GEOTECHNICAL DESIGN REPORT

For the Removal of:

B&M Railroad Tunnel under US Route 1 Bypass
Kittery, Maine

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York County                           Soils Report 2017-26
WIN 22156.00           Bridge No. 1361

12 May 2017
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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design considerations and recommendations for the proposed removal of the B&M Railroad (RR) Tunnel. The B&M RR Tunnel crosses underneath the US Route 1 Bypass (Bypass) in Kittery, Maine as shown on Sheet 1 – Location Map.

The proposed removal of the B&M RR Tunnel will include backfilling the existing tunnel and constructing permanent fill slopes that tie into the northern and southern openings of the tunnel. Per Sheet 6 of the Preliminary Plans¹, the top slab of the tunnel will be removed down to the construction joints and the tunnel will be backfilled with common borrow. Permanent 2H:1V (horizontal:vertical) to 1.5H:1V fill slopes are planned along the existing northern and southern retaining walls of the Bypass that are adjacent to the existing tunnel.

A gravel driveway parallels the northern retaining wall of the Bypass. Due to the proposed fill slope’s toe along the northern retaining wall, the centerline of the existing driveway will be shifted up to approximately 15 feet to the west. The shift of the existing driveway will require the construction of a “lower” fill slope (1.75H:1V) downslope and to the west of the driveway. The lower fill slope’s toe is near the eastern bank of a cove in the Piscataqua River.

2.0 GEOLOGIC SETTING

According to the Surficial Geology Map, Kittery Quadrangle, Maine, Open-File No. 07-52, 2007, by the Maine Geological Survey (MGS), the surficial soils in the site vicinity consist of marine nearshore deposits with contacts with the Presumpscot Formation. Marine nearshore deposits generally consist of sand, gravel, and silt deposited by wave and current action in shoreline and shallow nearshore environments. Presumpscot Formation soils general consist of silt, clay, and sand deposited on the sea floor.

According to the Bedrock Geology of the Kittery 1:100,000 Quadrangle, Maine and New Hampshire, the bedrock in the site vicinity consists of variably thin- to thick-bedded, buff-weathering, feldspathic and calcareous metawacke of the Kittery Formation.

¹ Maine Department of Transportation, Preliminary Plans, B&M Railroad Tunnel, 21 February 2017.
3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site\(^2\) were investigated by drilling borings BB-KRT-101, BB-KRT-102, BB-KRT-103, BB-KRT-104, BB-KRT-105, and BB-KRT-106. The borings were advanced to refusal (possible top of bedrock). The boring locations are shown on Sheet 2 – Boring Location Plan.

The borings were drilled on 11 April 2017 by New England Boring Contractors, Inc. (NEBC) using a track-mounted drill rig. Details and sampling methods used, field data obtained, and soil conditions encountered are presented on Sheet 3 – Boring Logs and in Appendix A – Boring Logs. The borings were drilled using solid stem augers. Soil samples were obtained continuously with depth or 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 NEBC drill rig is equipped with a 140-pound, rope and cathead hammer falling 30 inches. No correction of N-values is required for the N-values obtained with the standard rope and cathead system where common practice assumes rope and cathead systems have a theoretical 60 percent hammer efficiency. The theoretical hammer efficiency factor, the N-values, and the \(N_{60}\) values are shown on the boring logs.

The project geotechnical engineer selected the boring locations and drilling methods, designated the type and depth of sampling techniques, and identified field and laboratory testing requirements. A Maine Department of Transportation (MaineDOT) subsurface inspector, certified by the Northeast Transportation Technical Certification Program, logged the subsurface conditions encountered in 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 MaineDOT Survey.

4.0 LABORATORY TESTING

Soil 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 the borings to assist in soil classification, evaluation of engineering properties, and for geologic assessment of the site.

The laboratory testing program consisted of two standard grain size analyses with water content measurements and two standard grain size analyses with a hydrometer and water content measurements.

Soil laboratory testing was performed at the MaineDOT Laboratory in Bangor, Maine. The results of the soil laboratory testing are provided in Appendix B – Laboratory Test Results.

\(^2\) The site is defined within the Geotechnical Project Limits shown on Sheet 2 – Boring Location Plan.
Water content information and the soil test results are presented 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 at the site we conclude the site is, in general, underlain by sand, silty sand, silt, and bedrock. The approximate boring refusal depths are presented in Table 1.

The sand, silty sand, and silt strata encountered in borings BB-KRT-101, BB-KRT-102, BB-KRT-103, BB-KRT-104, BB-KRT-105, and BB-KRT-106 ranged from approximately 4.2- to 19.1-feet-thick. The soils encountered consisted of:

- Brown, moist to wet, fine sand, trace to some silt, little to some gravel, trace clay;
- Light brown, brown, moist to wet, fine sand, some silt, some coarse gravel;
- Brown, moist, silty sand, trace to some gravel, trace roots;
- Brown, wet, silty sand, little gravel, trace silt; and
- Light brown, olive, grey, moist to wet, silt, some fine sand, trace fine gravel, trace clay.

Cobbles were encountered 4 to 4.8 and 7.5 to 8 feet below existing ground surface (bgs) at boring BB-KRT-103.

N₆₀ values in the sand and silty sand stratums ranged from 4 to 48 blows per foot (bpf), indicating that the stratums are very loose to dense in density. N₆₀ values in the silt stratum ranged from 13 to 22 bpf, indicating that the stratum is stiff to very stiff in consistency.

Two grain size analyses and two grain size analyses with hydrometers conducted on samples indicate that the soils are classified as an A-4 and A-2-5 soil by the American Association of State Highway and Transportation Officials (AASHTO) Classification System and as a SM, CL, and SC-SM soil by the Unified Soil Classification System. The water contents of the samples tested ranged from approximately 11 to 15 percent.

Table 1 presents the approximate boring refusal depths.

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3 Approximate refusal depths are inferred to be the top of bedrock.
### Table 1. Summary of Approximate Boring Refusal Depths

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Approximate Refusal Depth, bgs (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BB-KRT-101</td>
<td>19.1</td>
</tr>
<tr>
<td>BB-KRT-102</td>
<td>4.8</td>
</tr>
<tr>
<td>BB-KRT-103</td>
<td>14.7</td>
</tr>
<tr>
<td>BB-KRT-104</td>
<td>4.2</td>
</tr>
<tr>
<td>BB-KRT-105</td>
<td>14.3</td>
</tr>
<tr>
<td>BB-KRT-106</td>
<td>4.4</td>
</tr>
</tbody>
</table>

#### 6.0 GEOTECHNICAL EVALUATIONS AND CONSTRUCTION CONSIDERATIONS

Geotechnical evaluations of elastic settlement and global slope stability and construction considerations are presented in the following sections and are in accordance with AASHTO Load Resistance and Factor Design (LRFD) Bridge Design Specifications, 7th Edition, 2014 with up to 2016 interims and relevant MaineDOT Bridge Design Guide (BDG) sections.

#### 6.1 Elastic Settlement

The very loose to dense silty sand and sand and stiff to very stiff silt beneath the proposed fill slopes will undergo elastic compression in response to the estimated stress increase. Elastic settlements due to these changes in the soil’s in-situ effective stress are anticipated to be small and will occur during construction as the backfill is placed and compacted for the fill slopes. Construction and/or surcharge loads could also introduce elastic settlements. However, these settlements are anticipated to be small and will also occur relatively quickly.

#### 6.2 Global Static and Seismic Slope Stability Analyses

Global static and seismic slope stability analyses were performed using the computer software program, Rocscience Slide v6.0, at the most critical section4 (Station 257+50).

The minimum factor of safety required for static stability evaluations is 1.3, where the slope does not support or contain a structural element, based on AASHTO LRFD Article 11.6.2.3. The minimum factor of safety required for seismic stability is 1.0 based on the requirements of the MaineDOT BDG. The lowest, calculated factors of safety for the static and seismic conditions were 3.4 and 2.3, respectively. Unrealistic failure surfaces (i.e. shallow, raveling slope failure) were not considered during the analyses. The Janbu simplified5 method of analysis was used.

Supporting slope stability analyses and calculations are provided in Appendix C – Calculations.

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4 The most critical section was determined by considering the subsurface conditions and proposed fill slopes.
5 The Janbu simplified method yielded the lowest factors of safety out of the three acceptable methods outlined in AASHTO LRFD Article 11.6.2.3.
6.3 Construction Considerations

Construction activities will include common earthwork grading.

The proposed tunnel backfill and fill slopes shall consist of MaineDOT Standard Specification 703.18 – Common Borrow Material. The common borrow should be placed in lifts of 6- to 8-inches-thick loose measure and compacted to the manufacturer’s specifications. In no case shall the backfill soil be compacted less than 90 percent of the AASHTO T-180 maximum dry density.


The Contractor shall install Turf Reinforcement Mat (per MaineDOT Special Provision – Section 613) over a minimum 2-inch-thick Compost Blanket (per MaineDOT Special Provision – Section 615) and seed with a hydraulically applied Flexible Growth Medium (per MaineDOT Special Provision – Section 618) together with a Special Seed Mix (per MaineDOT Special Provision – Section 707.03) over fill slopes that are inclined steeper than 1.75H:1V.

7.0 Closure

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed removal of the B&M RR Tunnel underneath the US Route 1 Bypass in Kittery, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

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

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

Boring Logs
### UNIFIED SOIL CLASSIFICATION SYSTEM

<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>GROUP SYMBOLS</th>
<th>TYPICAL NAMES</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE-GRAINED SOILS</td>
<td>CLEAN GRAVELS</td>
<td>GW</td>
</tr>
<tr>
<td></td>
<td>(little or no fines)</td>
<td>GP</td>
</tr>
<tr>
<td></td>
<td>GRAVEL WITH FINES</td>
<td>GM</td>
</tr>
<tr>
<td></td>
<td>(Approximate amount of fines)</td>
<td>GC</td>
</tr>
<tr>
<td>SANDS</td>
<td>CLEAN SANDS</td>
<td>SW</td>
</tr>
<tr>
<td></td>
<td>(little or no fines)</td>
<td>SP</td>
</tr>
<tr>
<td></td>
<td>SANDS WITH FINES</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>(Approximate amount of fines)</td>
<td>SC</td>
</tr>
<tr>
<td>SILTS AND CLAYS</td>
<td>INORGANIC SEDIMENTS</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>(liquid limit less than 50)</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>(liquid limit greater than 50)</td>
<td>OL</td>
</tr>
<tr>
<td>HIGHLY ORGANIC SOILS</td>
<td>Pt</td>
<td>Peat and other highly organic soils.</td>
</tr>
</tbody>
</table>

### MODIFIED BURMISTERS SYSTEM

<table>
<thead>
<tr>
<th>Descriptive Term</th>
<th>Portion of Total (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace</td>
<td>0 - 10</td>
</tr>
<tr>
<td>Little</td>
<td>11 - 20</td>
</tr>
<tr>
<td>Some</td>
<td>21 - 35</td>
</tr>
<tr>
<td>Adjective (e.g. sandy, clayey)</td>
<td>36 - 50</td>
</tr>
</tbody>
</table>

### TERMS DESCRIBING DENSITY/CONSISTENCY

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

<table>
<thead>
<tr>
<th>Density of Cohesionless Soils</th>
<th>Standard Penetration Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>0 - 4</td>
</tr>
<tr>
<td>Loose</td>
<td>5 - 10</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>11 - 30</td>
</tr>
<tr>
<td>Dense</td>
<td>31 - 50</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt; 50</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Approximate Consistency of Cohesionless Soils</th>
<th>SPT N-Value (blows per foot)</th>
<th>Shear Strength (psf)</th>
<th>Field Guidelines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>WOH, WOR, WOP, &lt; 2</td>
<td>0 - 250</td>
<td>Fist easily penetrates</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>250 - 500</td>
<td>Thumb easily penetrates</td>
</tr>
<tr>
<td>Medium Stiff</td>
<td>5 - 8</td>
<td>500 - 1000</td>
<td>Thumb penetrates with moderate effort</td>
</tr>
<tr>
<td>Stiff</td>
<td>9 - 15</td>
<td>1000 - 2000</td>
<td>Indented by thumb with great effort</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>16 - 30</td>
<td>2000 - 4000</td>
<td>Indented by thumbnail</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 30</td>
<td>&gt; 4000</td>
<td>Indented by thumbnail with difficulty</td>
</tr>
</tbody>
</table>

**Rock Quality Designation (RQD):**

RQD (%) = sum of the lengths of intact pieces of core* > 4 inches

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

**Desired Rock Observations (in this order, if applicable):**

- Color (Munsell color chart)
- Texture (aphanitic, fine-grained, etc.)
- Rock Type (granite, schist, sandstone, etc.)
- Hardness (very hard, hard, mod. hard, etc.)
- Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
- Geologic discontinuities/jointing:
  - spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
  - tightness (light, open, or healed)
  - infilling (grain size, color, etc.)
- Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
- RQD and correlation to rock mass quality (very poor, poor, etc.)
- ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
- Recovery (inch/inch and percentage)
- Rock Core Rate (X.X ft - Y.Y ft [min/sec])

### Sample Container Labeling Requirements:

- WIN
- Bridge Name / Town
- Sample Recovery
- Boring Number
- Date
- Sample Number
- Personnel Initials
- Sample Depth

---

**Maine Department of Transportation**

**Geotechnical Section**

**Key to Soil and Rock Descriptions and Terms**

**Field Identification Information**

October 2016
<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Sample No.</th>
<th>Pen./Rec. (in.)</th>
<th>Sample Depth (ft.)</th>
<th>Blows (/6 in.)</th>
<th>SG (lb)</th>
<th>N (uncorrected)</th>
<th>Shear Strength (psf) or RQD (%)</th>
<th>N60</th>
<th>Casing Blows</th>
<th>Elevation (ft)</th>
<th>Graphic Log</th>
<th>Visual Description and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1D</td>
<td>24/18</td>
<td>0.00 - 2.00</td>
<td>2/6/10/9</td>
<td>16</td>
<td>16</td>
<td>SSA</td>
<td></td>
<td></td>
<td>24.70</td>
<td></td>
<td>Brown, moist, medium dense, Silty SAND, some gravel, roots.</td>
</tr>
<tr>
<td>5</td>
<td>2D</td>
<td>24/16</td>
<td>5.00 - 7.00</td>
<td>3/5/8/12</td>
<td>13</td>
<td>13</td>
<td></td>
<td></td>
<td></td>
<td>3.50</td>
<td></td>
<td>Light brown, moist, stiff, SILT, some fine sand, trace fine gravel, trace clay.</td>
</tr>
<tr>
<td>10</td>
<td>3D</td>
<td>24/18</td>
<td>10.00 - 12.00</td>
<td>3/11/9/7</td>
<td>20</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td>9.10</td>
<td></td>
<td>Olive, wet, very stiff, SILT, some fine sand, trace fine gravel, trace clay.</td>
</tr>
<tr>
<td>15</td>
<td>4D</td>
<td>24/15</td>
<td>15.00 - 17.00</td>
<td>6/10/12/19</td>
<td>22</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
<td>19.10</td>
<td></td>
<td>Similar to Sample 2D; except, grey.</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Trace quartzite fragments at 19 feet bgs.</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom of Exploration at 19.10 feet below ground surface. AUGER REFUSAL.</td>
</tr>
</tbody>
</table>

Remarks:
-bgs = below existing 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.
### Soil/Rock Exploration Log

**US CUSTOMARY UNITS**

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Sample No.</th>
<th>Pen./Rec. (in.)</th>
<th>Sample Depth (ft.</th>
<th>Blows (/6 in.)</th>
<th>U/RQD (%)</th>
<th>N-uncorrected</th>
<th>N60</th>
<th>Casing</th>
<th>Blows</th>
<th>Elevation (ft.)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>0</strong></td>
<td>1D</td>
<td>24/13</td>
<td>0.00 - 2.00</td>
<td>1/2/2/3</td>
<td>4</td>
<td>4</td>
<td>SSA</td>
<td></td>
<td>26.10</td>
<td>Brown, moist, very loose, Silty SAND, trace roots.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2D</td>
<td>24/6</td>
<td>2.00 - 4.00</td>
<td>3/11/16/18</td>
<td>27</td>
<td>27</td>
<td></td>
<td></td>
<td></td>
<td>Brown, wet, medium dense, fine SAND, some silt, little gravel.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3D</td>
<td>9.6/8</td>
<td>4.00 - 4.80</td>
<td>22/50(3.6&quot;)</td>
<td>---</td>
<td></td>
<td></td>
<td></td>
<td>23.30</td>
<td>Brown, wet, fine SAND, some gravel, trace silt.</td>
<td></td>
</tr>
</tbody>
</table>

**Bottom of Exploration at 4.80 feet below ground surface. AUGER REFUSAL.**

---

**Definitions:**
- **D** = Split Spoon Sample
- **U** = Thin Wall Tube Sample
- **V** = Field Vane Shear Test, **PP** = Pocket Penetrometer
- **MV** = Unsuccessful Field Vane Shear Test Attempt
- **R** = Rock Core Sample
- **SSA** = Solid Stem Auger
- **HSA** = Hollow Stem Auger
- **RC** = Roller Cone
- **WOH** = Weight of 140lb. Hammer
- **WOP** = Weight of One Person

**Laboratory Testing Results/AASHTO and Unified Class:**
- **N-uncorrected** = Raw Field SPT N-value
- **N60** = SPT N-uncorrected Corrected for Hammer Efficiency
- **WC** = Water Content, percent
- **LL** = Liquid Limit
- **PL** = Plastic Limit
- **PI** = Plasticity Index
- **G** = Grain Size Analysis
- **C** = Consolidation Test

**Visual Description and Remarks:**
- Brown, moist, very loose, Silty SAND, trace roots.
- Brown, wet, medium dense, fine SAND, some silt, little gravel.
- Brown, wet, fine SAND, some gravel, trace silt.

---

**Remarks:**

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

---

**Boring Location:** Sta 257+47.1, 41.9 feet Lt.

---

**Project:** B&M Railroad Tunnel under US Route 1 Bypass

---

**Boring No.:** BB-KRT-102

---

**Driller:** New England Boring Contractors

---

**Operator:** Tom/Chris

---

**Datum:** NAVD 88

---

**Hammer Efficiency Factor:** 0.60

---

**Hammer Type:** Automatic

---

**Elevation (ft.):** 28.1

---

**Auger ID/OD:** 5-inch-diameter

---

**Datum:** NAVD 88

---

**Sample:** Standard Split Spoon

---

**Hammer Wt./Fall:** 140 lbs/30 inches

---

**Date Start/Finish:** 4/11/2017; 08:30-09:30

---

**Drilling Method:** Solid Stem Auger

---

**Core Barrel:** N/A

---

**Water Level:** Not Observed

---

**Definitions:**
- **R** = Rock Core Sample
- **SSA** = Solid Stem Auger
- **HSA** = Hollow Stem Auger
- **RC** = Roller Cone
- **WOH** = Weight of 140lb. Hammer
- **WOP** = Weight of One Person
- **WC** = Water Content, percent
- **LL** = Liquid Limit
- **PL** = Plastic Limit
- **PI** = Plasticity Index
- **G** = Grain Size Analysis
- **C** = Consolidation Test

---

**Reported Water Levels:**
- **0.00 - 2.00**
- **2.00 - 4.00**
- **4.00 - 4.80**

---

**Remarks:**

- 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.

---

**Bining:**

**Boring No.:** BB-KRT-102

---

**Elevation:**

- 0.00 - 2.00
- 2.00 - 4.00
- 4.00 - 4.80

---

**Rig Type:** Mobile B-53 Track

---

**Presentation:**

- **Win:** 22156.00

---

**Drilling Method:** Solid Stem Auger

---

**Casing ID/OD:** N/A

---

**Operator:** Tom/Chris

---

**Date Start/Finish:** 4/11/2017; 08:30-09:30

---

**Datum:** NAVD 88

---

**Sample:** Standard Split Spoon

---

**Hammer Efficiency Factor:** 0.60

---

**Hammer Type:** Automatic

---

**Elevation (ft.):** 28.1

---

**Auger ID/OD:** 5-inch-diameter

---

**Datum:** NAVD 88

---

**Sample:** Standard Split Spoon

---

**Hammer Wt./Fall:** 140 lbs/30 inches

---

**Date Start/Finish:** 4/11/2017; 08:30-09:30

---

**Drilling Method:** Solid Stem Auger

---

**Core Barrel:** N/A

---

**Water Level:** Not Observed

---

**Definitions:**
- **R** = Rock Core Sample
- **SSA** = Solid Stem Auger
- **HSA** = Hollow Stem Auger
- **RC** = Roller Cone
- **WOH** = Weight of 140lb. Hammer
- **WOP** = Weight of One Person
- **WC** = Water Content, percent
- **LL** = Liquid Limit
- **PL** = Plastic Limit
- **PI** = Plasticity Index
- **G** = Grain Size Analysis
- **C** = Consolidation Test

---

**Reported Water Levels:**
- **0.00 - 2.00**
- **2.00 - 4.00**
- **4.00 - 4.80**

---

**Remarks:**

- 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.
**Sample Information**

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Sample No.</th>
<th>Pen./Rec. (in.)</th>
<th>Sample Depth (ft.)</th>
<th>Blows (/6 in.)</th>
<th>Shear Strength (psf) or RQD (%)</th>
<th>N-uncorrected</th>
<th>Casing Blows</th>
<th>Elevation (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1D</td>
<td>24/16</td>
<td>5.00 - 7.00</td>
<td>8/7/7/7</td>
<td>14</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>2D</td>
<td>24/18</td>
<td>10.00 - 12.00</td>
<td>8/9/14/15</td>
<td>23</td>
<td>23</td>
<td></td>
<td></td>
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<tr>
<td>15</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Visual Description and Remarks**

- Cobble from 4 to 4.8 feet bgs.
  - Brown, moist, medium dense, Silty SAND, little gravel.

- Cobble from 7.5 to 8 feet bgs.
  - Bottom of Exploration at 14.70 feet below ground surface.
  - AUGER REFUSAL.

- Remarks:
  - "bgs = below existing ground surface"

**Definitions:**
- D = Split Spoon Sample
- MD = Unsuccessful Split Spoon Sample Attempt
- MU = Unsuccessful Thin Wall Tube Sample Attempt
- V = Field Vane Shear Test, PP = Pocket Penetrometer
- MV = Unsuccessful Field Vane Shear Test Attempt

---

**Laboratory Testing Results**

- G#270158 A-2-4, SM WC=10.7%

---

**Notes:**

- 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.
<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Sample No.</th>
<th>Pen./Rec. (in.)</th>
<th>Sample Depth (ft.)</th>
<th>Blows (/6 in.)</th>
<th>Shear Strength (psf) or RQD (%)</th>
<th>N-uncorrected</th>
<th>Casing Blows</th>
<th>Elevation (ft.)</th>
<th>Graphic Log</th>
<th>Visual Description and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1D</td>
<td>24/8</td>
<td>0.00 - 2.00</td>
<td>1/1/7/7</td>
<td>8</td>
<td>8</td>
<td>SSA</td>
<td>25.00</td>
<td></td>
<td>Brown, wet, loose, Silty SAND, little gravel, trace silt.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24/5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2D</td>
<td></td>
<td>24/5</td>
<td>2.00 - 4.00</td>
<td>7/5/6/21</td>
<td>11</td>
<td>11</td>
<td></td>
<td>22.80</td>
<td></td>
<td>Brown, wet, medium dense, fine SAND, some silt, some gravel.</td>
</tr>
<tr>
<td>3D</td>
<td>2D/2.4</td>
<td>4.00 - 4.20</td>
<td>50(2.4&quot;)</td>
<td>---</td>
<td></td>
<td></td>
<td></td>
<td>4.20</td>
<td></td>
<td>Bottom of Exploration at 4.20 feet below ground surface. AUGER REFUSAL.</td>
</tr>
</tbody>
</table>

**Definitions:**
- R = Rock Core Sample
- SSA = Solid Stem Auger
- S = Field Vane Undrained Shear Strength (psf)
- SP = Lab Vane Undrained Shear Strength (psf)
- EP = Unconfined Compressive Strength (ksi)
- WC = Water Content, percent
- TL = Liquid Limit
- PI = Plastic Index
- GM = Grain Size Analysis
- PR = Plasticity Index

**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.
Brown, moist, medium dense, fine SAND, some silt, some gravel.

Brown, wet, very loose, fine SAND, some silt, little fine gravel, trace clay.

Brown, wet, loose, fine SAND, some silt, little fine gravel, trace clay.

**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.
### Sample Information

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Sample No.</th>
<th>Sample Depth (ft.)</th>
<th>Blows (/6 in.)</th>
<th>Shear Strength or RQD (%)</th>
<th>N-uncorrected</th>
<th>Casing Blows</th>
<th>Elevation (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1D</td>
<td>0.00 - 2.00</td>
<td>21/36/12/8</td>
<td>48</td>
<td>48</td>
<td>SSA</td>
<td>26.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2D</td>
<td>2.00 - 4.00</td>
<td>4/5/6/17</td>
<td>11</td>
<td>11</td>
<td>SSA</td>
<td>26.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>3D</td>
<td>4.00 - 4.00</td>
<td>50(4.8&quot;)</td>
<td>---</td>
<td></td>
<td></td>
<td>21.80</td>
</tr>
</tbody>
</table>

### Visual Description and Remarks

- **1D**
  - Depth: 0.00 - 2.00 ft
  - Shear Strength or RQD: 21/36/12/8
  - Remarks: Brown, moist, dense, fine SAND, some silt, some gravel.

- **2D**
  - Depth: 2.00 - 4.00 ft
  - Shear Strength or RQD: 4/5/6/17
  - Remarks: Similar to Sample 2D; except, brown and wet.

- **3D**
  - Depth: 4.00 - 4.40 ft
  - Shear Strength or RQD: 50(4.8")
  - Remarks: Bottom of Exploration at 4.40 feet below ground surface.

---

**Definitions:**
- **D =** Split Spoon Sample
- **SSA =** Solid Stem Auger
- **HSA =** Hollow Stem Auger
- **RC =** Roller Cone
- **WOW/C =** Weight of 140lb. Hammer
- **WOP/C =** Weight of One Person

**Laboratory Testing Results/AASHTO and Unified Class:**
- **GIN270159**
- **A-2-4, SM**
- **WC=11.6%**

---

**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.
APPENDIX B

Laboratory Test Results
<table>
<thead>
<tr>
<th>Boring &amp; Sample Identification Number</th>
<th>Station Offset (Feet)</th>
<th>Depth (Feet)</th>
<th>Reference Number</th>
<th>G.S.D.C. Sheet</th>
<th>W.C. %</th>
<th>L.L.</th>
<th>P.I.</th>
<th>Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the &quot;Frost Susceptibility Rating&quot; from zero (non-frost susceptible) to Class IV (highly frost susceptible). The &quot;Frost Susceptibility Rating&quot; is based upon the MaineDOT and Corps of Engineers Classification Systems.</th>
</tr>
</thead>
<tbody>
<tr>
<td>BB-KRT-101, 2D</td>
<td>257+53.6</td>
<td>88.8 Lt.</td>
<td>5.0-7.0</td>
<td>270157</td>
<td>1</td>
<td>15.1</td>
<td>CL</td>
<td>A-4 IV</td>
</tr>
<tr>
<td>BB-KRT-103, 2D</td>
<td>257+90.9</td>
<td>80.7 Lt.</td>
<td>10.0-12.0</td>
<td>270158</td>
<td>1</td>
<td>10.7</td>
<td>SM</td>
<td>A-2-4 II</td>
</tr>
<tr>
<td>BB-KRT-105, 3D</td>
<td>258+71.5</td>
<td>86.5 Lt.</td>
<td>10.0-12.0</td>
<td>270160</td>
<td>1</td>
<td>14.6</td>
<td>SC-SM</td>
<td>A-4 III</td>
</tr>
<tr>
<td>BB-KRT-106, 2D</td>
<td>258+69</td>
<td>58.8 Lt.</td>
<td>2.0-4.0</td>
<td>270159</td>
<td>1</td>
<td>11.6</td>
<td>SM</td>
<td>A-2-4 II</td>
</tr>
</tbody>
</table>

GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98
LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98
PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98
NP = Non Plastic
State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE

SIEVE ANALYSIS
US Standard Sieve Numbers

HYDROMETER ANALYSIS
Grain Diameter, mm

GRAIN SIZE DISTRIBUTION CURVE

Percent Finer by Weight

Percent Retained by Weight

GRAVEL  SAND  SILT

UNIFIED CLASSIFICATION

<table>
<thead>
<tr>
<th>Boring/Sample No.</th>
<th>Station</th>
<th>Offset, ft</th>
<th>Depth, ft</th>
<th>Description</th>
<th>W, %</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>BB-KRT-101/2D</td>
<td>257+53.6</td>
<td>88.8 LT</td>
<td>5.0-7.0</td>
<td>SILT, some fine sand, trace fine gravel, trace clay.</td>
<td>15.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BB-KRT-103/2D</td>
<td>257+90.9</td>
<td>80.7 LT</td>
<td>10.0-12.0</td>
<td>fine SAND, some silt, little fine gravel.</td>
<td>10.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BB-KRT-105/3D</td>
<td>258+71.5</td>
<td>86.5 LT</td>
<td>10.0-12.0</td>
<td>fine SAND, some silt, little fine gravel, trace clay.</td>
<td>14.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BB-KRT-106/2D</td>
<td>258+69</td>
<td>58.8 LT</td>
<td>2.0-4.0</td>
<td>fine SAND, some silt, some coarse gravel.</td>
<td>11.6</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

WIN

022156.00

Town

Kittery

Reported by/Date

WHITE, TERRY A  4/28/2017
APPENDIX C

Calculations
Global Static and Seismic Slope Stability Analyses
OBJECTIVE
Estimate the soil properties (total unit weight, friction angle, and undrained shear strength) for the most critical section to perform static and seismic slope stability analyses with Rocscience Slide v6.0

GIVEN
Boring logs with limited lab data

ASSUMPTIONS
1) The total unit weights and friction angles were estimated and correlated from common geotechnical engineering references
2) The undrained shear strength of the silt was estimated using Lambe and Whitman, 1969, Table 7-4 and was estimated based on a blow count of 13 blows per foot
3) There are three different soil layers in the slope stability model (existing fill, proposed fill, and native silt)
4) The most critical section (Station 257+50 - see attached plan view) was determined by reviewing the anticipated quantity of new fill adjacent to the existing Bypass retaining wall and considering the likeliest lowest strength soil (native silt encountered at boring BB-KRT-101 from Elevation 24.7 to 9.1 feet) encountered during the subsurface investigation. This section also had the most shallow estimated bedrock surface, which will likely result in the lowest factor of safety due to the proposed final grading along the gravel driveway and the encountered soil conditions.

1. Estimate the total unit weight of each of the three soil layers

Reference: MaineDOT Bridge Design Guide (BDG), Table 3-3

Proposed fill = 125 pcf (proposed common borrow fill is assumed to be similar to granular borrow backfill, Soil Type 4)
Existing fill = 120 pcf (similar to medium dense to dense sand, Soil Type 2)
Native silt = 100 pcf (similar to stiff to very stiff clay or clayey silt, Soil Type 1)

2. Estimate the friction angles of each of the three soil layers

Reference: MaineDOT Bridge Design Guide (BDG), Table 3-3

Proposed fill = 34 degrees (similar to granular borrow, Soil Type 4, but increased from 32 deg. to 34 deg. to account for compaction)
Existing fill = 33 degrees (similar to medium dense to dense sand, Soil Type 2)
Native silt = 29 degrees (similar to stiff to very stiff clay or clayey silt, Soil Type 1)
3. Estimate the undrained shear strength of the native silt layer (the other soil layers are drained, cohesionless soils; therefore, they do not have any cohesion)

Reference: Lambe and Whitman, 1969, Table 7-4

Since no field vane tests were performed during the investigation, use the N _60 values to estimate the undrained shear strength of the silt. The lowest N _60 value in the silt layer at boring BB-KRT-101 was 13 blows per foot. Therefore, based on Table 7-4 in Lambe and Whitman for a stiff silt, assume the undrained shear strength of the silt layer is **1,000 psf**

4. Estimate the seismic horizontal acceleration coefficient (k_h) for dynamic slope stability analyses

Per AASHTO LRFD Article 11.6.5.2.1, the k_h value (**0.158g**) is equal to the A_s coefficient (PGA*F_PGA) which was calculated below the USGS 2007 Seismic Parameters CD for 7% probability exceedence in 75 years for a Site Class B site

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Sa (g)</th>
<th>Site Class B</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.099</td>
<td>PGA - Site Class B</td>
</tr>
<tr>
<td>0.2</td>
<td>0.189</td>
<td>Ss - Site Class B</td>
</tr>
<tr>
<td>1.0</td>
<td>0.044</td>
<td>S1 - Site Class B</td>
</tr>
</tbody>
</table>

Conterminous 48 States

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Sa (g)</th>
<th>Site Class D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td><strong>0.158</strong></td>
<td>As - Site Class D</td>
</tr>
<tr>
<td>0.2</td>
<td>0.302</td>
<td>SDs - Site Class D</td>
</tr>
<tr>
<td>1.0</td>
<td>0.106</td>
<td>SD1 - Site Class D</td>
</tr>
</tbody>
</table>
3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Soil Description</th>
<th>Internal Angle of Friction of Soil, $\phi$</th>
<th>Soil Total Unit Weight (pcf)</th>
<th>Coeff. of Friction, $\tan \delta$, Concrete to Soil</th>
<th>Interface Friction, Angle, Concrete to Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Very loose to loose silty sand and gravel</td>
<td>$29^\circ$*</td>
<td>100</td>
<td>0.35</td>
<td>19$^\circ$</td>
</tr>
<tr>
<td></td>
<td>Very loose to loose sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Very loose to medium density sandy silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stiff to very stiff clay or clayey silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Medium density silty sand and gravel</td>
<td>$33^\circ$</td>
<td>120</td>
<td>0.40</td>
<td>22$^\circ$</td>
</tr>
<tr>
<td></td>
<td>Medium density to dense sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dense to very dense sandy silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Dense to very dense silty sand and gravel</td>
<td>$36^\circ$</td>
<td>130</td>
<td>0.45</td>
<td>24$^\circ$</td>
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<td></td>
<td>Very dense sand</td>
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<td></td>
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</tr>
<tr>
<td>4</td>
<td>Granular underwater backfill</td>
<td>$32^\circ$</td>
<td>125</td>
<td>0.45</td>
<td>24$^\circ$</td>
</tr>
<tr>
<td></td>
<td>Granular borrow</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Gravel Borrow</td>
<td>$36^\circ$</td>
<td>135</td>
<td>0.50</td>
<td>27$^\circ$</td>
</tr>
</tbody>
</table>

* The value given for the internal angle of friction ($\phi$) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.
point out some of the significant influences of sample disturbance.

Field tests take on an increased importance in soils which are sensitive to disturbance and in subsoil conditions where the soils vary laterally and/or vertically. The most widely used field test method is penetration testing. Figure 7.3 shows some of the penetrometers that have been used for soil investigation. These penetrometers are driven or pushed into the ground and the resistance to penetration is recorded. The most widely used penetration test is the "standard penetration test", which consists of driving the spoon, shown in Fig. 7.4, into the ground by dropping a 140-lb weight from a height of 30 in. The penetration resistance is reported in number of blows of the weight to drive the spoon 1 ft.

Table 7.4 presents a correlation of standard penetration resistance with relative density for sand and a correlation of penetration resistance with unconfined compressive strength for clay. The standard penetration test is a very valuable method of soil investigation. It should, however, be used only as a guide, because there are many reasons why the results are only approximate. Figure 7.5 presents the results of some penetration tests run in a large tank in the laboratory. These test data show that the penetration resistance depends on factors other than relative density. As can be seen, the penetration resistance depends on the confining stress and on the type of soil. Further, the figures show that the test data scatter considerably. The influence of sand type on penetration resistance is particularly large at low densities—those of most interest. Another factor that may have a marked influence on the penetration resistance of a sand is the pore pressure conditions during the measuring operation. If the level of water in the drain hole is lowered prior to penetration measurement, a lowered resistance can result.

Experience has shown that the determination of the shear strength of a clay from the penetration test can be very unreliable. The standard penetration test should be used only as an approximation or in conjunction with other methods of exploration.

### Table 7.4 Standard Penetration Test

<table>
<thead>
<tr>
<th>Penetration Resistance (blows/ft)</th>
<th>Relative Density</th>
<th>Penetration Resistance (blows/ft)</th>
<th>Unconfined Compressive Strength (tons/ft²)</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-4</td>
<td>Very loose</td>
<td>&lt;2</td>
<td>&lt;0.25</td>
<td>Very soft</td>
</tr>
<tr>
<td>4-10</td>
<td>Loose</td>
<td>2-4</td>
<td>0.25-1.50</td>
<td>Soft</td>
</tr>
<tr>
<td>10-30</td>
<td>Medium</td>
<td>4-8</td>
<td>0.50-1.00</td>
<td>Medium</td>
</tr>
<tr>
<td>30-50</td>
<td>Dense</td>
<td>8-15</td>
<td>1.00-2.00</td>
<td>Stiff</td>
</tr>
<tr>
<td>&gt;50</td>
<td>Very dense</td>
<td>15-30</td>
<td>2.00-4.00</td>
<td>Very stiff</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;30</td>
<td>&gt;4.00</td>
<td>Hard</td>
</tr>
</tbody>
</table>

From Terzaghi and Peck, 1948.

In certain countries, such as Holland, subsoil conditions are such that penetration testing has proved to be a relatively reliable technique. More sophisticated techniques such as the friction jacket cone (Bogmann, 1953) have been widely used.

The vane test has proved to be a very useful method of determining the shear strength of soft clays and silts. Figure 7.6 shows various sizes and shapes of vanes which have been used for field testing. The vane is forced into the ground and then the torque required to rotate the vane is measured. The shear strength is determined from the torque required to shear the soil along the vertical and horizontal edges of the vane.

As later chapters in this book will show, a proper subsoil investigation should include the determination of water pressure at various depths within the subsoil. Methods of determining pore water pressure are discussed in Part IV. Part IV also notes how the permeability of a subsoil can be estimated from pumping tests.

Various load tests and field compaction tests may be highly desirable in important soil projects. In this type of test, a small portion of the subsoil to be loaded by the prototype is subjected to a stress condition in the field which approximates that under the completed structure. The engineer extrapolates the results of the field tests to predict the behavior of the prototype.

### 7.7 SUBSOIL PROFILES

Figures 7.7 to 7.17 present a group of subsoil profiles and Table 7.5 gives some information on the geological history of the various profiles. The purposes of presenting these profiles are:

1. Indicate how geological history influences soil characteristics.
2. Give typical values of soil properties.

3. Show dramatically the large variability in soil behavior with depth.
4. Illustrate how engineers have presented subsoil data.

Three considerations were used in the selection of the profiles: first, examples were chosen with different types of geological history; second, most of the profiles are ones for which there are excellent references giving considerably more detail on the characteristics of the soil and engineering problems involved with the particular profile; and finally, most of the profiles selected have been involved in interesting and/or important soil engineering projects.

Some of the soil characteristics shown in the profiles have already been described in this book. These characteristics include water content, soil weight, void ratio, porosity, Atterberg limits, and particle size. Other characteristics, particularly those referring to strength and compressibility, will be discussed in detail in later portions of this book. Reference will then be made back to these profiles.

The profiles illustrate many concepts presented in the preceding parts of this book; some of them are discussed in the remaining part of this section.

### Stress History

In a normally consolidated sedimentary soil both the void ratio and water content decrease with depth in the profile, and the strength therefore increases. This characteristic is illustrated in several of the profiles, e.g., the Norwegian marine clay (Fig. 7.7), the Thames Estuary clay (Fig. 7.10), and the Canadian clay (Fig. 7.11). The London clay is overconsolidated since it was compressed by a greater overburden than now exists. Erosion removed some of the original overburden. As would be expected, the overconsolidated London clay does not
### Material Table

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<tr>
<th>Material Name</th>
<th>Color</th>
<th>Unit Weight (lbs/ft³)</th>
<th>Strength Type</th>
<th>Cohesion (psf)</th>
<th>Phi (deg)</th>
<th>Water Surface Type</th>
<th>Hu Type</th>
<th>Hu</th>
<th>Ru</th>
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### Seismic Slope Stability Analyses

**Company:** MaineDOT  
**Scale:** 1:197  
**Drawn By:** N. Sherwood  
**File Name:** 22156_Kittery_Seismic_Slope Stability_rev1_c_phi.slim  
**Date:** 4/27/2017, 1:54:54 PM  
**Project:** B&M Railroad Tunnel  
**Analysis Description:** Seismic Slope Stability Analysis  
**SLIDEINTERPRET 6.039**