

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

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**GREAT BROOK BRIDGE
U.S. ROUTE 1 OVER GREAT BROOK
CAMDEN, MAINE**



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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and geotechnical recommendations for the replacement of Great Brook Bridge, which carries U.S. Route 1 over Great Brook in Camden, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation design recommendations and geotechnical design parameters for the design of the new bridge foundations.

Great Brook Bridge was constructed in 1934 and is a three-sided concrete frame structure with a span of 11 feet and a rise of approximately 6 feet. The total length of the culvert is 48 feet. The deck slab is supported by concrete capped and extended granite masonry abutments with integral concrete return walls. The concrete deck slab is structurally deficient and does not pass an HL-93 load rating. The structure has several places of concrete spalling, including the bottom of the concrete deck slab. There is moderate undermining at the upstream wingwalls.

A 2019 Maine Department of Transportation (MaineDOT) inspection report rates the deck slab/superstructure as “Poor” and the substructure as “Fair.” The bridge has a sufficiency rating of 45.3. Since the existing three-sided frame bridge is in poor condition and does not meet the load rating requirements, the project has been scoped as a bridge replacement. The bridge replacement will coincide with a highway reconstruction project on U.S. Route 1 in Camden.

A “detail-build” method of contracting will be utilized to deliver and construct the Great Brook Bridge replacement project. The contract “detail-build” special provision will require that the new bridge be a 13-foot span, precast concrete three-sided frame or arch (herein referred to as “frame”) on cast-in-place concrete footings (herein referred to as “pedestal walls”) on bedrock. The frame will have an approximate rise of 3.0 feet placed on cast-in-place 2-foot high (minimum) pedestal walls. The top of the inlet pedestal walls will be placed at Elev. 148.0 which is above the Q1.1 flow elevation. The wingwalls and headwalls on the upstream and downstream ends of the frame will be precast concrete or cast-in-place concrete. The length of the current bridge is just under 50 feet; the length of the replacement culvert is 100 feet.

The horizontal alignment will match the existing. The vertical alignment of the road will be raised by 2 feet. Traffic will be maintained by staged construction with one lane of alternating two-way traffic. The bridge replacement project will last one construction season.

2.0 GEOLOGIC SETTING

Great Brook Bridge in Camden carries U.S. Route 1 over Great Brook approximately 2.4 miles south of McKay Road, as shown on Sheet 1 – Location map.

The Maine Geologic Survey (MGS) Surficial Geology of the Lincolnville Quadrangle (2013), Open-File No. 13-7, indicates the surficial soils in the vicinity of the bridge project consist of glacial till. Glacial till is a loose to very compact, poorly sorted, massive to weakly stratified mixture of sand, silt, and gravel-size rock debris deposited by glacial ice.

The MGS Bedrock Geologic Map of Maine (1985) cites the bedrock at the bridge as a Pelite of the Megunticook Formation. The bedrock cores recovered in subsurface investigation were identified as a Metasandstone of the Megunticook Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two test borings on either side of the existing bridge culvert. Test boring BB-CGB-101 was located at the southwest corner and drilled in the northbound travel lane of U.S. Route 1. Test boring BB-CGB-102 was located at the northeast corner in the southbound travel lane of U.S. Route 1. The borings are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile provided at the end of this report.

The borings were drilled on June 24 and 25, 2015 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, soil and bedrock conditions, and groundwater conditions encountered are presented in the boring log provided in Appendix A – Boring Logs and on Sheet 3 - Boring Logs found at the end of this report.

The borings were drilled using cased wash boring, rock coring and solid stem auger techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633-05 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in March 2010 prior to the borings. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.908 to the raw field N-values. This hammer efficiency factor (0.908) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring log.

The borings were advanced to bedrock and terminated with a bedrock core. The bedrock was cored using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core was calculated.

A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations, drilling methods, designated type and depth of sampling techniques, reviewed the draft boring log and identified field and

laboratory testing requirements. The borings were located in the field using taped measurements at the completion of the drilling programs.

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 three standard grain size analyses with natural water contents. The results of soil tests are included as Appendix C – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings consisted of embankment fills underlain by native soils and bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered in detail.

5.1 Fill

A layer of granular fill was encountered in both borings. The encountered thickness was approximately 4 feet at the boring locations and generally consisted of:

- Brown, damp, gravel, some sand, some silt;
- Brown, damp, sand, little silt, some gravel.

SPT N-values in the fill soils ranged from 30 to 33 blows per foot (bpf), indicating the fill soils are medium dense to dense in consistency. Two grain size analyses conducted on samples from the fill unit classified the soils as A-1-b and A-2-4 under the AASHTO Classification System and as GM and SM under the United Soil Classification System (USCS). The moisture contents of the samples ranged from approximately 3 to 6 percent.

5.2 Native Sand

A deposit of native sand was encountered in boring BB-CGB-101. The encountered thickness was approximately 4.5 feet at the boring location and the deposit consisted of brown, damp, sand, some silt, little gravel.

One SPT N-value in the sand deposit was 6 bpf indicating the soil is loose in consistency. One grain size analysis conducted on a sample from the deposit classified the soil as A-2-4 under the AASHTO Classification System and as SM under the USCS. The moisture content of the sample was approximately 13 percent.

5.3 Bedrock

Bedrock was encountered and cored at depths of 4.5 and 9.5 feet below the roadway surface in the borings. Bedrock outcrops are visible in the streambed of Great Brook upstream and downstream of the existing culvert, and in the natural stream bottom inside the culvert.

The bedrock at the site is identified as white to medium grey, salt and peppery, medium grained, METASANDSTONE, very hard, fresh, joints/fractures at low angles to moderately dipping, moderately closely spaced and tight. There was little discernable geologic structure or fracturing. The RQD's of the bedrock cores were 90 and 100 percent, correlating to a rock quality of Good to Excellent. Detailed bedrock descriptions and RQD core run are provided on the boring logs in Appendix A – Boring Logs. Photographs of bedrock cores are provided in Appendix B – Bedrock Core Photographs.

Table 1 summarizes approximate bedrock surface elevations outside the rise of the existing culvert.

Boring	Station	Offset (feet)	Approx. Depth to Bedrock (feet)	Approx. Elevation of Bedrock Surface (feet)	RQD
BB-CGB-101	156+44.7	10.3 Rt.	9.5	142.7	90%
BB-CGB-102	157+28.9	7.4 Lt.	4.5	147.9	100%

Table 1. Summary of Approximate Bedrock Surface Depths, Elevations and RQD

5.4 Groundwater

Groundwater was not observed in the boreholes. Water was introduced into the boreholes during drilling operations. Therefore, water levels may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in river water elevation, seasonally, with precipitation, runoff, and construction activities.

6.0 BRIDGE AND FOUNDATION ALTERNATIVES

The Preliminary Design Report (PDR) (MaineDOT, 2019) investigated three bridge replacement options. The alternatives were:

- a three-sided, precast concrete frame with cast-in-place footings;
- a precast concrete Con/Span arch with cast-in-place footings;
- a steel multi-plate arch with cast-in-place footings.

A “detail-build” method of contracting will be utilized to deliver and construct the Great Brook Bridge replacement project. Based on the recommendations in the PDR, the contract “detail-build” special provision will require that the new bridge be a 13-foot span, precast concrete three-sided frame or arch (herein referred to as “frame”) on cast-in-place concrete footings (pedestal walls) on bedrock. The frame will have an approximate rise of 3.0 feet placed on cast-in-place 2-foot high (minimum) pedestal walls. The top of the inlet pedestal walls will be placed at Elev. 148.0 which is above the Q1.1 flow elevation. The wingwalls and headwalls on the upstream and downstream ends of the frame will be precast concrete modular gravity walls or cast-in-place concrete cantilever-type walls.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The project “detail-build” special provision will require cast-in-place spread footings on bedrock to support the bridge replacement alternative and wingwall alternative chosen by the contractor. The design recommendations in this Section are provided in accordance with AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 (herein referred to as LRFD).

7.1 Precast Concrete Frame and Arch Design and Construction

Precast concrete arches and frames (herein referred to as “frames”) will be 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. Precast concrete frames shall be designed in accordance with MaineDOT Standard Specification 534 - Precast Structural Concrete, MaineDOT Special Provision 531 – Detail-Build Bridge Structure, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures and AASHTO LRFD. The loading specified for the design of the structure 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 design should use Soil Type 4 as presented in the MaineDOT Bridge Design Guide (BDG) Section 3.6 to design earth loads from the soil envelope. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

The precast concrete frame shall be constructed in conformance with MaineDOT BDG Section 8 and MaineDOT Standard Specification 534. 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.

7.2 Spread Footings on Bedrock

The test borings conducted at the site encountered the bedrock surface at Elevations 142.7 and 147.9. These bedrock elevations are both above and below the bottom of the existing culvert. Since the span of the replacement precast frame is wider than the current span, the contractor should expect that bedrock excavation will be required in some areas and pedestal walls higher than the minimum 2 feet will be required at other locations.

The borings indicate that the bedrock is very hard, massive, with little fracturing, and has an approximate RQD correlating to a rock quality that is Good to Excellent. Excavation of bedrock material may be done using conventional excavation methods but will likely require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.7 of the MaineDOT Standard Specifications.

The thickness of the precast frame pedestal walls and wingwall footings may be designed to vary in thickness to accommodate variations in the bedrock surface with the top elevation of the pedestal walls as shown on the Plans.

7.3 Bearing Resistance of Spread Footings on Bedrock

Cast-in-place spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads shall be as specified in LRFD Article 11.5.5.

Nominal and factored bearing resistances were calculated for pedestal wall and wingwall spread footings using a Rock Mass Rating (RMR)-based empirical correlation² in accordance with LRFD Article C10.4.6.4 and 10.6.2.6.2. RMR was evaluated in accordance with Table 10.4.6.4-1 of the 2012 LRFD code, whereas the current edition of LRFD does not include the RMR formulation.

The bearing resistance of cast-in-place spread footings constructed on bedrock shall be investigated at the service limit state using factored loads and a factored bearing resistance of 40 ksf. Resistance factors for the service limit state are taken as 1.0. A service limit state bearing resistance of 40 ksf shall also be used for preliminary footing sizing as allowed in LRFD C10.6.2.1.

Based on the available data and the RMR-based methodology, the calculated factored bearing resistance at the strength limit state is 146 ksf, assuming a resistance factor, ϕ_b , for spread footings on bedrock of 0.45. From a practical perspective we recommend that the footing have a minimum width of 3 feet and the factored bearing resistance be limited to 90 ksf. However, the service limit state bearing resistance may govern the design.

The vertical bearing stress shall be calculated assuming a triangular or trapezoidal pressure distribution over an effective base as shown in LRFD Figure 11.6.3.2-2 for foundations on

² "Foundations on Rock" by Duncan Wyllie.

rock. In no instance shall the bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'_c$.

From a practical standpoint, footings shall have a minimum width of 3 feet wide regardless of the bearing resistance or bearing material.

For footings on rock, the location of the resultant of the reaction forces shall be within the middle nine-tenths (9/10) of the base width.

7.4 Frame Footing and Wingwall Spread Footing Design

Pedestal walls for precast frames and footings for wingwalls shall be designed for all relevant strength, service and extreme limit state load combinations specified in AASHTO LRFD Articles 3.4.1 and 11.5.5 and 12.5. Pedestal walls and spread footings shall be designed to resist all lateral earth loads, vehicular loads, arch/frame deadloads, and lateral thrust forces transferred through the bridge frames. The geotechnical design of spread footings at the strength limit state shall consider:

- bearing resistance,
- eccentricity,
- lateral sliding.

For the scour protection of precast frame spread footings and wingwall footings, project plans shall require construction of footings directly on bedrock surfaces cleaned of all soil, loose, fractured rock or potentially erodible rock. Further, the strength limit state design shall consider changes in foundation resistance after the design flood for scour. Buried structures and their wingwalls should be designed so that no movement of any part of the structure will occur as a result of scour.

For sliding analyses, a sliding resistance factor, ϕ_τ , of 0.80 shall be applied to the nominal sliding resistance of cast-in-place frame spread footings and cast-in-place wingwall spread footings constructed on bedrock.

Assuming that the rock subgrade will be prepared in-the-wet, some amount of sediment is expected to remain on the rock surface and the sliding computations for resistance of footings to lateral loads shall assume a maximum sliding resistance (friction) coefficient ($C \times \tan\phi_f$) of 0.60 at the bedrock-to-concrete interface assuming a level bedrock surface. If the rock subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance to lateral loads may assume a maximum sliding resistance (friction) coefficient of 0.70 at level bedrock-to-concrete interfaces, assuming a level bedrock surface.

Passive resistance shall be neglected in sliding and stability calculations.

If the exposed bedrock surface is steeper than 4H:1V at the footing subgrade elevation, the bedrock should be benched to create level steps. Alternatively, anchoring or doweling the footings to bedrock may be utilized to resist sliding forces and improve stability. Dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the engineer.

For spread footings cast directly on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 of the footing dimensions, in either direction. This eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the footing.

For the service limit state, a resistance factor, ϕ , of 1.0 shall be used to assess spread footing design for settlement, horizontal movement, bearing resistance, sliding and eccentricity. The overall global stability of foundations is typically investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the foundations, and therefore a global stability evaluation may be waived.

7.5 Frame Pedestal Walls - Earth Pressure and Surcharge Forces

Calculation of earth pressures acting on frame pedestal walls should assume an at-rest earth pressure coefficient, K_o , of 0.47, assuming the frame pedestal wall footings are to be prevented from movement. Minimum and maximum load factors for lateral at-rest and vertical earth pressures, for rigid frames and pedestal walls, are specified in LRFD Table 3.4.1-2.

The designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for frame pedestal wall backfill soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. The live load surcharge on frame pedestal wall footings may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 2 below:

Pedestal Wall Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
≥ 20	2.0

Table 2. Equivalent Height of Soil for Estimating Live Load Surcharge

Bridge frame pedestal walls shall include a drainage system behind the walls to intercept any groundwater. Drainage behind the structure shall be in accordance with BDG Section 5.4.1.4 - Drainage.

Backfill within 10 feet of arches, frames and wingwalls shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

7.6 Concrete Headwalls and Inlet/Outlet Wingwalls – Earth Pressures and Surcharge Forces

Concrete headwalls may be included in the buried structure design to retain riprap slopes and prevent riprap from dropping or eroding into the waterway. Nominal 1-foot by 1-foot concrete headwalls are recommended.

Larger precast or cast-in-place concrete headwalls and wingwalls shall be designed as conventional retaining walls for all relevant strength and service limit state load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist all lateral earth loads, live loads, creep and temperature and shrinkage deformations of the concrete arch. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 3 below:

Retaining Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic = 0 feet	Distance from wall pressure surface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥ 20	2.0	2.0

Table 3. Equivalent Height of Soil for Estimating Live Load Surcharge on Walls

Headwalls that are fixed to the arch or frame should be designed using an at-rest earth pressure coefficient, K_o , of 0.47, assuming the walls are to be prevented from movement.

Inlet and outlet wingwalls that are independent of the frame structure shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure using the Rankine active earth pressure coefficient, K_a , of 0.31 for level backfills, and a K_a of 0.47 for a backslope of 2H:1V. The designer may assume Soil Type 4 (MaineDOT BDG 3.6.1) for headwall and wingwall backfill soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

Footings for wingwalls at the frame inlet and outlet shall be cast directly on bedrock and meet the design requirements for spread footings on bedrock as specified in Sections 7.2, 7.3 and 7.4 of this report.

Inlet and outlet wingwall designs shall include a drainage system behind the wall stems to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

7.7 Precast Concrete Modular Gravity Walls

The contract documents may permit Precast Concrete Modular Gravity (PCMG) walls for the inlet and outlet wingwalls. In general, PCMG wingwalls should only be used in streams where the flow velocities are low, and the potential for scour is low. The PCMG walls shall be designed by a Professional Engineer in accordance the Standard Specification 674.

The bottom PCMG wall units shall be installed on a minimum 6 by 12-inch, cast-in-place concrete leveling pad. The concrete leveling pad shall be cast directly on bedrock, or on concrete fill sub-footing that is cast directly on bedrock. The excavation behind the PCMG wall facing shall be excavated to bedrock and area backfilled with compacted crushed stone prior to installing the base PCMG units.

The bearing resistance for PCMG walls constructed on crushed stone over bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 7 ksf. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state. The vertical stress may be calculated assuming a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1.

The PCMG 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}) from Table 3 of this report.

For the lowest PCMG unit, with the unit stem on compacted crushed stone, the location of the resultant of the reaction forces at the strength limit state should be within the middle two-thirds ($2/3$) of the footing width.

Failure by sliding shall be investigated by the PCMG wall designer-supplier. A sliding resistance factor, ϕ_r , of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall stems bearing on crushed stone. A sliding resistance factor of 0.90 shall be applied to the nominal sliding resistance of soil within the precast concrete units on granular bedding soils. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of $0.46 = 0.80 \times \tan 30^\circ$ (LRFD Eq. 10.6.3.4-2) at the foundation soil to concrete unit interfaces and a maximum frictional coefficient of $0.58 = \tan 30^\circ$ at foundation soil to soil-infill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Articles 11.11.4.2 and 10.6.3.4 and Table 10.5.5.2.2-1.

Passive resistance shall be neglected in sliding and stability calculations.

7.8 Settlement

A 2-foot raise in the roadway grade is planned at the bridge site. The granular fill soils encountered beneath the bridge approaches are generally medium dense in consistency. These coarse-grained materials undergo elastic, immediate compression in response to an increase in the vertical overburden pressure. As a result, any settlement is anticipated to be small and will occur relatively quickly during construction.

The backfill for the precast frame 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 minimize post-construction settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

We anticipate that all foundations will be constructed on bedrock. Per LRFD 10.6.2.4.4., for footings constructed on rock with an RMR-based rock quality of Fair to Very Good, elastic settlements can generally be assumed to be less than ½-inch.

7.9 Frost Protection

The pedestal wall footings of the frame, and its wingwalls, will be constructed directly on bedrock. For foundations on bedrock heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary.

Any foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Camden has a design freezing index of approximately 1275 F-degree days. An assumed water content of 10% was used for coarse-grained soils at the potential elevation of a footing. These components correlate to a frost depth of 6.3 feet.

We recommend that any foundations constructed on soil be designed with an embedment of 6.3 feet for frost protection.

7.10 Scour and Riprap

The precast bridge structure and wingwalls will be founded on spread footings founded on bedrock. For scour protection of the precast frame and wingwall footings, construct the footings directly on bedrock surfaces cleaned of soil, highly weathered rock and potentially erodible rock. All loose, highly fractured bedrock shall be removed by ripping. We anticipate that the remaining bedrock subgrade will be competent and is therefore not considered to be erodible or scourable. Therefore, no specific scour protection recommendations are needed for the foundations other than armoring with riprap.

Wingwalls shall be extended far enough from the structure to protect the structural portion of the soil envelope surrounding the bridge structure.

We recommend that sideslopes and footings supporting the structures be armored with a minimum 3-foot thick layer of riprap conforming to 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 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 unless the streambed consists of bedrock. The riprap slopes shall be constructed no steeper than a maximum 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

7.11 Seismic Design Considerations

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

7.12 Construction Considerations

Construction of the precast frame pedestal walls and wingwalls will require soil and rock excavation and removal of the existing culvert. Cofferdams and temporary earth support systems may be required to permit construction of frame and wingwall footings in the dry.

Construction activities should not be permitted to disturb the bedrock mass or create any open fissures. Irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with unreinforced concrete to the bearing elevation. Footings may be stepped for varying depths to bedrock along the centerline of the footing. The bottom of the footing elevations may vary based on the presence of fractured bedrock and variable bedrock.

The subgrade for spread footings for frames and wingwalls shall consist of sound bedrock. The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is made. The bedrock subgrade surface shall be cleaned of all overburden soils and loose, dislodged bedrock fragments by mechanical means. Mechanical means include expansive agents, hydraulic hoe ram, hydraulic splitters or wedging and prying. The final bearing surface of bedrock shall be washed with high pressure water and air prior to concrete being placed for the frame footings and wingwall footings.

The slope of the bedrock subgrade for foundations shall be no steeper than 4H:1V or it shall be benched in level steps or excavated to be completely level. This criterion also applies for the bedrock subgrade for any wingwall footings. Anchoring, doweling or other means of improving sliding resistance may also be employed where the prepared bedrock surface is steeper than 4H:1V in any direction.

The borings indicate that the site bedrock is very hard, massive, with little fracturing. Excavation of bedrock material may be done using conventional excavation methods and mechanical means but may likely require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.7 of the MaineDOT Standard Specifications. It

is also recommended that the contractor conduct pre-and post-blast surveys, at the nearby residences, and bridge structures in accordance with industry standards at the time of the blast.

The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Surface water should be diverted from the foundation excavation throughout the period of construction. Water encountered at the base of the foundation excavation should be removed by using a sump pump located in the corner of the excavation outside of the foundation footprint. The contractor should maintain the excavation so that all foundations are constructed in the dry.

Exposed soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

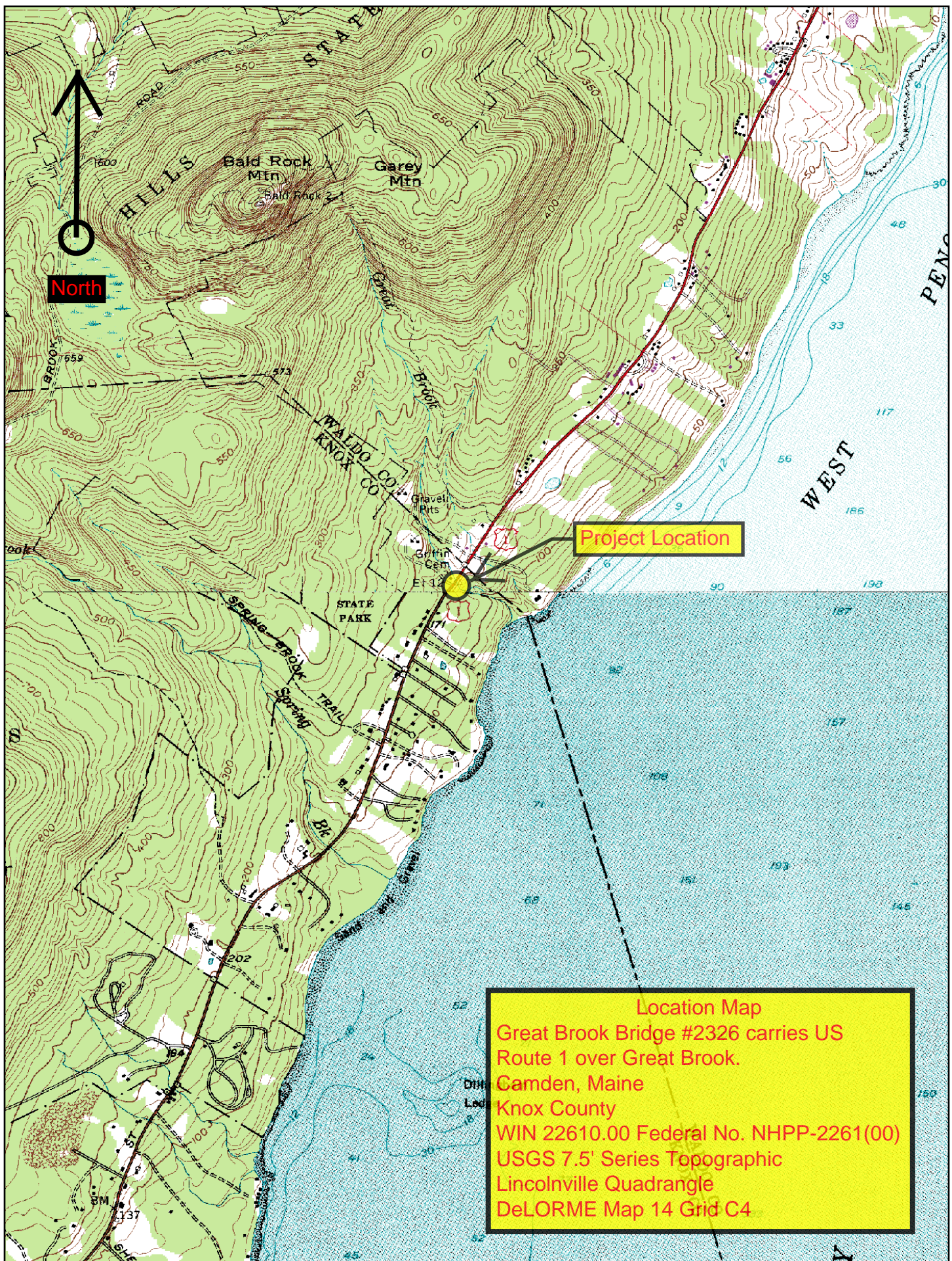
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Great Brook Bridge on U.S. Route 1 in Camden, 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. Further, these analyses and recommendations are based in part upon limited subsurface explorations 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.

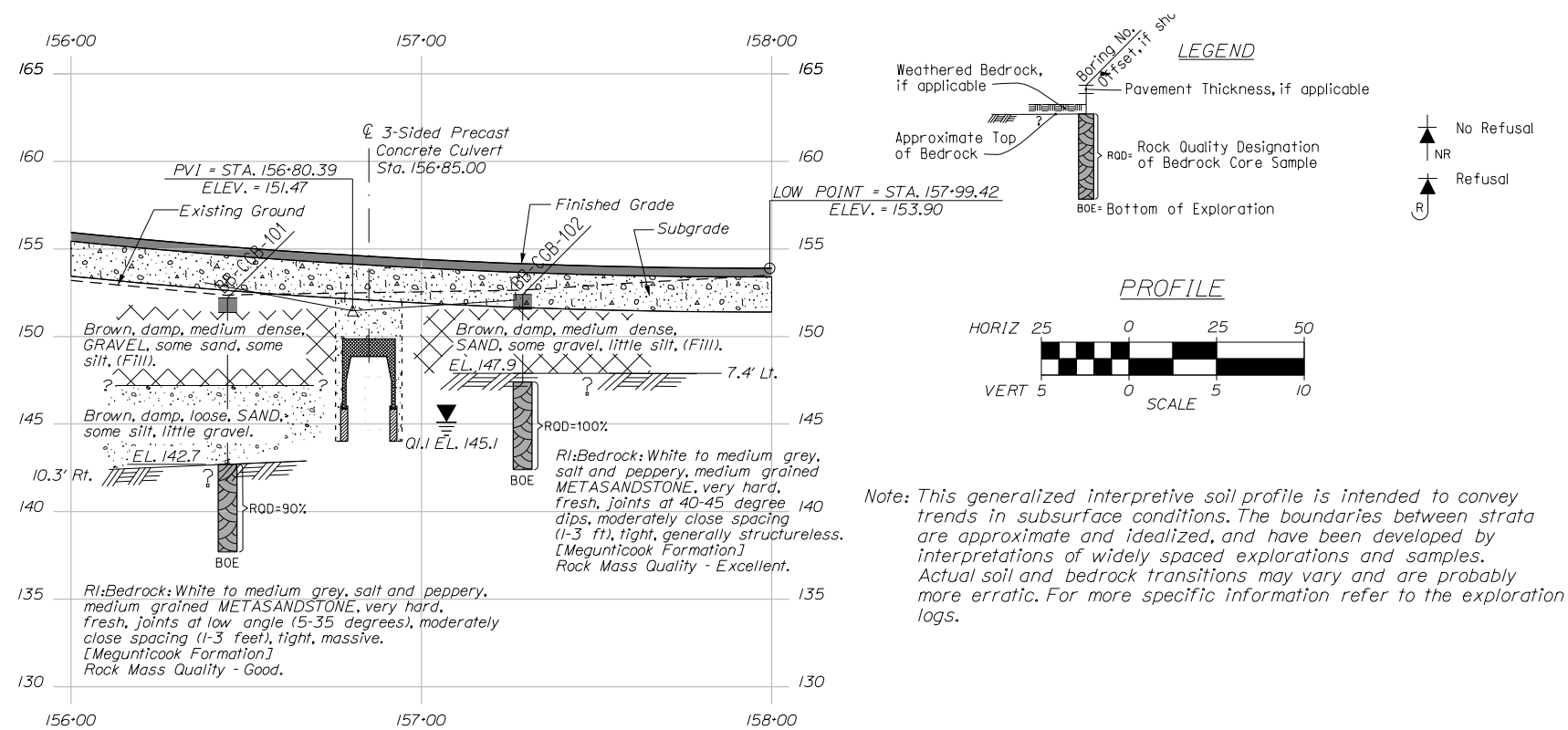
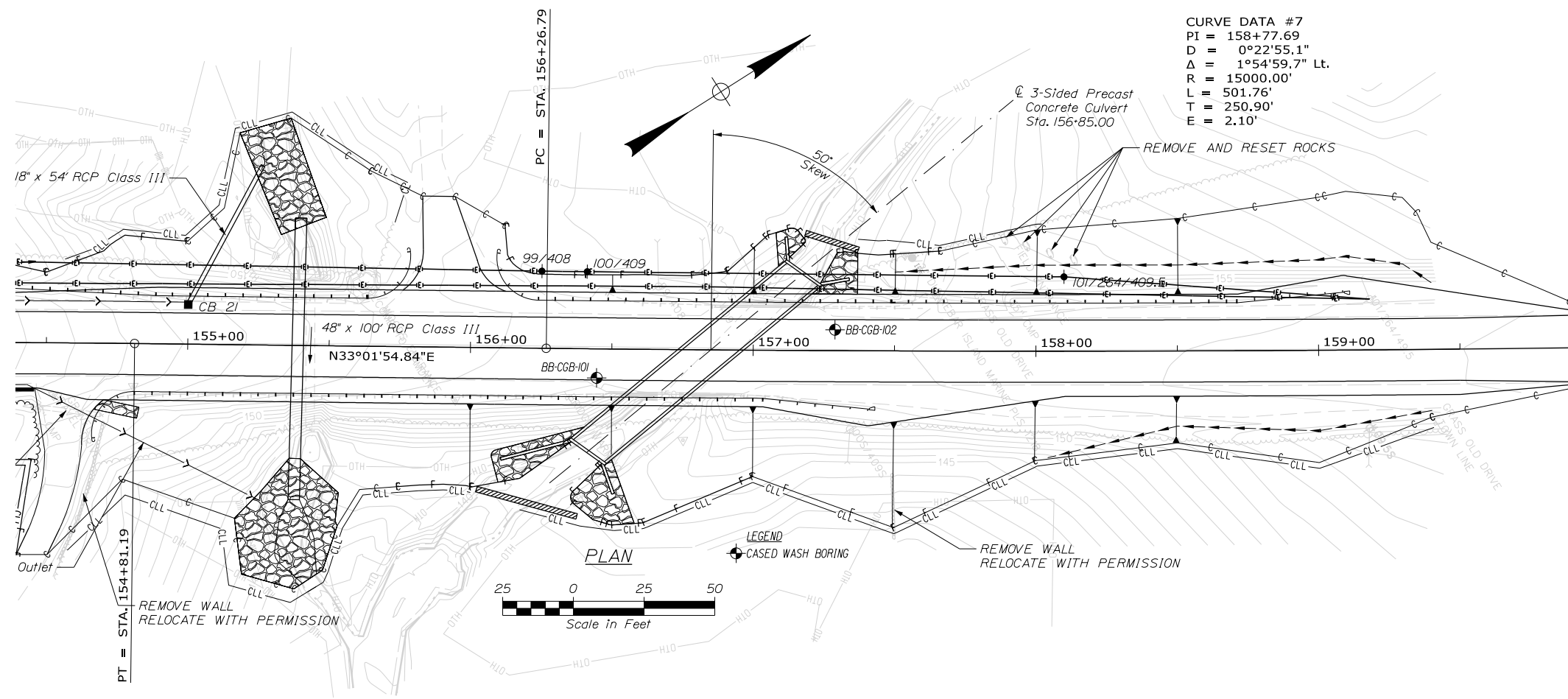
It is recommended that the geotechnical engineer be provided the opportunity for a review of the final design and specifications so that the earthwork and foundation recommendations and construction considerations in this report are properly interpreted and implemented in the design and specifications.

Sheets



Map Scale 1:24000

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. Road names used on this map may not match official road names.



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.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS										Project: Great Brook Bridge #2326 carries US Route 1 over Great Brook Location: Camden, Maine										Boring No.: BB-CGB-101 WIN: 22610.00									
Driller: MaineDOT										Elevation (ft.): 152.2										Auger ID/DD: 5" Solid Stem									
Operator: Giles/Daggett/Giles										Datum: NAVD88										Sampler: Standard Split Spoon									
Logged By: B. Wilder										Rig Type: CME 45C										Hammer Wt./Fall: 140#/30"									
Date Start/Finish: 6/24-25/2015: 5.0 hrs										Drilling Method: Cased Wash Boring										Core Barrel: NO-2"									
Boring Location: 156+44.7, 10.3 ft Rt.										Casing ID/DD: NW										Water Level*: None Observed									
Hammer Efficiency Factor: 0.908										Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>																			
<div>Definitions: D = Split Spoon Sample MB = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test MV = Unsuccessful Field Vane Shear Test Attempt</div> <div>R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone MBH = Weight of 140lb. Hammer MBHC = Weight of Rod or Casing MBHP = Weight of One Person</div> <div>S_u = Peak/Remained Field Vane Undrained Shear Strength (psf) S_{u(10)} = Lab Vane Undrained Shear Strength (psf) q_u = Unconfined Compressive Strength (psf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = Hammer Efficiency Factor/60%N-uncorrected</div> <div>WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit C = Consolidation Test</div>																													
Sample Information																													
Visual Description and Remarks																													
Laboratory Testing Results/ AASHTO and Unified Class																													
10" Pavement																													
Brown, damp, medium dense, GRAVEL, some sand, some silt, (fill).																													
GW264794 A-2-4, GM WC=6.2%																													
Brown, damp, loose, SAND, some silt, little gravel.																													
GW264796 A-2-4, SM WC=13.0%																													
0125 blows for 0.5 ft.																													
Top of Bedrock at Elev. 142.7 ft.																													
R1: Bedrock: White to medium grey, salt and peppery, medium grained, METASANDSTONE, very hard, fresh, joints at low angle (5-35 degrees), moderately close spacing (1-3 feet), tight, massive.																													
[Megunticook Formation]																													
Rock Mass Quality = Good.																													
R1: Core Times (min:sec)																													
9.5-10.5 ft (3:54)																													
10.5-11.5 ft (4:09)																													
11.5-12.5 ft (3:57)																													
12.5-13.5 ft (5:13)																													
13.5-14.5 ft (4:14) 100% Recovery																													
Bottom of Exploration at 14.5 feet below ground surface.																													
Remarks:																													
Stratification lines represent approximate boundaries between soil type transitions may be gradual.																													
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.																													
Page 1 of 1 Boring No.: BB-CGB-101																													

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS										Project: Great Brook Bridge #2326 carries US Route 1 over Great Brook Location: Camden, Maine										Boring No.: BB-CGB-102 WIN: 22610.00									
Driller: MaineDOT										Elevation (ft.): 152.4										Auger ID/DD: 5" Solid Stem									
Operator: Giles/Daggett/Giles										Datum: NAVD88										Sampler: Standard Split Spoon									
Logged By: B. Wilder										Rig Type: CME 45C										Hammer Wt./Fall: 140#/30"									
Date Start/Finish: 6/25/2015: 08:00-10:00										Drilling Method: Cased Wash Boring										Core Barrel: NO-2"									
Boring Location: 157+28.9, 7.4 ft Lt.										Casing ID/DD: NW										Water Level*: None Observed									
Hammer Efficiency Factor: 0.908										Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>																			
<div>Definitions: D = Split Spoon Sample MB = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test MV = Unsuccessful Field Vane Shear Test Attempt</div> <div>R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone MBH = Weight of 140lb. Hammer MBHC = Weight of Rod or Casing MBHP = Weight of One Person</div> <div>S_u = Peak/Remained Field Vane Undrained Shear Strength (psf) S_{u(10)} = Lab Vane Undrained Shear Strength (psf) q_u = Unconfined Compressive Strength (psf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = Hammer Efficiency Factor/60%N-uncorrected</div> <div>WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit C = Consolidation Test</div>																													
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10" Pavement																													
Brown, damp, dense, SAND, some gravel, little silt, (fill).																													
GW264796 A-1-b, SM WC=3.3%																													
Top of Bedrock at Elev. 147.9 ft.																													
Augered to 5.0 ft bgs and set NW casing.																													
R1: Bedrock: White to medium grey, salt and peppery, medium grained, METASANDSTONE, very hard, fresh, joints at 40-45 degree dips, moderately close spacing (1-3 ft), tight, generally structureless.																													
[Megunticook Formation]																													
Rock Mass Quality = Excellent.																													
R1: Core Times (min:sec)																													
5.0-6.0 ft (3:17)																													
6.0-7.0 ft (3:27)																													
7.0-8.0 ft (3:28)																													
8.0-9.0 ft (2:47)																													
9.0-10.0 ft (2:56) 100% Recovery																													
Bottom of Exploration at 10.0 feet below ground surface.																													
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Stratification lines represent approximate boundaries between soil type transitions may be gradual.																													
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Page 1 of 1 Boring No.: BB-CGB-102																													

Appendix A

Boring Logs

Maine Department of Transportation																																																																																																																																																																																																																																																																																																																																																																																																																																								
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<table><thead><tr><th rowspan="2">Depth (ft.)</th><th colspan="7">Sample Information</th><th rowspan="2">Elevation (ft.)</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/AASHTO Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (/6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th></tr></thead><tbody><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>SSA</td><td>151.4</td><td>[Pattern]</td><td>10" Pavement</td><td>G#264794 A-2-4, GM WC=6.2%</td></tr><tr><td></td><td>1D</td><td>24/14</td><td>1.00 - 3.00</td><td>9/11/9/10</td><td>20</td><td>30</td><td></td><td></td><td>[Pattern]</td><td>Brown, damp, medium dense, GRAVEL, some sand, some silt, (Fill).</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td>5</td><td>2D</td><td>24/9</td><td>5.00 - 7.00</td><td>4/2/2/2</td><td>4</td><td>6</td><td>11</td><td>147.2</td><td>[Pattern]</td><td>Brown, damp, loose, SAND, some silt, little gravel.</td><td>G#264795 A-2-4, SM WC=13.0%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td>10</td><td>R1</td><td>60/60</td><td>9.50 - 14.50</td><td>RQD = 90%</td><td></td><td></td><td>a125 NQ-2</td><td>142.7</td><td>[Pattern]</td><td>a125 blows for 0.5 ft. Top of Bedrock at Elev. 142.7 ft. R1:Bedrock: White to medium grey, salt and peppery, medium grained, METASANDSTONE, very hard, fresh, joints at low angle (5-35 degrees), moderately close spacing (1-3 feet), tight, massive. [Megunticook Formation] Rock Mass Quality = Good. R1:Core Times (min:sec) 9.5-10.5 ft (3:54) 10.5-11.5 ft (4:09) 11.5-12.5 ft (3:57) 12.5-13.5 ft (5:13) 13.5-14.5 ft (4:14) 100% Recovery</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td>15</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>137.7</td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td>20</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>[Pattern]</td><td></td><td></td></tr></tbody></table>										Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO 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	151.4	[Pattern]	10" Pavement	G#264794 A-2-4, GM WC=6.2%		1D	24/14	1.00 - 3.00	9/11/9/10	20	30			[Pattern]	Brown, damp, medium dense, GRAVEL, some sand, some silt, (Fill).											[Pattern]												[Pattern]												[Pattern]			5	2D	24/9	5.00 - 7.00	4/2/2/2	4	6	11	147.2	[Pattern]	Brown, damp, loose, SAND, some silt, little gravel.	G#264795 A-2-4, SM WC=13.0%										[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]			10	R1	60/60	9.50 - 14.50	RQD = 90%			a125 NQ-2	142.7	[Pattern]	a125 blows for 0.5 ft. Top of Bedrock at Elev. 142.7 ft. R1:Bedrock: White to medium grey, salt and peppery, medium grained, METASANDSTONE, very hard, fresh, joints at low angle (5-35 degrees), moderately close spacing (1-3 feet), tight, massive. [Megunticook Formation] Rock Mass Quality = Good. R1:Core Times (min:sec) 9.5-10.5 ft (3:54) 10.5-11.5 ft (4:09) 11.5-12.5 ft (3:57) 12.5-13.5 ft (5:13) 13.5-14.5 ft (4:14) 100% Recovery											[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]			15								137.7	[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]			20									[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]												[Pattern]			25									[Pattern]		
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10	R1	60/60	9.50 - 14.50	RQD = 90%			a125 NQ-2	142.7	[Pattern]	a125 blows for 0.5 ft. Top of Bedrock at Elev. 142.7 ft. R1:Bedrock: White to medium grey, salt and peppery, medium grained, METASANDSTONE, very hard, fresh, joints at low angle (5-35 degrees), moderately close spacing (1-3 feet), tight, massive. [Megunticook Formation] Rock Mass Quality = Good. R1:Core Times (min:sec) 9.5-10.5 ft (3:54) 10.5-11.5 ft (4:09) 11.5-12.5 ft (3:57) 12.5-13.5 ft (5:13) 13.5-14.5 ft (4:14) 100% Recovery																																																																																																																																																																																																																																																																																																																																																																																																																														
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[illegible]

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.	<u>Descriptive Term</u>		<u>Portion of Total (%)</u>	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace	0 - 10		
					little	11 - 20		
					some	21 - 35		
					adjective (e.g. sandy, clayey)		36 - 50	
	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.	TERMS DESCRIBING DENSITY/CONSISTENCY			
		GC	Clayey gravels, gravel-sand-clay mixtures.					
		CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	<u>Density of Cohesionless Soils</u>		<u>Standard Penetration Resistance N-Value (blows per foot)</u>	
		(little or no fines)	SP	Poorly-graded sands, gravelly sand, little or no fines.	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 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.	<u>Fine-grained soils</u> (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.				
		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.	<u>Approximate Undrained Shear Strength (psf)</u>		<u>Field Guidelines</u>		
		SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	<u>Consistency of Cohesive soils</u>	<u>SPT N-Value (blows per foot)</u>		
			CH	Inorganic clays of high plasticity, fat clays.	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	
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.	Rock Quality Designation (RQD):					
			RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core)					
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					Correlation of RQD to Rock Mass Quality			
					<u>Rock Mass Quality</u>		<u>RQD (%)</u>	
					Very Poor		≤25	
					Poor		26 - 50	
					Fair		51 - 75	
					Good		76 - 90	
					Excellent		91 - 100	
					Desired Rock Observations (in this order, if applicable):			
					Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
					Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information			
WIN		Blow Counts						
Bridge Name / Town		Sample Recovery						
Boring Number		Date						
Sample Number		Personnel Initials						
Sample Depth								

Appendix B

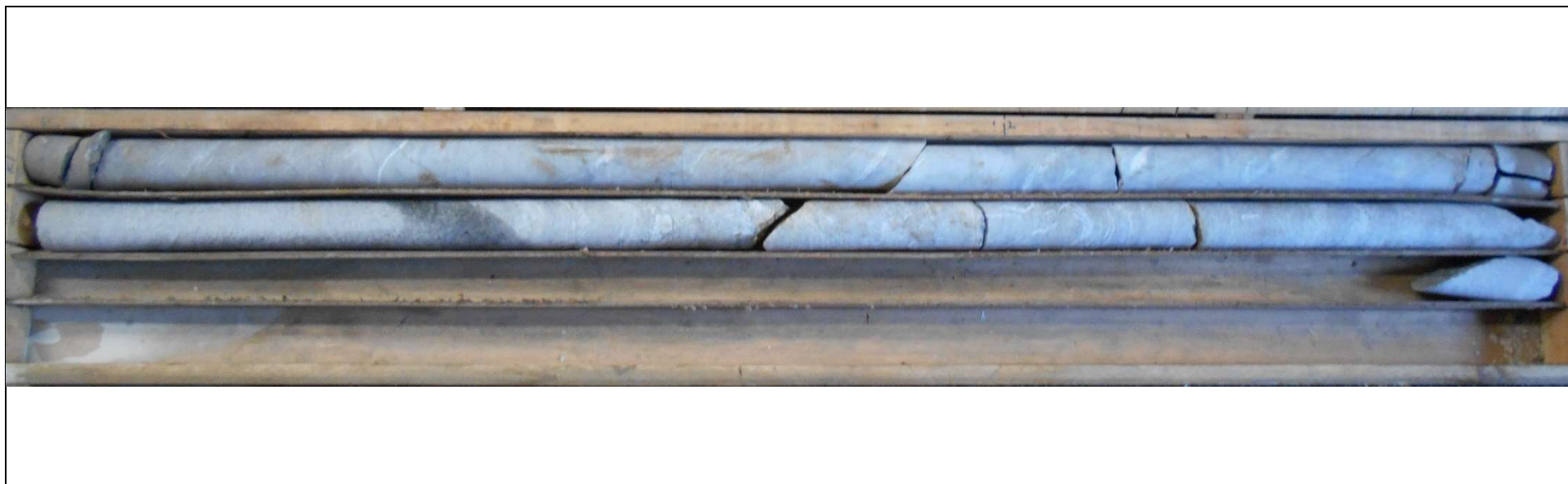
Bedrock Core Photographs



**Great Brook Bridge #2326 Carries US Route 1 Over Great Brook
Camden, ME**

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Penetration (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CGB-101	R1	9.5 – 14.5	60	60	54	90%	METASANDSTONE	1
BB-CGB-102	R1	5.0 – 10.0	60	60	60	100%	METASANDSTONE	2



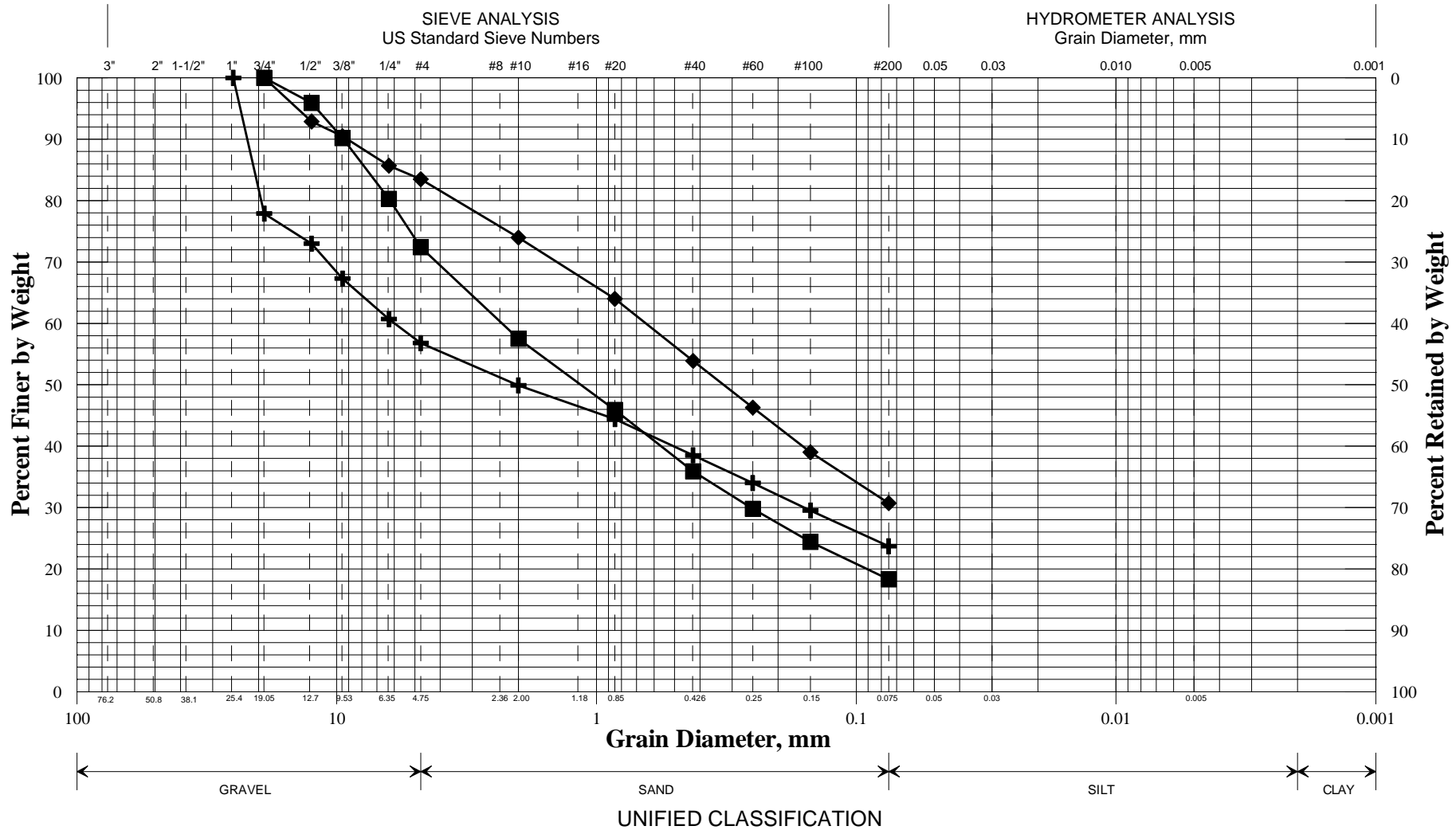
Notes: 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

Appendix C

Laboratory Test Results

Work Number: 22610.00

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-CGB-101/1D	156+44.7	10.3 RT	1.0-3.0	GRAVEL, some sand, some silt.	6.2			
◆	BB-CGB-101/2D	156+44.7	10.3 RT	5.0-7.0	SAND, some silt, little gravel.	13.0			
■	BB-CGB-102/1D	157+28.9	7.4 LT	1.0-3.0	SAND, some gravel, little silt.	3.3			
●									
▲									
×									

WIN
022610.00
Town
Camden
Reported by/Date
WHITE, TERRY A 7/15/2015

Appendix D

Calculations

Earth Pressure

Soil Parameters:

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

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

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

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

Frame pedestal walls and short headwalls fixed to frame - At-Rest Earth Pressure - Jaky

Reference: Das, Principles of Geotech Engr, 7th Edition, Eq. 13.5.

Formula for normally consolidated soils.

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

$$K_o = 0.47$$

Wingwalls 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 cantilever 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 $\beta := 27 \cdot \text{deg}$

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

$$K_{ar_slope} = 0.47$$

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

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.

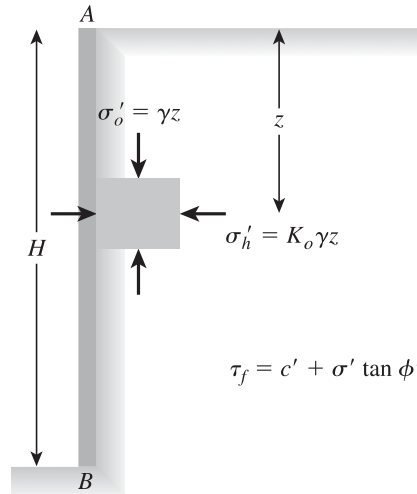


Figure 13.3
Earth pressure at rest

which shows a wall AB retaining a dry soil with a unit weight of γ . The wall is static. At a depth z ,

$$\text{Vertical effective stress} = \sigma'_o = \gamma z$$

$$\text{Horizontal effective stress} = \sigma'_h = K_o \gamma z$$

So,

$$K_o = \frac{\sigma'_h}{\sigma'_o} = \text{at-rest earth pressure coefficient}$$

For coarse-grained soils, the coefficient of earth pressure at rest can be estimated by using the empirical relationship (Jaky, 1944)

$$K_o = 1 - \sin \phi' \quad (13.5)$$

where $\phi' =$ drained friction angle.

While designing a wall that may be subjected to lateral earth pressure at rest, one must take care in evaluating the value of K_o . Sherif, Fang, and Sherif (1984), on the basis of their laboratory tests, showed that Jaky's equation for K_o [Eq. (13.5)] gives good results when the backfill is loose sand. However, for a dense, compacted sand backfill, Eq. (13.5) may grossly underestimate the lateral earth pressure at rest. This underestimation results because of the process of compaction of backfill. For this reason, they recommended the design relationship

$$K_o = (1 - \sin \phi) + \left[\frac{\gamma_d}{\gamma_{d(\min)}} - 1 \right] 5.5 \quad (13.6)$$

where $\gamma_d =$ actual compacted dry unit weight of the sand behind the wall
 $\gamma_{d(\min)} =$ dry unit weight of the sand in the loosest state (Chapter 3)

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:

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

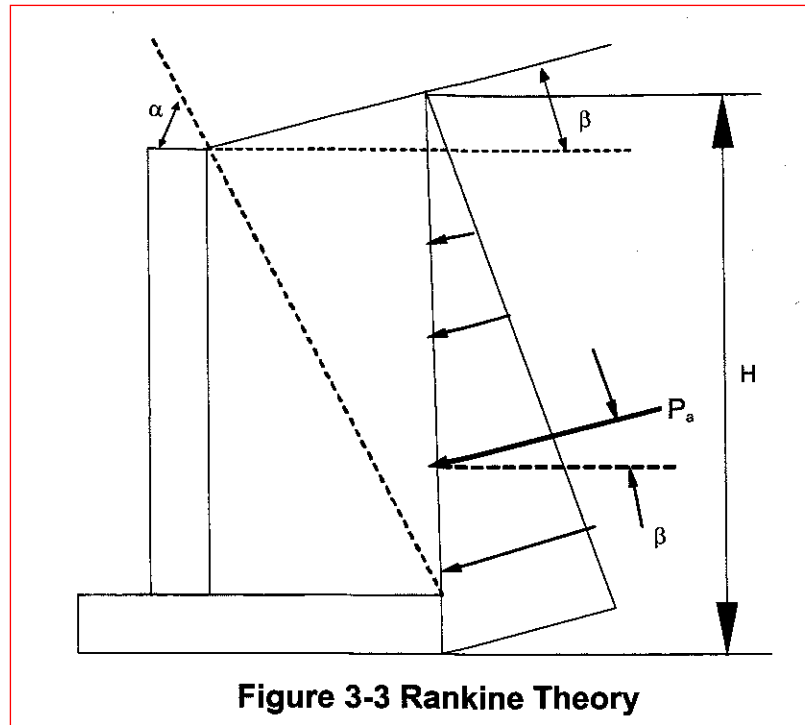


Figure 3-3 Rankine Theory

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.

Analysis

Calculation of nominal and factored bearing resistance of bedrock using Rock Mass Rating (RMR) based empirical correlation

Method

Use data from boring and calculate the nominal bearing resistance as follows:

1. Estimation of Rock Mass Rating
2. Determine rock property constants s and m
3. Calculate nominal bearing resistance of bedrock, q_n , using RMR method in Wylie "Foundations on Rock"/AASHTO (2012) LRFD 10.4.6.4 - Rock Mass Strength

References

1. AASHTO LRFD Bridge Design Specifications, 8th Ed, 2017, (C10.4.6.4 and 10.6.2.6.2)
2. AASHTO Standard Specifications for Highway Bridges, 17th Ed. 2002
3. Wylie, Duncan C, "Foundations on Rock", Second Edition, 2009.
4. "The Hoek-Brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

A. Design Bedrock Properties

Model site bedrock based on the country rock encountered in borings:

Medium grained, METASANDSTONE, very hard, fresh, massive.

RQD = 90-100%

Compressive Strength

Based on Table 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength as a Function of Rock Category and Rock Type, Ref. 2.

Rock Category C - sandstone.

$C_o = 9,700 - 25,000$ psi

Choose 9,700 psi

$$q_{uc1} := 9700 \text{ psi}$$

$$q_{uc1} = 1.397 \times 10^3 \cdot \text{ksf}$$

B. Frame/Arch and Wingwall Spread Footings

Determination of Rock Mass Rating (RMR) from LRFD (2012) Table 10.4.6.4-1 Geomechanics Classification of Rock Mass

Use RMR to supplement engineering judgment on rock competency according to LRFD 10.6.3.2.1. RMR is determined from the sum of five relative ratings listed in LRFD (2012) Table 10.4.6.4-1

1. Strength of intact rock

$$q_{u1} = 1397 \text{ ksf}$$

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength = 1080-2160 ksf **Relative Rating = 7**

2. Drill Core Quality

Minimum bedrock RQD = 90% From LRFD Table 10.4.6.4-1, RQD 75 to 90%; **Relative Rating = 17**

3. Spacing of joints

Assume broken or highly weathered rock is removed. Breaks of intact bedrock are 1 to 3 feet.

From LRFD Table 10.4.6.4-1; **Relative Rating = 20**

4. Condition of joints

Hard joint wall rock; **Relative Rating = 20**

5. Groundwater conditions

Water under moderate pressure; **Relative Rating = 4**

6. From LRFD Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations

Low angle to moderately dipping joints (5-55 degrees); **Relative Rating = -7**

ADJUSTED RMR

$$\text{RMR} := 7 + 17 + 20 + 20 + 4 - 7$$

$$\text{RMR} = 61$$

Determine Rock Type for LRFD Table 10.4.6.4-4

Rock Type - C

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.6.4-3, RMR = 61 is Class No. II and described as GOOD rock.

C. Rock Property Constants s and m (Ref. #1 and Ref. #4)

$$\text{RMR} = 61$$

Direct calculation of m and s is required, Reference 4 (Hoek and Brown, 1988), Equations 18 and 19 and Table 1. Assume isotropic behavior caused by the number and inconsistency of closely spaced discontinuity sets where none is significantly weaker than the other.

$$\text{For a disturbed rock mass: } m/m_i = \exp((\text{RMR}-100)/14)$$

$$s = \exp((\text{RMR}-100)/6)$$

$$m_i = m \text{ for intact rock}$$

For Rock Type C for intact rock, $m_i = 15$ (Ref. # 4, Table 1):

$$m_i := 15$$

$$m := m_i \cdot \exp\left(\frac{\text{RMR} - 100}{14}\right) \quad \text{Equation 18, Ref. 3}$$

$$m = 0.925$$

$$s := \exp\left(\frac{\text{RMR} - 100}{6}\right) \quad \text{Equation 19, Ref. 3}$$

$$s = 0.0015034$$

D. Nominal and Factored Bearing Resistance of Bedrock

Correction Factor for Foundation Shape, from Wyllie Table 5.4 Pg. 138 (Ref. #2)

$$C_{fl} := 1.0$$

Conservative selection of $C_{fl} = 1.0$ for $L/B > 6$

Nominal Bearing Resistance (Wyllie)

Reference #3: Wyllie "Foundations on Rock" Equation 5.4 Pg. 138

$$q_{n1} := C_{fl} \cdot \sqrt{s} \cdot q_{uc1} \cdot \left[1 + \sqrt{m \cdot \left(\frac{-1}{s} \right) + 1} \right]$$

$$q_{n1} = 324 \cdot \text{ksf}$$

Factored Bearing Resistances

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$$\phi_{bc} := 0.45$$

$$q_{rl} := q_{nl} \cdot \phi_{bc}$$

$$q_{rl} = 146 \cdot \text{ksf}$$

Strength Limit State

Factored Bearing Resistance

Use a bearing resistance factor of 0.80 LRFD 11.5.8 consistent with the design objective of no collapse.

$$\phi_{rec} := 0.8$$

$$q_{rl} := q_{nl} \cdot \phi_{rec}$$

$$q_{rl} = 259 \cdot \text{ksf}$$

Extreme Limit State

Verify Nominal Bearing Resistance per Carter and Kulhawy (1988)

Reference : NCHRP, Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, pg 40, Eq. 82b, and referred to in LRFD C.10.6.3.2.2. Same equation.

$$q_{nl} := q_{uc1} \cdot \left[\sqrt{s} + \sqrt{m \cdot (\sqrt{s}) + s} \right]$$

$$q_{nl} = 324 \cdot \text{ksf}$$

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 $0.3 f'_C$.

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Table 10.4.6.4-1—Geomechanics Classification of Rock Masses

Parameter			Ranges of Values							
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred			
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf	
	Relative Rating		15	12	7	4	2	1	0	
2	Drill core quality RQD		90% to 100%	75% to 90%		50% to 75%		25% to 50%		<25%
	Relative Rating		20	17		13		8		3
3	Spacing of joints		>10 ft	3–10 ft		1–3 ft		2 in.–1 ft		<2 in.
	Relative Rating		30	25		20		10		5
4	Condition of joints		<ul style="list-style-type: none">• Very rough surfaces• Not continuous• No separation• Hard joint wall rock	<ul style="list-style-type: none">• Slightly rough surfaces• Separation <0.05 in.• Hard joint wall rock		<ul style="list-style-type: none">• Slightly rough surfaces• Separation <0.05 in.• Soft joint wall rock		<ul style="list-style-type: none">• Slicken-sided surfaces or• Gouge <0.2 in. thick or• Joints open 0.05–0.2 in.• Continuous joints		<ul style="list-style-type: none">• Soft gouge >0.2 in. thick or• Joints open >0.2 in.• Continuous joints
	Relative Rating		25	20		12		6		0
5	Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal./hr.		400–2000 gal./hr.		>2000 gal./hr.		
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2		0.2–0.5		>0.5		
		General Conditions	Completely Dry	Moist only (interstitial water)		Water under moderate pressure		Severe water problems		
	Relative Rating		10	7		4		0		

Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

Table 10.4.6.4-3—Geomechanics Rock Mass Classes Determined from Total Ratings

RMR Rating	100–81	80–61	60–41	40–21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

The shear strength of fractured rock masses should be evaluated using the Hoek and Brown criteria, in which the shear strength is represented as a curved envelope that is a function of the uniaxial compressive strength of the intact rock, q_u , and two dimensionless constants m and s . The values of m and s as defined in Table 10.4.6.4-4 should be used.

The shear strength of the rock mass should be determined as:

$$\tau = (\cot \phi'_i - \cos \phi'_i) m \frac{q_u}{8} \quad (10.4.6.4-1)$$

in which:

$$\phi'_i = \tan^{-1} \left\{ 4h \cos^2 \left[30 + 0.33 \sin^{-1} \left(\frac{-3}{h^2} \right) \right] - 1 \right\}^{-\frac{1}{2}}$$

$$h = 1 + \frac{16(m\sigma'_n + sq_u)}{(3m^2q_u)}$$

where:

- τ = the shear strength of the rock mass (ksf)
- ϕ'_i = the instantaneous friction angle of the rock mass (degrees)
- q_u = average unconfined compressive strength of rock core (ksf)
- σ'_n = effective normal stress (ksf)
- m, s = constants from Table 10.4.6.4-4 (dim)

This method was developed by Hoek (1983) and Hoek and Brown (1988, 1997). Note that the instantaneous cohesion at a discrete value of normal stress can be taken as:

$$c_i = \tau - \sigma'_n \tan \phi'_i \quad (C10.4.6.4-1)$$

The instantaneous cohesion and instantaneous friction angle define a conventional linear Mohr envelope at the normal stress under consideration. For normal stresses significantly different than that used to compute the instantaneous values, the resulting shear strength will be unconservative. If there is considerable variation in the effective normal stress in the zone of concern, consideration should be given to subdividing the zone into areas where the normal stress is relative constant and assigning separate strength parameters to each zone. Alternatively, the methods of Hoek (1983) may be used to compute average values for the range of normal stresses expected.

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Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)

Rock Quality	Constants	Rock Type				
		A = Carbonate rocks with well developed crystal cleavage— <i>dolomite, limestone and marble</i> B = Lithified argillaceous rocks— <i>mudstone, siltstone, shale and slate (normal to cleavage)</i> C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage— <i>sandstone and quartzite</i> D = Fine grained polyminerallic igneous crystalline rocks— <i>andesite, dolerite, diabase and rhyolite</i> E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks— <i>amphibolite, gabbro gneiss, granite, norite, quartz-diorite</i>				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: <i>RM</i> = 100	<i>m</i> <i>s</i>	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft CSIR rating: <i>RM</i> = 85	<i>m</i> <i>s</i>	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft CSIR rating: <i>RM</i> = 65	<i>m</i> <i>s</i>	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft CSIR rating: <i>RM</i> = 44	<i>m</i> <i>s</i>	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: <i>RM</i> = 23	<i>m</i> <i>s</i>	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: <i>RM</i> = 3	<i>m</i> <i>s</i>	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	$C_o^{(1)}$	
			(ksf)	(psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale ⁽²⁾	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

⁽¹⁾Range of Uniaxial Compressive Strength values reported by various investigations.⁽²⁾Not including oil shale.

$$\rho = q_o (1 - \nu^2) B I_p / E_m, \text{ with } I_p = (L/B)^{1/2} / \beta_z \quad (4.4.8.2.2-2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (ν) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \quad (4.4.8.2.2-3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \quad (4.4.8.2.2-4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m .

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

order to permit construction of the models. Consequently, our ability to predict the strength of jointed rock masses on the basis of direct tests or of model studies is severely limited.

In searching for a solution to this problem in order to provide a basis for the design of underground excavations in rock, Hoek and Brown (1980a) felt that some attempt had to be made to link the constants m and s of their criterion to measurements or observations which could be carried out by any competent geologist in the field. Recognizing that the characteristics of the rock mass which control its strength and deformation behaviour are similar to the characteristics which had been adopted by Bieniawski (1974) and by Barton, Lien and Lunde (1974) for their rock mass classifications, Hoek and Brown (1980a) proposed that these rock mass classifications could be used for estimating the material constants m and s .

Because of the lack of suitable methods for estimating the strength of rock masses, the first table relating rock mass classifications to material properties published by Hoek and Brown (1980a) was widely accepted by the geotechnical community and has been used on a large number of projects. Experience gained from these applications showed that the estimated rock mass strengths were reasonable when used for slope stability studies in which the rock mass is usually disturbed and loosened by relaxation due to excavation of the slope. However, the estimated rock mass strengths generally appeared to be too low in applications involving underground excavations where the confining stresses do not permit the same degree of loosening as would occur in a slope.

In order to incorporate the lessons learned from practical applications, Brown and Hoek (1988) proposed a revised set of relationships between the rock mass rating (RMR) from Bieniawski's (1974) rock mass classification and the constants m and s . Following Priest and Brown (1983), the relationships were presented in the form of the following equations:

Disturbed rock masses :

$$\frac{m}{m_i} = \exp \left(\frac{\text{RMR} - 100}{14} \right) \quad (18)$$

$$s = \exp \left(\frac{\text{RMR} - 100}{6} \right) \quad (19)$$

Undisturbed or interlocking rock masses:

$$\frac{m}{m_i} = \exp \left(\frac{\text{RMR} - 100}{28} \right) \quad (20)$$

$$s = \exp \left(\frac{\text{RMR} - 100}{9} \right) \quad (21)$$

where

m and s are the rock mass constants and m_i is the value of m for the *intact* rock.

Equations 18 to 21 have been used to construct Table 1 which shows the approximate relationship between rock mass quality and the Hoek-Brown material constants. Note that the value of the Tunnelling Quality Index Q from the NGI rock mass classification by Barton, Lien and Lunde (1974) has been calculated from the relationship proposed by Bieniawski (1976) :

$$\text{RMR} = 9 \log_e Q + 44 \quad (22)$$

Limitations on using failure criterion

Figure 1 illustrates a jointed rock mass in to which a tunnel has been mined. The circles adjacent to the right hand wall of the tunnel enclose different rock mass volumes and the comments on the right hand side of the drawing indicate situations to which the Hoek-Brown failure criterion can be applied.

When the volume of rock under consideration is small enough that it does not contain any structural discontinuities, equation 1 can be applied, using the m and s values for *intact* rock. This condition would apply to small scale specimens which has been extracted for laboratory testing or to the analysis of concentrated forces such as those which may be exerted by an individual pick on a tunnel boring machine cutter.

When the volume of rock being considered is such that only a few structural discontinuities are contained in this volume, the Hoek-Brown criterion should not be used. The behaviour of this rock is likely to be highly anisotropic and the Hoek-Brown failure criterion, which is only applicable to isotropic rock, will give erroneous results.

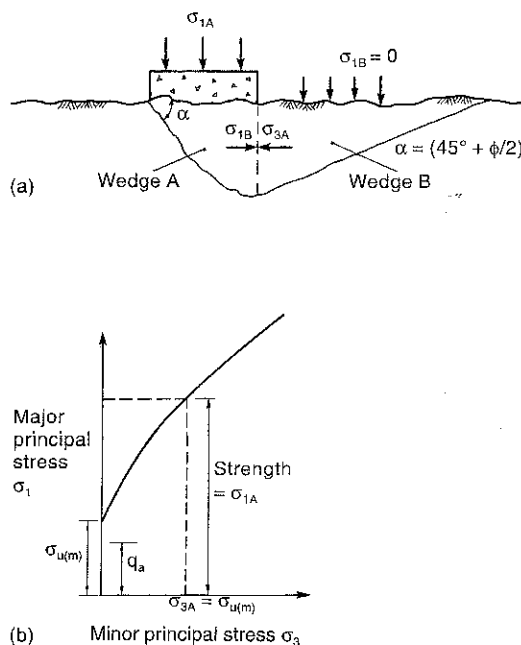


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a , strength of bearing rock = σ_{1A} , factor of safety $FS = \sigma_{1A}/q_a$.

$$\sigma_1 = (m\sigma_{u(r)}(s\sigma_{u(r)}^2)^{1/2} + s\sigma_{u(r)}^2)^{1/2} + (s\sigma_{u(r)}^2)^{1/2} \\ = s^{1/2}\sigma_{u(r)}[1 + (ms^{-1/2} + 1)^{1/2}] \quad (5.3)$$

The plot in Fig. 5.3(b) shows the relationship between the strength σ_{1A} and the confining stresses provided by the surrounding rock σ_{3A} . This illustrates that a very significant increase in the bearing capacity is produced by a small increase in the confining pressure.

The allowable bearing pressure q_a is related to the rock mass strength by the factor of safety FS and the correction factor C_{f1} :

$$q_a = \frac{C_{f1}s^{1/2}\sigma_{u(r)}[1 + (ms^{-1/2} + 1)^{1/2}]}{FS} \quad (5.4)$$

The factor C_{f1} is applied to the calculated allowable bearing pressure to account for the shape of

the foundation and has the values given in Table 5.4 (Sowers, 1970).

A more comprehensive procedure for calculating the ultimate bearing capacity of fractured rock is described by Serrano and Olalla (1994) in which the rock mass strength is defined by the Hoek and Brown strength criteria as above. The method of analysis can accommodate recessed footings, inclined loads and foundations located on sloping ground surfaces.

For most loading conditions on sound rock the factor of safety will be in the range 2–3 for which there is little risk of settlement. A factor of safety of 3 is used for the dead load plus the maximum live load. If part of the live load is temporary such as wind and earthquake, then a factor of safety of 2 can be used (US Department of the Navy, 1982).

In the equations to calculate the allowable bearing capacity for a fractured rock mass with the strength defined by curved strength envelopes, it is important to distinguish between the compressive strength of the intact rock and that of the rock mass. The intact rock strength $\sigma_{u(r)}$ is determined from laboratory tests on rock cores, while for fractured rock the strength is defined by equation 5.1 with the degree of fracturing of the rock mass being accounted for by the constants m and s .

5.2.3 Recessed footings

In the case of a footing which is recessed into the rock surface, it is necessary to modify equation 5.4 to account for the increase in the stress σ_{1s} as a result of the confining stress q_s applied at the ground surface. That is, the minor principal stress

Table 5.4 Correction factors for foundation shapes (L = length, B = width)

Foundation shape	C_{f1}	C_{f2}
Strip ($L/B > 6$)	1.0	1.0
Rectangular		
$L/B = 2$	1.12	0.9
$L/B = 5$	1.05	0.95
Square	1.25	0.85
Circular	1.2	0.7

Objective:

Estimate the factored bearing resistance for precast concrete modular gravity wall bearing on soil at the Service Limit State, Strength Limit State, and Extreme Limit State.

Given:

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

Assumptions:

1. The proposed bearing elevation is approximately Elev.144 (inlet) and 140 (outlet)
2. Proposed finish roadway grade elevation is approximately 2 feet higher than current.
3. Proposed precast concrete modular wall stem lengths can range from 8 to 12 ft
4. PCMG units will be installed on a CIP leveling pad cast on a concrete fill subfooting on bedrock - behind the leveling pad, Unit Stems can be installed on compacted gravel borrow or 3/4" crushed stone.
5. The bottom of the wall will be submerged for the structure's design life.

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

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

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Coarse to medium sand, with little gravel (SW, SP)	Medium dense to dense	4-8	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:

Assumed Foundation Width, Depth, and Water Surface

$$B' := \begin{pmatrix} 8 \\ 10 \\ 12 \end{pmatrix} \text{ ft} \quad B' = \text{Stem Length}$$

$$D_f := 0.0 \cdot \text{ft}$$

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

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

Total unit weight of the soil above the toe of the lower PCMG unit

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

Foundation soils:

$$\gamma_{1m} := 135 \cdot \text{pcf}$$

Compacted 3/4-inch crushed stone

$$\phi := 38 \cdot \text{deg}$$

Bowles Table 2-6

$$c := 0$$

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

Shape Factors for square footing (Bowles 5th Ed., pg 220)

$$s_{\gamma} := 0.8$$

$$s_c := 1.3$$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for $\phi = 38$ degrees.

$$N_c := 61.31$$

$$N_q := 48.9$$

$$N_{\gamma} := 64$$

Nominal Bearing Resistance per Terzaghi equation

$$q := D_f (\gamma_{\text{above}} - \gamma_w) \quad q = 0 \cdot \text{ksf}$$

Das Principles of Foundation Engineering 7th Ed. p. 142:
Eq. 3.16 Water table modification

$$q_n := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma_{1m} - \gamma_w) \cdot B' \cdot N_{\gamma} \cdot s_{\gamma}$$

Bowles Foundation Analysis and Design 5th Ed. p. 220:
Table 4-1 Bearing-capacity Equations

$$q_n = \begin{pmatrix} 14.9 \\ 18.6 \\ 22.3 \end{pmatrix} \cdot \text{ksf}$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\phi_b := 0.45$$

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

$$q_r = \begin{pmatrix} 6.7 \\ 8.4 \\ 10 \end{pmatrix} \cdot \text{ksf}$$

for

$$B' = \begin{pmatrix} 8 \\ 10 \\ 12 \end{pmatrix} \cdot \text{ft}$$

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)

$$N_c := 61.4$$

$$N_q := 48.8$$

$$N_\gamma := 78$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$$L := \begin{pmatrix} 8 \\ 10 \\ 12 \end{pmatrix} \cdot \text{ft} \quad B := 5 \cdot \text{ft}$$

PCMG unit lengths and width (5 feet)

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

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

$$s_\gamma = \begin{pmatrix} 0.75 \\ 0.8 \\ 0.83 \end{pmatrix} \quad s_q = \begin{pmatrix} 1.49 \\ 1.39 \\ 1.33 \end{pmatrix}$$

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 toe of the lower PCMG wall unit

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

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

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

$$\frac{D_f}{B} = 0$$

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 79.82$$

$$N_{\gamma m} = \begin{pmatrix} 58.5 \\ 62.4 \\ 65 \end{pmatrix}$$

$$N_{qm} = \begin{pmatrix} 72.629 \\ 67.863 \\ 64.686 \end{pmatrix}$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = \begin{pmatrix} 15.8 \\ 21.1 \\ 26.3 \end{pmatrix} \cdot \text{ksf}$$

Factored Bearing Resistance for Strength Limit States

$$\phi_b := 0.45$$

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

$$q_r = \begin{pmatrix} 7.1 \\ 9.5 \\ 11.8 \end{pmatrix} \cdot \text{ksf}$$

Recommend 7 ksf for the factored bearing resistance for PCMG wall base units 8-feet long, or longer at the Strength Limit State.

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 $0.3 f'_c$.

TABLE 2-6
Representative values for angle of internal friction ϕ

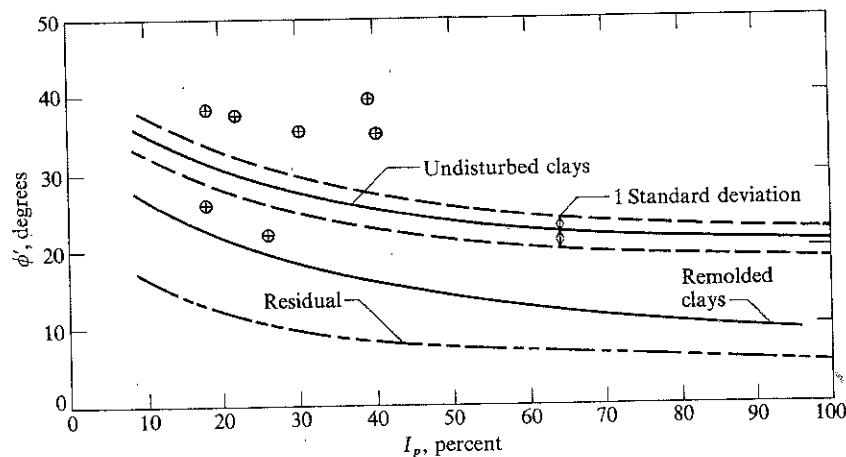
Soil	Type of test*		
	Unconsolidated- undrained, U	Consolidated- undrained, CU	Consolidated- drained, CD
Gravel			
Medium size	40–55°		40–55°
Sandy	35–50°		35–50°
Sand			
Loose dry	28–34°		
Loose saturated	28–34°		
Dense dry	35–46°		43–50°
Dense saturated	1–2° less than dense dry		43–50°
Silt or silty sand			
Loose	20–22°		27–30°
Dense	25–30°		30–35°
Clay	0° if saturated	3–20°	20–42°

*See a laboratory manual on soil testing for a complete description of these tests, e.g., Bowles (1992).

Notes:

1. Use larger values as γ increases.
2. Use larger values for more angular particles.
3. Use larger values for well-graded sand and gravel mixtures (GW, SW).
4. Average values for gravels, 35–38°; sands, 32–34°.

Figure 2-35 Correlation between ϕ' and plasticity index I_p for normally consolidated (including marine) clays. Approximately 80 percent of data falls within one standard deviation. Only a few extreme scatter values are shown [Data from several sources: Ladd et al. (1977), Bjerrum and Simons (1960), Kanja and Wolle (1977), Olsen et al. (1986).]



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 /Crushed Stone	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.

The Terzaghi Bearing-Capacity Equation

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

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

Terzaghi (1943). See Table 4-2 for typical values and for K_{py} values.

$$q_{ult} = cN_c s_c + \bar{q}N_q + 0.5\gamma B N_\gamma s_\gamma$$

$$N_q = \frac{a^2}{a \cos^2(45 + \phi/2)}$$

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

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

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

For: strip round	square
$s_c = 1.0$	1.3
$s_\gamma = 1.0$	0.6

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

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

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

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

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

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

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

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

$$\text{when } \phi = 0$$

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

$$N_q = \text{same as Meyerhof above}$$

$$N_c = \text{same as Meyerhof above}$$

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

Vesic (1973, 1975).* See Table 4-5 for shape, depth, and other factors.

Use Hansen's equations above.

$$N_q = \text{same as Meyerhof above}$$

$$N_c = \text{same as Meyerhof above}$$

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

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

†See Sec. 4-6 when $i_i < 1$.

N_γ values shows the following:

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

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

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

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

Meyerhof's Bearing-Capacity Equation

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

TABLE 4-4

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

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

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

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

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

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.

N_f = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

γ = total (moist) unit weight of soil above or below the bearing depth of the footing (kcf)

D_f = footing embedment depth (ft)

B = footing width (ft)

$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

s_c, s_γ, s_q = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)

d_q = correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4 (dim)

i_c, i_γ, i_q = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

For $\phi_f = 0$:

$$i_c = 1 - (nH/cBLN_c) \quad (10.6.3.1.2a-5)$$

For $\phi_f > 0$:

$$i_c = i_q - [(1 - i_q)/(N_q - 1)] \quad (10.6.3.1.2a-6)$$

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL \cot \phi_f)} \right]^n \quad (10.6.3.1.2a-7)$$

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

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

Most geotechnical engineers nationwide have not used the load inclination factors. This is due, in part, to the lack of knowledge of the vertical and horizontal loads at the time of geotechnical explorations and preparation of bearing resistance recommendations.

Furthermore, the basis of the load inclination factors computed by Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8 is a combination of bearing resistance theory and small scale load tests on 1.0 in. wide plates on London Clay and Ham River Sand (Meyerhof, 1953). Therefore, the factors do not take into consideration the effects of depth of embedment. Meyerhof further showed that for footings with a depth of embedment ratio of $D_f/B = 1$, the effects of load inclination on bearing resistance are relatively small. The theoretical formulation of load inclination factors were further examined by Brinch-Hansen (1970), with additional modification by Vesic (1973) into the form provided in Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8.

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

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.

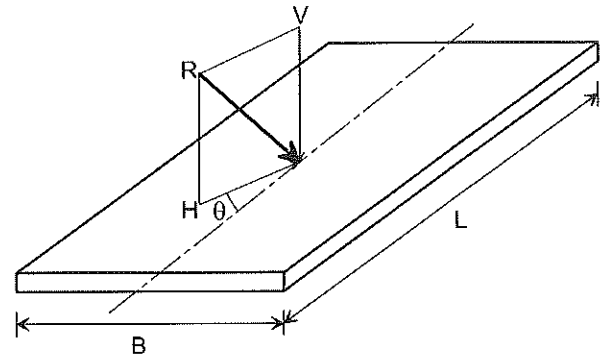


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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_{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.

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

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

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:

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

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

where:

c^* = reduced effective stress soil cohesion for punching shear (ksf)

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

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.

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

From Design Freezing Index Map: **Belfast, Maine**
Case 1 - coarse grained granular fill soils W=10%

$$DFI_1 := 1200 \quad d_1 := 73.1 \cdot \text{in}$$

$$DFI_2 := 1300 \quad d_2 := 76.3 \cdot \text{in}$$

Approximate DFI at project = 1275 find frost depth by interpolation:

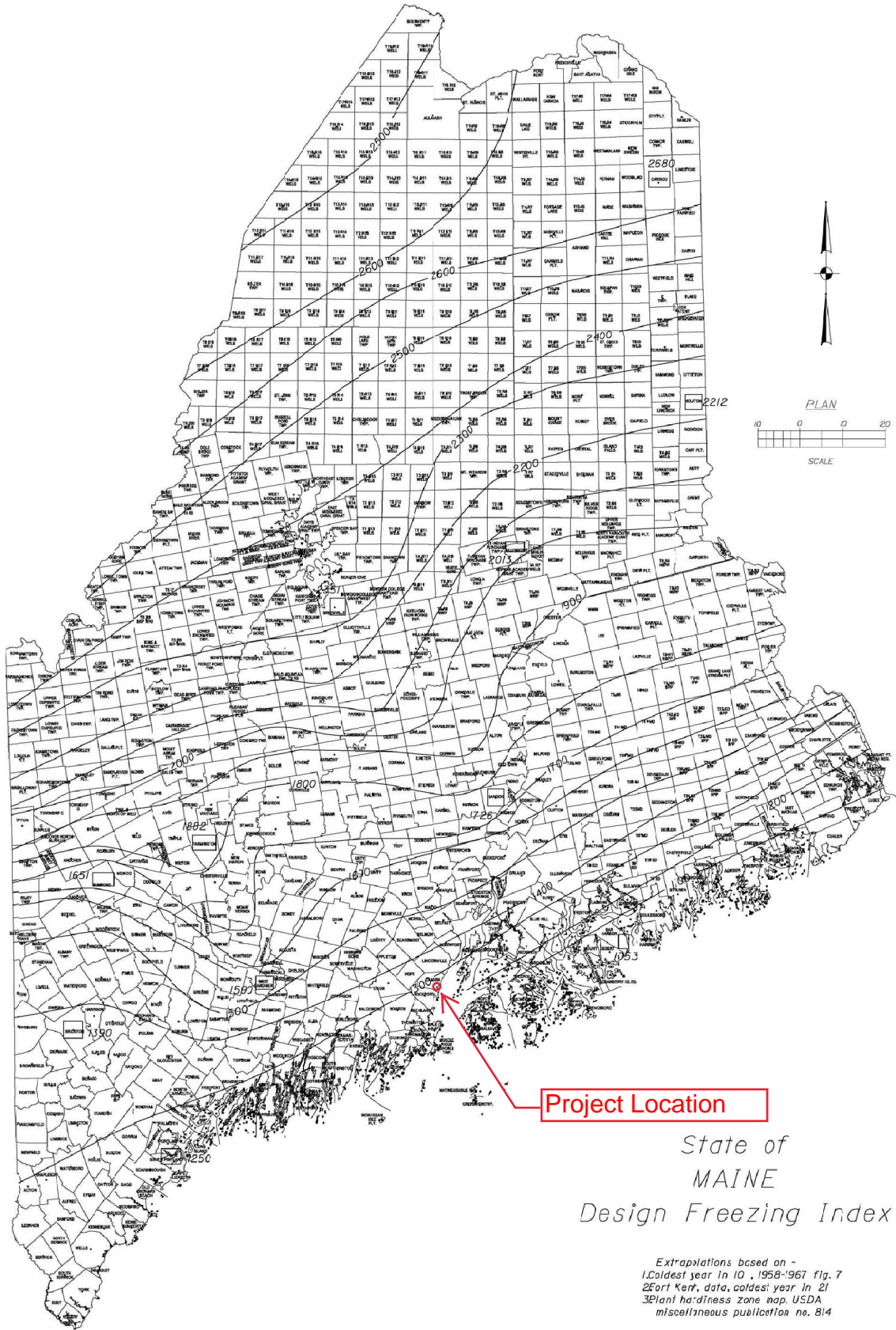
$$DFI_3 := 1275$$

$$d_3 := d_1 + \frac{(DFI_3 - DFI_1) \cdot (d_2 - d_1)}{(DFI_2 - DFI_1)} \quad d_3 = 75.5 \cdot \text{in}$$

$$\text{Depth of Frost Penetration} \quad d_3 = 6.3 \cdot \text{ft}$$

Recommendation: 6.3 feet for design of foundations constructed on coarse grained soils

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

Interpolate
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