

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

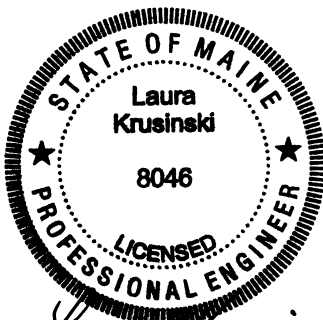
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**PEARCE BROOK NO. 6 BRIDGE
GREEN STREET OVER PEARCE BROOK
HOULTON, MAINE**

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and geotechnical design recommendations for the replacement of Pearce Brook No. 6 bridge, which carries Green Street over Pearce Brook in Houlton, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation recommendations, and geotechnical design parameters for design of the new bridge substructures.

The existing Pearce Brook No. 6 Bridge was constructed in 1919. The structure is comprised of a 10-foot span rigid concrete frame, mass concrete abutments, and mass concrete return wingwalls. The superstructure carries 22 feet of pavement between the bridge rails that are 27 feet apart. Ice jams are known to occur at the existing structure and are likely the result of the small and poorly aligned hydraulic opening.

The 2018 Maine Department of Transportation (MaineDOT) Bridge Maintenance Inspection Report rates the superstructure and deck a 4 for “poor condition” and the substructure a 3 for “serious condition”. All four retaining walls show heavy scaling and cracking. The existing bridge has a Federal Highway Administrations Sufficiency Rating of 36.6 and is considered Structurally Deficient.

The proposed replacement structure consists of a 22-foot span, 6-foot rise, 3-sided precast concrete frame, on cast-in-place stem walls, founded on cast-in-place spread footings on bedrock. Cast-in-place concrete wingwalls with spread footings founded on bedrock will be specified in contract documents along with an option to substitute precast concrete modular gravity (PCMG) walls placed on leveling concrete on bedrock.

The bridge replacement project with 275 feet of approach work will maintain nearly the same horizontal and vertical alignment as the existing Pearce Brook No. 6 Bridge. Green Street will be closed for the construction of the new bridge and traffic detoured on existing roads offsite.

2.0 GEOLOGIC SETTING

Pearce Brook No. 6 Bridge carries Green Street over Pearce Brook in Houlton, Maine approximately as shown on Sheet 1 – Location Map.

The Maine Geologic Survey (MGS) Surficial Geology of the Houlton Quadrangle, Maine, Open-file No. 81-9 (1981) indicates the surficial soils in the vicinity of the bridge project consist of glacial till. Glacial till is deposited by ice without sorting by water resulting in a generally unstratified and unsorted heterogenous mixture of clay, silt, sand, cobbles, and boulders. Glacial till commonly blankets and conforms to the bedrock surface.

The MGS Bedrock Geologic Map of Maine, (1985), indicates the bedrock in the vicinity of the bridge project as interbedded pelite and limestone or dolostone of the Carys Mills Formation.

3.0 SUBSURFACE INVESTIGATION

Two test borings explored subsurface conditions at the site. Boring BB-HPB-101 was located west of the existing structure behind the existing Abutment No. 1. Boring BB-HPB-102 was located east of the existing structure behind the existing Abutment No. 2. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The MaineDOT Drill Crew drilled the borings on May 11, 2017. Details and sampling methods used, field data obtained, and soil conditions encountered are presented on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

The borings were drilled using solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in April 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.854 to the raw field N-values. This hammer efficiency factor (0.854) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring log.

Bedrock was cored in both test borings using an NQ-2” bore barrel and the Rock Quality Designation (RQD) of the bedrock core was calculated. A subsurface inspector, certified by the Northeast Transportation Technician Certification Program (NETTCP), logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed boring logs, and identified field testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the site. Soil laboratory testing consisted of one standard grain size analysis with natural water content test, two grain size analyses with hydrometer and natural moisture content, and one Atterberg limits test. The results of laboratory soil tests are included in Appendix B – Laboratory Test Results. Moisture content information and other soil test results are shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings consisted of fill soils underlain by metamorphic 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 Soils

The fill unit encountered was approximately 11.4 to 13.6 feet thick at the test boring locations. The fill material generally consisted of:

- Brown, wet, silty sand, little gravel, wood;
- Olive-brown, wet, silt, some clay, trace sand;
- Brown, wet, gravel, some sand, little silt; and
- Grey, wet, sand, some gravel, some silt, trace clay.

Corrected SPT N-values in the fill soils ranged from 20 to 41 blows per foot (bpf), indicating the granular fill soils are medium dense to dense and the clayey silt soils are very stiff in consistency. Grain size analyses conducted on samples of the fill unit indicate the soil is classified as A-4, A-1-a, and A-2-4 by the AASHTO Soil Classification System and SC-SM, GM, and ML by the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 11 to 24 percent. One Atterberg limits test conducted on the silt-clay fill subunit indicates the material is preconsolidated.

5.2 Bedrock

Bedrock was encountered and cored in BB-HPB-101 and BB-HPB-102. Table 1 summarizes the approximate top of bedrock elevation at the boring locations and RQD's of the initial bedrock core runs.

Boring No.	Station	Offset (feet)	Approximate Depth to Bedrock, bgs (feet)	Approximate Top of Bedrock Elevation (feet)	RQD (%)	
BB-HPB-101	2+80.4	8.5 Lt.	12.7	335.8	R1	69
					R2	89
BB-HPB-102	3+19.6	7.4 Rt.	14.1	335.1	R1	45
					R2	48

Table 1 – Summary of Approximate Depth to Bedrock, Top of Bedrock Elevation, and RQD of initial core run

The bedrock is identified as grey, fine-grained, hard, fresh, ribbon limestone. Discontinuities consist of steep to vertical slaty cleavages that are moderately very close to wide, tight to open, rough, undulating, with occasional silt infilling, tight to healed, and at horizontal to low angle joints. The RQD of the bedrock cored ranged from 43 to 89 percent correlating to a Rock Mass Quality of poor to good.

5.3 Groundwater

The measured depths to groundwater in the test borings ranged from approximately 7.6 to 9.0 feet bgs. The measurements were recorded after completion of the test boring. Note that water was introduced into the borehole during drilling operations and the measurement may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

A precast concrete box culvert, a concrete voided slab on full height abutments with spread footings on bedrock, and a 3-sided precast concrete rigid frame with spread footings on bedrock, were identified as feasible alternatives during preliminary design. The necessary rise of a precast concrete box culvert, combined with the relatively shallow bedrock surface at the site, eliminated the box culvert from further consideration. The precast rigid frame alternative was determined to be more durable, would be faster to construct, and had a lower cost when compared to the voided concrete slab alternative. Therefore, a three-sided precast rigid frame is the recommended bridge alternative.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The proposed replacement structure will consist of a 22-foot span, 6-foot rise, rigid precast concrete frame, installed on cast-in-place stem walls on spread footings on bedrock. Cast-in-place, cantilever-type wingwalls with spread footings founded on bedrock will be specified in contract documents. The final contract documents may allow the option to substitute precast concrete modular gravity (PCMG) walls on leveling slabs/leveling concrete on bedrock. Engineered streambed material will restore the streambed profile. Riprap buried within the streambed will protect the corners of the frame footing and wingwall footings.

7.1 Precast Concrete Frame Design and Construction

Precast concrete frames are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The frame shall be designed in accordance with MaineDOT Standard Specification 534 – Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures, and American Association of State Highway and Transportation Officials (AASHTO) Load Resistance and Factor Design Bridge Design Specifications, 8th Edition, 2017 with up to 2018 interims (LRFD).

The loading specified for the structure should be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The precast concrete frame shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The design should use Soil Type 4 as presented in the MaineDOT BDG Section 3.6 to design earth loads from the soil envelope. The backfill properties are as follows: $\phi=32^\circ$, $\gamma = 125$ pcf. For vertical earth pressure, the maximum load factor for at rest earth pressure from LRFD Table 3.4.1-2 shall apply and wheel loads should be distributed through earth fills according to the provisions of LRFD 4.6.2.10 for flat-top frames with depths of fill less than 2.0 feet.

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

Earth pressure coefficients for the design of three-sided frames are discussed in Section 7.6 of this report.

7.2 Concrete Headwalls

Concrete headwalls may be included in the buried structure design to retain cover placed over the frame and prevent gravel from dropping or eroding into the waterway. Headwalls serving as parapets or located on the traffic side of bridge railing shall be considered an integral part of the railing and designed according to AASHTO LRFD Chapter 13.

7.3 Bedrock Removal and Bedrock Subgrade Preparation

The depth to bedrock in the borings drilled at the site ranged from approximately 12.7 to 14.1 feet below the roadway surface (brs). Spread footings can practically and economically be constructed to bear on bedrock at this location, possibly without temporary soil support systems. The borings indicate that bedrock with an RQD ranging from approximately 45 to 69 percent will be encountered at the bedrock surface at the footing locations.

The nature, slope, and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations for the frame stem wall footings and wingwall footings are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock, and soil. The final bearing surface shall be solid. Construction activities should not be permitted to disturb the bedrock mass or to create any rock falls or any open fissures. The cleanliness and condition of the final bedrock surface shall be approved by the Resident prior to placement of the footing concrete, fill concrete, or seal concrete.

If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level. The bedrock surface may be stepped along the centerline of bearing to create a workable bearing surface. The bottom of footing elevation may vary based on the presence of fractured or weathered bedrock and the variability of the bedrock surface.

The contractor should maintain the frame and wingwall excavations so that the foundations can be constructed in-the-dry. Where foundations are constructed in-the-dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing. For spread footings constructed in-the-dry, any irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with unreinforced concrete to the bearing elevation. In-the-dry excavation of highly sloped and loose fractured bedrock material may be done using conventional excavation methods.

7.4 Bearing Resistance of Cast-in-Place 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 11.5.6. The bearing vertical 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 rock.

The bearing resistance of cast-in-place spread footings constructed on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 26 ksf. This factored bearing resistance assumes a resistance factor, ϕ_b , for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. 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 20 ksf. Resistance factors for the service limit state are taken as 1.0. A factored bearing resistance of 20 ksf shall also be used to control settlement when analyzing the footing for service limit state load and for preliminary footing sizing as allowed in LRFD C10.6.2.1. See Appendix C – Calculations for supporting documentation.

For extreme limit state load combinations acting on project retaining walls, a factored bearing resistance of 47 ksf is recommended.

In no instance shall the bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'_c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

7.5 Spread Footing Design for Frames and Cast-in-Place Wingwalls

Spread footings for frames and cast-in-place 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. Spread footings for frames and cast-in-place wingwalls shall be designed to resist all lateral earth loads, vehicular loads, deadloads and lateral thrust forces

transferred through the frame, and temperature and shrinkage force effects. The design of spread footings at the strength limit state shall consider:

- bearing resistance,
- eccentricity,
- lateral sliding, and
- reinforced-concrete structural design.

Assuming that the bedrock 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 friction coefficient of 0.60 at the bedrock-to-concrete interface. If the bedrock 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 frictional coefficient of 0.70 at level bedrock-to-concrete interfaces.

Failure by sliding shall be investigated. A sliding resistance factor, ϕ_r , of 0.80 shall be applied to the nominal sliding resistance of cast-in-place concrete footings on bedrock.

The resistance due to passive pressure in front of walls shall be neglected due to consideration of scour, frost penetration, or other types of potential disturbance.

Anchorage of the footings or subfooting to bedrock may be required to resist sliding forces and improve stability. Dowels should be #9 reinforcing bars or larger and embedded into bedrock and into the footings by a depth determined by the designer. If bedrock is observed to slope steeper than 4H:1V at the footing subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

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. Shear failure along adversely oriented joint surfaces in the rock mass below the foundation is not anticipated; therefore, a global stability evaluation may be waived.

Extreme limit state design checks for wingwalls shall include bearing resistance, eccentricity, failure by sliding, and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice (if warranted by ice history or stream constriction by the abutments) or vehicle impact forces on retaining wall guardrail. Resistance factors, ϕ , for the

extreme limit state shall be taken as 1.0 with the exception of bearing resistance for which a resistance factor of 0.8 shall be used.

For scour protection of frame and wingwall spread footings, construct the spread footings or concrete seals directly on bedrock surfaces cleaned of all weathered, loose, and potentially erodible, or scourable rock. With these precautions, strength and extreme limit state designs do not need to consider rock scour due to the design or check floods for scour.

7.6 Earth Pressures and Traffic Surcharge Forces

Calculation of earth pressures acting on frame stem walls should assume an at-rest earth pressure coefficient, K_o , of 0.47, assuming they are prevented from movement. Calculation of earth pressures mobilized to resist outward thrust forces transferred from the frame to its footings shall also assume an at-rest earth pressure coefficient, K_o , of 0.47.

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

Independent precast or cast-in-place cantilever-type wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a , of 0.31 assuming a level backfill. For a 2H:1V backslope, the designer shall use a Rankine active earth pressure coefficient, K_a , of 0.46. The active earth pressure coefficient will change if the backslope conditions are different. See Appendix C – Calculations for supporting documentation.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. Live load surcharge on frame stem walls and their footings may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-1. The live load surcharge on wingwalls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 2 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 2 – Equivalent Height of Soil for Estimating Live Load Surcharge on Walls

Frame stem walls and 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.

Backfill within 10 feet of frames and wingwalls shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 703.19. This gradation specifies 7 percent or less of the material passing the No. 200 sieve. Limiting the amount of fines is intended to minimize frost action and eliminate the need to design for hydrostatic forces by promoting drainage behind the structure.

7.7 Precast Concrete Modular Gravity Walls

Contract documents may permit Precast Concrete Modular Gravity (PCMG) walls as an alternative to cast-in-place wingwalls. The assumed bearing elevation is below the $Q_{1.1}$ elevation; therefore, the wall base units shall be constructed on bedrock and no less than 2 feet below the maximum anticipated scour depth. 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 subcontracted by the Contractor as a design-build item.

Design recommendations for PCMG walls and their foundations are as follows:

- PCMG walls shall be designed to meet the design requirements specified in MaineDOT Standard Specification Subsection 674.03.
- The base PCMG units shall be founded on a minimum 6 by 12-inch leveling slab cast on bedrock, or a concrete subfooting cast directly on bedrock. Because the bedrock surface will be variable, and the height of the wall units are standardized, unreinforced concrete fill of variable thickness may be placed to simplify the design and construction of the wall.
- The bearing resistance for PCMG walls founded on a minimum 6-inch by 12-inch leveling slab or concrete subfooting, cast directly on bedrock, shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 10 ksf. A factored bearing resistance of 6 ksf shall be used to control settlement when analyzing the service limit state. A factored bearing resistance of 25 ksf shall be used when analyzing the extreme limit state.
- The factored vertical stress shall be calculated assuming a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1. The application of load factors for bearing stress is illustrated in LRFD Figure C11.5.6-1.
- 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}) of 2 feet, per LRFD Article 3.11.6.4.

- The ratio of factored resisting moments to overturning moments due to factored strength limit state loads, shall be greater than 1.0. For the lowest PCMG unit, with a stem on backfill soil behind the panel facing, 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 base width. The application of load factors for overturning and eccentricity are illustrated in LRFD Figure C11.5.6-2.
- Failure by sliding shall be investigated. A sliding resistance factor, ϕ_τ , of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on soil. A sliding resistance factor, ϕ_τ , 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 $0.46 = 0.80 \times \tan 30^\circ$ 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. The application of load factors for sliding resistance evaluations are illustrated in LRFD Figure C11.5.6-2.
- The resistance due to passive pressure in front of walls shall be neglected due to consideration of scour, frost penetration, or other types of potential disturbance.
- The wall shall be designed for hydrostatic forces resulting from a high-water elevation of El. 345 ($Q_{1.1}$). The buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

7.8 Settlement

Embankment settlement is a function of the grade raise and the overall thickness and compressibility of the soil deposits. The granular fill materials encountered in the east bridge approach are generally medium dense in consistency. The clay-silt fill material encountered in the west bridge approach is stiff in consistency and is judged to be moderately overconsolidated. The proposed vertical alignment is nearly the same as the existing vertical alignment. Therefore, we anticipated that approach roadway settlement will be negligible.

The structure and its foundations will be constructed on bedrock. Any settlement of foundations placed on properly prepared bedrock will be due to elastic compression of the bedrock and will be negligible.

7.9 Frost Protection

Foundations constructed on bedrock require no minimum depth of embedment for frost protection. Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Houlton has a design freezing index (DFI) of approximately 2200 F-

degree days. A water content of 10% was used for coarse-grained soils. These components correlate to a frost depth of 8.3 feet. Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 8.3 feet for frost protection. See Appendix C – Calculations for supporting calculations.

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

7.10 Scour and Riprap

For scour protection of the footings, construct the concrete footings or seal concrete subfooting directly on bedrock surfaces cleaned of soil and all weathered, loose, highly fractured, and potentially erodible rock. The remaining intact bedrock subgrade shall be competent.

Sideslopes and footings supporting the structure should be armored with riprap conforming to MaineDOT Standard Specification Section 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater backfill. The toe of the riprap sections shall be constructed 1 foot below the streambed elevation unless the streambed consists of bedrock. When riprap intersects bedrock, the largest riprap should be placed at the toe to buttress the slope above. The riprap slopes shall be constructed no steeper than a maximum of 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

Excavations will likely expose a stiff clay-silt stratum in the west bridge approach, and potentially the east approach. These soils could be difficult to manage; these soils are susceptible to disturbance and rutting as a result of exposure to water or construction traffic. Use of excavated, clayey silt as structural backfill or beneath the new pavement structure will not be permitted. Native soils and existing fill soils may only be used as common borrow in accordance with MaineDOT Standard Specification 203.

Construction of the frame spread footings, stem walls, and wingwalls will require soil and rock excavation and removal of the existing bridge abutments. Cofferdams will be required to permit construction of 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 elevation may vary based on the presence of fractured bedrock.

The subgrade for spread footings for frames and retaining walls 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 or dislodged bedrock fragments by mechanical means. Expansive agents, hydraulic hoe ram, hydraulic splitters, and wedging and prying are examples of tools and methods for removing bedrock by mechanical means. The final bedrock bearing surface shall be washed with high pressure water and air prior to concrete being placed for the arch and wingwall footings. The Resident shall approve the final bedrock surface prior to placement of the footing concrete.

The bedrock subgrade for foundations shall slope less than 4H:1V. If the bedrock subgrade slopes greater than 4H:1V, the subgrade shall be benched in level steps or excavated to be completely level. 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 at frame or wingwall footing locations.

The Contractor may excavate bedrock using conventional excavation methods, but excavation of the bedrock may 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.

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 use by the MaineDOT Bridge Program for the specific application to the proposed replacement of Pearce Brook No. 6 Bridge in Houlton, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

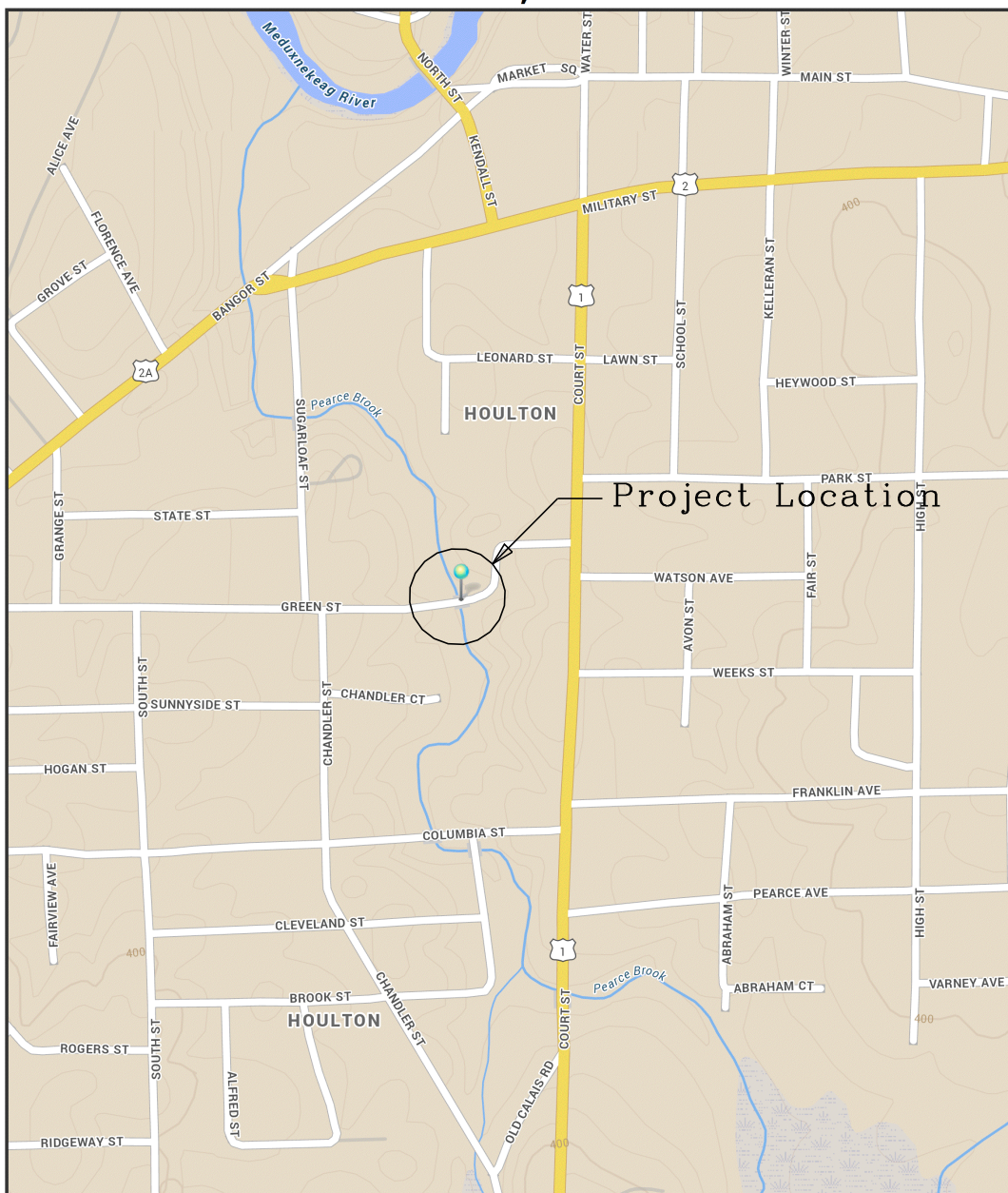
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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

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

Sheets



HOULTON, MAINE



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

0.1 Miles
1 inch = 0.14 miles

Date: 6/26/2018
Time: 8:35:46 AM

SHEET NUMBER

1

OF 3

PEARCE BROOK BRIDGE
PEARCE BROOK

HOULTON AROOSTOOK COUNTY

LOCATION MAP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

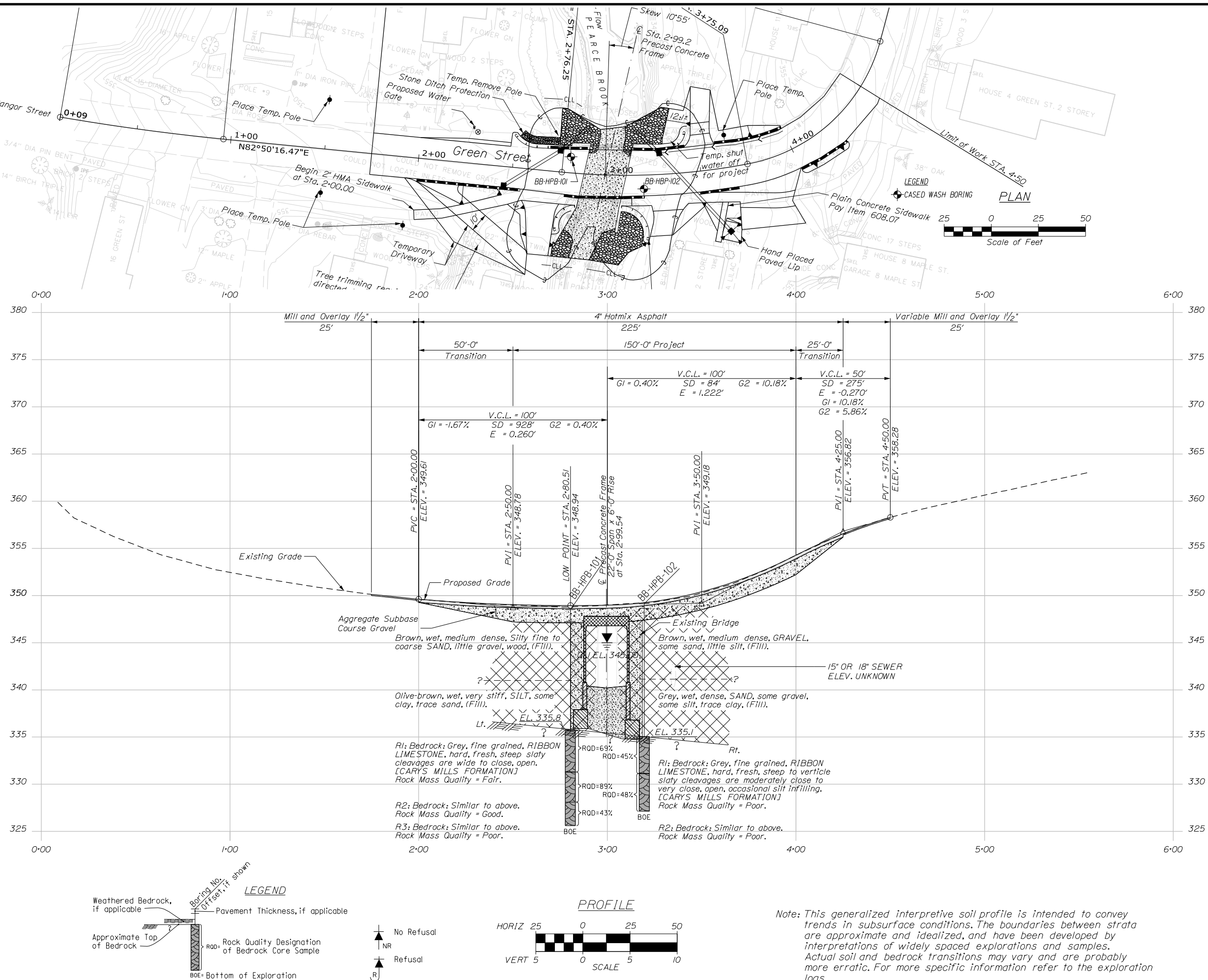
STP-2169(400)

WIN

BRIDGE NO. 5019

21694.00

BRIDGE PLANS



Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil 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: Pearce Brook No. 6 Bridge #5019 Location: Houlton, Maine				Boring No.: BB-HPB-101			
Driller: MaineDOT				Elevation (ft.): 348.5				Auger ID/OD: 5" Solid Stem			
Operator: Travis/James/Rick				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 5/11/2017: 08:30-11:30				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: 2+80.4, 8.5 ft Lt.				Casing ID/OD: NW-3"				Water Level*: 9.0 ft bgs.			
Hammer Efficiency Factor: 0.854				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WH = Weight of 140lb. Hammer ROR/C = Weight of Rock or Casing WGP = Weight of One Person				S _u = Peak/Remained Field Vane Undrained Shear Strength (psf) S _{ul} (lab) = Lab Vane Undrained Shear Strength (psf) q _u = Unconfined Compressive Strength (ksf) N = Uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-corrected Corrected for Hammer Efficiency N ₆₀ = Hammer Efficiency Factor/60%N-corrected			
Sample Information				Visual Description and Remarks				Laboratory Testing Results/AASHTO and Unified Class			
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows /6 in. Shear Strength or ROR (%)	N-corrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log		
0								347.3		15" HMA.	
5	10	24/14	5.00 - 7.00	7/9/5/3	14	20	26			Brown, wet, medium dense, Silty fine to coarse SAND, little gravel, wood. (Fill).	
10	20	24/20	9.00 - 11.00	7/9/10/13	19	27	20			Olive-brown, wet, very stiff, SILT, some clay, trace sand, (Fill).	
	R1	52.8/52.8	12.80 - 17.20	ROR = 69%			040 NQ-2	335.8		940 blows for 0.7 ft.	
15										Top of Bedrock at Elev. 335.8 ft. Roller Coned ahead to 12.8 ft bgs. R1: Bedrock: Grey, fine grained, RIBBON LIMESTONE, hard, fresh, steep slaty cleavages are wide to close, tight, rough, undulating. One horizontal to low angle joint. [CARYS MILLS FORMATION] Rock Mass Quality = Fair. R1: Core Times (min:sec) 12.8-13.8 ft (3:30) 13.8-14.8 ft (3:28) 14.8-15.8 ft (3:51) 15.8-16.8 ft (4:00) 16.8-17.2 ft (3:00) 100% Recovery	
20	R2	38.4/36	17.20 - 20.40	ROR = 89%						R2: Bedrock: Similar to above. Rock Mass Quality = Good. R2: Core Times (min:sec) 17.2-18.2 ft (2:45) 18.2-19.2 ft (2:40) 19.2-20.2 ft (3:00) 20.2-20.4 ft (3:00) 95% Recovery	
	R3	30/30	20.40 - 22.90	ROR = 43%				325.6		R3: Bedrock: Similar to above. Rock Mass Quality = Poor. R3: Core Times (min:sec) 20.4-21.4 ft (3:15) 21.4-22.4 ft (3:06) 22.4-22.9 ft (2:15) 100% Recovery	
25										Bottom of Exploration at 22.9 feet below ground surface.	
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.											
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											
Page 1 of 1											
Boring No.: BB-HPB-101											

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Pearce Brook No. 6 Bridge #5019 Location: Houlton, Maine				Boring No.: BB-HPB-102			
Driller: MaineDOT				Elevation (ft.): 349.2				Auger ID/OD: 5" Solid Stem			
Operator: Travis/James/Rick				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 5/11/2017: 11:30-14:30				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: 3+19.6, 7.4 ft Rt.				Casing ID/OD: NW-3"				Water Level*: 7.6 ft bgs.			
Hammer Efficiency Factor: 0.854				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WH = Weight of 140lb. Hammer ROR/C = Weight of Rock or Casing WGP = Weight of One Person				S _u = Peak/Remained Field Vane Undrained Shear Strength (psf) S _{ul} (lab) = Lab Vane Undrained Shear Strength (psf) q _u = Unconfined Compressive Strength (ksf) N = Uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-corrected Corrected for Hammer Efficiency N ₆₀ = Hammer Efficiency Factor/60%N-corrected			
Sample Information				Visual Description and Remarks				Laboratory Testing Results/AASHTO and Unified Class			
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows /6 in. Shear Strength or ROR (%)	N-corrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log		
0								348.7		6" HMA.	
5	10	24/13	5.00 - 7.00	9/17/5/17	22	31				Brown, wet, medium dense, GRAVEL, some sand, little silt, (Fill).	
10	20	24/17	10.00 - 12.00	9/11/18/20	29	41	36			Grey, wet, dense, SAND, some gravel, some silt, trace clay, (Fill).	
15	R1	46.8/44	14.10 - 18.00	ROR = 45%			050 NQ-2	335.1		950 blows for 0.1 ft.	
										Top of Bedrock at Elev. 335.1 ft. R1: Bedrock: Grey, fine grained, RIBBON LIMESTONE, hard, fresh, steep to vertical slaty cleavages are moderately close to very close, tight to open, occasional silt infilling, rough, undulating, infrequent horizontal to low angle healed joints. [CARYS MILLS FORMATION] Rock Mass Quality = Poor. R1: Core Times (min:sec) 14.1-15.1 ft (2:30) 15.1-16.1 ft (2:08) 16.1-17.1 ft (3:21) 17.1-18.0 ft (3:45) 94% Recovery	
20	R2	48/48	18.00 - 22.00	ROR = 48%				327.2		R2: Bedrock: Similar to above. Rock Mass Quality = Poor. R2: Core Times (min:sec) 18.0-19.0 ft (3:15) 19.0-20.0 ft (3:26) 20.0-21.0 ft (4:01) 21.0-22.0 ft (4:08) 100% Recovery	
25										Bottom of Exploration at 22.0 feet below ground surface.	
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.											
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											
Page 1 of 1											
Boring No.: BB-HPB-102											

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER SYSTEM			
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES				
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Descriptive Term		Portion of Total (%)	
		(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			
						TERMS DESCRIBING DENSITY/CONSISTENCY		
					Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).			
					Density of Cohesionless Soils		Standard Penetration Resistance N-Value (blows per foot)	
					Very loose		0 - 4	
					Loose		5 - 10	
					Medium Dense		11 - 30	
					Dense		31 - 50	
					Very Dense		> 50	
					Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to undrained shear strength as indicated.			
					Consistency of Cohesive soils		SPT N-Value (blows per foot)	
					Very Soft		WOH, WOR, WOP, <2	
					Soft		2 - 4	
					Medium Stiff		5 - 8	
					Stiff		9 - 15	
					Very Stiff		16 - 30	
					Hard		>30	
					Approximate Undrained Shear Strength (psf)		Field Guidelines	
					0 - 250		Fist easily penetrates	
					250 - 500		Thumb easily penetrates	
					500 - 1000		Thumb penetrates with moderate effort	
					1000 - 2000		Indented by thumb with great effort	
					2000 - 4000		Indented by thumbnail	
					over 4000		Indented by thumbnail with difficulty	
					Rock Quality Designation (RQD):			
					RQD (%) = sum of the lengths of intact pieces of core* > 4 inches length of core advance			
					*Minimum NQ rock core (1.88 in. OD of core)			
					Correlation of RQD to Rock Mass Quality			
					Rock Mass Quality		RQD (%)	
					Very Poor		≤25	
					Poor		26 - 50	
					Fair		51 - 75	
					Good		76 - 90	
					Excellent		91 - 100	
					Desired Rock Observations (in this order, if applicable):			
					Color (Munsell color chart)			
					Texture (aphanitic, fine-grained, etc.)			
					Rock Type (granite, schist, sandstone, etc.)			
					Hardness (very hard, hard, mod. hard, etc.)			
					Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)			
					Geologic discontinuities/jointing:			
					-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)			
					-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)			
					-tightness (tight, open, or healed)			
					-infilling (grain size, color, etc.)			
					Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)			
					RQD and correlation to rock mass quality (very poor, poor, etc.)			
					ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A			
					Recovery (inch/inch and percentage)			
					Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
					Sample Container Labeling Requirements:			
					WIN		Blow Counts	
					Bridge Name / Town		Sample Recovery	
					Boring Number		Date	
					Sample Number		Personnel Initials	
					Sample Depth			
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information								

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Pearce Brook No. 6 Bridge #5019 carries Green Road over Pearce Brook Location: Houlton, Maine				Boring No.: BB-HPB-101 WIN: 21694.00																																																																																																																																																																																																																			
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<table><tr><th colspan="8">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 and Unified Class</th></tr><tr><th>Depth (ft.)</th><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><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>SSA</td><td>347.3</td><td></td><td>15" HMA.</td><td rowspan="16">G#304376 A-4, ML WC=24.2% LL=26 PL=25 PI=1</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></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></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></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></td><td></td></tr><tr><td>5</td><td>1D</td><td>24/14</td><td>5.00 - 7.00</td><td>7/9/5/3</td><td>14</td><td>20</td><td>26</td><td></td><td></td><td>Brown, wet, medium dense, Silty fine to coarse SAND, little gravel, wood, (Fill).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>14</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>12</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>22</td><td></td><td></td><td></td></tr><tr><td>10</td><td>2D</td><td>24/20</td><td>9.00 - 11.00</td><td>7/9/10/13</td><td>19</td><td>27</td><td>20</td><td></td><td></td><td>Olive-brown, wet, very stiff, SILT, some clay, trace sand, (Fill).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>47</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>43</td><td></td><td></td><td></td></tr><tr><td></td><td>R1</td><td>52.8/52.8</td><td>12.80 - 17.20</td><td>RQD = 69%</td><td></td><td></td><td>a40 NQ-2</td><td>335.8</td><td></td><td>a40 blows for 0.7 ft.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Top of Bedrock at Elev. 335.8 ft. 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R3: Core Times (min:sec) 20.4-21.4 ft (3:15)</td></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>												Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class	Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	0							SSA	347.3		15" HMA.	G#304376 A-4, ML WC=24.2% LL=26 PL=25 PI=1																																													5	1D	24/14	5.00 - 7.00	7/9/5/3	14	20	26			Brown, wet, medium dense, Silty fine to coarse SAND, little gravel, wood, (Fill).								14											12											22				10	2D	24/20	9.00 - 11.00	7/9/10/13	19	27	20			Olive-brown, wet, very stiff, SILT, some clay, trace sand, (Fill).								47											43					R1	52.8/52.8	12.80 - 17.20	RQD = 69%			a40 NQ-2	335.8		a40 blows for 0.7 ft.											Top of Bedrock at Elev. 335.8 ft. 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	R1	52.8/52.8	12.80 - 17.20	RQD = 69%			a40 NQ-2	335.8		a40 blows for 0.7 ft.																																																																																																																																																																																																																	
										Top of Bedrock at Elev. 335.8 ft. Roller Coned ahead to 12.8 ft bgs. R1: Bedrock: Grey, fine grained, RIBBON LIMESTONE, hard, fresh, steep slaty cleavage are wide to close, tight, rough, undulating. One horizontal to low angle joint. [CARYS MILLS FORMATION] Rock Mass Quality = Fair. R1: Core Times (min:sec) 12.8-13.8 ft (3:30) 13.8-14.8 ft (3:28) 14.8-15.8 ft (3:51) 15.8-16.8 ft (4:00) 16.8-17.2 ft (3:00) 100% Recovery																																																																																																																																																																																																																	
15	R2	38.4/36	17.20 - 20.40	RQD = 89%						R2: Bedrock: Similar to above. Rock Mass Quality = Good. R2: Core Times (min:sec) 17.2-18.2 ft (2:45) 18.2-19.2 ft (2:40) 19.2-20.2 ft (3:00) 20.2-20.4 ft (3:00) 95% Recovery																																																																																																																																																																																																																	
20	R3	30/30	20.40 - 22.90	RQD = 43%				325.6		R3: Bedrock: Similar to above. Rock Mass Quality = Poor. R3: Core Times (min:sec) 20.4-21.4 ft (3:15)																																																																																																																																																																																																																	
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Pearce Brook No. 6 Bridge #5019 carries Green Road over Pearce Brook Location: Houlton, Maine				Boring No.: BB-HPB-101 WIN: 21694.00			
Driller: MaineDOT				Elevation (ft.) 348.5				Auger ID/OD: 5" Solid Stem			
Operator: Travis/James/Rick				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 5/11/2017; 08:30-11:30				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: 2+80.4, 8.5 ft Lt.				Casing ID/OD: NW-3"				Water Level*: 9.0 ft bgs.			
Hammer Efficiency Factor: 0.854				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_u(lab) = Lab Vane Undrained Shear Strength (psf) q_u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25									21.4-22.4 ft (3:06)		22.9 Bottom of Exploration at 22.9 feet below ground surface.
									22.4-22.9 ft (2:15) 100% Recovery		
30											
35											
40											
45											
50											

Remarks:

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

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

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Boring No.: BB-HPB-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Pearce Brook No. 6 Bridge #5019 carries Green Road over Pearce Brook Location: Houlton, Maine		Boring No.: BB-HPB-102 WIN: 21694.00	
Driller: MaineDOT		Elevation (ft.) 349.2		Auger ID/OD: 5" Solid Stem			
Operator: Travis/James/Rick		Datum: NAVD88		Sampler: Standard Split Spoon			
Logged By: B. Wilder		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 5/11/2017; 11:30-14:30		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"			
Boring Location: 3+19.6, 7.4 ft Rt.		Casing ID/OD: NW-3"		Water Level*: 7.6 ft bgs.			
Hammer Efficiency Factor: 0.854		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐					
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt						R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person	
S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected						T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test	

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0							SSA	348.7		6" HMA.	G#304377 A-1-a, GW-GM WC=10.8%	
5	1D	24/13	5.00 - 7.00	9/17/5/17	22	31				Brown, wet, medium dense, GRAVEL, some sand, little silt, (Fill).	G#304378 A-2-4, SC-SM WC=11.8%	
10	2D	24/17	10.00 - 12.00	9/11/18/20	29	41	36			Grey, wet, dense, SAND, some gravel, some silt, trace clay, (Fill).	G#304378 A-2-4, SC-SM WC=11.8%	
							60					
							79					
							175					
15	R1	46.8/44	14.10 - 18.00	RQD = 45%			a50	335.1		a50 blows for 0.1 ft.	G#304378 A-2-4, SC-SM WC=11.8%	
							NQ-2					
20	R2	48/48	18.00 - 22.00	RQD = 48%				327.2		Top of Bedrock at Elev. 335.1 ft. R1: Bedrock: Grey, fine grained, RIBBON LIMESTONE, hard, fresh, steep to vertical slaty cleavages are moderately close to very close, tight to open, occasional silt infilling, rough, undulating. Infrequent horizontal to low angle healed joints. [CARYS MILLS FORMATION] Rock Mass Quality = Poor. R1: Core Times (min:sec) 14.1-15.1 ft (2:30) 15.1-16.1 ft (2:08) 16.1-17.1 ft (3:21) 17.1-18.0 ft (3:45) 94% Recovery R2: Bedrock: Similar to above. Rock Mass Quality = Poor. R2: Core Times (min:sec) 18.0-19.0 ft (3:15) 19.0-20.0 ft (3:26) 20.0-21.0 ft (4:01) 21.0-22.0 ft (4:08) 100% Recovery	G#304378 A-2-4, SC-SM WC=11.8%	
25										Bottom of Exploration at 22.0 feet below ground surface.		

Remarks:

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

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

Page 1 of 1

Boring No.: BB-HPB-102

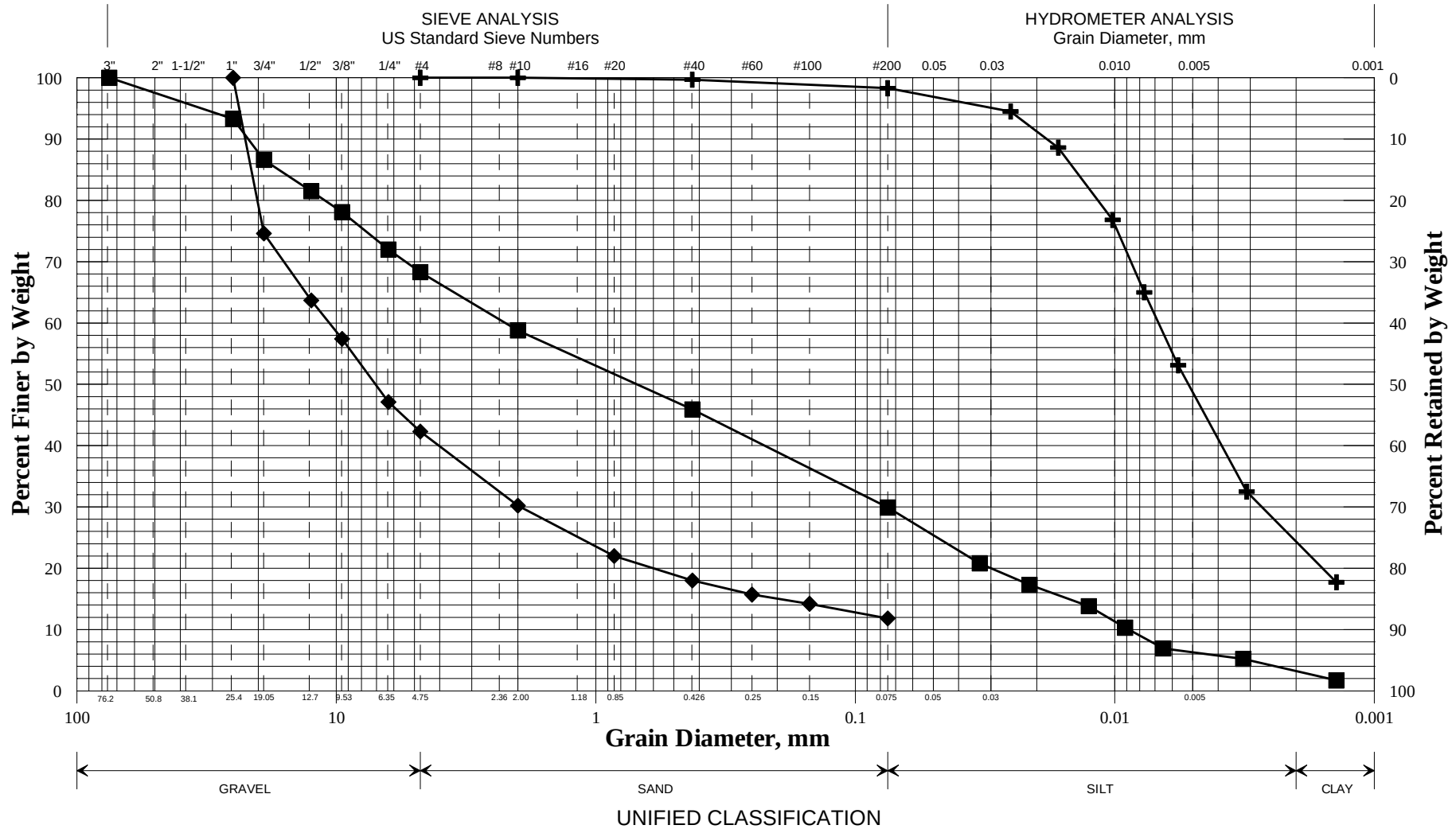
Appendix B

Laboratory Test Results

Work Number: 21694.00

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-HPB-101/2D	2+80.4	8.5 LT	9.0-11.0	SILT, some clay, trace sand.	24.2	26	25	1
◆	BB-HPB-102/1D	3+19.6	7.4 RT	5.0-7.0	GRAVEL, some sand, little silt.	10.8			
■	BB-HPB-102/2D	3+19.6	7.4 RT	10.0-12.0	SAND, some gravel, some silt, trace clay.	11.8			
●									
▲									
×									

WIN	
021694.00	
Town	
Houlton	
Reported by/Date	
WHITE, TERRY A	6/14/2017



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **304376** Boring No./Sample No. **BB-HPB-101/2D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **5/11/2017** Received **5/22/2017**

Sample Type: **GEOTECHNICAL** Location: Station: **2+80.4** Offset, ft: **8.5** LT Dbfg, ft: **9.0-11.0**

WIN/Town **021694.00 - HOULTON** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.7
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	98.3
[0.0252 mm]	94.5
[0.0165 mm]	88.6
[0.0102 mm]	76.8
[0.0077 mm]	65.0
[0.0057 mm]	53.1
[0.0031 mm]	32.5
[0.0014 mm]	17.7

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	26
Plastic Limit (T 90), %	25
Plasticity Index (T 90), %	1
Specific Gravity, Corrected to 20°C (T 100)	2.60
Loss on Ignition, % (T 267)	
Water Content (T 265), %	24.2

Consolidation (T 216)

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

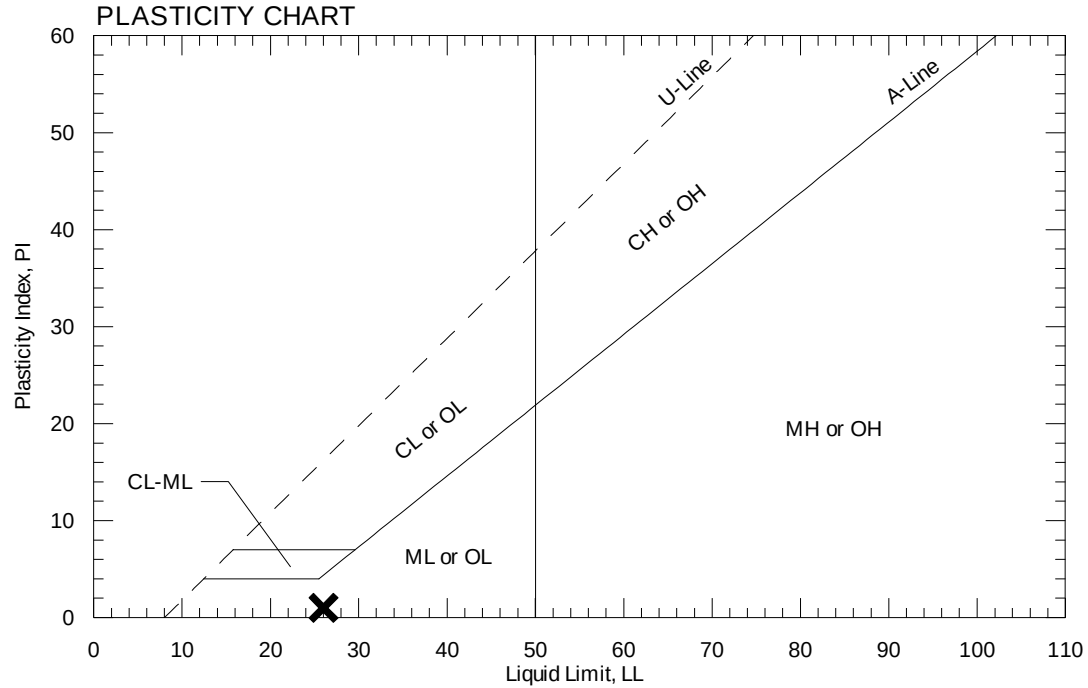
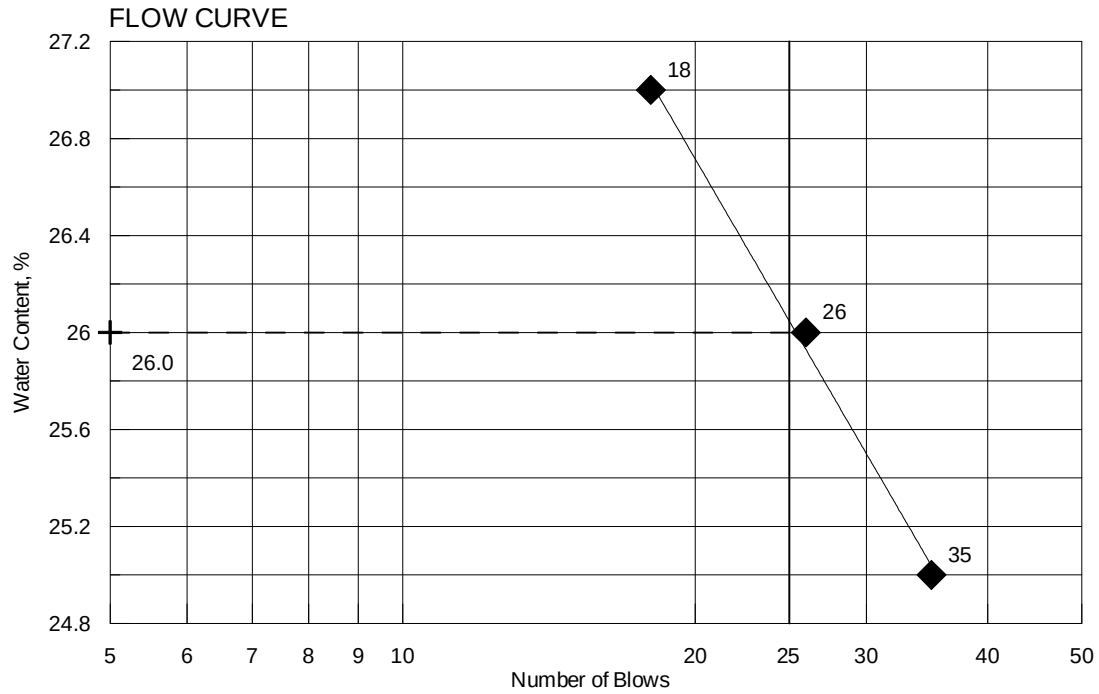
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **6/13/2017**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Houlton	Reference No.	304376
WIN	021694.00	Water Content, %	24.2
Sampled	5/11/2017	Liquid Limit @ 25 blows (T 89), %	26
Boring No./Sample No.	BB-HPB-101/2D	Plastic Limit (T 90), %	25
Station	2+80.4	Plasticity Index (T 90), %	1
Depth	9.0-11.0	Tested By	BBURR



Appendix C

Calculations

Earth Pressure

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

Rigid frame stem walls and footing (restrained) - At-Rest Earth Pressure - Rankine Theory

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

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

$$K_o = 0.47$$

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

Wingwalls and headwalls free to rotate - Active Earth Pressure - Rankine Theory

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

For cantilver walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

β = Angle of fill slope to the horizontal $\beta := 26.6 \cdot \text{deg}$

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

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

Recommend: Active Earth Pressure Coefficient, $K_a = 0.46$

6.1 AT-REST LATERAL PRESSURES

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

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

$$\sigma'_x = K_0(\sigma'_z) \quad (6.1)$$

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

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

$$K_0 = 1 - \sin \phi' \quad (6.2)$$

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

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

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

Soils that are overconsolidated and are in the process of being reloaded pose a difficulty in that Equation 6.3 does not apply. For this condition, a more complex equation is needed as well as a full knowledge of the stress history of the soil (Mayne and Kulhawy, 1982). For practical purposes, it may

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

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

^a Unloading cycle.

be enough to know that the K_0 during reloading falls about halfway between that for unloading and normally consolidated conditions. Also, K_0 might be directly determined through in-situ testing methods.

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

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

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

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

6.2.1 Active Pressure

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

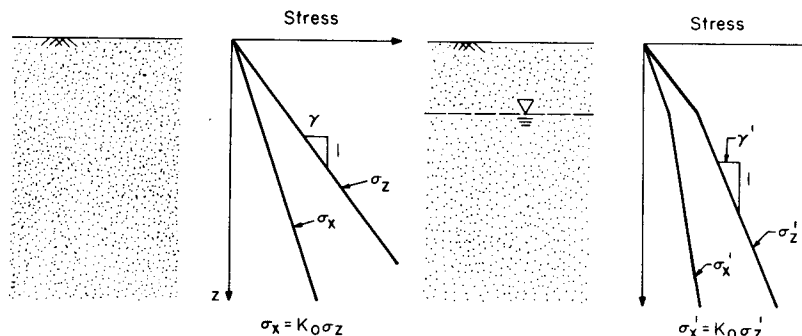


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

where:

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

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

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

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

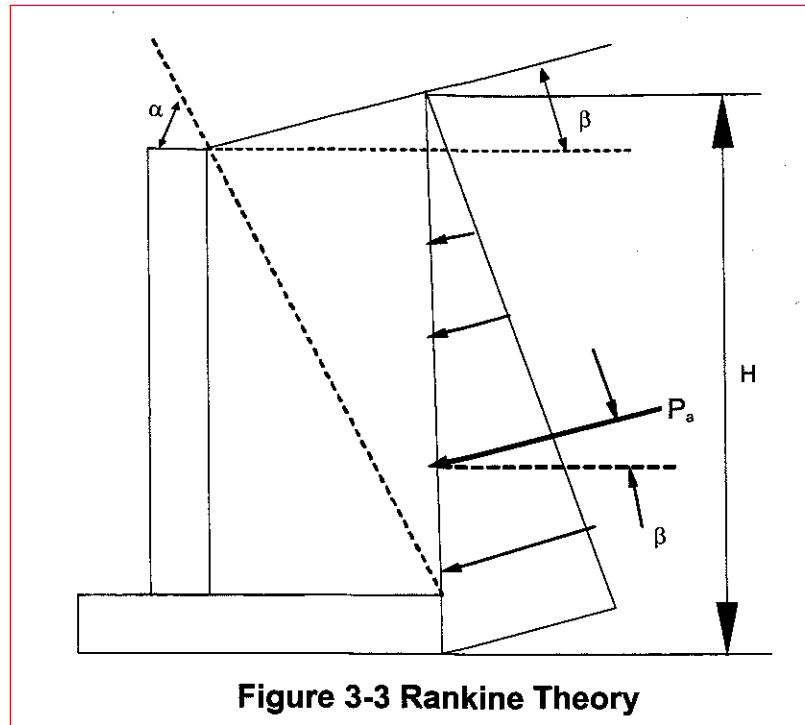


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

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

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

Bearing Resistance

Analysis

Calculation of nominal and factored bearing resistance on rock for Strength and Extreme Limit State Analysis.

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, with 2018 Interims
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

Grey, fine grained, Ribbon Lime (Mudstone/slate with thin limestone beds), hard, fresh, steep to vertical slaty cleavages are very close to wide.

RQD = 45%. Rock Mass Quality = Poor.

Compressive Strength

Assume a average design compressive strength of 9,200 psi for Cary's Mills Formation, which is called "ribbon lime". It is a limy slate - the original mudstone beds remained black, the limestone beds weathered to light tan or white "ribbons".

AASHTO Standard Specs, 17th Edition, 2002
Use middle of range for siltstone

Use $q_u = 9,200$ psi or 1325 ksf

$$q_{uc} := 9200 \cdot \text{psi}$$

B. 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

$q_{u1} = 1325$ ksf

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength = 1325 ksf: **Relative Rating = 7**

2. Drill Core Quality

Assume very poor rock is cleaned away.

Bedrock RQD = 45% (Poor) From LRFD Table 10.4.6.4-1, RQD between 25 and 50%; **Relative Rating = 8**

3. Spacing of joints

Assume broken or highly weathered rock is removed. Boring logs indicate breaks are wide to very close. Predominately close (2 - 12 in) breaks.

From LRFD Table 10.4.6.4-1 Spacing of joints 2-12 in; **Relative Rating = 10**

4. Condition of joints

Break surfaces are hard, slightly rough; **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

Breaks along bedding are horizontal to low angle. **Relative Rating = -7**

ADJUSTED RMR

$$RMR := 7 + 8 + 10 + 20 + 4 - 7$$

$$RMR = 42$$

Determine Rock Type for LRFD Table 10.4.6.4-4

Rock Type - B = Lithified Argillaceous Rocks
mudstone, siltstone, shale, and slate (normal to cleavage)

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.6.4-3, RMR = 42 is Class No. III and described as Fair rock.

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

$$\text{RMR} = 42$$

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.

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

$$m_i = m \text{ for intact rock} \quad s = \exp((\text{RMR}-100)/6)$$

For Rock Type B (mudstone, siltstone, slate) for intact rock, $\text{RMR}=100$, $m_i = 10$ (Ref. # 4, Table 1) and $s = 1$

$$m_i := 10$$

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

$$m = 0.159$$

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

$$s = 0.0000634$$

D. Nominal and Factored Bearing Resistance of Bedrock

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

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

Nominal Bearing Resistance (Wyllie)

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

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

$$q_{n1} = 59 \cdot \text{ksf}$$

Factored Bearing Resistances

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

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

$$q_{r1} := q_{n1} \cdot \phi_{bc}$$

$$q_{r1} = 26 \cdot \text{ksf}$$

Strength Limit State

Recommendation: Use 26 ksf for Factored Bearing Resistance at the Strength Limit State

Factored Bearing Resistances

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

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

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

$$q_{rl} = 47 \cdot \text{ksf}$$

Extreme Limit State

Recommendation: Use 47 ksf for Factored Bearing Resistance at the 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.

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

$$q_{nl} = 59 \cdot \text{ksf}$$

Analysis

Calculation of nominal and factored bearing resistance on bedrock for Service Limit State Analysis

Method 1

Per AASHTO LRFD 10.6.2.4.4 - Settlement of Footings on Rock, "For footings bearing on fair to very good rock according to Geomechanics Classification system (i.e. RMR), as defined in Article 10.4.6.4, and designed in accordance with the provisions of this Section, **elastic settlement may generally be assumed to be less than 0.5 inch.**"

Method 2

LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Abutment No. 1 and Abutment No. 2

Bearing Material:	Weathered or broken bedrock of any kind, except shale.
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16-24 ksf
AASHTO Recommended Value	20 ksf

Resistance Factor for Service Limit State

$$\phi_r := 1.0$$

$$q_{lnominal} := 20 \cdot \text{ksf}$$

Per LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values for the factored bearing resistance for Service Limit State Analyses, settlement is typically limited to 1 inch

$$q_{factored} := \phi_r \cdot q_{lnominal}$$

$$q_{factored} = 20 \cdot \text{ksf}$$

Service Limit State

Recommendation: Use 20 ksf for Factored Bearing Resistance at the Service Limit State

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

white "ribbons"

Cary's Mill Formation is "Ribbon Lime" or a limy slate. The mudstone beds are black, the white "ribbons" are limestone beds.

⁽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

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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		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• 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> <i>R</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> <i>R</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> <i>R</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> <i>R</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> <i>R</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> <i>R</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.

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.

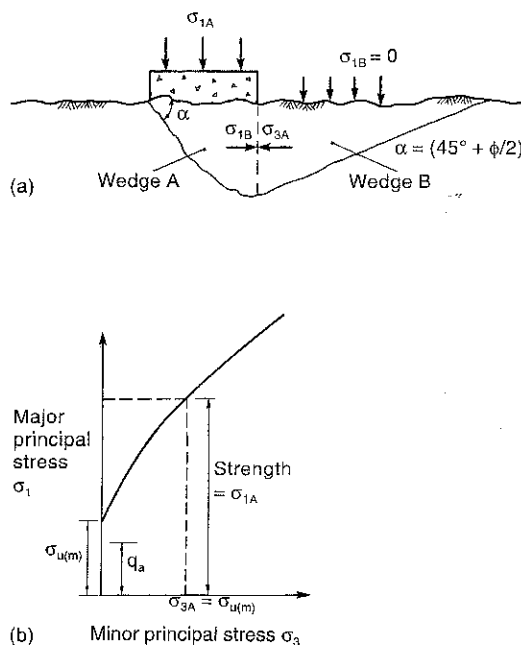


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a , strength of bearing rock = σ_{1A} , factor of safety $FS = \sigma_{1A}/q_a$.

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

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

21694.00 Houlton	Bearing Resistance Precast Concrete Modular Gravity Wall	B. Slaven May 2019 Checked by: LK 5/2019
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Objective:

Estimate the factored bearing resistance for precast concrete modular gravity wall bearing on soil placed on unreinforced concrete 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. Face of PCMG walls will be constructed on concrete leveling slab on bedrock
2. The stems and earth fill in the bottom unit will bear on soil behind the concrete leveling slab.
3. The bedrock elevation is approximately El. 335 to 336 feet.
4. The highest wall section will be 10 feet or 4 PCMG wall units.
5. Typical precast concrete modular wall stem lengths are 8-10-12-14 feet.
6. Most of the wall will be submerged for the structure's design life. (Q1.1 at El. 345.0)

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, and 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

$$B' := \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \text{ ft} \quad B' = \text{Stem Length}$$

$$D_f := 4.0 \cdot \text{ft}$$

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

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

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

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

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

Foundation soils:

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

$$w_{\text{sat}} := .10$$

$$\gamma_{1\text{sat}} := \gamma_{1t} \cdot (1 + w_{\text{sat}})$$

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

$$\gamma_{1\text{sat}} = 137.5 \cdot \text{pcf}$$

$$\phi := 36 \cdot \text{deg}$$

MaineDOT Bridge Design Guide p. 3-3
Soil Type 5

$$c := 0$$

Cohesion

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 $\phi = 36$ degrees.

$$N_c := 50.55$$

$$N_q := 37.7$$

$$N_{\gamma} := 44.4$$

Nominal Bearing Resistance per Terzaghi equation

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

$$q = 0.25 \cdot \text{ksf}$$

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

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

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

$$q_n = \begin{pmatrix} 20.1 \\ 22.8 \\ 25.4 \\ 28.1 \end{pmatrix} \cdot \text{ksf}$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\phi_b := 0.45$$

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

$$q_r = \begin{pmatrix} 9 \\ 10.3 \\ 11.5 \\ 12.7 \end{pmatrix} \cdot \text{ksf}$$

for

$$B' = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot \text{ft}$$

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)

$$N_c := 50.6$$

$$N_q := 37.8$$

$$N_\gamma := 56.3$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$$B' = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot \text{ft} \quad L := 5 \cdot \text{ft}$$

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

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

$$s_\gamma = \begin{pmatrix} 0.36 \\ 0.2 \\ 0.04 \\ -0.12 \end{pmatrix} \quad s_q = \begin{pmatrix} 2.16 \\ 2.45 \\ 2.74 \\ 3.03 \end{pmatrix}$$

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 invert of the structure for the entire design life.

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

$$\text{LRFD Table 10.6.3.1.2a-4} \quad \frac{D_f}{10 \cdot \text{ft}} = 0.4$$

$$\text{Therefore :} \quad d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 65.78$$

$$N_{\gamma m} = \begin{pmatrix} 20.268 \\ 11.26 \\ 2.252 \\ -6.756 \end{pmatrix}$$

$$N_{qm} = \begin{pmatrix} 81.741 \\ 92.727 \\ 103.712 \\ 114.697 \end{pmatrix}$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = \begin{pmatrix} 26 \\ 27.1 \\ 26.9 \\ 25.4 \end{pmatrix} \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$q_r = \begin{pmatrix} 11.7 \\ 12.2 \\ 12.1 \\ 11.4 \end{pmatrix} \cdot \text{ksf}$$

Recommend 10 ksf for the factored bearing resistance for precast concrete modular wall units of any stem length at the Strength Limit State.

3. Bearing Resistance for Extreme Limit States

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1) from above:

$$q_n = \begin{pmatrix} 26 \\ 27.1 \\ 26.9 \\ 25.4 \end{pmatrix} \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_e := 1.0$$

LRFD Article 10.5.5.3.3

$$q_r := q_n \cdot \phi_e$$

$$q_r = \begin{pmatrix} 26 \\ 27.1 \\ 26.9 \\ 25.4 \end{pmatrix} \cdot \text{ksf}$$

Recommend a factored bearing resistance of 25 ksf for precast concrete modular walls of any stem length, at the Extreme Limit State (traffic impact).

Frost Depth

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

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

DFI = 2200 degree-days.

Case 1 - coarse grained granular fill soils W=10% (based on soil lab testing).

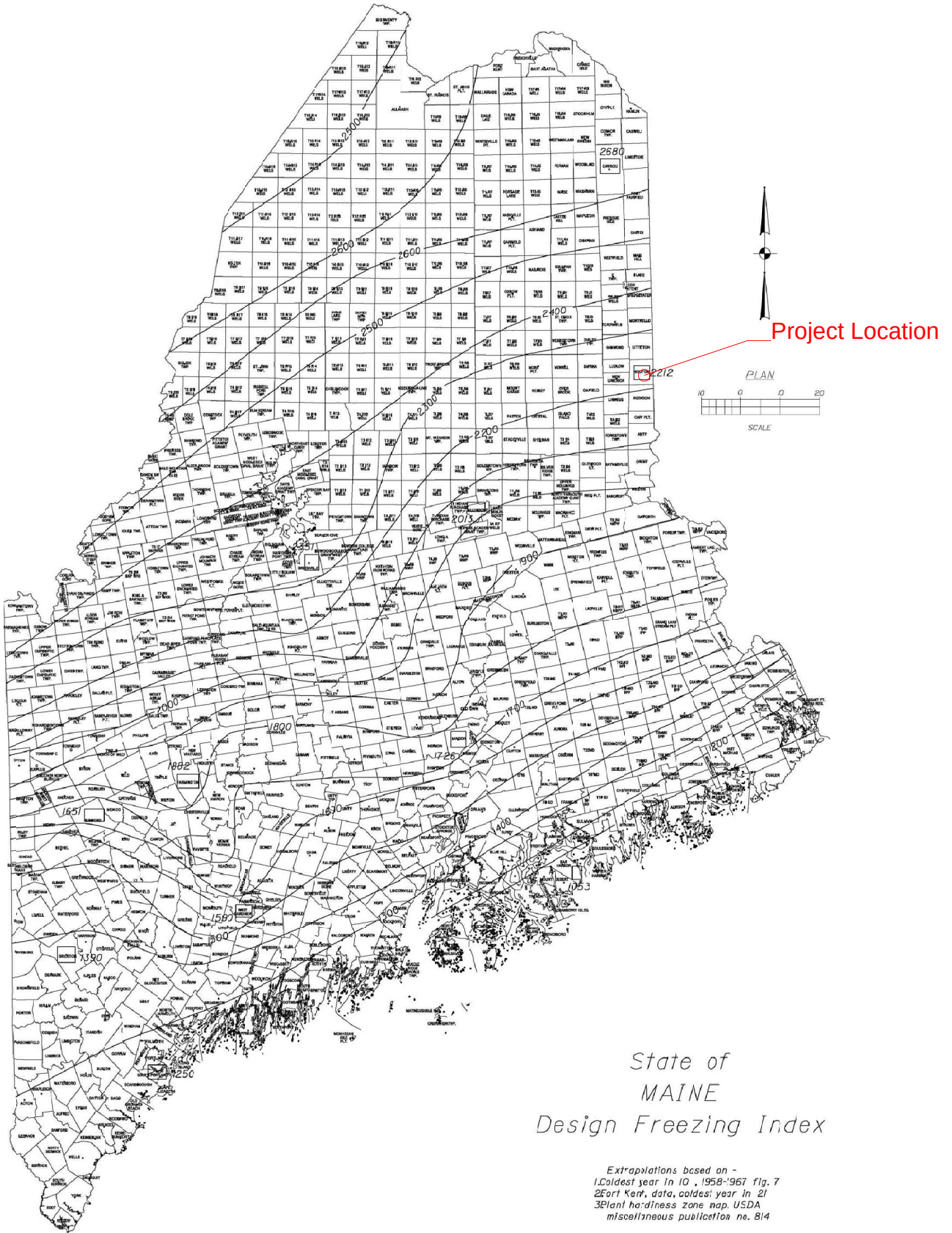
For DFI = 2200

$d := 100\text{in}$

Depth of Frost Penetration

$d = 8.3\cdot\text{ft}$

Figure 5-1 Maine Design Freezing Index Map



5.2 General**MaineDOT Bridge Design Guide****5.2.1 Frost**

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

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

Table 5-1 Depth of Frost Penetration

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