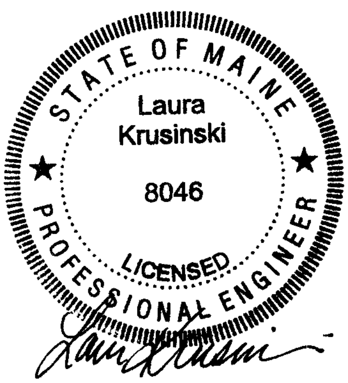
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

HODGDON STREAM BRIDGE
ROUTE 2A OVER SOUTH BRANCH MEDUXNEKEAG RIVER
HOULTON, MAINE



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PIN 15629.00

Soils Report No. 2008-13
Bridge No. 3458

Fed No. STP-1562(900)X
August 2008

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement Hodgdon Stream Bridge which carries Route 2A over South Branch Meduxnekeag River, in Houlton, Maine. The proposed replacement bridge will be a simply supported, single span bridge on cantilever-type abutments. The superstructure curb-to-curb width will be increased from 26 feet to 32 feet and will be centered on the existing alignment.

Preliminary foundation alternatives were provided by the geotechnical team member in an internal Geotechnical Design Memorandum, dated January 14, 2008. Subsequent preliminary design studies by Maine Department of Transportation (MaineDOT) Bridge Program identified the most practicable foundation type for this site to be cantilever-type abutments on spread footings founded directly on bedrock or seal concrete founded on bedrock. The following design recommendations are discussed in detail in this report:

Cantilever Abutments and Wingwalls - Abutments and wingwalls shall be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and any loads transferred through the superstructure. They shall be designed for all relevant strength and service limit states in accordance with AASHTO LRFD Bridge Design Specifications 4th Edition, 2007, (herein referred to as LRFD).

The design of project abutments founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths (3/8) of the footing dimensions, in either direction.

Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory for cantilever wingwalls. The Designer may assume Soil Type 4 [Bridge Design Guide (BDG) Section 3.6.1] for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pounds per cubic foot (pcf). Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG for the abutments and wingwalls if an approach slab is not specified. In the case a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD 3.11.6.2.

Bearing Capacity – The factored bearing pressure at the strength limit state for spread footings on sound bedrock should not exceed the factored bearing resistance of 35 kips per square foot (ksf). Based on presumptive bearing resistance values, a factored bearing resistance of 20 ksf may be used when analyzing the service limit state and for preliminary footing sizing, as allowed in LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the footing concrete, which may be taken as 0.3 *f'c*.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

Scour and Riprap - For scour protection, bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the BDG.

Settlement - The grades of bridge approaches and side slopes will be not raised, therefore post-construction settlement due to compression of the foundation soils is anticipated to be less than 0.5 inch and will have minimal effect on the finished structure. Any settlement of the bridge abutments will be due to elastic settlement of the bedrock, which is assumed to occur during construction and be less than 0.5 inches. Consolidation settlement of the rock mass due to soft, silt-infilled seams is estimated to be less than 0.2 inches.

Frost Protection - Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for project footings cast directly on sound bedrock. Retaining wall foundations placed on granular soils should be founded a minimum of 7.0 feet below finished exterior grade for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

Retaining Walls - A Precast Concrete Modular Gravity (PCMG) wall founded on fill soils will be used to retain approach fills. This wall shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The bearing resistance for the PCMG wall founded on a leveling slab founded on compacted fill soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 11 ksf. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state, and for preliminary footing sizing.

Failure by sliding shall be investigated by the wall subcontractor. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on sand and to the nominal sliding resistance of soil within the precast concrete units on soil. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.29 (0.80·tan 20°) at the foundation soil to concrete interfaces and a maximum frictional coefficient of 0.58 (tan 30°) at foundation soil to soil-infill interfaces.

For the lowest PCMG unit, the eccentricity of factored loads at the strength limit state shall not exceed one-fourth ($1/4^{th}$) of the footing dimensions, in either direction.

Seismic Design Considerations – In conformance with LRFD 4.7.4.2., seismic analysis is not required for single-span bridges, regardless of seismic zone, however superstructure connections and bridge seat dimensions shall be satisfied per LRFD 3.10.9 and 4.7.4.4., respectively.

Construction Considerations – Internally braced cofferdams and temporary lateral earth support systems will be required for abutment, wingwall and PCMG wall construction. Preparation of the bedrock subgrade for abutment footings may require excavation of bedrock to create level benches or a completely level surface. Excavation of bedrock may be conducted using conventional equipment, but may require drilling and blasting methods. All loose and fractured bedrock and soil debris should be removed from bearing surfaces and the surfaces washed with high-pressure water and air before concrete is poured for the abutment foundations.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Hodgdon Stream Bridge which carries State Route 2A over South Branch Meduxnekeag River, in Houlton Maine. This report presents the soils information obtained at the site during the subsurface investigation, foundation recommendations and geotechnical design parameters for bridge replacement.

Hodgdon Stream Bridge was built in 1943 and is a 35-foot single span, concrete T-beam superstructure, supported on full-height, retrofitted concrete gravity abutments. The abutments are comprised of portions of pre-1943 unreinforced concrete abutments with 1943 construction modifications. The 1943 construction modifications included new concrete bridge seats, 2-foot thick reinforced concrete jackets on the abutments and wingwalls, and the addition of French drains and weep holes in the abutment and wings.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports indicate substructure distress in areas in the form of concrete spall, scattered scaling, map cracking and efflorescence. There is minor undercutting and abrasion at the easterly abutment in front of the abutment toe and between the bedrock streambed. Portions of the riprap on the downstream wingwall of the easterly abutment are missing. Year 2006 MaineDOT Bridge Maintenance inspection reports assign the substructures a condition rating of 6 – satisfactory, and indicate a Bridge Sufficiency Rating of 47.3. However, year 2000 Bridge Maintenance inspection reports assign the structure a Bridge Sufficiency Rating of 25.7.

1943 bridge plans indicate the abutments are gravity shaped with a footing width of 7 feet and 2 feet wide at the top. This cross section includes the 2-foot reinforced concrete jacket. It should be noted that actual abutment geometries may vary from those dimensions shown on the Hodgdon Stream Bridge substructures plan sheet, dated September 1943, by the State Highway Commission Bridge Division.

Preliminary foundation alternatives were provided by the geotechnical team member in an internal Geotechnical Design Memorandum, dated January 14, 2008. The May 2008 Scope Review Team report considered the extensive deterioration of the superstructure and the narrow bridge width, and recommended bridge improvement, consisting of superstructure replacement or total bridge replacement. Subsequent preliminary engineering assessments by the MaineDOT Bridge Program resulted in the recommendation for a bridge replacement project with foundations consisting of cantilever-type abutments on spread footings founded directly on sound bedrock or seal concrete founded on bedrock.

2.0 GEOLOGIC SETTING

Hodgdon Stream Bridge on State Route 2A in Houlton, Maine crosses the South Branch Meduxnekeag River as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey (MGS) Surficial Geology of Houlton Quadrangle, Maine, Open-file No. 81-9 (1981) indicates that surficial soils in the vicinity of Hodgdon Bridge consist of glacial till and glacial stream deposits. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. Till deposits typically conform to the bedrock surface, and were deposited directly by the glacial ice. Glacial stream deposits generally consist of sand and gravel, and originated as meltwater stream deposits, during melting of the Late Wisconsinan glacier.

The Bedrock Geologic Map of Maine, MGS, (1985), cite the bedrock at the Hodgdon Stream Bridge site as the Carys Mills Formation and consists of interbedded pelite and limestone and/or dolostone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two test borings. Both borings were terminated with bedrock cores. Test borings BB-HHS-101 and BB-HHS-102 were drilled behind the locations of the existing west and east abutments, respectively. The boring locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. The borings were drilled on September 27, 2007 using the Maine Department of Transportation (MaineDOT) drill rig.

The borings were drilled using cased wash boring and solid stem auger techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is newly equipped with a CME automatic hammer. The hammer was calibrated by MaineDOT in August of 2007 and was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.77 to the raw field N-values. This hammer efficiency factor, 0.77, and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in the two borings using an NQ-2 core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. A MaineDOT Certified Subsurface Investigator logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs, found at the end of this report.

4.0 LABORATORY TESTING

There was no laboratory testing of soil samples.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at test borings BB-HHS-101 and BB-HHS-102 generally consisted of granular fill, reworked glacial till soils, and weathered bedrock, all underlain by metamorphic bedrock. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. A brief summary description of the strata encountered is as follows:

5.1 Fill

A layer of fill was encountered in borings located behind the existing abutments. The encountered fill layer is approximately 12 to 14.6 feet thick. The fill deposit generally consisted of brown, SAND, some fine and coarse gravel, gravelly SAND, and fine to coarse GRAVEL, with minor portions of silt and rock fragments.

SPT N-values in granular fill layers ranged from 12 to 73 blows per foot (bpf) indicating that the fill unit is medium dense to very dense in consistency. Two SPT N-values in the upper fill unit were greater than 50 bpf and were influenced by the coarse aggregate size.

5.2 Reworked Glacial Till

A 3.8-foot thick deposit of native till soil was encountered below the granular fill in BB-HHS-101. The deposit encountered consisted of grey, very dense, weathered bedrock fragments, some sand, trace silt.

5.3 Bedrock

Bedrock at the site was encountered and cored at a depth of 16.9 feet below ground surface (bgs) and Elevation 339.1 feet in boring BB-HHS-101. Bedrock was encountered and cored at depth of 14.6 feet bgs and Elevation 341.5 feet in boring BB-HHS-102.

The bedrock at the site is identified as grey, fine grained, metamorphosed LIMESTONE and calcareous SANDSTONE (SLATE), moderately hard to hard, moderately weathered to fresh, joint set along bedding, dipping at steep to vertical angles, very closely spaced, tight to open, some with silt infilling, highly fractured to massive. Geologists in Maine refer to rocks like the Carys Mills Formation as ‘ribbon lime’ because of the intebedded repetitive bands of light colored sandstone and dark colored limestone. The rock quality designation (RQD) of the bedrock was determined to range from 50 to 85 percent, correlating to a rock quality of poor to good.

The table below summarizes top of bedrock elevations at the proposed abutment locations.

Proposed Substructure	Boring	Station	Depth to Bedrock (feet)	Elevation of Bedrock Surface (feet)
Abutment 1	BB-HHS-101	5+24.1	16.9	339.1
Abutment 2	BB-HHS-102	5+84.3	14.6	341.5

6.0 FOUNDATION ALTERNATIVES AND RECOMMENDATIONS

Prior to the development of the Preliminary Design Report (PDR) for the Hodgdon Stream Bridge, foundation alternatives were provided to the Designer in an internal Geotechnical Design Memorandum, dated January 14, 2008. The following foundations were considered for the replacement bridge substructures and evaluated for practicality, durability and risk in the January 14, 2008 memorandum:

- Full height reinforced concrete abutments founded on new spread footings supported on bedrock or seal concrete founded on bedrock.
- Integral abutments supported on short piles, with piles driven behind the existing abutments, potentially with 1-foot rock sockets. The existing gravity abutments may be partially demolished and the remaining portion left in place as protection for the new pile-supported abutments.
- Concrete stub abutments founded on spread footings bearing within the approach fills. The existing abutments are left in place as protection for 3H:1V riprap slopes supporting the new stub abutments.

All of these foundation types are viable, with varying degrees of risk, as foundation alternatives for this site, however, cantilever-type abutments on spread footing founded directly on bedrock or on seal concrete on bedrock, were selected by the Designer and will be the recommended foundation type in the PDR. This report addresses only that foundation type.

6.1 General - Spread Footings on Bedrock

Full height, cast-in-place abutments supported on spread footings founded on bedrock is the most practical and durable foundation alternative from a geotechnical perspective. The borings encountered bedrock approximately 15 to 17 feet below the bridge approaches at the locations of the two borings. It is therefore considered feasible that cofferdams, seals (if required) and spread footings could be practically and economically constructed to bear on bedrock.

The borings indicate that suitable bedrock with a minimum RQD of 50 percent will be encountered at the bedrock surface, however, the bedrock surface shall be cleared of all loose, fractured and decomposed bedrock. Based on borings conducted at the site and top of bedrock elevation encountered, the bottom of footing or seal elevations are estimated to be approximately Elev. 339.0 feet at the Abutment 1 and approximately Elev. 341.0 at Abutment 2.

6.2 Abutment and Wingwall Design

Abutments and extension wingwalls shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength and service limit states. The design of project abutments and wingwalls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

Failure by sliding shall be investigated. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eights (3/8) of the footing dimensions, in either direction.

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and scour at the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65

Cantilever-type abutments and 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, calculated using Rankine Theory for cantilever-type abutments and wingwalls. See Sheet 4 - Rankine and Coulomb Active Earth Pressure Coefficients, at the end of this report, for guidance in calculating this value. The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG for the abutments and walls if an approach slab is not specified. In the case a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD 3.11.6.2. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) taken from the table below:

Wall Height (feet)	H_{eq}	
	Distance from wall backface to edge of traffic = 0 feet	Distance from wall backface to edge of traffic is >= 1.0 feet
5	5.0	2.0
10	3.5	2.0
>=20	2.0	2.0

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

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

Slopes in front of and sloping down to the wingwalls should be constructed with riprap and not exceed 1.75H:1V.

6.3 Bearing Capacity

Substructure spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 35 ksf. This assumes a bearing resistance factor, $\varphi_{b,}$ for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. The calculated factored bearing resistance is based on excavation of fractured bedrock to a depth where the RQD is at least 50%. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. See Appendix B – Calculations, for supporting documentation.

In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete, which may be taken as 0.3 *f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

6.4 PCMG Retaining Walls

Precast Concrete Modular Gravity (PCMG) walls will retain approach fills. The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The PCMG should be founded on bedrock or compacted granular borrow. The PCMG wall shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to 2.0 feet of soil.

The bearing resistance for the PCMG wall founded on a leveling slab founded on compacted granular fill soils shall be investigated at the strength limit stated using factored loads and a factored bearing resistance of 11 ksf. The stress distribution may be assumed to be a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing service limit state load combinations and for preliminary footing sizing. See Appendix B – Calculations, for supporting documentation.

The bearing resistance for the bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, the PCMG wall units should be designed so that the nominal bearing resistance, in conjunction with the depth of scour determined for the check flood for scour, provide adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated scour depth.

Failure by sliding shall be investigated by the wall subcontractor. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on sand and the nominal sliding resistance of soil within the precast concrete units on sand. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.29 (0.80·tan 20°) at the foundation soil to concrete interfaces, and a maximum frictional coefficient of 0.58 (tan 30°) at foundation soil to soil-infill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Article 11.11.4.2 and Table 10.5.5.2.2-1.

For lowest PCMG unit on soil, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth ($1/4^{th}$) of the footing dimensions, in either direction.

6.5 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limits states. These changes in foundation conditions shall be investigated at wingwalls, abutments and retaining walls.

In general, for scour protection, any footings which are constructed on soil deposits should be embedded at least 2 feet below the design scour depth and armored with 3 feet of riprap for scour protection. Refer to BDG Section 2.3.11 for information regarding scour design.

For scour protection, bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the BDG. Stone riprap shall conform to Item number 703.26 of the Standard Specification and be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to Item number 703.19 of the Standard Specification. Riprap may be placed at the toes of abutments, wingwalls and retaining walls, as required.

6.6 Settlement

The grades of the bridge approaches and side slopes will not be raised in the construction of the proposed bridge, therefore post-construction settlement due to compression of the foundation soils will be negligible. Settlement of the bridge abutments due to elastic settlement of the bedrock is anticipated to occur during construction of the abutments, and is generally assumed to be less than 0.5 inches. Consolidation settlement in the rock mass due to soft, silt-infilled seams in the rock is estimated to be less than 0.2 inches.

6.7 Frost Protection

Abutment and return wing spread footings at the site will be founded on bedrock. Therefore, heave due to frost is not a design issue, and no requirements for minimum depth of embedment are necessary.

PCMG retaining walls are proposed to retain the approach fills. These walls should be founded directly on the compacted granular borrow or bedrock. Foundations placed on compacted granular borrow should be designed with an appropriate embedment for frost protection. According to the BDG, Houlton, Maine has a design freezing index of approximately 2212 F-degree days. An assumed water content of 10% was used for moist, coarse grained soils above the water table. These components correlate to a frost depth of 7.0 feet. Therefore, any foundations placed on soil should be founded a minimum of 7.0 feet below finished exterior grade for frost protection.

6.8 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2, seismic analysis is not required for single-span bridges, regardless of seismic zone, however superstructure connections and bridge seat dimensions shall be satisfied per LRFD 3.10.9 and 4.7.4.4, respectively. The bridge is not classified as functional important. Furthermore, the bridge is not classified as a major structure, since the bridge construction costs will not exceed $10 million. These criteria eliminate the BDG requirement to design the foundations for seismic earth loads.

6.9 Construction Considerations

Construction activities may include internally braced cofferdam construction, earth support system construction and rock excavation.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is made. The bedrock surface shall be cleared of all loose, fractured and decomposed bedrock and loose soils. The final bearing surface shall be solid and unfractured. The bearing surface shall then be washed with high-pressure water and air prior to concrete being placed for the footing. Excavation of fractured and weathered bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Supplemental Specification 105.2.6. It is also recommended that the contractor conduct pre- and post-blast surveys, as well as blast vibration monitoring, at nearby residences in accordance with industry standards at the time of the blast.

Where the bedrock surface slopes toward the stream channel, the bedrock surface shall be stepped to create level benches or excavated to be level overall. Elsewhere, the bedrock surface slope shall be less than 4 horizontal to 1 vertical (4H:1V) or it 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.

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

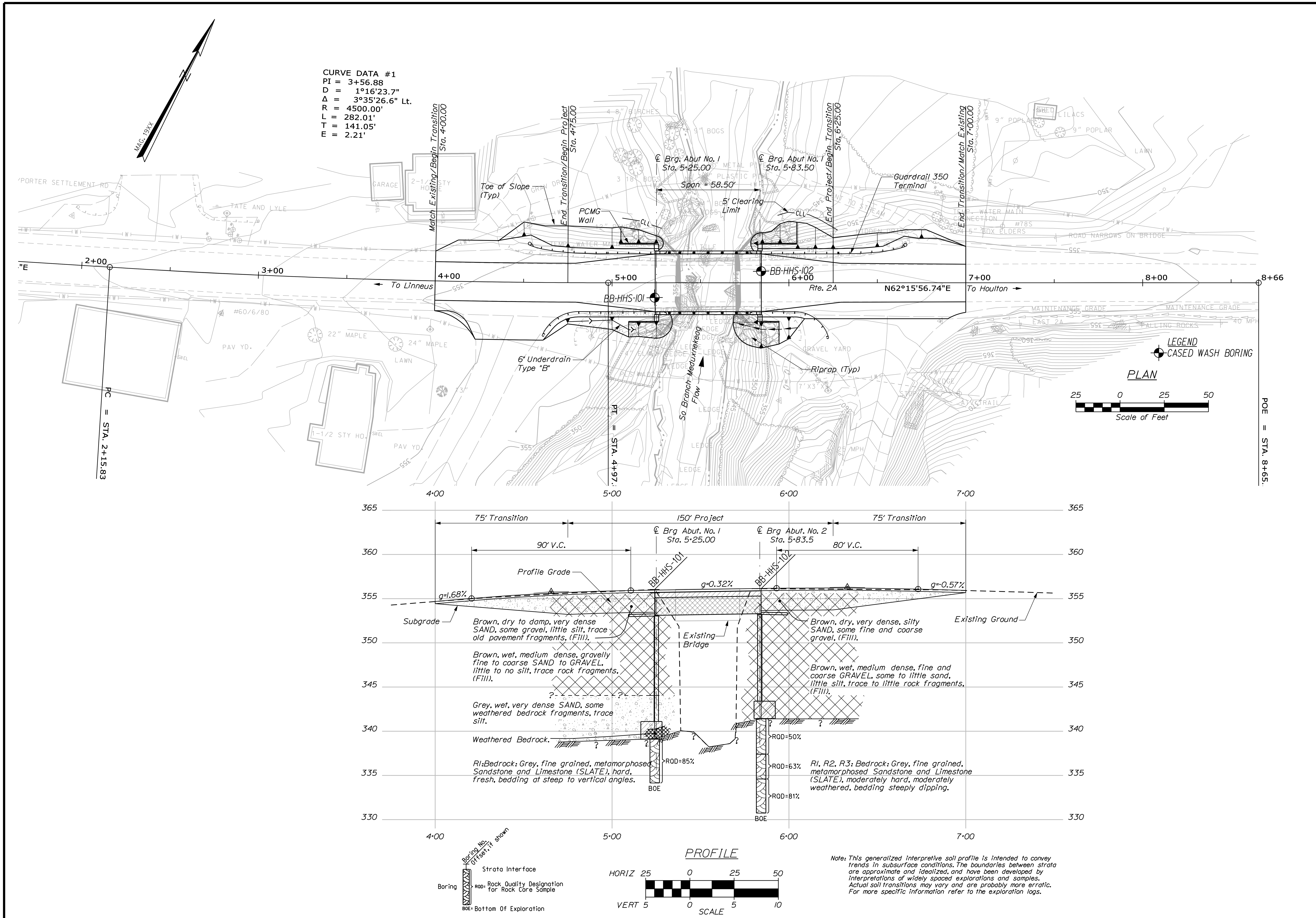
It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

7.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Hodgdon Stream Bridge in Houlton, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete 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.

We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets



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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1562(900)X

BRIDGE PLANS

PIN 15629.00

BRIDGE NO. 3458

		BY	DATE
PROJ. MANAGER	N. BENOIT		
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	JULY 2008
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

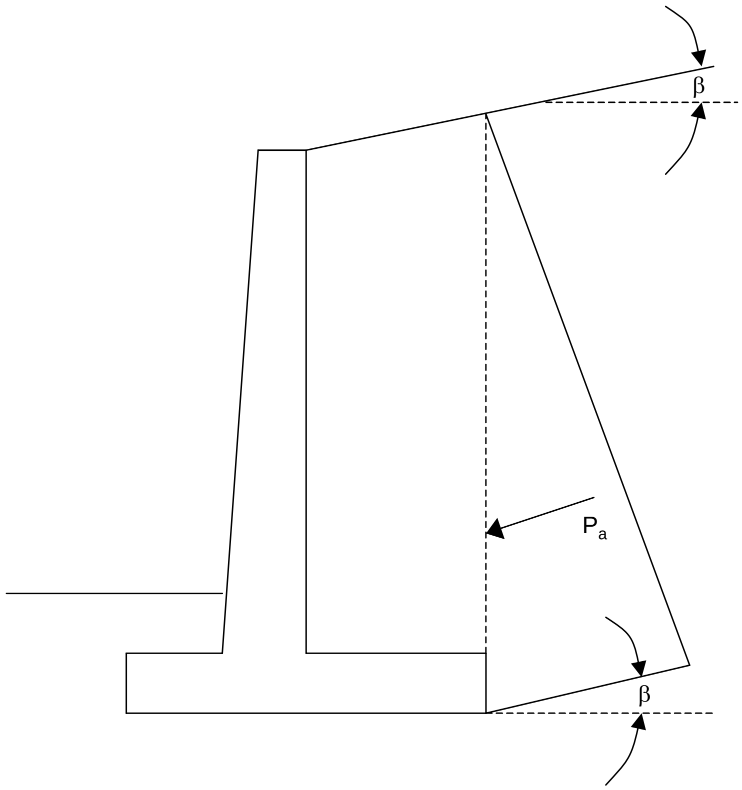
P.E. NUMBER

DATE

HODGDON STREAM BRIDGE
SOUTH BRANCH MEDUXNEKEAG RIVER
HOULTON AROOSTOOK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
6
OF _



For cases where interface friction between the backfill and wall are 0 or not considered, use Rankine.

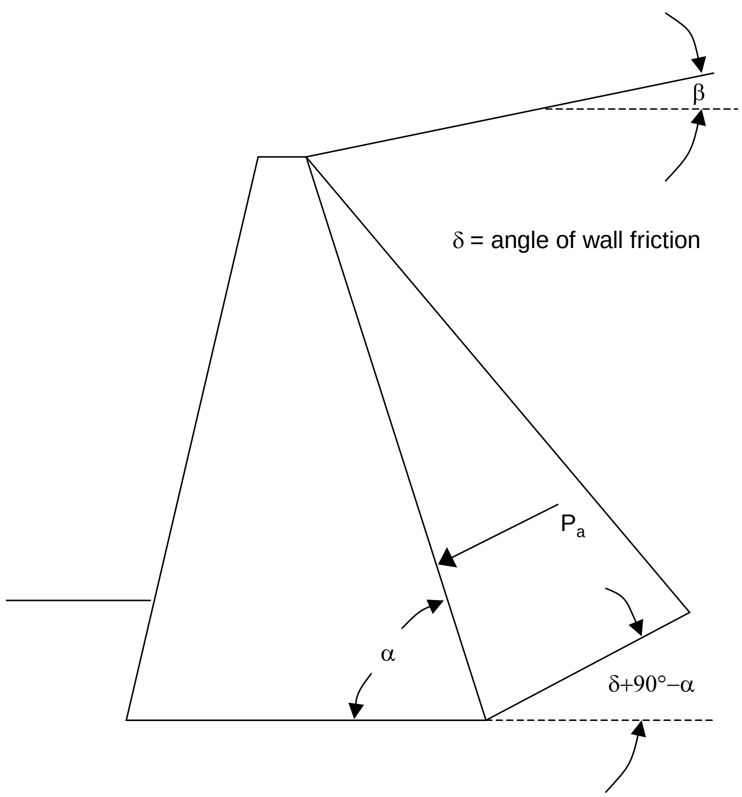
For a horizontal backfill surface, $\beta = 0°$:

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

For a sloped backfill surface, $\beta > 0°$:

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

P_a is oriented at β



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

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

P_a is oriented at $\delta + 90° - \alpha$

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hodgden Steam Bridge #3458 over S. Branch Meduxnekeag River
Location: Houlton, Maine

Boring No.: BB-HHS-101
PIN: 15629.00

Driller:	MaineDOT	**Elevation (ft.)**	356.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/27/07; 12:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+24.1, 8.4 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.77	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/18	0.80 - 2.80	35/35/22/15	57	73	SSA	355.60	PAVEMENT. —0.40	
									Brown, dry to damp, very dense, fine to coarse SAND, some fine rounded and coarse angulat gravel, little silt, trace old pavement fragments. (Fill)	
5	2D	24/8	5.00 - 7.00	7/5/4/4	9	12	10		Brown, wet, medium dense, gravelly fine to coarse SAND, little silt. (Fill)	
							23			
							33			
							50			
							57			
10	3D	24/9	10.00 - 12.00	8/4/10/11	14	18	28		Brown, wet, medium dense, coarse angular and fine rounded GRAVEL, trace rock fragments. (Fill)	
							31			
							43	344.00	—12.00	
							48			
							40			
15	4D	9.6/9.6	15.00 - 15.80	13/50(3.6")	---		65	340.20	Grey, wet, very dense, weathered bedrock fragments, some fine to coarse sand, trace silt. (Reworked Native Soil). —15.80	
	R1	60/57	16.90 - 21.90	RQD = 85%			a88 NQ-2	339.10	Weathered Bedrock. a88 blows for 10.8". —16.90	
									Top of intact Bedrock at Elev. 339.1'. R1: Bedrock: Grey, fine-grained, metamophosed LIMESTONE and calcareous SANDSTONE (Slate/Ribbon Lime), hard, fresh, joint set along bedding at steep to vertical angles, tight, unweathered; slightly fractured except for upper 6" which is very highly fractured. Carys Mills Formation R1:Core Times (min:sec) 16.9-17.9' (3:53) 17.9-18.9' (4:36) 18.9-19.9' (4:33) 19.9-20.9' (4:57) 20.9-21.9' (5:49) 95% Recovery No water return at 300 psi	
20										
								334.10	—21.90	
									Bottom of Exploration at 21.90 feet below ground surface.	
25										

Remarks:

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

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

Boring No.: BB-HHS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hodgden Steam Bridge #3458 over S. Branch Meduxnekeag River
Location: Houlton, Maine

Boring No.: BB-HHS-102
PIN: 15629.00

Driller:	MaineDOT	**Elevation (ft.)**	356.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/27/07; 08:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	5+84.3, 6.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15	0.60 - 2.60	29/25/23/13	48	62	SSA	355.60		PAVEMENT. 0.50 Brown, dry, very dense, silty fine to coarse SAND, some fine rounded and coarse angular gravel. (Fill).	
5	2D	24/12	5.00 - 7.00	6/6/10/14	16	21	4			Brown, wet, medium dense, angular fine and coarse GRAVEL, fine to coarse SAND, little silt, little broken rock fragments. (Fill)	
							20				
							17				
							40				
							58				
10	3D	24/8	10.00 - 12.00	8/12/7/7	19	24	20			Brown, wet, medium dense, fine to coarse angular GRAVEL, little sand, little silt, trace rock fragments. (Fill)	
							25				
							22				
							16			a50 blows for 7.8".	
15	R1	48/43	14.60 - 18.60	RQD = 50%			a50 NQ-2	341.50		14.60 Top of Bedrock at Elev. 341.5'. R1: Bedrock: Grey, fine grained, metamorphosed calcareous SANDSTONE and LIMESTONE (Slate/"Ribbon Lime"), moderately hard, moderately weathered, joint set along bedding, steeply dipping, very closely spaced ("slatey"), tight, occasional white vein; moderately fractured overall. Carys Mills Formation. R1:Core Times (min:sec) 14.6-15.6' (6:36) 15.6-16.6' (3:28) 16.6-17.6' (3:08) 17.6-18.6' (4:02) 90% Recovery Core Blocked	
20	R2	34/33.6	18.60 - 21.43	RQD = 63%				337.50		18.60 R2: Bedrock: Same as R1, except very highly fractured in first 7" of core run, open, weathered silt-infilled joint at 2'0" to 2'9" at steep angle. R2:Core Times (min:sec) 18.6-19.6' (6:36) 19.6-20.6' (4:13) 20.6-21.4' (4:18) 100% Recovery Core Blocked	
25	R3	48/42	21.40 - 25.40	RQD = 81%				334.70		21.40 R3:Bedrock: Grey, fine grained, metamorphosed calcareous	

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Hodgden Steam Bridge #3458 over S. Branch Meduxnekeag River	**Boring No.:**	BB-HHS-102		
Location:	Houlton, Maine	**PIN:**	15629.00		
Driller:	MaineDOT	**Elevation (ft.)**	356.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/27/07; 08:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	5+84.3, 6.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								330.70		SANDSTONE and LIMESTONE, hard, very slightly weathered to fresh, joint set along foliation, steep to vertical tight, very closely spaced; slightly fractured to massive overall. Carys Mills Formation. R3:Core Times (min:sec) 21.4-22.4' (3:26) 22.4-23.4' (5:11) 23.4-24.4' (5:37) 24.4-25.4' (5:39) 85% Recovery No water return at 300 psi 25.40 **Bottom of Exploration at 25.40 feet below ground surface.**	
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.

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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

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. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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 shear strength as indicated.

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

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*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)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, 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, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
-spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
-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: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

PIN
Bridge Name / Town
Boring Number
Sample Number
Sample Depth
Blow Counts
Sample Recovery
Date
Personnel Initials

Appendix B

Calculations

Frost Protection

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

From Design Freezing Index Map:
Houlton
DFI = 2212 degree-days

Case I - Soils at elevation of possible footings of are gravel, little sand - assume WC=20%

Interpolate between frost depth of 82.6 inches at 2200 DFI and 84.5 inches at 2300 DFI

Result:

Depth of Frost Penetration =

$$d := \frac{84.5 - 82.6}{100} \cdot 12 \cdot in + 82.6 \cdot in$$

$$d = 6.902 \cdot ft$$

Recommend an embedment depth of 7 feet for foundations constructed on compacted fill soils

Abutment and Wingwall Active Earth Pressure

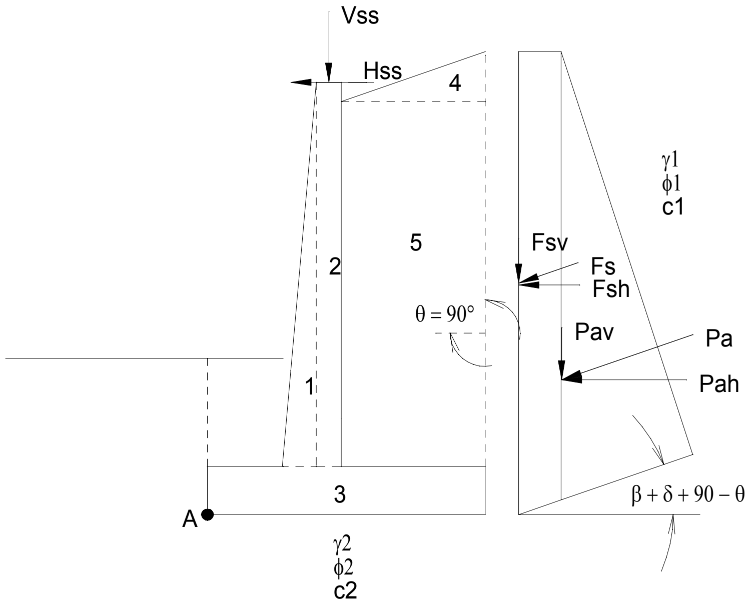
Backfill engineering strength parameters

Soil Type 4 Properties from Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot \text{pcf}$

Internal friction angle $\phi_1 := 32 \cdot \text{deg}$

Cohesion $c_1 := 0 \cdot \text{psf}$



Active Earth Pressure - Rankine Theory

Either Rankine or Coulomb may be used for **long heeled** cantilever walls, where the failure surface is uninterupted by the top of the wall stem. In general, use Rankine though. The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot \text{deg} - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.307$$

- For a sloped backfill

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot \text{deg}$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.307$$

- Pa is oriented at an angle of β to the vertical plane

Coulomb Theory

In general, for cases where the back face of the wall interferes with the development of a full sliding surface in the backfill, as assumed by Rankine Theory, use Coulomb.

- Coulomb theory applies for gravity, semigravity and prefab modular walls with steep back faces
- Coulomb theory also applies to concrete cantilever walls with short heels where the sliding surface in restricted by the top of wall - the wedge of soil does not move.
- Interface friction is considered in Coulomb.

Angle of back face of wall to the horizontal, θ :

$$\theta := 90 \cdot \text{deg}$$

Friction angle between fill and wall, δ :

Per LRFD Table 3.11.5.3-1, for "Clean sand, silty sand-gravel mixture, single-size hard rock fill against Formed or precast concrete" δ = 17 to 22 degrees; select 20 degrees.

$\delta := 20 \cdot \text{deg}$ for a gravity shaped wall where the interface friction is between soil and concrete

to $\delta := 24 \cdot \text{deg}$ per BDG Table 3-3

Per LRFD Figure C3.11.5.3-1, for a cantilever wall where the sliding surface is a plane from the footing heel to the top of the wall, δ=1/3 to 2/3 Φ

$$\delta := \frac{2}{3} \cdot \phi_1$$

$$\delta = 21.333 \cdot \text{deg}$$

(If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb)

$$K_{ac} := \frac{\sin(\theta + \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2} \qquad K_{ac} = 0.275$$

Orientation of Coulomb Pa

- In the case of gravity shaped walls and prefab walls, Pa is oriented δ degrees up from a perpendicular line to the backface.

- In the case of short heeled cantilever walls where the top of the wall interferes with the failure surface, Pa is oriented at an angle of $\phi/3$ to $2/3*\phi$ to the normal of a vertical line extending up from the heel of the wall

Assumptions:

1. Base of footing founded with 6 ft embedment for frost (conservative, 7 feet is recommended).

2. Assumed parameters for compacted granular backfill
saturated unit weight = 130 pcf
dry unit weight = 125 pcf
internal friction angle of 32 degree
undrained shear strength (c) 0 psf

3. Method used: Terzaghi, use strip equations since L>B

Foundation soil values

Available References:

ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
ϕ, SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
ϕ and γ correlations to soil description and N values, Bowles 1977 Table 3-4
ϕ: Bowles (4th Ed) Table 2-6
γ sat : Holtz, Kovacs, Table 2-1 1981

Footing Width and Depth

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \\ 20 \end{pmatrix} \cdot ft$$

$D_f := 6.0 \cdot ft$ $\qquad D_w := 6 \cdot ft$ $\qquad \gamma_w := 62.4 \cdot pcf$

Foundation Soil (Granular Fill)

$\gamma 1_{sat} := 130 \cdot pcf$

$\gamma 1_d := 125 \cdot pcf$

$\phi := 32 \cdot deg$

$c_1 := 0 \cdot psf$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Based on NavFac DM 7.2 pg 142-143 Table 1 - "Presumptive Values of Allowable Bearing Capacity Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Allowable Bearing Pressure (tons per sq. foot):	Recommended Value:
Coarse to medium sand, little gravel	Very compact	4 to 6	4 tsf
	Medium to compact	2 to 4	3 tsf
	Loose	1 to 3	1.5 tsf

Recommend 3 tsf or 6 ksf, to contol settlements for Service Limit State analyses and for preliminary footing sizing.

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

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

$s_\gamma := 1.0$ $\qquad$ $s_c := 1.0$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223)

$N_c := 35.47$ $\qquad$ $N_q := 23.2$ $\qquad$ $N_\gamma := 22$

Nominal Bearing Resistance per Terzaghi equation (Bowles, Table 4-1, 5th Ed., pg 220)

$$q := D_w \cdot \gamma 1_d + \left(D_f - D_w\right) \cdot \left(\gamma 1_{sat} - \gamma_w\right) \qquad q = 0.75 \cdot ksf$$

$$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot \left(\gamma 1_{sat} - \gamma_w\right) \cdot B \cdot N_\gamma \cdot s_\gamma \qquad q_n = \begin{pmatrix} 21.1 \\ 23.3 \\ 24.8 \\ 26.3 \\ 28.6 \\ 32.3 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance for strength limit states

$q_r := q_n \cdot 0.45$

$$q_r = \begin{pmatrix} 9.5 \\ 10.5 \\ 11.2 \\ 11.8 \\ 12.8 \\ 14.5 \end{pmatrix} \cdot ksf$$

Recommend a limiting factored bearing resistance of 11 ksf for footings 12 feet wide or less, on compacted granular fill.

Bearing Capacity - Abutment 1 and 2 Spread Footing Foundations

Method 1

Method: LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings, 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".

Description of Bearing Material:

Abutment 1: Boring BB-HHS-101, 5-ft core, Metamorphosed LIMESTONE, SANDSTONE, (Slate/Ribbon Lime), hard, fresh, except for upper 6 inches which is highly fractured. Assume upper 6 inches is removed. RQD=85%.

Abutment 2: Boring BB-HHS-102, 11 feet of rock cored: Metamorphosed LIMESTONE and SANDSTONE (Slate/Ribbon Lime). RQDs are 50%, 63%, 81%. Moderately hard, moderately weathered, moderately fractured overall, with silt infilled joint in R2. Use RQD of 50% for design.

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

Use a factored bearing resistance of 20 ksf for service limit state analysis - and for preliminary sizing of the footing.

Method 2

Method: *AASHTO Standard Specifications - 17th Edition, 2002*

Section 4.4.8.1.1 - Competent Rock

Figure 4.4.8.1.1.A - for footings supported on competent rock.

Averaged RQD of rock is 50%

Allowable contact stress 60 tsf (120 ksf)

Method 3

AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.2. Footings on Broken or Jointed Rock

Table 4.4.8.1.2.A - for footings supported on jointed rock.

a. estimated RMR, Rock Mass Rating,	Fair. RQD Range is 50-75
b. Rock Category per 4.4.8.1.2B	B, Slate
c. Unconfined compressive strength, Co	10,000 psi estimated (21,000 - 30,000 psi)
d. Nms, per Table 4.4.8.1.2A	Table states to use Nms=.056
e. Q ult	Nms x Co

Nominal Bearing Resistance

$Q_{nom} := 0.056 \cdot 10000 \cdot psi$ $\qquad Q_{nom} = 80.64 \cdot ksf$

Factored Bearing Resistance

$\phi := 0.45$

$Q_{factored} := Q_{nom} \cdot \phi$

$\boxed{Q_{factored} = 36.288 \cdot ksf}$

Recommend a factored bearing resistance 35 ksf for the Strength Limit State Analysis.

Assume an unfactored Service Load Combination of a maximum of 20 ksf - perform a settlement analysis (follows).

Settlement Analysis of Footings on Rock, LRFD 10.6.2.4.4

Per LRFD, 10.6.2.4.4, elastic settlements may generally be assumed to be less than 0.5 inches. However, the magnitude of consolidation settlement in rock masses containing soft seams should be estimated by applying procedures specified in Article 10.6.2.4.3.

Open silt infilled seam found in R-2 of BB-HHS-102, at a steep angle, between 2 ft and 2 ft 9 inches. Assume 9 inches thick.

Silt properties - assume OCR => 1.0 e_o=1.00 and C_c = 0.30 and Cr = 0.03. Assume preconsolidated since the silt seam is near the surface of the bedrock

$$e_o := 1.0$$

$$C_r := 0.030$$

Depth of seam is 48 inches (R1) + 2.0 feet = 6 feet below footing with applied load of 20 ksf (Unfactored Service Load Combination.

Per LRFD Figure 10.6.2.4.1-1, Boussinesq Vertical Stress Contours

Assume Footing Width, B = 15 feet

Depth of interest is approximately 0.4B

Stress is approximately $0.75q_o$

$$q_o := 20 \cdot \text{ksf}$$

$$\Delta\sigma_v := 0.75 \cdot q_o$$

$$\Delta\sigma_v = 15 \cdot \text{ksf}$$

Existing overburden stress

Profile is 14.6 feet of granular fill soils, assume water table at 5 ft bgs, and 6 feet of bedrock

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

$$\gamma_{rock} := 150 \cdot \text{pcf}$$

$$\sigma_v := \left(\gamma_{fill} \cdot 5 \cdot \text{ft}\right) + \left(\gamma_{fill} - 62.4 \cdot \text{pcf}\right) \cdot 9.6 \cdot \text{ft} + \left(\gamma_{rock} - 62.4 \cdot \text{pcf}\right) \cdot 6 \cdot \text{ft}$$

$$\sigma_v = 1.752 \cdot \text{ksf}$$

Calculate Settlement

$$\Delta H := 9 \cdot in \cdot \left(\frac{C_r}{1 + e_o}\right) \log\left(\frac{\sigma_v + \Delta\sigma_v}{\sigma_v}\right) \qquad \Delta H = 0.132 \cdot in$$

Settlment of up to 0.2 inches possible due to consolidation settlement in a soft seam