**To:** Carolyn Carlson, P.E., Structures Project Manager

IPD CEE

**From:** Ian Donovan, Geotechnical Engineer, via Callie Ewald, P.E., Geotechnical Engineering Manager

**Date:** December 18, 2018

**Subject:** Stowe BO 1446(37) Geotechnical Data Report

## 1.0 INTRODUCTION

This memorandum summarizes the field exploration and laboratory testing programs, provides a discussion of the exploration program results, presents select soil and foundation parameters, and provides geotechnical engineering recommendations for the design and construction of the replacement of Bridge 51 on Nebraska Valley Road in Stowe, VT.

Elevations (El.) noted herein are in feet (ft) and are referenced to the North American Vertical Datum of 1988 (NAVD88).

## 2.0 EXISTING CONDITIONS

The project site is located along Town Highway (TH) 43 (Nebraska Valley Road), approximately 0.5 miles northwest of its intersection with TH 1 (Moscow Road). The existing bridge, which carries TH 43 over Miller Brook, is a 54 ft long, single span, rolled beam, cast-in-place concrete deck bridge. Concrete abutments support the bridge, founded on spread footings bearing directly on bedrock. The bridge deck is at approximately El. 657. Record plans are not available for this structure.

The site is generally surrounded by pastoral or wooded land; one residential structure is located on the north side of the bridge, adjacent to the west bank of Miller Brook. Overhead utilities are present approximately 400 ft north of the existing bridge, and approximately 250 ft south of the bridge. No underground utilities are present at the site.

## 3.0 PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the subsurface conditions in the vicinity of the existing, and proposed new bridge, and provide geotechnical engineering recommendations for the proposed bridge foundation. Specifically, the scope of work included the following:

- Conduct a subsurface exploration program for the proposed replacement bridge consisting of a total of six (6) borings and two (2) hand steel probes to evaluate subsurface conditions and obtain soil and rock samples for laboratory testing as necessary;
- Develop geotechnical engineering recommendations for the design of the bridge foundation;
- Prepare this memorandum presenting the recommendation of the Geotechnical Section, including data collected as part of this investigation.

## 4.0 SUBSURFACE EXPLORATION PROGRAM

A subsurface exploration program was conducted to investigate the subsurface conditions in the vicinity of the proposed replacement bridge. The drilling program consisted of six (6) borings, B-101 through B-104, B-101A and B-103A, and two hand-steel probes, P-101 and P-102. Borings were drilled by the VTrans drilling crew using a CME 45 skid-mounted drill rig. The borings were completed between August 27 and November 26, 2018.

The borings were drilled using drive and wash techniques with 4-inch inside diameter (I.D.) casing. The borings were drilled to depths between 3.5 and 29.8 ft below ground surface (bgs).

Split-spoon sampling was conducted in soils at five-foot intervals in each of the borings. Sampling was conducted in general accordance with AASHTO T206 (using a 2-inch outside diameter (O.D.) sampler, driven 24 inches by blows from a 140-pound hammer falling freely for 30 inches). The number of blows required to drive the sampler each 6-inch increment was recorded and the Standard Penetration Test (SPT) resistance (N-value) was calculated as the sum of the blows over the middle 12 inches of penetration. Split spoon refusal is defined as less than 6 inches of penetration resulting from 50 blows from a 140-pound hammer.

Rock coring was conducted with an NX core barrel in general accordance with AASHTO T225 at three (3) boring locations. Rock core samples were described and logged in the VTrans Central Laboratory. The core descriptions include percent recovery, Rock Quality Designation (RQD), orientation and frequency of fractures, observed fracture infilling or coatings, the weathering state of the core, and other characteristics of note. The RQD was evaluated for each core run by dividing the total length of the rock core segments longer than four inches over the total length of the core run. The Rock Mass Rating (RMR) was also calculated and is included on the boring logs. RMR is AASHTO's recommended method of classifying rock for engineering purposes, and is based on rock strength, RQD, joint spacing, joint condition, and groundwater conditions.

Groundwater levels at the boring locations were estimated from the condition of the samples obtained and by observed water levels within the boreholes at the time of drilling.

The borings were located in the field using a handheld Trimble Geoexplorer 600 GPS unit. Elevations for the borings were read from the VTrans survey file dated March 2, 2018.

A boring location plan is included in **Attachment A**; boring logs are included in **Attachment B**.

## 5.0 GEOTECHNICAL LABORATORY TESTING

Based on visual identification of the soils and the characteristics of the proposed construction, laboratory testing of soil or rock samples was not deemed required as part of this project. No samples were tested.

## 6.0 SUBSURFACE CONDITIONS

In general, the subsurface conditions encountered at both abutments consisted of up to 0.8 ft of asphalt, sandy gravel, and schist bedrock.

**Sandy Gravel**

Sandy gravel was encountered at all boring locations. Where encountered, this layer is between at least 1.7 and greater than 19.3 ft thick and typically consists of moist, brown, loose to very dense, fine to coarse sand or fine to coarse gravel, with varying amounts of silt. SPT N-values in the sandy gravel stratum ranged from 4 blows per foot (bl/ft) to 63 bl/ft, with an average of 23 bl/ft at the boring locations.

**Schist Bedrock**

Bedrock was encountered at all boring locations except B-101 and B-103, and was cored in borings B-102, B-103A, and B-104. Where encountered, bedrock consisted of silvery gray to white, moderately hard, fresh to slightly weathered SCHIST. Where foliation planes and joints were apparent, they were tight and either fresh or with rust staining and were dipping between 30 and 50 degrees. RQD of the collected rock cores ranged from 57% to 98%, with an average of 77%, and RMR scores ranged from 46 to 66, indicating fair to good rock.

## 7.0 GROUNDWATER CONDITIONS

Groundwater levels were measured at four boring locations at the conclusion of drilling between August 27 and November 26, 2018. All measurements were taken from the ground surface. The recorded groundwater levels ranged between 3.5 and 11.8 ft bgs at the time of this investigation. A summary of groundwater levels encountered during the drilling program is presented in **Table 1**.

## 8.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

**Foundation Design**

As per section 10.5.5.1 of the 2016 AASHTO LRFD Bridge Design Specifications, a resistance factor of 1.0 should be applied to the nominal bearing resistance for use in service limit state design. Service limit state designs include, but are not limited to, settlement and scour. Section 10.5.5.2.2 specifies that a resistance factor of 0.45 will be applied to the nominal bearing resistance for use in strength limit state design for spread footings on rock and soil. Strength limit state design includes, but is not limited to, checks for bearing resistance, sliding and constructability.

Sections 10.5.2 and 10.5.3 of AASHTO outline all design states relevant to spread footing design and their respective resistance factors. Table 8.1 shows the appropriate resistance factors for various design states.

| Design State | Resistance Factor, φ |
|---|---|
| Settlement | 1.0 |
| Scour | 1.0 |
| Bearing Resistance | 0.45 |
| Sliding | 0.80 |

**Table 8.1** Summary of Resistance Factors

Potential for overturning is limited by controlling the location of the resultant of the reaction forces (eccentricity). Eccentricity, e, is measured from the footing's centroid to the location of the resultant force. Eccentricity should be considered for settlement and bearing resistance design of spread footings by using effective footing widths based on AASHTO Section 10.6.1.3.

It is anticipated that the new bridge may be supported by spread footings bearing directly on schist bedrock. The foundations for the proposed structure may be designed for a factored bearing resistance of 30 kips per square foot (ksf) where it bears on bedrock.

Settlement of the bridge, with maximum allowable bearing pressures indicated and designed as recommended herein, is expected to be less than 1 inch, with no more than 0.5 inches of differential settlement.

## Seismic Considerations

For the purpose of generating design earthquake forces for the proposed replacement bridge, in accordance with AASHTO LRFD Bridge Design Specifications the site should be considered as Site Class "B".

Based on the SPT N-values, field descriptions of the in-situ soils, and observed groundwater levels, the soils at this site are not considered susceptible to liquefaction.

## Design Groundwater Level

For design purposes, ground surface elevation should be used as the design groundwater elevation.

## Soil and Rock Properties

Design parameters for in-situ soil and rock, as well as for materials that may be used for backfill, are provided in **Table 2**. Soil and rock parameters were generated using our engineering judgement, generally accepted values for similar soils, and review of the VTrans database of previously conducted rock strength tests.

It is recommended that values for $K_0$ be used for calculating earth pressures in the case where the structure is not allowed to deflect horizontally away from or towards a soil mass. Values for $K_a$ should be used in an active earth pressure condition when the structure is allowed to move away from the retained soil mass, while values for $K_p$ should be used in a passive earth pressure condition when the structure is allowed to move toward the soil mass. The design earth pressure coefficients are based on horizontal backfill and a vertical wall face.

## 9.0 CONSTRUCTION CONSIDERATIONS

**Cofferdams and Temporary Excavation Support**

Should cofferdams and temporary excavation support be required as part of the project, the selection and type of system should be performed by the Contractor. The Contractor is required to retain a professional engineer registered in the State of Vermont to design the excavation support system. Detailed plans and a construction schedule should be provided to VTrans by the Contractor prior to construction of any cofferdams or temporary excavation support systems.

Where open (laid back) excavations are feasible, the sides should be sloped in accordance with OSHA regulations.

**Preparation of Foundation Subgrades**

Exposed bedrock subgrades should be cleared completely of unsuitable material prior to concrete, including soil, vegetation or other deleterious materials. A VTRANS geotechnical engineer or geologist should inspect the subgrade surface to ensure conformance with this requirement. If necessary, any weathered, fragmented or incompetent bedrock may be removed and a concrete sub-footing poured. The geotechnical engineer or geologist should evaluate rock type, degree of weathering and joint characteristics and orientation to verify design assumptions presented in this report.

**Dewatering**

Dewatering may be required during construction of the foundations. Temporary construction dewatering can likely be accomplished by open pumping from shallow sumps, temporary ditches, and trenches within and around the excavation limits. Sumps should be equipped with filters to prevent pumping of fine-grained soil particles. The water trapped by the temporary dewatering controls should be discharged to settling basins or to an approved filter sock so that the fine particles suspended in the water may settle out prior to discharge. All effluent should comply with all applicable permits and regulations.

**Placement and Compaction of Fills**

Fills should be placed in horizontal layers not more than 12 inches thick, prior to compaction. Cobbles larger than 8 inches in diameter should be removed from the fill prior to placement. Compaction equipment should consist of large, self-propelled vibratory rollers. Where hand-guided equipment is used, the loose lift thickness shall not exist 6 inches, and cobbles larger than 4 inches in diameter should be removed from the fill prior to placement.

General embankment fills should be compacted to a dry density of at least 90% of the maximum dry density measured in accordance with AASHTO T-99. Granular Backfill for Structures or other select materials placed within the roadway base section shall be compacted to a dry density equal to 95% of the maximum dry density as measured in accordance with AASHTO T-99.

## 10.0 CLOSING

We can provide assistance with developing plan details and notes for this project. If you have any questions or would like to discuss this report, please contact us at (802) 828-6911. Computer-generated boring logs are available in the *M:\Projects\12j660\MaterialsResearch* folder.

Tables
Table 1 – Summary of Borings
Table 2 – Recommended Soil and Rock Parameters

Attachments
Attachment A – Boring Location Plan
Attachment B – Boring Logs

cc: Electronic Read File/MG
Project File/CEE
IPD

Z:\Highways\CMB\GeotechEngineering\Projects\Stowe BO 1446(37)\REPORTS\Stowe BO 1446(37) Geotechnical Data Report.docx

**Tables**

**Table 1 - Summary of Borings**
**Stowe BO 1446(37)**

| Test Boring Number | Location | Northing | Easting | Approximate Ground Surface Elevation (ft) | Stratum Thickness (ft) | | Depth to Groundwater (ft) | Approximate Groundwater Elevation (ft) | Depth to Bedrock (ft) | Approximate Bedrock Elevation (ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Asphalt | Sand and Gravel | | | | |
| B-101 | Abutment #2 | 710226.914 | 1575588.757 | 658 | 0.8 | >1.7 | NR | NR | NE | NE |
| B-101A | Abutment #2 | 710226.368 | 1575589.876 | 658 | 0.6 | 10.7 | 4 | 654 | 11.3 | 646.7 |
| B-102 | Abutment #2 | 710215.648 | 1575584.080 | 658 | 0.8 | 9.2 | 3.5 | 654.5 | 10 | 648.0 |
| B-103 | Abutment #1 | 710246.257 | 1575523.357 | 657 | 0.8 | >19.3 | NR | NR | NE | NE |
| B-103A | Abutment #2 | 710246.803 | 1575522.231 | 657 | 0.7 | 18.2 | 10.4 | 646.6 | 18.9 | 638.1 |
| B-104 | Abutment #1 | 710257.796 | 1575530.486 | 657 | 0.4 | 19.4 | 11.8 | 645.2 | 19.8 | 637.2 |
| P-1 | Abutment #2 | 710231.334 | 1575599.328 | 657 | NE | 2.9 | NR | NR | 2.9 | 654.1 |
| P-2 | Abutment #2 | 710210.037 | 1575579.312 | 657 | NE | 7.1 | NR | NR | 7.1 | 649.9 |

**Notes**
1. Elevations are referenced to North American Vertical Datum of 1988 (NAVD88)
2. Groundwater depths were measured at the completion of drilling between August 27 and November 26, 2018

**Abbreviations**
NE indicates not encountered
NR indicates not recorded
> indicates stratum not fully penetrated

**Table 2 - Recommended Soil and Rock Parameters**
**Stowe BO 1446(37)**

| Stratum | Unit Weight, $\gamma$ ($lb/ft^3$) | Internal Friction Angle, $\phi$ (°) | Cohesion ($lb/ft^2$) | Coefficient of Friction, f | $K_0$ | $K_a$ | $K_p$ | Unconfined Compressive Strength ($lb/in^2$) |
|---|---|---|---|---|---|---|---|---|
| Sandy Gravel | 125 | 32 | 0 | 0.4 | 0.47 | 0.31 | 3.25 | -- |
| Schist Bedrock | 150 | 42 | -- | 0.7 | -- | -- | -- | 6000 |
| Granular Backfill for Structures | 140 | 35 | 0 | 0.55 | 0.43 | 0.27 | 3.69 | -- |
| Granular Borrow | 130 | 32 | 0 | 0.5 | 0.4 | 0.47 | 0.31 | -- |

# Attachment A – Boring Location Plan

## SOIL CLASSIFICATION

**AASHTO**

| | |
|---|---|
| A1 | Gravel and Sand |
| A3 | Fine Sand |
| A2 | Silty or Clayey Gravel and Sand |
| A4 | Silty Soil - Low Compressibility |
| A5 | Silty Soil - Highly Compressible |
| A6 | Clayey Soil - Low Compressibility |
| A7 | Clayey Soil - Highly Compressible |

## ROCK QUALITY DESIGNATION

| R.Q.D. (%) | ROCK DESCRIPTION |
|---|---|
| <25 | Very Poor |
| 25 to 50 | Poor |
| 51 to 75 | Fair |
| 76 to 90 | Good |
| >90 | Excellent |

## SHEAR STRENGTH

| UNDRAINED SHEAR STRENGTH IN P.S.F. | CONSISTENCY |
|---|---|
| <250 | Very Soft |
| 250-500 | Soft |
| 500-1000 | Med. Stiff |
| 1000-2000 | Stiff |
| 2000-4000 | Very Stiff |
| >4000 | Hard |

## CORRELATION GUIDE OF "N" TO DENSITY/CONSISTENCY

**DENSITY (GRANULAR SOILS)**

| N | DESCRIPTIVE TERM |
|---|---|
| <5 | Very Loose |
| 5-10 | Loose |
| 11-24 | Med. Dense |
| 25-50 | Dense |
| >50 | Very Dense |

**CONSISTENCY (COHESIVE SOILS)**

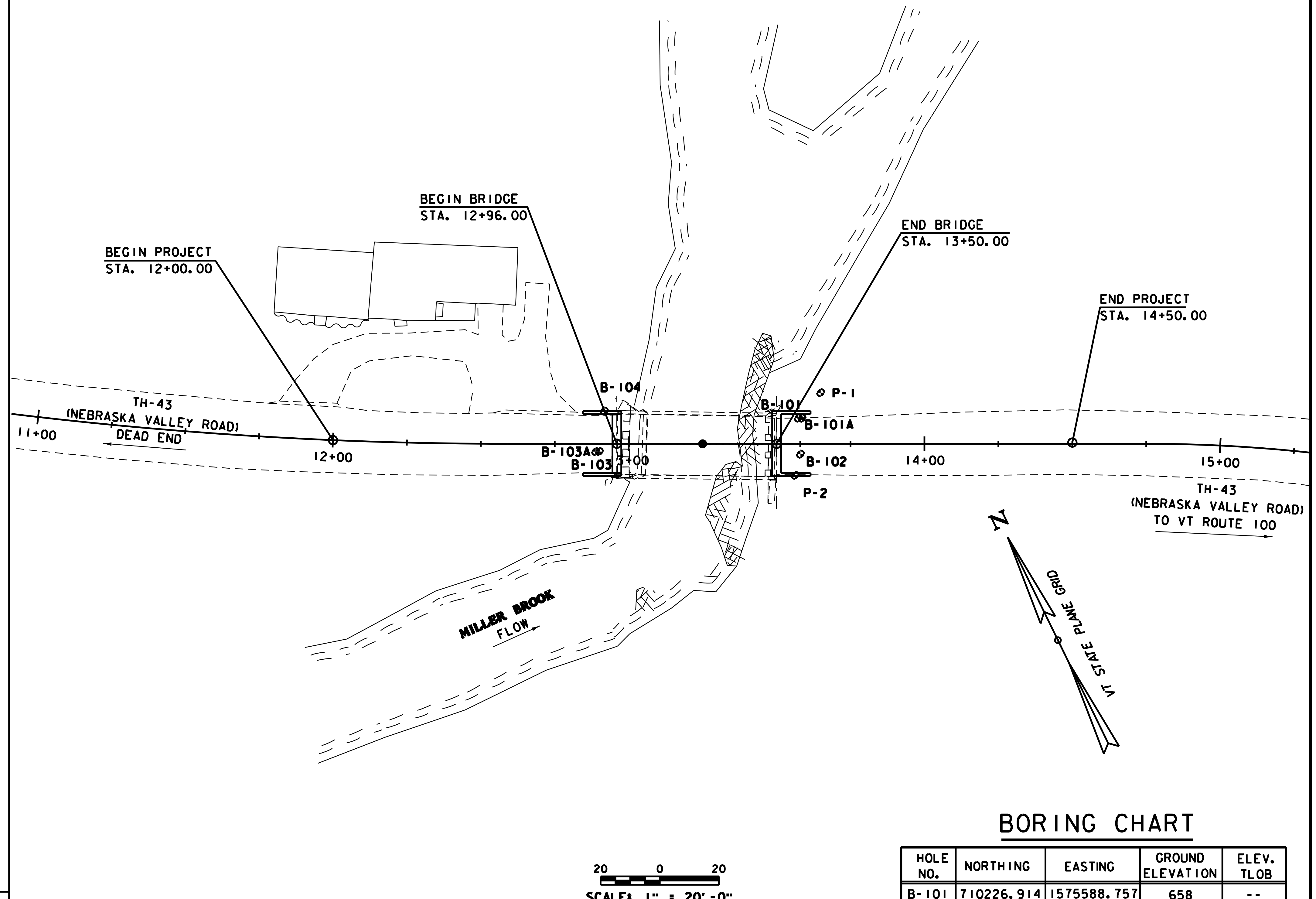
| N | DESCRIPTIVE TERM |
|---|---|
| <2 | Very Soft |
| 2-4 | Soft |
| 5-8 | Med. Stiff |
| 9-15 | Stiff |
| 16-30 | Very Stiff |
| 31-60 | Hard |
| >60 | Very Hard |

## COMMONLY USED SYMBOLS

| | |
|---|---|
| ▼ | Water Elevation |
| ⊘ | Standard Penetration Boring |
| ⊕ | Auger Boring |
| ⊙ | Rod Sounding |
| S | Sample |
| N | Standard Penetration Test<br>Blow Count Per Foot For:<br>2" O.D. Sampler<br>1 3/8" I.D. Sampler<br>Hammer Weight Of 140 Lbs.<br>Hammer Fall Of 30" |
| VS | Field Vane Shear Test |
| US | Undisturbed Soil Sample |
| B | Blast |
| DC | Diamond Core |
| MD | Mud Drill |
| WA | Wash Ahead |
| HSA | Hollow Stem Auger |
| AX | Core Size 1 1/8" |
| BX | Core Size 1 5/8" |
| NX | Core Size 2 1/8" |
| M | Double Tube Core Barrel Used |
| LL | Liquid Limit |
| PL | Plastic Limit |
| PI | Plasticity Index |
| NP | Non Plastic |
| w | Moisture Content (Dry Wgt. Basis) |
| D | Dry |
| M | Moist |
| MTW | Moist To Wet |
| W | Wet |
| Sat | Saturated |
| Bo | Boulder |
| Gr | Gravel |
| Sa | Sand |
| Si | Silt |
| Cl | Clay |
| HP | Hardpan |
| Le | Ledge |
| NLTD | No Ledge To Depth |
| CNPF | Can Not Penetrate Further |
| TLOB | Top of Ledge Or Boulder |
| NR | No Recovery |
| Rec. | Recovery |
| %Rec. | Percent Recovery |
| ROD | Rock Quality Designation |
| CBR | California Bearing Ratio |
| < | Less Than |
| > | Greater Than |
| R | Refusal (N > 100) |
| VTSPG | NAD83 - See Note 7 |

**COLOR**

| | | | |
|---|---|---|---|
| blk | Black | pnk | Pink |
| bl | Blue | pu | Purple |
| brn | Brown | rd | Red |
| dk | Dark | tn | Tan |
| gry | Gray | wh | White |
| gn | Green | yel | Yellow |
| lt | Light | mltc | Multicolored |
| or | Orange | | |




## BORING CHART

| HOLE NO. | NORTHING | EASTING | GROUND ELEVATION | ELEV. TLOB |
|---|---|---|---|---|
| B-101 | 710226.914 | 1575588.757 | 658 | -- |
| B-101A | 710226.368 | 1575589.876 | 658 | 646.7 |
| B-102 | 710215.648 | 1575584.080 | 658 | 648.0 |
| B-103 | 710246.257 | 1575523.357 | 657 | -- |
| B-103A | 710246.803 | 1575522.231 | 657 | 638.1 |
| B-104 | 710257.796 | 1575530.486 | 657 | 637.2 |
| P-1 | 710231.334 | 1575599.328 | 657 | 654.1 |
| P-2 | 710210.037 | 1575579.312 | 657 | 649.9 |

## DEFINITIONS (AASHTO)

BEDROCK (LEDGE) - Rock in its native location of indefinite thickness.

BOULDER - A rock fragment with an average dimension > 12 inches.

COBBLE - Rock fragments with an average dimension between 3 and 12 inches.

GRAVEL - Rounded particles of rock < 3" and > 0.0787" (#10 sieve).

SAND - Particles of rock < 0.0787" (#10 sieve) and > 0.0029" (#200 sieve).

SILT - Soil < 0.0029" (#200 sieve), non or slightly plastic and exhibits no strength when air-dried.

CLAY - Fine grained soil, exhibits plasticity when moist and considerable strength when air-dried.

VARVED - Alternate layers of silt and clay.

HARDPAN - Extremely dense soil, cemented layer, not softened when wet.

MUCK - Soft organic soil (containing > 10% organic material.

MOISTURE CONTENT - Weight of water divided by dry weight of soil.

FLOWING SAND - Granular soil so saturated (loose) that it flows into drill casing during extraction of wash rod.

STRIKE - Angle from magnetic north to line of intersection of bed with a horizontal plane.

DIP - Inclination of bed with a horizontal plane.

## GENERAL NOTES

1. The subsurface explorations shown herein were made between 12/03/2014 and 02/11/2015 by the Agency.

2. Soil and rock classifications, properties and descriptions are based on engineering interpretation from available subsurface information by the Agency and may not necessarily reflect actual variations in subsurface conditions that may be encountered between individual boring or sample locations.

3. Observed water levels and/or conditions indicated are as recorded at the time of exploration and may vary according to the prevailing rainfall, methods of exploration and other factors.

4. Engineering judgment was exercised in preparing the subsurface information presented herein. Analysis and interpretation of subsurface data was performed and interpreted for Agency design and estimating purposes. Presentation of the information in the Contract is intended to provide the Contractor access to the same data available to the Agency. The subsurface information is presented in good faith and is not intended as a substitute for personal investigation, independent interpretation, independent analysis or judgment by the Contractor.

5. Pictorial structure details shown on the boring plan layout or soils profile are for illustrative purposes only and may not accurately portray final contract details.

6. Terminology used on boring logs to describe the hardness, degree of weathering, and spacing of fractures, joints and other discontinuities in the bedrock is defined in the AASHTO Manual on Subsurface Investigations, 1988.

7. Northing and Easting coordinates are shown in Vermont State Plane Grid North American Datum 1983 in meters and survey feet.

PROJECT NAME: STOWE
PROJECT NUMBER: BO 1446 (37)

| | |
|---|---|
| FILE NAME: s12j660bor.dgn | PLOT DATE: 12/6/2018 |
| PROJECT LEADER: C. CARLSON | DRAWN BY: I. DONOVAN |
| DESIGNED BY: C. BURRALL | CHECKED BY: C. EWALD |
| BORING INFORMATION SHEET | SHEET 1 OF 1 |

# Attachment B – Boring Logs

| VTrans Working to Get You There | STATE OF VERMONT<br>AGENCY OF TRANSPORTATION<br>CONSTRUCTION AND<br>MATERIALS BUREAU<br>CENTRAL LABORATORY | BORING LOG<br>**Stowe**<br>**BO 1446(37)**<br>**Nebraska Valley Road Bridge 51** | Boring No.: **B-101**<br>Page No.: 1 of 1<br>Pin No.: 12j660<br>Checked By: SPM |
|---|---|---|---|

Boring Crew: Brochu, Gonyaw, Emerson, Judkins
Date Started: 8/27/18 Date Finished: 8/27/18
VTSPG NAD83: N 710226.90 ft E 1575588.80 ft
Station: 13+60 Offset: 12 LT
Ground Elevation: 658.0 ft

| | Casing | Sampler |
|---|---|---|
| Type: | WASH BORE | SS |
| I.D.: | 4 in | 1.5 in |
| Hammer Wt: | 140 lb. | 140 lb. |
| Hammer Fall: | 30 in. | 30 in. |

Hammer/Rod Type: Auto/AWJ
Rig: CME 45C SKID $C_E$ = 1.42

**Groundwater Observations**

| Date | Depth (ft) | Notes |
|---|---|---|
| | | Not recorded. |

| Depth (ft) | Strata (1) | CLASSIFICATION OF MATERIALS (Description) | Blows/6" (N Value) | Moisture Content % | Gravel % | Sand % | Fines % |
|---|---|---|---|---|---|---|---|
| | | Asphalt, 0.0 ft - 0.75 ft | | | | | |
| 2.5 | | Field Class:, SiSaGr, brn, Moist, Rec. = 0.9 ft | 5-13->50 (>63) | | | | |
| | | Hole stopped @ 3.5 ft | | | | | |
| 5.0 | | | | | | | |
| 7.5 | | | | | | | |
| 10.0 | | | | | | | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | | |
| 22.5 | | | | | | | |

Remarks:
Refusal in concrete encountered 3.5 ft bgs. Offset hole to B-101A.

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. $C_E$ is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

# BORING LOG

**Stowe**
**BO 1446(37)**
**Nebraska Valley Road Bridge 51**

Boring No.: **B-101A**

Pin No.: 12j660
Checked By: SPM

Boring Crew: Brochu, Judkins, Gonyaw
Date Started: 8/27/18 Date Finished: 8/27/18
VTSPG NAD83: N 710226.40 ft E 1575589.90 ft
Station: 13+61 Offset: 12 LT
Ground Elevation: 658.0 ft

| | Casing | Sampler |
|---|---|---|
| Type: | WASH BORE | SS |
| I.D.: | 4 in | 1.5 in |
| Hammer Wt: | 140 lb. | 140 lb. |
| Hammer Fall: | 30 in. | 30 in. |
| Hammer/Rod Type: | Auto/AWJ | |
| Rig: | CME 45C SKID | $C_E$ = 1.42 |

Groundwater Observations

| Date | Depth (ft) | Notes |
|---|---|---|
| 08/27/18 | 4.0 | WT During drilling. |

| Depth (ft) | Strata (1) | CLASSIFICATION OF MATERIALS (Description) | Blows/6" (N Value) | Moisture Content % | Gravel % | Sand % | Fines % |
|---|---|---|---|---|---|---|---|
| 0.0 | | Asphalt, 0.0 ft - 0.6 ft | | | | | |
| 1.0 | | Field Class:, GrSa, brn, Moist, Rec. = 0.6 ft | 8-4-5-17 (9) | | | | |
| 5.0 | | Rec. = 0.0 ft, 5.0 ft - 7.0 ft | 3-3-3-3 (6) | | | | |
| 10.0 | | Field Class:, SiSa with broken rock, gray-brn, Moist, Rec. = 0.7 ft | 1-2->100 (>100) | | | | |
| 11.3 | | Rollerbit 2 ft to confirm bedrock, 11.3 ft - 13.2 ft | | | | | |

Hole stopped @ 13.2 ft

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. $C_E$ is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

| | | |
|---|---|---|
| VTrans Working to Get You There — Vermont Agency of Transportation | STATE OF VERMONT<br>AGENCY OF TRANSPORTATION<br>CONSTRUCTION AND<br>MATERIALS BUREAU<br>CENTRAL LABORATORY | BORING LOG<br>**Stowe**<br>**BO 1446(37)**<br>**Nebraska Valley Road Bridge 51** |

| | |
|---|---|
| Boring No.: | **B-102** |
| Page No.: | 1 of 1 |
| Pin No.: | 12j660 |
| Checked By: | SPM |

Boring Crew: Gonyaw, Judkins, Brochu
Date Started: 8/29/18 Date Finished: 8/30/18
VTSPG NAD83: N 710215.60 ft E 1575584.10 ft
Station: 13+60 Offset: 2 RT
Ground Elevation: 658.0 ft

| | Casing | Sampler |
|---|---|---|
| Type: | WASH BORE | SS |
| I.D.: | 4 in | 1.5 in |
| Hammer Wt: | 140 lb. | 140 lb. |
| Hammer Fall: | 30 in. | 30 in. |
| Hammer/Rod Type: | Auto/AWJ | |
| Rig: | CME 45C SKID | $C_E$ = 1.42 |

**Groundwater Observations**

| Date | Depth (ft) | Notes |
|---|---|---|
| 08/30/18 | 3.5 | WT During drilling. |

| Depth (ft) | Strata (1) | CLASSIFICATION OF MATERIALS (Description) | Run (Dip deg.) | Core Rec. % (RQD %) | Drill Rate minutes/ft | Blows/6" (N Value) | Moisture Content % | Gravel % | Sand % | Fines % |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Asphalt, 0.0 ft - 0.8 ft | | | | | | | | |
| 2.5 | | Field Class:, SaGr w/ broken rock fragments, brn, Moist, Rec. = 1.4 ft | | | | 11-11-40->50 (51) | | | | |
| 5.0 | | Field Class:, SiGrSa, brn, Moist, Rec. = 1.1 ft | | | | 5-3-3-4 (6) | | | | |
| 7.5 | | | | | | | | | | |
| 10.0 | | Field Note:, Refusal, Rec. = 0.0 ft<br>10.01 ft - 15.0 ft, Gray and white, Sulfidic and carbonaceous biotite-muscovite-plagioclase-quartz SCHIST, rust staining along open foliation and joint planes. Moderately hard, Slightly weathered, Fair rock, NX, RMR=46 | R-1 (30) | 92 (57) | 4<br>5<br>5<br>5<br>5 | >50 (>100) | | | | |
| 12.5 | | | | | | | | | | |
| 15.0 | | 15.0 ft - 20.0 ft, Gray and white, Sulfidic and carbonaceous biotite-muscovite-plagioclase-quartz SCHIST, foliation planes and joints are tight and unweathered. Moderately hard, Unweathered, Good rock, NX, RMR=66 | R-2 (30-40) | 98 (98) | 5<br>6<br>8<br>7<br>7 | | | | | |
| 17.5 | | | | | | | | | | |
| 20.0 | | Hole stopped @ 20.0 ft | | | | | | | | |
| 22.5 | | | | | | | | | | |

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. $C_E$ is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

# BORING LOG

**Stowe**
**BO 1446(37)**
**Nebraska Valley Road Bridge 51**

Boring No.: **B-103**
Page No.: 1 of 1
Pin No.: 12j660
Checked By: SPM

Boring Crew: Brochu, Emerson, Judkins
Date Started: 8/28/18 Date Finished: 8/28/18
VTSPG NAD83: N 710246.30 ft E 1575523.40 ft
Station: 12+92 Offset: 2 RT
Ground Elevation: 657.0 ft

| | Casing | Sampler |
|---|---|---|
| Type: | WASH BORE | SS |
| I.D.: | 4 in | 1.5 in |
| Hammer Wt: | 140 lb. | 140 lb. |
| Hammer Fall: | 30 in. | 30 in. |
| Hammer/Rod Type: | Auto/AWJ | |
| Rig: | CME 45C SKID | $C_E$ = 1.42 |

Groundwater Observations

| Date | Depth (ft) | Notes |
|---|---|---|
| | | Not Recorded. |

| Depth (ft) | Strata (1) | CLASSIFICATION OF MATERIALS (Description) | Blows/6" (N Value) | Moisture Content % | Gravel % | Sand % | Fines % |
|---|---|---|---|---|---|---|---|
| 0.0 | | Asphalt, 0.0 ft - 0.75 ft | | | | | |
| 1.0 | | Field Class:, SaGr, brn, Moist, Rec. = 1.4 ft | 11-14-16-16 (30) | | | | |
| 5.0 | | Field Class:, SaGr w/ broken rock, brn, Moist, Rec. = 0.5 ft | 3-2-2-1 (4) | | | | |
| 10.0 | | Field Class:, SaGr w/ broken rock, brn, Moist, Rec. = 0.9 ft | 6-8-15-20 (23) | | | | |
| 15.0 | | Field Class:, SaGr, brn, Moist, Rec. = 1.0 ft | 9-12-12-10 (24) | | | | |
| 20.0 | | Hole stopped @ 20.0 ft | | | | | |

Depth scale (ft): 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0, 22.5

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. $C_E$ is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

# BORING LOG

**Stowe**
**BO 1446(37)**
**Nebraska Valley Road Bridge 51**

Boring No.: **B-103A**

Pin No.: 12j660
Checked By: SPM

Boring Crew: Brochu, Gonyaw, Emerson
Date Started: 11/26/18 Date Finished: 11/26/18
VTSPG NAD83: N 710246.80 ft E 1575522.20 ft
Station: 12+91 Offset: 2 RT
Ground Elevation: 657.0 ft

| | Casing | Sampler |
|---|---|---|
| Type: | WASH BORE | SS |
| I.D.: | 4 in | 1.5 in |
| Hammer Wt: | 140 lb. | 140 lb. |
| Hammer Fall: | 30 in. | 30 in. |

Hammer/Rod Type: Auto/AWJ
Rig: CME 45C SKID $C_E$ = 1.42

Groundwater Observations

| Date | Depth (ft) | Notes |
|---|---|---|
| 11/26/18 | 10.4 | WT During drilling. |

| Depth (ft) | Strata (1) | CLASSIFICATION OF MATERIALS (Description) | Run (Dip deg.) | Core Rec. % (RQD %) | Drill Rate minutes/ft | Blows/6" (N Value) | Moisture Content % | Gravel % | Sand % | Fines % |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Apshalt, 0.0 ft - 0.7 ft | | | | | | | | |
| | | Field Note:, Blind bore to bedrock. | | | | | | | | |
| 2.5 | | | | | | | | | | |
| 5.0 | | | | | | | | | | |
| 7.5 | | | | | | | | | | |
| 10.0 | | | | | | | | | | |
| 12.5 | | | | | | | | | | |
| 15.0 | | | | | | | | | | |
| 17.5 | | | | | | | | | | |
| 20.0 | | 18.9 ft - 23.9 ft, Silvery-gray to white, Sulfidic, muscovite-plagioclase-quartz SCHIST, Open foliation planes and joints are slightly rusty. Core is pitted/vuggy from 21.9 ft bgs to 22.4 ft bgs. Moderately hard, Very slightly weathered, Fair rock, NX | R-1 (30-40) | 88 (68) | 3<br>3<br>2<br>2<br>3 | | | | | |
| 22.5 | | | | | | | | | | |
| | | Hole stopped @ 23.9 ft | | | | | | | | |

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. $C_E$ is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

# BORING LOG

**Stowe**
**BO 1446(37)**
**Nebraska Valley Road Bridge 51**

Boring No.: **B-104**

Pin No.: 12j660
Checked By: SPM

Boring Crew: Judkins, Emerson, Whitlock
Date Started: 9/24/18 Date Finished: 9/25/18
VTSPG NAD83: N 710257.80 ft E 1575530.50 ft
Station: 12+94 Offset: 13 LT
Ground Elevation: 657.0 ft

| | Casing | Sampler |
|---|---|---|
| Type: | WASH BORE | SS |
| I.D.: | 4 in | 1.5 in |
| Hammer Wt: | 140 lb. | 140 lb. |
| Hammer Fall: | 30 in. | 30 in. |

Hammer/Rod Type: Auto/AWJ
Rig: CME 45C SKID $C_E$ = 1.42

Groundwater Observations

| Date | Depth (ft) | Notes |
|---|---|---|
| 09/25/18 | 11.8 | WT During Drilling. |

| Depth (ft) | Strata (1) | CLASSIFICATION OF MATERIALS (Description) | Run (Dip deg.) | Core Rec. % (RQD %) | Drill Rate minutes/ft | Blows/6" (N Value) | Moisture Content % | Gravel % | Sand % | Fines % |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Asphalt, 0.0 ft - 0.38 ft | | | | | | | | |
| 2.5 | | Field Class:, GrSa, brn, Moist, Rec. = 0.9 ft | | | | 10-9-9-10 (18) | | | | |
| 5.0 | | Field Class:, SiSa w/ wood, brn, Moist, Rec. = 0.9 ft | | | | 4-2-5-5 (7) | | | | |
| 7.5 | | | | | | | | | | |
| 10.0 | | Field Class:, GrSa, gry, Moist, Rec. = 1.2 ft | | | | 18-16-26-21 (42) | | | | |
| 12.5 | | | | | | | | | | |
| 15.0 | | Field Class:, SiSa w/ broken rock, brn, Moist, Rec. = 0.7 ft | | | | 4-6-11-19 (17) | | | | |
| 17.5 | | | | | | | | | | |
| 20.0 | | 19.8 ft - 24.8 ft, Silver-gray to white, Sulfidic and carbonaceuos biotite-muscovite-plagioclase-quartz SCHIST, rust along open foliation planes. From 23.6 ft to 24.2 ft rust staining is more pronounced. Moderately hard, Slightly to moderately weathered, Fair rock, NX, RMR=58 | R-1 (40-50) | 100 (76) | 2<br>3<br>3<br>3<br>2 | | | | | |
| 22.5 | | | | | | | | | | |

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. $C_E$ is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

| STATE OF VERMONT<br>AGENCY OF TRANSPORTATION<br>CONSTRUCTION AND<br>MATERIALS BUREAU<br>CENTRAL LABORATORY | BORING LOG<br>**Stowe**<br>**BO 1446(37)**<br>**Nebraska Valley Road Bridge 51** | Boring No.: **B-104**<br>Page No.: 2 of 2<br>Pin No.: 12j660<br>Checked By: SPM |
|---|---|---|

Boring Crew: Judkins, Emerson, Whitlock
Date Started: 9/24/18 Date Finished: 9/25/18
VTSPG NAD83: N 710257.80 ft E 1575530.50 ft
Station: 12+94 Offset: 13 LT
Ground Elevation: 657.0 ft

| | Casing | Sampler |
|---|---|---|
| Type: | WASH BORE | SS |
| I.D.: | 4 in | 1.5 in |
| Hammer Wt: | 140 lb. | 140 lb. |
| Hammer Fall: | 30 in. | 30 in. |
| Hammer/Rod Type: | Auto/AWJ | |
| Rig: | CME 45C SKID | $C_E$ = 1.42 |

Groundwater Observations

| Date | Depth (ft) | Notes |
|---|---|---|
| 09/25/18 | 11.8 | WT During Drilling. |

| Depth (ft) | Strata (1) | CLASSIFICATION OF MATERIALS (Description) | Run (Dip deg.) | Core Rec. % (RQD %) | Drill Rate minutes/ft | Blows/6" (N Value) | Moisture Content % | Gravel % | Sand % | Fines % |
|---|---|---|---|---|---|---|---|---|---|---|
| 27.5 | | 24.8 ft - 29.8 ft, Silver-gray to white, Sulfidic and carbonaceous biotite-muscovite-plagioclase-quartz SCHIST, faint rust staining along joints. Moderately hard, Very slightly weathered, Good rock, NX, RMR=63 | R-2 (30-40) | 100 (86) | 3<br>2<br>2<br>2<br>2 | | | | | |
| 30.0 | | Hole stopped @ 29.8 ft | | | | | | | | |
| 32.5 | | | | | | | | | | |
| 35.0 | | | | | | | | | | |
| 37.5 | | | | | | | | | | |
| 40.0 | | | | | | | | | | |
| 42.5 | | | | | | | | | | |
| 45.0 | | | | | | | | | | |
| 47.5 | | | | | | | | | | |

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. $C_E$ is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

AGENCY OF TRANSPORTATION OFFICE MEMORANDUM

**To:** Carolyn Carlson, P.E., Structures Project Manager

**From:** Callie Ewald, P.E., Geotechnical Engineering Manager CEE

**Date:** June 19, 2019

**Subject:** Stowe BO 1446(37) Geotechnical Addendum

## 1.0 INTRODUCTION

This memorandum summarizes the additional field investigation conducted on the Southwest side of Bridge 51 located over Miller Brook on Nebraska Valley Road (TH 43) in Stowe, VT. A previous Geotechnical Report summarizing our initial subsurface investigation and geotechnical recommendations for the bridge foundations was put together by Ian Donovan in December of 2018. The purpose of this additional investigation was to evaluate the bedrock elevation across the approach footprint for the temporary bridge location for use in depicting the permanent topography of the area post construction.

## 2.0 SUBSURFACE EXPLORATION & RESULTS

The additional subsurface investigation was conducted between April 22nd and 24th, 2019 utilizing a Diedrich D-25 drill rig. Eighteen probes were performed in a grid pattern to determine the depth to presumed bedrock. During drilling operations, no soil samples were collected, and a boulder breaker was advanced until refusal. Borings were previously performed in the area with a minimum of 10 foot of coring into bedrock to confirm the presence and type of bedrock. Based on the surficial nature of the soils, while not confirmed with coring, these elevations are presumed to be the top of bedrock surface in this area.

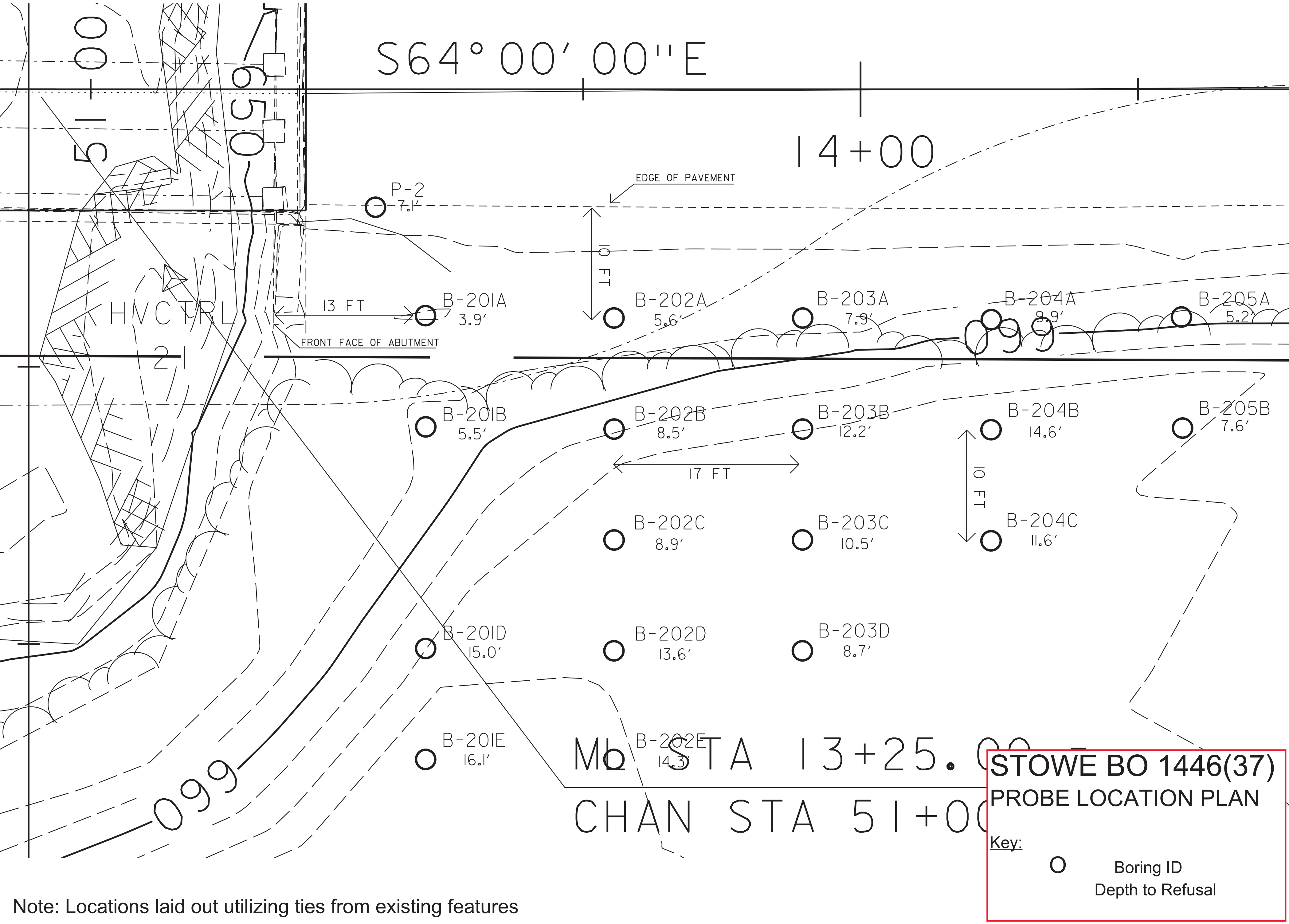
The probes were laid out in the field tying off from features such as the edge of roadway and front face and corner of the southern abutment. The probes were then located in the field post drilling using a handheld Trimble Geoexplorer 600 GPS unit, however heavy vegetation and cloud cover prevented the accuracies from being within acceptable standards. Therefore, the ties to existing features were used in the final probe layout attached. We see this level of accuracy as satisfactory for the purposes described above, however if the borings are to be used to aid in any design as well as make decisions, then we recommend having survey locate the probes.

A Probe Location Plan is attached illustrating probe locations and the tie distances used to locate them. The Probe Field Notes are also attached which depict the ties, field notes, and depth to refusal encountered in each probe.

## 3.0 CLOSING

If you have any questions or would like to discuss the information provided herein, please contact me at (802) 595-4589.

Enclosures: Probe Location Plan
Probe Field Notes




Note: Locations laid out utilizing ties from existing features

# VERMONT AGENCY OF TRANSPORTATION

SHEET 1 OF 1.

## CONSTRUCTION & MATERIALS BUREAU
## GEOTECHNICAL ENGINEERING SECTION
## PROBE / HAND STEEL NOTES

FEET ☑
METERS ☐

PROJECT: STOWE BO 1446(37)    EA No.: 1446037-100

DRILLERS: Brochu, Gonyaw, Emerson    CHECKED BY: Ewald

| Boring No. | Date Drilled | Distance from SW Abutment | Distance from Pavement Edge | Refusal Depth | Sample | Field Notes |
|---|---|---|---|---|---|---|
| | | | | | | |
| B-201A | 4/24/19 | 13' | 10' | 3.9' | P | |
| B-201B | 4/24/19 | 13' | 20' | 5.5' | P | |
| B-201C | | 13' | 30' | | | UNABLE TO PROBE DUE TO STEEP TERRAIN |
| B-201D | 4/22/19 | 13' | 40' | 15.0' | P | |
| B-201E | 4/22/19 | 13' | 50' | 16.1' | P | |
| | | | | | | |
| B-202A | 4/24/19 | 30' | 10' | 5.6' | P | |
| B-202B | 4/22/19 | 30' | 20' | 8.5' | P | |
| B-202C | 4/22/19 | 30' | 30' | 8.9' | P | |
| B-202D | 4/22/19 | 30' | 40' | 13.6' | P | |
| B-202E | 4/22/19 | 30' | 50' | 14.3' | P | |
| | | | | | | |
| B-203A | 4/24/19 | 47' | 10' | 7.9' | P | |
| B-203B | 4/24/19 | 47' | 20' | 12.2' | P | |
| B-203C | 4/24/19 | 47' | 30' | 10.5' | P | |
| B-203D | 4/24/19 | 47' | 40' | 8.7' | P | |
| | | | | | | |
| B-204A | 4/24/19 | 64' | 10' | 9.9' | P | |
| B-204B | 4/23/19 | 64' | 20' | 14.6' | P | |
| B-204C | 4/23/19 | 64' | 30' | 11.6' | P | |
| | | | | | | |
| B-205A | 4/23/19 | 81' | 10' | 5.2' | P | END OF ROD ( BOLDER BUSTER) LOST IN HOLE |
| B-205B | 4/23/19 | 81' | 20' | 7.6' | P | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| P = Probe | | | | | | |

**AGENCY OF TRANSPORTATION** **OFFICE MEMORANDUM**

**To:** Cory Burrall, P.E., Structures Project Manager

END CEE

**From:** Eric Denardo, P.E., Geotechnical Engineer via Callie Ewald, P.E
Geotechnical Engineering Manager

**Date:** April 23, 2024

**Subject:** Stowe BO 1446 (37) - Integral Abutment Recommendations

## 1.0 INTRODUCTION

We have completed the geotechnical and geological analysis for the replacement of Bridge No. 51 on (Nebraska Valley Road) in Stowe VT which crosses over the Miller Brook. This is an addendum to the previous Geotechnical reports submitted by the Geotechnical Engineering Section dated December 18, 2018, and June 2019 which included boring logs and a summary of the preliminary subsurface investigation, and subsequent bedrock probes, respectively. Contained herein are the results from our geotechnical and geological analyses and recommendations for pile supported integral abutments founded on spread footings as determined using the 2020 AASHTO *LRFD Bridge Design Specifications.*

## 2.0 FIELD INVESTIGATION

Two phases of field investigation have been completed for this project. An initial subsurface investigation was completed by the VTrans Geotechnical Engineering Section in 2018 to determine the subsurface conditions and depth to bedrock. A second field investigation completed by VTrans took place in 2019, where eighteen probes were advanced to profile the surface of shallow bedrock encountered in the initial investigation. Refer to these reports for detailed descriptions of the field sampling and testing, laboratory analysis of soil and rock samples, and boring logs. Information gathered for these reports was used to estimate the soil and rock parameters used in these analyses.

## 3.0 ANALYSIS

### Abutment 1 – Pile Supported Integral Abutment:

Developed by the Florida Bridge Software Institute, FB-MultiPier, version 6.0, is a multi-aspect software that allows the user to analyze a bridge pier system in three dimensions. The program's analysis factors in the subsurface strata, pile group including cap, and the structural capabilities of the pier system. For this integral abutment analysis, only the piles and cap were modeled in FB-MultiPier.

**3.1 Loads:** Unfactored loads were provided by Cory Burrall with the Geotechnical Services Request Form dated February 16, 2024, and can be found in Table 3.1.

Our common practice, as outlined in the 2008 VTrans Integral Abutment Manual, is to apply vertical live and dead loading. Longitudinal effects from thermal deformations, wind loads as well as transverse effects from vehicle centrifugal force and wind loads were applied. When analyzing a single abutment, FB-Pier does not consider the longitudinal and transverse stiffness provided by the entire bridge structure; it models the abutment or pier standing alone. Due to this as well as guidance from other states' bridge manuals, it is assumed that all braking forces are to be resisted by the stiffness of the frame that is not accounted for in the analyses.

**Table 3.1:** Unfactored Loads Abutment 1

| Load Type | Load | Value | Elevation(ft) | Direction |
|---|---|---|---|---|
| Superstructure Dead Load* | DC | 331.90 kips | 648.00 | Vertical |
| Superstructure Super Dead | DW | 39.20 kips | 648.00 | Vertical |
| Vehicular Live Load | LL | 163.67 kips | 648.00 | Vertical |
| Vehicular Centrifugal | CE | 5.66 kips | 664.61 | Transverse |
| Wind on Structure | WS | 1.60 kips | 658.61 | Longitudinal |
| | | 6.45 kips | | Transverse |
| Wind on Live Load | WL | 1.10 kips | 664.61 | Longitudinal |
| | | 2.79 kips | | Transverse |
| Thermal Contraction | TU, $\Delta_t$ | 0.30 in | 648.00 | Longitudinal |

**Includes pile cap self-weight*

Abutment 1 was analyzed for both scour and non-scour conditions. A scour elevation of 638.18 feet(ft) was provided in the Abutment Loads and Deformations document provided by the Structures Section dated February 12, 2024.

According to loads provided in Table 3.1 and AASHTO LRFD Table 3.4.1-1, Limit State Strength I and Strength V were analyzed. It was determined that conditions from Strength V governed in the non-scour and scour conditions at Abutment 1. The maximum factored axial load was determined from Strength I, to be 760.6 kips distributed over the abutment, resulting in maximum factored axial load equal to 152.2 kips per pile for a 5-pile layout.

**3.2 Soil Profile:** The following soil strata have been identified based on our review of the boring logs. It should be noted that groundwater elevations are subject to change. Because groundwater elevations can fluctuate seasonally and are affected by temperature and precipitation, groundwater may be encountered during construction when not previously noted in the logs. While all of the subsurface information was used to develop the models, the highest elevation of bedrock, encountered in B-103A, was used in the analysis.

**Abutment 1 - (B-103, B-103A):** The ground surface elevations at boring B-103, was approximately 657 ft. Groundwater was measured during drilling on November 26, 2018, at a depth of 10.4 ft below ground surface (bgs) corresponding to an approximate elevation of 646.6 ft. Bedrock was not encountered to a depth of 20 ft bgs. A second boring, B-103A was advanced next to B-103 down to bedrock. Bedrock was encountered at a depth of 18.9 ft bgs corresponding with a bedrock elevation of 638.1 ft.

The soil parameters used in the analysis for Abutment 1 are displayed below in Table 3.2. Rock parameters are summarized in Table 3.3. It should be noted that the torsional shear stress values are skin friction values for that given layer of soil or rock.

**Table 3.2** FB-Pier Analysis Soil Parameters – Abutment 1

| Elevation (feet) | Description | Friction Angle (deg.) | Unit Weight (pcf) | Subgrade Modulus (pci) | Shear Modulus (ksi) | Torsional Shear Stress (psf) |
|---|---|---|---|---|---|---|
| 648 - 647 | Loose Sandy Gravel | 30 | 100 | 20 | 0.624 | 363.39 |
| 647 - 638 | M. Dense Sandy Gravel | 38 | 125 | 90 | 2.05 | 1130.23 |
| < 638 | Bedrock | 25 | 170 | - | 2218.75 | - |

**Table 3.3** FB-Pier Analysis Rock Parameters

| Parameter | Value |
|---|---|
| Unconfined Compressive Strength (ksf) | 769 |
| Modulus of Elasticity (ksi) | 4970 |
| Poisson's Ratio | 0.12 |
| Shear modulus (ksi) | 2218.75 |

**3.3 Modeling:** The models were analyzed for strength and service loading combinations in both the scour and non-scour conditions. Abutment 1 was modeled as having a 7.38 ft high, 3.0 ft wide, and 27.3 ft long pile cap, with 5 HP 12x84 piles spaced at 5 ft 5 ½ in. on center. A bottom of pile cap elevation of 648.0 ft was used in the analysis for Abutment 1. Dimensions and elevations for the pile cap and pile spacing were taken from the Final Plans dated February 2024.

Due to shallow bedrock encountered in the location of Abutment 1, the piles were modeled to best replicate conditions of pre-drilled holes in rock for the pile installment using a custom P-Y curve developed based on the type and strength of the rock as outlined in the *RSPile Laterally Loaded Piles Manual*. FB MultiPier has an option to create a custom curve based on soil/rock resistance as a function of pile deflection. This was developed using the diameter of the pile and the unconfined compressive strength of the rock encountered. This method was used to model the piles placed in 17.4 in diameter pre-drilled hole through soil and rock then backfilled with sand. The piles were modeled as 14.9 ft long and were assumed to be predrilled and seated in rock sockets drilled a minimum of 3 ft into competent bedrock. All piles were oriented for weak axis bending in a single row. Figure 3.1 shows the pile layout of Abutment 1.

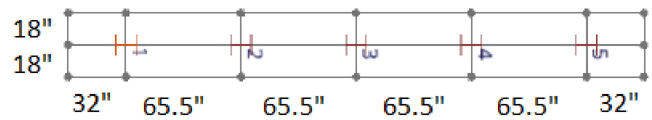



**Figure 3.1:** Abutment 1 Pile Layout

A scour elevation of 638.18 ft was provided by the Structures Section resulting in the piles being modeled as having 9.82 ft of free-standing length in the scour condition Both non-scour and scour models were created in FB-MultiPier to ensure the final pile size satisfied all design requirements for strength and service load cases at both abutments.

## 4.0 RESULTS

### 4.1 Abutment 1 – Pile Supported Integral Abutment:

**4.1.1 Pile Stresses:** Five HP 12x84 piles were modeled for both the non-scour and scour conditions. The models were designed for strength limit state and then evaluated for deflection in the service limit state. The piles were checked for combined axial compression and flexure under the non-scour and scour conditions using the requirements of AASHTO LRFD 6.9.2.2, 6.9.4.1, and 6.10.8.2. FB-MultiPier analyses were performed by applying an axial load, moment, and deflection at the top of each pile. For both the non-scour conditions, Limit State Strength V and Limit State Service I were found to be the two controlling load combinations for both abutments.

The output from FB-MultiPier was used to calculate the factored structural and flexure pile resistance as well as the moment that would cause a plastic hinge in the pile, in accordance with the VTrans 2008 Integral Abutment Bridge Design Guidelines. FB-MultiPier outputs as well as calculated values are displayed below in Table 4.1.

**Table 4.1** FB-Pier Output for AASHTO Governing Strength Case

| Abutment | Soil Condition | Max. Applied Moment (kip-ft) | 2nd Pile Segment Interaction | Factored Lateral Load (kips) | Top Segment Unbraced Length (feet) | Fixity* (feet) |
|---|---|---|---|---|---|---|
| 1 | Non-Scour | 30.4 | 0.11 | 7.4 | 12.4 | 11.9 |
| 1 | Scour | 33.4 | 0.12 | 18.1 | 13.9 | 13.9 |

* Measured from bottom of pile cap

The maximum applied moments are less than the plastic moments calculated in both the scour and non-scour conditions; therefore, a plastic hinge does not develop in the top segment of the pile in these conditions given these loadings. The factored lateral load provided in the tables is the load applied to the top of the pile to achieve the required deflection/rotation times a load factor of 1.2.

The final design resulted in a total of 5 piles spaced at 5 ft 5 ½ in on-center as shown in Figure 3.3. The pile size needed to satisfy design requirements was found to be HP 12x84 piles. The piles will be predrilled so that they are a minimum of 15 ft long and have a 3 ft minimum length rock socket. For the strength limit state, the piles were found to be within the acceptable stress limits. For the service limit state, the maximum deflections were found to be 0.36 and 0.48 in, in the non-scour and scour conditions, respectively.

**4.1.2 Driving Resistances:** Because the piles will be predrilled, the driving resistance of the soil is not a concern. We anticipate drilling to be feasible given the soil conditions present in the location of Abutment 1. <u>The piles should be installed with a rock socket of a minimum of 3 ft drilled into competent rock</u>.

Section 10.7.8 of the AASHTO LRFD BDS stipulates that the maximum tension and compression stresses allowed in the piles shall not exceed $\sigma = 0.9 * \phi_{da} * f_y$. $\phi_{da}$ as defined in AASHTO LRFD 6.5.4.2 as 1.0, resulting in a maximum induced stress in the pile of $0.9 * f_y$ or 45 ksi for grade 50 (50 ksi) piles.

**4.1.3 Nominal Axial Pile Resistance:** The piles will be predrilled and seated in rock sockets. All of the required axial capacity will be generated from the end bearing of the piles on rock. The nominal bearing resistance, $R_N$, shall be factored using the resistance factors, $\Phi_{dyn}$, in Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Specifications. The factored resistance, $R_R$, may be taken as $R_R = \Phi_{dyn} * R_n$. Due to the piles being seated in predrilled rock sockets, no dynamic tests will be performed. For this reason, the resistance factor, $\Phi_{dyn}$, is 0.5. Given the loads provided in Table 3.1, <u>the nominal axial pile resistance, or resistance the piles should be driven to, is 312 kips.</u>

**4.1.4 Pile Cap Design:** The backwalls can be designed as horizontal beams resisting lateral earth pressures. The lateral earth pressure is generated by the movement of the abutment either into (passive earth pressure) or away from (active earth pressure) the soil mass. Passive earth pressure conditions may govern during the warmer months as the structure expands. Similarly, an active earth pressure condition may control during the colder months of the year as the superstructure contracts.

Assuming a distance of 7.38 feet from the bottom of the bridge seat to the bottom of the pile cap for Abutment 1, and the abutment experiencing all of the lateral movement, when the full passive pressure condition would be met. This would produce a passive earth pressure coefficient larger than an active earth pressure coefficient. Therefore, it is conservative to design for the full passive pressure condition at the abutments.

*Equation 1:* $$K_P = \frac{1+\sin\varphi}{1-\sin\varphi}$$

*Equation 2:* $$W_P = \frac{1}{2}\gamma H^2 K_P$$

The passive earth pressure per unit length of backwall can be calculated by inserting the value of $K_p$, computed in Equation 1, into Equation 2. The backfill unit weight is assumed to be equal to 140 pcf with an internal friction angle of 34 degrees. Based on these assumptions and Equations 1 and 2, the total passive earth pressure per unit length of the backwall is calculated to be equal to 13.5 k/ft for Abutment 1.

**4.1.5 Downdrag Analysis:** Negative skin friction, or downdrag, is considered when the relative settlement between the pile and soil equals or exceeds 0.4 inches according to AASHTO 3.11.8. The proposed roadway does not vary significantly in grade with the existing roadway and as a result will not require large amounts of fill. Therefore, neither settlement nor downdrag due to an additional roadway surcharge is expected.

**4.1.6 Settlement Analysis:** Settlement of the abutment is anticipated to be negligible due to the piles being seated in bedrock. Any settlement that does occur should be caused by the elasticity in the piles, which should occur as the piles are loaded.

### 4.2 Abutment 2 – Spread Footing on Rock:

**4.2.1 Bearing Resistance & Settlement:** Due to shallow bedrock at the location of Abutment 2, it is anticipated that it will be supported on a spread footing directly on bedrock. As per section 10.5.5.1 of the 2020 AASHTO LRFD Bridge Design Specifications, a resistance factor of 1.0 should be applied to the unfactored bearing resistance for use in service limit state design. Service limit state design includes, but is not limited to, settlement and scour. Section 10.5.5.2.2 specifies that a resistance factor of 0.45 should be applied to the unfactored bearing resistance for use in strength limit state design for spread footings on rock.

Strength limit state design includes, but is not limited to, checks for bearing resistance, sliding, and constructability. The potential for overturning is limited by controlling the location of the resultant of the reaction forces (eccentricity). Eccentricity, e, shall be limited as follows:

Foundations on rock: $|e| < 0.45b$

Eccentricity should be considered for settlement and bearing resistance design of spread footings by using effective footing widths based on AASHTO Section 10.6.1.3.

The bedrock at the abutment location has fair to good rock quality designation. Classified in B-102 as moderately hard and slightly weathered SCHIST, AASHTO recommends a presumptive bearing resistance of 20 ksf per Table C10.6.2.6.1-1 for "weathered or broken bedrock of any kind except highly argillaceous rock." Taken as the nominal bearing resistance, in combination with a resistance factor of 0.45 for spread footings on rock, per AASHTO 10.5.5.2.2-1, this yields a factored bearing resistance of 9 ksf.

Settlement of Abutment 2, with maximum allowable bearing pressures indicated and footing founded on bedrock, is anticipated to be negligible.

It is recommended that any incompetent, weathered, and fractured bedrock encountered during construction of the spread footing be removed until competent bedrock is encountered. During excavation, the Agency Geologist should inspect the bedrock to determine the amount and extent of excavation needed. If uneven bedrock contours are encountered, the concrete subfooting should be stepped along the existing bedrock in order to transfer the footing pressure directly to the bedrock.

**4.2.2 Resistance Factors:** Sections 10.5.2 and 10.5.3 of AASHTO outline all design states relevant to spread footing design and their respective resistance factors. Eccentricity should be considered for bearing resistance design of spread footings by using effective footing widths based on AASHTO Section 10.6.1.3. Table 4.2 shows the appropriate resistance factors for various design states.

**Table 4.2** Resistance Factors for Design States

| Design State | Resistance Factor, φ |
|---|---|
| Service (Scour) | 1.0 |
| Strength (Bearing Resistance) | 0.45 |
| Sliding | 0.80 |

Additional sliding resistance can be accomplished by doweling the footing into bedrock.

## 5.0 RECOMMENDATIONS

### 5.1 Abutment 1: Integral Abutment Foundations

The following table provides a summary of the requirements for the piles at Abutment 1. The piles should meet the requirements for both minimum pile embedment and minimum pile embedment in bedrock.

**Table 5.1:** Summary of requirements of H-piles at each abutment

| Requirement | Abutment 1 |
|---|---|
| Pile Size | HP 12x84 |
| Number of Piles | 5 |
| Piles Spacing | 65.5 in |
| Pile Orientation | Weak axis |
| Minimum Pile Embedment* | 15 ft |
| Minimum Embedment into Competent Bedrock | 3 ft |
| Method of Installation | Pre-bored and placed |
| Backfill Material | Sand |
| Nominal Axial Pile Resistance | 312 kips |
| Resistance Factor | 0.5 |

*Length of pile below bottom of pile cap

The piles for Abutment 1 will be pre-bored and placed into position in rock sockets. For this reason, dynamic tests are not required; however, the pre-bored piles should be seated in the rock sockets with a pile driving hammer.

**5.2 Construction Considerations:**

**5.2.1 Cofferdams/Temporary Earthwork Support:** Cofferdams or temporary shoring may be necessary during construction of the abutments. If required, the Contractor should be reminded that Section 208.06 of VTrans' *2024 Standard Specifications for Construction* indicates that "The Contractor shall prepare detailed plans and a schedule of operations for each cofferdam specified in the Contract. Construction drawings shall be submitted in accordance with Subsection 105.06."

**5.2.2 Construction Dewatering:** Temporary construction dewatering may be required to construct the foundations. Temporary dewatering can likely be accomplished by open pumping from shallow sumps, temporary ditches, and trenches within and around the excavation limits. Sumps should be provided with filters suitable to prevent pumping of fine-grained soil particles. The water trapped by the temporary dewatering controls should be discharged to settling basins or an approved filter "sock" so that the fine particles suspended in the discharge have adequate time to "settle out" prior to discharge. All effluent water, or discharge, should comply with all applicable permits and regulations.

**5.2.3 Placement and Compaction of Soils:** Fills should be placed systematically in horizontal layers not more than 12 inches in thickness, prior to compaction. Cobbles larger than 8 inches should be removed from the fill prior to placement. Compaction equipment should preferably consist of large, self-propelled vibratory rollers. Where hand-guided equipment is used, such as a small vibratory plate compactor, the loose lift thickness shall not exceed 6 inches. Cobbles larger than 4 inches should be removed from the fill prior to placement.

Embankment fills should be compacted to a dry density of no less than 95% of the maximum dry density determined in accordance with AASHTO T-99, Method C. Granular Backfill for Structures, or other select materials placed within the roadway base section, shall be compacted to a dry density of 95% of the maximum dry density determined in accordance with AASHTO T-99.

**5.2.4 Roadway/Embankment Design:** No geotechnical problems are expected assuming standard Agency construction practices are utilized.

**5.3 Design Parameters:** Engineering properties of common construction materials are shown in Table 5.1. These values should be used when designing the substructure units. It is recommended that values of $K_o$ be used for calculating earth pressures where the structure is not allowed to deflect longitudinally, away from or into the retained soil mass. Values for $K_a$ should be utilized for an active earth pressure condition where the structure is moving away from the soil mass and $K_p$ where the structure is moving toward the soil mass. The design earth pressure coefficients are based on horizontal surfaces (non-sloping backfill) and a vertical wall face.

**Table 5.1:** Engineering Properties of Construction Materials

| | **703.04 – Granular Borrow** | **704.08 – Granular Backfill for Structures** | **In-Situ Bedrock** |
|---|---|---|---|
| Unit Weight, $\gamma$ (lbs/ft$^3$): | 130 | 140 | 170 |
| Internal Friction Angle, $\phi$ (degrees): | 32 | 34 | 25 |
| | | | |
| Coefficient of Friction, f | | | |
| - mass concrete cast against soil: | 0.45 | 0.55 | 0.7 |
| - soil against precast/formed concrete: | 0.40 | 0.48 | N/A |
| | | | |
| Active Earth Pressure Coef., $K_a$: | 0.31 | 0.28 | - |
| Passive Earth Pressure Coef., $K_p$: | 3.26 | 3.54 | - |
| At-Rest Earth Pressure Coefficient, $K_o$: | 0.47 | 0.44 | - |

## 7.0 CONCLUSION

If any further analysis is needed or you would like to discuss this report, please contact us by email. Final FB-MultiPier input files used in the analyses are located in the *M:\Projects\12j660\MaterialsResearch\FB-Multipier*

*Abutment 1 STR I B103 3 FT EMBED [5 PILES] end.in*
*Abutment 1 STR I B103 3 FT EMBED SCOUR [5 PILES] end.in*
*Abutment 1 STR V B103 3 FT EMBED [5 PILES] end.in*
*Abutment 1 STR V B103 3 FT EMBED SCOUR [5 PILES] end.in*
*Abutment 1 SER I B103 3 FT EMBED [5 PILES] end.in*
*Abutment 1 SER I B103 3 FT EMBED SCOUR [5 PILES] end.in*
*Abutment 1 SER II B103 3 FT EMBED [5 PILES] end.in*
*Abutment 1 SER II B103 3 FT EMBED SCOUR [5 PILES] end.in*

cc: Electronic Read File/MG
Project File/CEE
END

Z:\Highways\CMB\GeotechEngineering\Projects\Stowe BO 1446(37)\REPORTS \ Stowe BO 1446 (37) Integral Abutment Reccomendations.docx