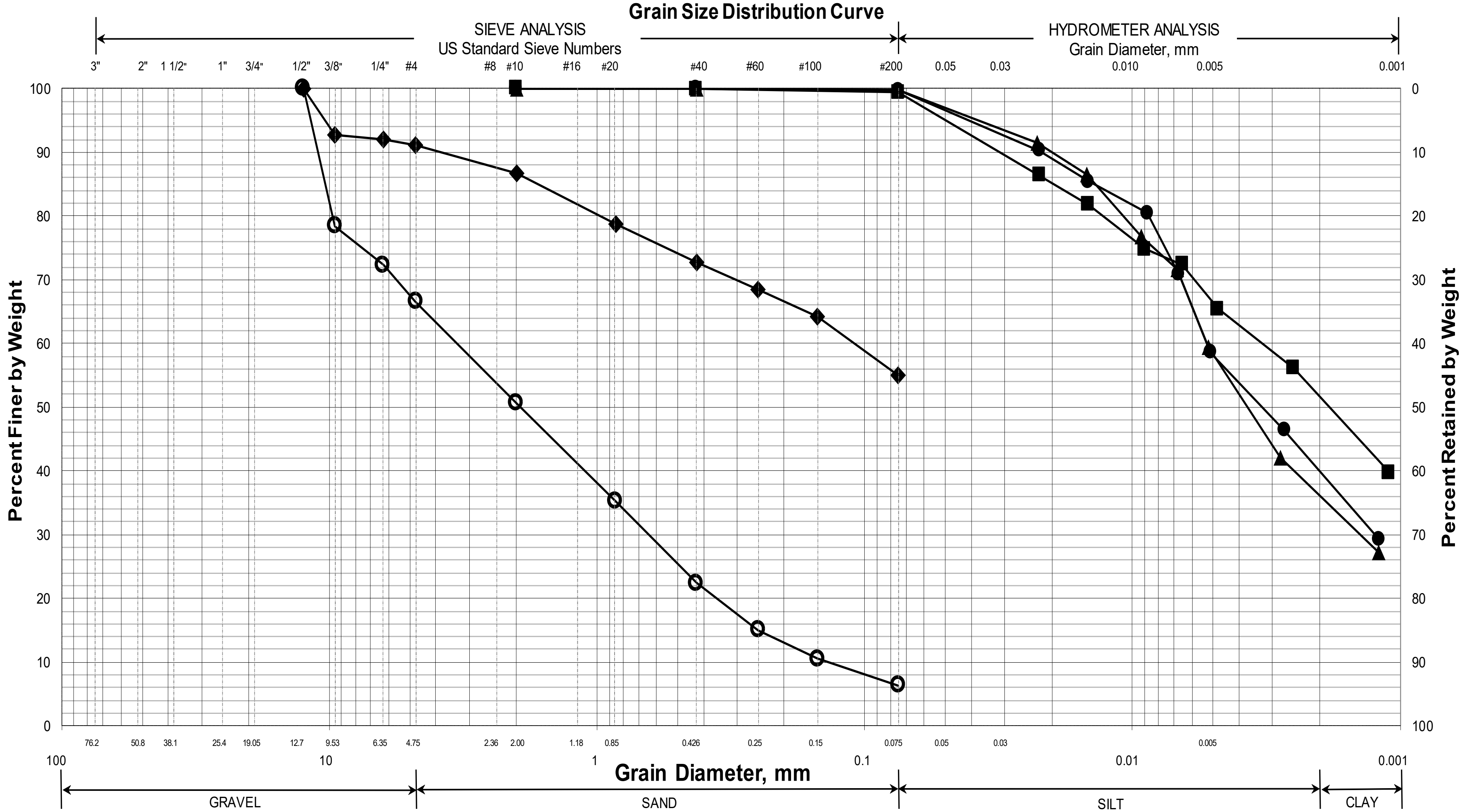
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-MED-101/1D	12+72.6	7.2 LT	2.0-4.0	SAND, some gravel, trace silt.	5.6			
◆	HB-MED-101/2D	12+72.6	7.2 LT	5.0-7.0	Sandy SILT, trace gravel.	30.6			
■	HB-MED-101/3D	12+72.6	7.2 LT	10.0-12.0	Silty CLAY, trace sand.	23.6	31	20	11
●	HB-MED-101/4D	12+72.6	7.2 LT	15.0-17.0	Clayey SILT, trace sand.	22.6	27	19	8
▲	HB-MED-101/6D	12+72.6	7.2 LT	20.0-22.0	Clayey SILT, trace sand.	21.9	28	20	8
×									

WIN	
027222.00	
Town	
Medford	
Reported by/Date	
WHITE, TERRY A	2/19/2026

SHEET 1

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
379722	HB-MED-101/3D	GEOTECHNICAL (DISTURBED)	2/20/2024	4/1/2024

Sample Type: **GEOTECHNICAL** Location: Station: **12+72.6** Offset, ft: **7.2** **LT** Dbfg, ft: **10.0-12.0**

WIN/Town **027222.00 - MEDFORD** 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]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.4
[0.0223 mm]	86.4
[0.0146 mm]	81.8
[0.0090 mm]	74.8
[0.0065 mm]	72.4
[0.0048 mm]	65.4
[0.0025 mm]	56.1
[0.0011 mm]	39.7

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	31
Plastic Limit (T 90), %	20
Plasticity Index (T 90), %	11
Specific Gravity, Corrected to 20°C (T 100)	2.72
Loss on Ignition, % (T 267)	
Water Content (T 265), %	23.6

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

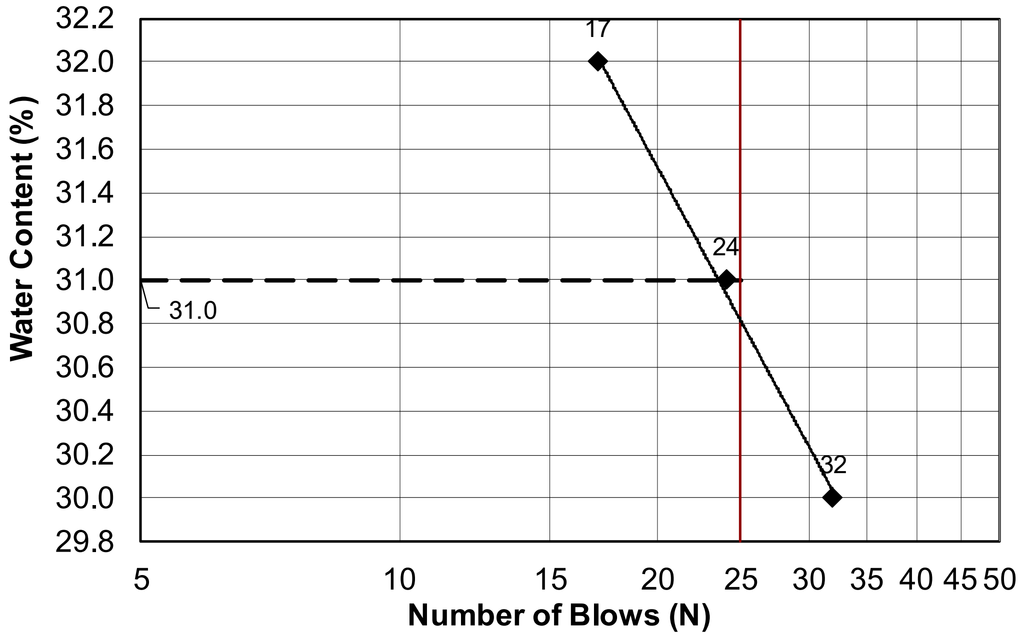
Comments:

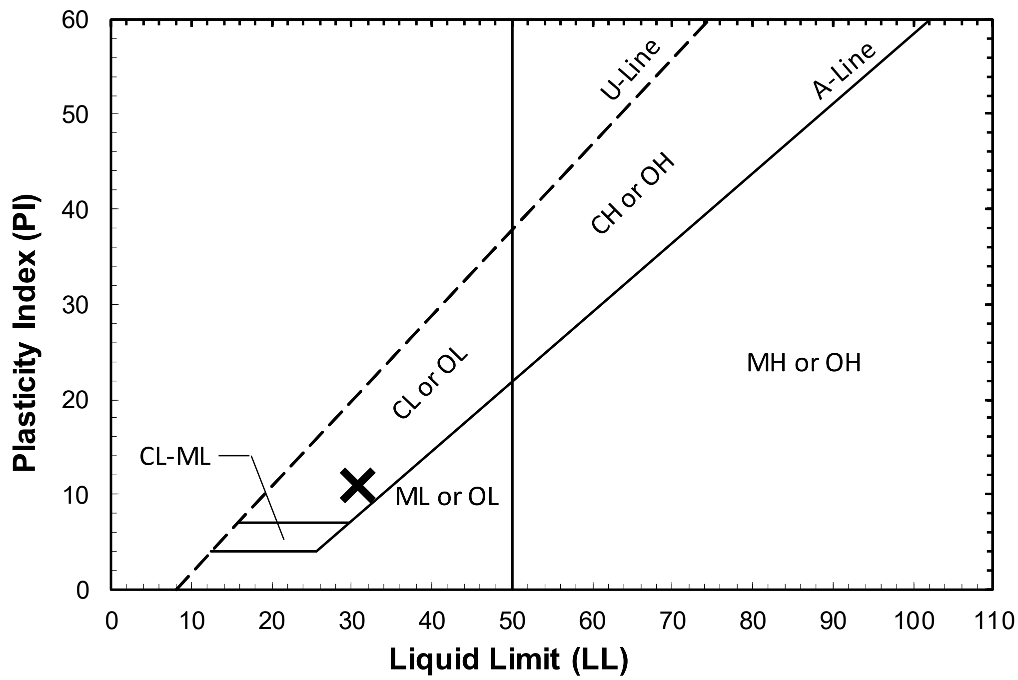
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **4/18/2024**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Medford	Reference No.	379722
WIN	027222.00	Water Content, %	23.6
Sampled	2/20/2024	Liquid Limit @ 25 blows (T 89), %	31
Boring No./Sample No.	HB-MED-101/3D	Plastic Limit (T 90), %	20
Station	12+72.6	Plasticity Index (T 90), %	11
Depth	10.0-12.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
379723	HB-MED-101/4D	GEOTECHNICAL (DISTURBED)	2/20/2024	4/1/2024

Sample Type: **GEOTECHNICAL** Location: Station: **12+72.6** Offset, ft: **7.2 LT** Dbfg, ft: **15.0-17.0**

WIN/Town **027222.00 - MEDFORD** 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]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.8
[0.0223 mm]	90.3
[0.0146 mm]	85.4
[0.0088 mm]	80.5
[0.0067 mm]	70.8
[0.0051 mm]	58.6
[0.0027 mm]	46.4
[0.0012 mm]	29.3

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	27
Plastic Limit (T 90), %	19
Plasticity Index (T 90), %	8
Specific Gravity, Corrected to 20°C (T 100)	2.72
Loss on Ignition, % (T 267)	
Water Content (T 265), %	22.6

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

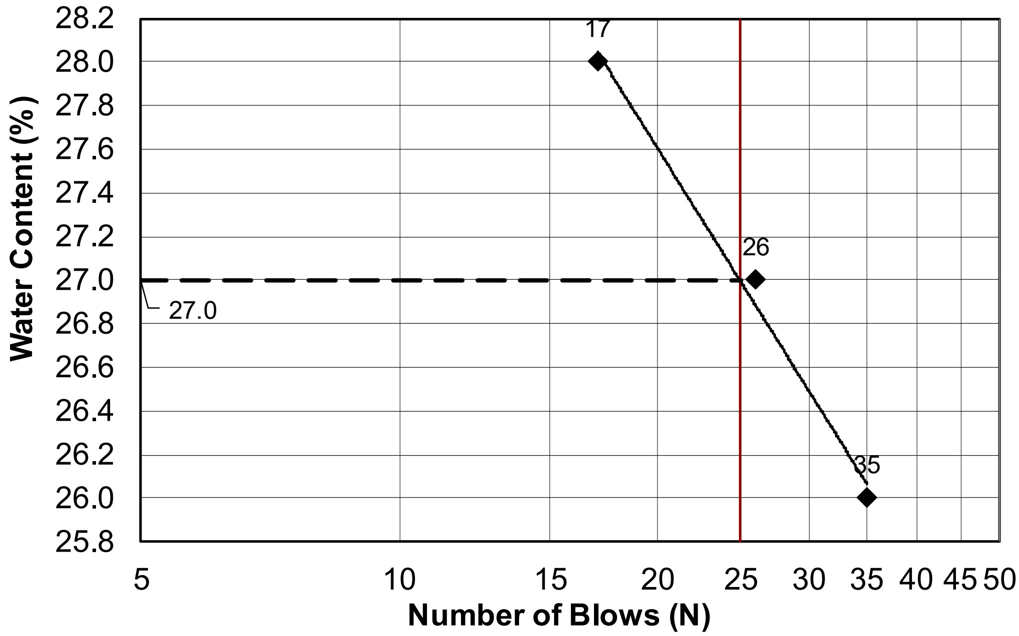
Comments:

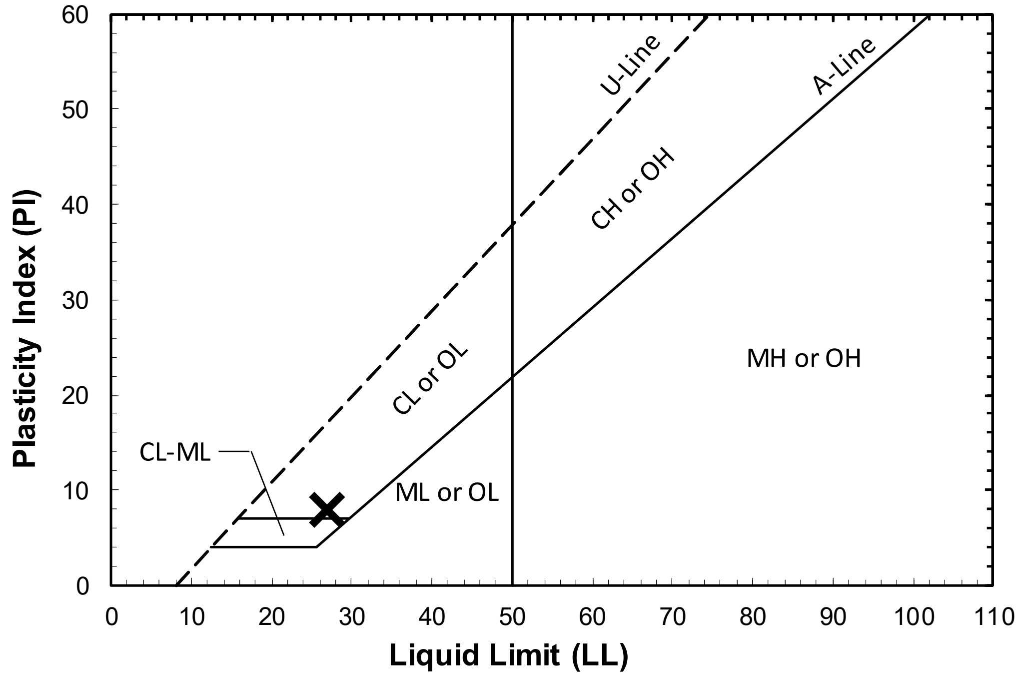
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **4/18/2024**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Medford	Reference No.	379723
WIN	027222.00	Water Content, %	22.6
Sampled	2/20/2024	Liquid Limit @ 25 blows (T 89), %	27
Boring No./Sample No.	HB-MED-101/4D	Plastic Limit (T 90), %	19
Station	12+72.6	Plasticity Index (T 90), %	8
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
379724	HB-MED-101/6D	GEOTECHNICAL (DISTURBED)	2/20/2024	4/1/2024

Sample Type: **GEOTECHNICAL** Location: Station: **12+72.6** Offset, ft: **7.2** **LT** Dbfg, ft: **20.0-22.0**

WIN/Town **027222.00 - MEDFORD** 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]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.8
[0.0227 mm]	91.5
[0.0148 mm]	86.6
[0.0093 mm]	76.7
[0.0068 mm]	71.7
[0.0052 mm]	59.4
[0.0028 mm]	42.1
[0.0012 mm]	27.2

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	28
Plastic Limit (T 90), %	20
Plasticity Index (T 90), %	8
Specific Gravity, Corrected to 20°C (T 100)	2.66
Loss on Ignition, % (T 267)	
Water Content (T 265), %	21.9

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

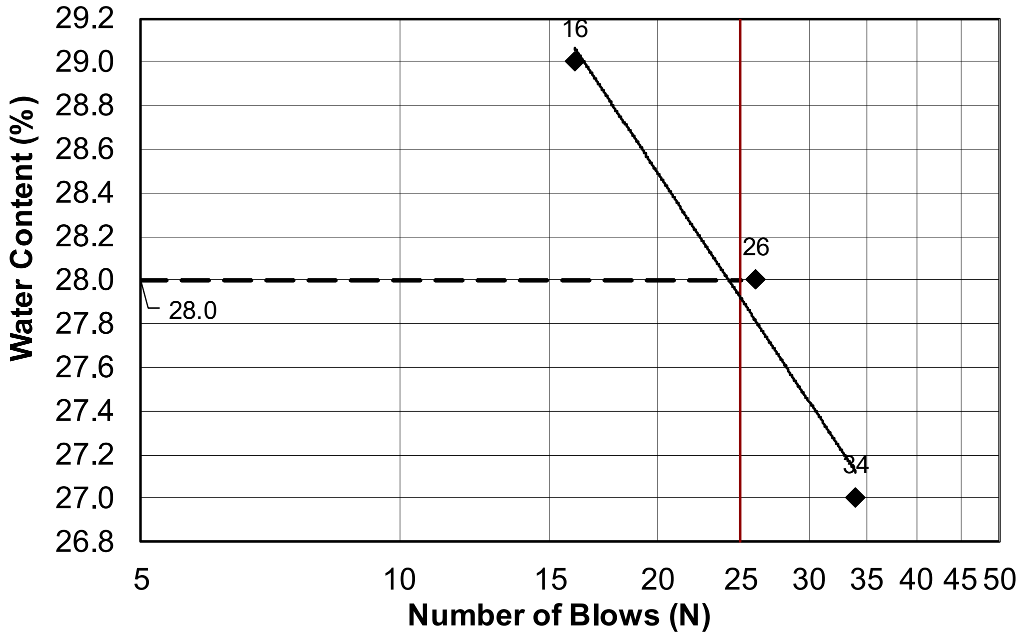
Comments:

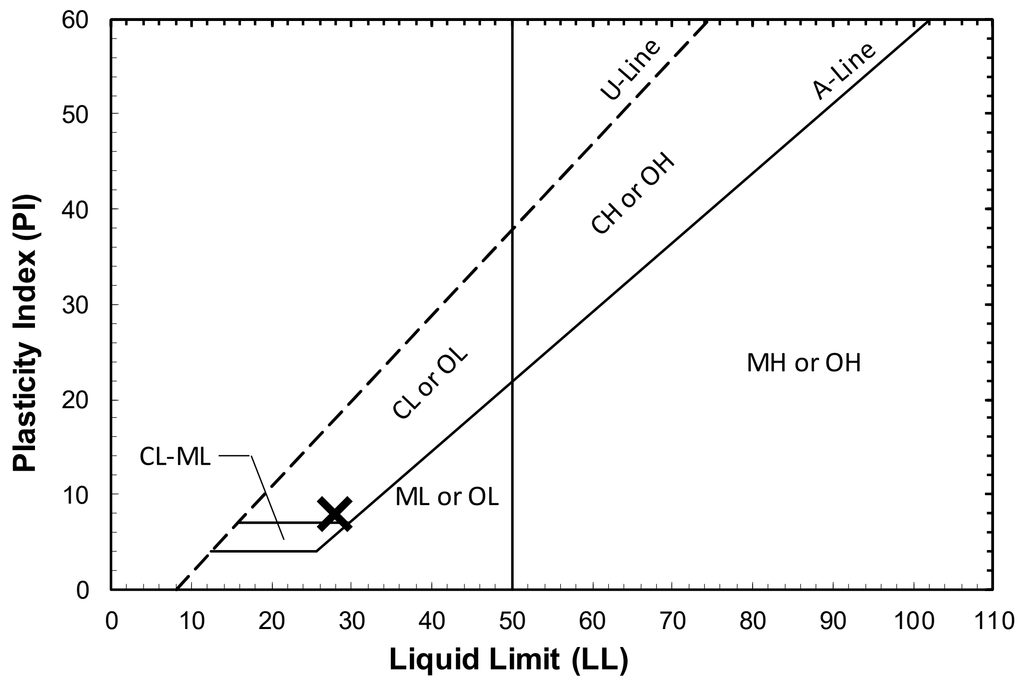
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **4/18/2024**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Medford	Reference No.	379724
WIN	027222.00	Water Content, %	21.9
Sampled	2/20/2024	Liquid Limit @ 25 blows (T 89), %	28
Boring No./Sample No.	HB-MED-101/6D	Plastic Limit (T 90), %	20
Station	12+72.6	Plasticity Index (T 90), %	8
Depth	20.0-22.0	Tested By	BBURR





Appendix C

Calculations

Liquidity Index

Liquidity Index

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

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

HB-MED-101, 3D

$WC := 23.6$

$LL := 31$

$PL := 20$

$$LI := \frac{WC - PL}{LL - PL} = 0.33$$

HB-MED-101, 4D

$WC := 22.6$

$LL := 27$

$PL := 19$

$$LI := \frac{WC - PL}{LL - PL} = 0.45$$

HB-MED-101, 6D

$WC := 21.9$

$LL := 28$

$PL := 20$

$$LI := \frac{WC - PL}{LL - PL} = 0.24$$

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

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

$K_o = 0.47$

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:

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

$K_{ar} = 0.31$

For a sloped 2H:1V backfill

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

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

$K_{ar\_slope} = 0.46$

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:

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

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

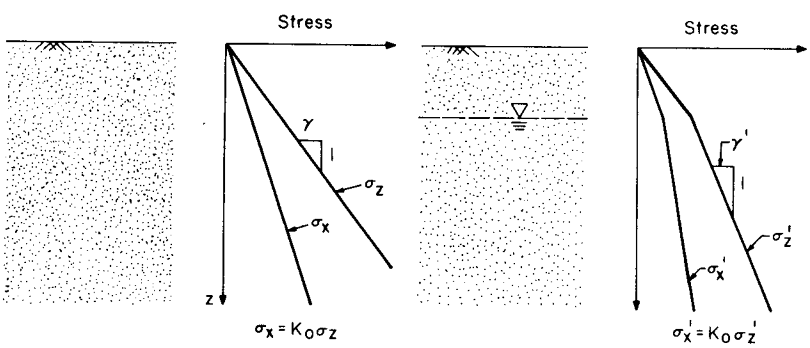


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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:

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

where:

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

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

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

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

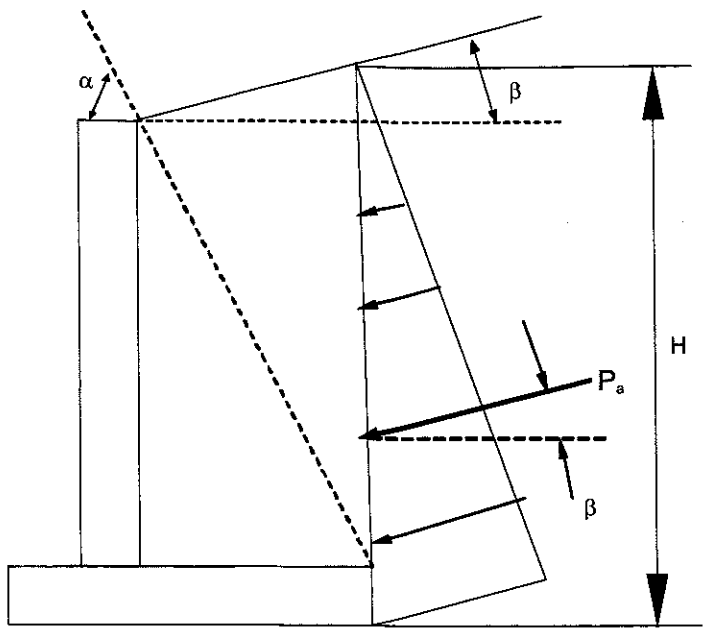


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

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 Silty Clay

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: Silty Clay (CL)

Based on N-values, soils are very stiff near the bearing elevation

Density In Place: very stiff

Bearing Resistance: Ordinary Range (ksf) 2 to 6

Recommended Value of Use: $q_{nom} := 4 \cdot ksf$

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

$q_{factored\_service\_bc} := q_{nom} \cdot \phi_{service\_bc}$ $q_{factored\_service\_bc} = 4 \cdot ksf$

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 Silty Clay

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

Assumptions:

1. The box will be founded at ~ Elev 265.20 feet

   Bottom of Construction will be 2 feet below box invert $D_{footing} := 2.0 \cdot ft$

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 $B_{box} := 15 \cdot ft$

   Length of box culvert, L $L_{box} := 98 \cdot ft$

Foundation soils:

Foundation soils properties based on HB-MED-101

$\gamma1_d := 95 \cdot pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Soft to Stiff Clay

$w_{sat} := 22.7\%$

HB-MED-101 3D, 4D, 6D 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

$\gamma1_{sat} = 116.565 \cdot pcf$

$\phi := 0 \cdot deg$

$c := 2000psf$

MaineDOT Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Approximate undrained shear stress for very stiff fine grained soils:
2000 - 4000 psf

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

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

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

Shape Factors - per LRFD Table 10.6.3.1.2a-3

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

$s_\gamma := 1$

$s_q := 1$

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

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: $D_w := 0 \cdot ft$ $C_{wq} := 0.5$ $C_{w\gamma} := 0.5$

Load Inclination factors

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

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

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

LRFD Table 10.6.3.1.2a-3 $\frac{D_{footing}}{B_{box}} = 0.13$

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

$N_{cm} = 5.2973$ $N_{\gamma m} = 0$ $N_{qm} = 1$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$q_n = 10.7 \cdot ksf$

Factored Bearing Resistance

$\phi_b := 0.5$

$q_r := q_n \cdot \phi_b$

$q_r = 5.4 \cdot ksf$

Recommend a limiting factored bearing resistance of 5.0 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	$\frac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\frac{\gamma}{1+w}$	G_s, e	$\frac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\frac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\frac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\frac{(1+w)G_s\gamma_w}{1+\frac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\frac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\frac{G_s\gamma_w}{1+\left(\frac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\frac{e}{w_{sat}}\right)\left(\frac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\frac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\frac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\frac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\frac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\frac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\frac{1}{G_s}\right)\gamma_d+\gamma_w$
				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	Dry unit weight, γ_d 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

AASHTO LRFD 10th edition, 2024

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	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:

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

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.

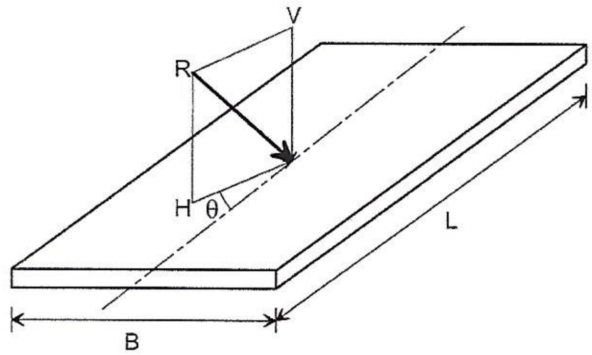


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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.

AASHTO LRFD 10th edition, 2024

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
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

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

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:

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

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

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

Modulus of Subgrade Reaction:

Reference: Foundation Analysis and Design 5th Edition JE Bowles Section 9-6

Width of box culvert, B $B_{box} = 15\,ft$

Length of box culvert, L $L_{box} = 98\,ft$

Thickness of box culvert, t $t_{box} := 12 \cdot in$ assumed

Depth of box, D $D_{box} := 12.2 \cdot ft$

Bearing Resistance: $q_{factored\_service\_bc} = 4 \cdot ksf$ Calculated above

Modulus of Elasticity: Site soils at bearing elevation are Silty Clay. Use values for Silty Clay (very stiff)
From Bowles Table 2-8 Modulus Es for Medium Clay, ranges from 313 - 1044 ksf

Use: Modulus of Elasticity, Es $E_s := 400 \cdot ksf$

Poisson's Ratio: Site conditions at bearing elevation are Silty Clay. Use values for Silty Clay (very stiff)
From Bowles Table 2-7 Poisson's Ration μ for Saturated Clay ranges from 0.4 - 0.5

Use: Possion's Ratio, μ $\mu := 0.5$

$$E_{prime\_s} := \frac{1 - \mu^2}{E_s} \qquad E_{prime\_s} = 0.001875 \cdot \frac{ft^2}{kip}$$

Analyze corner:

Take H as 5*B as recommended in Bowles Chapter 5

$$H_{inf} := \frac{5 \cdot B_{box}}{B_{box}} \qquad H_{inf} = 5$$ N in Table 5-2

$$\frac{L_{box}}{B_{box}} = 6.5333$$ M in Table 5-2

From Table 5-2 for N=5 and M=6.5

$I_1 := 0.546$

$I_2 := 0.124$

by interpolation

Determine Steinbrenner influence factor - Bowles Section 5-6:

$$I_s := I_1 + \left[\frac{1 - (2 \cdot \mu)}{1 - \mu}\right] \cdot I_2 \qquad I_s = 0.546$$

Determine Influence factor for footing depth - Bowles Figure 5-7

Depth ratio: $\frac{D_{box}}{B_{box}} = 0.8133$ $\frac{L_{box}}{B_{box}} = 6.5333$ $\mu = 0.5$ $I_F := 0.87$

Calculate modulus of subgrade reaction - Bowles Eq. 9-7

$$k_s := \frac{1}{B_{box} \cdot E_{prime\_s} \cdot I_s \cdot I_F}$$ Bowles Eq. 9-7

$$k_s = 43 \cdot pci$$

Recommend Modulus of Subgrade Reaction of 45 pci

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

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

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

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

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

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

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

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

$$E'_s = \frac{E_s}{1-\mu^2} \qquad \mu' = \frac{\mu}{1-\mu} \qquad (2\text{-}65)$$

Bowles, Foundation Design and Analysis, 5th Ed.

TABLE 2-8
Value range* for the static stress-strain modulus E_s for selected soils (see also Table 5-6)

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

Soil	E_s, MPa
Clay	
Very soft	2–15
Soft	5–25
Medium	15–50
Hard	50–100
Sandy	25–250
Glacial till	
Loose	10–150
Dense	150–720
Very dense	500–1440
Loess	15–60
Sand	
Silty	5–20
Loose	10–25
Dense	50–81
Sand and gravel	
Loose	50–150
Dense	100–200
Shale	150–5000
Silt	2–20

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

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

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

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

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

5-6 IMMEDIATE SETTLEMENT COMPUTATIONS

The settlement of the corner of a rectangular base of dimensions $B' \times L'$ on the surface of an elastic half-space can be computed from an equation from the Theory of Elasticity [e.g., Timoshenko and Goodier (1951)] as follows:

$$\Delta H = q_o B' \frac{1-\mu^2}{E_s}\left(I_1 + \frac{1-2\mu}{1-\mu} I_2\right) I_F \qquad (5\text{-}16)$$

where q_o = intensity of contact pressure in units of E_s

B' = least lateral dimension of contributing base area in units of ΔH

I_i = influence factors, which depend on L'/B', thickness of stratum H, Poisson's ratio μ, and base embedment depth D

E_s, μ = elastic soil parameters—see Tables 2-7, 2-8, and 5-6

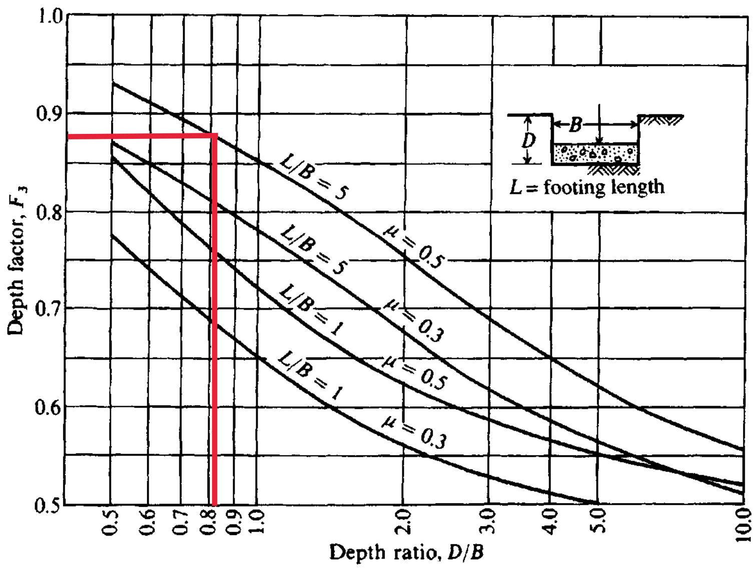
The influence factors (see Fig. 5-7 for identification of terms) I_1 and I_2 can be computed using equations given by Steinbrenner (1934) as follows:

$$I_1 = \frac{1}{\pi}\left[M \ln \frac{(1+\sqrt{M^2+1})\sqrt{M^2+N^2}}{M(1+\sqrt{M^2+N^2+1})} + \ln \frac{(M+\sqrt{M^2+1})\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}}\right] \qquad (a)$$

$$I_2 = \frac{N}{2\pi}\tan^{-1}\left(\frac{M}{N\sqrt{M^2+N^2+1}}\right) \quad (\tan^{-1} \text{ in radians}) \qquad (b)$$

where $M = \dfrac{L'}{B'}$

Figure 5-7 Influence factor I_F for footing at a depth D. Use actual footing width and depth dimension for this D/B ratio. Use program FFACTOR for values to avoid interpolation.



Bowles, Foundation Design and Analysis, 5th Ed.

TABLE 5-2
Values of I_1 and I_2 to compute the Steinbrenner influence factor I_s for use in Eq. (5-16*a*) for several $N = H/B'$ and $M = L/B$ ratios *(continued)*

N		*M* = 2.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.2	I_1 =	0.007	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
	I_2 =	0.043	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044
0.4		0.026	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024
		0.074	0.075	0.075	0.075	0.076	0.076	0.076	0.076	0.076	0.076	0.076
0.6		0.053	0.051	0.050	0.050	0.050	0.049	0.049	0.049	0.049	0.049	0.049
		0.094	0.097	0.097	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
0.8		0.086	0.082	0.081	0.080	0.080	0.080	0.079	0.079	0.079	0.079	0.079
		0.107	0.111	0.112	0.113	0.113	0.113	0.113	0.114	0.114	0.114	0.114
1.0		0.121	0.115	0.113	0.112	0.112	0.112	0.111	0.111	0.110	0.110	0.110
		0.114	0.120	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.5		0.207	0.197	0.194	0.192	0.191	0.190	0.190	0.189	0.188	0.188	0.188
		0.118	0.130	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
2.0		0.284	0.271	0.267	0.264	0.262	0.261	0.260	0.259	0.257	0.256	0.256
		0.114	0.131	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148
3.0		0.402	0.392	0.386	0.382	0.378	0.376	0.374	0.373	0.368	0.367	0.367
		0.097	0.122	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
4.0		0.484	0.484	0.479	0.474	0.470	0.466	0.464	0.462	0.453	0.451	0.451
		0.082	0.110	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
5.0		0.553	0.554	0.552	0.548	0.543	0.540	0.536	0.534	0.522	0.519	0.519
		0.070	0.098	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
6.0		0.585	0.609	0.610	0.608	0.604	0.601	0.598	0.595	0.579	0.576	0.575
		0.060	0.087	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157
7.0		0.618	0.653	0.658	0.658	0.656	0.653	0.650	0.647	0.628	0.624	0.623
		0.053	0.078	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
8.0		0.643	0.688	0.697	0.700	0.700	0.698	0.695	0.692	0.672	0.666	0.665
		0.047	0.071	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
9.0		0.663	0.716	0.730	0.736	0.737	0.736	0.735	0.732	0.710	0.704	0.702
		0.042	0.064	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
10.0		0.679	0.740	0.758	0.766	0.770	0.770	0.770	0.768	0.745	0.738	0.735
		0.038	0.059	0.071	0.082	0.091	0.099	0.106	0.112	0.147	0.156	0.158
20.0		0.756	0.856	0.896	0.925	0.945	0.959	0.969	0.977	0.982	0.965	0.957
		0.020	0.031	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
500.0		0.832	0.977	1.046	1.102	1.150	1.191	1.227	1.259	1.532	1.721	1.879
		0.001	0.001	0.002	0.002	0.002	0.003	0.003	0.003	0.008	0.016	0.031

Settlement

Objective:

1) To estimate soil parameters for Settle 3D analysis

Given:

1) Boring Log HB-MED-101 and lab test data.

Assumptions:

1) Groundwater is at El. 268.2.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 is used to construct the proposed raise roadway grade (approximately 4 feet).
3) Unless otherwise noted, HB-MED-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. (2016). 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 = 4 feet

$H_{fill} := 4ft$

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

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

$\sigma_{z\_induced} = 0.5 \cdot ksf$

Existing Ground Elevation = El. 273.2 ft

Soil Layer 1 (Elev. 273.2 - 264.7) Fill: Granular Borrow, with drainage system

$N_1 := 30$

N60 of HB-MED-101, 1D = 92 due to frozen soil, not applicable. Assumed N60 = 30

$E_{s1} := \frac{500(N_1 + 15)}{50} ksf$

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

$E_{s1} = 450 \cdot ksf$

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

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

$E_{ur1} = 1800 \cdot ksf$

$\gamma_{dry1} := 125 pcf$

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

$w_{sat1} := 5.6\%$

HB-MED-101, 1D Natural Water Content

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

$\gamma_{sat1} = 132 \cdot pcf$

Soil Layer 2 (Elev. 264.7 - 251.2) Persumpscot Formation: Silty CLAY/ Clayey SILT, trace sand

$N_{60\_2} := 16$

N60 of HB-MED-101, 3D = 16
HB-MED-101, 4D = 18
HB-MED-101, 5D = 25
HB-MED-101, 6D = 16, Conservatively use N60 =16

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

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

$E_{s2} = 132 \cdot ksf$

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

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

$E_{ur2} = 528 \cdot ksf$

$\gamma_{dry2} := 95 \cdot pcf$

Das, Table 3.2: Dry Unit Weights, Soft to Stiff Clay

$w_{sat2} := 22.7\%$

HB-MED-101, 3D, 4D, 6D mean Natural Water Content

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

$\gamma_{sat2} = 117 \cdot pcf$

$C_{c2} := 0.4$ — Andrews, Table III

$C_{r2} := 0.04$ — Assume 10% of Cc

$d_2 := 8.5ft$ — Depth to top of soil layer 2

$\sigma_{p2} := 8\ ksf$ — Shear Strength estimated in bearing calc = 2000 psf
Sansep Concept:
Assume Preconsolidation Stress = Shear Strength/0.25
= 8000 psf

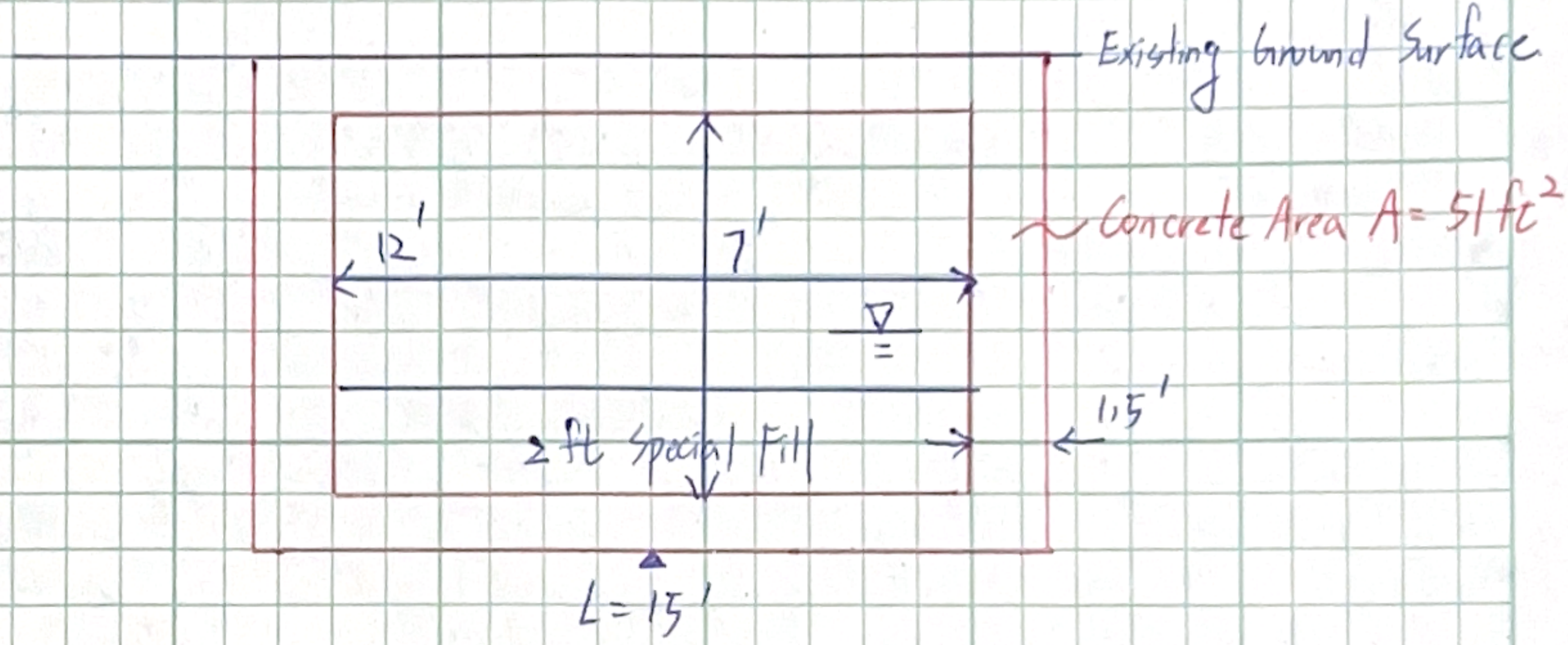
$e_2 := 1$ — Assumed

$C_{v2} := 0.15 \frac{ft^2}{day}$ — Andrews, pg. 11, Cv range from 0.05-0.15 square feet per day. Overconsolidated, choose high end.

$C_{v2} = 54.786 \cdot \frac{ft^2}{yr}$

Assumptions:

1. Fill Unit Weight $\gamma_{fill} = 125\ pcf$
2. Special Fill Unit Weight $\gamma_{special\text{-}fill\ (saturated)} = 135\ pcf$
3. Concrete Unit Weight $\gamma_{conc} = 150\ pcf$
4. Groundwater = 5' Below Ground Surface.
5. Water Unit Weight $\gamma_{water} = 62.4\ pcf$.



Existing Effective Stress:

$$\sigma'_{v\,existing} = 125 pcf \times 9 ft - 62.4 pcf \times 4 ft = 875.4\ psf$$

Proposed Effective Stress:

$$\sigma'_{v\,proposed} = \frac{150 pcf \times 51 ft^2}{15 ft} + (135 pcf - 62.4 pcf) \times 2 ft$$

$$= 655.2\ psf$$

Change in Effective Stress:

$$\Delta\sigma'_v = \sigma'_{v\,proposed} - \sigma'_{v\,existing} = 655.2\ psf - 875.4\ psf$$

$$= -220.2\ psf = -0.22\ ksf$$

#

Settle3D Analysis Information
27222 Medford Paddy Hill Road Bridge

Project Settings

Document Name	27222 Medford Settlement.s3z
Project Title	27222 Medford Paddy Hill Road Bridge
Analysis	Immediate and consolidation settlement
Author	Yueh-Ti Lee
Company	MaineDOT
Date Created	4/21/2026, 11:41:38 AM
Comments	
Delta q = 0.5 ksf surcharge, and -0.22 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 seconds

Stage: Stage 1 = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.64897
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.64897
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.135909	0.5
Effective Stress [ksf]	0.5	2.7202
Total Stress [ksf]	0.5	5.16491
Total Strain	0.000525036	0.00177605
Pore Water Pressure [ksf]	0	2.44471
Excess Pore Water Pressure [ksf]	0	0.491621
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.50625	8
Over-consolidation Ratio	1	8.20956
Void Ratio	0	0.99794
Permeability [ft/y]	0	0.182316
Coefficient of Consolidation [ft^2/y]	0	54.79
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.633535

Stage: Stage 2 = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.01119
Consolidation Settlement [in]	0	0.36222
Immediate Settlement [in]	0	0.64897
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.135909	0.5
Effective Stress [ksf]	0.5	2.85611
Total Stress [ksf]	0.5	5.16491
Total Strain	0.000525036	0.00392344
Pore Water Pressure [ksf]	0	2.3088
Excess Pore Water Pressure [ksf]	0	0.0296164
Degree of Consolidation [%]	0	99.9064
Pre-consolidation Stress [ksf]	0.50625	8
Over-consolidation Ratio	1	6.6177
Void Ratio	0	0.997093
Permeability [ft/y]	0	0.182316
Coefficient of Consolidation [ft^2/y]	0	54.79
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.648169

Stage: Stage 3 = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.99728
Consolidation Settlement [in]	0	0.38304
Immediate Settlement [in]	0	0.64897
Secondary Settlement [in]	0	1.96527
Loading Stress [ksf]	0.135909	0.5
Effective Stress [ksf]	0.5	2.85611
Total Stress [ksf]	0.5	5.16491
Total Strain	0.000525036	0.0242731
Pore Water Pressure [ksf]	0	2.3088
Excess Pore Water Pressure [ksf]	-9.95183e-019	2.61939e-019
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.50625	8
Over-consolidation Ratio	1	6.61752
Void Ratio	0	0.993187
Permeability [ft/y]	0	0.182316
Coefficient of Consolidation [ft^2/y]	0	54.79
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.648172

Loads

1. Rectangular Load

Length	98 ft
Width	15 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1470 ft^2
Load	-0.22 ksf
Depth	8 ft
Installation Stage	Stage 1 = 0 y

Coordinates

X [ft]	Y [ft]
-41.128	8.036
56.872	8.036
56.872	23.036
-41.128	23.036

2. Rectangular Load

Length	28 ft
Width	150 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4200 ft^2
Load	0.5 ksf
Depth	0 ft
Installation Stage	Stage 1 = 0 y

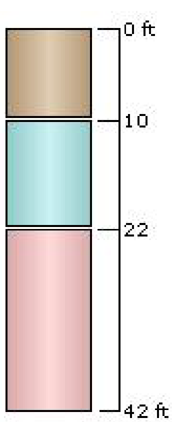
Coordinates

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	10	0	No
2	Presumpscot: Silty Clay/ Clayey Silt	12	10	Yes
3	Weaker Presumpscot	20	22	Yes



Soil Properties

Property	Fill	Presumpscot: Silty Clay/ Clayey Silt	Weaker Presumpscot
Color			
Unit Weight [kips/ft^3]	0.125	0.095	0.095
Saturated Unit Weight [kips/ft^3]	0.132	0.117	0.117
Immediate Settlement	Enabled	Enabled	Enabled
Es [ksf]	450	132	132
Esur [ksf]	1800	528	528
Primary Consolidation	Disabled	Enabled	Enabled
Material Type		Non-Linear	Non-Linear
Cc		0.4	0.4
Cr		0.04	0.04
e0		1	1
Pc [ksf]		8	
OCR	1		1.1
Cv [ft^2/y]		54.79	54.79
B-bar		1	1
Secondary Consolidation	Disabled	Mesri	Mesri
Ca/Cc		0.04	0.06
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

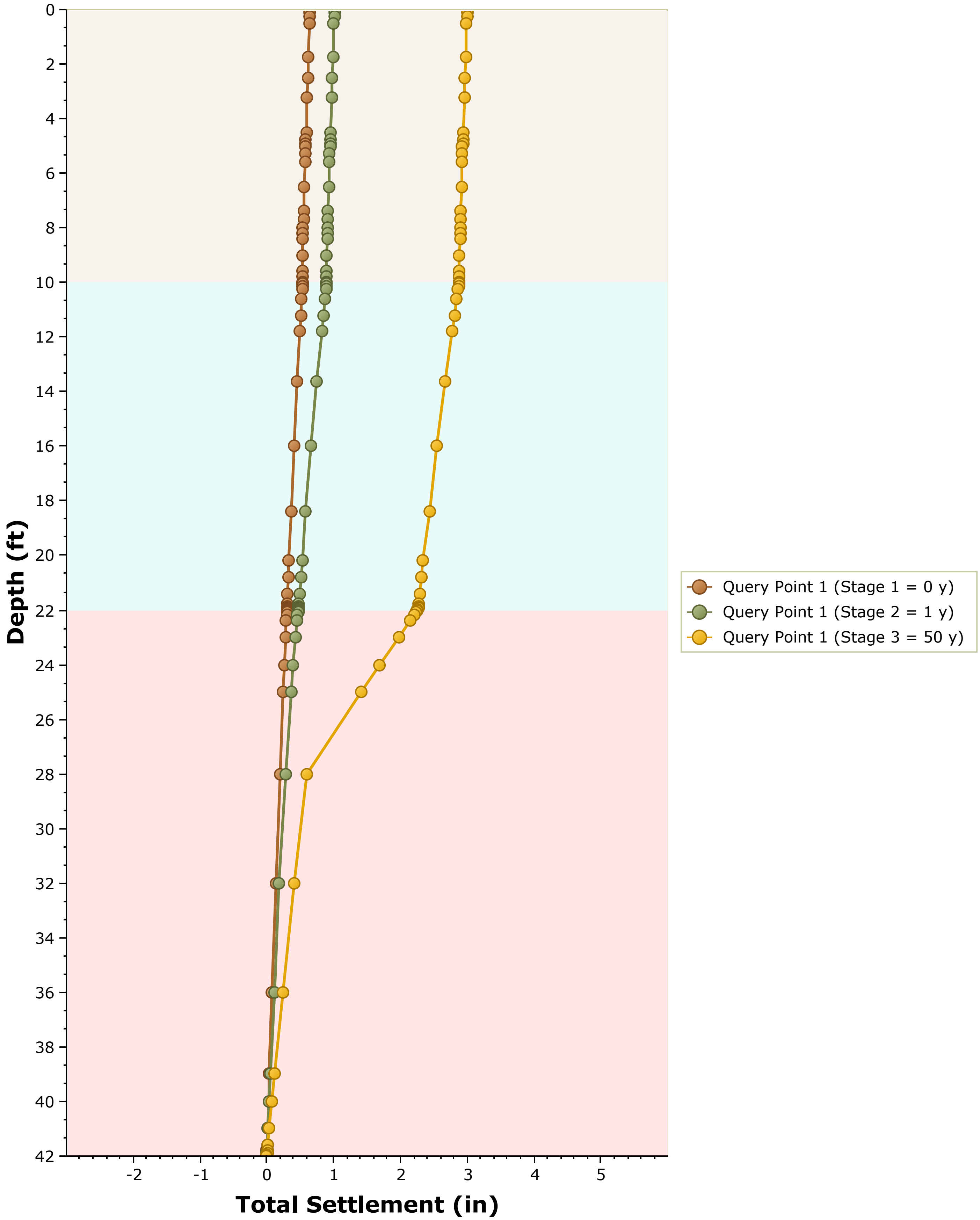
Piezometric Line Entities

ID	Depth (ft)
1	5 ft

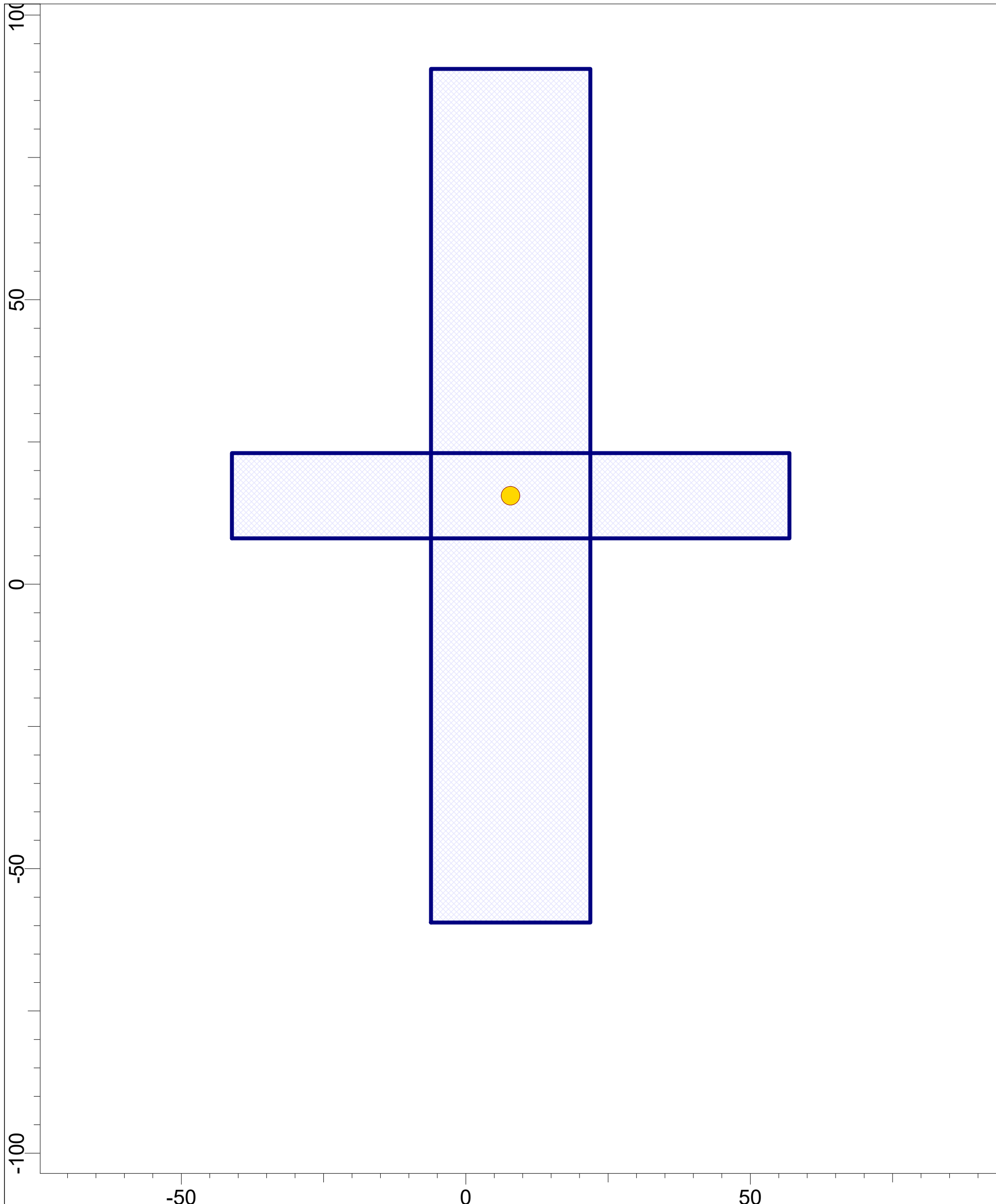
Query Points

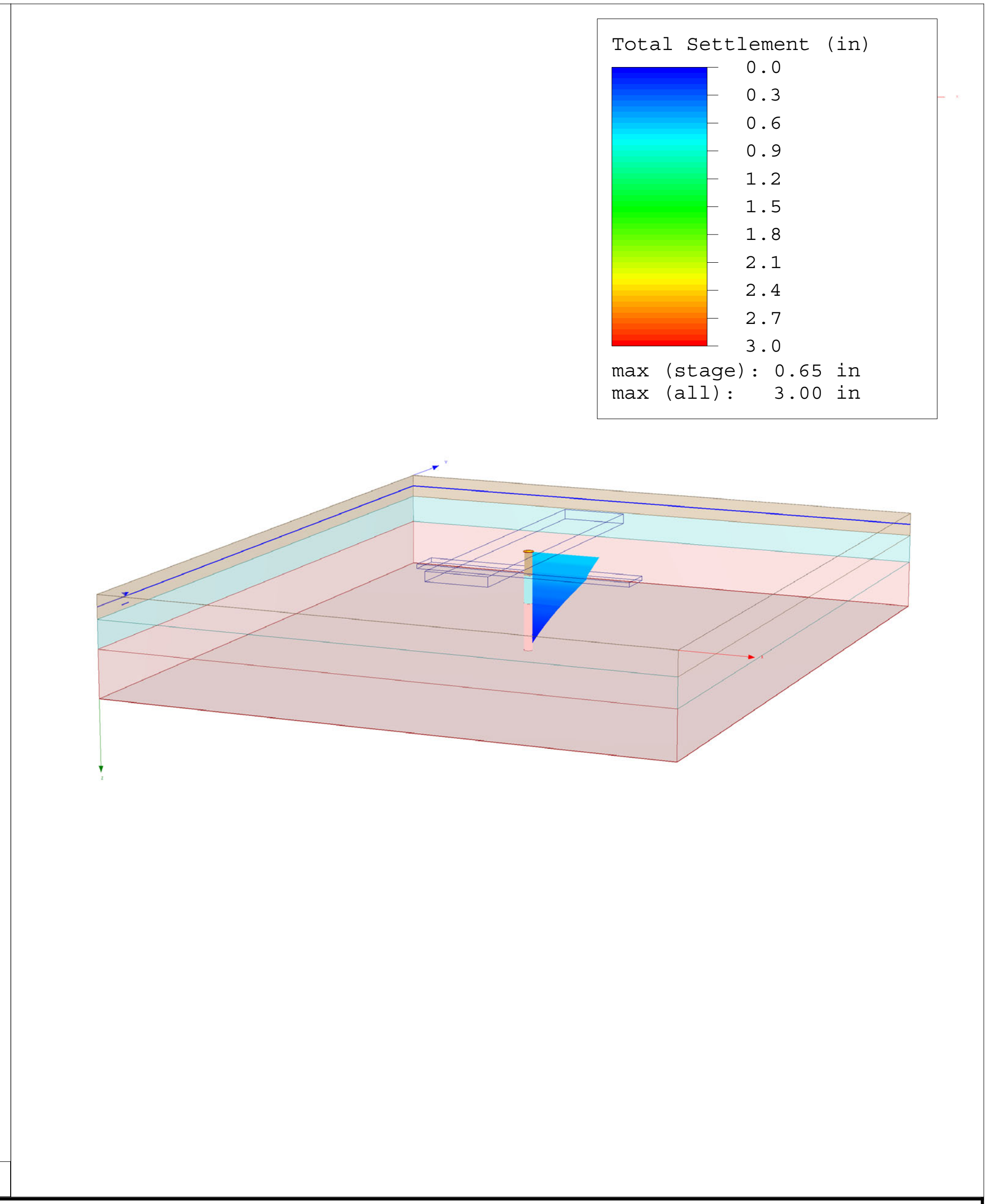
Point #	(X,Y) Location	Number of Divisions
1	7.872, 15.536	Auto: 59

Total Settlement vs. Depth

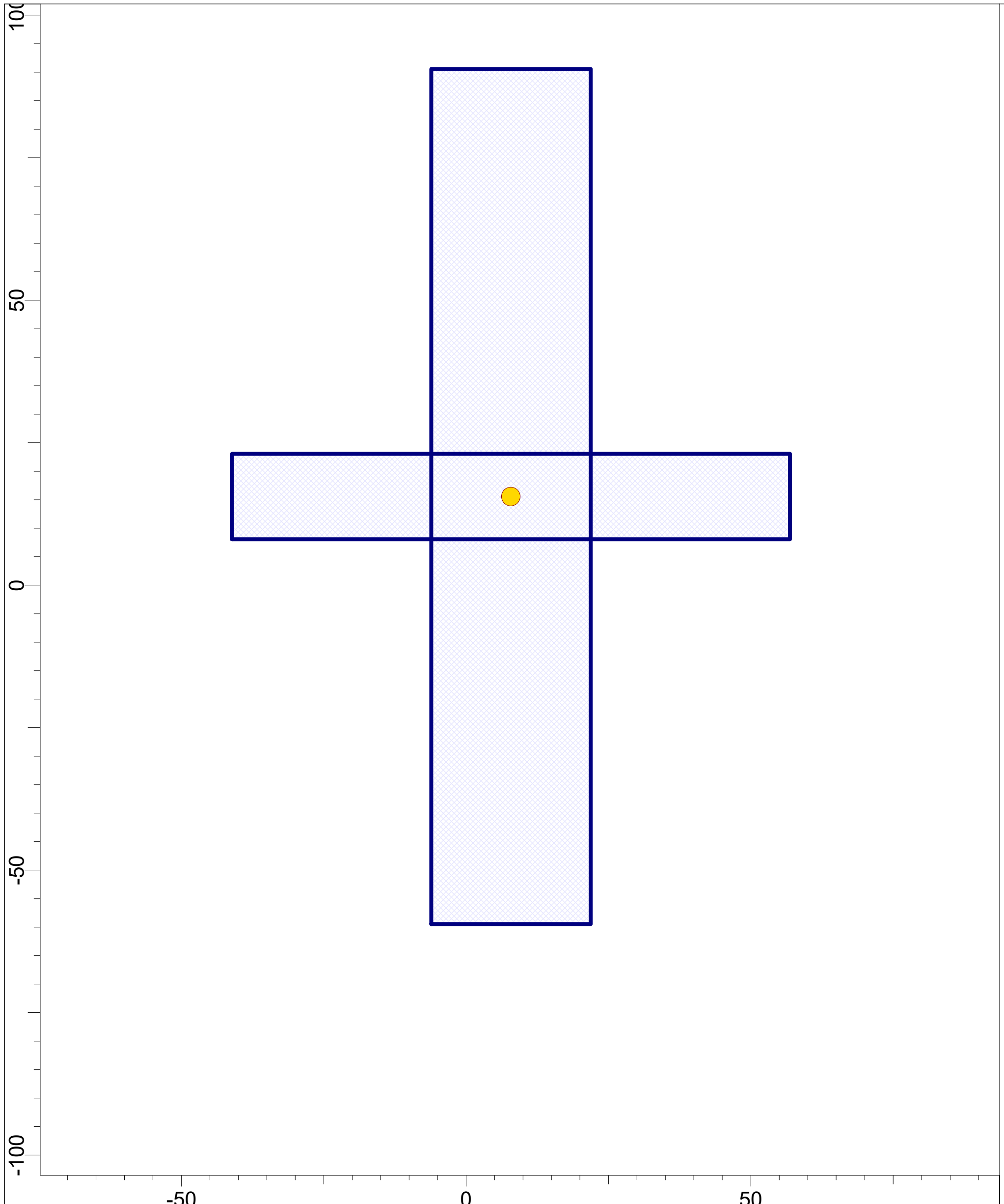


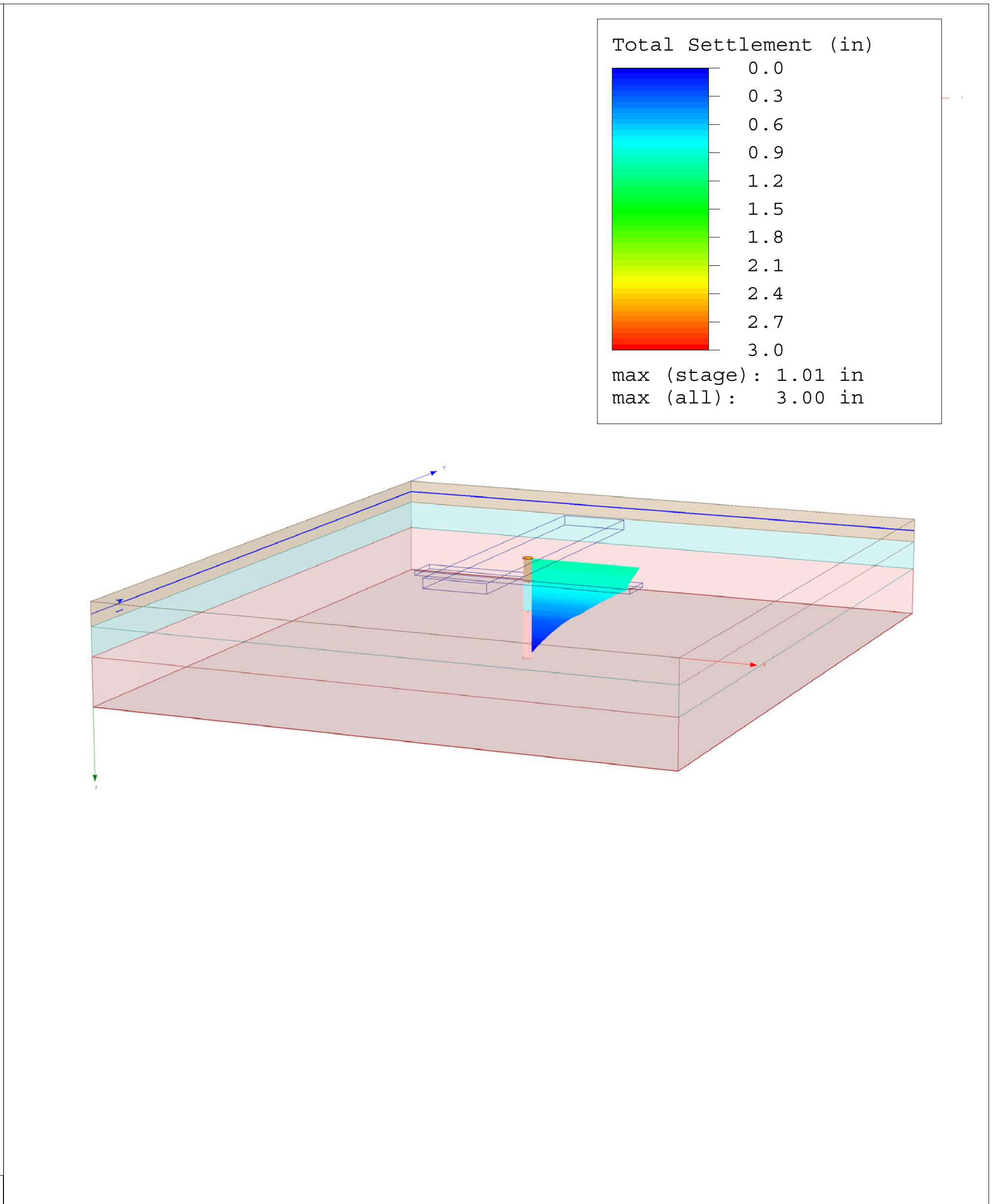
Reference Stage: None



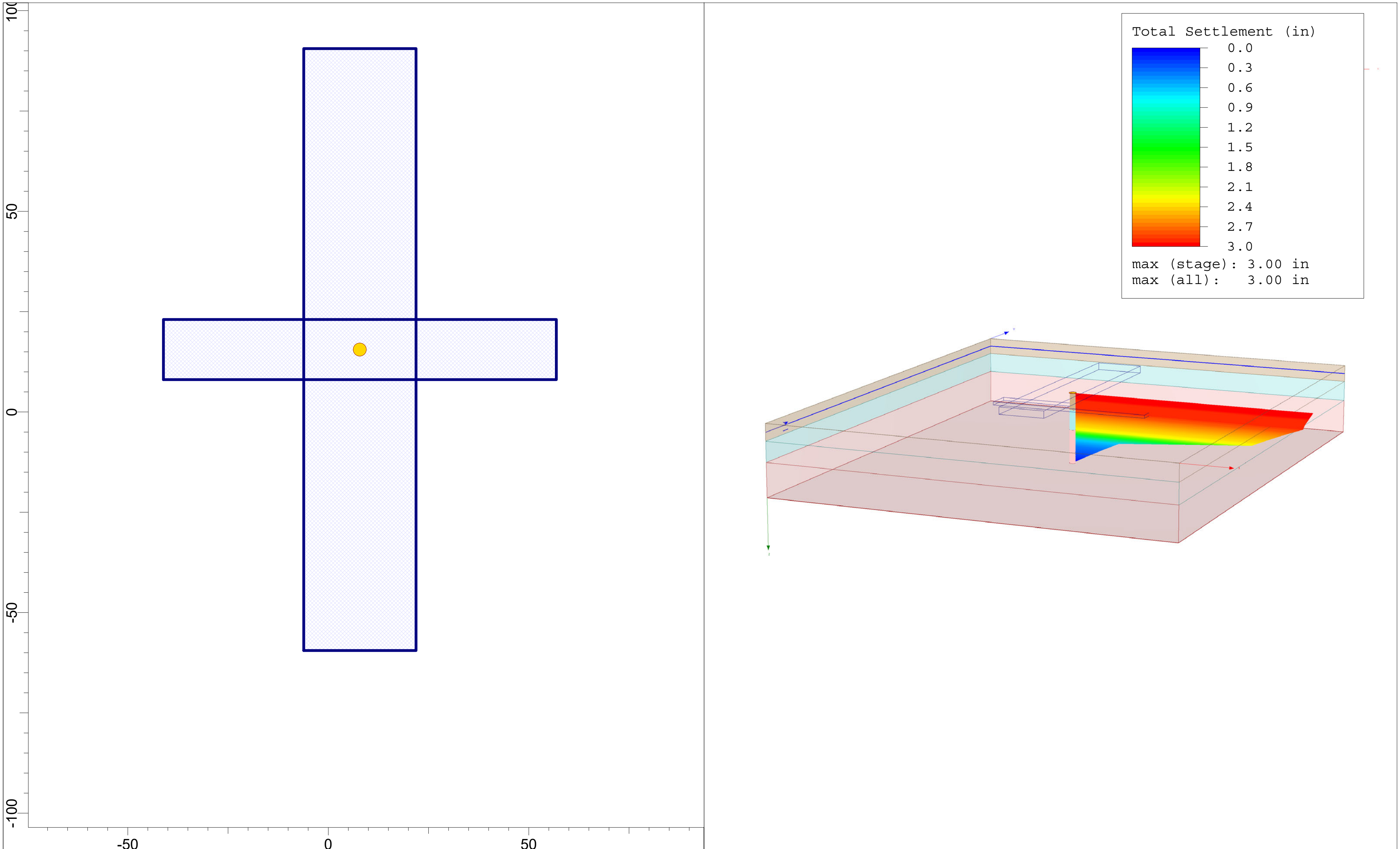


rocscience	*Project* 27222 Medford Paddy Hill Road Bridge		
	Analysis Description Immediate and consolidation settlement		
	Drawn By Yueh-Ti Lee	*Company* MaineDOT	
SETTLE3D 3.015	*Date* 4/21/2026, 11:41:38 AM	*File Name* 27222 Medford Settlement.s3z	





SETTLE3D 3.015	*Project*	27222 Medford Paddy Hill Road Bridge	
	Analysis Description	Immediate and consolidation settlement	
	Drawn By	Yueh-Ti Lee	*Company* MaineDOT
	Date	4/21/2026, 11:41:38 AM	*File Name* 27222 Medford Settlement.s3z



SETTLE3D 3.015	*Project* 27222 Medford Paddy Hill Road Bridge		
	Analysis Description Immediate and consolidation settlement		
	Drawn By Yueh-Ti Lee	*Company* MaineDOT	
	Date 4/21/2026, 11:41:38 AM	*File Name* 27222 Medford Settlement.s3z	

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)	†$E_s = 40\,000 + 1050N$ $E_{s(\mathrm{OCR})} \approx E_{s,\mathrm{nc}}\sqrt{\mathrm{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
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	$E_s = (1 \text{ to } 2)q_c$ $^{\$}E_s' = 2.5q_c$ $E_s' = 4q_c + 5000$
	where $E_s' = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

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

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

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

5-9.1 Effects on Settlements

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

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	Dry unit weight, γ_d 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

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 if the drainage characteristics of the deposit, such as sand layer spacings and overlying soil permeability, are known.

Over-Consolidation Ratio, OCR

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:

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

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: **Medford, Maine**
DFI = 1950 degree-days.
Case 1 - fine grained soils W=5.6% (HB-MED-101 1D).

For DFI = 1900
at w=10%

$d_1 := 65.8in$

For DFI = 2000
at w=10%

$d_2 := 67.6in$

Depth of Frost Penetration

$$d := \frac{d_1 + d_2}{2} = 5.6 \cdot ft$$

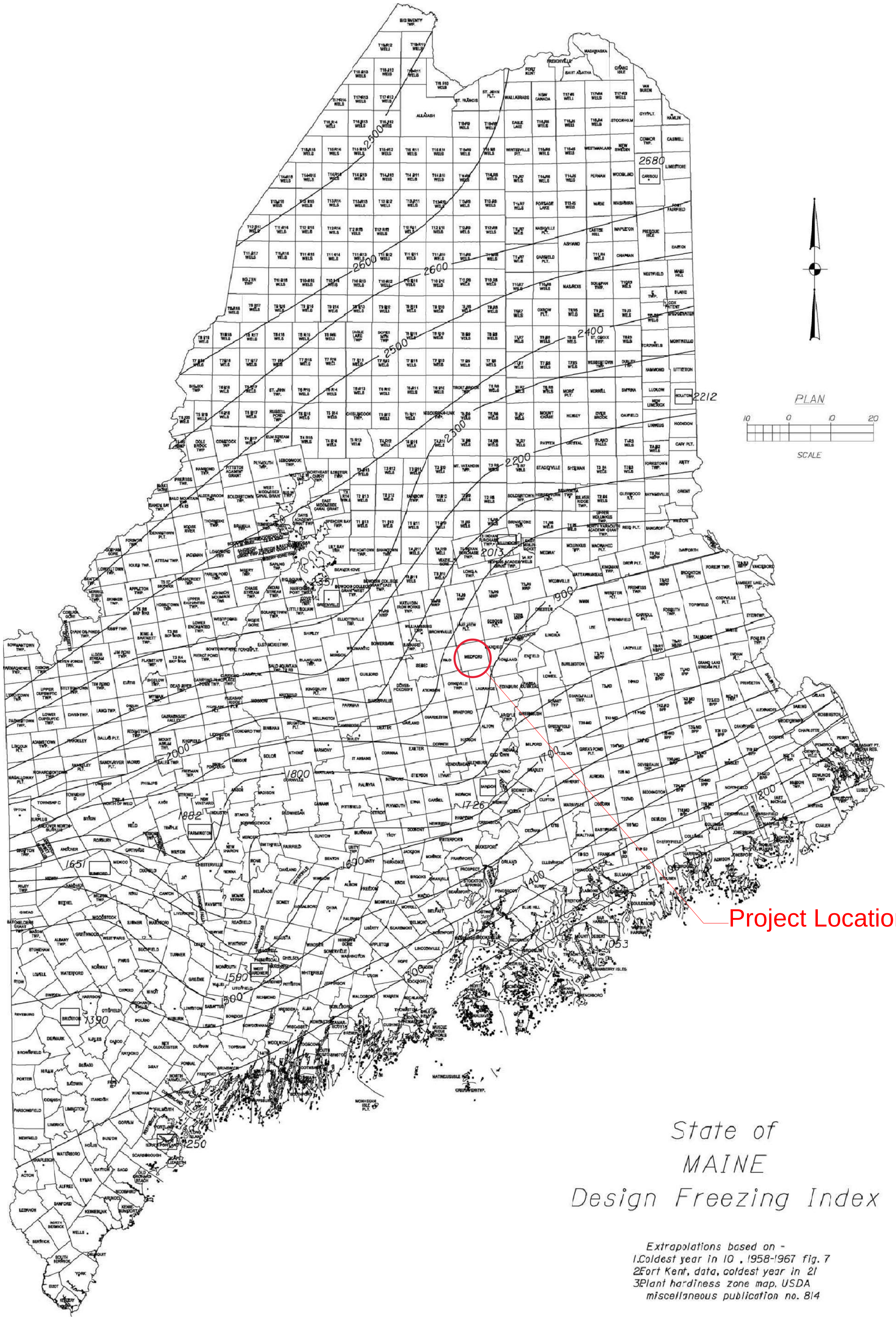


Figure 5-1 Maine Design Freezing Index Map

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