MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

STIMSON BRIDGE
STATE ROUTE 5 OVER LITTLE OSSIPEE RIVER
WATERBORO AND LIMERICK, MAINE

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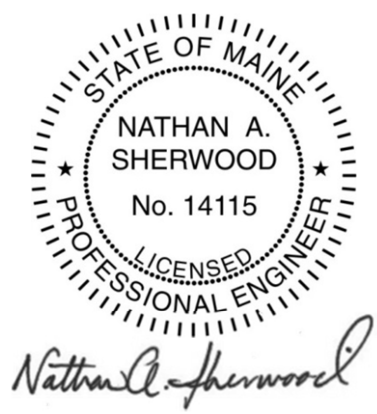

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TABLE OF CONTENTS

GEOTECHNICAL DESIGN SUMMARY 1

1.0 INTRODUCTION 6

2.0 GEOLOGIC SETTING 7

3.0 SUBSURFACE INVESTIGATION 7

4.0 LABORATORY TESTING 8

5.0 SUBSURFACE CONDITIONS 8

5.1 ABUTMENT NO. 1 – WATERBORO SIDE 8
5.2 PIER 9
5.3 ABUTMENT NO. 2 – LIMERICK SIDE 9
5.4 BEDROCK 10
5.5 GROUNDWATER 10

6.0 FOUNDATION ALTERNATIVES 11

6.1 ABUTMENTS NO. 1 AND 2 11
6.2 PIER 11

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS 12

7.1 INTEGRAL ABUTMENT H-PILES 12
7.1.1 Strength Limit State Design 13
7.1.2 Service and Extreme Limit State Design 15
7.1.3 Lateral Pile Resistance 16
7.1.4 Bedrock Socketed H-Piles 19
7.1.5 Driven Pile Resistance and Field Quality Control 20
7.2 INTEGRAL ABUTMENT DESIGN 21
7.3 WINGWALLS 22
7.4 SPREAD FOOTING/CONCRETE SEAL ON BEDROCK 23
7.5 FACTORED BEARING RESISTANCES FOR PIER 23
7.6 PIER DESIGN 23
7.7 PIER SPREAD FOOTING SUBGRADE PREPARATION 25
7.8 GROUND SETTLEMENT 25
7.9 FROST PROTECTION 26
7.10 SCOUR AND RIPRAP 26
7.11 SEISMIC DESIGN PARAMETERS 27
7.12 CONSTRUCTION CONSIDERATIONS 28

8.0 CLOSURE 29

TABLES

Table 1 - Summary of Depth to Bedrock, Top of Bedrock Elevations, and RQD
Table 2 - Estimated Pile Lengths for Integral Abutments No. 1 and No. 2
Table 3 - Factored Axial Compressive Resistances of Driven Piles at the Strength Limit State
Table 4 - Factored Axial Compressive Resistances of Driven Piles at the Service and Extreme Limit States
Table 5 - Soil Parameters for the Generation of Soil-Resistance (p-y) Curves at Abutment No. 1
Table 6 - Soil Parameters for the Generation of Soil-Resistance (p-y) Curves at Abutment No. 2
Table 7 - Bedrock Parameters for the Generation of Bedrock-Resistance (p-y) Curves at Abutments
Table 8 - Static Geotechnical Resistances of H-Piles Installed in Bedrock Sockets (No Steel Plate Installed Across Pile Tip)
Table 9 - Static Geotechnical Resistances of H-Piles Installed in Bedrock Sockets (*Steel Plate Installed Across Pile Tip*)
Table 10 - Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments
Table 11 - Factored Bearing Resistances for Service, Strength, and Extreme Limit State Design
Table 12 - Seismic Design Parameters

SHEETS

Sheet 1 - Location Plan
Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile
Sheet 3 - Boring Logs

APPENDICES

Appendix A - Boring Logs
Appendix B - Laboratory Test Results
Appendix C - Calculations
Appendix D - Special Provision

GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information, provide geotechnical design considerations, and provide geotechnical design recommendations for the replacement of the Stimson Bridge which carries State Route 5 over the Little Ossipee River between the towns of Waterboro and Limerick in Maine.

The proposed bridge will be a 168-foot-long, two-span, steel girder superstructure supported on integral abutments and a center pier. The proposed integral abutments will be supported by H-piles that are driven to bedrock or installed in bedrock sockets. The pier will consist of a mass reinforced concrete shaft founded on a spread footing and concrete seal bearing on competent bedrock. The following summarized geotechnical design considerations and recommendations are discussed further in Section 7.0.

Integral Abutment H-piles

The H-piles shall be end bearing and driven to the required resistance on bedrock or constructed in bedrock sockets. The H-piles shall be designed for the service, strength, and extreme limit states. It is recommended that lateral pile resistance analyses using LPile® Plus 5.0 (LPile) be performed to determine the pile lengths required to prevent translation of the pile tip and to evaluate the pile stresses due to combined axial loads, flexure, and thermal displacements. We understand that the Structural Engineer (Hoyle, Tanner & Associates, Inc., referred to herein as HTA) will perform the LPile analyses using the soil and bedrock parameters presented in Section 7.1.3. The structural resistance of the piles should then be evaluated for structural compliance with the interaction equation, by HTA.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and dynamic pile tests with signal matching at each abutment. The first pile driven at each abutment shall be dynamically tested to confirm nominal pile resistance and verify preliminary stopping criteria developed by the Contractor in the wave equation analysis. 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, φ_{dyn}, of 0.65.

Bedrock Socketed H-Piles

To satisfy requirements for fixity, piles at the abutments may be installed in drilled bedrock sockets. A bedrock socket length should be selected such that: 1) the pile tips are installed 1 to 5 feet beyond the pile length required to achieve fixity and 2) the piles have adequate free length to control bending moments and limit stresses in the pile. The pile tips should be end bearing on bedrock and be “fixed” in the bedrock sockets with a nominal 2-foot-thick zone (“plug”) of concrete placed at the bottom of the bedrock socket.

The nominal static geotechnical resistances of steel H-piles were computed using the Intact Rock Method (IRM) proposed by Sandford (2013) and based on Rowe and Armitage (1987b) for bearing resistance on bedrock. The factored static geotechnical resistances for two H-pile sections are presented in Section 7.1.4.

Integral Abutment Design

Integral abutments shall be designed for all relevant service, strength, and extreme limit states and load combinations. Calculation of passive earth pressures for integral abutment design shall assume a Coulomb theory passive earth pressure coefficient, K_p, of 6.73. If the ratio of the calculated lateral abutment movement to abutment height (y/H) is less than 0.005, HTA may use a Rankine theory passive earth pressure coefficient, K_p, of 3.25. For purposes of the integral abutment backwall reinforcing steel design, HTA may use a maximum load factor, γ_{EH}, of 1.50 to calculate factored passive earth pressures.

The abutment design shall include a drainage system behind the abutment to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the wingwalls. The approach slab shall be positively connected to the integral abutment. Additional lateral earth pressures due to construction surcharges or live load surcharges are required if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted.

Wingwalls

Wingwalls should preferably be straight, cantilevered extension wings not to exceed 10 feet in length. Design wingwall reinforcement for the passive earth pressure with results on the back face of the wall when the bridge expands using the Coulomb theory passive earth pressure coefficient and a passive earth pressure load factor, γ_{EH}, of 1.5. The design of in-line cantilevered wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off of the abutment.

Spread Footing/Concrete Seal on Bedrock

The proposed pier will be founded on a spread footing/concrete seal constructed on competent bedrock. The approximate depth to bedrock and top of bedrock elevation[1] encountered in the pier boring is presented in Table 1 (Section 5.4).

[1] All elevations presented herein reference the North American Vertical Datum 1988 (NAVD88)

Factored Bearing Resistances for Pier

When analyzing the service, strength, and extreme limit state factored load combinations, the following factored bearing resistances shall be used to design the spread footing and concrete seal on competent bedrock:

- Service Limit State – 20 kips per square foot (ksf); assuming settlement will be limited to 1 inch;
- Strength Limit State – 20 ksf; and
- Extreme Limit State – 36 ksf.

Pier Design

The pier shall be designed for all applicable load combinations for all relevant service, strength, and extreme limit states. The pier shall be designed to transmit all loads from the superstructure and the self-weight of the pier to the spread footing and concrete seal.

The service limit state design analyses shall consider settlement, horizontal movement, bearing resistance, sliding, and eccentricity. The strength limit state design shall consider bearing resistance, eccentricity (overturning), failure by sliding, and reinforced concrete structural failure. For the extreme limit state design, analyses shall consider bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice, and seismic forces. Anchoring of the spread footing to the concrete seal is required by the Maine Department of Transportation Bridge Design Guide. Rock anchors/dowels may be used to resist sliding forces at the base of the spread footing and concrete seal.

Pier Spread Footing Subgrade Preparation

The pier spread footing/concrete seal subgrade shall consist of competent bedrock. The nature, slope, and degree of fracturing in the bedrock bearing surface will not be evident until the foundation excavation is completed. Regardless of the type of footing excavation (submerged or in-the-dry), the bedrock surface shall be cleared of all fractured and loose bedrock and soil to expose competent bedrock.

Portions, or all of the pier foundation excavation, may be submerged. The Contractor shall prepare and submit a written procedure for cleaning and inspecting the bedrock subgrade in accordance with Section 5.11 of the Standard Specifications. If bedrock slopes steeper than 4:1 (horizontal:vertical) at the spread footing subgrade elevation, the bedrock shall be benched to create level steps or excavated to provide a completely level bearing surface.

Ground Settlement

No significant new fills are proposed at the bridge approaches, but it is anticipated some minimal modifications to the existing vertical profile will occur. Elastic settlements due to these modifications are anticipated to be small and will occur relatively quickly. Post-construction induced settlement will be minimal. Any settlement of the abutments will be due to axial shortening of the foundation piles and is anticipated to be less than ½ inch.

Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4 feet for frost protection. For foundations bearing on bedrock, heave due to frost is not a design issue and no requirements for embedment for frost protection are necessary. Any foundation bearing on soil shall be founded a minimum of 5.5 feet beneath the finished exterior grade for frost penetration.

Scour and Riprap

The consequences of changes in foundation conditions resulting from riverbed material loss due to the design flood for scour shall be considered for all foundations at the service and strength limit states. For scour protection of the pile-supported abutments, bridge approach slopes and slopes at abutments shall be armored with 3 feet of plain riprap or 4 feet of heavy riprap. Stone riprap shall be placed at a maximum slope of 1.75:1.

For scour protection of the pier footing, the bottom of the concrete seals should be constructed directly on competent bedrock that is cleaned of all weathered, loose, and potentially erodible or scourable bedrock.

Seismic Design

Seismic analysis is not required for multispan bridges in Seismic Zone 1. However, superstructure connections and minimum support length requirements shall be designed per American Association of State Highway and Transportation Officials, Load and Resistance Factor Design, Bridge Design Specifications, 7th Edition, 2014 with 2016 interims.

Construction Considerations

Construction of the integral abutments will require pile driving and/or bedrock coring. Temporary earth support systems may be required to permit construction of pile foundations at the proposed abutments.

There is potential that the existing abutments, if not entirely removed, may obstruct pile driving or bedrock coring operations. The Contractor shall be responsible for excavating those portions of the existing abutments and footings that conflict with piles by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Excavation by these methods shall be made incidental to related pay items.

Occasional cobbles were encountered in the fill beneath the bridge approaches. There is potential for these obstructions to impede the driving of sheet piles, H-piles, or coring bedrock sockets. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, or spudding. Alternative methods to clear obstructions may be used as approved by the Resident. Care should take to install piles within allowable tolerances.

Construction of the pier will require a cofferdam to support overburden soils and control river flow. The bedrock shall be cleared of all loose fractured bedrock, loose decomposed bedrock, and soil. The seal foundation subgrade should be confirmed to be relatively level. If bedrock is observed to slope steeper than 4:1, the bedrock should be benched to create level steps or excavated to be completely level. The condition of the bedrock surface prior to placing tremie-seal concrete should be inspected with the use of remote underwater cameras, divers, or other methods approved by the Resident. The cleanliness and condition of the final bedrock surface for tremie-seals shall be approved by the Resident prior to placement of seal concrete.

Cobbles were encountered in the river alluvium at the location of the proposed pier. There is potential for these obstructions to impact or impede the driving of sheeting and cofferdam excavation.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information, provide geotechnical design considerations, and provide geotechnical design recommendations for the replacement of the Stimson Bridge which carries State Route 5 over the Little Ossipee River between the towns of Waterboro and Limerick in Maine.

The existing Stimson Bridge was built in 1931 and is comprised of three, 45-foot-long simple span concrete tee-beams, for a total length of 135 feet. The bridge substructure system consists of mass concrete abutments supported on spread footings bearing on gravel and mass concrete piers supported on spread footings bearing on an unknown material (likely granular soils). The substructure elements are differentially skewed in order to incorporate the kinked superstructure.

The bridge superstructure was rehabilitated in 1996 and 1997. The abutments and piers have not been rehabilitated over the life of the bridge and are showing signs of significant deterioration. The substructure system is in overall poor condition. The northern abutment has large areas of spalled concrete with exposed reinforcing steel at bearing areas. The abutments generally have minor cracking with some concrete spalls. The piers have large areas of severe concrete spalling and delaminations with exposed reinforcing steel at the bearing areas. The piers generally have areas of moderate cracking, staining, and spalling.

The Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection report (dated 17 November 2014) assigned the substructures a condition rating of 4 (poor) with a Bridge Sufficiency Rating of 14.1. The Inspection Notes state that the bridge is in overall poor condition with “moderate/isolated heavy areas of deterioration” of the substructures’ concrete elements.

The proposed bridge consists of a 168-foot-long, two span superstructure, comprised of weathering steel welded plate I-girders. The bridge will have an approximate 27.5 degree skew. A two span option reduces the required superstructure depth for the bridge by shortening the span lengths, resulting in an increase of hydraulic clearance.

As shown in Table 1, bedrock was encountered at depths of 25.7 and 19.2 feet below ground surface (bgs) at Abutments No. 1 and 2, respectively and at 3.9 feet bgs in the river (pier location) during the subsurface investigation. As a result of the relatively shallow bedrock depths, the substructures for the proposed bridge consist of integral abutments supported on H-piles driven to or installed in bedrock and a cast-in-place reinforced concrete mass shaft supported by a spread footing and concrete seal bearing on competent bedrock.

An off-alignment temporary bridge maintaining with traffic signals will be constructed downstream of the existing bridge to maintain one-way alternating traffic during the construction of the proposed bridge.

2.0 GEOLOGIC SETTING

The Stimson Bridge crosses the Little Ossipee River as shown on Sheet 1 – Location Map. The Little Ossipee River is approximately 120 feet wide at the location of Stimson Bridge, which is located approximately 14.3 miles upstream of its confluence with the Saco River.

According to the Surficial Geology Map, Limerick Quadrangle, Maine, Open-File No. 99-89, 1999 by the Maine Geological Survey (MGS), the surficial soils in site vicinity consist of river alluvium and glaciolacustrine delta deposits. River alluvium deposits generally consist of fine- to coarse-grained sand, silt and clay with some gravel, and organic matter in areas. The unit is generally deposited in flood plains of rivers and brooks. Glaciolacustrine delta deposits generally consist of gravel and very well sorted sand and are likely associated with Glacial Lake Arrowhead near Limerick and Ossipee Mills.

According to the MGS Bedrock Geologic Map of Maine (1985) the bedrock at the site is identified as interbedded pelite and limestone and/or dolostone, a lower member of the Rindgemere Formation that dates back to the Devonian – Silurian Age.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were investigated by drilling the following three borings: 1) BB-WLLOR-101 was drilled near the location of the proposed Abutment No. 1 (Waterboro side), 2) BB-WLLOR-102 was drilled near the location of the proposed pier, and 3) BB-WLLOR-103 was drilled near the location of proposed Abutment No. 2 (Limerick side). Two additional power auger borings will be drilled at a later date at the proposed integral abutment locations to further define the top of bedrock elevations across the abutments. The boring locations and an interpretive subsurface profile depicting the soil and bedrock stratigraphy across the site are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The borings were drilled between 14 and 15 April 2015 by the MaineDOT Materials Testing and Exploration drill crew using a trailer-mounted drill rig. Details and sampling methods used, field data obtained, soil, and bedrock conditions encountered are presented on Sheet 3 – Boring Logs and in Appendix A – Boring Logs. The borings were drilled using solid stem auger, cased wash boring and rock coring techniques. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler was driven 24 inches and the hammer blows for each 6-inch interval of penetration were recorded. The standard penetration resistance value (N-value) is the sum of the blows for the second and third intervals of the 24-inch drive. The MaineDOT drill rig is equipped with a 140-pound, automatic hammer falling 30 inches. The hammer was calibrated per ASTM D 4633-05 "Standard Test Method for Energy Measurement for Dynamic Penetrometers". The MaineDOT automatic hammer was calibrated in October 2014 and was found to deliver approximately 51.3 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 0.908 to the raw field N-values. The hammer efficiency factor (0.908), the raw field N-values, and the corrected N-values (N_{60}) are shown on the boring logs.

The bedrock was cored using an NQ 2-inch core barrel. The Rock Quality Designation (RQD) of each bedrock core was calculated and is presented on the boring logs. The Geotechnical Engineer selected the boring locations and drilling methods, designated the type and depth of sampling techniques, and identified field and laboratory testing requirements. The MaineDOT subsurface inspector, certified by the Northeast Transportation Technical Certification Program, logged the subsurface conditions encountered at the borings in accordance with the MaineDOT Key to Soil and Rock Descriptions, provided in Appendix A – Boring Logs. The borings were located in the field by tape after the completion of drilling and later confirmed by MaineDOT Survey.

4.0 LABORATORY TESTING

Soil and rock samples obtained during the subsurface investigation were examined in our Bangor, Maine office to confirm the field classifications and samples were selected for laboratory testing. A laboratory testing program was conducted on selected soil samples recovered from borings to assist in soil classification, evaluate engineering properties, and for geologic assessment of the site. The laboratory testing consisted of six standard grain size analyses with natural water content measurements. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. The results of the laboratory testing are provided in Appendix B – Laboratory Test Results. Moisture content information and other soil test results are included on the boring logs on Sheet 3 – Boring Logs and in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

On the basis of our interpretation of the borings drilled by us at the site, we conclude the site is, in general, underlain by sandy fill, native sand with varying amounts of gravel and silt, gravel, and underlying bedrock. Section 5.4 presents information regarding the bedrock encountered in the borings. A brief summary description of the soil encountered at each substructure is presented in Sections 5.1 through 5.3 and Section 5.4 (bedrock).

5.1 Abutment No. 1 – Waterboro Side

In general, sandy fill, native soil, and bedrock were encountered at BB-WLLOR-101, which is located near the proposed Abutment No. 1.

Fill

A layer of fill was encountered immediately beneath the pavement in BB-WLLOR-101. The fill layer encountered at BB-WLLOR-101 was approximately 8.6-feet-thick. The fill consisted of brown, moist to wet, well-graded sand, some gravel, trace silt, with occasional cobbles. Corrected SPT N-values in the fill ranged from 24 to 27 blows per foot (bpf) indicating that the soil is medium dense in consistency. The water content of one fill sample tested was 3.7 percent. One grain size analysis conducted on a fill sample indicates the fill is classified as an A-1-b soil by the American Association of State Highway and Transportation Officials (AASHTO) Soil Classification System and as an SW-SM soil by the Unified Soil Classification System (USCS).

Native Soil (River Alluvium and/or Glaciolacustrine Delta deposits)

The alluvial or glaciolacustrine deposit encountered at BB-WLLOR-101 was 16.7-feet-thick. The deposit consisted of grey-brown and grey, wet, sand, some gravel, and little silt. Corrected SPT N-values in the deposit ranged from 15 to 50 bpf indicating that the soil is medium dense to very dense in consistency. Water contents obtained from tested samples ranged from approximately 12.6 to 13.5 percent. Grain size analyses conducted on samples of the deposit indicate the soils are classified as an A-1-b or A-2-4 to A-2-7 soil by the AASHTO Soil Classification System and as an SM soil by the USCS.

5.2 Pier

A layer of river alluvium was encountered overlying bedrock at BB-WLLOR-102, which is located near the proposed pier. The thickness of the alluvium at the boring location was approximately 3.9-feet-thick. The alluvium consisted of brown, wet, well-graded gravel, some sand, trace silt, and occasional cobbles. The corrected SPT N-value in the deposit was 80 bpf indicating that the soil is very dense in consistency. The water content of one sample tested was 9.2 percent. One grain size analysis conducted on a sample indicates the alluvium is classified as an A-1-a soil by the AASHTO Soil Classification System and as a GW-GM soil by the USCS.

5.3 Abutment No. 2 – Limerick Side

In general, sandy fill, native soil, and bedrock were encountered at BB-WLLOR-103, which is located near the proposed Abutment No. 2.

Fill

A layer of fill was encountered beneath the pavement in BB-WLLOR-103. The thickness of the fill layer at BB-WLLOR-103 was approximately 10.2-feet-thick. The fill consisted of brown, moist, poorly-graded sand, trace gravel, and trace silt. Corrected SPT N-values in the fill ranged from 12 to 23 bpf indicating that the soil is medium dense in consistency. The water content of one sample tested was 3.3 percent. One grain size analysis conducted on a sample indicates the fill is classified as an A-1-b soil by the AASHTO Soil Classification System and as an SP-SM soil by the USCS.

Native Soil (River Alluvium and/or Glaciolacustrine Delta deposits)

A layer of river alluvium and glaciolacustrine deposits was encountered beneath the fill. The thickness of the deposit encountered at BB-WLLOR-103 was 8.7-feet-thick. The deposit consisted of brown, wet, sand, some gravel, and little silt. Corrected SPT N-values in the deposit ranged from 15 to 50 bpf indicating that the soil is medium dense to very dense in consistency. Water contents obtained from tested samples ranged from approximately 8.5 to 22.8 percent. Grain size analyses conducted on samples of the deposit indicate the soil is classified as an A-1-b soil by the AASHTO Soil Classification System and as an SM soil by the USCS.

5.4 Bedrock

Bedrock was encountered and cored at all boring locations. Table 1 summarizes the approximate depths to bedrock, corresponding top of bedrock elevations, and RQD at the boring locations.

Boring No. *Proposed Substructure Name*	Approximate Depth to Bedrock, bgs (feet)	Approximate Top of Bedrock Elevation (feet)	RQD[2] (R1, R2) (%)
BB-WLLOR-101 *Abutment No. 1*	25.7	295.8	22, 48
BB-WLLOR-102 *Pier*	3.9	298.6	10, 46
BB-WLLOR-103 *Abutment No. 2*	19.2	301.8	52, 40

Table 1. Summary of Approximate Depth to Bedrock, Approximate Top of Bedrock Elevations, and RQD

In general, the bedrock in the borings is identified as red-brown to grey, poorly bedded garnet-mica schist and pegmatite/migmatite with muscovite, feldspar, and quartz present, moderately hard, moderately fractured, and moderately weathered. The bedrock is identified as part of the Lower Member of the Rindgemere Formation. The RQD of the bedrock ranged from approximately 10 to 52 percent indicating a rock mass quality of very poor to fair.

5.5 Groundwater

Groundwater was not observed during the subsurface investigation. The existing ordinary high water elevation ($Q_{1.1}$) at the site is approximately Elevation 307.2 feet[3]. Groundwater levels will fluctuate with precipitation, seasonal changes, runoff, and construction activity.

[2] R1 is the first bedrock run length and R2 is the second length.
[3] Preliminary Design Report, 15 October 2015, page 23.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered at the site and discussions with the Structural Engineer, Hoyle, Tanner and Associates, Inc. (HTA), regarding expected loading conditions, the following foundation alternatives for the substructures were considered feasible as discussed in Sections 6.1 and 6.2.

6.1 Abutments No. 1 and 2

Bedrock was encountered at Elevation 295.8 feet and 301.8 feet at the proposed Abutment No. 1 and Abutment No. 2, respectively. The historical bridge plans[4] indicate the existing abutments are likely founded on native granular soil. The design team considered supporting the integral abutments on the following three types of piles:

- H-piles;
- Steel N80 casings spun into bedrock; and
- Micropiles.

However, based on previous bridge replacement projects in the State with similar subsurface conditions and discussions with HTA regarding expected loading conditions, it was decided that the most cost effective and practical deep foundation type was to support the integral abutments on H-piles driven to bedrock or installed in bedrock sockets. To satisfy requirements for fixity and/or elastic behavior, piles may require installation in bedrock sockets, as detailed in Section 7.1.4.

6.2 Pier

Bedrock was encountered at Elevation 298.6 feet at the proposed pier location. Due to the shallow depth to bedrock in this location, the pier will consist of a mass reinforced concrete shaft that is constructed on a spread footing that will bear directly on either competent bedrock or on a concrete seal bearing on competent bedrock.

[4] "Stimson Bridge over Little Ossipee River between the Towns of Limerick and Waterboro, York County, Survey Plan," by Maine Highway Commission, Bridge Division, Sheet 1 of 8, dated 22 May 1930.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

Geotechnical design considerations and recommendations regarding the integral abutments, H-piles, wingwalls, pier, scour mitigation, frost protection, seismic design, and construction are presented in the following sections and are in accordance with AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 7th Edition, 2014 with 2016 interims and relevant MaineDOT Bridge Design Guide (BDG) sections.

7.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be founded on a single row of H-piles. The piles shall be end bearing and driven to the required resistance on bedrock or installed in bedrock sockets. Per discussions with HTA, the piles may be either HP 14x89 or HP 14x117 sections, depending on the factored design axial loads and the ability to effectively resist lateral loads. The piles shall be 50 kips per square inch (ksi), Grade A572 steel. The piles should be oriented for weak axis bending. If the H-piles are driven, the piles shall be fitted with pile tips conforming to MaineDOT Standard Specification 711.10 to protect section ends, improve friction, and increase bearing area at the pile tip.

Due to the relatively shallow depth to bedrock at both abutments, if the results of the LPile analyses indicate that the H-piles do not achieve fixity or require additional pile free length to control bending moments and stresses; the piles may need to be installed in bedrock sockets (see Section 7.1.4). Pile lengths at the proposed abutments may be estimated based on the information presented in Table 2.

Substructure	Estimated Bottom Elevation of Proposed Abutment (feet)	Interpolated Top of Bedrock Elevation at Proposed Centerline[5] (feet)	Estimated Pile Lengths (feet)
Abutment No. 1	311.0	295.8	15.2
Abutment No. 2	312.0	301.8	10.2

Table 2. Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths do not account for embedment in the abutment, penetration into bedrock, embedment in bedrock sockets, locations where bedrock may be deeper or shallower than that encountered in the borings, damaged pile, the additional 5 feet of pile required for dynamic testing instrumentation (per ASTM D4945), or additional pile length needed to accommodate the Contractor's leads and driving equipment.

[5] The top of bedrock elevations may change after the additional power auger borings are completed.

7.1.1 Strength Limit State Design

The design of pile foundations bearing on or within bedrock for the strength limit state shall consider the following:

- The axial compressive geotechnical resistance of individual piles;
- The axial compressive structural resistance of individual piles;
- The axial compressive drivability resistance of individual piles; and
- The structural resistance of individual piles in combined axial compression and flexure.

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor, ϕ_c, of 0.60 for good driving conditions, shall be applied to the structural compressive resistance of the pile. Since the piles will be subjected to lateral loading, the piles shall also be checked for resistance against combined axial compression and flexure per LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor, ϕ_c, of 0.70 and the flexural resistance factor, ϕ_f, of 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Equation 6.9.2.2-1 or -2).

We understand the piles will be analyzed by HTA for determination of unbraced lengths and fixity using the LPile software. The calculated unbraced lengths shall be used to analyze the piles in combined axial compression and flexure resistance per LRFD Articles 6.9.2.2 and 6.15.2.

Structural Resistance

The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the axial compressive structural resistance of two H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 for good driving conditions. The unbraced pile lengths (ℓ) and effective length factors (K) in these evaluations have been assumed. It is the responsibility of HTA to calculate the nominal axial compressive structural resistance based on unbraced lengths and effective length factors determined from the LPile analyses.

Geotechnical Resistance

The nominal axial compressive geotechnical resistance at the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3, which states that the nominal axial compressive resistance of piles driven to end bearing on hard bedrock shall not exceed the axial compressive structural resistances obtained from LRFD Article 6.9.4.1 with a resistance factor, ϕ_c, of 0.50 for severe driving conditions. The factored axial compressive geotechnical resistances for piles driven to hard bedrock are presented in Table 3.

Drivability Analyses

Drivability analyses were performed using the GRLWEAP 2003 software to determine the axial compressive drivability resistance that might be achieved using a Delmag 19-42 diesel hammer. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The axial compressive drivability resistances were calculated using a resistance factor, φ_{dyn}, of 0.65 for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

A summary of the calculated factored axial compressive structural, geotechnical, and drivability resistances of two H-pile sections for the strength limit state is presented in Table 3. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Strength Limit State Factored Axial Compressive Pile Resistances			
	Structural Resistance[6] ϕ_c=0.60 (kips)	Geotechnical Resistance[7] φ_c=0.50 (kips)	Drivability Resistance φ_{dyn}=0.65 (kips)	Governing Resistance (kips)
HP 14x89	782	652	341	341
HP 14x117	1,031	859	471	471

Table 3. Factored Axial Compressive Resistances of Driven Piles at the Strength Limit State

Local experience supports the estimated factored resistances from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the strength limit state not exceed the governing resistances presented in the rightmost column "Governing Resistance (kips)" in Table 3.

[6] Based on the preliminary assumption that the unbraced length (ℓ) is 1-foot and the effective length factor (K) is 1.2.
[7] Based on guidance in LRFD Article 10.7.3.2.3, *Piles Driven to Hard Rock.*

7.1.2 Service and Extreme Limit State Design

The design of piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall global stability, and pile group movements/stability considering changes in soil conditions due to scour at the check flood (Q_{500}). For the service limit state, a resistance factor, ϕ, of 1.0 should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination with a resistance factor, ϕ, of 0.65.

The design of piles at the extreme limit state shall consider pile axial compressive resistance, overall global stability of the pile group, pile failure by uplift in tension, and structural failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood (Q_{500}) can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as ϕ = 1.0, with the exception of uplift of piles, for which the resistance factor, ϕ_{uplift}, is 0.80 per LRFD Article 10.5.5.3.2.

The nominal axial compressive geotechnical resistances at the service and extreme limit states were calculated per LRFD Article 10.7.3.2.3. The calculated factored axial compressive structural, geotechnical, and drivability resistances of two H-pile sections for the service and extreme limit states are presented in Table 4. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Service and Extreme Limit States Factored Axial Compressive Pile Resistances			
	Structural Resistance (normal conditions)[8] ϕ_c=1.0 (kips)	Geotechnical Resistance φ_c=1.0[9] (kips)	Drivability Resistance φ_{dyn} = 1.0 (kips)	Governing Resistance (kips)
HP 14x89	1,303	1,303	525	525
HP 14x117	1,718	1,718	725	725

Table 4. Factored Axial Compressive Resistances for Driven Piles for Service and Extreme Limit States

Local experience supports the estimated factored resistances from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the service and extreme

[8] Based on the preliminary assumption that the unbraced length (ℓ) is 1-foot and the effective length factor (K) is 1.2.
[9] Based on guidance in LRFD Article 10.7.3.2.3, *Piles Driven to Hard Rock.*

limit states shall not exceed the governing resistances presented in the rightmost column "Governing Resistance (kips)" in Table 4.

7.1.3 Lateral Pile Resistance

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects in accordance with LRFD Article 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip shall be confirmed with soil-structure interaction analyses.

We understand that HTA will perform a series of lateral pile resistance analyses to evaluate pile behavior at both abutments using LPile software with pile head deflections, design rotations, and axial loads. HTA should utilize the results of the LPile analyses to recalculate axial compressive structural pile resistances based on unbraced pile segments and verify that pile bending stresses and total stresses do not exceed allowable stresses.

Geotechnical parameters used for the generation of soil/bedrock-resistance (p-y) curves in lateral pile analyses are presented in Tables 5 through 7. The models developed for LPile analyses shall emulate the soil and bedrock at the site by using the recommended properties (presented in Tables 5 through 7) and using appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed.

Description of Soil Layer	Approximate Top and Bottom Elevations of Soil Layer (feet)	Above/Below Groundwater Table	Effective Unit Weight, γ' lbs/ft^3 (lbs/in^3)	Soil Modulus, k_s (lbs/in^3)	Internal Angle of Friction, ϕ' (degrees)
Medium dense, SAND, (Fill)	321.1 – 316.1	Above	117 (0.068)	90	33
Medium dense, SAND, (Fill)	316.1 – 312.5	Below	55 (0.032)	60	33
Medium dense, SAND, (Native)	312.5 – 303.5	Below	48 (0.028)	50	32
Dense, SAND, (Native)	303.5 – 295.8	Below	80 (0.046)	125	38

Table 5. Soil Parameters for the Generation of Soil-Resistance (p-y) Curves at Abutment No. 1

Description of Soil Layer	Approximate Top and Bottom Elevations of Soil Layer (feet)	Above/Below Groundwater Table	Effective Unit Weight, γ' lbs/ft^3 (lbs/in^3)	Soil Modulus, k_s (lbs/in^3)	Internal Angle of Friction, ϕ' (degrees)
Medium dense, SAND, (Fill)	320.7 – 315.7	Above	117 (0.068)	90	33
Medium dense, SAND, (Fill)	315.7 – 310.5	Below	50 (0.029)	50	31
Medium dense, SAND, (Native)	310.5 – 307.0	Below	48 (0.028)	50	32
Dense, SAND, (Native)	307.0 – 301.8	Below	80 (0.046)	125	38

Table 6. Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2

Subsurface Material	Model Name and (Reference)	Uniaxial Compressive Strength, UCS (lbs/in^2)	Strain Parameter, k_{rm} (unitless)	Young's Modulus, E_r (lbs/in^2)	RQD (%)	Effective Unit Weight, γ' (lbs/in^3)
Bedrock	Weak Rock (Reese, 1997)	6,943	0.0005	3,446,400	41	0.066
Bedrock	Strong Rock (Vuggy Limestone)	6,943	-	-	-	0.066

Table 7. Bedrock Parameters for Generation of Bedrock-Resistance (p-y) Curves at Abutments

7.1.4 Bedrock Socketed H-Piles

To satisfy requirements for fixity, piles at the abutments may require installation in bedrock sockets. A bedrock socket length should be selected such that: 1) the pile tips are installed 1 to 5 feet beyond the pile length required to achieve fixity and 2) the piles have adequate free length to control bending moments and limit stresses in the pile. The pile tips should be end bearing on bedrock and be "fixed" in the bedrock sockets with a nominal 2-foot-thick zone ("plug") of concrete placed at the bottom of the bedrock socket.

The nominal static (i.e. pile is not driven) geotechnical resistances of two H-pile sections were computed using the Intact Rock Method (IRM) proposed by Sandford (2013) and based on Rowe and Armitage (1987b) for bearing resistance on bedrock. The resistances presented in Table 8 were computed using a resistance factor, φ_{stat}, of 0.45.

Per discussions with HTA, the maximum factored pile load is approximately 315 kips. As shown in Table 8, if the piles are installed in bedrock sockets, the factored static geotechnical pile resistances at the strength limit state do not achieve the required maximum factored pile load of 315 kips. Therefore, a steel plate may be welded across each pile tip (i.e. from flange-to-flange) to provide an increased bearing surface area; which in turn, will yield increased factored static geotechnical pile resistances that will exceed the required maximum factored pile load of 315 kips, as shown in Table 9. The selection of the steel plate thickness and welding detail shall be the responsibility of HTA. Supporting calculations are provided in Appendix C – Calculations.

The maximum applied factored axial pile load should not exceed the governing factored pile resistances shown in Table 9. Therefore, to prevent loading the H-piles beyond their structural capacities, the recommended governing resistances for H-piles installed in bedrock sockets for the strength limit state are the resistances presented in the rightmost column "Factored Governing Resistance (kips)" in Table 9. The governing resistances are equivalent to the nominal structural resistances of the pile sections.

Bedrock sockets may be drilled using rotary duplex methods with down-the-hole hammers, rotary percussive methods, or solid rock coring methods. The bedrock socket should have a diameter of at least 2 inches greater than the diagonal H-pile section dimension. Bedrock sockets shall be constructed to have a clean, planar bottom. Once the bedrock socket is drilled and the socket is adequately cleaned out, a 2-foot-thick concrete plug shall be placed on the bottom of the bedrock socket and the H-pile shall be installed in the bedrock socket and the annular space between the sidewall of the lower portion of the bedrock socket and the H-pile shall be tremie-filled with Class A concrete. The annular space in the portion of the bedrock socket from the concrete plug to the top of the bedrock socket shall be backfilled with Type C Underwater Backfill to achieve the free length of pile required for adequate pile behavior.

Pile Section	Factored Static Geotechnical Pile Resistance ϕ=1.0 (Service and Extreme Limit State Design) (kips)	Factored Static Geotechnical Pile Resistance φ_{stat}=0.45 (Strength Limit State Design) (kips)
HP 14x89	522	235
HP 14x117	688	310

Table 8. Static Geotechnical Resistances of H-Piles Installed in Bedrock Sockets (No Steel Plate Installed Across Pile Tip)

Pile Section	Factored Static Geotechnical Pile Resistance ϕ=1.0 (Service and Extreme Limit State Design) (kips)	Factored Static Geotechnical Pile Resistance φ_{stat}=0.45 (Strength Limit State Design) (kips)	Nominal Factored Governing Resistance ϕ=1.0 (Service and Extreme Limit State Design) (kips)	Factored Governing Resistance ϕ=0.50 (Strength Limit State Design) (kips)
HP 14x89	4,057	1,826	1,303	782
HP 14x117	4,232	1,904	1,718	1,031

Table 9. Static Geotechnical Resistance of H-Piles Installed in Bedrock Sockets (*Steel Plate Installed Across Pile Tip*)

7.1.5 Driven Pile Resistance and Field Quality Control

For piles that are not installed in bedrock sockets, but are driven to bear on or within bedrock, the contract plans shall require the Contractor to perform a wave equation analysis of the proposed pile-hammer system and dynamic pile tests with signal matching. The first pile driven at each abutment (without a bedrock socket) shall be dynamically tested to confirm nominal pile resistance and verify preliminary stopping criteria developed by the Contractor in the wave equation analysis. The pile driving acceptance criteria developed shall prevent pile damage. Restrike tests or additional pile tests will be required as part of the pile quality and assurance program should pile behavior vary radically between adjacent piles, the pile behavior indicates a pile is refusing on a boulder or cobble layer above bedrock or is not firmly seated on bedrock, or if piles "walk" out of position.

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, ϕ_{dyn}, of 0.65. The maximum factored axial pile load should be shown on the plans.

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 and Geotechnical Engineer. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. A hammer shall be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving may be terminated when the penetration is less than ½ inch in 10 consecutive blows.

7.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant service, strength, and extreme limit states and load combinations as specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design. Strength limit state design shall also consider changes in foundation conditions and foundation resistance after scour due to the design (Q_{100}) flood.

A resistance factor, ϕ, of 1.0 shall be used to assess abutment design at the service limit state, that considers settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation shall be investigated at the Service I Load Combination with a resistance factor, ϕ, of 0.65.

Extreme limit state design of integral abutment supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit states shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor, ϕ, of 1.0.

HTA may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for abutment and wingwall backfill material and the following associated soil and engineering properties of the backfill for design:

- Internal angle of friction (ϕ') = 32 degrees;
- Total unit weight (γ) = 125 pounds per cubic foot (pcf); and
- Soil-concrete interface friction angle (δ) = 24 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equivalent to the passive earth pressure state. Calculation of passive earth pressures should assume a Coulomb theory passive earth pressure coefficient, K_p, of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure HTA may consider using the Rankine theory passive earth pressure coefficient, K_p, of 3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, HTA may use a maximum load factor, γ_{EH}, of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressures due to live load surcharge are required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) presented in Table 10.

Abutment Height (feet)	h_{eq} (feet)
5	4
10	3
greater than 20	2

Table 10. Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The abutment design shall include a drainage system behind the abutment to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the wingwalls. Weep holes, if required, shall be constructed approximately 6 inches above the riprap shelf. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

Backfill within 10 feet of the abutments and side slope fill shall conform to Granular Borrow for Underwater Backfill – MaineDOT Specification 703.19. This gradation specifies 7 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost penetration behind the abutment.

Slopes in front of the integral abutments shall be set back from the riverbank and shall be constructed with riprap and erosion control geotextile. The slopes shall not exceed 1.75:1 (horizontal:vertical) in accordance with MaineDOT Standard Detail 610(03).

7.3 Wingwalls

Wingwalls should preferably be straight, cantilevered extension wings not to exceed 10 feet in length. Design wingwall reinforcement for the passive earth pressure with results on the back face of the wall when the bridge expands using the Coulomb theory passive earth pressure and a passive earth pressure load factor, $\gamma_{EH,}$ of 1.5. The design of in-line cantilevered wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off of the abutment.

7.4 Spread Footing/Concrete Seal on Bedrock

For design purposes, the top of the bedrock elevation at the proposed pier is Elevation 298.6 feet. The nature, slope, and degree of fracturing of the bedrock bearing surface will not be evident until pier cofferdam excavation is completed. Prior to the placement of the pier concrete seal, the bedrock subgrade surface shall be cleared of all loose, fractured bedrock, and soil to expose competent bedrock.

7.5 Factored Bearing Resistances for Pier

The pier spread footing shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads is specified in LRFD Article 11.5.6. 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 factored bearing resistances at the service, strength, and extreme limit states are presented in Table 11. Supporting calculations are provided in Appendix C – Calculations.

Assumed Bearing Material	Limit State	Resistance Factor, φ_b	LRFD Reference	Factored Bearing Resistance, q_R (ksf)
Competent Bedrock	Service	1.0	Article 10.5.5.1	20[10]
	Strength	0.45	Table 10.5.5.2.2-1	20
	Extreme	0.80	Article C11.5.8	36

Table 11. Factored Bearing Resistances for Service, Strength, and Extreme Limit State Design

In no instance shall the factored bearing stress exceed the factored compressive resistance of the spread footing concrete or seal concrete, which may be taken as 30 percent of the concrete's compressive strength. No spread footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

7.6 Pier Design

The solid shaft pier shall be proportioned for all applicable load combinations in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant service, strength, and extreme limit states. The pier shall be designed to transmit the loads of the superstructure and the self-weight of the pier to the spread footing/concrete seal.

The design of the reinforced concrete pier on a spread footing/concrete seal bearing on competent bedrock at the strength limit state shall consider bearing resistance, eccentricity, failure by sliding,

[10] This factored bearing resistance is settlement-controlled to 1 inch.

and reinforced concrete structural failure. A modified strength limit state analysis shall be performed that includes the ice pressures per MaineDOT BDG Section 3.9 – Ice Loads.

For the spread footing or concrete seal bearing on competent bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 of the spread footing dimensions in either direction. The eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the base width and length.

For the service limit state, a resistance factor, ϕ, of 1.0 shall be used to assess the spread footing design for settlement, horizontal movement, bearing resistance, sliding, and eccentricity. The overall stability of the spread footing is typically investigated at the Service I Load Combination with a resistance factor, ϕ, of 0.65. Shear failure along adversely oriented joint surfaces in the bedrock mass below the spread footing is not anticipated; therefore, a global stability evaluation is not required.

Extreme limit state design checks for the pier shall include bearing resistance, eccentricity, failure by sliding, and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice, and seismic forces. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0 with the exception of bearing resistance for which a resistance factor of 0.8 shall be used. The ice pressures for Extreme Event II shall be applied at the $Q_{1.1}$ and Q_{50} elevations as defined in MaineDOT BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

For scour protection of the pier foundation, the spread footing or concrete seal shall be constructed directly on competent bedrock that is cleaned of all weathered, loose, and potentially erodible or scourable bedrock. With these precautions, strength and extreme limit state designs do not need to consider rock scour due to the design or check floods for scour.

For sliding analyses at the strength limit state, a sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of the pier founded on a spread footing or concrete seal bearing on competent bedrock assuming the spread footing subgrade will be prepared in-the-wet and some amount of sediment will remain on the bedrock surface. If the spread footing subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing concrete, a different sliding resistance factor, φ_τ, of 0.9 may be used.

Assuming that the bedrock subgrade will be prepared in-the-wet, some amount of sediment is expected to remain on the bedrock surface and the sliding computations for resistance of the pier foundation to lateral loads shall assume a maximum friction coefficient of 0.60 at the bedrock-concrete seal interface. If the bedrock subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance of the pier foundation to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

Anchorage of the spread footing to the concrete seal is required by MaineDOT BDG Section 5.2.2. The dowels shall be drilled and grouted into the concrete seal after dewatering and prior to placing the concrete. Anchorage of the foundation concrete or of the concrete seal to the bedrock may also be required to resist sliding forces and improve stability. The dowels shall be drilled and grouted into the concrete seal after dewatering and prior to placing the foundation concrete.

Site conditions may warrant that the pier nose be designed to effectively break up or deflect floating ice or debris. Facing the pier nose with a steel plate/angle or facing the pier with granite should be considered.

7.7 Pier Spread Footing Subgrade Preparation

The spread footing or concrete seal subgrade shall consist of competent bedrock. The nature, slope, and degree of fracturing of the bearing surface will not be evident until the foundation excavation for the pier is completed. Regardless of the type of foundation construction (submerged or in-the-dry), the bearing surface shall be cleared of all fractured and loose bedrock and soil to expose competent bedrock. If the spread footing or concrete seal is constructed in-the-dry, any irregularities in the existing bearing surface or irregularities created during the excavation process shall be backfilled with unreinforced concrete to the bottom of the spread footing elevation. It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water shall be controlled by pumping from sumps. The Contractor shall maintain the excavation so that the foundation is constructed in-the-dry. The cleanliness and condition of the bearing surface shall be approved by the Resident prior to placing concrete.

Portions, or all, of the pier foundation may be submerged. The Contractor shall prepare and submit a written procedure for cleaning and inspection of the bedrock subgrade to the Resident in accordance with Section 511 of the MaineDOT Standard Specifications. If bedrock slopes steeper than 4:1 at the subgrade elevation, the bedrock shall be benched to create level steps or excavated to provide a completely level bearing surface. The bearing surface may be stepped along the centerline of the spread footing to create a workable bearing surface. The bottom of the spread footing or concrete seal elevation may vary based on the presence of fractured bedrock and the variability of the bedrock surface.

Submerged or in-the-dry excavation of highly sloped and/or loose, fractured bedrock may be completed using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.7 of the MaineDOT Standard Specifications. It is also recommended that the Contractor conduct pre- and post-blast surveys, as well as blast vibration monitoring at nearby structures in accordance with industry standards at the time of the blast.

7.8 Ground Settlement

The sandy fill and native silty sand encountered in the borings located behind the abutments will undergo elastic compression when a load greater than the existing overburden pressure is applied. No significant new fills are proposed at the bridge approaches, but it is anticipated some minimal modifications to the existing vertical profile will occur. Elastic settlements due to these modifications are anticipated to be small and will occur relatively quickly. Construction/surcharge loads could also induce elastic settlements. However, these anticipated settlements will be small and will occur relatively quickly. Post-construction induced settlement will be minimal. Any settlement of the abutments will be due to axial shortening of the foundation piles and is anticipated to be less than ½ inch.

7.9 Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4 feet for frost protection as shown in Figure 5-2 of the MaineDOT BDG.

The pier spread footing and concrete seal will be constructed directly on bedrock. For foundations bearing on bedrock, heave due to frost is not a design issue and no requirements for embedment for frost protection are necessary.

Foundations bearing on soil should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Waterboro-Limerick has a design freezing index of approximately 1,300 F-degree days. A water content of 5 percent was assumed for granular soils. These components correlate to a frost depth of approximately 6.5 feet.

A similar analysis was performed using ModBerg software by the United States Army Cold Regions Research and Engineering Laboratory. For the ModBerg analysis, Waterboro-Limerick was assigned a design freezing index of approximately 1,123 F-degree days, for Sanford, the closest location in the ModBerg database. A water content of 5 percent was assumed for granular soils above the groundwater table. These components correlate to a frost depth of approximately 4.6 feet.

Based on the average of the two frost depths calculated (4.6 and 6.5 feet), it is recommended that foundations bearing on soil be designed with an embedment of 5.5 feet for frost protection. Supporting calculations are provided in Appendix C – Calculations.

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

7.10 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design (Q_{100}) and check (Q_{500}) floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state shall consider loss of lateral and vertical support of the foundation due to scour. Design at the extreme limit state shall check that the nominal foundation resistance due to the check flood (Q_{500}) event is no less that the extreme limit state loads. At the service limit state, the design shall limit movements and ensure overall stability considering scour at the design load.

The PDR indicates the proposed bridge will only be subject to contraction scour and not local scour. The calculated contraction scour depth for the design flood (Q_{100}) event is 5.76 feet at proposed Abutments and Pier. A scour depth of this magnitude will destabilize the abutment pile groups if left unprotected. The PDR indicates the bridge approach slopes and abutment slopes will be armored with riprap which will provide a sufficient level of scour protection for the pile-supported abutments. Pier seal plan notes will require the seal to be constructed directly on competent bedrock that is cleaned of all potentially erodible or scourable rock; therefore, contraction scour at the pier will not be an issue.

For scour protection of the pile-supported abutments, the bridge approach slopes and slopes at abutments should be armored with 3 feet of plain riprap or 4 feet of heavy riprap. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding riprap design. The top of the riprap shall extend up the slope to a minimum elevation of Q_{50}.

Stone riprap shall conform to Sections 703.26 and 703.28 of Standard Specifications and shall be placed at a maximum slope of 1.75:1. The toe of the riprap section shall be constructed 1 foot below the riverbed elevation. The riprap section shall be underlain by a 1-foot-thick layer of bedding material conforming to Section 703.19 of the Standard Specification and Class 1 nonwoven erosion control geotextile per Standard Details 610(02) through 610(04).

7.11 Seismic Design Parameters

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the LRFD Manual and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provides the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. These results are presented in Table 12. Supporting calculations are provided in Appendix C – Calculations.

Parameter	Value
Peak Ground Acceleration (PGA)	0.100g
Acceleration Coefficient (A_S)	0.120g
S_{DS} (Period = 0.2 second)	0.233g
S_{D1} (Period = 1.0 second)	0.080g
Site Class[11]	C
Seismic Zone	1

Table 12. Seismic Design Parameters

In conformance with LRFD Table 4.7.4.3.1-1, seismic analysis is not required for multi-span bridges in Seismic Zone 1. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

[11] The site class was determined per LRFD Table C3.10.3.1-1 – Method B.

7.12 Construction Considerations

Construction of the integral abutments will require pile driving and/or bedrock coring equipment. Temporary earth support systems may be required to permit construction of the abutments. The new integral abutments will be constructed behind the existing abutments avoiding placement of fills or cofferdams in the river. There is a potential that the existing abutments and their footings, if not removed entirely, may obstruct pile driving operations or bedrock coring. The Contractor shall be responsible for excavating those portions of the existing abutments and footings that conflict with piles by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Excavation by these methods shall be made incidental to related pay items. It is assumed that the existing substructures will be removed to the riverbed or slightly below. Care should be taken to ensure suitable materials are not disturbed unnecessarily.

Occasional cobbles were encountered in the sandy fill underneath the bridge approaches. There is potential for these obstructions to impact construction activities. Impacts include but are not limited to impeding the driving of sheet piles for temporary earth support systems and driving H-piles for abutment foundations or impede bedrock coring operations. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident. Care should take to install piles within allowable tolerances.

Excavations for the proposed abutments will expose soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The Contractor should control groundwater, surface water infiltration, and soil erosion. Water should be controlled by pumping from sumps.

Construction of the center pier will require a cofferdam to support overburden soils and control river flow during construction of the tremie-seal and footing. The bedrock shall be cleared of all loose fractured bedrock, loose decomposed bedrock, and soil. The foundation subgrade should be confirmed to be relatively level. If bedrock is observed to slope steeper than 4:1, the bedrock should be benched to create level steps or excavated to be completely level. An alternative approach is to design reinforcing dowels to anchor the seal to the sloping bedrock as a means of improving sliding resistance. Where the foundation will not be constructed in-the-dry, the condition of the bedrock surface prior to placing tremie-seal concrete should be inspected with the use of remote underwater cameras, divers, or other methods approved by the Resident. The cleanliness and condition of the final bedrock surface for tremie-seals shall be approved by the Resident prior to the placement of concrete seal.

Underwater excavation of highly sloping and loose fractured bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.7 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre-and post-blast surveys, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

Cobbles were encountered in the river alluvium at the location of the proposed pier. There is potential for these obstructions to impact or impede the driving of sheeting and cofferdam excavation.

Use of excavated native soils as structural backfill or beneath the new pavement structure should not be permitted. The native soils may only be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703.

The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. The materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below the subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

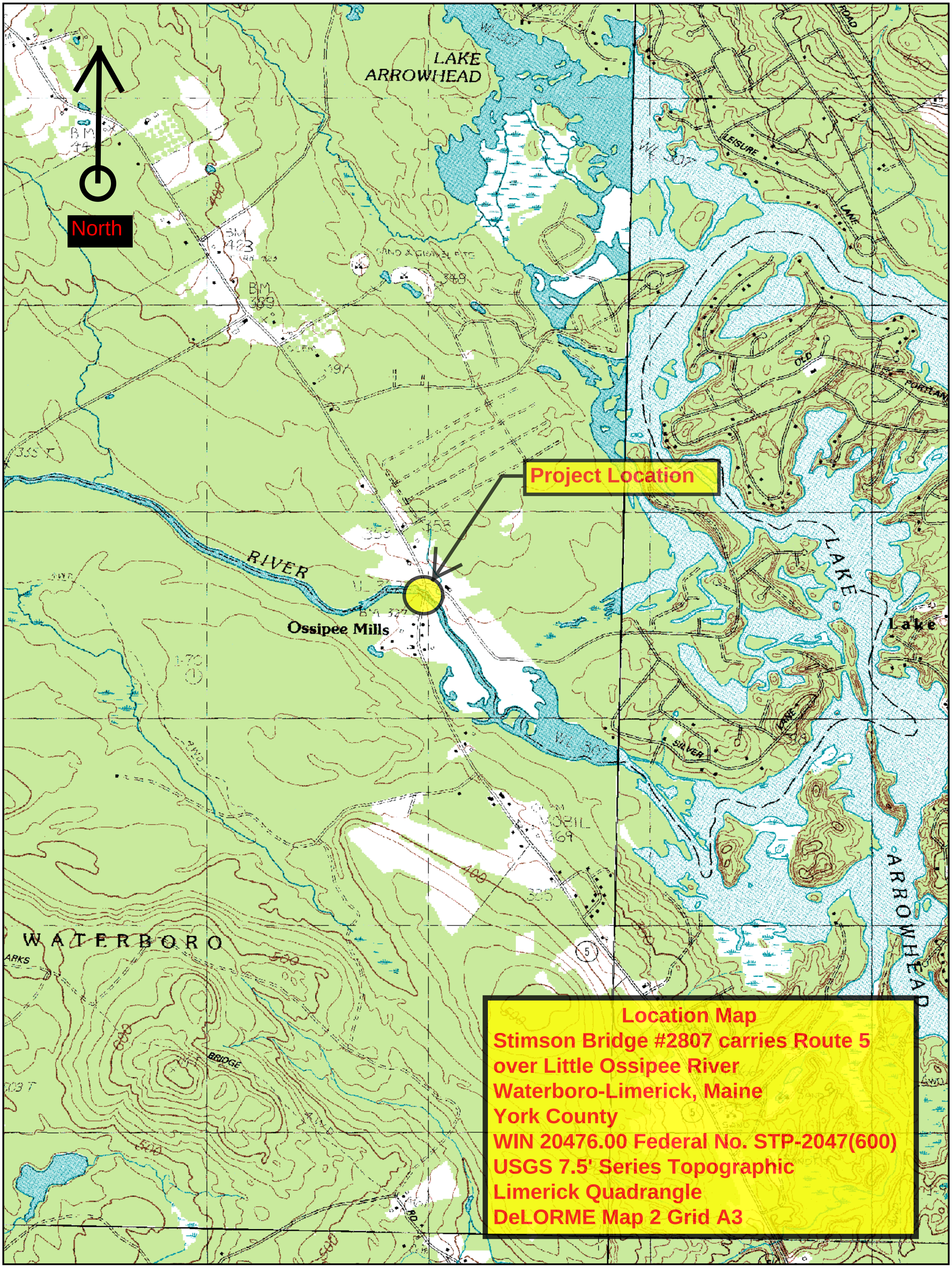
8.0 CLOSURE

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

In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by the 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.

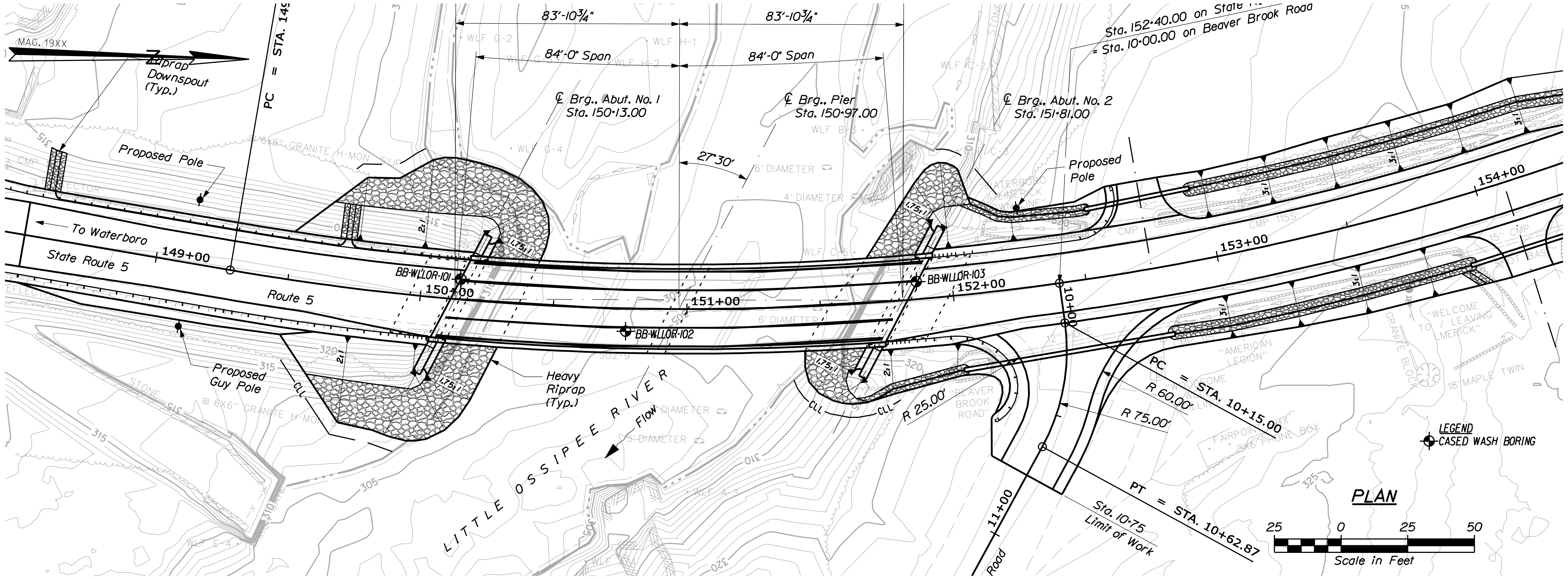
We also recommend that the Geotechnical Engineer be provided the opportunity for a general review of the final design and specifications in order to check that the geotechnical recommendations presented herein are properly interpreted and implemented in final design.

SHEETS



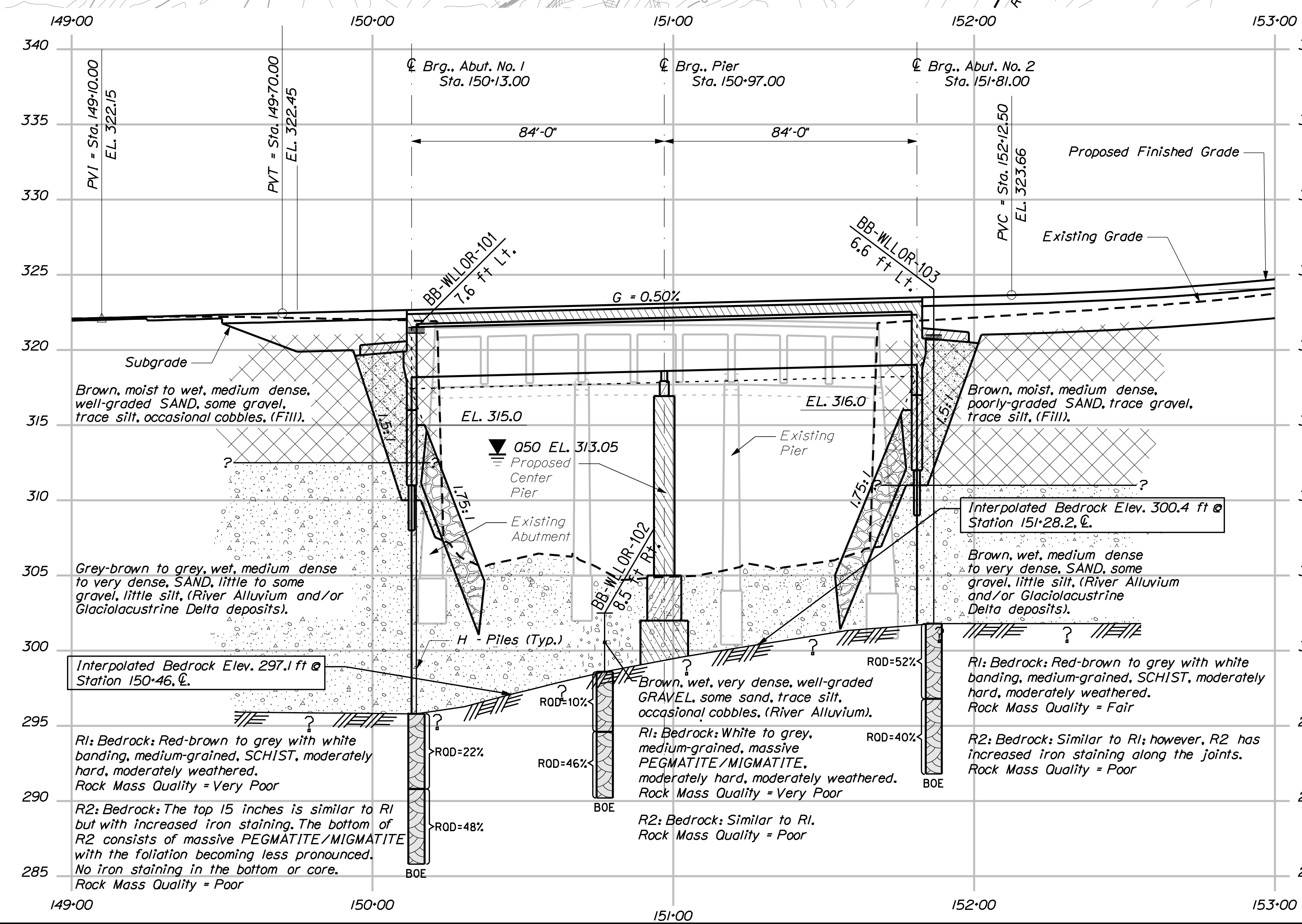
Map Scale 1:24000

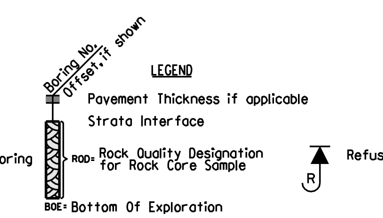
Filename: ...\geotech\msta\006_BLP&ISP1.dgn   Division: GEOTECH   Username: terry.white   Date:3/4/2016



PLAN

25 0 25 50
Scale in Feet





PROFILE

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

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

STP-2047(600)

WIN 20476.00

BRIDGE NO. 2807

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. Parlin		
DESIGN-DETAILED	D. Eaton	-----	-----
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	N.SHERWOOD	T.WHITE	FEB 2016
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----		-----
REVISIONS 2	-----		-----
REVISIONS 3	-----		-----
REVISIONS 4	-----		-----
FIELD CHANGES	-----		-----

SIGNATURE

P.E. NUMBER

DATE

STIMSON BRIDGE
LITTLE OSSIPEE RIVER
WATERBORO \ LIMERICK YORK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\msta\007_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:3/4/2016

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

Project: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine
Boring No.: BB-WLLOR-101   WIN: 20476.00

Driller:	MaineDOT	Elevation (ft.)	321.5	Auger ID/OD:	5-inch-diameter Solid Stem
Operator:	Giles/Doggett/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140 pounds/30 inches
Date Start/Finish:	4/14/2015; 08:00-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ - 2 inch
Boring Location:	Sta 150+14.6, 7.6 feet left	Casing ID/OD:	NW (3 inches/3.5 inches)	Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.908		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	321.13	4.5-inch-thick layer of pavement (0.38)	
	1D	24/18	2.00 - 4.00	4/10/6/10	16	24			Brown, moist, medium dense, well-graded SAND, some gravel, trace silt, occasional cobbles, (Fill).	G#263919 A-1-b, SW-SM WC=3.7%
5	2D	24/17	5.00 - 7.00	4/9/9/8	18	27			Similar to above, except wet.	
								312.50	(9.00)	
10	3D	24/6	10.00 - 12.00	5/5/5/5	10	15	19		Grey-brown, wet, medium dense, SAND, some gravel, little silt, (River Alluvium and/or Glaciolacustrine Delta deposits).	
							33			
							37			
							37		Wood fragments from 14.0-14.4 feet bgs.	
							34			
15	4D	24/2	15.00 - 17.00	12/7/6/6	13	20	10		Similar to above.	
							16			
							25			
							66			
							108			
20	5D	24/12	20.00 - 22.00	16/16/15/36	31	47	5		Grey, wet, dense, SAND, some gravel, little silt.	G#263920 A-1-b, SM WC=12.6%
							5			
							35			
							146		RC to 25.0 feet bgs.	
							189			
25	6D / R1	8.4/8.4 / 60/59	25.00 - 25.70 / 25.70 - 30.70	26/50(2.4") / RQD = 22%	---		a50 / NQ-2	295.80	a50 blows for 8.4 inches. Similar to above, except very dense and little gravel. (25.70) Top of Bedrock at Elevation 295.8 feet. R1: Bedrock: Red-brown to grey with white banding (composed of primarily quartz and feldspar that are not foliated), garnet SCHIST with foliated quartz, muscovite mica, feldspar, and amphibole, medium-grained, moderately hard, moderately weathered. Lower Member of the Rindgemere Formation. Rock Mass Quality = Very Poor. 98% Recovery. R1: Core Times (min:sec) 25.7-26.7 feet (2:00); 26.7-27.7 feet (2:00); 27.7-28.7 feet (3:18); 28.7-29.7 feet (3:36); 29.7-30.7 feet (3:10)	G#263921 A-2-4 to A-2-7, SM WC=13.5%
30	R2	60/60	30.70 - 35.70	RQD = 48%					R2: Bedrock: The top 15 inches is similar to R1 but with increased iron staining. The bottom of R2 consists of massive PEGMATITE/MIGMATITE with the foliation becoming less pronounced. No iron staining in the bottom of the core. Rock Mass Quality = Poor. 100% Recovery. R2: Core Times (min:sec) 30.7-31.7 feet (2:40); 31.7-32.7 feet (2:14); 32.7-33.7 feet (2:11); 33.7-34.7 feet (2:40); 34.7-35.7 feet (2:30)	
35								285.80	(35.70) Bottom of Exploration at 35.70 feet below ground surface.	
40										
45										
50										

Remarks:

- 400 pounds of down pressure on Core Barrel
- bgs = below ground surface

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-WLLOR-101

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

Project: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine
Boring No.: BB-WLLOR-102   WIN: 20476.00

Driller:	MaineDOT	Elevation (ft.)	302.5	Auger ID/OD:	5-inch-diameter Solid Stem
Operator:	Giles/Doggett/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140 pounds/30 inches
Date Start/Finish:	4/15/2015; 12:30-15:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ - 2 inch
Boring Location:	Sta 150+77.2, 8.5 feet right	Casing ID/OD:	NW (3 inches/3.5 inches)	Water Level*:	Boring located in River
Hammer Efficiency Factor: 0.908		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	1D	24/17	0.00 - 2.00	26/30/23/18	53	80	36		Brown, wet, very dense, well-graded GRAVEL, some sand, trace silt, occasional cobbles, (River Alluvium).	G#263922 A-1-a, GW-GM WC=9.2%
							15			
							62		a115 blows for 10.8 inches	
	R1	48/48	3.90 - 7.90	RQD = 10%			a115 / NQ-2	298.60	(3.90) Top of Bedrock at Elevation 298.6 feet. R1: Bedrock: White to grey, massive PEGMATITE/MIGMATITE with a high muscovite content in the upper 18 inches of the core, mostly feldspar and quartz with numerous black beryl crystals scattered throughout the rest of the core, moderately hard, moderately weathered, joints are typically horizontal with several steeply dipping joints (45-60 degrees), iron staining with lime green minerals on the lower joints. Lower Member of the Ringdgemere Formation. Rock Mass Quality = Very Poor. 100% Recovery. R1: Core Times (min:sec) 3.9-4.9 feet (2:36); 4.9-5.9 feet (5:57); 5.9-6.9 feet (2:49); 6.9-7.9 feet (4:00)	
5										
	R2	52.8/52.8	7.90 - 12.30	RQD = 46%					R2: Bedrock: Similar to R1; however, the quartz is forming a distinctive graphic pattern and the joints creating smooth breaks in the core dipping at 0-20, 45, and 60 degrees. Most of the joints have iron staining. Rock Mass Quality = Poor. 100% Recovery. R2: Core Times (min:sec) 7.9-8.9 feet (3:51); 8.9-9.9 feet (5:30); 9.9-10.9 feet (5:30); 10.9-11.9 feet (9:38); 11.9-12.3 feet (5:00)	
10										
								290.20	(12.30) Bottom of Exploration at 12.30 feet below ground surface.	
15										
20										
25										

Remarks:

- 400 pounds of down pressure on core barrel
- 18.5 feet from bridge deck to bottom of riverbed
- 11-inch-thick concrete deck
- bgs = below ground surface

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-WLLOR-102

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

Project: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine
Boring No.: BB-WLLOR-103   WIN: 20476.00

Driller:	MaineDOT	Elevation (ft.)	321.0	Auger ID/OD:	5-inch-diameter Solid Stem
Operator:	Giles/Doggett/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140 pounds/30 inches
Date Start/Finish:	4/15/2015; 08:00-12:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ - 2 inch
Boring Location:	Sta 151+86.6, 6.6 feet left	Casing ID/OD:	NW (3 inches/3.5 inches)	Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.908		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	320.67	4-inch-thick layer of pavement (0.33)	
	1D	24/17	1.00 - 3.00	8/8/7/7	15	23			Brown, moist, medium dense, poorly-graded SAND, trace gravel, trace silt, (Fill).	G#263923 A-1-b, SP-SM WC=3.3%
5	2D	24/14	5.00 - 7.00	8/4/4/5	8	12	23		Similar to above.	
							19			
							17			
							13			
							14			
10	3D	24/13	10.00 - 12.00	2/2/8/13	10	15	3	310.50	(10.50) Brown, wet, medium dense, SAND, some gravel, little silt, (River Alluvium and/or Glaciolacustrine Delta deposits).	G#263924 A-1-b, SM WC=22.8%
							29			
							19			
							34			
							69			
15	4D	24/18	15.00 - 17.00	37/13/13/18	26	39	40		Brown, wet, dense, SAND, some gravel, little silt.	G#263925 A-1-b, SM WC=8.5%
							189		RC to 19.0 feet bgs.	
							56			
							127			
	5D / R1	2.4/2.4 / 60/60	19.00 - 19.20 / 19.20 - 24.20	50(2.4") / RQD = 52%	---		NQ-2	301.80	Similar to above, except very dense. Weathered bedrock observed in tip of sampler. (19.20) Top of Bedrock at Elevation 301.8 feet. R1: Bedrock: Red-brown to grey with white banding (composed of primarily quartz and feldspar which are not foliated), garnet SCHIST with foliated quartz, muscovite mica, feldspar, amphibole, medium-grained, moderately hard, moderately weathered, the foliation created by the mica plates, dips from 0-10 degrees and is locally contorted by folds. Lower Member of the Ringdgemere Formation. Rock Mass Quality = Fair. 100% Recovery. R1: Core Times (min:sec) 19.2-20.2 feet (2:32); 20.2-21.2 feet (2:34); 21.2-22.2 feet (2:46); 22.2-23.2 feet (3:20); 23.2-24.2 feet (3:35)	
25	R2	60/55	24.20 - 29.20	RQD = 40%					R2: Bedrock: Similar to R1 however, R2 has increased iron staining along the joints. Rock Mass Quality = Poor. 92% Recovery. R2: Core Times (min:sec) 24.2-25.2 feet (2:10); 25.2-26.2 feet (2:11); 26.2-27.2 feet (2:46); 27.2-28.2 feet (3:00); 28.2-29.2 feet (3:00)	
								291.80	(29.20) Bottom of Exploration at 29.20 feet below ground surface.	
30										
35										
40										
45										
50										

Remarks:

- 400 pounds of down pressure on core barrel
- bgs = below ground surface

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-WLLOR-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2047(600)

WIN 20476.00

BRIDGE PLANS

BRIDGE NO. 2807

	BY	DATE
PROJ. MANAGER	M. Parisi	
DESIGN-DETAILED	D. Eaton	
CHECKED-REVIEWED	N. SHERWOOD	
DESIGN2-DETAILED2	T. WHITE	MAR 2016
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

STIMSON BRIDGE
LITTLE OSSIPEE RIVER
WATERBORO \ LIMERICK   YORK COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

APPENDIX A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.

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

Rock Quality Designation (RQD):

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

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Stimson Bridge #2807 carries Route 5 over Little Ossipee River **Location:** Waterboro-Limerick, Maine	**Boring No.:** BB-WLLOR-101 **PIN:** 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	321.5	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/14/2015; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 150+14.6, 7.6 feet left	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.908	**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	321.13		4.5-inch-thick layer of pavement 0.38	
	1D	24/18	2.00 - 4.00	4/10/6/10	16	24				Brown, moist, medium dense, well-graded SAND, some gravel, trace silt, occasional cobbles, (Fill).	G#263919 A-1-b, SW-SM WC=3.7%
5	2D	24/17	5.00 - 7.00	4/9/9/8	18	27				Similar to above, except wet.	
								312.50		9.00	
10	3D	24/6	10.00 - 12.00	5/5/5/5	10	15	19			Grey-brown, wet, medium dense, SAND, some gravel, little silt, (River Alluvium and/or Glaciolacustrine Delta deposits).	
							33				
							37				
							37				
							34			Wood fragments from 14.0-14.4 feet bgs.	
15	4D	24/2	15.00 - 17.00	12/7/6/6	13	20	10			Similar to above.	
							16				
							25				
							66				
							108				
20	5D	24/12	20.00 - 22.00	16/16/15/36	31	47	5			Grey, wet, dense, SAND, some gravel, little silt.	G#263920 A-1-b, SM WC=12.6%
							5				
							35				
							146			RC to 25.0 feet bgs.	
							189				
25											

Remarks:

- 400 pounds of down pressure on Core Barrel
- bgs = below ground surface

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: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine

Boring No.: BB-WLLOR-101
PIN: 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	321.5	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/14/2015; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 150+14.6, 7.6 feet left	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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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.
25	6D	8.4/8.4	25.00 - 25.70	26/50(2.4")	---		a50	295.80	a50 blows for 8.4 inches Similar to above, except very dense and little gravel. 25.70	G#263921 A-2-4 to A-2-7, SM WC=13.5%
	R1	60/59	25.70 - 30.70	RQD = 22%			NQ-2		Top of Bedrock at Elevation 295.8 feet. R1: Bedrock: Red-brown to grey with white banding (composed of primarily quartz and feldspar that are not foliated), garnet SCHIST with foliated quartz, muscovite mica, feldspar, and amphibole, medium-grained, moderately hard, moderately weathered. Lower Member of the Rindgemere Formation Rock Mass Quality = Very Poor 98% Recovery R1: Core Times (min:sec) 25.7-26.7 feet (2:00) 26.7-27.7 feet (2:00) 27.7-28.7 feet (3:18) 28.7-29.7 feet (3:36) 29.7-30.7 feet (3:10)	
30	R2	60/60	30.70 - 35.70	RQD = 48%					R2: Bedrock: The top 15 inches is similar to R1 but with increased iron staining. The bottom of R2 consists of massive PEGMATITE/ MIGMATITE with the foliation becoming less pronounced. No iron staining in the bottom of the core. Rock Mass Quality = Poor 100% Recovery R2: Core Times (min:sec) 30.7-31.7 feet (2:40) 31.7-32.7 feet (2:14) 32.7-33.7 feet (2:11) 33.7-34.7 feet (2:40) 34.7-35.7 feet (2:30)	
35								285.80	35.70	
									Bottom of Exploration at 35.70 feet below ground surface.	
40										
45										
50										

Remarks:

- 400 pounds of down pressure on Core Barrel
- bgs = below ground surface

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: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine

Boring No.: BB-WLLOR-102
PIN: 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	302.5	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/15/2015; 12:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 150+77.2, 8.5 feet right	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Boring located in River
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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	1D	24/17	0.00 - 2.00	26/30/23/18	53	80	36			Brown, wet, very dense, well-graded GRAVEL, some sand, trace silt, occasional cobbles, (River Alluvium).	G#263922 A-1-a, GW-GM WC=9.2%
							15				
							62				
	R1	48/48	3.90 - 7.90	RQD = 10%			a115			a115 blows for 10.8 inches	
							NQ-2	298.60		3.90	
5										Top of Bedrock at Elevation 298.6 feet. R1: Bedrock: White to grey, massive PEGMATITE/MIGMATITE with a high muscovite content in the upper 18 inches of the core, mostly feldspar and quartz with numerous black beryl crystals scattered throughout the rest of the core, moderately hard, moderately weathered, joints are typically horizontal with several steeply dipping joints (45-60 degrees), iron staining with lime green minerals on the lower joints. Lower Member of the Rindgemere Formation. Rock Mass Quality = Very Poor 100% Recovery R1: Core Times (min:sec) 3.9-4.9 feet (2:36) 4.9-5.9 feet (5:57) 5.9-6.9 feet (2:49) 6.9-7.9 feet (4:00)	
	R2	52.8/52.8	7.90 - 12.30	RQD = 46%							
10										R2: Bedrock: Similar to R1; however, the quartz is forming a distinctive graphic pattern and the joints creating smooth breaks in the core dippping at 0-20, 45, and 60 degrees. Most of the joints have iron staining. Rock Mass Quality = Poor 100% Recovery R2: Core Times (min:sec) 7.9-8.9 feet (3:51) 8.9-9.9 feet (5:30) 9.9-10.9 feet (5:30) 10.9-11.9 feet (9:38) 11.9-12.3 feet (5:00)	
								290.20			
15										12.30	
										Bottom of Exploration at 12.30 feet below ground surface.	
20											
25											

Remarks:

- 400 pounds of down pressure on core barrel
- 18.5 feet from bridge deck to bottom of riverbed
- 11-inch-thick concrete deck
- bgs = below ground surface

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: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine

Boring No.: BB-WLLOR-103
PIN: 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	321.0	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/15/2015; 08:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 151+86.6, 6.6 feet left	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	320.67		4-inch-thick layer of pavement 0.33	
	1D	24/17	1.00 - 3.00	8/8/7/7	15	23				Brown, moist, medium dense, poorly-graded SAND, trace gravel, trace silt, (Fill).	G#263923 A-1-b, SP-SM WC=3.3%
5	2D	24/14	5.00 - 7.00	8/4/4/5	8	12	23			Similar to above.	
							19				
							17				
							13				
							14				
10	3D	24/13	10.00 - 12.00	2/2/8/13	10	15	3	310.50		10.50 Brown, wet, medium dense, SAND, some gravel, little silt, (River Alluvium and/or Glaciolacustrine Delta deposits).	G#263924 A-1-b, SM WC=22.8%
							29				
							39				
							34				
							69				
15	4D	24/18	15.00 - 17.00	37/13/13/18	26	39	40			Brown, wet, dense, SAND, some gravel, little silt. RC to 19.0 feet bgs.	G#263925 A-1-b, SM WC=8.5%
							189				
							56				
							127				
	5D	2.4/2.4	19.00 - 19.20	50(2.4")	---		NQ-2	301.80		Similar to above, except very dense Weathered bedrock observed in tip of sampler 19.20	
20	R1	60/60	19.20 - 24.20	RQD = 52%						Top of Bedrock at Elevation 301.8 feet. R1: Bedrock: Red-brown to grey with white banding (composed of primarily quartz and feldspar which are not foliated), garnet SCHIST with foliated quartz, muscovite mica, feldspar, amphibole, medium-grained, moderately hard, moderately weathered, the foliation created by the mica plates, dips from 0-10 degrees and is locally contorted by folds. Lower Member of the Rindgemere Formation Rock Mass Quality = Fair 100% Recovery R1: Core Times (min:sec)	
	R2	60/55	24.20 - 29.20	RQD = 40%							
25											

Remarks:

- 400 pounds of down pressure on core barrel
- bgs = below ground surface

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine

Boring No.: BB-WLLOR-103
PIN: 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	321.0	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/15/2015; 08:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 151+86.6, 6.6 feet left	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										19.2-20.2 feet (2:32) 20.2-21.2 feet (2:34) 21.2-22.2 feet (2:46) 22.2-23.2 feet (3:20) 23.2-24.2 feet (3:35) R2: Bedrock: Similar to R1; however, R2 has increased iron staining along the joints. Rock Mass Quality = Poor 92% Recovery	
								291.80		R2: Core Times (min:sec) 24.2-25.2 feet (2:10) 25.2-26.2 feet (2:11) 26.2-27.2 feet (2:46) 27.2-28.2 feet (3:00) 28.2-29.2 feet (3:00)	
30										29.20 **Bottom of Exploration at 29.20 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

- 400 pounds of down pressure on core barrel
- bgs = below ground surface

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-WLLOR-103

APPENDIX B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Waterboro-Limerick Work Number: 20476.00

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-WLLOR-101, 1D	150+14.6	7.6 Lt.	2.0-4.0	263919	1	3.7			SW-SM	A-1-b	0
BB-WLLOR-101, 5D	150+14.6	7.6 Lt.	20.0-22.0	263920	1	12.6			SM	A-1-b	II
BB-WLLOR-101, 6D	150+14.6	7.6 Lt.	25.0-25.7	263921	1	13.5			SM	A-2-4	II
BB-WLLOR-102, 1D	150+77.2	8.5 Rt.	0.0-2.0	263922	1	9.2			GW-GM	A-1-a	0
BB-WLLOR-103, 1D	151+86.6	6.6 Lt.	1.0-3.0	263923	2	3.3			SP-SM	A-1-b	0
BB-WLLOR-103, 3D	151+86.6	6.6 Lt.	10.0-12.0	263924	2	22.8			SM	A-1-b	II
BB-WLLOR-103, 4D	151+86.6	6.6 Lt.	15.0-17.0	263925	2	8.5			SM	A-1-b	II
STREAMBED			0.0-0.33	263500	3	13.8			GW	A-1-a	0

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)

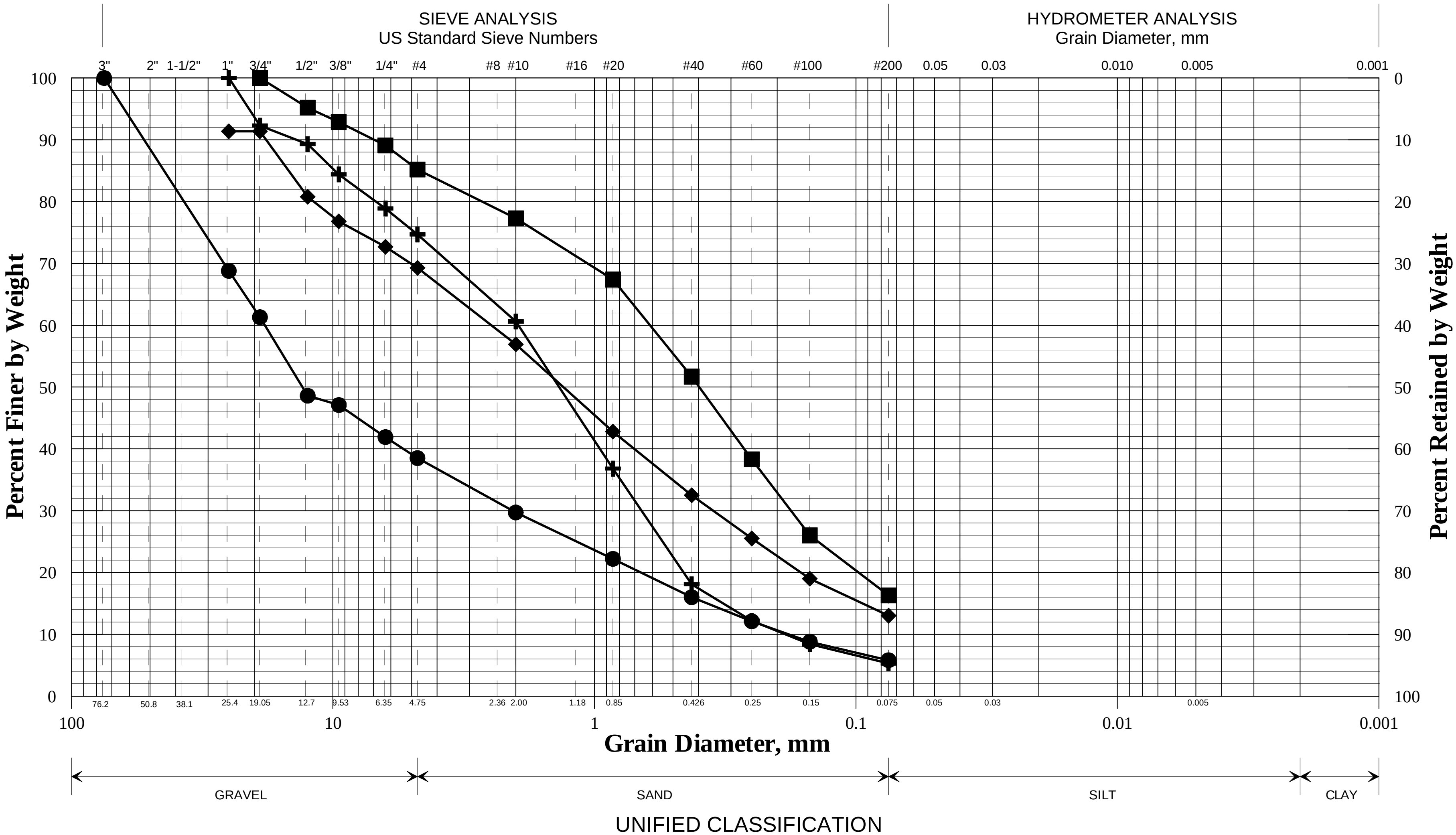
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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-WLLOR-101/1D	150+14.6	7.6 LT	2.0-4.0	SAND, some gravel, trace silt.	3.7			
◆	BB-WLLOR-101/5D	150+14.6	7.6 LT	20.0-22.0	SAND, some gravel, little silt.	12.6			
■	BB-WLLOR-101/6D	150+14.6	7.6 LT	25.0-25.7	SAND, little silt, little gravel.	13.5			
●	BB-WLLOR-102/1D	150+77.2	8.5 RT	0.0-2.0	GRAVEL, some sand, trace silt.	9.2			
▲									
✖									

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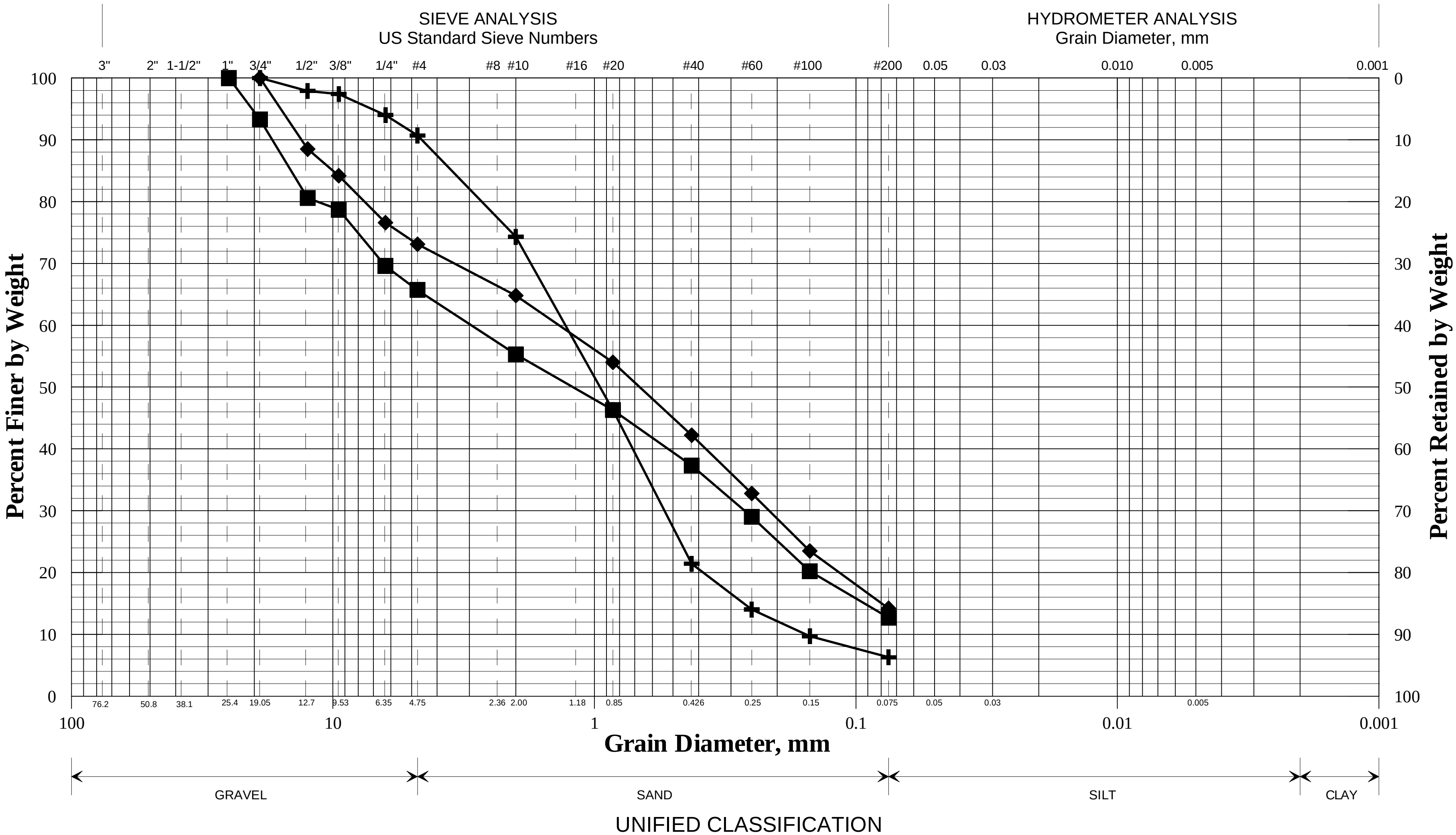
SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-WLLOR-103/1D	151+86.6	6.6 LT	1.0-3.0	SAND, trace gravel, trace silt.	3.3			
◆	BB-WLLOR-103/3D	151+86.6	6.6 LT	10.0-12.0	SAND, some gravel, little silt.	22.8			
■	BB-WLLOR-103/4D	151+86.6	6.6 LT	15.0-17.0	SAND, some gravel, little silt.	8.5			
●									
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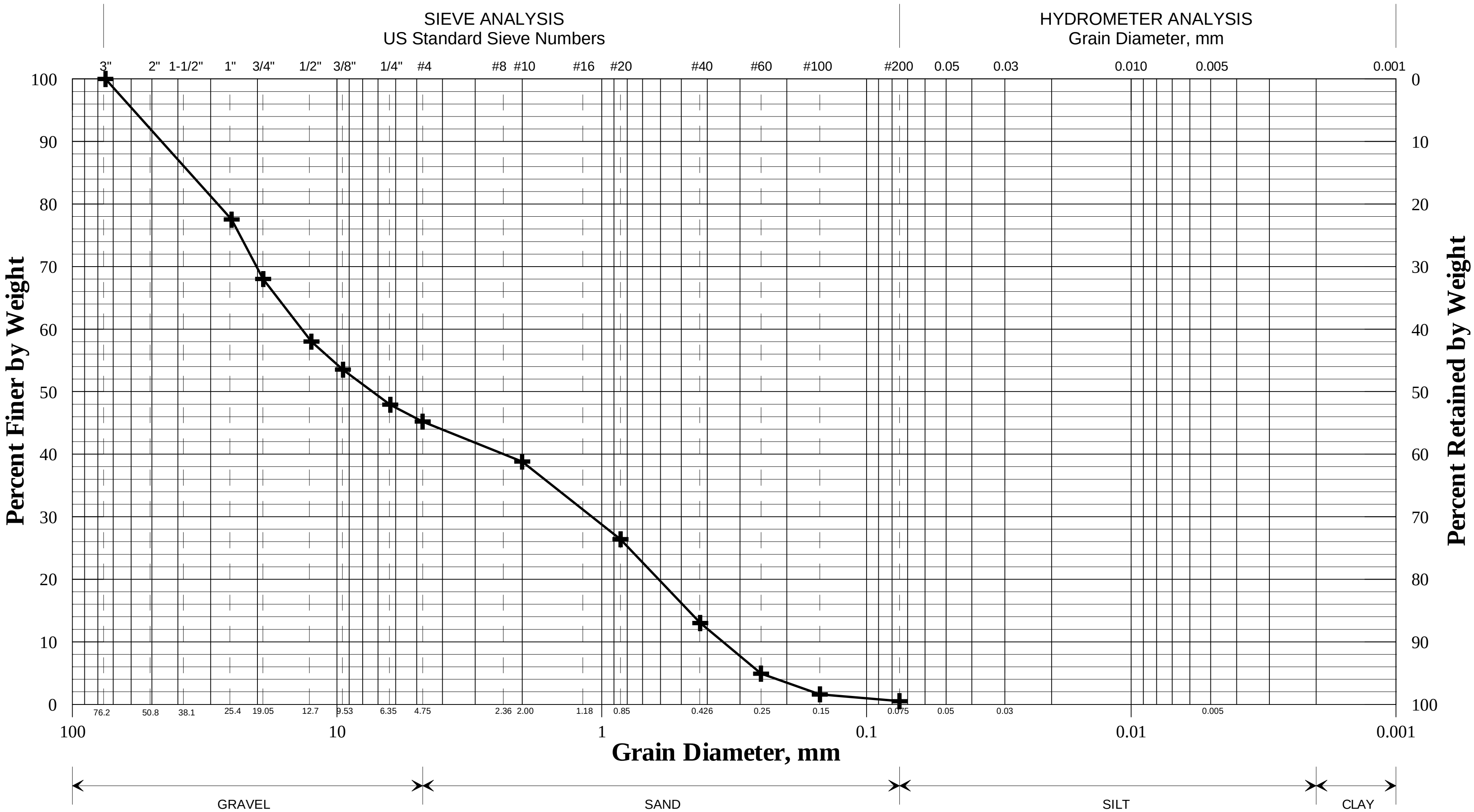
SHEET 2

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	STREAMBED			0.0-0.33	Sandy GRAVEL, trace silt.	13.8			
◆									
■									
●									
▲									
✖									

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SHEET 3

APPENDIX C

Calculations

Factored Axial Pile Compressive Resistances Calculations

OBJECTIVE
Estimate the nominal and factored axial compressive structural, geotechnical, and drivability resistances for two different H-pile sections at the service, strength, and extreme limit states considering the piles are either: 1) driven or 2) placed in bedrock sockets without driving.

GIVEN
1) Limited lab data and boring logs
2) HTA (project structural engineer) specified H-pile types: HP 14x89 and HP 14x117.

ASSUMPTIONS
1) Estimated soil properties.
2) An estimated unconfined compressive strength of the bedrock = 8,000 psi (refer to Page 5).
3) Pile design per AASHTO LRFD Bridge Design Specifications, 7th edition, 2014 with 2016 interims.
4) Per discussions with HTA, to increase lateral resistance, the piles may be installed in bedrock sockets. The geotechnical axial compressive resistances provided consider both proposed installation methods: 1) driven and 2) installing the piles in bedrock sockets without driving.

ESTIMATE THE NOMINAL AND FACTORED AXIAL COMPRESSIVE STRUCTURAL RESISTANCES AT THE SERVICE, STRENGTH, AND EXTREME LIMIT STATES

1. Define pile properties per AISC Steel Design Manual for HP 14x89 and HP 14x117 sections

HP 14x89 **Note: all matrices are set up in this order.**
HP 14x117

Area of selected H-piles, As $A_s := \begin{bmatrix} 26.1 \\ 34.4 \end{bmatrix} \ in^2$

Yield strength of H-piles, Fy $F_y := 50 \ ksi$

2. Determine equivalent nominal yield resistance of the H-piles per LRFD Article 6.9.4.1.1

Slender element reduction factor, Q (per LRFD Article C6.9.4.2.1): $Q := 1$

Equivalent nominal yield resistance, Po: $P_o := Q \cdot F_y \cdot A_s$

Therefore, the equivalent nominal yield resistances are: $P_o = \begin{bmatrix} 1305 \\ 1720 \end{bmatrix} \ kip$

3. Determine elastic critical buckling resistance of the H-piles per LRFD Eq. 6.9.4.1.2-1

Steel modulus, E: $E := 29000\ ksi$

Effective length factor, K [per LRFD Table C4.6.2.5-1; assuming case "d" (rotation fixed and translation free at pile head and rotation and translation fixed at the pile tip)]: $K_{eff} := 1.2$

Unbraced length, l_unbraced (Assume 12 inches is unbraced and scour is unlikely): $l_{unbraced} := 12\ in$

Radius of gyration, rs (LRFD Article C6.9.4.1.2 states that the critical flexural buckling resistances be calculated about the x- and y-axes with the smaller value used to calculate the resistance. Therefore, use the y-axes values):

$$r_s := \begin{bmatrix} 3.53 \\ 3.59 \end{bmatrix} in$$

Elastic critical buckling resistance, Pe, (per LRFD Article Eq. 6.9.4.1.2-1):

$$P_e := \overrightarrow{\left(\frac{(\pi^2 \cdot E)}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s} \right)^2} \right) \cdot A_s}$$

$$P_e = \begin{bmatrix} 448914 \\ 611956 \end{bmatrix} kip$$

4. Determine nominal axial compressive resistance, Pn, of the H-piles per LRFD Eq. 6.9.4.1.1-1

Pe/Po:

$$\frac{P_e}{P_o} = \begin{bmatrix} 344 \\ 356 \end{bmatrix}$$

If Pe/Po is greater than 0.44, then:

$$P_n := \overrightarrow{\left(.658^{\left(\frac{P_o}{P_e} \right)} \right) \cdot P_o}$$

$$P_n = \begin{bmatrix} 1303 \\ 1718 \end{bmatrix} kip$$

5. Determine the factored axial compressive resistance, Pr, of the H-piles per LRFD Eq. 6.9.2.1-1 at the Strength Limit State

Assume the driving conditions are "good" based on the subsurface information.

$\phi_c := 0.60$

Therefore, under "good" driving conditions, the resistance factor is 0.60

$P_{r\_strength} := \phi_c \cdot P_n$

The factored axial compressive resistance, Pr:

$P_{r\_strength} = \begin{bmatrix} 782 \\ 1031 \end{bmatrix}$ ***kip***

At the strength limit state, the static geotechnical resistances for H-piles installed in bedrock sockets with steel plates welded across the pile tips should be limited to the factored axial structural compressive resistances, P_r_strength, shown to the left.

6. Determine the factored compressive axial resistance, Pr, of the H-piles per LRFD Eq. 6.9.2.1-1 at the Service/Extreme Limit States

Assume the driving conditions are "good" based on the subsurface information.

The resistance factor is 1.0 (per LRFD 10.5.5.1 and 10.5.5.3)

$\phi := 1$

The factored axial compressive resistance, Pr:

$P_{r\_service\_extreme} := \phi \cdot P_n$

$P_{r\_service\_extreme} = \begin{bmatrix} 1303 \\ 1718 \end{bmatrix}$ ***kip***

At the service and extreme limit states, the static geotechnical resistances for H-piles installed in bedrock sockets with steel plates welded across the pile tips should be limited to the factored axial structural compressive resistances, P_r_service_extreme, shown to the left.

ESTIMATE THE NOMINAL AND FACTORED AXIAL COMPRESSIVE GEOTECHNICAL RESISTANCES AT THE SERVICE, STRENGTH, AND EXTREME LIMIT STATES PER LRFD 10.7.3.2.3 - PILES DRIVEN TO HARD ROCK

LRFD Article 10.7.3.2.3 states, "The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile driving acceptance criteria shall be developed that will prevent pile damage."

Therefore, limit the nominal axial compressive geotechnical pile resistance with a resistance factor for severe driving conditions of 0.50 per LRFD Article 10.7.3.2.3.

The nominal structural resistance was previously calculated as:

$$P_n = \begin{bmatrix} 1303 \\ 1718 \end{bmatrix} kip$$

1. Determine the factored axial compressive resistance, Pr, of the H-piles per LRFD Eq. 6.9.2.1-1 at the STRENGTH LIMIT STATE

Assume severe driving conditions, the resistance factor is 0.50

$$\phi_{geotechnical\_LRFD} := 0.50$$

The factored axial compressive resistance, Pr_geotechnical_strength_LRFD:

$$P_{r\_geotechnical\_strength\_LRFD} := \phi_{geotechnical\_LRFD} \cdot P_n$$

$$P_{r\_geotechnical\_strength\_LRFD} = \begin{bmatrix} 652 \\ 859 \end{bmatrix} kip$$

2. Determine the factored axial compressive resistance, Pr, of the H-piles per LRFD Eq. 6.9.2.1-1 at the SERVICE/EXTREME LIMIT STATES

The resistance factor is 1.0 (per LRFD 10.5.5.1 and 10.5.5.3)

$$\phi := 1$$

The factored axial compressive resistance, Pr_geotechnical_service_extreme_LRFD:

$$P_{r\_geotechnical\_service\_extreme\_LRFD} := \phi \cdot P_n$$

$$P_{r\_geotechnical\_service\_extreme\_LRFD} = \begin{bmatrix} 1303 \\ 1718 \end{bmatrix} kip$$

ESTIMATE THE NOMINAL AND FACTORED AXIAL COMPRESSIVE GEOTECHNICAL RESISTANCES AT THE SERVICE, STRENGTH, AND EXTREME LIMIT STATES FOR PILES PLACED IN BEDROCK SOCKETS AND DRIVEN PILES PER THE INTACT ROCK METHOD (IRM)

The axial compressive geotechnical resistance for piles end-bearing on bedrock as determined by the IRM was proposed by Thomas Sandford, PhD, PE of the University of Maine (MaineDOT Transportation Research Division Technical Report 14-01, Phase 2 (January 2014), based on the Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, 2006.

1. Determine the nominal unit bearing resistance of the pile point, qp

Assume the design value of the unconfined compressive strength (qu) of the rock is **8,000 psi**. This is based on:

$$q_u := 8000 \ psi$$

a) The unconfined compressive strength tests on schist from the Waterville Formation (schist) ranged from 5,120-8,766 psi.
b) The average unconfined compressive strength of schist from the Richmond-Dresden project was approximately 7,000 psi.
c) Goodman recommends a qu = 8,000 psi for quartz mica schist (Rock Mechanics, 1989, Table 3-1, page 61), qu:

$$q_p := 2.5 \cdot q_u$$

The geotechnical tip resistance, qp:

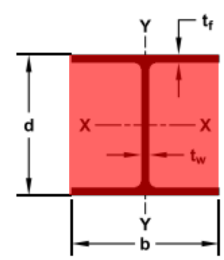
$$q_p = 20 \ ksi$$

2a. Determine the factored axial compressive resistance, Pr, of the H-piles at the Service and Extreme Limit States considering the piles are placed in bedrock sockets (i.e. NOT driven)

The resistance factor is 1.0 (per LRFD 10.5.5.1 and 10.5.5.3):

$$\phi := 1$$

To increase the static geotechnical resistance, a steel plate (shown as the shaded red area on the schematic sketch to the right) can be welded across the tip of the pile to provide a greater tip bearing surface.



A HP 14x89 section has a depth (d) and flange width (b) of:

$$b_{14x89} := 14.7 \ in$$
$$d_{14x89} := 13.8 \ in$$

The area of the steel plate, As_plate_14x89, of the HP 14x89 is:

$$A_{s\_plate\_14x89} := b_{14x89} \cdot d_{14x89}$$

$$A_{s\_plate\_14x89} = 202.9 \ in^2$$

A HP 14x117 section has a depth (d) and flange width (b) of:

$b_{14x117} := 14.9\ in$
$d_{14x117} := 14.2\ in$

The area of the steel plate, As_plate_14x117, of the HP 14x117 is:

$A_{s\_plate\_14x117} := b_{14x117} \cdot d_{14x117}$

$A_{s\_plate\_14x117} = 211.6\ in^2$

The factored axial resistance of a HP 14x89 placed in a bedrock socket (not driven), Pr_geotechnical_service_extreme_IRM_plate_14x89, is:

$$P_{r\_geotechnical\_service\_extreme\_IRM\_plate\_14x89} := \overrightarrow{\phi \cdot q_p \cdot A_{s\_plate\_14x89}}$$

$$P_{r\_geotechnical\_service\_extreme\_IRM\_plate\_14x89} = 4057\ kip$$

Limit to the nominal structural resistance of 1,303 kips

The factored axial resistance of a HP 14x117 placed in a bedrock socket (not driven), Pr_geotechnical_service_extreme_IRM_plate_14x117, is:

$$P_{r\_geotechnical\_service\_extreme\_IRM\_plate\_14x117} := \overrightarrow{\phi \cdot q_p \cdot A_{s\_plate\_14x117}}$$

$$P_{r\_geotechnical\_service\_extreme\_IRM\_plate\_14x117} = 4232\ kip$$

Limit to the nominal structural resistance of 1,718 kips

2b. Determine the factored axial compressive resistance, Pr, of the H-piles at the Service and Extreme Limit States considering the piles are driven (i.e. NOT placed in bedrock sockets)

The resistance factor is 1.0 (per LRFD 10.5.5.1 and 10.5.5.3):

In this scenario, since the piles are not placed in bedrock sockets, the area of the piles will not be increased with a steel plate.

The factored axial compressive resistances of the piles that are driven, Pr_geotechnical_service_extreme_IRM_no_plate, are:

$$P_{r\_geotechnical\_service\_extreme\_IRM\_no\_plate} := \overrightarrow{\phi \cdot q_p \cdot A_s}$$

$$P_{r\_geotechnical\_service\_extreme\_IRM\_no\_plate} = \begin{bmatrix} 522 \\ 688 \end{bmatrix} kip$$

3a. Determine the factored axial compressive resistance, Pr, of the H-piles at the Strength Limit State considering the piles are placed in bedrock sockets (i.e. NOT driven)

Per LRFD Table 10.5.5.2.3-1, the resistance factor, for end bearing on rock, Canadian Geotechnical Society (CGS), is 0.45:

$$\phi_{geotechnical\_CGS} := 0.45$$

The factored axial resistance of a HP 14x89 placed in a bedrock socket (not driven), Pr_geotechnical_strength_IRM_plate_14x89, is:

$$P_{r\_geotechnical\_strength\_IRM\_plate\_14x89} := \phi_{geotechnical\_CGS} \cdot P_{r\_geotechnical\_service\_extreme\_IRM\_plate\_14x89}$$

$$P_{r\_geotechnical\_strength\_IRM\_plate\_14x89} = 1826\ kip$$

Limit to the factored axial structural compressive resistance of 782 kips.

The factored axial resistance of a HP 14x117 placed in a bedrock socket (not driven), Pr_geotechnical_strength_IRM_plate_14x117, is:

$$P_{r\_geotechnical\_strength\_IRM\_plate\_14x117} := \phi_{geotechnical\_CGS} \cdot P_{r\_geotechnical\_service\_extreme\_IRM\_plate\_14x117}$$

$$P_{r\_geotechnical\_strength\_IRM\_plate\_14x117} = 1904\ kip$$

Limit to the factored axial structural compressive resistance of 1,031 kips.

3b. Determine the factored axial compressive resistance, Pr, of the H-piles at the Strength Limit State considering the piles are driven (i.e. NOT placed in bedrock sockets)

Per LRFD Table 10.5.5.2.3-1, the resistance factor, for end bearing on rock, Canadian Geotechnical Society (CGS), is 0.45.

In this scenario, since the piles are not placed in bedrock sockets, the area of the piles will not be increased with a steel plate.

The factored axial compressive resistances of the piles that are driven, Pr_geotechnical_strength_IRM_no_plate, are:

$$P_{r\_geotechnical\_strength\_IRM\_no\_plate} := \overrightarrow{\phi_{geotechnical\_CGS} \cdot q_p \cdot A_s}$$

$$P_{r\_geotechnical\_strength\_IRM\_no\_plate} = \begin{bmatrix} 235 \\ 310 \end{bmatrix} kip$$

Notes:

1) Per discussions with HTA, the maximum factored pile load is approximately 315 kips. As shown above, if the piles are driven (i.e. not installed in bedrock sockets), the governing geotechnical resistances at the strength limit state for HP 14x89 and HP 14x117 pile sections (235 and 310 kips, respectively) **do not** achieve the required 315 kips. Therefore, if the piles are installed in bedrock sockets, we recommend steel plates be welded to each pile tip to provide an adequate axial compressive resistance. The steel plate size may be decreased; however, the geotechnical engineer should be consulted to provide appropriate corresponding resistances with the decreased plate area .

1a) To prevent loading the piles beyond their structural capacities, HTA should limit the resistances to the nominal structural resistances: **HP 14x89 = 1,303 kips and HP 14x117 = 1,718 kips** at the Service/Extreme Limit States and **HP 14x89 = 782 kips and HP 14x117 = 1,031 kips** at the Strength Limit State.

2) Although the IRM yields lower resistance values than LRFD; previous dynamic load tests have confirmed capacities closer to LRFD values. Therefore, we recommend LRFD resistances be used rather than IRM resistances.

ESTIMATE THE NOMINAL AND FACTORED AXIAL COMPRESSIVE DRIVABILITY RESISTANCES AT THE SERVICE, STRENGTH, AND EXTREME LIMIT STATES PER LRFD ARTICLE 10.7.8 AND USING GRLWEAP software

1. For steel piles in either compression or tension, the driving stresses are limited to 90% of the yield strength (Fy).

Drivability analyses resistance factor, 1.0 (per LRFD Table 10.5.5.2.3-1): $\phi_{da} := 1$

$\sigma_{dr} := 0.9 \cdot F_y \cdot \phi_{da}$

Allowable driving stress: $\sigma_{dr} = 45\ ksi$

Limit driving stress to 45 ksi or limit blow counts to 5-15 blows per inch (bpi) per Section 501 (Note: 6-10 bpi is considered optimal for diesel hammers).

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

Resistance factor, 0.65 when dynamic testing is performed during construction(per LRFD Table 10.5.5.2.3-1) for strength limit state: $\phi_{dynamic\_strength} := 0.65$

For service and extreme limit states: $\phi_{dynamic\_service\_extreme} := 1.0$

3. Determine GRLWEAP Soil and Pile Model assumptions

-Assume the pile length will be 20 feet.

-Assume the contractor drives 25-foot-long piles (5 feet for testing + 2 feet for pile cap + 3 feet for misc.)

-Assume minimal soil side shaft resistance along shaft from Elevation 312 to 302 feet; medium dense to very dense sand.

3a. Estimate skin friction (f_s) contribution of medium dense to very dense sand per Fang Foundation Engineering 1991, Second Edition, Chapter 13 - Pile Foundations, by B. Fellenius, using the Beta-method

Using the Beta-method, estimate beta value (for a med. dense to dense sand, per Fang, Table 13.1):

$$\beta := 0.45$$

Estimate the effective overburden stress (sigma v') near the midpoint of the pile (10 feet bgs) and assuming the effective unit weights (gamma) used to recommend LPile parameters in report.

Groundwater is approximately 5 feet bgs (conservative estimate)

$$\gamma_{layer1} := 117\ pcf$$

$$\gamma_{layer2} := 112\ pcf$$

$$\sigma'_{v\_at\_midpt} := 5\ ft \cdot \gamma_{layer1} + 5\ ft \cdot \left(\gamma_{layer2} - 62.4\ pcf\right)$$

$$\sigma'_{v\_at\_midpt} = 0.833\ ksf$$

$$f_{s\_at\_midpt} := \beta \cdot \sigma'_{v\_at\_midpt}$$

$$f_{s\_at\_midpt} = 374.85\ psf$$

As a check, consider the Meyerof (1976) approach in estimating the skin friction :

Average N60 for use in design at Abutment No. 2:

$$N_{60} := 23$$

$$f_{s\_Meyerof} := (0.02) \cdot 2000 \cdot \left(N_{60}\right)\ psf$$

(per Das, Principles of Foundation Engineering, Seventh Edition, Eq. 11-45)

$$f_{s\_Meyerof} = 920\ psf$$

The unit skin friction per the Beta-method controls.

Now apply unit skin friction assuming a HP 14x89
(less area, so less skin friction)

Perimeter of pile section, P_14x89

$P_{14x89} := 4\ ft$

Length of pile embedded, L_pile

$L_{pile} := 20\ ft$

Skin friction capacity, R_skin:

$R_{skin} := f_{s\_at\_midpt} \cdot P_{14x89} \cdot L_{pile}$

(Assume that the skin friction capacity will be uniformily distrubuted along the pile length for GRLWEAP analyses)

$R_{skin} = 30\ kip$

GRLWEAP Analysis No. 1: HP 14x89

The pile can be driven to the resistances below with a Delmag 19-42 with a reasonable blow count and level of driving stress. The GRLWEAP results are below:

State of Maine Dept. Of Transportation
20476_Waterboro-Limerick_Stimson Bridge

04-Feb-2016
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	30.71	0.11	3.8	8.29	15.90
325.0	32.28	0.37	4.1	8.43	15.95
350.0	33.79	0.64	4.4	8.56	16.03
375.0	35.23	0.70	4.8	8.71	16.14
400.0	36.79	0.94	5.1	8.89	16.36
425.0	38.24	0.80	5.5	9.03	16.53
450.0	39.47	0.90	5.9	9.19	16.69
475.0	40.89	1.00	6.3	9.36	16.89
500.0	42.20	1.64	6.7	9.54	17.14
525.0	43.44	2.42	7.2	9.70	17.47

To limit the driving stress to less than 45 ksi and target the blow counts to 7 bpi; assume the ultimate capacity is approximately **525 kips** for a HP 14x89 pile

Therefore, the nominal axial compressive resistance (Rn_dr_14x89) is:

$$R_{n\_dr\_14x89} := 525\ \textbf{kip}$$

Factored axial compressive esistance at the Strength Limit State (Rf_dr_strength_14x89):

$$R_{f\_dr\_strength\_14x89} := R_{n\_dr\_14x89} \cdot \phi_{dynamic\_strength}$$

$$R_{f\_dr\_strength\_14x89} = 341\ \textbf{kip}$$

Factored axial compressive resistance at the Service and Extreme Limit States (Rf_dr_service_extreme_14x89):

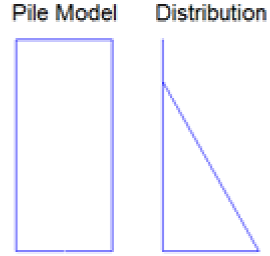
$$R_{f\_dr\_service\_extreme\_14x89} := R_{n\_dr\_14x89} \cdot \phi_{dynamic\_service\_extreme}$$

$$R_{f\_dr\_service\_extreme\_14x89} = 525\ \textbf{kip}$$

DELMAG D 19-42

Efficiency	0.800	
Helmet	1.90	kips
Hammer Cus	60155	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetrati	20.00	ft
Pile Top Area	26.10	in2

Pile Model　　Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

GRLWEAP Analysis No. 1: HP 14x117

The pile can be driven to the resistances below with a Delmag 19-42 with a reasonable blow count and level of driving stress. The GRLWEAP results are below:

State of Maine Dept. Of Transportation
20476_Waterboro-Limerick_Stimson Bridge

05-Feb-2016
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
550.0	36.05	1.83	8.2	9.45	17.07
575.0	36.98	2.12	8.7	9.56	17.33
600.0	37.99	2.40	9.2	9.66	17.57
625.0	38.87	2.62	9.7	9.75	17.79
650.0	39.65	2.75	10.3	9.84	17.94
675.0	40.52	3.01	10.9	9.93	18.17
700.0	41.25	3.18	11.5	10.02	18.36
725.0	42.07	3.32	12.1	10.11	18.57
750.0	42.71	3.46	12.8	10.20	18.76
775.0	43.51	3.62	13.5	10.29	18.96

To limit the driving stress to less than 45 ksi and target the blow counts to 12 bpi (optimal range of diesel hammer); assume the ultimate capacity is approximately **725 kips** for a HP 14x117 pile

Therefore, the nominal axial compressive resistance (Rn_dr_14x117) is:

$$R_{n\_dr\_14x117} := 725\ \mathbf{kip}$$

Factored axial compressive resistance at the Strength Limit State (Rf_dr_strength_14x117):

$$R_{f\_dr\_strength\_14x117} := R_{n\_dr\_14x117} \cdot \phi_{dynamic\_strength}$$

$$R_{f\_dr\_strength\_14x117} = 471\ \mathbf{kip}$$

Factored axial compressive resistance at the Service and Extreme Limit States (Rf_dr_service_extreme_14x117):

$$R_{f\_dr\_service\_extreme\_14x117} := R_{n\_dr\_14x117} \cdot \phi_{dynamic\_service\_extreme}$$

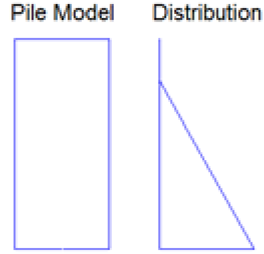
$$R_{f\_dr\_service\_extreme\_14x117} = 725\ \mathbf{kip}$$

DELMAG D 19-42

Efficiency	0.800	
Helmet	1.90	kips
Hammer Cus	60155	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetrati	20.00	ft
Pile Top Area	34.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

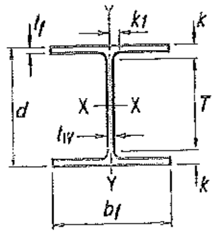


Table 1-4
HP Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance k_1	Distance T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.
HP14×117[f]	34.4	14.2	14 1/4	0.805	13/16	7/16	14.9	14 7/8	0.805	13/16	1 1/2	1 1/16	11 1/4	5 1/2
×102[f]	30.0	14.0	14	0.705	11/16	3/8	14.8	14 3/4	0.705	11/16	1 3/8	1		
×89[f]	26.1	13.8	13 7/8	0.615	5/8	5/16	14.7	14 3/4	0.615	5/8	1 5/16	15/16		
×73[c,f]	21.4	13.6	13 5/8	0.505	1/2	1/4	14.6	14 5/8	0.505	1/2	1 3/16	7/8		
HP12×84	24.6	12.3	12 1/4	0.685	11/16	3/8	12.3	12 1/4	0.685	11/16	1 3/8	1	9 1/2	5 1/2
×74[f]	21.8	12.1	12 1/8	0.605	5/8	5/16	12.2	12 1/4	0.610	5/8	1 5/16	15/16		
×63[f]	18.4	11.9	12	0.515	1/2	1/4	12.1	12 1/8	0.515	1/2	1 1/4	7/8		
×53[f]	15.5	11.8	11 3/4	0.435	7/16	1/4	12.0	12	0.435	7/16	1 1/8	7/8		
HP10×57[f]	16.8	9.99	10	0.565	9/16	5/16	10.2	10 1/4	0.565	9/16	1 1/4	15/16	7 1/2	5 1/2
×42[f]	12.4	9.70	9 3/4	0.415	7/16	1/4	10.1	10 1/8	0.420	7/16	1 1/8	13/16	7 1/2	5 1/2
HP8×36[f]	10.6	8.02	8	0.445	7/16	1/4	8.16	8 1/8	0.445	7/16	1 1/8	7/8	5 3/4	5 1/2

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Western Avenue over Messalonskee Stream Location: Waterville, Maine	Boring No.: BB-WMS-101 PIN: 18234.00

Driller:	MaineDOT	Elevation (ft.)	109.5 (Approx.)	Auger ID/OD:	--
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Split spoon - 1.375 in. ID
Logged By:	B. Babcock	Rig Type:	CME 45 Skid on Truck	Hammer Wt./Fall:	NW-S 140#/30 in.
Date Start/Finish:	07-18-2012/07-18-2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ - 2.0 in. ID
Boring Location:	See Plan	Casing ID/OD:	NW - 3.0 in. ID	Water Level*:	9.6
Hammer Efficiency Factor:	0.84	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
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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	109.1		-BITUMINOUS CONCRETE- 0.4	
	1D	24/18	1.0 - 3.0	11/9/7/6	16	22				Brown, dry, medium dense, silty fine to medium SAND, trace coarse sand, trace fine to coarse gravel, trace asphalt -FILL-(SC-SM)	G#244214 A-4, SC-SM WC=3.9%
	2D	24/14	3.0 - 5.0	4/4/5/5	9	13				Brown, dry, loose, silty fine to medium SAND, little fine to coarse gravel, trace coarse sand -FILL-(SC-SM)	G#244215 A-4, SC-SM WC=3.8%
5	3D	24/12	5.0 - 7.0	4/4/4/4	8	11	5			Brown, dry, loose, fine to medium SAND, some silt, trace fine to coarse gravel -FILL-(SC-SM)	G#244216 A-2-4, SC-SM WC=5.0%
							16				
	4D	24/5	7.0 - 9.0	8/4/1/1	5	7	13			Brown, wet, loose, fine to coarse SAND, some silt, some fine gravel, trace cinders -FILL-(SC-SM)	G#244217 A-2-4, SC-SM WC=14.6%
							6	100.5		9.0	
	5D	24/6	9.0 - 11.0	2/1/1/1	2	3	5			Brown, wet, very loose, silty fine to coarse SAND, little fine to coarse gravel, trace wood and plant fibers -FILL-(SC-SM)	G#244218 A-4, SC-SM WC=21.7 PI=NP
10							4	98.5		11.0	
	6D	10/5	11.0 - 11.8	4/100(3.0")			4	97.7		Light brown, saturated, very loose, fine to medium SAND, some silt, trace fine to coarse gravel, trace wood -ALLUVIAL DEPOSIT-(SC-SM) 11.8	G#244219 A-2-4, SC-SM WC=16.2%
	R1	60/60	12.9 - 17.9	RQD = 0%			6	96.6		-PROBABLE WEATHERED BEDROCK- 12.9	
							16			Top of Bedrock at El.-12.9 ft Gray, aphanitic to fine grained SCHIST. Hard, very slightly weathered. Primary joints dipping at steep to vertical angles, close, smooth, planar to stepped, discolored, open. Rock Mass Quality=Very Poor -WATERVILLE FORMATION- R1 Core Times (min:sec): 12.9-13.9' (2:10); 13.9-14.9' (2:08); 14.9-15.9' (1:55); 15.9-16.9' (2:22); 16.9-17.9' (3:28)	
15											
	R2	60/60	17.9 - 22.9	RQD = 80%						Gray, aphanitic to fine grained SCHIST. Hard, very slightly weathered. Primary joints dipping at steep to vertical angles, moderately close, smooth, planar, discolored, open. Rock Mass Quality=Good -WATERVILLE FORMATION- R2 Core Times (min:sec): 17.9-18.9' (1:22); 18.9-19.9' (2:01); 19.9-20.9' (2:08); 20.9-21.9' (2:43); 21.9-22.9' (2:18)	GTX#12141 qp=5,120 psi GTX#12141 qp=8,977 psi
20											
								86.6		22.9 **Bottom of Exploration at 22.9 feet below ground surface.**	
25											

Remarks:

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

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

Unconfined compression (Figure 3.3*a*) is the most frequently used strength test for rocks, yet it is not simple to perform properly and results can vary by a factor of more than two as procedures are varied. The test specimen should be a rock cylinder of length-to-width ratio in the range 2 to 2.5 with flat, smooth, and parallel ends cut perpendicularly to the cylinder axis. Procedures are recommended in ASTM designation D2938-71a and by Bieniawski and Bernede (1979). Capping of the ends with sulfur or plaster to specified smoothness is thought to introduce artificial end restraints that overly strengthen the rock. However, introduction of Teflon pads to reduce friction between the ends and the loading surfaces can cause outward extrusion forces producing a premature splitting failure, especially in the harder rocks. When mine pillars are studied, it is sometimes preferable to machine the compression specimen from a large cylinder to achieve loading through rock of the upper and lower regions into the more slender central region. In the standard laboratory compression test, however, cores obtained during site exploration are usually trimmed and compressed between the crosshead and platen of a testing machine. The compres-

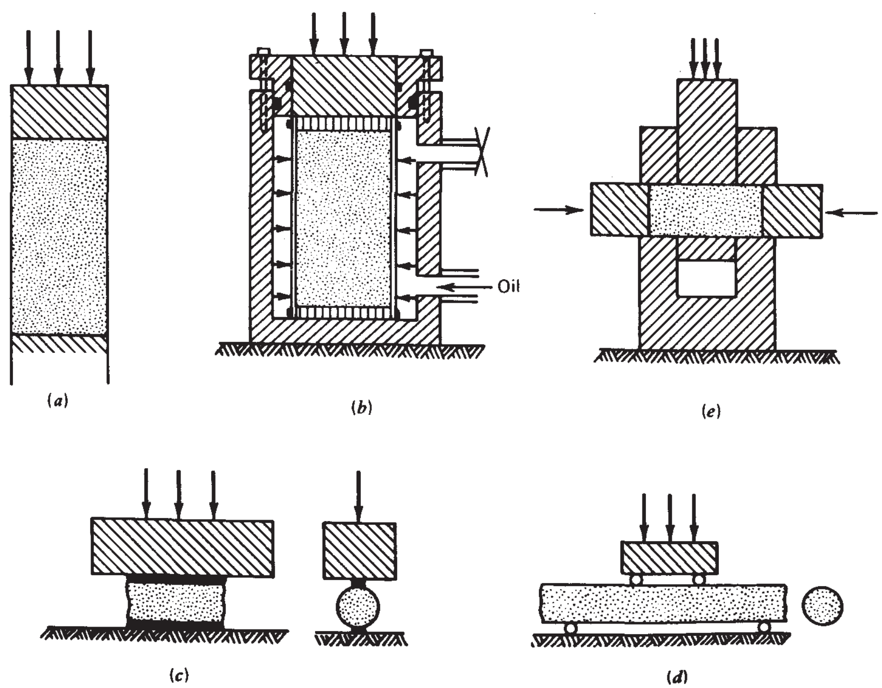


Figure 3.3 Common laboratory tests for characterizing rock strength criteria. (*a*) Unconfined compression. (*b*) Triaxial compression. (*c*) Splitting tension (Brazilian). (*d*) Four-point flexure. (*e*) Ring shear.

sive strength q_u is expressed as the ratio of peak load P to initial cross-sectional area A:

$$q_u = \frac{P}{A} \tag{3.1}$$

Representative values of q_u are listed in Table 3.1.

Triaxial compression (Figure 3.3*b*) refers to a test with simultaneous compression of a rock cylinder and application of axisymmetric confining pressure. Recommended procedures are described in ASTM designation D2664-67 (1974) and in an ISRM Committee report by Vogler and Kovari (1978).

***Table 3.1** Unconfined Compressive Strength* (q_u) *and Ratio of Compressive to Indirect Tensile Strength* (q_u/T_0) *for Specimens of Representative Rocks*

Description[a]	q_u MPa	q_u psi	q_u/T_0[b]	Reference[c]
Berea sandstone	73.8	10,700	63.0	5
Navajo sandstone	214.0	31,030	26.3	5
Tensleep sandstone	72.4	10,500		1
Hackensack siltstone	122.7	17,800	41.5	5
Monticello Dam s.s. (greywacke)	79.3	11,500		4
Solenhofen limestone	245.0	35,500	61.3	5
Bedford limestone	51.0	7,400	32.3	5
Tavernalle limestone	97.9	14,200	25.0	5
Oneota dolomite	86.9	12,600	19.7	5
Lockport dolomite	90.3	13,100	29.8	5
Flaming Gorge shale	35.2	5,100	167.6	3
Micaceous shale	75.2	10,900	36.3	2
Dworshak Dam gneiss				
45° to foliation	162.0	23,500	23.5	5
Quartz mica schist ⊥ schistocity	55.2	8,000	100.4	5
Baraboo quartzite	320.0	46,400	29.1	5
Taconic marble	62.0	8,990	53.0	5
Cherokee marble	66.9	9,700	37.4	5
Nevada Test Site granite	141.1	20,500	12.1	7
Pikes Peak granite	226.0	32,800	19.0	5
Cedar City tonalite	101.5	14,700	15.9	6
Palisades diabase	241.0	34,950	21.1	5
Nevada Test Site basalt	148.0	21,500	11.3	7
John Day basalt	355.0	51,500	24.5	5
Nevada Test Site tuff	11.3	1,650	10.0	7

[a] Description of rocks listed in Table 3.1:

Berea sandstone, from Amherst, Ohio; fine grained, slightly porous; cemented. *Navajo sandstone*, from Glen Canyon Dam site, Arizona; friable, fine to medium grained. (Both sandstones are

Table Footnote (*continued*)

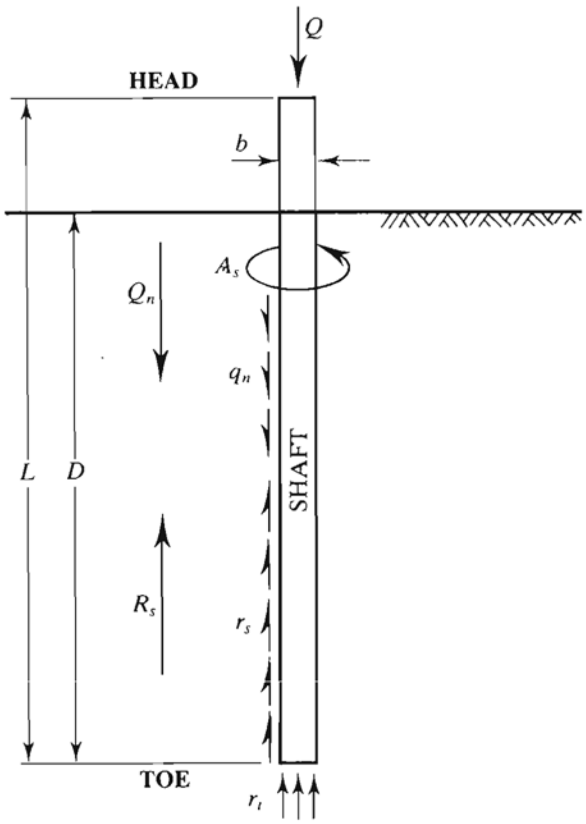


Fig. 13.4 Terms and symbols for pile analysis. Q_d = dead load; Q_l = live load; Q_n = drag load; Q_u = ultimate load (= capacity); R_s = shaft resistance; R_t = toe resistance; R_u = ultimate resistance (= capacity); L = pile length; D = embedment depth; b = pile diameter; A_s = circumferential area; A_t = toe cross-sectional area; N_t = toe bearing capacity coefficient.

$$R_t = A_t r_t - A_t \sigma'_{z=0} N_t$$

$$R_s = \Sigma A_s r_s = \Sigma A_s \beta \sigma'_z \quad \text{or} \quad \Sigma A_s (c' + \beta \sigma'_z)$$

function of both friction and cohesion. Equation 13.1 then changes to:

$$r_s = c' + \beta \sigma'_z \tag{13.3}$$

where c' = effective cohesion intercept.

Although it has been proven conclusively that the transfer of load from a pile to the soil by means of shaft resistance is governed by the effective stress, for piles in clay, a total stress analysis can be useful in site-specific instances. Also, enough information is often not available to support a reliable design based on effective stress analysis. A total stress analysis may then be used, which means that the shaft resistance is equal to the undrained shear strength of the soil and independent of the overburden stress:

$$r_s = \alpha \tau_u \tag{13.4}$$

TABLE 13.1 RANGES OF BETA-COEFFICIENTS.

Soil Type	*phi*	*beta*
Clay	25–30	0.23–0.4
Silt	28–34	0.27–0.5
Sand	32–40	0.30–0.8
Gravel	35–45	0.35–0.8

where

τ_u = undrained shear strength
α = proportionality coefficient

However, the total stress analysis can only lead so far and effective stress analysis according to Equations 13.1 and 13.3 provides the better means for analysis of test data and for putting experience to use in a design. Of course, more sophisticated effective stress theories for unit shaft resistance exist. However, in contrast to most of these, the effective stress approach according to Equations 13.1 and 13.3 is not restricted to homogeneous soils, but applies equally well to piles in layered soils and it can easily accommodate non-hydrostatic pore pressures.

The proportionality coefficient is equal to unity in soft and firm clays, but smaller than unity in stiff and hard clays, especially if they are overconsolidated. A useful qualitative reference is illustrated in Figure 13.5, showing that for wood and concrete piles the proportionality coefficient is equal to unity up to a shear strength of about 30 kPa, whereupon it becomes progressively smaller. For steel piles, the coefficient is indicated as smaller than unity even for soft clays.

Equation 13.5 gives the total shaft resistance as the integral of the unit shaft resistance over the embedment depth:

$$R_s = \int_0^D r_s dz = \int_0^D A_s(c' + \beta \sigma'_z) dz \tag{13.5}$$

where

R_s = total shaft resistance (fully mobilized)
A_s = pile unit circumferential area
D = pile embedment depth

It is important to realize that even simple axial loading of a single pile can be made in several different ways. Figure 13.6 illustrates six cases, A through F, of axial loading. Case A shows a pile loaded with a compression load (push load) at the pile head. The transfer of load to the soil increases the effective stress in the soil and produces compression stress in the pile. The increased stress in the pile causes an increase of pile diameter (Poisson's ratio effect; a minimal increase, of course). These aspects are symbolically indicated in the figure.

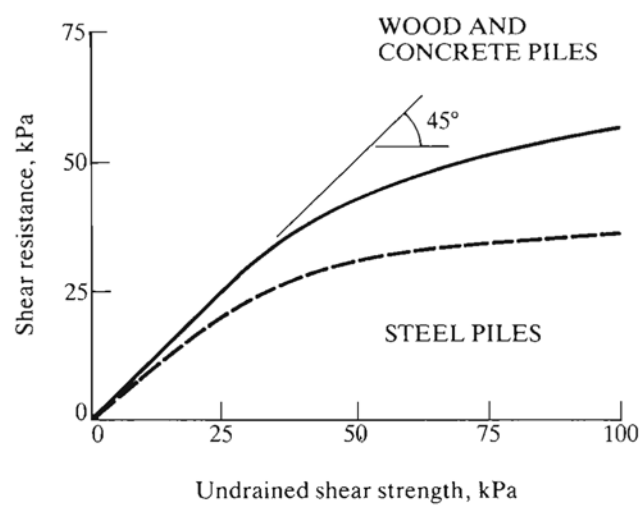


Fig. 13.5 Shaft resistance in clay as a function of undrained shear strength. (*After Tomlinson, 1957.*)

the at-rest pressure coefficient, K_o, at a greater depth. Based on presently available results, the following average values of K are recommended for use in Eq. (11.41):

Pile type	***K***
Bored or jetted	$\approx K_o = 1 - \sin\phi'$
Low-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.4K_o = 1.4(1 - \sin\phi')$
High-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.8K_o = 1.8(1 - \sin\phi')$

The values of δ' from various investigations appear to be in the range from $0.5\phi'$ to $0.8\phi'$.

Based on load test results in the field, Mansur and Hunter (1970) reported the following average values of K.

H-piles. . . . . . $K = 1.65$

Steel pipe piles. . . . . . $K = 1.26$

Precast concrete piles. . . . . . $K = 1.5$

Coyle and Castello (1981), in conjunction with the material presented in Section 11.9, proposed that

$$Q_s = f_{av} pL = (K\overline{\sigma}'_o \tan\delta')pL \tag{11.43}$$

where

$\overline{\sigma}'_o$ = average effective overburden pressure
δ' = soil–pile friction angle $= 0.8\phi'$

The lateral earth pressure coefficient K, which was determined from field observations, is shown in Figure 11.17. Thus, if that figure is used,

$$Q_s = K\overline{\sigma}'_o \tan(0.8\phi')pL \tag{11.44}$$

Correlation with Standard Penetration Test Results

Meyerhof (1976) indicated that the average unit frictional resistance, f_{av}, for high-displacement driven piles may be obtained from average standard penetration resistance values as

$$f_{av} = 0.02p_a(\overline{N}_{60}) \tag{11.45}$$

where

$(\overline{N}_{60})$ = average value of standard penetration resistance
p_a = atmospheric pressure ($\approx$100 kN/m^2 or 2000 lb/ft^2)

For low-displacement driven piles

$$f_{av} = 0.01p_a(\overline{N}_{60}) \tag{11.46}$$

Factored Bearing Resistances for Pier Calculations

OBJECTIVE

Estimate the factored bearing resistances of the center pier at the service, strength, and extreme limit states.

GIVEN

1) Limited lab data and boring logs

ASSUMPTIONS

1) Estimated soil properties.
2) An estimated unconfined compressive strength of the bedrock = 8,000 psi (refer to Page 2).
3) Design per AASHTO LRFD Bridge Design Specifications, 2014, 7th edition, with 2016 interims.

ESTIMATE THE FACTORED BEARING RESISTANCE AT THE SERVICE LIMIT STATE

Reference: LRFD Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

For broken bedrock of any kind:

Type of Bearing Material: Weathered or broken bedrock of any kind, except shale.

Consistency of in-situ bedrock: Medium hard rock

Nominal bearing resistance: Ordinary range is 16 to 24 ksf

Recommended nominal bearing resistance value to use (based on prior projects with similar subsurface conditions): 20 ksf

Resistance factor at the **service limit state** = 1.0 (LRFD Article 10.5.5.1)

$$\phi_{bearing\_service} := 1.0$$

$$q_{nominal\_bearing} := 20\ ksf$$

$$q_{factored\_bearing} := \phi_{bearing\_service} \cdot q_{nominal\_bearing}$$

$$q_{factored\_bearing} = 20\ ksf$$

Recommend a factored bearing resistance of 20 ksf at the Service Limit State

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

ESTIMATE THE FACTORED BEARING RESISTANCE AT THE STRENGTH LIMIT STATE

Determine the factored bearing resistance at the strength limit state using the Rock Mass Rating (RMR) system per LRFD Article 10.6.3.2.2 and C4.6.4 and Table 10.4.6.4-1 (LRFD, 6th Ed., 2012).

Bedrock at the proposed center mass pier location (BB-WLLOR-102) was found to be "very poor" in quality. The RQD was 10%. The borings indicate that the bedrock ranges from a SCHIST to a PEGMATITE/MIGMATITE.

Determine RMR from Table 10.4.6.4-1 (LRFD, 6th Ed., 2012) Geomechanics Classification of Rock Mass

Per LRFD, the RMR is determined as the sum of the five relative ratings listed in Table 10.4.6.4-1 (LRFD, 2012)

Parameter No. 1: Strength of intact rock material

Assume the design value of the unconfined compressive strength (qu) of the rock is **8,000 psi**. This is based on:

a) The unconfined compressive strength tests on schist from the Waterville Formation (schist) ranged from 5,120-8,766 psi.
b) The average unconfined compressive strength of schist from the Richmond-Dresden project was approximately 7,000 psi.
c) Goodman recommends a qu = 8,000 psi for quartz mica schist (Rock Mechanics, 1989, Table 3-1, page 61), qu:

$q_u := 8000\ psi$

$q_u = 1152\ ksf$

Therefore, since the unconfined compressive strength of the rock is between 7.5 and 14 ksi, the Relative Rating (RR) for this parameter is 7.

$RR_{parameter1} := 7$

Parameter No. 2: Drill core quality RQD

The average of the RQDs at the two cores at the center pier location was 28%. Therefore, since the RQD is between 25-50%, the Relative Rating for this parameter is **8**.

$RR_{parameter2} := 8$

Parameter No. 3: Spacing of joints

The jointing of the rock cores at the center pier location were characterized as "closely spaced". Therefore, since the spacing of joints is between 2 to 12 inches, the Relative Rating for this parameter is **10**.

$RR_{parameter3} := 10$

Parameter No. 4: Condition of joints

Assume the rock has slightly rough surfaces, joint separations of less than 0.05 inches, and soft joint wall rock. Therefore, the Relative Rating for this parameter is **12**.

$RR_{parameter4} := 12$

Parameter No. 5: Groundwater conditions - general conditions

There were no groundwater measurements during the investigation. Assume the "water under moderate pressure." Therefore, the Relative Rating for this parameter is **4**.

$RR_{parameter5} := 4$

Now, sum up the five Relative Ratings for the five parameters above to calculate the Raw RMR:

$$RMR_{raw} := RR_{parameter1} + RR_{parameter2} + RR_{parameter3} + RR_{parameter4} + RR_{parameter5}$$

$$RMR_{raw} = 41$$

Parameter No. 6: Now, adjust the Raw RMR considering the Joint Orientations from LRFD Table 10.4.6.4-2 (LRFD, 2012)

At the center pier location, the jointing was described as typically horizontal with several steeply dipping joints (45-60 degrees). Assume that for foundations, the strike and dip orientations of joints are FAIR. Therefore, the Relative Rating for this parameter is **-7**.

$RR_{parameter6} := -7$

The Adjusted RMR can now be calculated by summing Parameters No. 1 through No. 6

$$RMR_{adjusted} := RMR_{raw} + RR_{parameter6}$$

$$RMR_{adjusted} = 34$$

Determine Rock Mass Class from Adjusted RMR Rating

Reference: LRFD, 2012, 6th Edition Table 10.4.6.4-3.

With an Adjusted RMR = 34, the rock can be described as "Poor rock - Class IV."

Determine Rock Type

Reference: LRFD, 2012, 6th Edition Table 10.4.6.4-4.

The bearing material is assumed to be a schist and/or a pegmatite/migmatite. Therefore, the Rock Type is E, since the bearing material is likely a "coarse-grained polyminerallic igneous and metamorphic rock - *amphibolite, gabbro gneiss, granite, norite, quartz-diorite."*

Determine the Intact Rock Mass "m" and "s" constants

Reference: The Hoek and Brown Failure Criterion - an 1988 Update, 15th Canadian Rock Mechanics Symposium.

To calculate the disturbed rock mass constants (m and s), the m and s constants assuming "intact rock samples" are used with Rock Type E. Therefore, the intact rock mass constants are mi = 25 and s = 1

$$m_i := 25$$

$$s := 1$$

Now, calculate the disturbed rock mass constants per Equation 18 and 19 (Hoek and Brown, 1988).

$$m := m_i \cdot \exp\left(\frac{RMR_{adjusted} - 100}{14}\right)$$

$$m = 0.224$$

$$s := \exp\left(\frac{RMR_{adjusted} - 100}{6}\right)$$

$$s = 0.000017$$

Determine nominal bearing resistance of the center pier

Per Table 5.4 (page 138 of Wyllie), the foundation shape factor for a rectangular shape (assuming L/B = 2) is 1.12

Reference: Foundations on Rock, 2nd Edition, Dr. Duncan Wyllie, P.Eng, 2009

$$C_{f1} := 1.12$$

Use the lower and middle bounds of the recommended uniaxial compressive strength values for a Schist.

Reference: Standard Specifications for Highways, 17th Edition, 2002, Table 4.4.8.1.2B

$$q_{uc} := \begin{bmatrix} 1400 \\ 8000 \\ 14500 \\ 21000 \end{bmatrix} psi$$

Reference: Foundations on Rock, 2nd Edition, Dr. Duncan Wyllie, P.Eng, 2009

Per Equation 5.4 (page 138 of Wyllie), the nominal bearing resistance may be taken as (excluding the factory of safety term since we are using LRFD):

$$q_{nominal\_bearing} := C_{f1} \cdot \sqrt{s} \cdot q_{uc} \cdot \left(1 + \sqrt{m \cdot \left(s^{\frac{-1}{2}}\right) + 1}\right)$$

$$q_{nominal\_bearing} = \begin{bmatrix} 8 \\ 45 \\ 81 \\ 117 \end{bmatrix} ksf$$

Determine the factored bearing resistance of the center pier at the Strength Limit State

The resistance factor for bedrock is 0.45

Reference: LRFD Table 10.5.5.2.2-1

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

Therefore, per LRFD Eq. 10.6.3.1.1-1, the factored bearing resistance is:

$$q_{bearing\_strength} := q_{nominal\_bearing} \cdot \phi_{bearing}$$

$$q_{bearing\_strength} = \begin{bmatrix} 4 \\ 20 \\ 36 \\ 53 \end{bmatrix} ksf$$

Recommend a factored bearing resistance of 20 ksf at the Strength Limit State

Determine the factored bearing resistance of the bedrock at the Extreme Limit State

Use a bearing resistance factor of 0.80 for Extreme Limit State for gravity and semigravity walls per LRFD Article C11.5.8. Use for piers for consistency with the theory of preventing collapse for the Extreme Event.

$$\phi_{bearing\_extreme} := 0.80$$

$$q_{bearing\_extreme} := q_{nominal\_bearing} \cdot \phi_{bearing\_extreme}$$

$$q_{bearing\_extreme} = \begin{bmatrix} 6 \\ 36 \\ 65 \\ 94 \end{bmatrix} ksf$$

Recommend a factored bearing resistance of 36 ksf at the Extreme Limit State

Table 1 : Approximate relationship between rock mass quality and material constants

Disturbed rock mass m and s values — *undisturbed rock mass m and s values*

EMPIRICAL FAILURE CRITERION $\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2}$ σ_1' = major principal effective stress σ_3' = minor principal effective stress σ_c = uniaxial compressive strength of intact rock, and m and s are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTAL-LINE ROCKS – *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES *Laboratory size specimens free from discontinuities* CSIR rating: RMR = 100 NGI rating: Q = 500	m s *m* *s*	7.00 1.00 *7.00* *1.00*	10.00 1.00 *10.00* *1.00*	15.00 1.00 *15.00* *1.00*	17.00 1.00 *17.00* *1.00*	25.00 1.00 *25.00* *1.00*
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.* CSIR rating: RMR = 85 NGI rating: Q = 100	m s *m* *s*	2.40 0.082 *4.10* *0.189*	3.43 0.082 *5.85* *0.189*	5.14 0.082 *8.78* *0.189*	5.82 0.082 *9.95* *0.189*	8.56 0.082 *14.63* *0.189*
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.* CSIR rating: RMR = 65 NGI rating: Q = 10	m s *m* *s*	0.575 0.00293 *2.006* *0.0205*	0.821 0.00293 *2.865* *0.0205*	1.231 0.00293 *4.298* *0.0205*	1.395 0.00293 *4.871* *0.0205*	2.052 0.00293 *7.163* *0.0205*
FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1m.* CSIR rating: RMR = 44 NGI rating: Q = 1	m s *m* *s*	0.128 0.00009 *0.947* *0.00198*	0.183 0.00009 *1.353* *0.00198*	0.275 0.00009 *2.030* *0.00198*	0.311 0.00009 *2.301* *0.00198*	0.458 0.00009 *3.383* *0.00198*
POOR QUALITY ROCK MASS *Numerous weathered joints at 30-500mm, some gouge. Clean compacted waste rock* CSIR rating: RMR = 23 NGI rating: Q = 0.1	m s *m* *s*	0.029 0.000003 *0.447* *0.00019*	0.041 0.000003 *0.639* *0.00019*	0.061 0.000003 *0.959* *0.00019*	0.069 0.000003 *1.087* *0.00019*	0.102 0.000003 *1.598* *0.00019*
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced <50mm with gouge. Waste rock with fines.* CSIR rating: RMR = 3 NGI rating: Q = 0.01	m s *m* *s*	0.007 0.0000001 *0.219* *0.00002*	0.010 0.0000001 *0.313* *0.00002*	0.015 0.0000001 *0.469* *0.00002*	0.017 0.0000001 *0.532* *0.00002*	0.025 0.0000001 *0.782* *0.00002*

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

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

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

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

Disturbed rock masses :

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

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

Undisturbed or interlocking rock masses:

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

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

where

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

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

$$\text{RMR} = 9 \,\text{Log}_e\, \text{Q} + 44 \qquad (22)$$

Limitations on using failure criterion

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

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

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

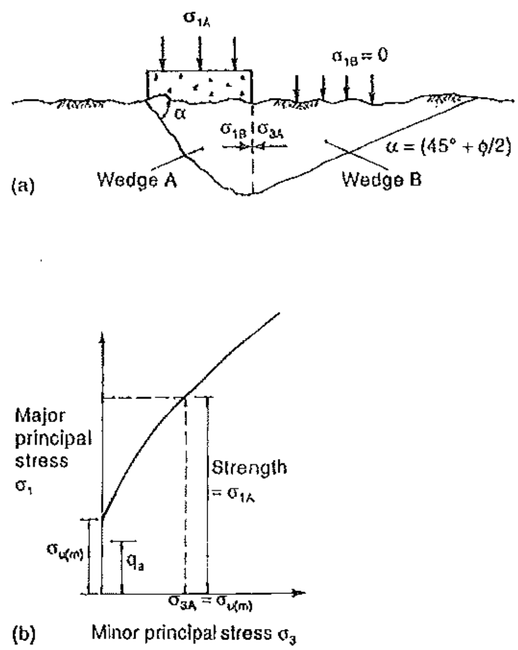


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a, strength of bearing rock = σ_{1A}, factor of safety $FS = \sigma_{1A}/q_a$.

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

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

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

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

The factor C_{f1} is applied to the calculated allowable bearing pressure to account for the shape of the foundation and has the values given in Table 5.4 (Sowers, 1970).

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

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

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

5.2.3 Recessed footings

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

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

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

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

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

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

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

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

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

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

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

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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

LPile (Soil and Bedrock) Parameter Calculations

LK 12/15/2015

Soil Layer	Approximate Top and Bottom Elevations of Soil Layer	Above/Below Groundwater?	Effective Unit Weight, γ'	Soil Modulus, k_s	Internal Angle of Friction, ϕ
-	feet	-	lbs/ft^3 (lbs/in^3)	lbs/in^3	degrees
Medium dense, SAND, (Fill)	321.1 – 316.1	Above	117 (0.068)	90	33
Medium dense, SAND, (Fill)	316.1 – 312.5	Below	55 (0.032)	60	33
Medium dense, Silty SAND, (Native)	312.5 – 303.5	Below	48 (0.028)	50	32
Dense, Silty SAND, (Native)	303.5 – 295.8	Below	80 (0.046)	125	38

Table 1. Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 1

Soil Layer	Approximate Top and Bottom Elevations of Soil Layer	Above/Below Groundwater?	Effective Unit Weight, γ'	Soil Modulus, k_s	Internal Angle of Friction, ϕ
-	feet	-	lbs/ft^3 (lbs/in^3)	lbs/in^3	degrees
Medium dense, SAND, (Fill)	320.7 – 315.7	Above	117 (0.068)	90	33
Medium dense, SAND, (Fill)	315.7 – 310.5	Below	50 (0.029)	50	31
Medium dense, Silty SAND, (Native)	310.5 – 307.0	Below	48 (0.028)	50	32
Dense, Silty SAND, (Native)	307.0 – 301.8	Below	80 (0.046)	125	38

Table 2. Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2

OBJECTIVE

Provide the structural engineer, Hoyle Tanner Assoc. (HTA) with soil parameters at both abutments for LPILE analyses.

ASSUMPTIONS

- Will utilize common engineering correlations to estimate integral soil parameters
- Based on field descriptions, the groundwater table could be at 5 feet bgs (damp to wet) at BB-WLLOR-101

GIVEN

Lab testing data from geotechnical investigation.

EVALUATION

1) ABUTMENT No. 1 (see attached draft boring log' BB-WLLOR-101)

a) Estimate the following parameters for FILL layer (from Elev. 321 ft to 312.5 ft, 9 ft):

- Effective Unit Weight, γ_t' (pcf and pci)
- Soil modulus parameter, K_s (pci)
- Angle of internal friction, ϕ', (degrees)
- Layer 1: 0-5 feet & Layer 2: 5-9 feet

b) Using Reese (1979), use the corrected blow counts to estimate K_s and ϕ' values utilizing the relative density correlations presented in the Reese paper.

c) $D_R = \left(\dfrac{N}{20(1+2\sigma_v')} \right)^{0.5}$ (Equation 5.1)

↳ Need estimated σ_v' at midpoint of FILL layer (4.5' bgs)

↳ Estimate γ_t of FILL per NAVFAC 7.1-22

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↳ Per NAVFAC, Table 6, 7.1-22, a well-graded sand has a min. γ_{sat} of 86 pcf and a max. γ_{sat} of 148 pcf (medium dense → 117 pcf)

$\gamma' = \gamma_{sat} - \gamma_w$

$\therefore \gamma'_{Layer\,1} = \underline{117\text{ pcf}}$

$\gamma'_{Layer\,2} = 117\text{ pcf} - 62.4\text{ pcf} = \underline{55\text{ pcf}}$

↳ $\therefore \sigma_v' @ 5' bgs = 5\text{ feet}(112\text{ pcf})$

$= \underline{560\text{ psf}} \; (= 0.56\text{ ksf})$

↳ $\therefore D_{R,FILL} = \left(\frac{\left(\frac{24+27}{2}\right)}{20(1+2(0.56\text{ksf}))}\right)^{0.5}$

$\underline{D_{R,FILL} \simeq 80\%}$ 77%

d) Estimate internal angle of friction

↳ Per Schmertmann, 1975, Fig. 5.1 of the Reese paper:

$\phi_{FILL_{1,2}} = 33°$ (value was decreased to consider unknown origin of fill placement and considering that a $\phi = 38°$ is too high). OK < 34°

e) Estimate K_s

↳ Above groundwater table (Layer 1) = 90 pci

↳ Below groundwater table (Layer 2) = 60 pci

f) Now, estimate the same soil parameters for Layer 3 (Elev. 312.5 ft to Elev. 303.5 ft)

↳ This native silty sand layer has N_{60} values from 15-20.

↳ The density increases at Elev. 303.5 ft (start of Layer 4)

g) Estimate γ'

↳ Using Table 6: Silty sand has a max γ_{sat} of 142 pcf and a min. γ_{sat} of 88 pcf.

↳ The silty sand was found to be medium dense; therefore, assume the γ_{sat} of Layer 3 is 115 pcf.

However the blow counts were on the lower end of "medium dense".

Say γ_{sat} = 110 pcf

$\therefore \gamma'_{Layer\ 3} = 110\text{ pcf} - 62.4\text{ pcf} = 48\text{ pcf}$

h) Estimate K_s of Layer 3

$K_{s,\ layer\ 3}$ = 50 pci (per Table 5.1)

i) Estimate ϕ

↳ σ_v' @ 14.5 ft bgs = 5 ft (117 pcf) + 4 ft (117 - 62.4 pcf)

+ 5.5 ft (110 - 62.4 pcf)

= 1065 psf ~ 1.065 ksf

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PRELIM. BY ________ DATE ________ PROJ. NO. ________ FILE NO. ________ OF ________
FINAL CHK. BY ________ DATE ________ LOCATION ________ SHEET NO. 4 OF 19
ITEM NO. ________ SUBJECT ________

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↳ $D_R = \left(\dfrac{N}{20(1+2\sigma_v')} \right)^{0.5}$

$= \left[\dfrac{\left(\frac{15+20}{2}\right)}{20\,(1+2(1.065))} \right]^{0.5}$

$D_{R,\,layer\,3} = 53\%$

↳ Per Schmertmann (1975), $\phi_{layer\,3} = 35°$ (too high!)

↳ Say $\phi_{layer\,3} = 32°$ (Japan National Railways) & Peck, Hansen, Thornburn. ✓ OK 32°

j) Now, estimate soil parameters for Layer 4 (Elev. 303.5' to Elev. 295.8 ft → top of bedrock)

K) Estimate γ'

↳ This layer is a dense silty sand w/ some gravel.

↳ ∴ γ_{sat} of dense silty sand is 142 pcf.

↳ Say $\gamma'_{layer\,4}$ = 142 pcf − 62.4 pcf = 80 pcf

l) Estimate K_s

↳ $K_{s,\,layer\,4}$ = 125 pci

m) Estimate ϕ

↳ Per Japan National Railways & Peck & Hansen:

Schmertmann 38°
PHT 40°

$\phi = 0.3N + 27$

$= 0.3(47) + 27 \rightarrow \phi = 41.1° \rightarrow$ say $\phi = 38°$

KMR-1

n) See attached table for soil parameters of Abut. # 1

2) ABUTMENT No. 2 (see attached BB-WLLOK-103 for layer delineation).

a) Estimate γ' for Layer 1 (sandy fill above groundwater)

↳ γ_{sat} (86 - 148 pcf)

↳ γ_{sat} for medium dense = 117 pcf

↳ $\gamma'_{LAYER 1}$ = 117 pcf - 0 pcf

↳ $\gamma'_{LAYER 1}$ = 117 pcf

b) Estimate ϕ for Layer 1

↳ $\phi = 0.3N + 27$ (degrees)

$= 0.3(23) + 27$

$\phi_{LAYER 1} = 33°$ (use same value as Abut No. 1)

OK < 34°

c) Estimate K_s for Layer 1

$K_{s\,LAYER 1}$ = 90 pci (above water)

d) Estimate $\gamma'_{LAYER 2}$

↳ $\gamma'_{LAYER 2}$ = 112 pcf - 62.4 pcf

lowered value due to decreased density from layer 1

↳ $\gamma'_{LAYER 2}$ = 50 pcf

MaineDOT

PRELIM. BY ______ DATE ______ PROJ. NO. ______ FILE NO. ______ OF ______
FINAL CHK. BY ______ DATE ______ LOCATION ______ SHEET NO. 6 OF 19
ITEM NO. ______ SUBJECT ______

e) Estimate $K_{s\,LAYER\,2}$

↳ $K_{s\,LAYER\,2} = 50$ pci (below water)

same logic as decreased γ' value in step d)

f) Estimate $\phi_{LAYER\,2}$

↳ $\phi_{LAYER\,2} = 0.3N + 27$ (degrees)

$= 0.3(12) + 27$

↳ $\phi_{LAYER\,2} = 31°$

✓ OK 31°

g) Now estimate properties for Layer 3 (Elev. 310.5 ft to Elev. 306.5 ft)

h) Estimate $\gamma'_{LAYER\,3}$

↳ $\gamma_{sat} = 88 - 142$ pcf $\rightarrow (\gamma_{sat})_{medium\ dense} \cong 110$ pcf.

↳ $\gamma'_{LAYER\,3} = 110$ pcf $- 62.4$ pcf

↳ $\gamma'_{LAYER\,3} = 48$ pcf

i) Estimate $K_{s\,LAYER\,3}$

↳ $K_{s\,LAYER\,3} = 50$ pci

j) Estimate $\phi_{LAYER\,3}$

↳ $\phi_{LAYER\,3} = 0.3N + 27$ (degrees)

$= 0.3(15) + 27$

$\phi_{LAYER\,3} = 32°$

≈ 31.5

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KMR-1

PRELIM. BY ______ DATE ______ PROJ. NO. ______ FILE NO. ______ OF ______
FINAL CHK. BY ______ DATE ______ LOCATION ______ SHEET NO. 7 OF 19
ITEM NO. ______ SUBJECT ______

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k) Now, estimate parameters for Layer 4 (dense silty sand from Elev. 307 ft to 301.8 ft)

l) Estimate $\gamma'_{LAYER\,4}$

↳ γ_{sat} dense SM w/ some gravel = 142 pcf

↳ $\gamma' = \gamma_{sat} - \gamma_w$

$= 142\text{ pcf} - 62.4\text{ pcf}$

$\gamma'_{LAYER\,4} = 80\text{ pcf}$

m) Estimate $K_{s\,LAYER\,4}$

↳ $K_{s,LAYER\,4} = 125\text{ pci}$

n) Estimate $\phi_{LAYER\,4}$

↳ $\phi_{LAYER\,4} = 0.3N + 27$ (degrees)

$= 0.3(39) + 27$

$\phi_{LAYER\,4} = 38°$ OK (38.1

KMR-1

(8/19)

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Stimson Bridge #2807 carries Route 5 over Little Ossipee River Location: Waterboro-Limerick, Maine	Boring No.: BB-WLLOR-101 PIN: 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	321.5	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/14/2015; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 150+14.6, 7.6 feet left	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.908	**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	321.13		4.5-inch-thick layer of pavement 0.38	
	1D	24/18	2.00 - 4.00	4/10/6/10	16	24				Brown, damp, medium dense, SAND, some gravel, trace silt, well-graded, occasional small cobbles, (Fill).	G#263919 A-1-b, SW-SM WC=3.7%
										LAYER 1	
5	2D	24/17	5.00 - 7.00	4/9/9/8	18	27				Similar to above, except wet.	
										LAYER 2	
								312.50		9.00	
10	3D	24/6	10.00 - 12.00	5/5/5/5	10	15	19			Grey-brown, wet, medium dense, Silty SAND, some gravel, little silt, (River Alluvium and/or Glaciolacustrine Delta Deposits).	
							33				
							37			LAYER 3	
							37				
							34			Wood fragments from 14.0-14.4 feet bgs.	
15	4D	24/2	15.00 - 17.00	12/7/6/6	13	20	10			Similar to above.	
							16				
							25	303.5			
							66				
							108				
20	5D	24/12	20.00 - 22.00	16/16/15/36	31	47	5			Grey, wet, dense, Silty SAND, some gravel, little silt.	G#263920 A-1-b, SM WC=12.6%
							5			LAYER 4	
							35				
							146			RC to 25.0 feet bgs.	
							189				
25											

Remarks:

- 400 pounds of down pressure on Core Barrel
- bgs = below ground surface

Piles in Biaxial Bending → Curved Bridge on Skew

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.

(9/19)

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine

Boring No.: BB-WLLOR-101
PIN: 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	321.5	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/14/2015; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 150+14.6, 7.6 feet left	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.908	**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	6D	8.4/8.4	25.00 - 25.70	26/50(2.4")	---		a50	295.80		a50 blows for 8.4 inches very dense 25.70	G#263921 A-2-4 to A-2-7, SM WC=13.5%
	R1	60/59	25.70 - 30.70	RQD = 22%			NQ-2			Top of Bedrock at Elev. 295.8 feet. R1: Bedrock: Red-brown to grey with white banding (composed of primarily quartz and feldspar that are not foliated) foliated quartz, muscovite mica, feldspar, amphibole, and garnet SCHIST, medium-grained, moderately hard, moderately weathered, Lower Member of the Rindgemere Formation. Rock Mass Quality = Very Poor R1:Core Times (min:sec) 25.7-26.7 ft (2:00) 26.7-27.7 ft (2:00) 27.7-28.7 ft (3:18) 28.7-29.7 ft (3:36) 29.7-30.7 ft (3:10) No Water return 98% recovery 30.70	
30	R2	60/60	30.70 - 35.70	RQD = 48%				290.80		R2: Bedrock: The top 15" is similar to R1 but with increased iron staining. The bottom of R2 consists of massive PEGMATITE/ MIGMATITE with the foliation becoming less pronounced. No iron staining in the bottom of the core. Rock Mass Quality = Poor R2:Core Times (min:sec) 30.7-31.7 ft (2:40) 31.7-32.7 ft (2:14) 32.7-33.7 ft (2:11) 33.7-34.7 ft (2:40) 34.7-35.7 ft (2:30) 100% Recovery 35.70	
35								285.80		**Bottom of Exploration at 35.70 feet below ground surface.**	
40											
45											
50											

Remarks:

- 400 pounds of down pressure on Core Barrel
- bgs = below ground surface

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.

10|9

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Stimson Bridge #2807 carries Route 5 over Little Ossipee River
Location: Waterboro-Limerick, Maine

Boring No.: BB-WLLOR-103
PIN: 20476.00

Driller:	MaineDOT	**Elevation (ft.)**	321.0	**Auger ID/OD:**	5-inch-diameter Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30 inches
Date Start/Finish:	4/15/2015; 08:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ - 2 inch
Boring Location:	Sta 151+86.6, 6.6 feet left	**Casing ID/OD:**	NW (3 inches/3.5 inches)	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	320.67		4-inch-thick layer of pavement 0.33	
	1D	24/17	1.00 - 3.00	8/8/7/7	15	23				Brown, damp, medium dense, SAND, trace gravel, trace silt, poorly-graded, (Fill).	G#263923 A-1-b, SP-SM WC=3.3%
										LAYER 1	
5										Similar to above.	
	2D	24/14	5.00 - 7.00	8/4/4/5	8	12	23			assumed based on GWT @ 101	
							19				
							17				
							13			LAYER 2	
							14				
10	3D	24/13	10.00 - 12.00	2/2/8/13	10	15	3	310.50		10.50 Brown, wet, medium dense, Silty SAND, some gravel, little silt, (River Alluvium and/or Glaciolacustrine Delta Deposits).	G#263924 A-1-b, SM WC=22.8%
							29				
							39			LAYER 3	
							34				
							69				
15	4D	24/18	15.00 - 17.00	37/13/13/18	26	39	40			Brown, wet, dense, Silty SAND, some gravel, little silt.	G#263925 A-1-b, SM WC=8.5%
							189			RC to 19.0 feet bgs.	
							56			LAYER 4	
							127				
	5D	2.4/2.4	19.00 - 19.20	50(2.4")	---		NQ-2	301.80		Similar to above, except very dense Weathered bedrock observed in tip of sampler 19.20	
20	R1	60/60	19.20 - 24.20	RQD = 52%						Top of Bedrock at Elev. 301.8 feet. R1: Bedrock: Red-brown to grey with white banding (composed of primarily quartz and feldspar which are not foliated), foliated quartz, muscovite mica, feldspar, amphibole, and garnet SCHIST, medium-grained, moderately hard, moderately weathered, the foliation created by the mica plates, dips from 0-10 degrees and is locally contorted by folds, Lower Member of the Rindgemere Formation Rock Mass Quality = Fair R1:Core Times (min:sec) 19.2-20.2 ft (2:32)	
	R2	60/55	24.20 - 29.20	RQD = 40%				296.80			
25											

Remarks:

- 400 pounds of down pressure on Core Barrel
- bgs = below ground surface

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.

11/19

ANALYSIS OF SINGLE PILES UNDER LATERAL LOADING

by

Barry J. Meyer
Lymon C. Reese

Research Report 244-1

Development of Procedures for the Design of Drilled Foundations
For Support of Overhead Signs

Research Study 3-5-78-244

conducted for

Texas

State Department of Highways and Public Transportation

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH

THE UNIVERSITY OF TEXAS AT AUSTIN

December 1979

12/19

CHAPTER 5. ANALYSIS OF LATERAL LOAD TESTS FOR PILES IN SAND

INTRODUCTION

The criteria suggested by Reese, et al., (1974) for analyzing the behavior of single vertical piles embedded in sand appear to be the best criteria available at the present time. The piles may be subjected to either static or cyclic loading. To determine how accurately this method can predict the behavior of laterally loaded piles, it is necessary to compare analytical results obtained by using these criteria with the measured results from load tests.

In most of the tests that were analyzed, all the necessary soils information had to be inferred from the Standard Penetration Test, SPT, and a certain range in the results of the analyses was possible. This range in results is due to the different assumptions regarding the correlation of the results of the SPT with the relevant soil properties. In performing the analysis, the most reasonable assumptions were made in selecting soil properties. All of the available information was carefully analyzed, and the best estimate of the in-situ soil properties was made. There is no implication that the soil properties selected are the "exact" soil properties, but they are the best estimate in view of the limited information that was presented in each case.

METHOD OF OBTAINING SOIL PROPERTIES

As previously stated, when the important soil properties such as ϕ, γ, k, and K_o were not reported, they can either be obtained from correlations with some in-situ testing method or they can be assumed. The approach used in this report was to select a particular method for relating the blow count, from an SPT, to the relative density, D_r, and to then relate D_r to ϕ and k. The angle of internal friction could then be related to the void ratio for a particular soil, and then γ could be calculated. The value of K_o was reported in none of the experiments and there is no method by which an exact value of K_o can be determined from the

13/19

SPT for an overconsolidated sand deposit.

There are many methods available for correlating the SPT blow count to D_r (Gibbs and Holtz, 1957; Bazarra, 1967; Peck et al., 1974). The method proposed by Bazarra seemed to be the best method because it took the overburden pressure into consideration and because the method was developed from the results of actual field tests. The two equations which Bazarra proposed to obtain the relative density are

$$D_r = \left(\frac{N}{20\ (1 + 2\bar{p})}\right)^{0.5} \tag{5.1}$$

for $\bar{p} < 1.5\ \text{kip/ft}^2$

and

$$D_r = \left(\frac{N}{20\ (3.25 + 0.5\bar{p})}\right)^{0.5} \tag{5.2}$$

where

N = blow count (blows/ft),

$\bar{p}$ = effective overburden pressure (kip/ft^2).

The angle of internal friction can now be determined from correlations with D_r. The angle of internal friction is not only a function of D_r, but also a function of particle size and shape, gradation, and confining pressure. In most cases, the particle size and gradation were reported. Because the pressures around the top portion of a laterally loaded pile are not large, the effect of confining pressure was not considered.

The correlation that was used to relate D_r to ϕ was given by Schmertman (1975). His curves, shown in Fig. 5.1, show ϕ as a function of D_r and some of the previously mentioned parameters (e.g., grain size). The

$\frac{lb}{ft^2} \times \frac{1\ kip}{1000}$

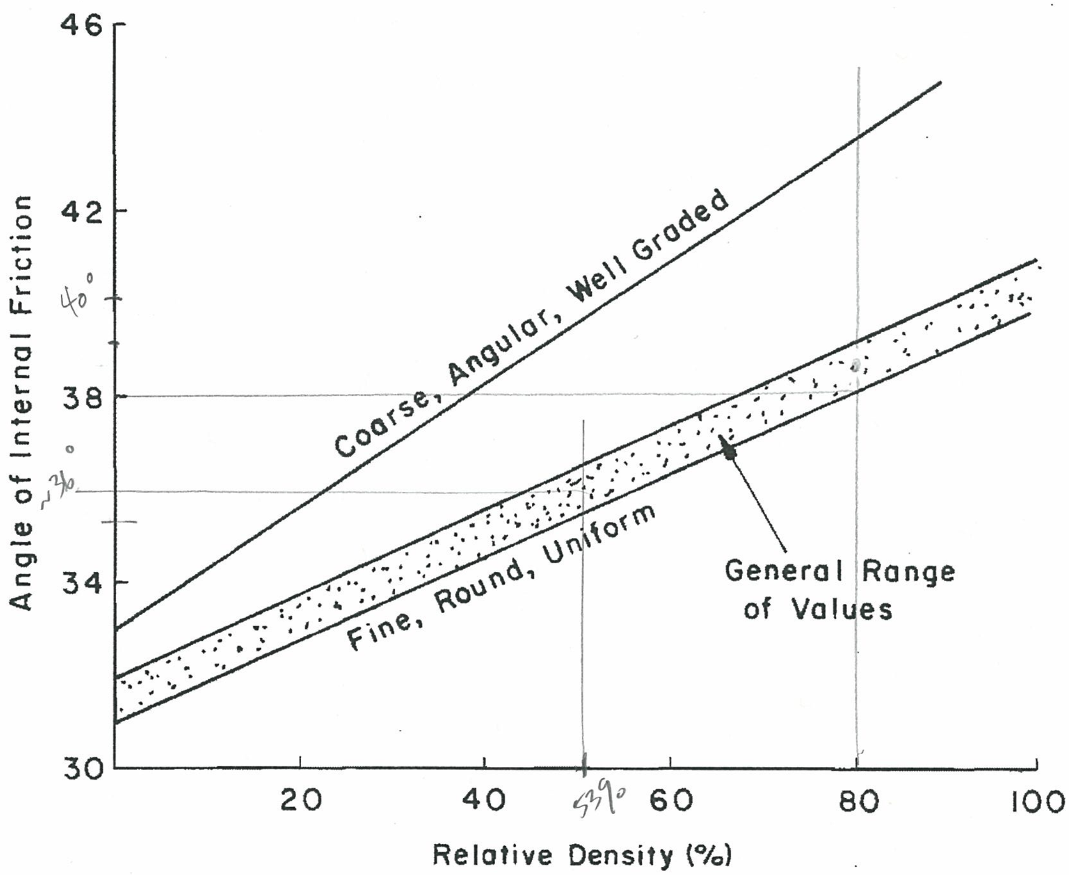


Fig. 5.1. Correlation between angle of internal friction and relative density (Schmertmann, 1975).

Note: yields high ϕ values...

15/19

upper curve is for angular, well-graded materials and the lower curve is for the rounded, poorly-graded materials. Most of the sands that were described in connection with the load tests that were studied were classified as SP in the Unified Soil Classification System, which would place them closer to the lower curve in Fig. 5.1.

Touma (1972) recommended the determination of D_r and ϕ from the work of Gibbs and Holtz (1957) and he further obtained a correlation between the N-value from the Standard Penetration Test and the N-value from the penetrometer test of the State Department of Highways and Public Transportation, State of Texas. Figure 5.2 presents Touma's recommendations and allows correlations to be developed if one has N-values from the SDHPT test (PEN test).

The constant of subgrade reaction is necessary to establish the initial portion of the p-y curve. Values of k, as a function of the general classifications of loose, medium, and dense, have been reported by Reese (1975) and are shown in Tables 5.1 and 5.2, for sands below the water table and for sands above the water table, respectively. The values of k were plotted as a function of D_r, instead of tabulating them as Reese (1975) suggested. The two curves of k versus D_r are plotted in Fig. 5.3.

Depending on the gradation of the sand and the particle size, a rough correlation between ϕ and the void ratio can be made, as shown in Fig. 5.4. The submerged unit weight was calculated using a degree of saturation, S_r, of 100%, and the total unit weight was calculated using an S_r of 50%.

Reese, et al., (1974) used a value of 0.4 for K_o in their analyses. In analyzing tests which were similar to the tests from which the criteria were developed, a value of 0.4 for K_o is reasonable. Larger values of K_o could be used where sands are overconsolidated. In this report, the soil-pile behavior was determined as accurately as possible at the time of testing and every effort was made to take all extraneous factors into consideration. In instances where a large amount of soil was excavated, a value of 1.0 was used for K_o.

In the following sections, the results of analyses of the results of a number of lateral load tests will be discussed. In cases where the important soil properties were reported, these values were simply used directly in the

16/19

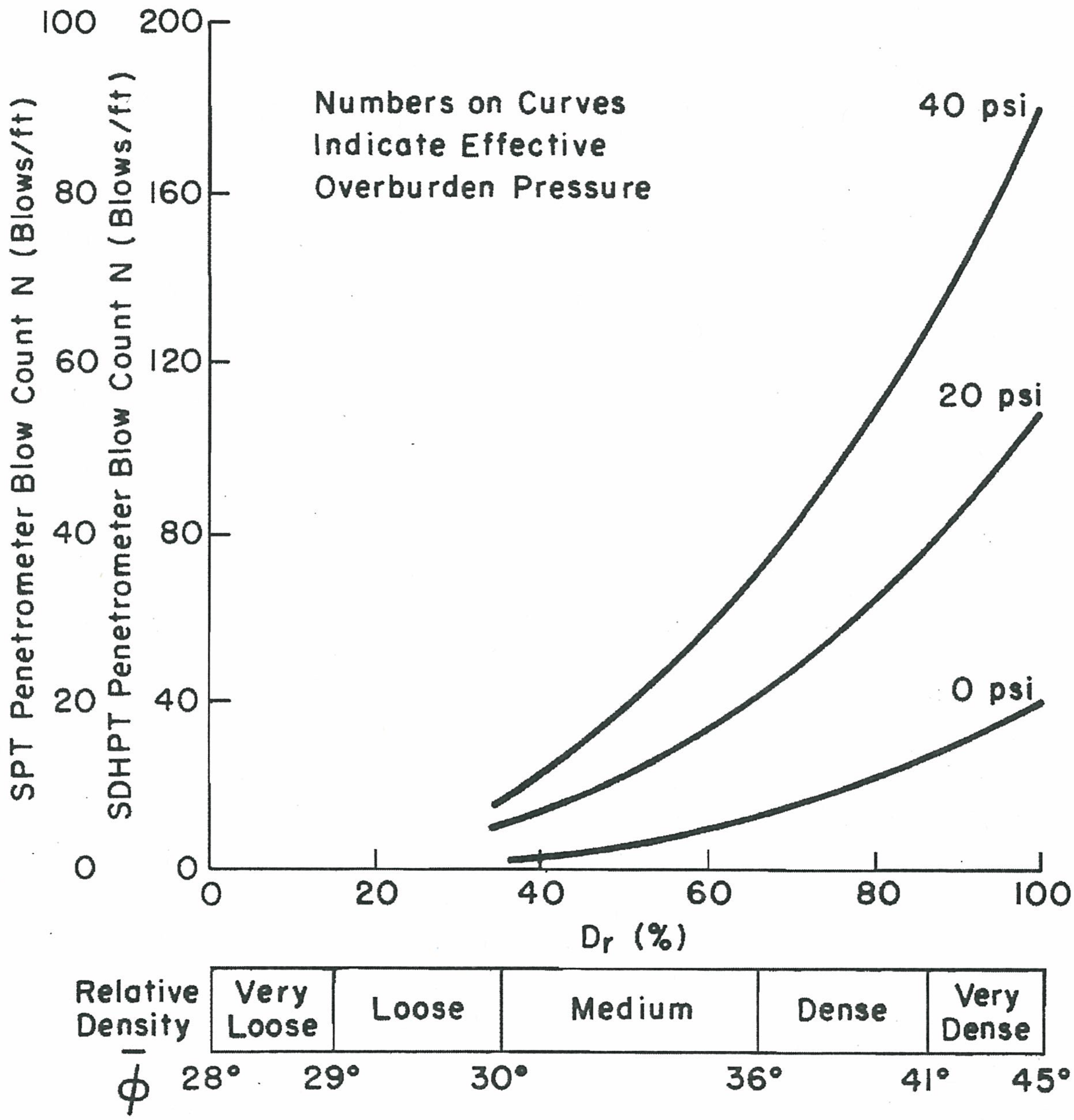


Fig. 5.2. Relation between SDHPT and SPT penetrometer blow count and the friction angle.

17/19

TABLE 5.1. RECOMMENDED VALUES OF k FOR SANDS BELOW THE WATER TABLE

Relative Density	Loose	Medium	Dense
Recommended k ($lb/in.^3$)	20	60	125

TABLE 5.2. RECOMMENDED VALUES OF k FOR SANDS ABOVE THE WATER TABLE

Relative Density	Loose	Medium	Dense
Recommended k ($lb/in.^3$)	25	90	225

NAVFAC 7.01

18/19

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids(1)					Unit Weight(2) (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, Inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146(3)	90	155(3)	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148(4)	125	156(4)	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62

19/19

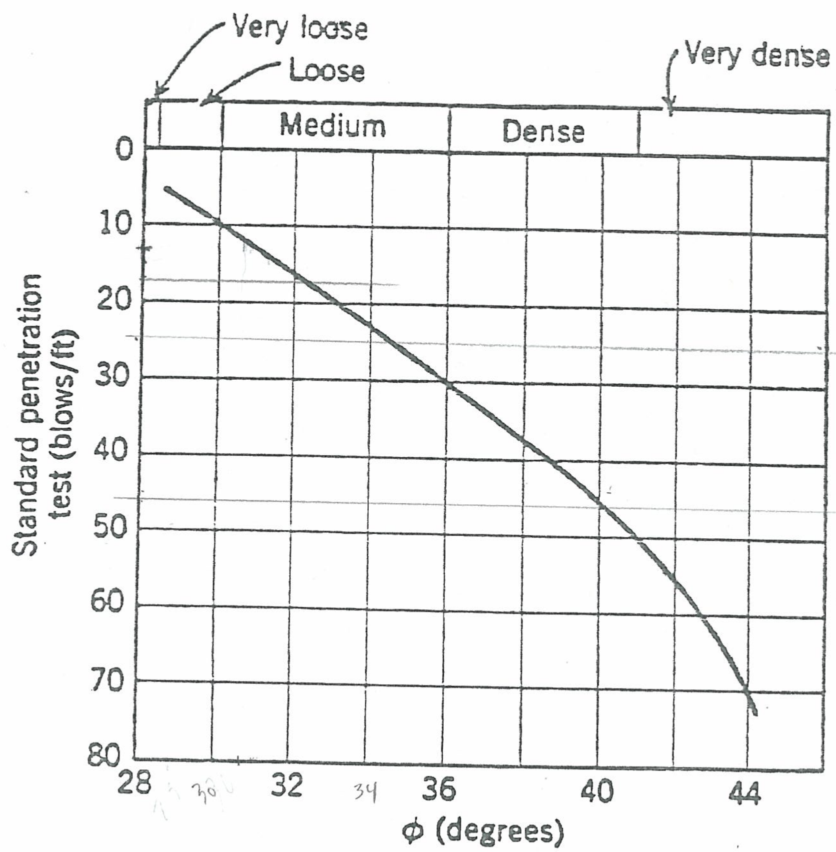


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

JAPAN NATIONAL RAILWAYS

$$\phi = 0.3N + 27 \quad \text{deg.}$$

PECK

$$C_N = 0.77 \log \frac{20}{\bar{p}_o} \quad ; \quad \bar{p}_o \geq 0.25 \text{ tsf}$$

$\bar{p}_o$ = effective overburden pressure (tsf)

OBJECTIVE

Provide LPile parameters for the proposed integral abutments assuming the H-piles installed will be installed in bedrock sockets.

GIVEN

1) Boring logs and lab data

ASSUMPTIONS

1) Assume the design "median" value of the uniaxial compressive strength of the rock is 6,943 psi (unconfined compressive strength tests on schist from Waterville Formation (5,120-8,766 psi)). These values are from the Western Avenue project in Waterville (WIN 18234.00)
2) Average the RQDs across the site since they are relatively comparable (see below).
3) LPile parameters are based on geotechnical engineering correlations and suggested values per the LPile 5.0 technical manual and noted references.

ROCK TYPE AND RQDs AT ABUTMENTS

Abutment No. 1 (Boring BB-WLLOR-101)
R1 RQD = Schist, 22%
R2 RQD = Pegmatite/Migmatite, 48%

Average RQD = 41%

Abutment No. 2 (Boring BB-WLLOR-103)
R1 RQD = Schist, 52%
R2 RQD = Schist, 40%

ABUTMENTS NO. 1 AND 2 - WEAK ROCK (REESE, 1997) MODEL

The LPile parameters for the proposed abutments are based on the subsurface information encountered at Borings BB-WLLOR-101 and -103.

1) Determine the effective unit weight of the bedrock

Per R.E. Goodman (Introduction to Rock Mechanics, 1980, pg. 33, Table 2.3), the typical dry unit weight of quartz, mica, and **schist** is 176 pcf (equivalent to 0.101 pci).

$\gamma_{dry} := 0.101 \ \textit{pci}$

Per Vallejo (Geological Engineering, 2011, pg. 119, Table 3.2), the typical porosity of schist is 3%, where n = porosity (decimal).

$n := 0.03$

Per Goodman (pg. 31, Table 2.2), the specific gravity, G, of quartz (similar to schist) can be taken as 2.65

$G := 2.65$

Per Das (7th Edition, pg. 65, Equation 3.33), the void ratio can be calculated as:

$$e := \frac{n}{1-n}$$

$e=0.031$

Per Das (7th Edition, pg. 55, Equation 3.18), the water content, wc, (assuming 100% saturation, S, since the bedrock is beneath the groundwater table) can be calculated as:

$S:=1.0$

$$wc:=\frac{(S \cdot e)}{G}$$

$wc=0.012$ (12% water content)

Therefore, per Das (Equation 3.12), the saturated unit weight can be calculated as:

$$\gamma_{saturated}:=\gamma_{dry} \cdot (1+wc)$$

$\gamma_{saturated}=0.102$ ***pci***

The effective (buoyant) unit weight can then be calculated as:

$\gamma_{water}:=0.036$ ***pci***

$$\gamma':=\gamma_{saturated}-\gamma_{water}$$

$\gamma'=0.066$ ***pci*** **This value will be used for the strong rock model as well.**

2) Determine the Young's modulus of the bedrock

Use the Young's modulus, E_r, parameter corresponding to the UCS value selected for design (from the UCT that yielded 6,943 psi); therefore, use the average of the Young's moduli from the UCT laboratory data:

$$E_r:=\frac{(5710000+8650000)}{2} \textit{ psi}$$

$E_r=7180000$ ***psi***

Per the LPile 5.0 User Manual (pg. 43), to account for jointing in the bedrock, the Young's modulus should be reduced by a reduction factor (alpha_E).

Per LRFD 2012 Table 10.4.6.5.1b-1 (O'Neill and Reese, 1999), the reduction factor is a function of the Young's modulus of the rock mass (E_m) divided by the Young's modulus of the bedrock core (E_i), which is the value above (E_r) from the UCT test.

The Young's modulus of the bedrock mass can be calculated per LRFD 2012 Equation 10.4.6.5-1.

The adjusted Rock Mass Rating (RMR = 31) of the schist was previously calculated for estimating the bearing resistance of the center pier. Since the RMR was calculated assuming the same UCS value, the RMR value can be used in this application to determine the Young's modulus of the bedrock mass.

Therefore:

$$RMR := 31$$

$$E_m := 145 \cdot \left(10^{\frac{RMR - 10}{40}}\right) \; \textit{ksi}$$

$$E_m = 485700 \; \textit{psi}$$

Now, calculate the ratio of the moduli:

$$\frac{E_m}{E_r} = 0.068$$

The reduction factor corresponding to this ratio, per LRFD 2012 Table 10.8.3.5.4b-1 is approximately 0.48.

Therefore, the Young's modulus (reduced for jointing in bedrock) is:

$$\alpha_E := 0.48$$

$$E_{design} := E_r \cdot \alpha_E$$

$$E_{design} = 3446400 \; \textit{psi}$$

3) Determine the uniaxial compressive strength of the bedrock

Since the RQDs of the core runs at the proposed abutments are relatively similar and given the expected nature of varying bedrock conditions and properties, assume that the UCS is the same at both abutments. Therefore, use a UCS of 6,943 psi. **This value will be used in the strong rock model.**

4) Determine the RQD of the bedrock

As shown on Page 1, the average RQD of the borings located at the proposed abutments is 41%. Use this value for all RQD values for LPile analyses.

5) Determine the bedrock strain parameter (k_rm)

Per the National Cooperative Highway Research Program (NCHRP) Synthesis 360, 2006, "Rock-Socketed Shafts for Highway Structure Foundations (pg. 58), "Typically, the k_rm value is taken as the strain at 50% of the maximum strength of the core sample. Because limited experimental data are available for weak rock during the derivation of the p-y criteria, the k_rm from a particular site may not be in the range between 0.0005 and 0.00005. For such cases, you may use the upper bound value (0.0005) to get a larger value of y_rm which in turn will provide a more consdervative result."

Past MaineDOT projects in Maine has suggested using a k_rm value of 0.0005 (i.e. Sarah Mildred Long Bridge LPile analyses). Therefore, use a k_rm value of 0.0005 for this project.

ABUTMENTS NO. 1 AND 2 - STRONG ROCK (VUGGY LIMESTONE) MODEL

The required geotechnical LPile parameters the strong rock model consist of the effective unit weight and the UCS. HTA should use the effective unit weights and UCS value from the weak rock model.

Note: HTA will determine the layer thickness of the bedrock based on the desired depth of bedrock socket from the LPile analyses.

The dry density is related to the wet density by the relationship

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{wet}}}{1 + w} \tag{2.4}$$

where w is the water content of the rock (dry weight basis).

Water content and porosity are related by

$$n = \frac{w \cdot G}{1 + w \cdot G} \tag{2.5}$$

If the pores of the rock are filled with mercury, and the mercury content is determined to be w_{Hg} (as a proportion of the dry weight of the rock before mercury injection), the porosity can be calculated more accurately as follows:

$$n = \frac{w_{\text{Hg}} \cdot G/G_{\text{Hg}}}{1 + (w_{\text{Hg}} \cdot G/G_{\text{Hg}})} \tag{2.6}$$

The specific gravity of mercury (G_{Hg}) equals 13.546.

The densities of some common rocks are given in Table 2.3. These figures are only sample values, of course, since special factors can cause wide variations in individual formations.

Rocks exhibit a far greater range in density values than do soils. Knowledge of rock density can be important to engineering and mining practice. For example, the density of a rock governs the stresses it will experience when acting as a beam spanning an underground opening; unusually high density in a roof rock implies a shortened limiting safe span. A concrete aggregate with higher than average density can mean a smaller volume of concrete required for a gravity retaining wall or dam. Lighter than average aggregate can mean lower stresses in a concrete roof structure. In oil shale deposits, the density indicates the value of the mineral commodity because the oil yield correlates directly with the unit weight; this is true because oil shale is a mixture of a relatively light constituent (kerogen) and a relatively heavy constituent (dolomite). In coal deposits, the density correlates with the ash content and with the previous depth of cover, accordingly with the strength and elasticity of the rock. It is easy to measure the density of a rock; simply saw off the ends of a dried drill core, calculate its volume from the dimensions, and weight it. In view of the possible significance of variations from the norm, density should therefore be measured routinely in rock investigations.

2.5 Hydraulic Permeability and Conductivity

Measurement of the permeability of a rock sample may have direct bearing on a practical problem, for example, pumping water, oil, or gas into or out of a

Table 2.3 *Dry Densities of Some Typical Rocks*[a]

Rock	Dry (g/cm³)	Dry (kN/m³)	Dry (lb/ft³)
Nepheline syenite	2.7	26.5	169
Syenite	2.6	25.5	162
Granite	2.65	26.0	165
Diorite	2.85	27.9	178
Gabbro	3.0	29.4	187
Gypsum	2.3	22.5	144
Rock salt	2.1	20.6	131
Coal	0.7–2.0		
(density varies with the ash content)			
Oil shale	1.6–2.7		
(density varies with the kerogen content, and therefore with the oil yield in gallons per ton)			
30 gal/ton rock	2.13	21.0	133
Dense limestone	2.7	20.9	168
Marble	2.75	27.0	172
Shale, Oklahoma[b]			
1000 ft depth	2.25	22.1	140
3000 ft depth	2.52	24.7	157
5000 ft depth	2.62	25.7	163
Quartz, mica schist	2.82	27.6	176
Amphibolite	2.99	29.3	187
Rhyolite	2.37	23.2	148
Basalt	2.77	27.1	173

[a] Data from Clark (1966), Davis and De Weist (1966), and other sources.
[b] This is the Pennsylvanian age shale listed in Table 2.1.

porous formation, disposing of brine wastes in porous formations, storing fluids in mined caverns for energy conversion, assessing the water tightness of a reservoir, dewatering a deep chamber, or predicting water inflows into a tunnel. In many instances the system of discontinuities will radically modify the permeability values of the rock in the field as compared to that in the lab, so that some sort of in situ pumping test will be required for an acceptable forecast of formation permeabilities. Our motivation for selecting permeability as an index property of rock is that it conveys information about the degree of interconnection between the pores or fissures—a basic part of the rock framework. Furthermore, the variation of permeability with change in normal stress, especially as the sense of the stress is varied from compression to tension, evaluates the degree of fissuring of the rock, since flat cracks are greatly affected by normal stress whereas spherical pores are not. Also, the degree to which the permeability changes by changing the permeant from air to water expresses progresses, the porosity tends to increase to 20% or more. As a result, measurement of porosity can serve as an accurate index to rock quality in such rocks. In several projects in granitic rocks the National Civil Engineering Laboratory of Portugal was able to classify the rock for the purposes of engineering design mainly on the basis of a quick porosity measurement, obtained from the water content of the rock after immersion for 24 hours at a standard temperature and pressure (Hamrol, 1961). Among unweathered rocks, there is also a general correlation between porosity and mechanical properties such as unconfined compressive strength and modulus of elasticity; but such relationships are usually marked by enormous scatter. In the case of weak sandstones (having saturated compressive strength less than 20 MPa) Dobereiner and de Freitas (1986) have demonstrated good correlations of density, modulus of elasticity, and compressive strength with the saturated moisture content. The moisture content of a saturated specimen is linked with its porosity by Equation 2.5. Saturation can be approached by soaking a specimen in water while it is subjected to a laboratory vacuum.

Porosity can be measured in rock specimens by a variety of techniques. Since it is the pore space that governs the quantity of oil contained in a saturated petroleum reservoir, accurate methods for porosity determination in sandstones have been developed by the oil industry. However, these methods are not always suitable for measurements in hard rocks with porosities of less than several percent. Porosities can be determined from the following calculations.

1. Measured density.
2. Measured water content after saturation in water.
3. Mercury content after saturation with mercury using a pressure injector.
4. Measured solid volume and pore air volume using Boyle's law.

These are considered further below.

2.4 Density

The density or "unit weight" of a rock, γ, is its specific weight (FL^{-3}),[2] for example, pounds per cubic foot or kilonewtons per cubic meter. The *specific gravity* of a solid, G, is the ratio between its density and the unit weight of water γ_w; the latter is approximately equal to 1 g-force/cm^3 (9.8 kN/m^3 or approximately 0.01 MN/m^3).[3] Rock with a specific gravity of 2.6 has a density

[2] The terms in parenthesis indicate the dimensions of the preceding quantity. F, L, T indicate force, length, and time, respectively.

[3] At 20°C, the unit weight of water is 0.998 g/cm^3 × 980 cm/s^2 = 978 dynes/cm^3 or = 0.998 g-force/cm^3.

of approximately 26 kN/m^3. In the English system, the density of water is 62.4 pounds per cubic foot. (Mass density ρ equals γ/g.)

It was stated previously that the porosity of a rock can be calculated from knowledge of its weight density. This assumes that the specific gravity of the grains or crystals is known; grain specific gravity can be determined by grinding the rock and adapting methods used in soils laboratories. If the percentages of different minerals can be estimated under a binocular microscope, or from a thin section, the specific gravity of the solid part of a rock can then be calculated as the weighted average of the specific gravities of the component grains and crystals:

$$G = \sum_{i=1}^{n} G_i V_i \tag{2.2}$$

where G_i is the specific gravity of component i, and V_i is its volume percentage in the solid part of the rock. The specific gravities of a number of common rock-forming minerals are listed in Table 2.2. The relation between porosity and dry density γ_{dry} is

$$\gamma_{dry} = G\gamma_w(1 - n) \tag{2.3}$$

Table 2.2 Specific Gravities of Common Minerals[a]

Mineral	G
Halite	2.1–2.6
Gypsum	2.3–2.4
Serpentine	2.3–2.6
Orthoclase	2.5–2.6
Chalcedony	2.6–2.64
Quartz	2.65
Plagioclase	2.6–2.8
Chlorite and illite	2.6–3.0
Calcite	2.7
Muscovite	2.7–3.0
Biotite	2.8–3.1
Dolomite	2.8–3.1
Anhydrite	2.9–3.0
Pyroxene	3.2–3.6
Olivine	3.2–3.6
Barite	4.3–4.6
Magnetite	4.4–5.2
Pyrite	4.9–5.2
Galena	7.4–7.6

[a] A. N. Winchell (1942).

onse of the rock to changes in external forces. Many ant rock properties for engineering depend on the structure of the mineral particles and how they are linked.

The physical or **index properties** of rocks are determined in the laboratory. Porosity, unit weight, permeability, ability, strength and the elastic wave propagation city have the most significant influence on an understanding of the mechanical behaviour to be expected. Some these properties are directly related to the strength and rmational characteristics of the rocks and are used to ify them.

Porosity is the ratio between the rock pore volume, and the total volume V (solid particles + pore spaces or s): $n(\%) = V_v/V$. This is the property that most affects ngth and mechanical characteristics, as it is inversely proportional to strength and density, and directly proportional to rmability. In crystalline, igneous or metamorphic rocks, s may be microcracks or cracks in the intact rock. In eral, porosity decreases with depth and the age of the s.

The value of n varies between 0% and 90%, with nal values ranging from 15%–30%. Carbonate bioclas- edimentary rocks and volcanic rocks may have very high osity values, the same as altered or weathered rocks. le *3.2* shows porosity data for some rocks.

Effective porosity is the ratio between the interconnected pore void volume and the total volume V of the nple rock; it can be obtained from the dry and saturated ights of the sample:

$$n_e = (W_{sat} - W_{dry}) / (\gamma_w V)$$

iere γ_w = unit weight of water.

Pores in rocks are often not interconnected, so that l porosity is greater than effective porosity. The void ratio defined as the ratio of the volume of void space, V_v, to the lume of solid particles, V_{sol}: $e = V_v/V_{sol}$.

The **unit weight** of the rock depends on its components and is defined as the weight per unit volume. The units ed are those of force per unit volume. Care should be taken cause in geotechnical literature "density" ρ (ρ = mass/ lume) is sometimes referred to as specific or unit weight; hen working with weight ($\gamma = \rho$ g) it should be made clear at units of force (i.e. mass × acceleration), not mass, are ing used; i.e. 1 $kg_{mass}/m^3 \times 1\ m/s^2 = 1\ N/m^3$, making $kg_{mass}/m^3 \times 9.81\ m/s^2 = 9.81\ N/m^3$ the unit weight of kg_{mass}/m^3 on earth. Water has a density of 1,000 kg_{mass}/m^3 giving 1 m^3 a weight of 9.81 kN.

Unlike soils, the specific weight values of rocks vary idely. *Table 3.2* gives average values for some rocks.

Permeability is the water-transmitting capacity of a ck. Most rocks have low or very low permeability. Water infiltrates and flows through intact rock through pores and cracks, and the permeability is determined by how these are interconnected and other factors, such as the degree of weathering, anisotropy and the state of stress the material is subjected to.

The permeability of a rock is measured by the coefficient of permeability or hydraulic conductivity, k, expressed in m/s, cm/s or m/day.

Darcy's law states that the rate of flow Q per unit area is proportional to the gradient of the potential head, i, measured in the direction of flow:

$$Q = kiA$$

For most types of rock, the flow in intact rock can be considered to follow Darcy's law:

$$q_x = k(dh/dx)A$$

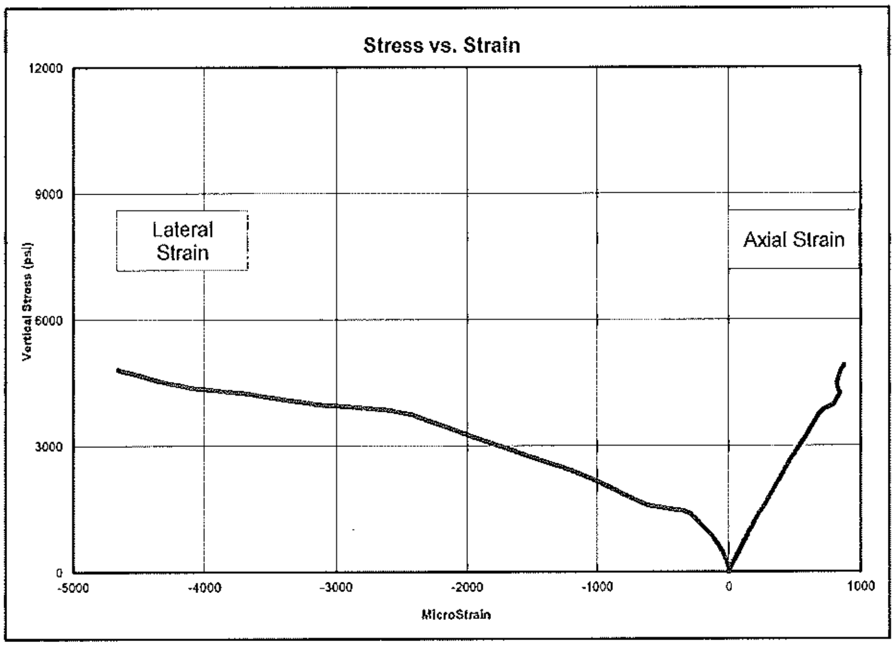
Table 3.2 **TYPICAL VALUES FOR UNIT WEIGHT AND POROSITY OF ROCKS**

Rock	Unit weight (kN/m³)	Porosity (%)
Andesite	22–23.5	10–15
Amphibolite	29–30	–
Basalt	27–29	0.1–2
Chalk	17–23	30
Coal	10–20	10
Diabase	29	0.1
Diorite	27–28.5	–
Dolomite	25–26	0.5–10
Gabbro	30–31	0.1–0.2
Gneiss	27–30	0.5–1-5
Granite	26–27	0.5–1.5 (0.9)
Greywacke	28	3
Gypsum	23	5
Limestone	23–26	5–20 (11)
Marble	26–28	0.3–2 (0.6)
Mudstone	22–26	2–15
Quartzite	26–27	0.1–0.5
Rhyolite	24–26	4–6
Salt	21–22	5
Sandstone	23–26	5–25 (16)
Schist	25–28	3
Shale	25–27	0.1–1
Tuff	19–23	14–40

GeoTesting
EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	Western Avenue over Messalonskee Stream
Project Location:	Waterville, ME
GTX #:	12141
Test Date:	8/26/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-WMS-101
Sample ID:	R2
Depth, ft:	19.12-19.47
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 5,120 psi

The graph above does not include all data up to the peak stress value. The strain gauges failed before the peak stress value was recorded.

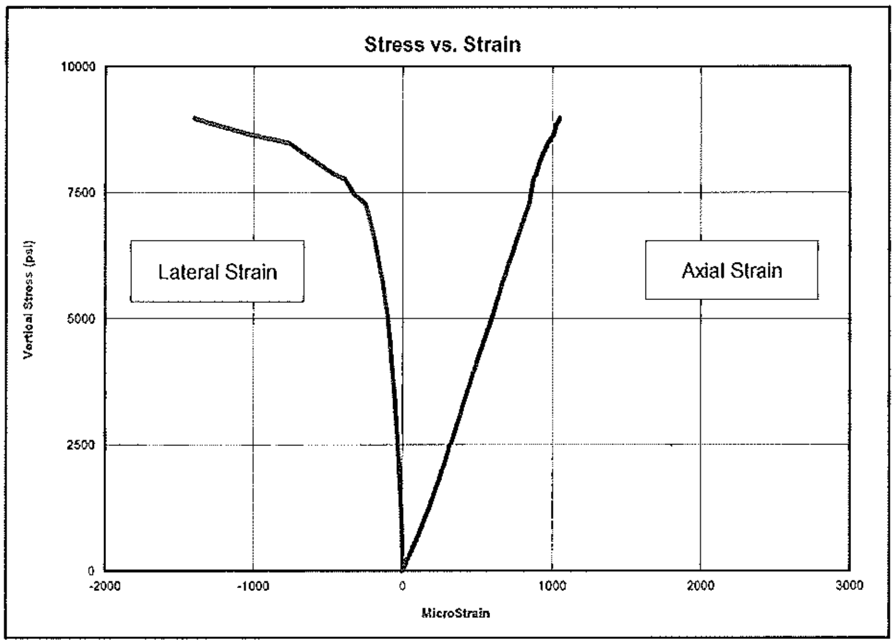
Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
500-1900	5,710,000	---
1900-3200	5,270,000	---
3200-4600	3,980,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	Western Avenue over Messalonskee Stream
Project Location:	Waterville, ME
GTX #:	12141
Test Date:	8/26/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-WMS-101
Sample ID:	R2
Depth, ft:	21.15-21.51
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 8,977 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
900-3300	8,650,000	0.17
3300-5700	9,130,000	0.32
5700-8100	9,870,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Table 3.3 Qualitative Description of Granular Soil Deposits

Relative density (%)	Description of soil deposit
0–15	Very loose
15–50	Loose
50–70	Medium
70–85	Dense
85–100	Very dense

have relative densities less than 20 to 30%. Compacting a granular soil to a relative density greater than about 85% is difficult.

The relationships for relative density can also be defined in terms of porosity, or

$$e_{max} = \frac{n_{max}}{1 - n_{max}} \tag{3.31}$$

$$e_{min} = \frac{n_{min}}{1 - n_{min}} \tag{3.32}$$

$$\boxed{e = \frac{n}{1 - n} \qquad (3.33)}$$

where n_{max} and n_{min} = porosity of the soil in the loosest and densest conditions, respectively. Substituting Eqs. (3.31), (3.32), and (3.33) into Eq. (3.30), we obtain

$$D_r = \frac{(1 - n_{min})(n_{max} - n)}{(n_{max} - n_{min})(1 - n)} \tag{3.34}$$

By using the definition of dry unit weight given in Eq. (3.16), we can express relative density in terms of maximum and minimum possible dry unit weights. Thus,

$$D_r = \frac{\left[\dfrac{1}{\gamma_{d(min)}}\right] - \left[\dfrac{1}{\gamma_d}\right]}{\left[\dfrac{1}{\gamma_{d(min)}}\right] - \left[\dfrac{1}{\gamma_{d(max)}}\right]} = \left[\frac{\gamma_d - \gamma_{d(min)}}{\gamma_{d(max)} - \gamma_{d(min)}}\right]\left[\frac{\gamma_{d(max)}}{\gamma_d}\right] \tag{3.35}$$

where $\gamma_{d(min)}$ = dry unit weight in the loosest condition (at a void ratio of e_{max})
γ_d = *in situ* dry unit weight (at a void ratio of e)
$\gamma_{d(max)}$ = dry unit weight in the densest condition (at a void ratio of e_{min})

In terms of density, Eq. (3.35) can be expressed as

$$D_r = \left[\frac{\rho_d - \rho_{d(min)}}{\rho_{d(max)} - \rho_{d(min)}}\right]\frac{\rho_{d(max)}}{\rho_d} \tag{3.36}$$

Now, using the definitions of unit weight and dry unit weight [Eqs. (3.9) and (3.11)], we can write

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + wG_s\gamma_w}{1 + e} = \frac{(1 + w)G_s\gamma_w}{1 + e} \tag{3.15}$$

and

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w}{1 + e} \tag{3.16}$$

or

$$e = \frac{G_s\gamma_w}{\gamma_d} - 1 \tag{3.17}$$

Because the weight of water for the soil element under consideration is $wG_s\gamma_w$, the volume occupied by water is

$$V_w = \frac{W_w}{\gamma_w} = \frac{wG_s\gamma_w}{\gamma_w} = wG_s$$

Hence, from the definition of degree of saturation [Eq. (3.5)],

$$S = \frac{V_w}{V_v} = \frac{wG_s}{e}$$

or

$$Se = wG_s \tag{3.18}$$

This equation is useful for solving problems involving three-phase relationships.

If the soil sample is *saturated*—that is, the void spaces are completely filled with water (Figure 3.3)—the relationship for saturated unit weight (γ_{sat}) can be derived in a similar manner:

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + e\gamma_w}{1 + e} = \frac{(G_s + e)\gamma_w}{1 + e} \tag{3.19}$$

Also, from Eq. (3.18) with $S = 1$,

$$e = wG_s \tag{3.20}$$

As mentioned before, due to the convenience of working with densities in the SI system, the following equations, similar to unit–weight relationships given in Eqs. (3.15), (3.16), and (3.19), will be useful:

$$w = \frac{W_w}{W_s} \tag{3.8}$$

Unit weight (γ) is the weight of soil per unit volume. Thus,

$$\gamma = \frac{W}{V} \tag{3.9}$$

The unit weight can also be expressed in terms of the weight of soil solids, the moisture content, and the total volume. From Eqs. (3.2), (3.8), and (3.9),

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{W_s\left[1 + \left(\frac{W_w}{W_s}\right)\right]}{V} = \frac{W_s(1 + w)}{V} \tag{3.10}$$

Soils engineers sometimes refer to the unit weight defined by Eq. (3.9) as the *moist unit weight.*

Often, to solve earthwork problems, one must know the weight per unit volume of soil, excluding water. This weight is referred to as the *dry unit weight,* γ_d. Thus,

$$\gamma_d = \frac{W_s}{V} \tag{3.11}$$

From Eqs. (3.10) and (3.11), the relationship of unit weight, dry unit weight, and moisture content can be given as

$$\boxed{\gamma_d = \frac{\gamma}{1 + w} \qquad (3.12)}$$

Unit weight is expressed in English units (a gravitational system of measurement) as pounds per cubic foot (lb/ft^3). In SI (Système International), the unit used is kilo Newtons per cubic meter (kN/m^3). Because the Newton is a derived unit, working with mass densities (ρ) of soil may sometimes be convenient. The SI unit of mass density is kilograms per cubic meter (kg/m^3). We can write the density equations [similar to Eqs. (3.9) and (3.11)] as

$$\rho = \frac{M}{V} \tag{3.13}$$

and

$$\rho_d = \frac{M_s}{V} \tag{3.14}$$

where ρ = density of soil (kg/m^3)
ρ_d = dry density of soil (kg/m^3)
M = total mass of the soil sample (kg)
M_s = mass of soil solids in the sample (kg)

The unit of total volume, V, is m^3.

- *Strain Factor E50:* Values of ε_{50} strain at 50% of the maximum stress. The strain factor ε_{50} for clays and/or silts at each soil depth are entered in dimensionless units of strain.

If soil test data are available, the user may enter the value based on the stress-strain curves measured in the soil laboratory. The *p-y* curves for weak rocks need a strain parameter k_{rm} which is equivalent to ε_{50}. More information regarding k_{rm} and ε_{50} can be found in the *Technical Manual.*

Initial Mass Modulus for Weak Rock: The initial mass modulus for weak rock should be entered for this value. This value may be measured in the field using an appropriate test or may be obtained from the product of the modulus reduction ratio and Young's modulus measured on intact rock specimens in the laboratory

Uniaxial Compressive Strength: This value is the uniaxial compressive strength of weak rock at the specified depth. Values at elevations between the top and bottom elevations will be determined by linear interpolation.

Any input values that are considered unreasonable are flagged in the output file and a warning dialog box is displayed. However, the analysis is performed normally.

Rock Quality Designation: The secondary structure of the weak rock is described using the Rock Quality Designation (*RQD*). Enter the value of *RQD* in percent for the weak rock.

Strain Factor k_{rm}: The parameter k_{rm} for weak rock typically ranges between 0.0005 and 0.00005. The input dialog for weak rock is shown in Figure 3-26 as an example.

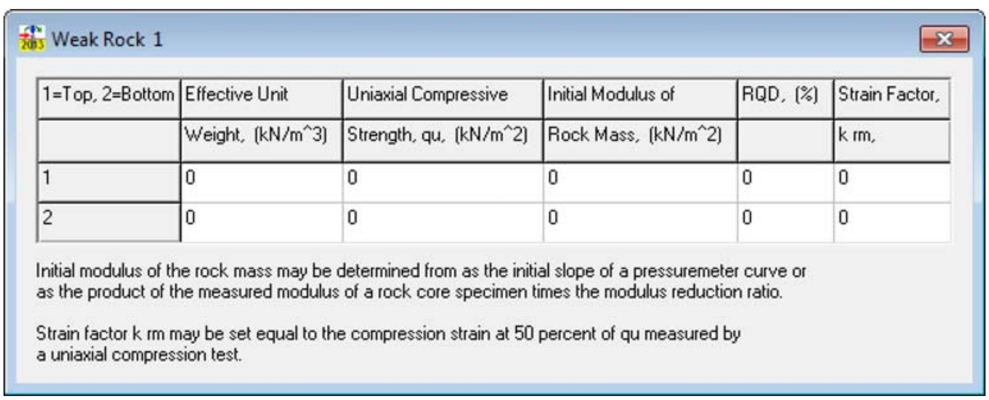


Figure 3-26 Dialog for Properties of Weak Rock

User Input p-y Curves

Data for user-input *p-y* curves are input using two linked dialog boxes. The first dialog box is used to enter values of effective unit weight at the top and bottom of the soil layer and to open the input dialog box for entry of the *p-y* curve data.

used for *p-y* curve development were: $k_{rm} = 0.00005$; q_u = 1.86 MPa for depth of 0–3.9 m, 6.45 MPa for depth of 3.9–8.8 m, and 16.0 MPa for depth of more than 8.8 m; $E_{ir} = 10q_u$ (MPa) for each layer, B = 2.25 m, and $EI = 35.15 \times 10^3$ MN-m^2. The value of k_{rm} was adjusted to provide agreement between displacements given by the *p*-y method of analysis and measured displacements from the load test.

Figure 39 shows a comparison of the measured load-displacement curve with results produced by the *p-y* method of analysis, for various methods of computing the flexural rigidity (*EI*) of the test shaft. Methods that account for the nonlinear relationship between bending moment and *EI* provide a better fit than *p-y* analysis with a constant vale of *EI*. The curve labeled "Analytical" in Figure 39 was obtained using an analytical procedure described by Reese to incorporate the nonlinear moment–*EI* relationships directly into the numerical solution of Eq. 97, whereas the curve labeled "ACI" incorporates recommendations by the American Concrete Institute for treating the nonlinear moment–*EI* behavior.

Fitting of *p-y* curves to the results of the two load tests as described previously forms the basis for recommendations that have been incorporated into the most widely used computer programs being used by state DOTs for analysis of laterally loaded rock-socketed foundations. The program COM624 (Wang and Reese 1991) and its commercial version, LPILE (Ensoft, Inc. 2004), allow the user to assign a limited number of soil or rock types to each subsurface layer. One of the options is "weak rock." If this geomaterial selection is made, additional required input parameters are unit weight, modulus, uniaxial compressive strength, RQD, and k_{rm}. The program then generates *p-y* curves using Eqs. 98–107. The program documentation recommends assigning "weak rock" to geomaterials with uniaxial compressive strengths in the range of 0.5–5 MPa. The user assigns a value to k_{rm}. The documentation (Ensoft, Inc. 2004) recommends to:

> . . . examine the stress–strain curve of the rock sample. Typically, the k_{rm} is taken as the strain at 50% of the maximum strength of the core sample. Because limited experimental data are available for weak rock during the derivation of the *p-y* criteria, the k_{rm} from a particular site may not be in the range between 0.0005 and 0.00005. For such cases, you may use the upper bound value (0.0005) to get a larger value of y_{rm}, which in turn will provide a more conservative result.

The criteria recommended for *p-y* curves in the LPILE^{PLUS} users manual (Ensoft Inc. 2004) for "strong rock" is illustrated in Figure 40. Strong rock is defined by a uniaxial strength of intact rock $q_u \geq 6.9$ MPa. In Figure 40, s_u is defined as one-half of q_u and *b* is the shaft diameter. The *p-y* curve is bilinear, with the break in slope occurring at a deflection *y* corresponding to 0.04% of the shaft diameter. Resistance (*p*) is a function of intact rock strength for both portions of the curve. The criterion does not account explicitly for rock mass properties, which would appear to limit its applicability to massive rock. The authors recommend verification by load testing if deflections exceed 0.04% of the shaft diameter, which would exceed service limit state criteria in most practical situations. Brittle fracture of the rock is assumed if the resistance *p* becomes greater than the shaft diameter times one-half of the uniaxial compressive strength of the rock. The deflection *y* corresponding to brittle fracture can be determined from the diagram as 0.0024 times the shaft diameter. This level of displacement would be exceeded in many practical situations. It is concluded that the recommended criteria applies only for very small lateral deflections and is not valid for jointed rock masses. Some practitioners apply the weak rock criteria, regardless of material strength, to avoid the limitations cited earlier. The authors state that the *p-y* curve shown in Figure 40 "should be employed with

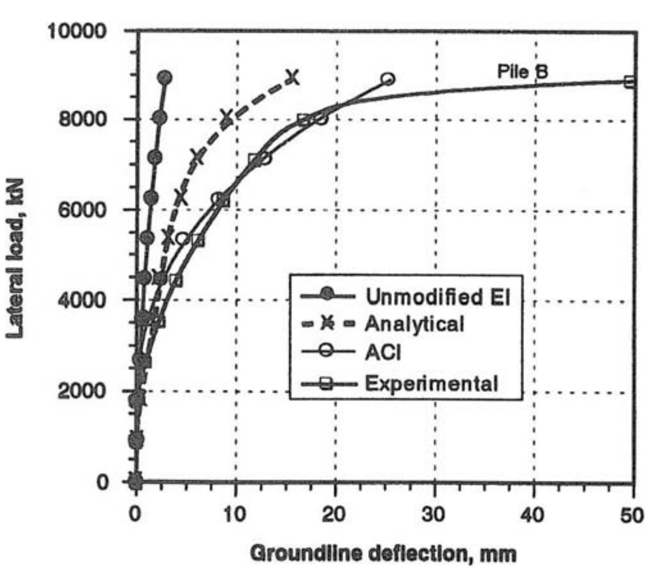


FIGURE 39 Measured and analytical deflection curves, socket in sandstone (Reese 1997).

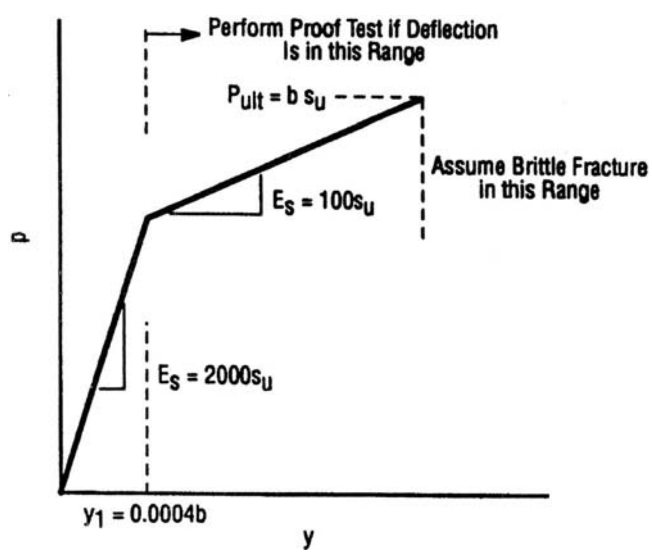


FIGURE 40 Recommended *p-y* curve for strong rock (Ensoft, Inc. 2004).

Passive Earth Pressure Parameters Calculations

OBJECTIVE

Estimate the passive earth pressure coefficients using both Rankine and Coulomb theories.

GIVEN

1) Limited lab data and boring logs
2) PDR plans indicating backfill slope behind in-line wingwalls (2:1) and slopes behind proposed abutments (0.50%, assume horizontal backslope)

ASSUMPTIONS

1) Use 2003 MaineDOT Bridge Design Guide (BDG) backfill soil parameters (Soil Type 4).
2) Design per AASHTO LRFD Bridge Design Specifications, 2014, 7th edition, with 2016 interims.

Recommend that all wingwalls and abutments be backfilled with free-draining (granular) material (i.e. Soil Type 4, Table 3-3, MaineDOT BDG).

The following are the soil properties of Soil Type 4:

Total unit weight: $\gamma_{type\_4} := 125 \cdot pcf$

Internal angle of friction of soil: $\phi'_{type\_4} := 32 \cdot deg$

Interface friction angle (concrete to soil): $\delta_{type\_4} := 24\ deg$

Coefficient of friction (concrete to soil): $\delta_{friction\_type\_4} := 0.45$

Cohesion: $c_{type\_4} := 0 \cdot psf$

ESTIMATE THE PASSIVE EARTH PRESSURE COEFFICIENT USING RANKINE THEORY (K_p) FOR USE IN DESIGNING WINGWALLS AND ABUTMENTS

Bowles does not recommend the use of Rankine theory for calculating the passive earth pressure coefficient when the backfill surface is sloped greater than 0 degrees.

The passive earth pressure coefficient using the Rankine theory per Das, Principles of Geotechnical Engineering, 7th Edition, Eq. 13.22:

$$K_p := \tan\left(45\ \mathbf{deg} + \frac{\phi'_{type\_4}}{2}\right)^2$$

$$K_p = 3.25$$

The resultant earth pressure force, Pp, is oriented at an angle, Beta, to the vertical plane.

ESTIMATE THE PASSIVE EARTH PRESSURE COEFFICIENT USING COULOMB THEORY (K_p) FOR USE IN DESIGNING WINGWALLS AND ABUTMENTS

For cases where the interface friction is considered (this is for gravity shaped structures), use Coulomb theory.

For precast integral abutments bearing against clean sand, silty sand-gravel mixtures; use an interface friction angle ranging from 17 to 22 degrees per LRFD Table 3.11.5.3-1.

The interface friction angle between the backfill and wall taken as specified in LRFD Table 3.11.5.3-1: $\delta_{LRFD} := 19.5 \cdot \mathbf{deg}$

The angle of the backface of the wall to the horizontal, θ: $\theta := 90\ \mathbf{deg}$

Beta value is 0 degrees (the Cuolomb theory coefficient does not change based on the slope of backfill), $\beta_{horizontal}$: $\beta_{horizontal} := 0\ \mathbf{deg}$

$$K_p := \frac{\sin\left(\theta - \phi'_{type\_4}\right)^2}{\sin(\theta)^2 \cdot \sin\left(\theta + \delta_{LRFD}\right) \cdot \left(1 - \sqrt{\frac{\sin\left(\phi'_{type\_4} + \delta_{LRFD}\right) \cdot \sin\left(\phi'_{type\_4} + \beta_{horizontal}\right)}{\sin\left(\theta + \delta_{LRFD}\right) \cdot \sin\left(\theta + \beta_{horizontal}\right)}}\right)^2}$$

$$K_p = 6.73$$

The resultant earth pressure force, Pp, is oriented at the interface friction angle (19.5 degrees) to the normal drawn to the backface of the wall. The resultant passive earth pressure force should be assumed to act a distance of H/3 measured from the bottom of the footing.

The derivation is similar to that for Rankine's active state.

Figure 13.8c shows the variation of passive pressure with depth. For cohesionless soils ($c' = 0$),

$$\sigma'_p = \sigma'_o \tan^2\left(45 + \frac{\phi'}{2}\right)$$

or

$$\frac{\sigma'_p}{\sigma'_o} = K_p = \tan^2\left(45 + \frac{\phi'}{2}\right) \tag{13.22}$$

K_p (the ratio of effective stresses) in the preceding equation is referred to as the *coefficient of Rankine's passive earth pressure.*

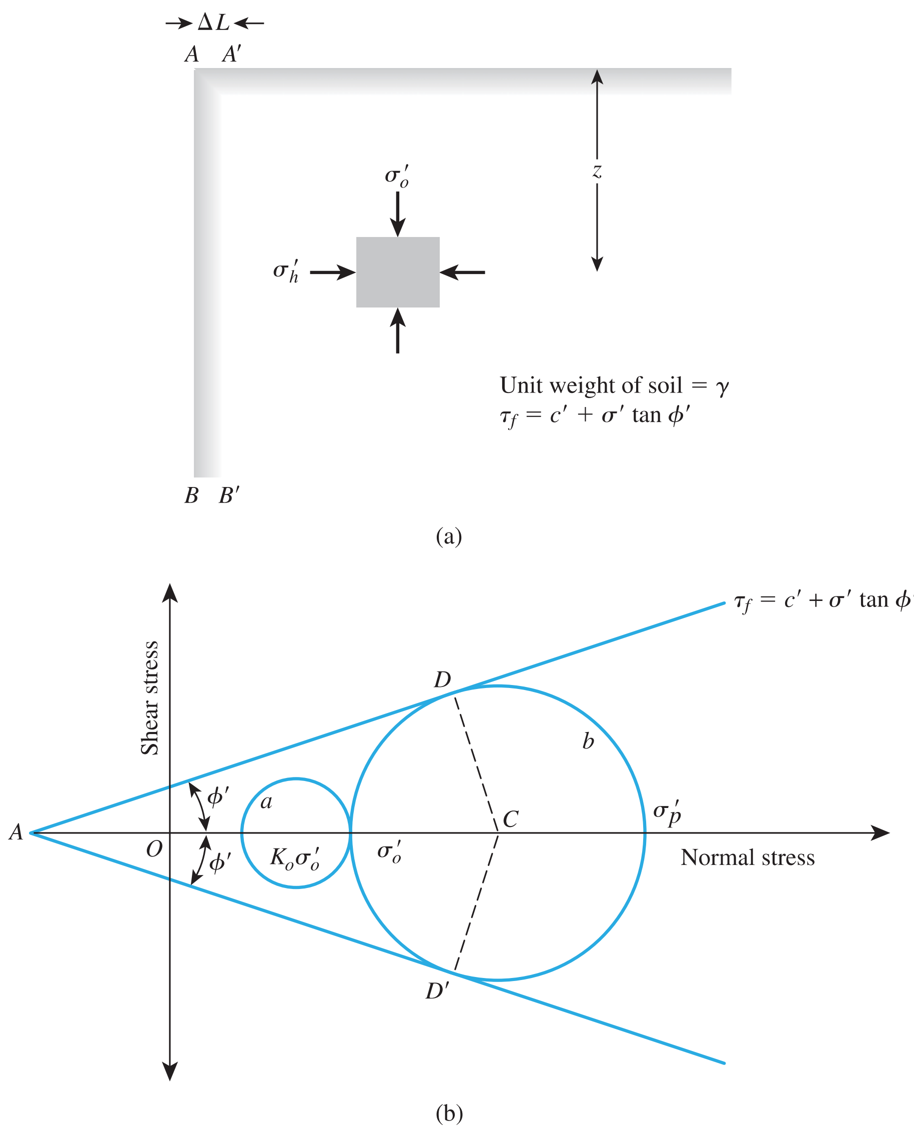


Figure 13.8 Rankine's passive earth pressure

Frost Protection Design Calculations

OBJECTIVE

Estimate the depth foundations bearing on soil should be founded to prevent against frost penetration using the Design Freezing Index (MaineDOT BDG) and ModBerg software methods.

GIVEN

1) Limited lab data and boring logs.

ASSUMPTIONS

1) Estimated soil properties.
2) References: MaineDOT Bridge Design Guide (BDG) 5.2.1 and the ModBerg software.

METHOD NO. 1 - MAINEDOT DESIGN FREEZING INDEX (DFI) MAP AND DEPTH OF FROST PENETRATION TABLE (BDG SECTION 5.2.1)

From the Design Freezing Index Map:

Waterboro and Limerick, Maine
DFI approximately 1,300 degree-days

From lab testing: soils are coarse-grained with a water content (wc) = ~5% (average from all borings)

From MaineDOT Table 5-1:

For a Design Freezing Index of 1,300 and wc less than 10%, the frost penetration is 76.3 inches.

$$Frost\_depth\_BDG := 76.3 \cdot in$$

$$Frost\_depth\_BDG = 6.4\ ft$$

METHOD NO. 2 - CALCULATE FROST DEPTH PER MODBERG SOFTWARE

Closest Station is Sanford

```
------------------------
--- ModBerg Results ---
-----------------------

Project Location: Sanford 2 NNW, Maine

Air Design Freezing Index        =  1123 F-days
N-Factor                         =  0.80
Surface Design Freezing Index    =   898 F-days
Mean Annual Temperature          =  46.8 deg F
Design Length of Freezing Season =  116 days

---------------------------------------------------------
Layer
#:Type        t    w%    d    Cf  Cu   Kf   Ku    L
---------------------------------------------------------
1-Coarse     55.3  5.0 125.0  24  28   1.2  1.3   900
---------------------------------------------------------

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 = 4.61 ft = 55.3 in.
*********************************************************"
```

Therefore, considering both methods, use a frost depth that averages both methods:

$$Frost\_depth\_modberg := 4.61\ ft$$

$$Frost\_depth\_design := \frac{Frost\_depth\_BDG + Frost\_depth\_modberg}{2}$$

$$Frost\_depth\_design = 5.5\ ft$$

Seismic Design Parameters Calculations

OBJECTIVE

1) Determine the site classification per LRFD Table C3.10.3.1-1 - Method B and calculate the following site-specific seismic design parameters: the design spectral response parameter at short periods (Sds) and the design spectral response parameter at a period of 1 second (Sd1).
2) Calculate the peak ground acceleration (PGA), short- and long-period spectral acceleration coefficients (Ss and S1, respectively) for a rock site (Site Class B) using the USGS 2007 Seismic Parameters CD for 7% probability exceedence in 75 years.

GIVEN

1) Limited lab data and boring logs
2) Project lattitude and longitude

ASSUMPTIONS

1) Estimated soil properties.

DETERMINE SITE-SPECIFIC SPECTRAL ACCELERATION RESPONSE PARAMETERS PER THE CALCULATED SITE CLASS (C)

Per Method B of LRFD Table C3.10.3.1-1, the site is defined as Site Class C. The table below was used to classify the site based on the corrected blow counts at the three boring locations.

Seismic Site Classification
Reference: LRFD Tables 3.10.3.1-1 and C3.10.3.1-1
Method B: Average N_{60} for the top 100 feet soil and/or bedrock at the site

Boring BB-WLLOR-101

Depth of Blow Count (bgs, feet)	N_{60} [1,2,3,4] (bpf)	Soil Type	Depth to Top of Layer (bgs, feet)	Depth to Bottom of Layer (bgs, feet)	Layer thickness, d_i (feet)	d_i/N_{60}
3	24	sand (fill)	0.0	4.5	4.5	0.19
6	27	sand (fill)	4.5	9.0	4.5	0.17
11	15	sand	9.0	13.5	4.5	0.30
16	20	sand	13.5	18.5	5.0	0.25
21	47	sand	18.5	23.5	5.0	0.11
25.7	50	sand	23.5	25.7	2.2	0.04
25.7	100	BEDROCK	-	-	74.3	0.74
				Sum of d_i and d_i/N_{60} =	**100.0**	**1.80**

Boring BB-WLLOR-103

Depth of Blow Count (bgs, feet)	N_{60} [1,2,3,4] (bpf)	Soil Type	Depth to Top of Layer (bgs, feet)	Depth to Bottom of Layer (bgs, feet)	Layer thickness, d_i (feet)	d_i/N_{60}
2	23	sand (fill)	0.0	5.0	5.0	0.22
6	12	sand (fill)	5.0	10.5	5.5	0.46
11	15	sand	10.5	13.5	3.0	0.20
16	39	sand	13.5	19.2	5.7	0.15
19.2	100	BEDROCK	-	-	80.8	0.81
				Sum of d_i and d_i/N_{60} =	**100**	**1.83**

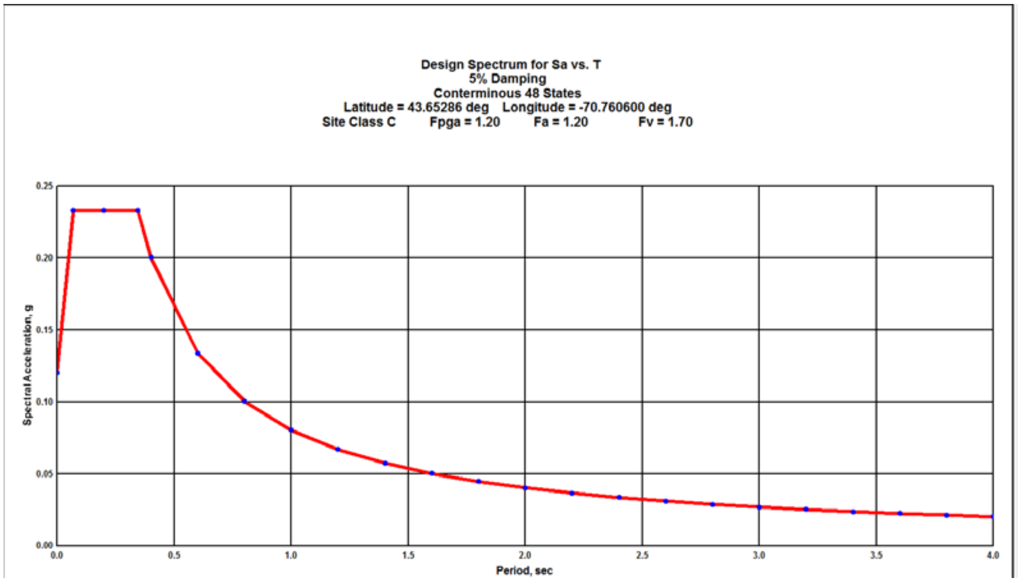
Notes:
1) Weight of rod (WOR) and weight of hammer (WOH) values are taken as N_{60} = 1
2) N_{60} values are limited to less than 100
3) N_{60} values for bedrock layers are 100
4) bpf = blows per foot

Average N_{60} value in the upper 100 feet at Boring BB-WLLOR-101 = 55.6
Average N_{60} value in the upper 100 feet at Boring BB-WLLOR-103 = 54.6
Average N_{60} value in borings = 55.1

The average N_{60} value (55 bpf) of the borings is greater than 50 bpf; therefore, the site is classified as 'Site Class C' per LRFD Table 3.10.3.1-1

DETERMINE THE SPECTRAL ACCELERATION RESPONSE PARAMETERS AND PGA PER SITE CLASS C

Using the USGS 2007 Seismic Parameters CD, the following are the results:



Graph Data

Period, sec	Sa, g
0.00	0.1204
0.07	0.2330
0.20	0.2330
0.34	0.2330
0.40	0.2007
0.60	0.1338
0.80	0.1004
1.00	0.0803
1.20	0.0669
1.40	0.0573
1.60	0.0502
1.80	0.0446
2.00	0.0401
2.20	0.0365
2.40	0.0335
2.60	0.0309
2.80	0.0287
3.00	0.0268
3.20	0.0251
3.40	0.0236
3.60	0.0223
3.80	0.0211
4.00	0.0201

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 43.652000
Longitude = -070.760000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.100	PGA - Site Class B
0.2	0.194	Ss - Site Class B
1.0	0.047	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 43.652000
Longitude = -070.760000
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.120	As - Site Class C
0.2	0.233	SDs - Site Class C
1.0	0.080	SD1 - Site Class C

APPENDIX D

Special Provision

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Rock-Socketed H-Pile Foundation)

501.01 Description. The following is added to subsection 501.01:

This work shall consist of providing all materials, equipment, and labor necessary for construction of a Rock-Socketed H-pile Foundation, as shown on the Plans or as authorized by the Resident. Construction of Rock-Socketed H-Pile Foundation shall be as specified in Section 501 of the Standard Specifications, except as amended herein.

501.02 Materials. The following is added to subsection 501.02:

Aggregate for Rock-Socketed H-pile Foundation shall meet the requirements of 703.22 Underdrain Backfill Material, Type C.

501.03 Quality Control Plan. The following is added to subsection 501.03:

No later than 30 days prior to constructing the Rock-Socketed H-pile Foundation, the Contractor shall submit an installation plan for review by the Resident and project geotechnical engineer. This plan shall provide information on the following:

- List of proposed equipment to be used including: drilling equipment, drills, drill bits, augers, buckets, casing, final cleaning equipment, rock coring equipment, tremies or concrete pumps, etc.;
- Details of overall Rock-Socketed H-Pile Foundation construction operation sequence;
- Details of excavation methods in soils and bedrock, including methods of removing any obstruction such as boulders or cobbles;
- Details of methods to clean bedrock-sockets and bearing surface;
- Details of installing H-Piles and supporting H-piles laterally in their final positions until the abutment is complete and in place; and
- Details of concrete placement.

The Resident and project geotechnical engineer will evaluate the Rock-Socketed H-Pile Foundation installation plan, and all procedural approvals given by the Resident shall be subject to trial in the field and shall not relieve the Contractor of the responsibility to satisfactorily complete the work as detailed in the Plans and Specifications.

501.042 Equipment. The following is added to subsection 501.042:

Rock Sockets. Drilling of bedrock-sockets for Rock-Socketed H-Pile Foundation shall be using cased-hole drilling methods. Excavation equipment and methods shall be designed so that the completed socket will have a planer bottom.

The excavation and drilling equipment shall have adequate capacity including power, torque, and down thrust to excavate a drill socket of both the diameter and to a depth of 20 percent beyond the depth indicated on the plans. When the material encountered cannot be drilled using conventional earth augers with soil or rock teeth, drill buckets, the Contractor shall provide drilling equipment including but not limited to: rock core barrels, rock tools, air tools, and other equipment as necessary to construct the shaft excavation to the size and depth required.

Failure by the Contractor to demonstrate adequate methods and equipment shall be reason for the Resident to require alterations in equipment and/or method by the Contractor to eliminate unsatisfactory results. Any altered methods or construction equipment shall be at the Contractors expense and incidental to this item.

The Contractor shall perform the excavations required for H-pile rock sockets as shown on the Plans, through whatever materials are encountered, to the dimensions and elevations shown on the Plans or otherwise required by the specifications and special provisions. The Contractor's methods and equipment shall be suitable for the intended purpose and materials encountered. Blasting shall not be permitted.

The following sections are added to 501.04 <u>Construction Requirements</u>:

<u>501.049 Drilling and Rock-Socket Excavation</u>.

Bedrock excavations shall be made at locations and to the elevations and dimensions shown on the Plans. Planar bottom socket elevations shall be adjusted when the Resident determines that the material encountered during excavation is unsuitable or differs from that anticipated in the design of the Rock-Socketed H-Pile Foundation.

The Contractor shall maintain a construction method log during bedrock socket drilling and excavation. The log shall contain information such as: drilling methods, drilling resistance, cleaning methods, obstructions, seepage of groundwater through casing/bedrock seal, etc.

Excavated materials which are removed from socket excavations shall be disposed of by the Contractor in accordance with the applicable specifications for disposal of excavated materials.

The Contractor shall perform the necessary excavation for the Rock-Socketed H-pile Foundation under this item. No separate payment will be made for either excavation of materials of different densities or employment of special tools and procedures necessary to accomplish the excavation in an acceptable fashion.

After removal of the soil cuttings from within the casing, the casing shall be further advanced into bedrock, where shown on the plans or directed by the Resident, if necessary to achieve sealing against the entry of overburden. Then the excavation shall continue into bedrock as an uncased or cased bedrock socket of the length and diameter indicated. The bedrock socket shall not be constructed until the casing is sealed in bedrock and until the casing has been checked for plumbness. A method of excavating the bedrock socket that is capable of providing a cylindrical opening of the specific diameter and to full-depth as shown on the plans or to the depth directed by the Resident shall be used. Overbreakage of the bedrock surface shall be avoided, so as to not destroy the seal at the bottom of the casing. The bedrock socket shall be constructed so as to have a planar bottom.

The Contractor shall keep a daily construction record. The Contractor shall provide access and equipment for checking the alignment of the casing and for checking the dimension, alignment and cleanliness of the rock socket. Final pile and socket depths shall be measured with suitable weighted tape or other approved methods after final cleaning. A minimum of 50 percent of the base of each socket shall have less than ½-in. of sediment at the time of placement of the concrete. Socket cleanliness shall be demonstrated by the Contractor to the satisfaction of the Resident. Concrete placement shall not begin until the Resident's approval has been obtained.

<u>501.050 Obstructions</u>

Surface and subsurface obstructions at the pile locations shall be removed by the Contractor. Such obstruction may include man made materials such as old concrete foundations, and natural materials such as boulders. Special procedures and/or tools shall be employed by the Contractor after the casing cannot be advanced using conventional augers fitted with soil or rock teeth, or drilling buckets. Such special procedures/tools may include but are not limited to: chisels, boulder breakers, core barrels, air tools, etc.

Drilling tools which are lost in the excavation shall not be considered obstructions and shall be promptly removed by the Contractor without compensation. All costs due to lost tool removal shall be borne by the Contractor.

<u>501.051 H-Pile and Concrete Installation</u>

The casing shall be installed in a manner that will produce a positive seal at the bottom of the casing so that no piping of water or other materials occurs into the socket.

The H-pile shall be lowered into the cased-hole so as to bear on the planer bottom of the rock socket. The Contractor will be required to support the H-Piles laterally in their final positions until the abutment is complete and in place. The socket shall then be filled with structural concrete, as specified in this Section, to the elevation shown on the Contract Plans.
Rock-Socketed H-Pile Foundation concrete shall conform to Standard Specification 502 - Structural Concrete with the following additional requirements:

Chutes, troughs, pipes and buckets may be permitted in dry sockets only. Concrete placement under water shall be performed by tremie.

After the concrete has been allowed to cure, the cased hole shall be filled with aggregate. Aggregate shall be dropped. Casing may be withdrawn as aggregate is placed.

Where the Contractor is to excavate after rock-socketed H-pile installation, the cased hole does not need to be filled with aggregate. The Contractor is responsible for maintaining the integrity of the fixed, concrete bottom of the H-pile when excavating after installation, as determined by the Resident. Additional length of rock socket, bracing, and other incidentals associated with maintaining the integrity of the concrete bottom of the foundation and maintaining the lateral positions of the piles shall be incidental.

501.047 Splicing Piles. The following is added to subsection 501.047:

Splicing of H-piles for Rock-Socketed H-Pile Foundation shall not be permitted.

501.05 Method of Measurement. The following is added to subsection 501.05:

b) Piles Furnished Furnishing of H-piles for Rock-Socketed H-Pile Foundation shall be as outlined in subsection 501.05.

c) Piles in Place. Method of measurement for constructing Rock-Socketed H-pile Foundation as described in this Section shall be measured by the linear foot of piles in place. Method of measurement shall include all materials, excavation, construction methods, mobilization of equipment, and incidentals necessary to complete the work, as described herein.

501.06 Basis of Payment. The following is added to subsection 501.06:

The accepted quantities of Rock-Socketed H-piles will be paid for at the Contract Unit Price per linear foot, delivered, and complete, in place. Such payment will include full compensation for all material, excavation, construction methods, mobilization of equipment, and incidentals necessary to complete the work as specified herein.

Payment shall be under:

Pay Items		Pay Unit
501.50	Steel H-beam Piles 89 lb/ft, delivered	Linear foot
501.502	Rock-Socketed H-Piles 89 lb/ft, in place	Linear foot
501.XXX	Equipment for Installing H-Piles – Mobilization	Lump Sum