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Letter of Transmittal

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Attached is one (1) copy of Soils Report 2020-40, "GEOTECHNICAL DESIGN REPORT: For the Replacement of: DUGWAYBRIDGE, ROUTE 17 OVER BALDWIN BROOK, BYRON, MAINE" dated: October 26, 2020.

This report is available in TEDOCS as Document # 1910168.

att: 1 of 2020-40

**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE**

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**DUGWAY BRIDGE
ROUTE 17 OVER BALDWIN BROOK
BYRON, MAINE**

Prepared by:

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Oxford County
WIN 23559.00

Soils Report 2020-40
Bridge No. 2236

October 26, 2020
Federal Aid Project No. 23559.00

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Dugway Bridge which carries State Route 17 over Baldwin Brook in Byron, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical design parameters, and construction recommendations for the new box culvert.

The existing Dugway Bridge was constructed in 1964 and it consists of a 10-foot diameter corrugated galvanized steel pipe. According to the 2017 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the structure is in overall poor condition (rating of 4). The steel culvert has moderate to heavy pitting and corrosion and bolts fastening the steel plates together are missing or have failed. The channel at the inlet and outlet ends is severely eroded.

The proposed replacement structure is a 23-foot span and 10-foot rise, 101-foot long, precast concrete box culvert. The box culvert shall have 1-foot tall precast headwalls and 2-foot deep toe walls at the inlet and outlet. The upstream and downstream ends of the culvert will be slope-tapered at approximately 1.7H:1V (horizontal:vertical) to match the sideslopes. The box culvert invert will be embedded 3 feet into the streambed and 3 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. Four rock bands will be placed on the bottom of the culvert slowing the stream to allow fish passage. The box shall be placed on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill.

The new box culvert will be located on nearly the same horizontal alignment as the existing bridge. The finished grade over the proposed precast box culvert will closely match the existing. Alternating one-way traffic will be maintained by staged construction.

2.0 GEOLOGIC SETTING

Dugway Bridge carries State Route 17 over Baldwin Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geologic Map of the Lewiston 1° X 2° Quadrangle, Open-File No. 87-7 (1987), indicates the surficial soils in the vicinity of the bridge project consist of glacial till. Glacial till is typically a loose to very compact, poorly sorted, massive to weakly stratified mixture of sand, silt, and gravel-size rock, deposited by glacial ice. Glacial till may include lenses of water-laid sand and gravel. Boulders are common in the glacial till mapped in the Lewiston Quadrangle.

The MGS Bedrock Geologic Map of the Maine (1985), maps the bedrock at the project site as Perry Mountain Formation with nearby contact to the Rangeley Formation. Both formations consist of Schist and Granofels, with the Perry Mountain Formation also including Quartzite.

3.0 SUBSURFACE INVESTIGATION

Two test borings explored subsurface conditions at the project location. Boring BB-BBB-101 was drilled south of the existing pipe culvert and BB-BBB-102 was drilled north of the existing pipe culvert. The test boring locations are shown on Sheet 2 – Boring Location Plan.

The test borings were drilled on March 27 and 28, 2019. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

Borings were performed by using a combination of solid stem auger and cased wash boring techniques. The borings were completed by backfilling and compacting the borehole with drill cuttings. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rig that performed the borings was equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in April 2018. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.928 to the raw field N-values. This hammer efficiency factor (0.928) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field-testing requirements, and reviewed the field logs for accuracy. A MaineDOT NETTCP Certified Subsurface Inspector logged the subsurface conditions encountered in the borings and probes. The borings were located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of five standard grain size analyses with natural water content. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill and Glacial Till. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A fill layer was encountered in the test borings. The thickness of the fill layer was approximately 16 to 18 feet. The unit generally consisted of light brown, damp to moist sand, some to little gravel, some to trace silt, and occasional cobbles and boulders.

Corrected SPT N-values in the fill layer ranged from 28 to 116 blows per foot (bpf), indicating the fill unit is medium dense to very dense in consistency. Blow counts greater than 50 and one unsuccessful SPT test indicate the presence of cobbles and boulders. Three grain size analyses of the fill material resulted in the material being classified as A-1-b and A-2-4 under the AASHTO Soil Classification System and SW-SM and SM under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 7 to 18 percent.

5.2 Glacial Till

A glacial till deposit, comprised of primary cobbles and boulders, was encountered beneath the upper fill layer in the borings. Rock coring techniques were required in one boring to advance the borehole through the cobble and boulder deposit. The upper cobble and boulder till layer ranged in thickness from 7.2 to 10.0 feet and it was underlain by a very dense, cemented glacial till deposit. The cemented glacial till encountered was variable from:

- Grey, wet, Silty SAND, little gravel, to
- Grey, wet, Sandy SILT, little to trace gravel, to
- Grey, wet, SAND, little gravel, trace silt, to
- Grey, wet, Gravely SAND.

Corrected SPT N-values exceeded 100 bpf indicating the layer is very dense to hard in consistency. Three SPT tests failed to advance the minimum eighteen inches required to record an N-value. Grain size analyses conducted on samples from the cemented glacial till deposit indicated the material is classified as A-1-b and A-4 under the AASHTO Soil Classification System and SM and SW under the USCS. The natural water content of one samples tested was approximately 9 percent.

The two borings did not explore the full vertical limit of the glacial till deposit, but were terminated in the glacial till at depths of approximately 27 to 29 feet into the till.

5.3 Groundwater

No groundwater was observed during the borings. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Given the span length of 23 feet, it was determined that a buried structure alternative would be the most suitable structure type to satisfy the purpose and need of this bridge replacement project. A precast concrete box culvert was chosen because of the structure's durability, ease and speed of construction, and economic advantage compared to similar buried structures. Therefore, a 101-foot long precast concrete box culvert with a 23-foot span and 10-foot rise is the preferred replacement structure. The box will be embedded approximately 3 feet into the streambed and 3 feet of special fill will be placed inside to create a natural streambed. Rock bands will be constructed on the special fill inside of the culvert to slow the water and facilitate fish passage.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 101-foot-long precast concrete box culvert with slope tapered inlet and outlet walls. The box culvert will have 1-foot tall precast headwalls. To prevent undermining, the box culvert will have 2-foot tall inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 3 feet into the streambed and 3 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. Riprap aprons will be constructed at the inlet and outlet of the box culvert. The riprap aprons should be embedded a minimum of 3 feet into the streambed and covered with the engineered streambed material to provide continuity of the natural streambed.

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be designed 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, 9th Edition, 2020 (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: $\phi = 32^\circ$, $\gamma = 125$ pcf.

The box culvert will be bedded on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill conforming to Standard Specification 703.19. The soil envelope and backfill

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

7.1.1 Precast Concrete Box Culvert Headwalls

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

7.1.2 Precast Concrete Inlet and Outlet Walls

The precast concrete box culvert’s outlet and inlet walls will be slope-tapered to match the sideslopes. The left and right outlet walls will share the same precast base slab. The sloped 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.

Inlet and outlet walls that are fixed to the box culvert should be designed to resist movement using an at-rest earth pressure coefficient, K_o , of 0.47. Wingwalls sections that are independent of the box culvert and free to rotate should be designed using the Rankine active earth pressure coefficient, K_a , of 0.31 assuming a 2H:1V backslope. The active earth pressure coefficient will change if the backslope conditions are different.

7.1.3 Precast Concrete Inlet and Outlet Toe Walls

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

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill with a bottom elevation ranging from approximately El. 960.5 to 955 feet.

For a precast concrete box culvert with a base width of 25 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 21 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 6 ksf. Due to the large size of the concrete box culvert base, controlling deflection and not bearing resistance may govern the design. The service limit state bearing resistance may govern the design. In no instance shall bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'c$.

7.1.5 Modulus of Subgrade Reaction

Large span precast box culverts can be viewed similarly to a mat foundation. A common approach to the design of precast box culverts is to use beam on elastic foundation theory to compute the soil-structure interaction and deflections.

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction, k_s , equal to 102 pounds per cubic inch (pci).

7.2 Subgrade Preparation for Box Culvert

The soils encountered in the borings at the proposed box subgrade elevation (El. 955 to 960) consist of very dense cobbles, boulders, and sands. Any cobbles or boulders exposed at the subgrade should be removed, so as to not create contact stresses on the box and replaced with compacted granular borrow. Any unsuitable soils (i.e. low strength silts and clays and loose sands) that may be encountered at the subgrade elevation, should be excavated down to expose competent, firm material and replaced with compacted granular borrow.

7.3 Settlement

The fill unit encountered at the site is medium dense to very dense in consistency. The glacial till deposit is very dense in consistency. These coarse-grained materials are cohesionless and undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The proposed vertical alignment will remain nearly the same as the existing vertical alignment. As a result, little to no increase in vertical overburden pressure is expected. Any settlement is anticipated to be small and will occur relatively quickly.

Any loose or soft soils encountered at the subgrade elevation for the precast box culvert should be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill. With these provisions, post-construction settlement at the location of the replacement structure is anticipated to be minimal.

7.4 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, Byron has a design freezing index (DFI) of approximately 1900 F-degree days. A water content of 10% was used for coarse-grained soils. These components correlate to a frost depth of 7.7 feet.

It is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 7.7 feet for frost protection.

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

7.5 Scour and Riprap

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

Where required, slopes shall be armored with a 3-foot thick layer of 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. Riprap aprons will be installed at both ends of the culvert.

7.6 Seismic Design Considerations

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

8.0 CONSTRUCTION CONSIDERATIONS

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 and compacted to the manufacturer's specifications. To minimize future settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

The proposed box culvert should be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill, conforming to Standard Specification 703.19.

The Contractor should maintain the excavation so that the box culvert is constructed in-the-dry. It is anticipated that there will be seepage of water. Water should be controlled by pumping from sumps.

The glacial till deposit encountered in borings at proposed bottom elevation of the box culvert is primarily a deposit of nested boulders and cobbles. All cobbles and boulders encountered at the excavation subgrade, and projecting into the zone of the granular borrow bedding, shall be removed and replaced with compacted granular borrow or compacted $\frac{3}{4}$ -inch crushed stone.

Any unsuitable soils (i.e. low strength silts and loose sands) that may be encountered at the subgrade elevation, should be excavated down to expose competent, firm material and replaced with compacted granular borrow.

The Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary construction traffic. 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.

9.0 CLOSURE

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

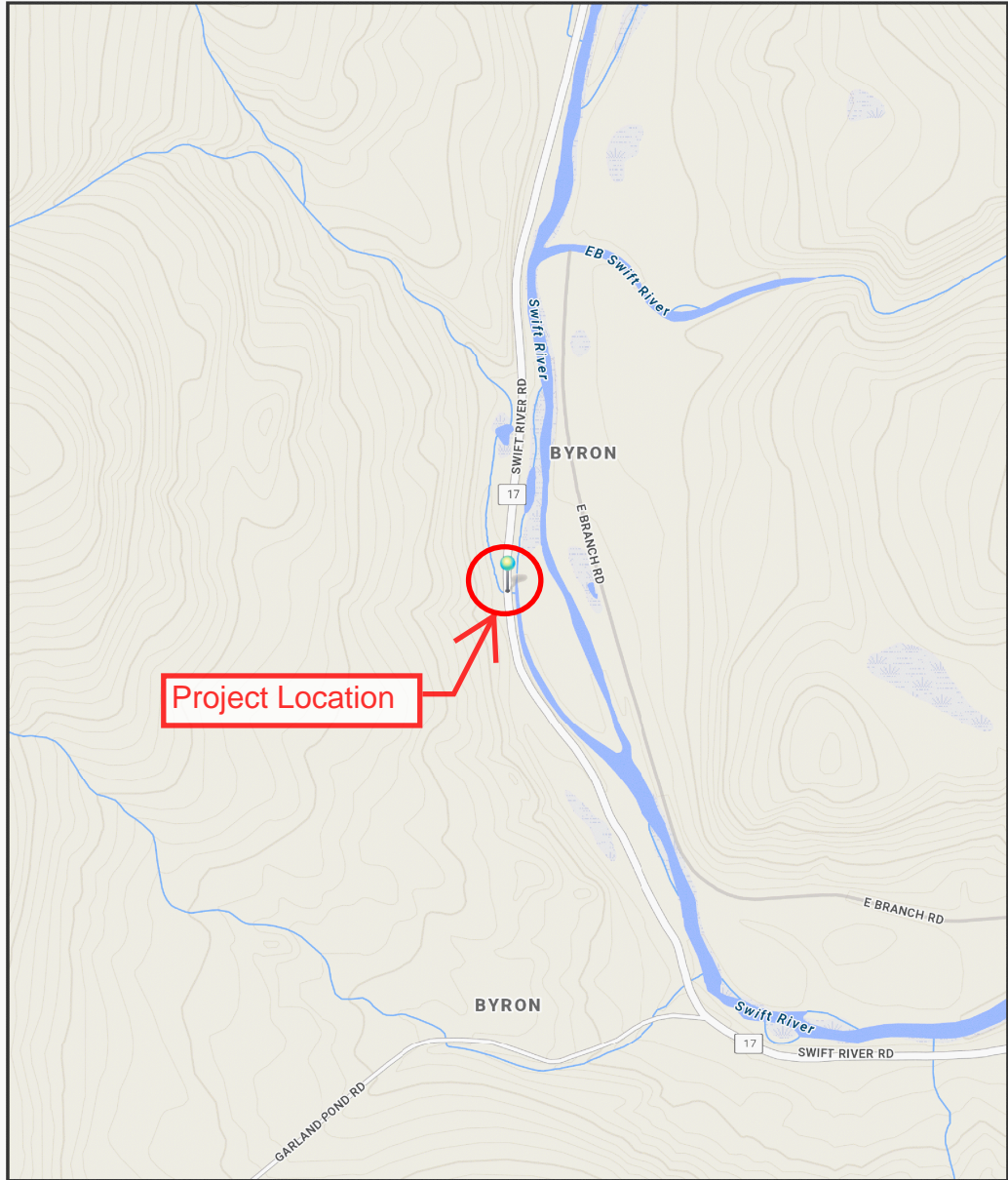
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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

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

Sheets



BYRON, MAINE

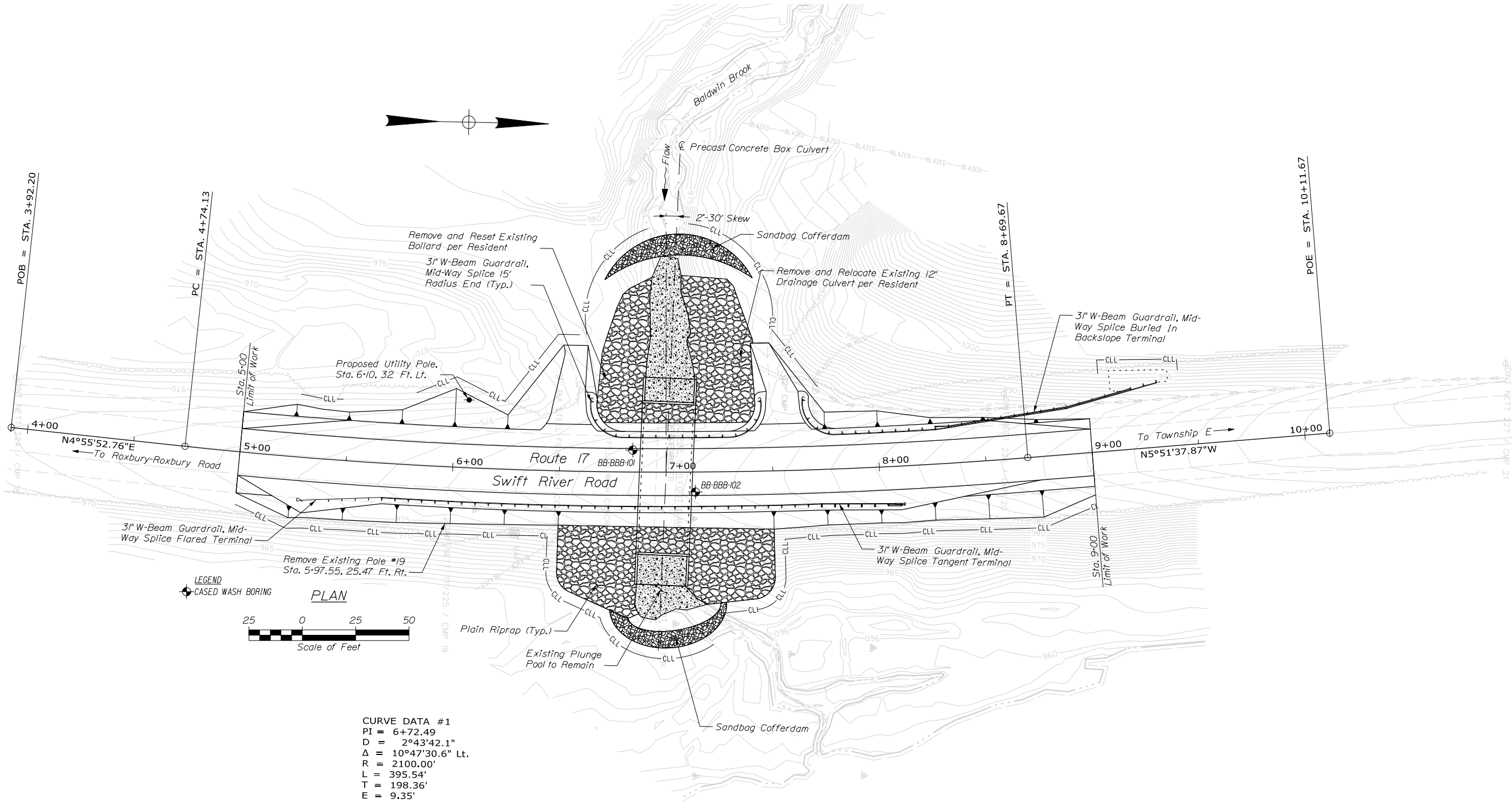


The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch.

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1 inch = 0.23 miles

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SHEET NUMBER 1	DUGWAY BRIDGE BALDWIN BROOK	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
	BYRON OXFORD COUNTY	2355900
OF 4	LOCATION MAP	WIN 23559.00 BRIDGE PLANS



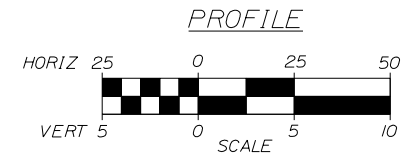
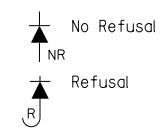
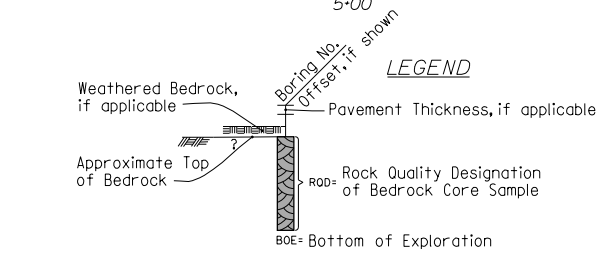
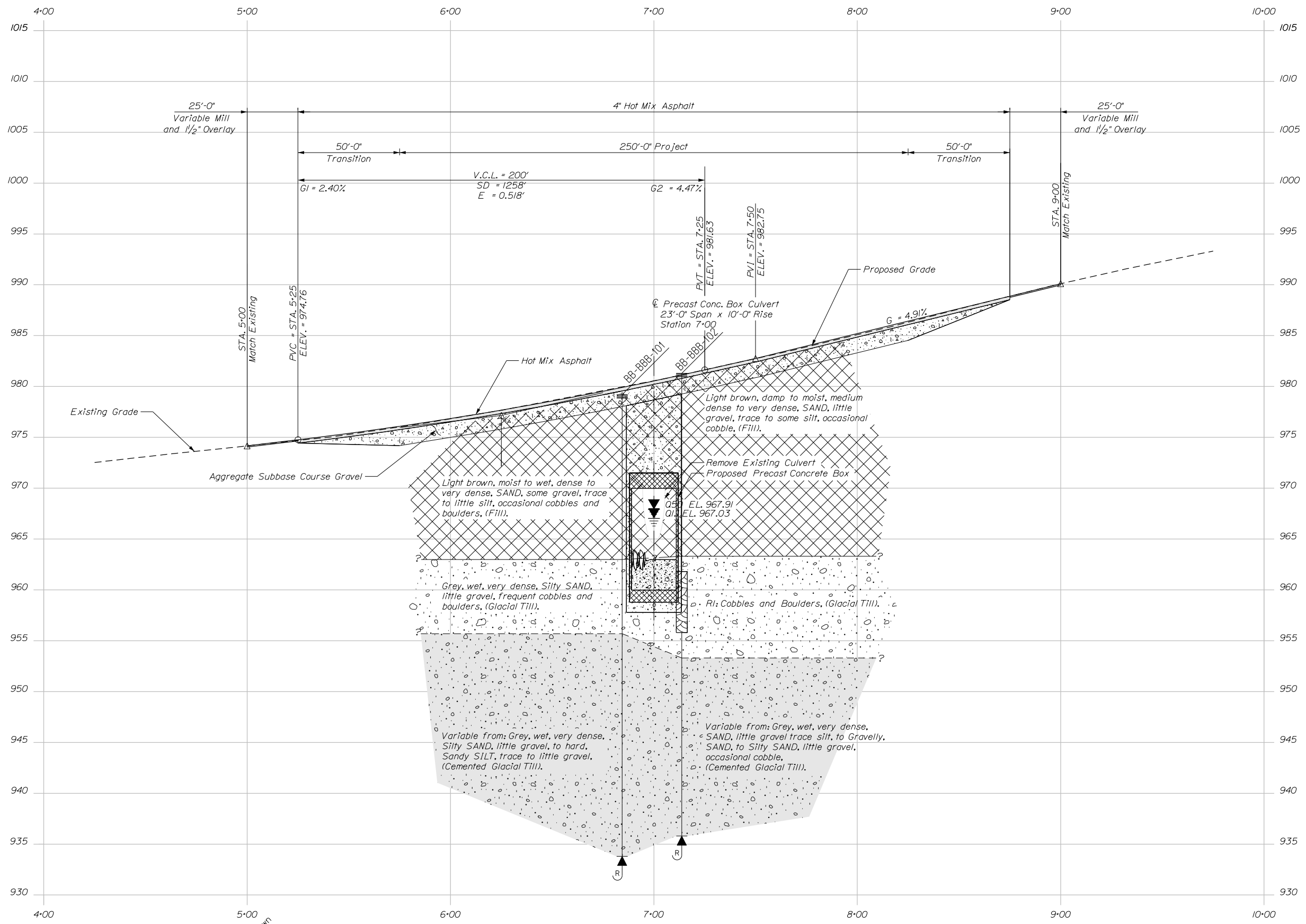
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BYRON		BORING LOCATION PLAN		2	
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REVISIONS 2		REVISIONS 2		DATE	
REVISIONS 3		REVISIONS 3			
REVISIONS 4		REVISIONS 4			
FIELD CHANGES		FIELD CHANGES			
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BRIDGE PLANS		BRIDGE PLANS			

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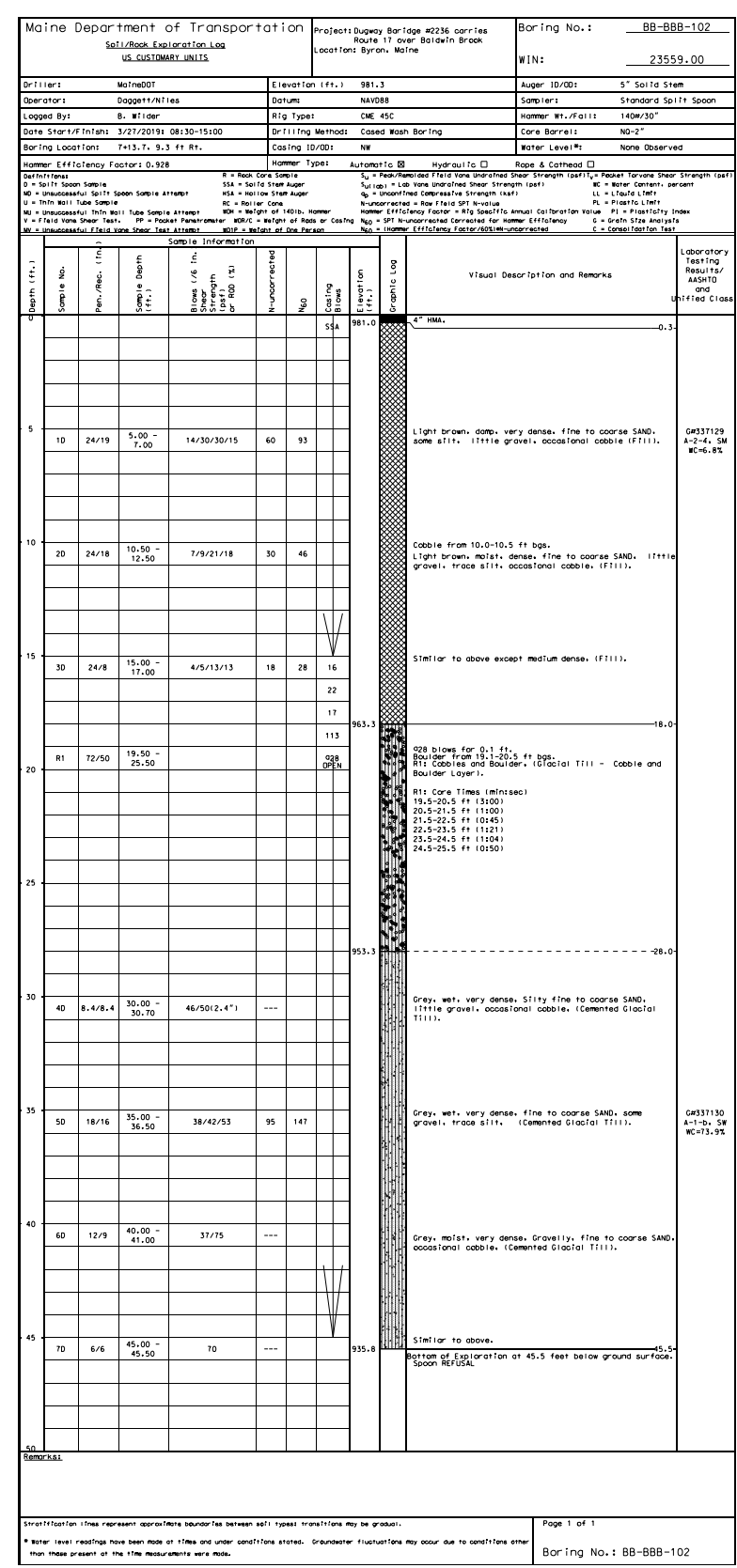
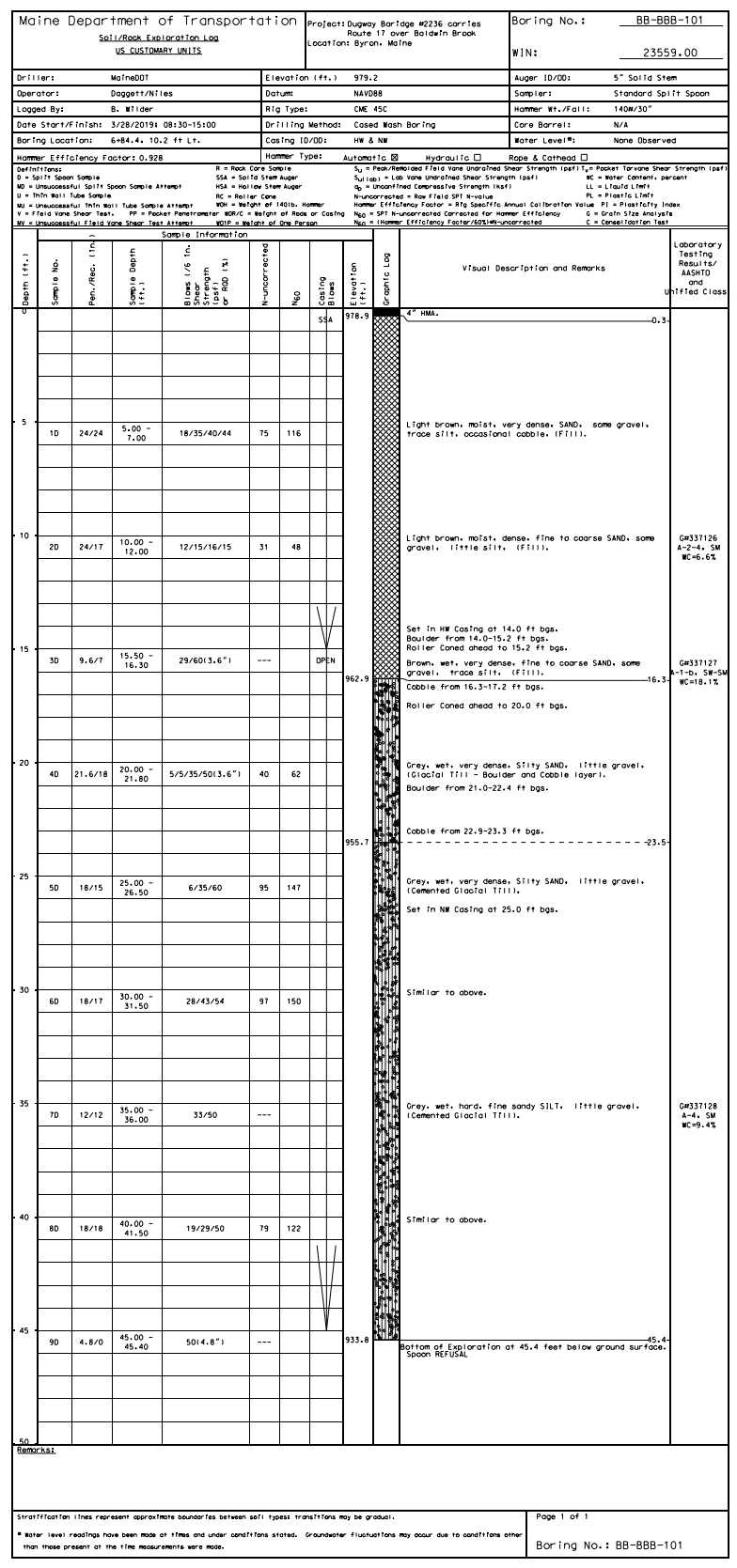
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Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples.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

STATE OF MAINE DEPARTMENT OF TRANSPORTATION		2355900	
BYRON		WIN	
DUGWAY BRIDGE BALDWIN BROOK OXFORD COUNTY		BRIDGE NO. 2236	
INTERPRETIVE SUBSURFACE PROFILE		BRIDGE PLANS	
SHEET NUMBER		23559.00	
3		DATE	
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T. WHITE		DATE	
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DESIGNS-DETAILED		DATE	
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REVISIONS 2		DATE	
REVISIONS 3		DATE	
REVISIONS 4		DATE	
FIELD CHANGES		DATE	



STATE OF MAINE DEPARTMENT OF TRANSPORTATION 2355900		BRIDGE NO. 2236 WIN 23559.00	
DUGWAY BRIDGE BALDWIN BROOK BYRON		OXFORD COUNTY BORING LOGS	
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SHEET NUMBER 4 OF 4			

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER 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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	
			GC	Clayey gravels, gravel-sand-clay mixtures.	
		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.	
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	SM	Silty sands, sand-silt mixtures		
		SC	Clayey sands, sand-clay mixtures.		
		ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.		
	SILTS AND CLAYS (liquid limit greater than 50)	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.			
MH		Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.			
HIGHLY ORGANIC SOILS	Pt	CH	Inorganic clays of high plasticity, fat clays.		
		OH	Organic clays of medium to high plasticity, organic silts.		
		Pt	Peat and other highly organic soils.		
Desired Soil Observations (in this order, if applicable):				Desired Rock 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				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))	
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				Sample Container Labeling Requirements:	
				WIN	Blow Counts
				Bridge Name / Town	Sample Recovery
				Boring Number	Date
				Sample Number	Personnel Initials
				Sample Depth	

Driller: MaineDOT	Elevation (ft.): 979.2	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 3/28/2019; 08:30-15:00	Drilling Method: Cased Wash Boring	Core Barrel: N/A
Boring Location: 6+84.4, 10.2 ft Lt.	Casing ID/OD: HW & NW	Water Level*: None Observed
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

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
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	978.9	4" HMA.		
5	1D	24/24	5.00 - 7.00	18/35/40/44	75	116				Light brown, moist, very dense, SAND, some gravel, trace silt, occasional cobble, (Fill).		
10	2D	24/17	10.00 - 12.00	12/15/16/15	31	48				Light brown, moist, dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#337126 A-2-4, SM WC=6.6%	
15	3D	9.6/7	15.50 - 16.30	29/60(3.6")	---	OPEN			962.9	Set in HW Casing at 14.0 ft bgs. Boulder from 14.0-15.2 ft bgs. Roller Coned ahead to 15.2 ft bgs. Brown, wet, very dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#337127 A-1-b, SW-SM WC=18.1%	
20	4D	21.6/18	20.00 - 21.80	5/5/35/50(3.6")	40	62			955.7	Cobble from 16.3-17.2 ft bgs. Roller Coned ahead to 20.0 ft bgs. Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till - Boulder and Cobble layer). Boulder from 21.0-22.4 ft bgs. Cobble from 22.9-23.3 ft bgs.		

Remarks:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Dugway Baridge #2236 carries Route 17 over Baldwin Brook Location: Byron, Maine				Boring No.: BB-BBB-101 WIN: 23559.00							
Driller: MaineDOT				Elevation (ft.): 979.2				Auger ID/OD: 5" Solid Stem							
Operator: Daggett/Niles				Datum: NAVD88				Sampler: Standard Split Spoon							
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 3/28/2019; 08:30-15:00				Drilling Method: Cased Wash Boring				Core Barrel: N/A							
Boring Location: 6+84.4, 10.2 ft Lt.				Casing ID/OD: HW & NW				Water Level*: None Observed							
Hammer Efficiency Factor: 0.928				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{u(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected				T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)							
25	5D	18/15	25.00 - 26.50	6/35/60	95	147		933.8	Grey, wet, very dense, Silty SAND, little gravel, (Cemented Glacial Till). Set in NW Casing at 25.0 ft bgs.	G#337128 A-4, SM WC=9.4%					
30	6D	18/17	30.00 - 31.50	28/43/54	97	150		Similar to above.							
35	7D	12/12	35.00 - 36.00	33/50	---			Grey, wet, hard, fine sandy SILT, little gravel, (Cemented Glacial Till).							
40	8D	18/18	40.00 - 41.50	19/29/50	79	122		Similar to above.							
45	9D	4.8/0	45.00 - 45.40	50(4.8")	---			Bottom of Exploration at 45.4 feet below ground surface. Spoon REFUSAL							
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.															
									Page 2 of 2						
									Boring No.: BB-BBB-101						

Driller: MaineDOT	Elevation (ft.): 981.3	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 3/27/2019; 08:30-15:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 7+13.7, 9.3 ft Rt.	Casing ID/OD: NW	Water Level*: None Observed
Hammer Efficiency Factor: 0.928		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>

Definitions:
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample Attempt
 U = Thin Wall Tube Sample
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 MV = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. Hammer
 WOR/C = Weight of Rods or Casing
 WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
 S_{u(lab)} = Lab Vane Undrained Shear Strength (psf)
 q_p = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw Field SPT N-value
 Hammer Efficiency Factor = Rig Specific Annual Calibration Value
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows ((6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	981.0	4" HMA.	-0.3	
5	1D	24/19	5.00 - 7.00	14/30/30/15	60	93						Light brown, damp, very dense, fine to coarse SAND, some silt, little gravel, occasional cobble (Fill).
10	2D	24/18	10.50 - 12.50	7/9/21/18	30	46				Cobble from 10.0-10.5 ft bgs. Light brown, moist, dense, fine to coarse SAND, little gravel, trace silt, occasional cobble, (Fill).		
15	3D	24/8	15.00 - 17.00	4/5/13/13	18	28	16			Similar to above except medium dense, (Fill).		
20	R1	72/50	19.50 - 25.50				a28 OPEN		963.3	a28 blows for 0.1 ft. Boulder from 19.1-20.5 ft bgs. R1: Cobbles and Boulder, (Glacial Till - Cobble and Boulder Layer).		
25										R1: Core Times (min:sec) 19.5-20.5 ft (3:00) 20.5-21.5 ft (1:00) 21.5-22.5 ft (0:45) 22.5-23.5 ft (1:21) 23.5-24.5 ft (1:04) 24.5-25.5 ft (0:50)		

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Dugway Baridge #2236 carries Route 17 over Baldwin Brook Location: Byron, Maine	Boring No.: BB-BBB-102 WIN: 23559.00
--	---	---

Driller: MaineDOT Operator: Daggett/Niles Logged By: B. Wilder Date Start/Finish: 3/27/2019; 08:30-15:00 Boring Location: 7+13.7, 9.3 ft Rt.	Elevation (ft.): 981.3 Datum: NAVD88 Rig Type: CME 45C Drilling Method: Cased Wash Boring Casing ID/OD: NW	Auger ID/OD: 5" Solid Stem Sampler: Standard Split Spoon Hammer Wt./Fall: 140#/30" Core Barrel: NQ-2" Water Level*: None Observed
---	---	--

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person	S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{u(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.			
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows								
25									953.3	-----28.0					
30	4D	8.4/8.4	30.00 - 30.70	46/50(2.4")	---									Grey, wet, very dense, Silty fine to coarse SAND, little gravel, occasional cobble, (Cemented Glacial Till).	
35	5D	18/16	35.00 - 36.50	38/42/53	95	147								Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, (Cemented Glacial Till).	G#337130 A-1-b, SW WC=73.9%
40	6D	12/9	40.00 - 41.00	37/75	---									Grey, moist, very dense, Gravelly, fine to coarse SAND, occasional cobble, (Cemented Glacial Till).	
45	7D	6/6	45.00 - 45.50	70	---				935.8	-----45.5					
50														Similar to above. Bottom of Exploration at 45.5 feet below ground surface. Spoon REFUSAL	

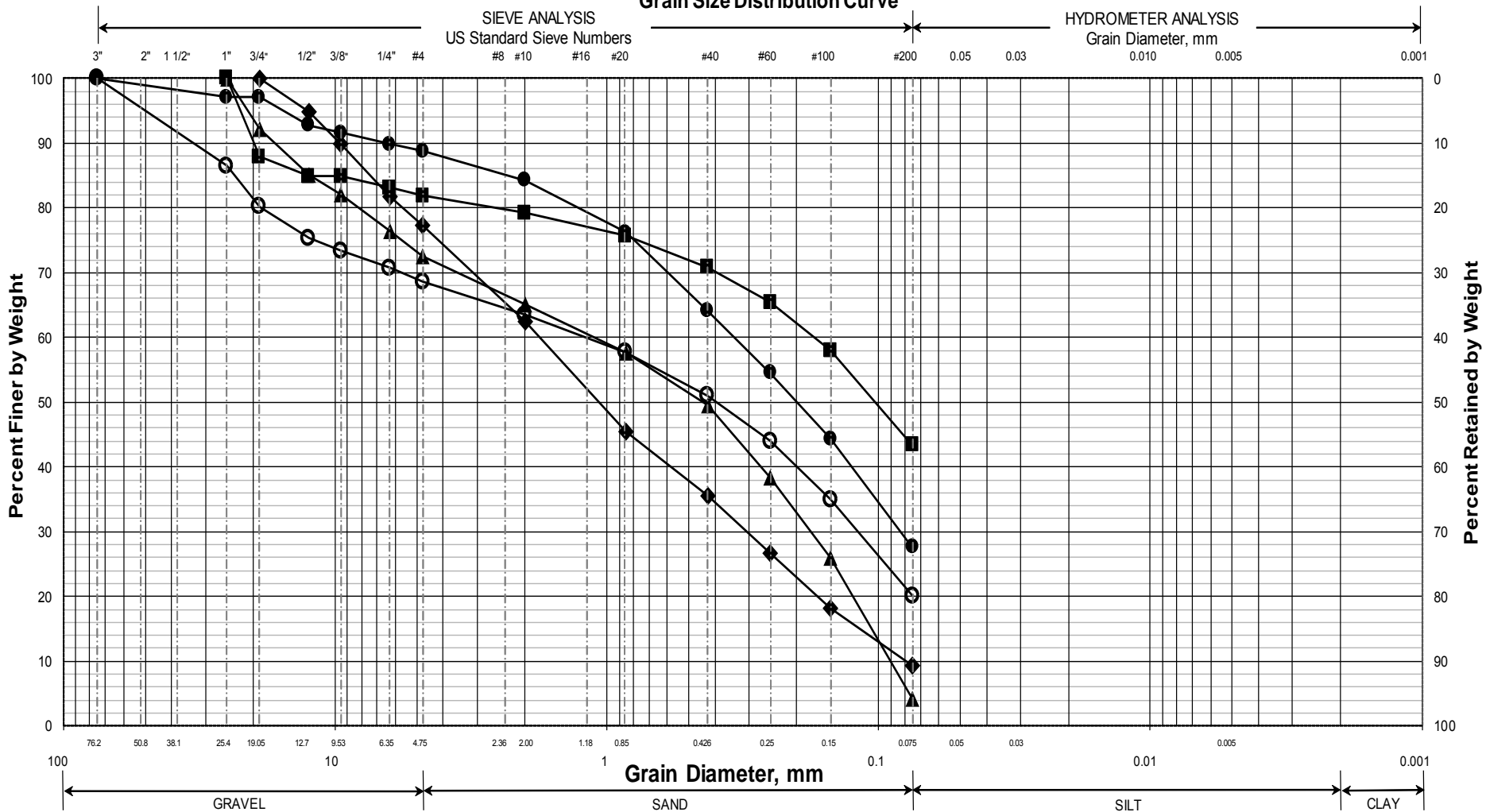
Remarks:

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

Appendix B

Laboratory Test Results

Maine Department of Transportation Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-BBB-101/2D	6+84.4	10.2 LT	10.0-12.0	SAND, some gravel, little silt.	6.6			
◆	BB-BBB-101/3D	6+84.4	10.2 LT	15.5-16.3	SAND, some gravel, trace silt.	18.1			
■	BB-BBB-101/7D	6+84.4	10.2 LT	35.0-36.0	Sandy SILT, little gravel.	9.4			
●	BB-BBB-102/1D	7+13.7	9.3 RT	5.0-7.0	SAND, some silt, little gravel.	6.8			
▲	BB-BBB-102/5D	7+13.7	9.3 RT	35.0-36.5	SAND, some gravel, trace silt.	73.9			
X									

WIN
023559.00
Town
Byron
Reported by/Date
WHITE, TERRY A 4/11/2019

Appendix C

Calculations

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 $\beta := 26.56 \cdot \text{deg}$

$$K_{ar_slope} := \cos(\beta) \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}} \quad K_{ar_slope} = 0.46$$

P_a 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

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

Soil type	Coefficient of Lateral Earth Pressure			
	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.

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 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

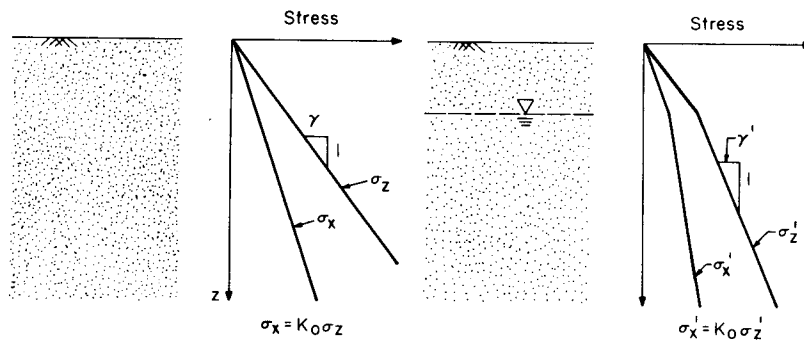


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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 $\beta = 0^\circ$, the value of the coefficient of active earth pressure (Rankine), K_a , may be taken as:

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

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 $\beta > 0^\circ$, the coefficient of active earth pressure (Rankine), K_a , may be taken as:

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

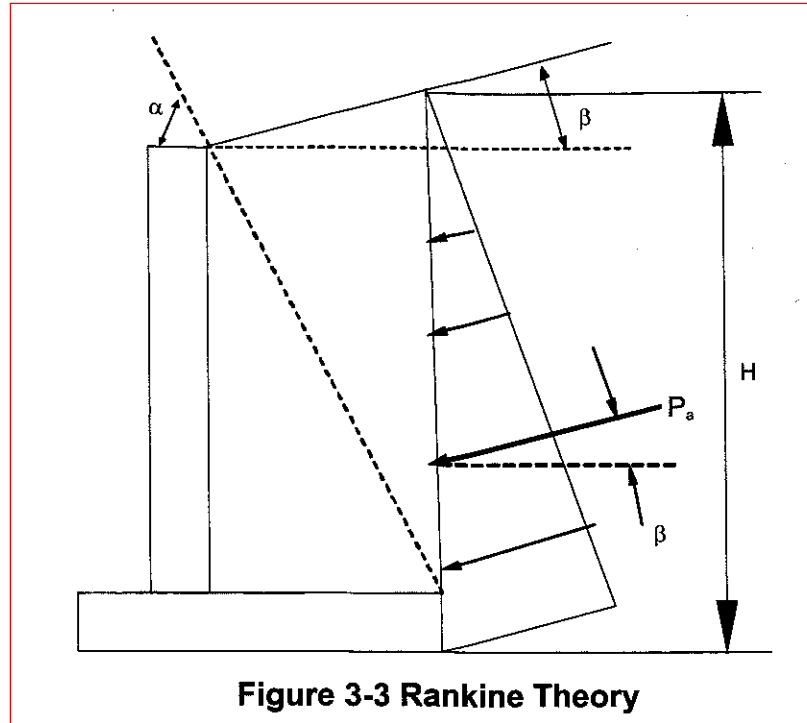


Figure 3-3 Rankine Theory

The resultant earth pressure force, P_a , is oriented at an angle, β , as shown in 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, K_a , may be determined using a β value adjusted per AASHTO LRFD Figures 3.11.5.8 -1 through 3, or substituted with β^* , as shown in Figure 3-2.

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

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

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

Bearing Resistance

Objective:

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

Given:

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

Assumptions:

1. The box culvert's embedment into the streambed is conservatively assumed as 1 foot, which accounts for the possible scouring away of 1 foot of special fill.
2. The one foot thick layer of proposed Granular Borrow bedding material modeled as subgrade.
3. The proposed bearing elevation is ranges from El. 955.0 to 960.0.
4. Proposed finish roadway grade elevation is approximately 980.6 feet at the low point.
5. Proposed precast concrete box base is 25 feet wide.
6. The bottom of the box culvert will be submerged for the structure's design life.

Estimate the factored bearing resistance at the Service Limit State:

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

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Coarse to medium sand and with little gravel (SW, SP)	Medium dense to dense	6-10	6

Recommend 6 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Foundation Width, Depth, and Water Surface

$$B := 25\text{ft}$$

$$D_f := 1.0\text{ft}$$

$$D_w := 0\text{ft}$$

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

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

$$\gamma_{\text{above}} := 125 \cdot \text{pcf} \quad \text{MaineDOT Bridge Design Guide p. 3-3 Soil Type 4}$$

$$\gamma_{\text{above_sat}} := 135 \cdot \text{pcf}$$

Foundation soils:

Foundation soils based on BB-BBB-101 3D

$$\gamma_{1d} := 125 \cdot \text{pcf}$$

$$w_{\text{sat}} := 18.1\%$$

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

$$\gamma_{1\text{sat}} = 147.6 \cdot \text{pcf}$$

$$\phi := 36 \cdot \text{deg} \quad \text{Compacted Granular Borrow over Glacial Till}$$

$$c := 0$$

Nominal Bearing Resistance for Strength Limit States

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

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

$$N_c := 50.6$$

$$N_q := 37.8$$

$$N_\gamma := 56.3$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$$L := 72 \cdot \text{ft}$$

$$s_c := 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right)$$

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

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

$$s_c = 1.3 \quad s_\gamma = 0.9 \quad s_q = 1.3$$

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.

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

Load Inclination factors

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

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

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

LRFD Table 10.6.3.1.2a-4

$$\frac{D_f}{B} = 0.04$$

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 63.7$$

$$N_{\gamma m} = 48.5$$

$$N_{qm} = 47.3$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 47.9 \cdot \text{ksf}$$

Byron, Dugway Bridge
23559.00

Bearing Resistance
Precast Box Culvert

Calculation by J. Manahan
April 2020
Checked by: LK 10/20/20

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$q_r = 21.6 \text{ ksf}$$

Recommend a factored bearing resistance of 21 ksf for box bottom slabs 25 ft or greater on compacted granular borrow.

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, $\tan \delta$, 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/ft ³	kN/m ³
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Table C10.6.2.6.1-1—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	12–20	14
	Medium dense to dense	8–14	10
	Loose	4–12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8–12	8
	Medium dense to dense	4–8	6
	Loose	2–6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6–12	8
	Medium dense to dense	2–6	4
	Loose	1–2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4–8	6
	Medium stiff to stiff	2–6	3
	Soft	1–2	1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, 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 <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	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

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

C10.6.3.1.2a

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:

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

in which:

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

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

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

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)

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.

where:

- B = footing width (ft)
- L = footing length (ft)
- H = unfactored horizontal load (kips)
- V = unfactored vertical load (kips)
- θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees)

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

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

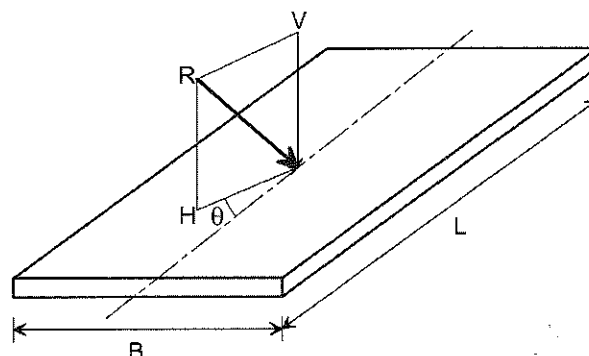


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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

Table 10.6.3.1.2a-2—Coefficients C_{wg} and C_{wy} for Various Groundwater Depths

D_w	C_{wg}	C_{wy}
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.

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

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

Friction Angle, ϕ_f (degrees)	D_f/B	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

The parent information from which Table 10.6.3.1.2a-4 was developed covered the indicated range of friction angle, ϕ_f . Information beyond the range indicated is not available at this time.

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.

Linear interpolations may be made for friction angles in between those values shown in Table 10.6.3.1.2a-4.

10.6.3.1.2b—Considerations for Punching Shear

C10.6.3.1.2b

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:

$$c^* = 0.67c \tag{10.6.3.1.2b-1}$$

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

where:

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

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

Modulus of Subgrade Reaction

Objective:

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

Given:

1. Limited lab data and SPT N-values.

Assumptions:

1. The proposed bearing elevation of base slab ranges from El. 955.0 to 960.0.
2. Proposed finished roadway grade is approximately 980.6.
3. Proposed precast concrete box is 25 feet wide and approximately 72 feet long.
4. The subsurface conditions present at the proposed bearing elevation consists of cemented glacial till.
5. The bottom of the box culvert will be submerged for the structure's design life.

Published values of subgrade modulus

Published values of subgrade modulus in dense sand:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:

Range of modulus of subgrade reaction 236 to 472 pci

Subgrade of dense sand, average of upper and lower limit: $k_s = 354$ pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:

Saturated coarse grained very dense sand K_{v1} , $260 \text{ pci} / 2 = 130$ pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:

Typical subgrade reaction values for 0.3 m x 0.3 m plate

Saturated dense sand, 478 - 553 pci: $k_{0.3} (k_1) = 515$ pci

Terzaghi Geotechnique, Vol. 5, No. 4, Table 1:

Values of vertical subgrade reaction for 1 ft x 1 ft plate on sand

Submerged dense sand: $k_{s1} = 347$ pci

Adjust Published values for dimensions of base slab

Select a subgrade modulus of 260 pci for saturated dense sand based on GEC No. 6. and divided by 2 per Note 2 (see references attached).

Value of $k_{s1} = 130$ pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base (Width B = 25 ft, Length L = 72 ft).

Square to rectangle base adjustment:

$$k_{s1} := 130 \text{ pci} \quad B := 25 \text{ ft} \quad L := 72 \text{ ft}$$

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

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

$$k = 101.7 \cdot \text{pci}$$

Recommend a subgrade modulus of 102 pci

for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \quad (9-10)$$

for either horizontal or vertical members

for depth variation

of interest below ground

to give k_s the best fit (if load test or other data are available)

variation may be zero; at the ground surface A_s is zero for a lateral k_s

> 0 . For footings and mats (plates in general), $A_s > 0$ and $B_s \cong 0$.

used with the proper interpretation of the bearing-capacity equa-

tion d_i factors dropped) to give

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

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

to estimate k_s . In these equations the Terzaghi or Hansen bearing-

capacity is used. The C factor is 40 for SI units and 12 for Fps, using the same

settlement but with no SF, since this equation is based on a settlement

of 0.0254-m and 1-in. settlement but with no SF, since this equation

is based on a settlement of 0.0254-m and 1-in. settlement but with no SF, since this equation

is based on a settlement of 0.0254-m and 1-in. settlement but with no SF, since this equation

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

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

depth of interest, say, the length of a pile

depth of interest

estimate of the exponent

to estimate a value of k_s to determine the correct order of magnitude

obtained using one of the approximations given here. Obviously if a

value is three times larger than the table range indicates, the computations

will have a possible gross error. Note, however, if you use a reduced value of

of 12 mm) instead of 0.0254 m you may well exceed the table range.

If a computational error (or a poor assumption) is found then use judgment

to estimate the exponent. The reader should not use, say, a

value even as a "good" estimate.

shown in Fig. 9-9c (and used in your diskette program FADBEMLP as

estimated at some small value of, say, 6 to 25 mm, or from inspection

of a load test was done. It might also be estimated from a triaxial

test "ultimate" or at the maximum pressure from the stress-strain plot.

compute

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

Range of modulus of subgrade

reaction k_s

Use values as guide and for comparison when using approximate equations $\frac{kN}{M^3} \rightarrow \frac{lb}{in^3} : \frac{224.8 lb}{1 kN} * \frac{1M^3}{61023.7 in^3} = .003684 \frac{kN}{M^3} = 1 \frac{lb}{in^3}$

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

354 pci

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

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

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

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

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

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

$$E'_s = \frac{1 - \mu^2}{E_s} = \frac{1 - 0.3^2}{11.72} = 0.07765 \text{ m}^2/\text{MN}$$

For center:

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

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

From these we may write

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

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

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

$$k_s = \frac{1}{0.61(0.07765)(4 \times 0.597)(0.8)} = 11.05 \text{ MN/m}^3$$

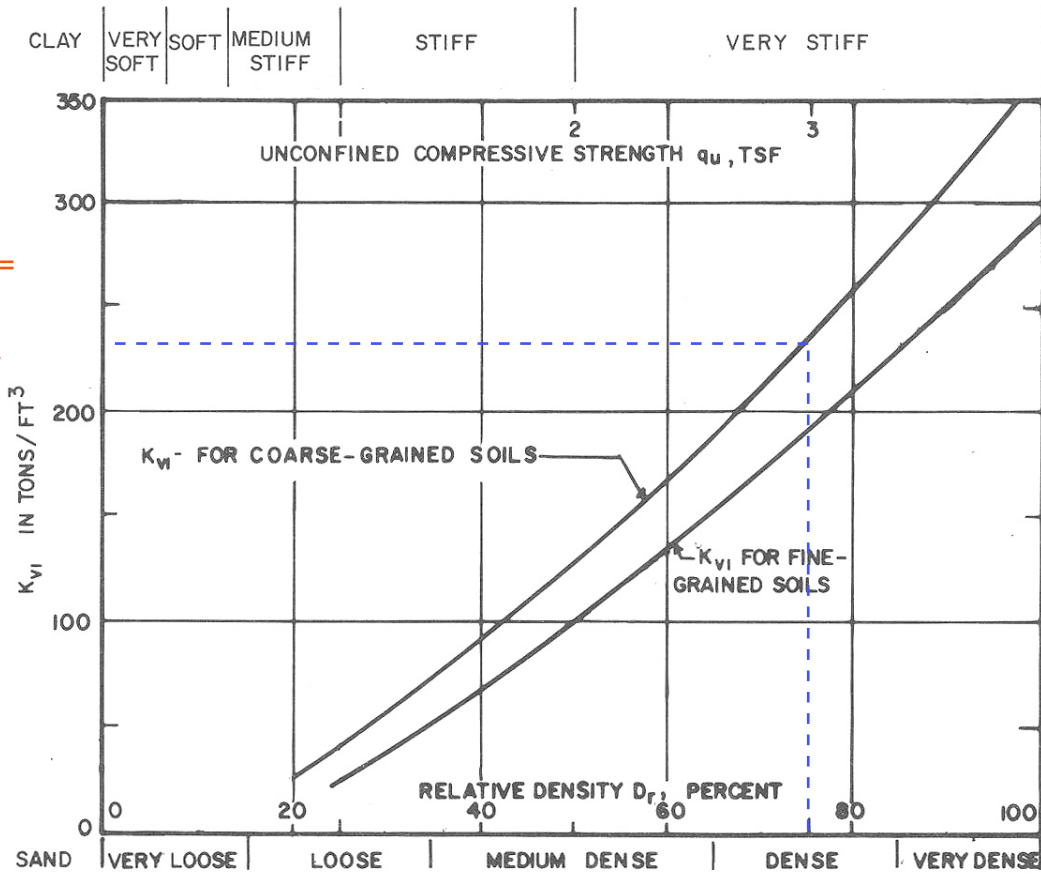
You should note that k_s does not depend on the contact pressure of the base q_o .

For corner:

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

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

225 TCF =
260 PCI
Reduce per
Note 2



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING
 q = FOOTING UNIT LOAD IN tsf
 B = FOOTING WIDTH

D = DEPTH OF FOOTING BELOW GROUND SURFACE

K_{vi} = MODULUS OF VERTICAL SUBGRADE REACTION

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

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)
 SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

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

FOR $B \geq 40$ FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

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

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

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

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

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

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

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

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

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

or

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

where

E_s = modulus of elasticity of soil

B = foundation width

E_F = modulus of elasticity of foundation material

I_F = moment of inertia of the cross section of the foundation

μ_s = Poisson's ratio of soil

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

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

Soil type	$k_{0.3}(k_1)$ MN/m ³	pci
Dry or moist sand:		
Loose	8–25	29 - 92
Medium	25–125	92 - 461
Dense	125–375	461 - 1382
Saturated sand:		
Loose	10–15	37 - 55
Medium	35–40	129 - 147
Dense	130–150	478 - 553
Clay:		
Stiff	10–25	37 - 92
Very stiff	25–50	92 - 184
Hard	>50	> 184

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

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

EVALUATION OF COEFFICIENTS OF SUBGRADE REACTION

General procedure

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

Vertical subgrade reaction

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

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

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

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

Dense sand

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

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

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

$$300 \text{ ton/ft}^3 \times 1.157 = 347 \text{ pci}$$

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

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

Values Range Proposed For rec High

If the subgrade ately in simple pr basis of our presen numerical values of pressures which ar The latter is indep

The proposed v medium sand, Tab of the loaded area normally consolida beams and rafts sl perfectly rigid.

The \bar{k}_{s1} values of the tests can be of such tests is to the test results can of the block shoul

If the contact the value :

For $l = \infty$, $k_{s1} =$ loaded subgrade 1

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

Foundations on Sandy Soils

For foundations on sandy soils,

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

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

Foundations on Clays

For foundations on clays,

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

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

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

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

Method 1:

where

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

Frost

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

From Design Freezing Index Map: Byron, **Maine**

DFI = 1900 degree-days.

Case 1 - coarse grained granular fill soils W=10% (assumed).

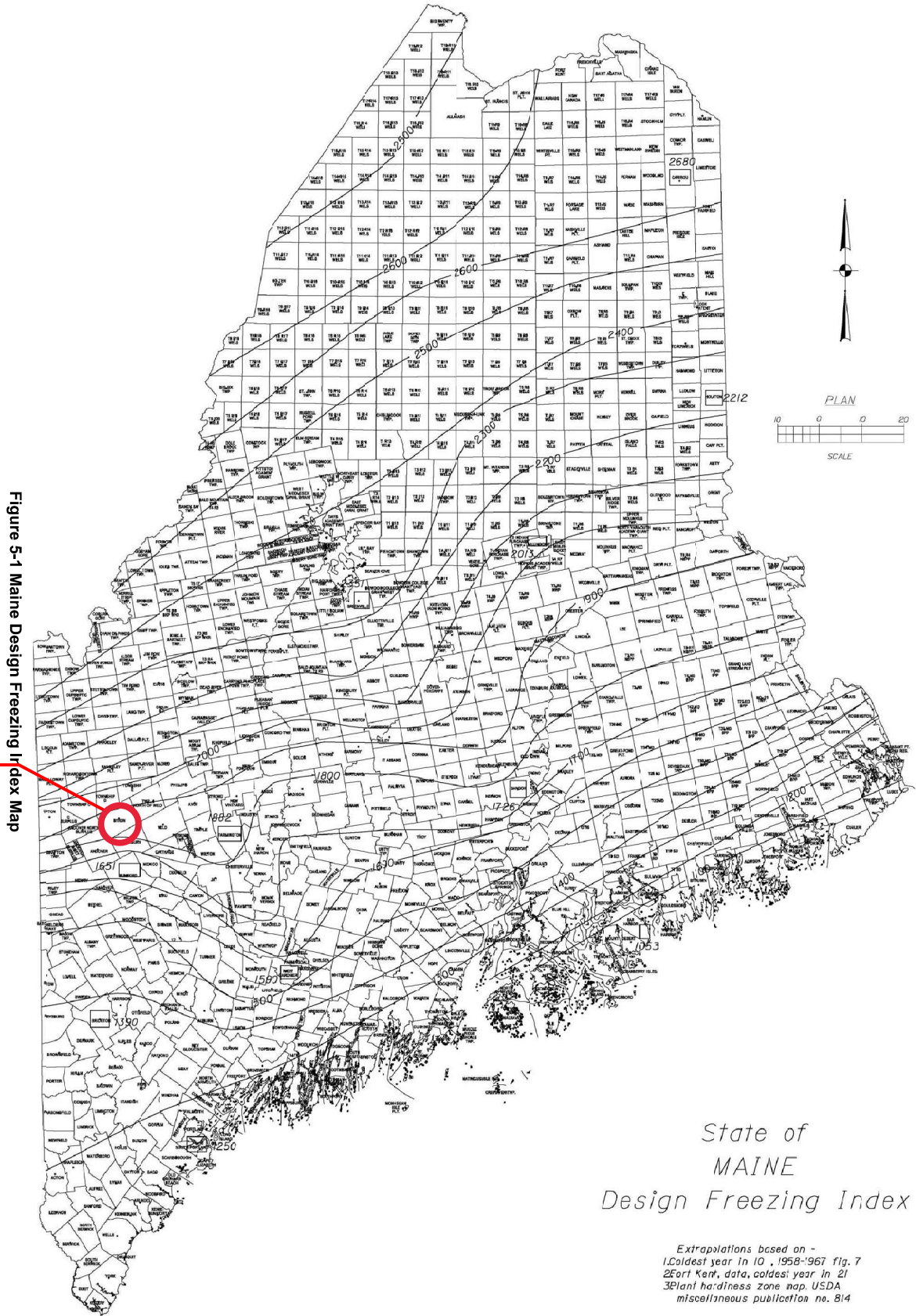
For DFI = 1900

at w=10%

d := 92.6in

Depth of Frost Protection

d = 7.72·ft



Project Location

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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