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

# GEOTECHNICAL DESIGN REPORT

*For the Replacement of:*

**ORLAND RIVER BRIDGE**
**ROUTE 175 OVER ORLAND RIVER**
**ORLAND, MAINE**


*Prepared by:*

Laura Krusinski, P.E.
Senior Geotechnical Engineer

*Reviewed by:*

Michael J. Moreau, P.E.
Geotechnical Engineer



Hancock County
PIN 15103.00

Soils Report No. 2008-17
Bridge No. 2632

Fed No. STP-1510(300)X
November 24, 2008

**Table of Contents**

## GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Orland River Bridge which carries Route 175 over Orland River, in Orland, Maine. The proposed replacement bridge will be a 70-foot single span bridge. The superstructure curb-to-curb width will be increased from 24 feet to 28 feet and the new alignment will be located approximately on the existing alignment.

Preliminary foundation alternatives were provided by the geotechnical team member in an internal Geotechnical Design Memorandum, dated September 24, 2008. Preliminary geotechnical studies identified the more conventional and effective foundation type for the site to be spread footings founded directly on bedrock or seal concrete founded on bedrock. A less expensive but less effective option is abutments founded on short piles (10 feet and less). The following design recommendations for both pile supported abutments and abutments founded on spread footings on bedrock are discussed in detail in this report.

**Cantilever Abutments and Wingwalls** – Cantilever 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, with 2008 Interims (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, $\varphi_\tau$, 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.

The Designer may assume backfill soil properties of $\phi = 32°$ and $\gamma = 125$ pcf. Earth loads shall be calculated using an active earth pressure coefficient, $K_a$, of 0.31, calculated using Rankine Theory for cantilever wingwalls. 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. If a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD 3.11.6.5.

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

## GEOTECHNICAL DESIGN SUMMARY – CONTINUED

**Integral Abutments on Driven H-piles.** In consideration of (a) the consequences of scour and pile exposure, (b) the need to limit pile tip movement, and (c) obtaining pile behavior associated plastic stress redistribution and inelastic rotation in the pile, a minimum pile length of 8 to 10 feet is recommended. Bedrock is relatively shallow at the site and we estimated pile lengths of 5.3 to 9.7 feet for integral structures at Abutments 1 and 2, respectively. Abutments founded on H-piles driven behind 1.75H:1V slopes is considered only feasible if rock sockets are drilled at Abutment 1. A pile supported foundation at either abutment is only feasible if there is no likelihood of removal of the dam downstream.

Any substructure design that includes re-use of old substructures to protect pile groups should include repairing and patching areas of concrete that are spalling or cracked, repairing undermined portions of the abutments at the streambed, and repointing or reseting, any dry laid granite block walls as required, to ensure serviceability.

**Integral Pile Design.** Abutments founded on H-piles driven behind 1.75H:1V slopes is considered only feasible at Abutment 2. The piles should be end bearing and driven to the required resistance on, or within, bedrock. Piles may be HP 12x53, 14x73, 14x89, or 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel. Driven piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption.

The Structural Designer shall design H-piles for all relevant strength, service and extreme limit state load groups. The structural resistance check should include checking axial, lateral and flexural resistance. Our analysis indicates the factored axial drivability pile resistances control and those values are:

| Pile Section | Strength Limit State,<br>Factored Axial Pile Drivablity<br>Resistance (kips) |
|---|---|
| HP 12 x 53 | 197 |
| HP 14 x 73 | 270 |
| HP 14 x 89 | 329 |
| HP 14 x 117 | 416 |

The maximum factored axial pile load should not exceed the calculated factored drivability pile resistances provided above.

The top of the piles should be checked for resistance against combined axial load and flexure, per LRFD Article 6.15.2. As integral H-piles would be short and not achieve fixity, the resistance of the piles should be analyzed for combined axial compression and flexure resistance and evaluated for structural compliance with the interaction equation.

## GEOTECHNICAL DESIGN SUMMARY – CONTINUED

For strength limit state load combinations, a resistance factor of 0.70 for axial resistance ($\phi_c$) and 1.0 for flexural resistance ($\phi_f$) should be applied to the combined axial and flexural resistance of the pile in the interaction equation.

**Driven Pile Quality Control.** The contractor is required to perform a wave equation analysis of the proposed pile-hammer system. The first pile driven at any substructure should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. The ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, $\varphi_{dyn}$, of 0.52. The maximum factored pile load should be shown on the plans.

**Integral Stub Abutment Design.** Integral abutment sections shall be designed for all relevant strength, service and extreme limit states specified in LRFD Articles 3.4.1 and 11.5.5. Integral abutment sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure. The Rankine passive case is recommended. Wing wall sections that are integral with the abutment, should also be designed to withstand a maximum earth pressure equal to the Rankine passive earth pressure state. All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

**PCMG Retaining Walls** - Precast Concrete Modular Gravity (PCMG) walls will retain approach fills. A PCMG wall along the southeast approach will be constructed in front of the existing wingwalls. 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 and embedded for frost protection.

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 state using factored loads and a factored bearing resistance of 11 ksf. Footings with a width 8.0 feet or less should be assessed for a factored bearing resistance of 9 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.

**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.

If pile supported abutments are used, the stream velocity should be low and there should be low potential for dam removal, scour action, wave action, storm surge and ice damage. This is to maintain the integrity of the bridge approach fills and riprap abutment slopes, which provide the only lateral support to the pile groups.

## GEOTECHNICAL DESIGN SUMMARY – CONTINUED

**Existing Abutments.** Any substructure design that includes re-use of old substructures should include repairing and patching areas of concrete that are spalling or cracked. The scope of work should also include repairing any undermined portions of the abutments at the streambed, and repointing or reseting, any dry laid granite block walls as required, to ensure serviceability.

**Settlement** - The grades of bridge approaches and side slopes will be raised 2.4 inches, 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 or piles, which is assumed to occur during construction and be less than 0.5 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 5.0 feet below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

**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 the seismic requirements in LRFD 3.10.9 and 4.7.4.4., respectively.

**Construction Considerations** – Internally braced cofferdams and temporary lateral earth support systems may 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. The contractor should conduct pre- and post-blast surveys, as well as blast vibration monitoring.

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 placed for the abutment and wingwall foundations.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Groundwater and surface water should be controlled by pumping from sumps.

If integral abutment piles foundations are used, they will require removal of existing return wingwalls and the associated 3 foot thick footings and up to 8 feet wide. The pile foundation area would require placement and compaction of granular fill up to the abutment subgrade level.

# 1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Orland River Bridge which carries State Route 175 over the Orland River, in Orland, Maine. This report presents the soils information obtained at the site during the subsurface investigation, foundation recommendations and geotechnical design parameters for replacement bridge foundations.

Orland River Bridge was built in approximately 1932 and is a 55-foot single span, concrete T-beam superstructure, supported on full-height, concrete gravity abutments. The substructure concrete is unreinforced with the exception of K-bars at the abutment wingwall junctions and the bridge seat. The wingwalls are constructed at 90 degrees to the abutments, and consist of unreinforced concrete gravity walls, ranging from 12 to 58 feet in length. The downstream westerly wingwall includes a dry laid, hewn granite block wall. The pre-existing bridge was a 2-span bridge with abutments and center pier constructed of dry laid, split granite blocks. There is a dam approximately 500 feet downstream from the Orland River Bridge.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports indicate minor substructure distress in areas in the form of concrete spall and minor scaling. There is minor abrasion below the water line. No undermining or scour is noted. 2007 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 22.3.

It should be noted that actual abutment geometries may vary from those dimensions shown on the State Highway Commission Bridge Commission Plans, dated 1931 and 1932.

This project is a bridge replacement project. This scope was supported by the Scope Review Team (SRT) Final Report.

Preliminary foundation alternatives were provided by the geotechnical team member in an internal Geotechnical Design Memorandum, dated September 24, 2008. Subsequent preliminary engineering assessments by CLD Engineers and the MaineDOT Bridge Program resulted in the recommendation for a bridge replacement project with foundations consisting of either:

- A pile supported integral abutment (Abutment 2) and a cantilever-type abutment (Abutment 1) on spread footings founded directly on sound bedrock or seal concrete founded on bedrock, or,
- A pile supported integral abutment (Abutment 2) and a semi-integral abutment (Abutment 1) founded on a pile group consisting of 3.5-foot long driven piles.

## 2.0 GEOLOGIC SETTING

Orland River Bridge on State Route 175 in Orland, Maine crosses the Orland River as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey "Surficial Geology of Orland Quadrangle, Maine, Open-file No. 82-21" (1982) indicates that the project site is located at the contact boundary of two surficial soil units, a glacial marine deposit and glacial till. The glacial marine geologic unit consists of silt, clay and sand. The unit is commonly a clayey silt, but sand is very abundant at the surface in some places. The unit may include small areas of till, sand and gravel that are not completely covered by the marine sediment. The glacial marine unit is composed of sediment that washed out of the Late Wisconsinan glacier and accumulated on the ocean floor during the most recent glacial period, when the relative sea level was higher than present and seawater flooded coastal and interior Maine. 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.

According to the Bedrock Geologic Map of Maine, Maine Geologic Survey, 1985, the bedrock at the project site is the Penobscot Formation and consists of sulfidic/carbonaceous pelite.

## 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-OOR-101 and BB-OOR-102 were drilled 30 feet and 19 feet behind 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 3, 2008 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. The MaineDOT Geotechnical Team Member 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

Laboratory testing for samples obtained in the borings consisted of three (3) standard grain size analyses, with natural water contents. The results of soil laboratory tests are included as Appendix B - Laboratory Data, at the end of this report. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

## 5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at test borings BB-OOR-101 and BB-OOR-102 generally consisted of fill material, ranging from granular soils to reworked native silt soils, boulders, and wood, 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. The boring logs are provided at Appendix A –Boring Logs. A brief summary description of the strata encountered follows:

### 5.1 Fill

A layer of fill was encountered in borings located behind the existing abutments. The encountered fill layer is approximately 16.8 to 18.2 feet thick. The upper fill subunit generally consisted of brown, SAND, some silt, trace gravel; gravelly SAND, trace silt; and brown, sandy SILT, little gravel, with minor portions of organics and wood fragments. The frequency of boulders, timber construction materials and soft, native silt soils increased with depth. A 2.8 foot thick layer of wood was encountered at a depth of approximately 15 feet in BB-OOR-102. Boring BB-OOR-102 also encountered a 3-foot thick concrete footing supporting the downstream, easterly wingwall at a depth of 18.2 feet. The concrete footing was cored and found to be directly founded on bedrock.

SPT N-values in fill unit ranged from 3 blows per foot (bpf) to greater than 50 bpf, indicating that the fill unit is very loose to very dense in consistency.

### 5.2 Bedrock

Bedrock at the site was encountered and cored at a depth of 16.8 feet bgs and Elevation -0.8 feet in boring BB-OOR-101. Bedrock was encountered and cored at depth of 21.2 feet bgs and Elevation 1.3 feet in boring BB-OOR-102.

The bedrock at the site is identified as grey, fine grained, metasedimentary PHYLLITE, moderately hard to hard, moderately weathered to fresh, joint set along foliation, dipping at steep to vertical to chaotic angles, very closely spaced, tight to very open and silt infilled, slightly fractured to massive. The rock quality designation (RQD) of the bedrock was determined to range from 22 to 100 percent, correlating to a rock mass quality of very poor to excellent.

Table 1 summarizes top of bedrock elevations at the exploration locations.

| Proposed Substructure | Boring | Station | Depth to Bedrock (feet) | Elevation of Bedrock Surface (feet) |
|---|---|---|---|---|
| Abutment 1 | BB-OOR-101 | 4+32.6 | 16.8 | -0.8 |
| Abutment 2 | BB-OOR-102 | 5+37.3 | 21.2 | 1.3 |

**Table 1.** Elevation of Bedrock Surface at Exploration Locations

## 6.0 FOUNDATION ALTERNATIVES

Prior to the development of the Preliminary Design Report (PDR) for Orland River Bridge, foundation alternatives were provided to the Designer in an internal Geotechnical Design Memorandum, dated September 24, 2008. The following foundations were considered for the replacement bridge substructures and evaluated for practicality and effectiveness in the Memorandum:

- Full height, cantilever 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, with 3-foot rock sockets at Abutment 1. The existing gravity abutments may be partially demolished and the remaining portion left in place as protection for the new pile-supported abutments.
- A mix of the two foundation alternatives described above: a pile-supported integral abutment and a semi-integral full height cantilever concrete abutment founded on spread footings supported on bedrock or seal concrete on bedrock.

All of these foundation types are viable, with varying degrees of effectiveness and cost, however, cantilever-type abutments on spread footing founded directly on bedrock or on seal concrete on bedrock, are recommended. In light that all foundation alternatives described above are still under consideration, this report addresses two foundation types: pile supported integral abutments and full height, cantilever-type abutments founded spread footings supported by bedrock.

# 7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

This section provides geotechnical design recommendations for two foundation alternatives: pile supported integral abutments and full height, cantilever-type abutments founded spread footings directly on bedrock.

## 7.1 Abutments and Wingwalls Founded on Spread Footings on Bedrock

### 7.1.1 General - Spread Footings on Bedrock

Full height, cantilever abutments supported on spread footings founded on bedrock is the most practical and effective foundation alternative from a geotechnical perspective. The borings encountered bedrock approximately 17 to 22 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 near 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. -0.8 feet at the Abutment 1 and approximately Elev. 1.3 feet at Abutment 2.

### 7.1.2 Abutment and Wingwall Design

Abutments and 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.

Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the design flood can support the unfactored Strength Limit States loads with a resistance factor of 1.0. The unfactored Strength Limit State loads shall include any debris loads occurring during the flood event.

Failure by sliding shall be investigated. A sliding resistance factor, $\varphi_\tau$, 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 of cast-in-place concrete on bedrock shall assume a maximum frictional coefficient of 0.70 at the concrete-bedrock 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 movement resulting from scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, $\varphi$, 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, $K_a$, of 0.31, calculated using Rankine Theory for cantilever-type abutments and wingwalls. Sheet 4 - Rankine and Coulomb Active Earth Pressure Coefficients, at the end of this report, illustrates the calculation of earth pressure coefficients. The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi$ = 32 degrees, $\gamma$ = 125 pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG for the abutments and walls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads is permitted per LRFD 3.11.6.5. The live load surcharge on walls 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 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ($H_{eq}$) taken from Table 2 below:

| Abutment Height (feet) | $H_{eq}$ (feet) |
|---|---|
| 5 | 4.0 |
| 10 | 3.0 |
| >=20 | 2.0 |

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

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.

### 7.1.3 Bearing Resistance

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 for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. See Appendix C – 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.

## 7.2 Integral Abutment Founded on Driven H-piles

For an 80-foot span integral structure, we anticipate the combined New England Bulb Tee (NEBT) girder and deck depth to be approximately 5.5 feet, and abutment breastwall height of approximately 6 feet. This implies a depth of 11.5 feet may be required to accommodate a NEBT, 8-inch deck, and abutment breastwall. Bedrock was encountered at a depth of approximately 17 feet bgs below the west bridge approach and approximately 21 feet bgs below the east bridge approach. This results in estimated pile lengths of 5.3 to 9.7 feet at Abutments 1 and 2, respectively. This data is summarized in Table 3.

| Location/ Boring | Depth to Bedrock From Ground Surface (feet) | Top of Bedrock Elevation (feet) | Estimated Integral Pile Lengths (feet) |
|---|---|---|---|
| Abutment 1 BB-OOR-101 | 16.8 | -0.8 | 5.3 |
| Abutment 2 BB-OOR-102 | 21.2 | 1.3 | 9.7 |

**Table 3.** Estimate Pile Lengths when Piles installed to Bedrock Surface

Based on the data presented in Table 3, integral abutments founded on H-piles driven behind 1.75H:1V slopes is considered only feasible if rock sockets are drilled at Abutment 1.

The MaineDOT and the University of Maine (UMO) have investigated the performance of integral abutment bridges at sites with shallow bedrock and have monitored the instrumented Nash Stream Bridge in Coplin Plantation, Maine. Preliminary evaluation of the field data from the research study indicate that integral abutment bridges with 'short' steel piles (defined as piles less than 13 feet) may not develop fixity but perform adequately and do not experience stresses larger than those seen by longer piles. The shortest pile instrumented by the researchers was an H-pile embedded in 14 feet of soil.

To accommodate integral abutment piles at the Orland River Bridge site, the following design features are recommended:

- In consideration of (a) the consequences of scour and pile exposure, (b) the need to limit pile tip movement, and (c) obtaining pile behavior associated plastic stress redistribution and inelastic rotation in the pile, a minimum pile length of 8 to 10 feet is recommended. This recommendation is based on finite element analyses and limited field data from the UMO study. Due to the shallow depth of bedrock at the Orland River Bridge at Abutment 1, piles should be installed in bedrock sockets of approximately 3 feet to provide the minimum 8-foot pile length recommended. If a fixed condition at the pile tip is desired, the bottom 6-inches of the rock sockets should be tremie-filled with concrete. However, the UMO research indicates some rotation at the pile tip is acceptable.

- Short piles supporting integral abutments should be designed in accordance AASHTO LRFD criteria and checked for pile tip movement as described in the design example found in Appendix B of Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock – Phase I", and Chapter 5 of that report.

- Since the abutment piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribe in LRFD Articles 6.9.2.2 and 6.15.2. An L-Pile analysis is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, moments and pile head displacements applied. Achievement of an assumed pinned condition at the pile tip should be also confirmed with an L-Pile analysis. As the proposed piles for this project will be short and will not achieve fixity, the resistance for the piles should be determined for compliance with the interaction equation.

- Driven piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption.

- The stream velocity should be low and there should be low potential for dam removal, scour action, wave action, storm surge and ice damage. This is to maintain the integrity of the bridge approach fills and riprap abutment slopes, which provide the only lateral support to pile groups.

- The existing abutments may be left in place as protection for the pile supported abutments with 1.75H:1V slopes constructed to the tops of the partially demolished, existing abutments. Slopes should be protected with riprap over an erosion control geotextile. The existing return wingwalls are on spread footings up to 8 feet wide, and will obstruct embedment of piles. Removal of the existing wingwalls and 3 foot thick footings would be necessary. This would also necessitate placement and compaction of granular fill up to the abutment subgrade level to permit driving pile.

- The condition of the existing concrete abutments should be assessed, if the abutments are incorporated as protection into the replacement bridge. Any substructure design that includes re-use of old substructures should include repairing and patching areas of concrete that are spalling or cracked. Any undermined portions of the abutments at the bedrock/riverbed interface should be repaired. Requirements for lateral support of pile foundations dictate that the any existing dry laid granite block walls be repointed or reset, as required, to ensure serviceability.

### 7.2.1 Integral Pile Design

The piles should be end bearing and driven to the required resistance on rock or within bedrock. Piles may be HP 12x53, 14x73, 14x89, or 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel. The piles should be oriented for weak axis bending. Piles should be fitted with driving pile points to protect the tips and improve penetration.

The Structural Designer shall design H-piles at the strength limit states considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 6.2.b below.

The design of H-piles at the service limit state shall consider tolerable horizontal movement of the piles, and overall stability of the pile group and displacements considering scour at the design flood event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored Strength Limit States loads with a resistance factor of 1.0. The design flood for scour is defined in LRFD Articles 2.6.4.4.2 and 3.7.5.

### 7.2.2 Strength Limit State Design

The nominal compressive resistance ($P_n$) in the structural limit state for piles loaded in compression shall be as specified in LRFD 6.9.4.1. If the H-piles are fully embedded, $\lambda$ may be taken as 0. For the portion of the pile which is theoretically in pure compression, i.e. below the point of fixity, the factored structural axial resistances of four H-pile sections were calculated using a resistance factor, $\phi_c$, of 0.60. Short pile will not achieve a fixed condition, therefore the factored structural axial resistance will be controlled by the combined axial and flexural resistance of the pile. This analysis is the responsibility of the Structural Designer.

The nominal and factored axial geotechnical resistance in the strength limit state was calculated using the Canadian Geotechnical Society method and a resistance factor, $\varphi_{stat}$, of 0.45. The calculated factored geotechnical resistances of four H-pile sections were calculated and are provided in Table 4, below.

A drivability analyses of the four proposed H-pile sections were conducted. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be no more that 45 ksi. The resistance factor for a single pile in axial compression when a dynamic test is performed given in LRFD Table 10.5.5.2.3-1 is $\phi_{dyn}$ = 0.65. Table 10.5.5.2.3-3 requires that no less than three to four dynamic tests be conducted for sites with low to medium variability. When a pile group is nonredundant, i.e., there are less than five (5) piles, LRFD Article 10.5.5.2.3 dictates a 20 percent reduction of the resistance factor value of 0.65. This results in a resistance factor, $\phi_{dyn}$, of 0.52, which was used to determine the drivability resistance of the pile sections.

For the strength limit state, the calculated factored axial compressive structural resistance, geotechnical and drivability resistances of four (4) proposed H-piles sections are summarized in Table 4 below. Supporting calculations can be found in Appendix C – Calculations, at the end of this report.

| | Strength Limit State Factored Axial Pile Resistance (kips) | | | |
|---|---|---|---|---|
| | Structural Resistance $\phi_c$=0.60[1] | Geotechnical Resistance $\varphi_{stat}$ = 0.45 | Drivablity Resistance | Governing Pile Resistance |
| HP 12 x 53 | 465 | 47 | 197 | 197 |
| HP 14 x 73 | 642 | 64 | 270 | 270 |
| HP 14 x 89 | 783 | 78 | 329 | 329 |
| HP 14 x 117 | 1032 | 103 | 416 | 416 |

**Table 4.** Strength Limit State Factored Axial Structural Resistances for Four H-Pile Sections

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural limit state. However, the factored axial drivablity resistance is less than the factored axial structural resistance, and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is

[1] Assuming $\lambda$ = 0 and $\phi_c$ =0.60. Short pile will not achieve fixity, therefore the factored structural resistance will be controlled by combined the axial and flexural resistance of the pile.

recommended that the governing resistance used in design be the factored drivability resistance in the Table 4. The maximum factored axial pile load should not exceed the calculated factored drivability pile resistances in Table 4.

Per LRFD 10.5.5.3.2 the pile groups shall be designed so that the nominal resistance remaining after the design scour event is no less than the unfactored Strength Limit State loads with a resistance factor of 1.0, including any debris loads occurring from the flood event.

The top of the piles should be checked for resistance against combined axial load and flexure, per LRFD Article 6.15. This axial load will govern the design. The upper portion of the pile is defined per LRFD Figure C6.15.2-1 as that portion of the pile above the point of second inflection in the moment vs. pile depth curve, or at the lowest point of zero deflection. For strength limit state load combinations, a resistance factor of 0.70 for axial resistance ($\phi_c$) and 1.0 for flexural resistance ($\phi_f$) should be applied to the combined axial and flexural resistance of the pile in the interaction equation. The resistance of the pile in the lower zone need only be checked against axial load, but only if the piles are fully fixed.

### 7.2.3 Service and Extreme Limit State Design

The design of piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and deflections resulting from scour at the design flow event. For the service and extreme limit states, resistance factors of 1.0 are recommended for the calculation of structural, geotechnical and drivability axial pile resistances.

Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the design flood can support the unfactored Strength Limit States loads with a resistance factor of 1.0. The unfactored Strength Limit State loads shall include any debris loads occurring during the flood event.

The calculated factored axial structural, geotechnical and drivablity axial resistances of four (4) H-pile sections were calculated for the service and extreme limit states and are provided below in Table 5. Supporting documentation is provided in Appendix C – Calculations.

| | Service and Extreme Limit State Factored Axial Pile Resistance (kips) | | | |
|---|---|---|---|---|
| | Structural Resistance $\phi_c$=0.60[2] | Geotechnical Resistance $\varphi_{stat}$ = 0.45 | Drivablity Resistance | Governing Pile Resistance |
| HP 12 x 53 | 775 | 105 | 379 | 379 |
| HP 14 x 73 | 1070 | 143 | 519 | 519 |
| HP 14 x 89 | 1305 | 174 | 633 | 633 |
| HP 14 x 117 | 1720 | 229 | 800 | 800 |

**Table 5.** Factored Axial Pile Resistance for Piles at the Service and Extreme Limit States.

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural limit state. However, the factored axial drivability resistance is less than the factored axial structural resistance, and local experience supports the estimated factored resistance from the drivablity analyses. Therefore, it is recommended that the governing resistance used in design be the factored drivability resistance in the Table 5. The maximum factored axial pile loads for the service and extreme limit states should not exceed the calculated factored drivablity pile resistance in Table 5.

A resistance factor of 1.0 shall be used to assess pile/abutment design at the service limit state, including: settlement, excessive horizontal movement and deflections resulting from scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, $\varphi$, of 0.65.

### 7.2.4 Driven Pile Resistance and Pile Quality Control

Contract documents should require the contractor to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each substructure. The first pile driven at any substructure should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. Restrikes will be not be required as part of the pile field quality control program.

With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, $\phi_{dyn}$, of 0.65 provided that a minimum of three (3) to four (4) piles out of the total number of piles driven at the project site are dynamically tested, in accordance with LRFD Tables 10.5.5.2.3-1 and -3. LRFD Article 10.5.5.2.3 further specifies that the resistance factor, $\phi_{dyn}$, of 0.65 be reduced by 20 percent when applied to nonredundant pile groups, i.e. less than five (5) piles in the group. This results in a resistance factor, $\phi_{dyn}$, of

[2] Assuming $\lambda = 0$ and $\phi_c$ =0.60. Short pile will not achieve fixity, therefore the factored structural resistance will be controlled by combined the axial and flexural resistance of the pile.

0.52. With the use of a reduced resistance factor, the $\eta_R$ factor provided in Article 1.3.4 should not be increased to address the lack of foundation redundancy. The maximum factored pile load should be shown on the plans. Calculations for the maximum pile resistance determine by drivability analyses are provided in Appendix C – Calculations.

Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than $0.90\phi_{da}$ $F_y$, where $\phi_{da}$ is equal to 1.0, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 5 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

### 7.2.5 Integral Stub Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states specified in LRFD Articles 3.4.1 and 11.5.5. The design of abutments at the strength limit state shall consider pile group failure and structural failure. Strength limit state shall also consider the foundation/pile group resistance after scour due to the design flood, using unfactored loads and nominal pile/foundation resistances. The design of independent return wings at the strength limit stat shall consider nominal bearing resistance, overturning, lateral sliding and structural failure.

The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material. The backfill properties are as follows: $\phi$ = 32 degrees, $\gamma$ = 125 pcf and a soil-concrete friction coefficient of 0.45. Cast-in-place integral abutment sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure. The Rankine passive case is recommended, $K_p = 3.3$.

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 abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ($H_{eq}$) taken from Table 2 provided in Section 6.1.b. of this report.

Wing wall sections that are integral with the abutment, should also be designed to withstand a maximum earth pressure equal to the passive earth pressure state. A Rankine passive earth pressure coefficient, $K_{p,}$ of 3.3 is recommended.

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 ft 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 pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and not exceed 1.75H:1V.

### 7.3 PCMG Retaining Walls

Precast Concrete Modular Gravity (PCMG) walls will retain approach fills on the corners of Abutments 1 and 2. A PCMG along the southeast bridge approach may be constructed in front of the existing wingwalls. 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 units 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. Modular units with a width 8.0 feet or less should be assessed for a factored bearing resistance of 9 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 C – 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. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, $\varphi$, of 0.65

Failure by sliding shall be investigated by the wall subcontractor. A sliding resistance factor, $\varphi_\tau$, 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 0.36 (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 material frictional coefficients are based on LRFD Article 11.11.4.2 and Table 3.11.5.3-1.

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

### 7.4 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at all limits states. Design at the strength limit state should consider loss of lateral and vertical support to due to scour. Design at the extreme limit state should check that the nominal foundation resistance after the design flood event provides adequate resistance to support the unfactored Strength Limit State loads. At the service limit state the design shall limit movements and overall stability considering scour at the design flood. 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 Plain and Hand Laid Riprap 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.

### 7.5 Settlement

The grades of the bridge approaches will be raised approximately 3 inches above to existing grades. 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 or piles is anticipated to occur during construction of the abutments, and is generally assumed to be less than 0.5 inches.

### 7.6 Frost Protection

Heave due to frost is not a design issue for abutment and retaining wall spread footings founded on bedrock, and in those situations, no requirements for minimum depth of embedment are necessary.

PCMG retaining walls will retain sideslope fills at the corners of the abutments and along the southeast bridge approach. These walls should be founded directly on 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, Orland, Maine has a design freezing index of approximately 1475 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 6.8 feet. Modberg, a computer program, developed by U.S. Army Cold Regions Research and Engineering Laboratory, was used to check the calculated maximum depth of frost penetration. The calculated depth of frost according to the Modberg solution, which is based on the Modified Berggren Equation, is 4.8 feet.

We recommend that any foundation placed on soil should be founded a minimum of 5.0 feet below finished exterior grade for frost protection.

### 7.7 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. Orland River Bridge is not on the National Highway System, and is therefore 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.

However, superstructure connections and bridge seat dimensions shall be satisfied per LRFD 3.10.9 and 4.7.4.4, respectively. The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD Manual.

- Peak ground acceleration coefficient (PGA) = 0.062g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.136g
- Long-term (1.0-second period) spectral acceleration coefficient = 0.042g

Per LRFD Article 3.10.3.1 the site is assigned to Site Class D due to the presence of stiff soils with a blow count between 15 and 50 bpf. Per LRFD Article 3.10.6 the site is assigned to Seismic Zone 1 based on a calculated SD1 of 0.1.

### 7.8 Construction Considerations

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

The existing return wingwalls are on spread footings up to 8 feet wide and will obstruct embedment of piles. Removal of the existing wingwalls will be necessary. This will also necessitate the replacement of excavated materials with compacted granular fill before pile driving can commence.

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.

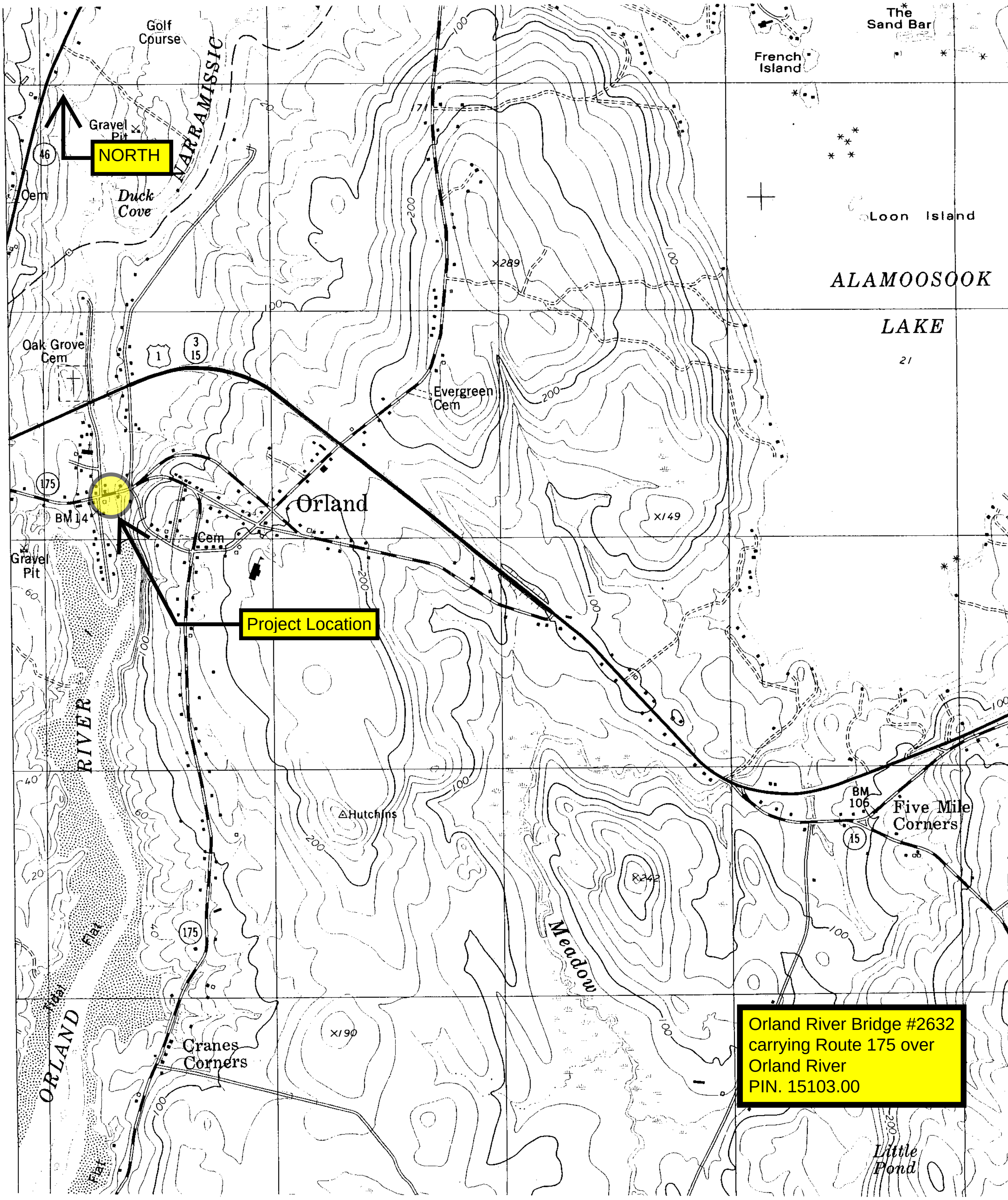
It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

## 8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Orland River Bridge in Orland, 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**



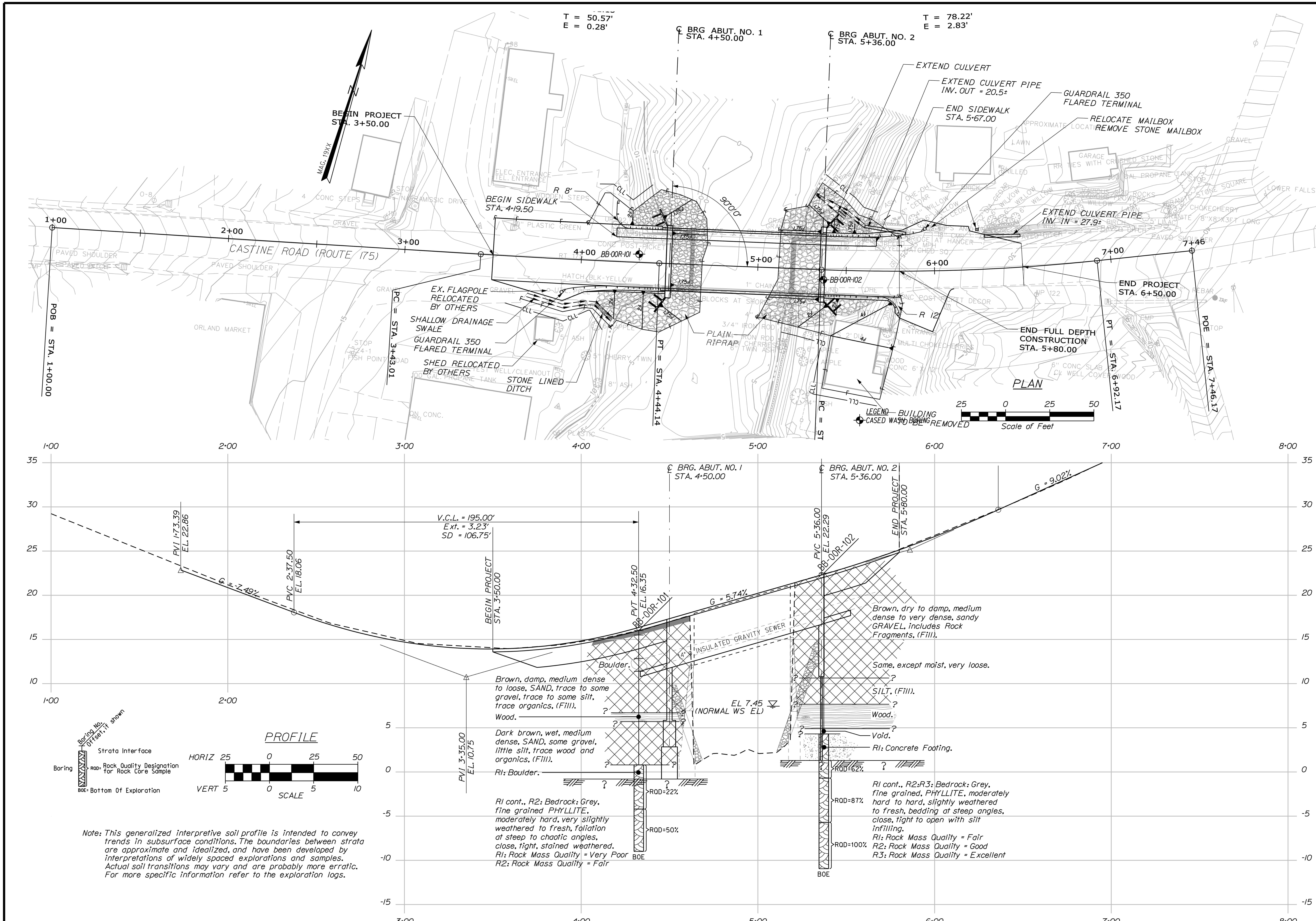

ORLAND QUADRANGLE
MAINE–HANCOCK CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
SW/4 ORLAND 15' QUADRANGLE

SCALE 1:24 000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER




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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1510(300)X

PIN 15103.00

BRIDGE NO. 2632

BRIDGE PLANS

| | BY | DATE |
|---|---|---|
| PROJ. MANAGER | | |
| DESIGN-DETAILED | L. KRUSINSKI | |
| CHECKED-REVIEWED | T. WHITE | SEPT 2008 |

SIGNATURE

P.E. NUMBER

DATE

ORLAND RIVER BRIDGE
ORLAND RIVER
ORLAND HANCOCK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER 2 OF 4

Filename: ...\geotech\msta\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:12/1/2008

Filename: ...\msta\007_BORING LOGS1.dgn    Division: GEOTECH    Username: terry.white    Date:12/1/2008

## Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Orland River Bridge #2632 carring Route 175 over Orland River
Location: Orland River

Boring No.: BB-OOR-101
PIN: 15103.00

| | | | | | |
|---|---|---|---|---|---|
| Driller: | MaineDOT | Elevation (ft.) | 16.0 | Auger ID/OD: | 5" Solid Stem |
| Operator: | E. Giguere/C. Giles | Datum: | NAVD 88 | Sampler: | Standard Split Spoon |
| Logged By: | L. Krusinski | Rig Type: | CME 45C | Hammer Wt./Fall: | 140#/30" |
| Date Start/Finish: | 9/3/08; 08:00-12:00 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ-2" |
| Boring Location: | 4+32.6, 4.7 Lt. | Casing ID/OD: | HW | Water Level*: | Not Measured |
| Hammer Efficiency Factor: 0.77 | | 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 | $N_{60}$ | Casing Blows | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 15.50 | PAVEMENT. 0.50 | |
| | 1D | 24/16 | 1.00 - 3.00 | 12/10/5/6 | 15 | 19 | | | Brown, damp, medium dense, fine to coarse SAND, some fine gravel, trace silt, (Fill). Boulder from 2.8-4.0' bgs. | G#210031 A-1-b, SM WC=5.3% |
| 5 | 2D | 24/9 | 5.00 - 7.00 | 2/1/4/1 | 5 | 6 | 4 | 11.00 | 5.00 Brown, damp, loose, fine to coarse SAND, some silt, trace gravel, trace organics, (Fill). | |
| | | | | | | | 10 | | | |
| | | | | | | | 25 | | | |
| | | | | | | | 64 | | | |
| | | | | | | | 350 | 6.70 | 9.30 Wood from 9.3-10.3' bgs. Roller Coned ahead from 9.3-10.6' bgs. | |
| 10 | MD | 24/0 | 10.50 - 12.50 | 4/3/3/8 | 6 | 8 | 98 | 5.70 | 10.30 Failed sample attempt, wood fragments in spoon. | |
| | | | | | | | 25 | | | |
| | | | | | | | 25 | | | |
| | 3D | 24/14 | 13.00 - 15.00 | 9/9/3/23 | 12 | 15 | 48 | | Dark brown, wet, medium dense, fine to coarse SAND, some fine to coarse gravel, little silt, trace wood fragments and organics. Roller Coned ahead after 3D to 15.2' bgs. | G#210032 A-1-b, SM WC=27.5% |
| | | | | | | | 311 | | | |
| 15 | R1 | 60/48 | 15.20 - 20.20 | RQD = 22% | | | NQ-2 | 0.80 | 15.20 R1: Boulder from 15.2-16.8' bgs. R1:Core Times (min:sec) 15.2-16.2' (2:02) 16.2-17.2' (1:42) | |
| | | | | | | | | -0.80 | 16.80 Top of Bedrock at Elev. -0.8'. R1: Bedrock: Grey, fine grained PHYLLITE, moderately hard, very slightly weathered, chaotic foliation to steep, joint surfaces tight, stained, weathered. Rock Mass Quality: Very Poor. R1:Core Times cont. 17.2-18.2' (1:41) 18.2-19.2' (2:20) 19.2-20.2' (2:50) 80% Recovery | |
| 20 | R2 | 57.6/ 57.6 | 20.20 - 25.00 | RQD = 50% | | | | -4.20 | 20.20 R2: Bedrock: Grey, fine grained PHYLLITE, moderately hard, fresh, bedding close at chaotic to steep angles, joint surfaces tight, unweathered, moderately fractued overall. Rock Mass Quality: Fair. R2:Core Times (min:sec) 20.2-21.2' (2:42) 21.2-22.2' (3:05) 22.2-23.2' (3:06) 23.2-24.2' (3:05) 24.2-25.0' (3:30) 100% Recovery Bottom of Exploration at 25.00 feet below ground surface. | |
| 25 | | | | | | | | -9.00 | 25.00 Bottom of Exploration at 25.00 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.

Page 1 of 1
Boring No.: BB-OOR-101

## Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Orland River Bridge #2632 carring Route 175 over Orland River
Location: Orland River

Boring No.: BB-OOR-102
PIN: 15103.00

| | | | | | |
|---|---|---|---|---|---|
| Driller: | MaineDOT | Elevation (ft.) | 22.5 | Auger ID/OD: | 5" Solid Stem |
| Operator: | E. Giguere/C. Giles | Datum: | NAVD 88 | Sampler: | Standard Split Spoon |
| Logged By: | L. Krusinski | Rig Type: | CME 45C | Hammer Wt./Fall: | 140#/30" |
| Date Start/Finish: | 9/3/08; 12:00-16:00 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ-2" |
| Boring Location: | 5+37.3, 5.6 Rt. | Casing ID/OD: | HW & NW | Water Level*: | 16.0' bgs. |
| Hammer Efficiency Factor: 0.77 | | 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 | $N_{60}$ | Casing Blows | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | | | |
| | 1D | 24/17 | 1.00 - 3.00 | 24/28/13/11 | 41 | 53 | | | Brown, dry to damp, very dense, sandy fine to coarse GRAVEL (coarse portion: rock fragments), trace silt, (Fill). | G#210033 A-1-a, GM WC=3.3% |
| 5 | MD | 24/0 | 5.00 - 7.00 | 3/7/4/4 | 11 | 14 | 25 | | Failed sample attempt, drove spoon second time; Brown, dry, medium dense, silty rock fragments, some fine sand, (Fill). | |
| | | | | | | | 20 | | | |
| | | | | | | | 8 | | | |
| | | | | | | | 6 | | | |
| | | | | | | | 5 | | | |
| 10 | | | | | | | 4 | | | |
| | 2D | 24/4 | 11.00 - 13.00 | 1/1/1/5 | 2 | 3 | 12 | | Brown, moist, very loose, angular GRAVEL, changed to fine sandy silt in basket, (Fill). | |
| | | | | | | | 8 | 10.50 | 12.00 Change to brown, SILT. | |
| | | | | | | | 10 | | | |
| | | | | | | | a62 RC | 7.67 | a62 blows for 10". 14.83 Roller coned ahead of casing from 14.83-18.2' bgs, then telescoped NW Casing. WOOD layer from 14.83-17.6' bgs. | |
| 15 | 3D | 24/17 | 15.00 - 17.00 | 20/10/34/31 | 44 | 56 | | | | |
| | | | | | | | | 4.90 | 17.60 | |
| | R1 | 60/57 | 18.20 - 23.20 | | | | NQ-2 | 4.30 | 18.20 R1: CONCRETE Footing. R1:Core Times (min:sec) 18.2-19.2' (2:37) 19.2-20.2' (2:29) 20.2-21.2' (2:14) | |
| 20 | | | | | | | | | | |
| | | | 21.20 - 23.20 | RQD = 62% | | | | 1.30 | 21.20 Top of Bedrock at Elev. 1.3'. R1:Bedrock: Grey, fine grained, hard PHYLLITE, bedding at steep angles, very close, joints tight and weathered to open and silt infilled. Rock Mass Quality: Fair. R1:Core Times cont. 21.2-22.2' (1:50), grey silt seam 22.2-23.2' (2:10), dark grey water 95% Recovery | |
| | R2 | 60/60 | 23.20 - 28.20 | RQD = 87% | | | | -0.70 | 23.20 R2: Bedrock: Grey, fine grained PHYLLITE, moderately hard, fresh to slightly weathered, bedding close at steep angles, joints typically tight and fresh to open with silt infilling, 1/4" silt seam at 4.4' into core run. Rock Mass Quality: Good. R2:Core Times (min:sec) 23.2-24.2' (2:22) 24.2-25.2' (2:01) 25.2-26.2' (2:02) 26.2-27.2' (1:01) 27.2-28.2' (1:07) 100% Recovery | |
| 25 | | | | | | | | | | |
| | R3 | 60/63 | 28.20 - 33.20 | RQD = 100% | | | | -5.70 | 28.20 R3:Bedrock: Grey, fine grained PHYLLITE, hard, fresh, bedding at steep angles, very close, joints tight, no infilling. Rock Mass Quality: Excellent. R3:Core Times (min:sec) 28.2-29.2' (1:40) 29.2-30.2' (1:54) 30.2-31.2' (1:20) 31.2-32.2' (1:40) 32.2-33.45' (1:52) 105% Recovery | |
| 30 | | | | | | | | | | |
| | | | | | | | | -10.95 | 33.45 Bottom of Exploration at 33.45 feet below ground surface. | |
| 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.

Page 1 of 1
Boring No.: BB-OOR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1510(300)X

PIN 15103.00

BRIDGE PLANS

BRIDGE NO. 2632

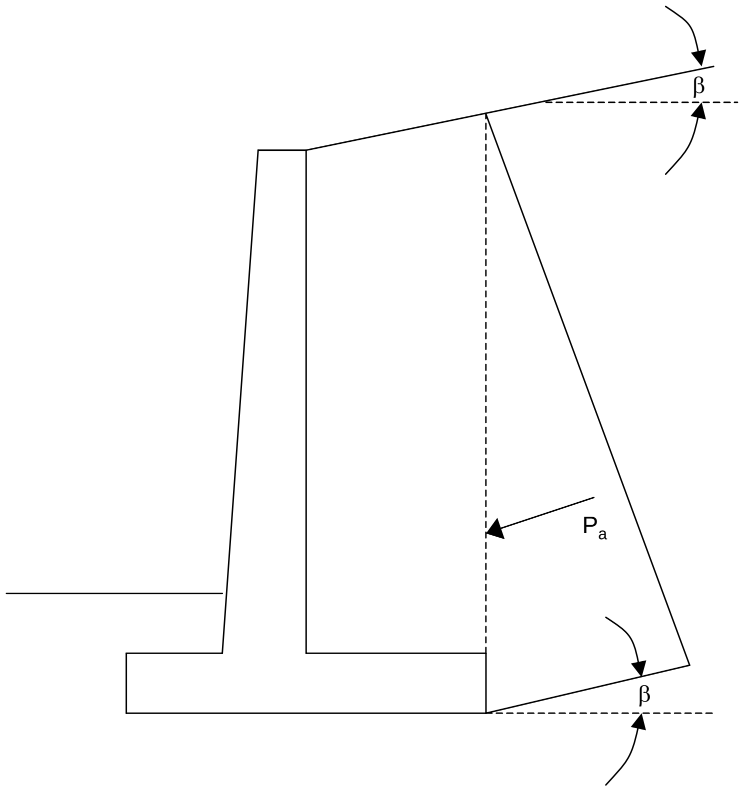
| | BY | DATE |
|---|---|---|
| PROJ. MANAGER | | |
| DESIGN-DETAILED | L. KRUSINSKI / T. WHITE | SEPT 2008 |
| CHECKED-REVIEWED | | |
| DESIGN2-DETAILED2 | | |
| DESIGN3-DETAILED3 | | |
| REVISIONS 1 | | |
| REVISIONS 2 | | |
| REVISIONS 3 | | |
| REVISIONS 4 | | |
| FIELD CHANGES | | |

SIGNATURE
P.E. NUMBER
DATE

ORLAND RIVER BRIDGE
ORLAND RIVER
HANCOCK COUNTY
ORLAND

# BORING LOGS

SHEET NUMBER
3
OF 4




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

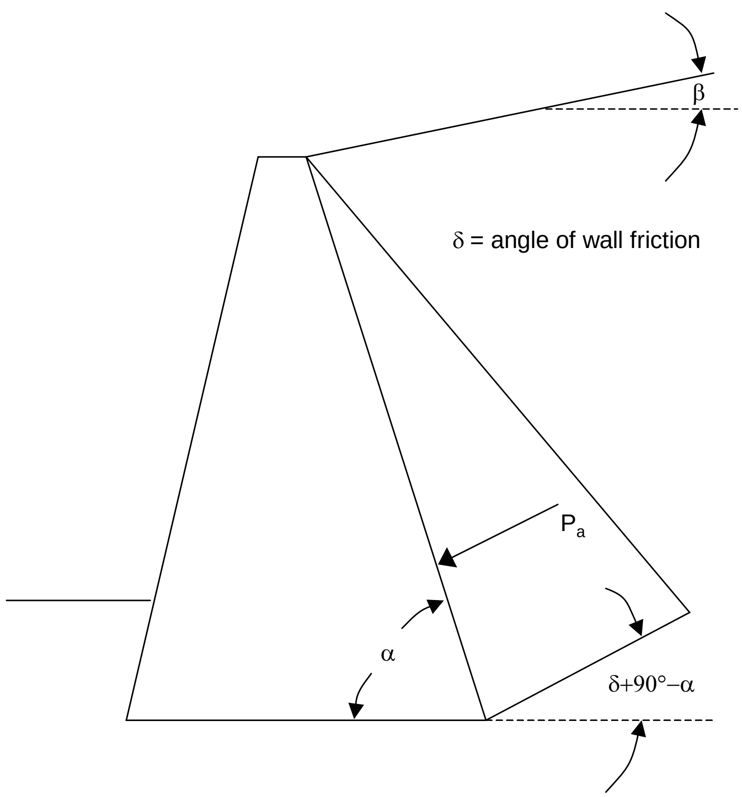
For a horizontal backfill surface, β = 0°:

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

For a sloped backfill surface, β > 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 δ + 90° - α

Rankine and Coulomb Active Earth Pressure Coefficients

# Appendix A

Boring Logs

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Orland River Bridge #2632 carring Route 175 over Orland River
**Location:** Orland River

**Boring No.:** BB-OOR-101
**PIN:** 15103.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 16.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | E. Giguere/C. Giles | **Datum:** | NAVD 88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | L. Krusinski | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/3/08; 08:00-12:00 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+32.6, 4.7 Lt. | **Casing ID/OD:** | HW | **Water Level*:** | Not Measured |
| **Hammer Efficiency Factor:** | 0.77 | **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 | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 15.50 | | PAVEMENT. 0.50 | |
| | 1D | 24/16 | 1.00 - 3.00 | 12/10/5/6 | 15 | 19 | | | | Brown, damp, medium dense, fine to coarse SAND, some fine gravel, trace silt. (Fill). | G#210031 A-1-b, SM WC=5.3% |
| | | | | | | | | | | | |
| | | | | | | | | | | Boulder from 2.8-4.0' bgs. | |
| | | | | | | | | | | | |
| 5 | 2D | 24/9 | 5.00 - 7.00 | 2/1/4/1 | 5 | 6 | 4 | 11.00 | | 5.00 Brown, damp, loose, fine to coarse SAND, some silt, trace gravel, trace organics. (Fill). | |
| | | | | | | | 10 | | | | |
| | | | | | | | 25 | | | | |
| | | | | | | | 64 | | | | |
| | | | | | | | 350 | 6.70 | | 9.30 Wood from 9.3-10.3' bgs. Roller Coned ahead from 9.3-10.6' bgs. | |
| 10 | MD | 24/0 | 10.50 - 12.50 | 4/3/3/8 | 6 | 8 | 98 | 5.70 | | 10.30 Failed sample attempt, wood fragments in spoon. | |
| | | | | | | | 25 | | | | |
| | | | | | | | 25 | | | | |
| | 3D | 24/14 | 13.00 - 15.00 | 9/9/3/23 | 12 | 15 | 48 | | | Dark brown, wet, medium dense, fine to coarse SAND, some fine to coarse gravel, little silt, trace wood fragments and organics. Roller Coned ahead after 3D to 15.2' bgs. | G#210032 A-1-b, SM WC=27.5% |
| | | | | | | | 311 | | | | |
| 15 | R1 | 60/48 | 15.20 - 20.20 | RQD = 22% | | | NQ-2 | 0.80 | | 15.20 R1: Boulder from 15.2-16.8' bgs. R1:Core Times (min:sec) 15.2-16.2' (2:02) 16.2-17.2' (1:42) | |
| | | | | | | | | -0.80 | | 16.80 Top of Bedrock at Elev. -0.8'. R1: Bedrock: Grey, fine grained PHYLLITE, moderately hard, very slightly weathered, chaotic foliation to steep, joint surfaces tight, stained, weathered. Rock Mass Quality: Very Poor. R1:Core Times cont. 17.2-18.2' (1:41) 18.2-19.2' (2:20) 19.2-20.2' (2:50) 80% Recovery | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 20 | R2 | 57.6/57.6 | 20.20 - 25.00 | RQD = 50% | | | | -4.20 | | 20.20 R2: Bedrock: Grey, fine grained PHYLLITE, moderately hard, fresh, bedding close at chaotic to steep angles, joint surfaces tight, unweathered, moderately fractued overall. Rock Mass Quality: Fair. R2:Core Times (min:sec) 20.2-21.2' (2:42) 21.2-22.2' (3:05) | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 25 | | | | | | | | | | | |

**Remarks:**

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

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

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| **Project:** | Orland River Bridge #2632 carring Route 175 over Orland River | **Boring No.:** | BB-OOR-101 | | |
| **Location:** | Orland River | **PIN:** | 15103.00 | | |
| **Driller:** | MaineDOT | **Elevation (ft.)** | 16.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | E. Giguere/C. Giles | **Datum:** | NAVD 88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | L. Krusinski | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/3/08; 08:00-12:00 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+32.6, 4.7 Lt. | **Casing ID/OD:** | HW | **Water Level*:** | Not Measured |
| **Hammer Efficiency Factor:** | 0.77 | **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 | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | | -9.00 | | 22.2-23.2' (3:06)<br>23.2-24.2' (3:05)<br>24.2-25.0' (3:30) 100% Recovery<br>**Bottom of Exploration at 25.00 feet below ground surface.** 25.00<br>**Bottom of Exploration at 25.00 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.



**Boring No.:** BB-OOR-101

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Orland River Bridge #2632 carring Route 175 over Orland River
**Location:** Orland River

**Boring No.:** BB-OOR-102
**PIN:** 15103.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 22.5 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | E. Giguere/C. Giles | **Datum:** | NAVD 88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | L. Krusinski | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/3/08; 12:00-16:00 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 5+37.3, 5.6 Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 16.0' bgs. |
| **Hammer Efficiency Factor:** | 0.77 | **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)
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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 | | | | | | | SSA | | | | |
| | 1D | 24/17 | 1.00 - 3.00 | 24/28/13/11 | 41 | 53 | | | | Brown, dry to damp, very dense, sandy fine to coarse GRAVEL (coarse portion: rock fragments), trace silt, (Fill). | G#210033 A-1-a, GM WC=3.3% |
| 5 | MD | 24/0 | 5.00 - 7.00 | 3/7/4/4 | 11 | 14 | 25 | | | Failed sample attempt, drove spoon second time; Brown, dry, medium dense, silty rock fragments, some fine sand, (Fill). | |
| | | | | | | | 20 | | | | |
| | | | | | | | 8 | | | | |
| | | | | | | | 6 | | | | |
| | | | | | | | 5 | | | | |
| 10 | | | | | | | 4 | | | | |
| | 2D | 24/4 | 11.00 - 13.00 | 1/1/1/5 | 2 | 3 | 12 | | | Brown, moist, very loose, angular GRAVEL, changed to fine sandy silt in basket, (Fill). | |
| | | | | | | | 8 | 10.50 | | 12.00 Change to brown, SILT. | |
| | | | | | | | 10 | | | | |
| | | | | | | | a62 | | | a62 blows for 10". | |
| 15 | 3D | 24/17 | 15.00 - 17.00 | 20/10/34/31 | 44 | 56 | RC | 7.67 | | 14.83 Roller coned ahead of casing from 14.83-18.2' bgs, then telescoped NW Casing. WOOD layer from 14.83-17.6' bgs. | |
| | | | | | | | | 4.90 | | 17.60 | |
| | R1 | 60/57 | 18.20 - 23.20 | | | | NQ-2 | 4.30 | | 18.20 R1: CONCRETE Footing. R1:Core Times (min:sec) 18.2-19.2' (2:37) 19.2-20.2' (2:29) 20.2-21.2' (2:14) | |
| 20 | | | | | | | | | | | |
| | | | 21.20 - 23.20 | RQD = 62% | | | | 1.30 | | 21.20 Top of Bedrock at Elev. 1.3'. R1:Bedrock: Grey, fine grained, hard PHYLLITE, bedding at steep angles, very close, joints tight and weathered to open and silt infilled. Rock Mass Quality: Fair. R1:Core Times cont. 21.2-22.2' (1:50), grey silt seam 22.2-23.2' (2:10), dark grey water | |
| | R2 | 60/60 | 23.20 - 28.20 | RQD = 87% | | | | -0.70 | | | |
| 25 | | | | | | | | | | | |

**Remarks:**

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

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

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| **Project:** | Orland River Bridge #2632 carring Route 175 over Orland River | **Location:** | Orland River | **Boring No.:** | BB-OOR-102 |
| | | | | **PIN:** | 15103.00 |

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 22.5 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | E. Giguere/C. Giles | **Datum:** | NAVD 88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | L. Krusinski | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 9/3/08; 12:00-16:00 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 5+37.3, 5.6 Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 16.0' bgs. |
| **Hammer Efficiency Factor:** | 0.77 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
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N-uncorrected = Raw field SPT N-value
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
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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 | | | | | | | | | | 95% Recovery | |
| | | | | | | | | | | 23.20 R2: Bedrock: Grey, fine grained PHYLLITE, moderately hard, fresh to slightly weathered, bedding close at steep angles, joints typically tight and fresh to open with silt infilling, 1/4" silt seam at 4.4' into core run. Rock Mass Quality: Good. R2:Core Times (min:sec) 23.2-24.2' (2:22) 24.2-25.2' (2:01) 25.2-26.2' (2:02) 26.2-27.2' (1:01) 27.2-28.2' (1:07) 100% Recovery | |
| | R3 | 60/63 | 28.20 - 33.20 | RQD = 100% | | | | -5.70 | | 28.20 R3:Bedrock: Grey, fine grained PHYLLITE, hard, fresh, bedding at steep angles, very close, joints tight, no infilling. Rock Mass Quality: Excellent. R3:Core Times (min:sec) 28.2-29.2' (1:40) 29.2-30.2' (1:54) 30.2-31.2' (1:20) 31.2-32.2' (1:40) 32.2-33.45' (1:52) 105% Recovery | |
| 30 | | | | | | | | | | | |
| | | | | | | | | -10.95 | | | |
| 35 | | | | | | | | | | 33.45 **Bottom of Exploration at 33.45 feet below ground surface.** | |
| 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 | Blow Counts |
| Bridge Name / Town | Sample Recovery |
| Boring Number | Date |
| Sample Number | Personnel Initials |
| Sample Depth | |

# <u>Appendix B</u>

Laboratory Test Results

State of Maine - Department of Transportation

## Laboratory Testing Summary Sheet

# Town(s): Orland      Project Number: 15103.00

| Boring & Sample Identification Number | Station (Feet) | Offset (Feet) | Depth (Feet) | Reference Number | G.S.D.C. Sheet | W.C. % | L.L. | P.I. | Classification | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Unified | AASHTO | Frost |
| BB-ORR-101, 1D | 4+32.6 | 4.7 Lt. | 1.0-3.0 | 210031 | 1 | 5.3 | | | SM | A-1-b | II |
| BB-ORR-101, 3D | 4+32.6 | 4.7 Lt. | 13.0-15.0 | 210032 | 1 | 27.5 | | | SM | A-1-b | II |
| BB-ORR-102, 1D | 5+37.3 | 5.6 Rt. | 1.0-3.0 | 210033 | 1 | 3.3 | | | GM | A-1-a | I |

**Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.**

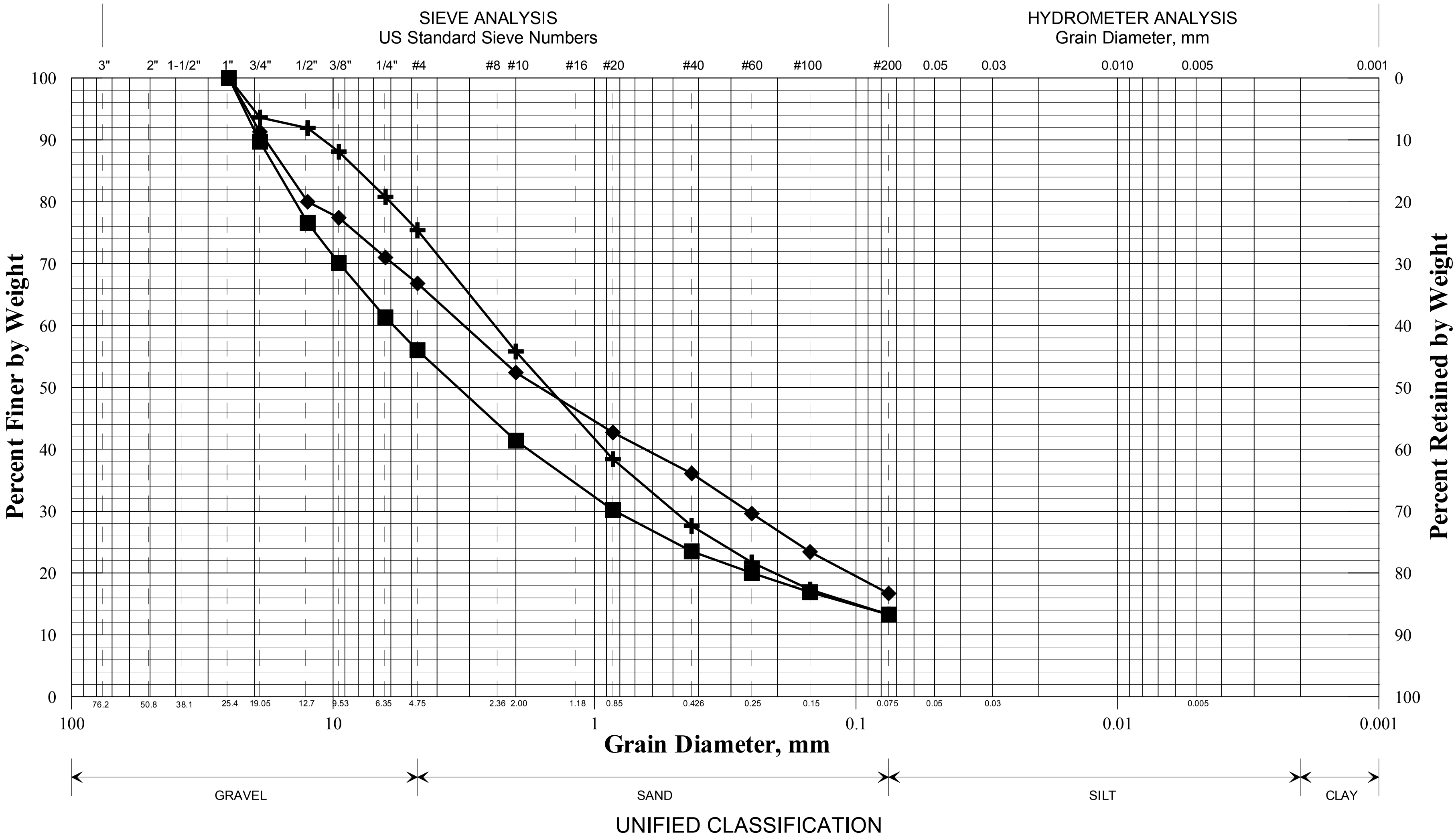
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

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-OOR-101/1D | 4+32.6 | 4.7 LT | 1.0-3.0 | SAND, some gravel, little silt. | 5.3 | | | |
| ◆ | BB-OOR-101/3D | 4+32.6 | 4.7 LT | 13.0-15.0 | SAND, some gravel, little silt. | 27.5 | | | |
| ■ | BB-OOR-102/1D | 5+37.3 | 5.6 RT | 1.0-3.0 | Sandy GRAVEL, trace silt. | 3.3 | | | |
| ● | | | | | | | | | |
| ▲ | | | | | | | | | |
| ✖ | | | | | | | | | |

| PIN |
|---|
| 015103.00 |
| Town |
| Orland |
| Reported by/Date |
| WHITE, TERRY A 10/1/2008 |

**SHEET 1**

# Appendix C

Calculations

## Bearing Resistance - 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-OOR-101, R1, PHYLITTE, MOD. hard, very slightly weathered, chaotic foliation, tight, stained, weathered, RQD=22%, improving to fresh PHYLLITE, RQD=50%

Abutment 2: Boring BB-OOR-102, R1: PHYLITTE. RQDs are 62% and 87% and 100%. Moderately hard to hard, fresh to slightly weathered, some joints open and 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***

***Assumption: Poor rock will be removed. RQD for upper 3 feet is 22% in BB-OOR-101.***

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, PYLITTE |
| c. Unconfined compressive strength, Co | 10,000 psi estimated (3,500 - 35,000 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 factored Service Load Combination of a maximum of 20 ksf for initial sizing of footing**

**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:

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

**Footing Width and Depth**

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \\ 20 \end{pmatrix} \cdot ft \qquad D_f := 6.0 \cdot ft \qquad D_w := 6 \cdot ft \qquad \gamma_w := 62.4 \cdot pcf$$

**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 - $\phi$ 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)

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

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

### *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 10 to 20 feet wide, on compacted granular fill. Recommend a bearing resistance of 9 ksf for smaller footings 8 feet wide or less.**

## Bedrock Properties at the Site

RQD from bedrock cores
22% to 50% in BB-OOR-101
62% to 100% in BB-OOR-102

Rock Type: Phyllite

$\phi$ **= 20-27** (AASHTO LRFD Table C.10.4.6.4-1);

uniaxial compressive strength = Cu= 3500 to 35,000 psi - use **10,000 psi** for design AASHTO TABLE 4.4.8.2.B

## Pile Properties

Use the following piles: 12x53, 14x73, 14x89, 14x117

$$A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.89 \\ 198.356 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$$

## Nominal and Factored Structural Compressive Resistance of HP piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock
Use LRFD Equation 6.9.2.1-1

Normalized column slenderness factor, $\lambda$, in equation 6.9.4.1-1 is assumed to be zero since the unbraced length is zero.

$$F_y := 50 \cdot ksi$$

$$\lambda := 0$$

### Nominal Axial Structural Resistance

From LRFD 6.9.4.1-1

$$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$$

$$\boxed{P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip}$$

**Factored Axial Structural Resistance of single H pile**

Resistance factor or H-pile in compression, no damage anticipated, LRFD 6.5.4.2

$\phi_c := 0.6$

Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1

$P_r := \phi_c \cdot P_n$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 465 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot kip$$

## Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Talbe 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example www.fhwa.gov/bridge/lrfd/us_dsp.htm

**Nominal unit bearing resistance of pile point, qp**

Design value of compressive strength of rock core

Phylitte $q_{u\_1} := 10000 \cdot psi$

Spacing of discontinuities $s_d := 4 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $t_d := \frac{1}{64} \cdot in$

Pile width is b - matrix $D := b$

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 0 \cdot ft$

Diameter of socket: $D_s := 12 \cdot in$

Depth factor $dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$ and dd < 3.4

$dd = 1$ OK

Ksp

$$K_{sp} := \frac{3 + \frac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \frac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.226 \\ 0.222 \\ 0.222 \\ 0.222 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$$

$$q_{p\_1} = \begin{pmatrix} 977 \\ 960 \\ 959 \\ 958 \end{pmatrix} \cdot ksf$$

**Nominal geotechnical tip resistance, Rp -** *Extreme Limit States and Service Limit States*

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$$

$$R_{p\_1} = \begin{pmatrix} 105 \\ 143 \\ 174 \\ 229 \end{pmatrix} \cdot kip$$

**Factored Axial Geotechnical Compressive Resistance** - *Strength Limit States*

Resistance factor, end bearing on rock Candadian Geotechnical Society method

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

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$$

$$R_{r\_p1} = \begin{pmatrix} 47 \\ 64 \\ 78 \\ 103 \end{pmatrix} \cdot kip$$

## Drivability Analysis

Ref: LRFD Article 10.7.8

For steel piles in compression or tension, driving stresses are limited to 90% of fy

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1, Drivablity Analysis, steel piles

$\sigma_{dr} := 0.90 \cdot 50 \cdot (ksi) \cdot \phi_{da}$

$\sigma_{dr} = 45 \cdot ksi$ driving stress cannot exceed 45 ksi

**Compute the resistance that can be achieved in a drivablity analysis:**

The resistance that must be achieved in a drivablity analysis will be the maximum factored pile load divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 page 10-38 gives resistance factor for dynamic test,

$\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium variablity. Only 1 to 2 piles will be tested, and the pile group would be nonredundant, i.e. less than five piles. Therefore reduce Φ by 20%.

$\phi_{dyn\_red} := 0.65 \cdot 0.8$ $\phi_{dyn\_red} = 0.52$

## Pile Size is 12 x 53

**The 12x53 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:**

State of Maine Dept. Of Transportation — 24-Oct-2008
Orland 12 x 53 fuel set 9 ft str — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 100.0 | 21.36 | 0.15 | 1.1 | 5.50 | 15.45 |
| 200.0 | 30.88 | 0.42 | 2.4 | 6.66 | 14.74 |
| 300.0 | 39.23 | 1.52 | 3.8 | 7.45 | 15.79 |
| 350.0 | 42.85 | 2.24 | 4.6 | 7.88 | 16.60 |
| 400.0 | 46.53 | 2.45 | 5.5 | 8.43 | 17.74 |
| 450.0 | 49.49 | 2.97 | 6.6 | 8.86 | 18.54 |
| 500.0 | 52.35 | 3.44 | 8.1 | 9.33 | 19.45 |

DELMAG D 19-42

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 2.70 | kips |
| Hammer Cus | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.060 | in |
| Skin Damping | 0.100 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 30.00 | ft |
| Pile Penetratic | 10.00 | ft |
| Pile Top Area | 15.50 | in2 |

Limiting driving stress to 45 ksi:

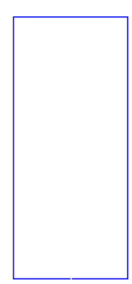
$$R_{ndr} := \left(\frac{45 - 42.85}{46.53 - 42.85}\right) \cdot (400 \cdot kip - 350 \cdot kip) + 350 \cdot kip$$

$$R_{ndr} = 379 \cdot kip$$

$$R_{fdr} := R_{ndr} \cdot \phi_{dyn\_red}$$

$$\boxed{R_{fdr} = 197 \cdot kip}$$

Pile Model



Res. Shaft = 1 %
(Constant Res. Shaft)

## Pile Size is 14 x 74

**The 14x 73 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:**

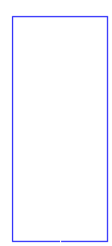
State of Maine Dept. Of Transportation — 24-Oct-2008
Orland14 x 73 fuel set 9 ft str — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 100.0 | 19.60 | 0.09 | 1.1 | 5.55 | 15.29 |
| 300.0 | 32.52 | 0.97 | 3.9 | 7.26 | 14.71 |
| 400.0 | 38.62 | 1.60 | 5.4 | 7.99 | 15.87 |
| 450.0 | 41.37 | 3.46 | 6.4 | 8.36 | 16.53 |
| 500.0 | 44.01 | 5.29 | 7.5 | 8.77 | 17.20 |
| 550.0 | 46.60 | 5.78 | 8.8 | 9.21 | 17.99 |

DELMAG D 19-42

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 2.70 | kips |
| Hammer Cus | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.060 | in |
| Skin Damping | 0.100 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 30.00 | ft |
| Pile Penetrati | 10.00 | ft |
| Pile Top Area | 21.40 | in2 |

Pile Model



Res. Shaft = 1 %
(Constant Res. Shaft)

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.01}{46.60 - 44.01}\right)\cdot(550\cdot kip - 500\cdot kip) + 500\cdot kip$$

$$R_{ndr} = 519\cdot kip$$

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn\_red}$$

$$\boxed{R_{fdr} = 270\cdot kip}$$

## Pile Size is 14 x 89

**The 14 x 89 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:**

State of Maine Dept. Of Transportation — 24-Oct-2008
Orland 14 x 89 fuel setting 1 — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 100.0 | 19.83 | 0.19 | 1.0 | 6.03 | 17.59 |
| 300.0 | 30.22 | 0.35 | 3.6 | 7.82 | 16.16 |
| 400.0 | 35.62 | 1.35 | 4.9 | 8.35 | 16.81 |
| 500.0 | 40.93 | 2.85 | 6.5 | 9.15 | 18.11 |
| 550.0 | 43.34 | 3.07 | 7.5 | 9.56 | 18.75 |
| 600.0 | 45.62 | 3.38 | 8.6 | 9.95 | 19.51 |
| 700.0 | 49.55 | 4.18 | 11.9 | 10.64 | 21.15 |

DELMAG D 19-42

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 2.70 | kips |
| Hammer Cı | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.060 | in |
| Skin Dampin | 0.100 | sec/ft |
| Toe Dampin | 0.150 | sec/ft |
| Pile Length | 30.00 | ft |
| Pile Penetra | 10.00 | ft |
| Pile Top Are | 26.10 | in2 |

Limiting driving stress to 45 ksi:

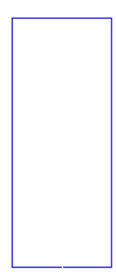
$$R_{ndr} := \left(\frac{45 - 43.34}{45.62 - 44.62}\right)\cdot(600\cdot kip - 550\cdot kip) + 550\cdot kip$$

$$R_{ndr} = 633\cdot kip$$

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn\_red}$$

$$\boxed{R_{fdr} = 329\cdot kip}$$

Pile Model



Res. Shaft = 1 %
(Constant Res. Shaft)

## Pile Size is 14 x 117

**The 14 x 117 pile can be driven to the resistances below with a D 19-42 at Fuel Setting 1 and a 3.2 kip helmet, at a reasonable blow count and level of driving stress. See GRLWEAP results below:**

State of Maine Dept. Of Transportation — 27-Oct-2008
Orland 14 x 117 fuel setting 1 H-3.2 kip — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 100.0 | 17.90 | 1.40 | 0.9 | 6.11 | 17.44 |
| 300.0 | 24.19 | 0.35 | 3.7 | 7.73 | 15.89 |
| 500.0 | 32.72 | 1.30 | 6.6 | 8.64 | 16.84 |
| 700.0 | 40.24 | 2.49 | 11.0 | 9.93 | 19.65 |
| 750.0 | 41.77 | 3.19 | 12.8 | 10.22 | 20.34 |
| 800.0 | 43.16 | 3.75 | 14.9 | 10.48 | 20.97 |
| 850.0 | 44.76 | 4.15 | 16.9 | 10.81 | 21.78 |

Limiting blow count to 15 bpi:

$R_{ndr} := 800 \cdot kip$

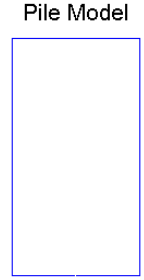
$R_{fdr} := R_{ndr} \cdot \phi_{dyn\_red}$

$\boxed{R_{fdr} = 416 \cdot kip}$

DELMAG D 19-42

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 3.20 | kips |
| Hammer Cushion | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.060 | in |
| Skin Damping | 0.100 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 30.00 | ft |
| Pile Penetration | 10.00 | ft |
| Pile Top Area | 34.40 | in2 |

Pile Model



Res. Shaft = 1 %
(Constant Res. Shaft)

## Calibration back to ASD - Structural Capacity

**Geotechnical design capacity shall not exceed the pile structural allowable design load , based on allowable steel stress for integral piles, use 50 ksi steel, therefore 0.25Fy is the allowable stress.**

*For 50 ksi steel* $F_y := 50 \cdot ksi$ $\sigma_a := \frac{F_y}{4}$ $Q_{all} := \sigma_a \cdot A_s$

$$Q_{all} = \begin{pmatrix} 194 \\ 268 \\ 326 \\ 430 \end{pmatrix} \cdot kip$$

**50 ksi steel piles driven to 2.25 times the structural capacity**

$Q_{ult} := Q_{all} \cdot 2.25$

$$Q_{ult} = \begin{pmatrix} 436 \\ 602 \\ 734 \\ 968 \end{pmatrix} \cdot kip$$

**Assume the above equals the nominal geotechnical capacity**

Factored resistance = 2.25 times the structural capacity times a resistance factor of 0.65

$R_{factored} := Q_{ult} \cdot 0.65$

$$R_{factored} = \begin{pmatrix} 283 \\ 391 \\ 477 \\ 629 \end{pmatrix} \cdot kip$$

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

From Design Freezing Index Map:
**Orland**
DFI = 1475 degree-days

Case I - Soils at elevation of possible footings of WC=10%

Interpolate between frost depth of 82.1 inches at 1500 DFI and 79.2. inches at 1400 DFI

Depth of Frost Penetration =

$$d := \frac{82.1 - 79.2}{100} \cdot 75 \cdot in + 79.2 \cdot in \qquad d = 6.781 \cdot ft$$

**Method 2 - ModBerg Software**

--- ModBerg Results ---

Project Location: Ellsworth, Maine

Air Design Freezing Index = 1256 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1005 F-days
Mean Annual Temperature = 44.6 deg F
Design Length of Freezing Season = 126 days

| Layer #:Type | t | w% | d | Cf | Cu | Kf | Ku | L |
|---|---|---|---|---|---|---|---|---|
| 1-Coarse | 57.4 | 5.0 | 120.0 | 23 | 26 | 1.0 | 1.2 | 864 |

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.

Total Depth of Frost Penetration = 4.79 ft = 57.4 in.

Recommendation: use 5.0 feet for for design

# Seismic Design Parameters

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04472
Zip Code Latitude = 44.595400
Zip Code Longitude = -068.704100
Site Class B
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.062 | PGA - Site Class B |
| 0.2 | 0.136 | Ss - Site Class B |
| 1.0 | 0.042 | S1 - Site Class B |

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04472
Zip Code Latitude = 44.595400
Zip Code Longitude = -068.704100
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.100 | As - Site Class D |
| 0.2 | 0.218 | SDs - Site Class D |
| 1.0 | 0.100 | SD1 - Site Class D |

**Seismic Design Parameters for**
**2007 AASHTO Seismic Design Guidelines**

**Purpose** - The ground motion parameters obtained in this analysis are for use with the design procedures described in AASHTO Guidelines for the Seismic Design of Highway Bridges (2007) The user may calculate seismic design parameters and response spectra (both for period and displacement), for Site Class A through E.

**Description** - This program allows the user to obtain seismic design parameters for sites in the 50 states of the United States, Puerto Rico and the U.S. Virgin Islands. In most cases the user may perform an analysis for a site by specifying location by either latitude-longitude (recommended) or zip code. However, locations in Puerto and the Virgin Islands may only be specified by latitude-longitude.

Ground motion maps are included in PDF format. These maps may be opened using a map viewer that is part of the software package.

**Data** - The 2007 AASHTO maps are based on 5% in 50 year probabilistic data from the U.S. Geological Survey data sets for the following regions: 48 conterminous states (2002), Alaska (2006), Hawaii (1998), Puerto Rico and the Virgin Islands (2003). These were the most recent data available at the time of preparation of the AASHTO maps. The AASHTO maps are labelled with a probability of exceedance of 7% in 75 years which is approximately equal to the 5% in 50 year data.

**Disclaimer** - Correct application of the data obtained from the use of this program and/or maps is the responsibility of the user. This software is not a substitute for technical knowledge of seismic design and/or analysis.

## Abutment and Wingwall Active Earth Pressure

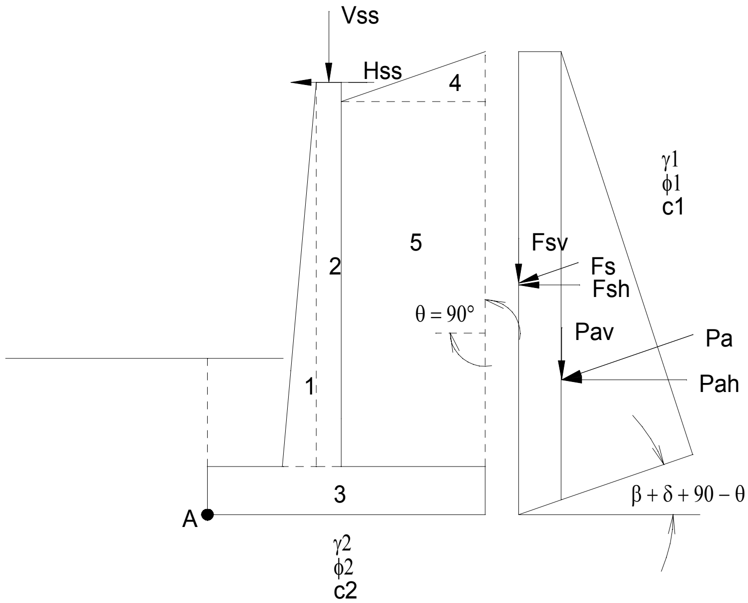
### Backfill engineering strength parameters

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

$\beta$ = 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 $\beta$ 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 $\delta$ 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 $\delta$ 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

**Passive Earth Pressure - Rankine Theory**

Bowles does not recommend use of Rankine method for Kp when B>0.

$\beta$ = Angle of fill slope to the horizontal

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

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

$K_{pslope} = 3.255$

Pp is oriented at an angle of $\beta$ to the vertical plane

**Passive Earth Pressure - Coulomb Theory**

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.

- 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.

For a smooth vertical wall with horizontal backfill $\delta = \beta = 0$ and $\theta = 90$ degrees (refer: Bowles, 5th edition, pag 596

$\theta =$ Angle of back face of wall to the horizontal

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

$\delta$ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

$\delta := \frac{2}{3} \cdot \phi_1 \qquad \delta = 0.372$

$$K_{pc} := \frac{\sin(\theta - \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 + \beta)}{\sin(\theta + \delta) \cdot \sin(\theta + \beta)}}\right)^2} \qquad K_{pc} = 7.333$$