AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Ken Upmal, P.E., Roadway Design Project Manager

END CEE

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

Date: May 23, 2016

Subject: Cabot-Danville FEGC-F 028-3(26) – Bridge No. 88 Integral Abutments

1.0 INTRODUCTION

We have completed our geotechnical investigation for the subject project located on US Route 2 at approximately MM 3.4 in Cabot, Vermont crossing the Hooker Brook. The proposed project includes the removal of the existing Bridge No. 88 and replacing it with a 48.1 foot single span NEXT Beam Bridge with associated roadway and channel work. A previous geotechnical report dated July 26, 2012 provided results from the initial subsurface investigation and recommendations for spread footing foundations. Following review of preliminary plans and additional discussions with the Structures section, the substructure was changed from abutments supported on spread footings to integral abutments supported on piles. Contained herein are the results from our additional subsurface investigation, geotechnical analysis and recommendations for integral abutments supported on piles as determined using the 2014 AASHTO *LRFD Bridge Design Specifications.*

2.0 FIELD INVESTIGATION

The field investigation was conducted between March 2, 2016 and March 22, 2016. Two standard penetration borings were drilled to determine the subsurface profiles in order to aid in design and construction of the bridge foundations supported on piles. Boring locations were provided by Mike Cruz and Erik Atkins of Green International with the Geotechnical Services Request Form dated August 7, 2015. A summary of the location of each boring and corresponding ground surface elevation can be found in Table 2.1. The values for the Northings and Eastings are based on the Vermont State Plane Grid Coordinate System NAD 83, and were located by a handheld GPS. Elevations for the borings were then taken off a VTrans survey file. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used to determine them. A boring location plan for the bridge is attached.

Table 2.1 Boring Locations

Boring Number	Station	Offset (ft)	Northing (ft)	Easting (ft)	Elevation (ft)
B-209	41+37.5	21.73	683112.65	1702839.88	1449.6
B-210	41+90.6	23.40	683168.42	1702866.59	1449.3

The borings were performed in general accordance with AASHTO T206, *Standard Method of Test for Penetration Test and Split-Barrel Sampling of Soils.* During boring operations, for boring B-209, split spoon samples and standard penetration tests (SPT) were taken continuously to 25 feet, then at 5 foot intervals to 70 feet, and at 10 foot intervals to a depth of 102 feet. For boring B-210, split spoon samples and SPTs were taken continuously to 25 feet, then at 5 foot intervals to 70 feet, and 10 foot intervals to a depth of 181.6 feet. A boulder was encountered at a depth of approximately 148.1 feet to 158.1 feet. Bedrock was not encountered in either of the borings. Soil samples were visually identified in the field and SPT blow counts were recorded on the boring logs when applicable. Soil samples were preserved and returned to the Construction and Materials Bureau Central Laboratory for testing and further evaluation. Upon completion of the laboratory testing, the boring logs were revised to reflect the results of the laboratory classification analysis.

3.0 FIELD AND LABORATORY TESTING

The standard penetration resistance of the in-situ soil is determined by the number of blows required to drive a 2 inch OD split barrel sampler into the soil with a 140 pound hammer dropped from a height of 30 inches, in accordance with procedures specified in AASHTO T206. During the standard penetration test (SPT), the sampler is driven for a total length of 2 feet, while counting the blows for each 6 inch increment. The SPT N-value, which is defined as the sum of the number of blows required to drive the sampler through the second and third increments, is commonly used with established correlations to estimate a number of soil parameters, particularly the shear strength and density of cohesionless soils. The N-values provided on the boring logs are raw values and have not been corrected for energy, borehole diameter, rod length, or overburden pressure. The VT Agency of Transportation has determined a hammer correction value, C_E, to account for the efficiency of the SPT hammer on the drill rig. For borings B-209 and B-210, a CME 55 Track Rig was used, with a hammer energy correction factor of 1.41. This value, included on the boring logs, was used in calculations to determine soil parameters. Laboratory tests were conducted on all samples to evaluate grain size, moisture content, and percent finer than No. 200 sieve. Results from this testing can be found on the attached boring logs.

4.0 SOIL PROFILE

Review of laboratory data and boring logs revealed the following information pertaining to the soil strata. It should be noted that groundwater elevations are subject to change given the fact that boreholes were generally left open for a short period of time. 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.

B-209 (Abutment 1): The ground surface elevation at B-209 was approximately 1449.6 feet. The groundwater was measured before drilling on March 8, 2016 at a depth of 0.2 feet below the ground surface. No bedrock was encountered to a depth of 102 feet.

Depth (Below Ground Surface Elevation)	Soil Profile
0 – 8 feet	Medium Dense Silty Sand
8 – 14 feet	Dense Sandy Silt
14 – 102 feet	Medium Dense Silt

B-210 (Abutment 2): The ground surface elevation at B-210 was approximately 1449.3 feet. The groundwater was measured before drilling on March 14, 2016 at a depth of 3.2 feet below the ground surface. No bedrock was encountered to a depth of 181.6 feet. A boulder was encountered and cored through from 148.1 feet to 158.1 feet.

Depth (Below Ground Surface Elevation)	Soil Profile
0 – 6 feet	Loose Silty Sand
6 – 20 feet	Medium Dense Sandy Silt
20 – 148 feet	Medium Dense Silt
148 – 158 feet	Boulder
158 – 168 feet	Very Loose Silty Sand
168 – 181.6	Very Dense Sandy Gravel

5.0 ANALYSIS

Developed by the Florida Bridge Software Institute, FB-Multipier, version 4.19.4, is a multi-aspect software that allows the user to analyze a bridge pier system in three dimensions. Its 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.

5.1 Loads: Unfactored loads were provided by Mike Cruz and Erik Atkins of Green International in the Geotechnical Services Request Form dated August 7, 2015. The dead load (DC) provided did not include the weight of the pile cap so the weight of the pile cap was calculated and included in the analyses. Due to the addition of approximately 8.5 feet of fill material at the abutments, downdrag analyses were performed and a load was added to each pile in accordance with AASHTO 3.11.8 and described in more detail in Section 6.4 below. A summary of the loads used can be found in Tables 5.1 and 5.2 for Abutments No. 1 and No. 2, respectively. A total expected thermal movement of 0.14 inches per abutment was calculated based on the thermal expansion coefficient of the concrete NEXT Beams, the expected temperature range, and the length of the structure in accordance with AASHTO Section 3.12.2. A live load rotation of 0.01 radians per abutment was assumed for the analysis based on past projects of similar type and magnitude.

Table 5.1: Unfactored Loads – Abutment No. 1

Type	Unfactored Loads/Abutment	Load Orientation
Dead Loads, DC*	905.67 kips	Vertical
Wearing Loads, DW	40.25 kips	Vertical
Live Loads, LL	182.24 kips	Vertical
Downdrag Loads, DD**	26.91 kips	Vertical

**Includes weight of pile cap* ***Load per pile*

Table 5.2: Unfactored Loads – Abutment No. 2

Type	Unfactored Loads/Abutment	Load Orientation
Dead Loads, DC*	897.57 kips	Vertical
Wearing Loads, DW	40.25 kips	Vertical
Live Loads, LL	182.24 kips	Vertical
Downdrag Loads, DD**	27.15 kips	Vertical

**Includes weight of pile cap* ***Load per pile*

Our common practice, as outlined in the 2008 VTrans Integral Abutment Manual, is to apply vertical live and dead loading, as well as longitudinal effects from thermal deformations, brake forces, and rotation due to live loading. 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 state's bridge manuals, it is assumed that all wind and braking forces are to be resisted by the stiffness of the frame that is not accounted for in design.

The loads provided in Tables 5.1 and 5.2 were factored according to AASHTO LRFD Table 3.4.1-1. The governing Strength I load case resulted in factored axial loads of 1775 kips/abutment and 1767 kips/abutment resulting in a maximum axial loads equal to 254 kips per pile and 253 kips per pile for Abutment No. 1 and No. 2 respectively, for a seven pile layout.

5.2 Modeling: Due to the varying soil conditions on either side of the bridge, two soil profiles were developed and modeled in FB-Pier. Soil information from boring B-209 was used for Abutment No. 1 and soil information from B-210 was used for Abutment No. 2. The piles were analyzed for strength and service at both the non-scour and scour condition. A bottom of pile cap elevation of 1445 feet, provided in the revised preliminary plans dated July 2014, was used for both abutments in the analyses. Per general recommendations from the VTrans Hydraulics Section, a minimum scour depth of 6 feet below the thalweg was used in analyses, which resulted in a scour elevation of 1437.2 feet.

Because bedrock was not encountered in the borings, a 7 pile layout was chosen for analyses to reduce the axial load on each pile. This should allow for decreased pile lengths and a lower risk in construction with the piles being driven to lower resistances than if fewer piles were being used. A 6 pile layout was analyzed as well, and does satisfy design requirements, however we recommend the 7 pile layout to minimize required driving resistances.

Both abutments were modeled as having 11.4 foot high, 4 feet thick, and 47 feet long pile caps with 7 HP 12x63 piles spaced at 6 feet on center. All piles are assumed to be driven plumb and oriented for weak axis bending. Figure 5.1 below shows the pile layout for both abutments.

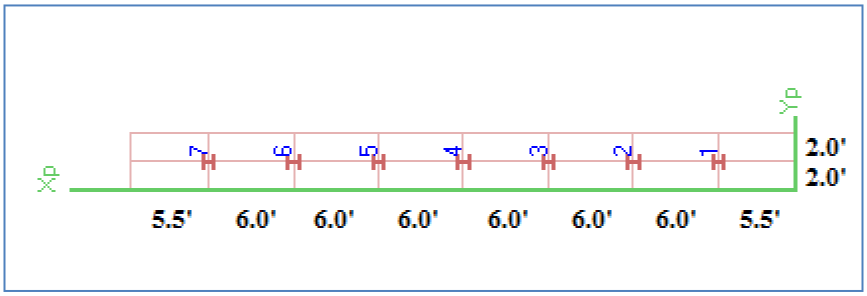


Figure 5.1: Abutment Pile Layout

6.0 RESULTS

6.1 Pile Stresses: Seven HP 12x63 piles were modeled for both the non-scour and scour condition. The piles were checked for combined axial compression and flexure under both non-scour and scour conditions using the requirements of AASHTO LRFD 6.9.2.2. An FB-Pier analysis was performed by applying an axial load, a deflection, and a rotation at the top of each pile under AASHTO LRFD Strength Case I. The output from FB-Pier 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 VTrans 2008 Integral Abutment Bridge Design Guidelines. A plastic hinge consistently formed in the top segment of the pile in the analysis run with the non-scour soil condition. This occurred when the applied moment exceeded the plastic moment. An analysis was then performed to ensure that a plastic hinge would not form in the second segment of the pile, which would overstress the pile and cause the pile to fail. The second segment of the pile was considered to be between the two points of zero moment when a fixed head condition was modeled. FB-Pier outputs as well as calculated values are displayed below in Table 6.1 and 6.2, for assumed 75 foot piles and 65 foot piles for Abutments No. 1 and No. 2 respectively.

Table 6.1: FB-Pier Output for AASHTO Strength Case I –Abutment 1

Soil Condition	Max. Applied Moment (kip-ft)	Plastic Moment** (kip-ft)	2nd Pile Segment Interaction	Factored Lateral Load (kips)	Unbraced Length (feet)	Fixity* (feet)
Non-Scour	106.0	95.7	0.53	22.1	8	19
Scour	66.8	102.4	0.43	8.16	11	21

** Moment resulting in plastic hinge development. This moment becomes constant at pile head after pile begins to plastically deform.
* Measured from top of pile head

Table 6.2: FB-Pier Output for AASHTO Strength Case I –Abutment 2

Soil Condition	Max. Applied Moment (kip-ft)	Plastic Moment** (kip-ft)	2nd Pile Segment Interaction	Factored Lateral Load (kips)	Unbraced Length (feet)	Fixity* (feet)
Non-Scour	101.1	95.7	0.47	20.6	9	16
Scour	69.6	102.7	0.44	8.9	11	23

** Moment resulting in plastic hinge development. This moment becomes constant at pile head after pile begins to plastically deform.
* Measured from top of pile head

As shown in Tables 6.1 and 6.2, in the scour condition, the maximum applied moment is less than the plastic moment calculated; therefore a plastic hinge does not develop in the top segment of the pile during scour condition. The factored lateral load in this table is the load applied to the top of the pile to achieve the required deflection times a load factor of 1.2.

6.2 Axial Capacity Analysis: To aid in estimating pile lengths, the minimum length needed to resist the factored design load based on dynamic testing needed to be calculated. This is assuming the pile could achieve sufficient resistance primarily in skin friction as bedrock was not encountered in the borings. With factored loads of 254 kips and 253 kips, and a resistance factor, ϕ_{dyn} = 0.65, nominal axial pile resistances of 391 and 389 kips are required for Abutments No. 1 and No. 2 respectively. To simplify things in construction, we recommend all piles be driven to a nominal axial resistance of 391 kips.

Using the Nordlund method for cohesionless soils, unit skin friction values were calculated for each soil layer. Based on these values, the lengths of piles needed to resist the 391 kip and 389 kip loads was calculated to be 65 feet and 52 feet, measured from the bottom of the pile cap for Abutments No. 1 and No.2 respectively. However, based on past experience with piles tending to run in similar soil conditions, we recommend pile lengths of 80 feet and 70 feet for Abutments No. 1 and No. 2 respectively, be used for estimating and plan preparation purposes.

The resistance factor of 0.65 requires a minimum of 2 dynamic tests performed per site condition, but no less than 2% of the production piles, during installation in accordance with Table 10.5.5.2.3-1 of the AASHTO LRFD code. No less than 1 test shall be performed at each abutment. The remaining piles should be calibrated by wave equation analysis.

6.3 Pile Cap Design: The backwall can be designed as a horizontal beam 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 distances of 10.1 feet and 9.8 feet from the bottom of the approach slab to the bottom of the pile cap for Abutments No. 1 and No. 2 respectively, and the abutment experiencing all of the lateral movement, then 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 abutment.

Equation 1: $K_p = (1 + \sin\phi)/(1 - \sin\phi)$

Equation 2: $w_p = {}^1/_2\,\gamma\, H^2\, K_p$

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 135 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 24.5 k/ft and 22.8 k/ft for Abutments No. 1 and No. 2 respectively.

6.4 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 varies significantly in grade with the existing site and as a result will require large amounts of fill. The proposed roadway will require up to 8.4 feet and 8.1 feet of fill to be placed behind Abutments No.1 and No. 2, respectively. A settlement analysis was performed and anticipated settlements of up to 2.5 inches were calculated at both Abutments. Assuming the full downdrag condition is met, where the downward movement of the settling soil acts on the pile, an additional axial load should be modeled acting on each pile. The settlement will induce axial loads of 26.9 kips and 27.2 kips per HP 12x63 pile for Abutments No. 1 and No. 2, respectively.

6.5 Driving Resistances: Past experience suggests that the HP 12x63 piles analyzed in this report could be driven through the soils encountered by pile-driving equipment commonly used by contractors in the region. Section 10.7.8 of the AASHTO LRFD Bridge Design Specifications stipulates that the maximum tension and compression stresses allowed in the piles shall not exceed $\sigma = 0.9 * \phi_{da} * f_y$. ϕ_{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. However, wave equation analyses only verify that the piles can be driven to a factored resistance; the program is not able to determine the location and size of boulders.

7.0 RECOMMENDATIONS

7.1 Integral Abutment Foundations: 7 HP 12x63 piles organized in a single row spaced at 6 feet center to center spacing will satisfy the requirements for design at both abutments. The piles are anticipated to be driven to a nominal axial resistance of 391 kips at both Abutments. The minimum required embedment for the piles is 20 feet below bottom of footing for both abutments. Pile lengths for estimating purposes should be assumed to be 80 feet and 70 feet below the bottom of footing for Abutments No. 1 and No. 2, respectively.

7.2 Construction Considerations:

7.2.1 Cofferdams/Temporary Earthwork Support: With the bottom of pile cap (Elevation 1445 ft) estimated to be located at or below ordinary high water (Elevation 1445.3 ft), cofferdams may be necessary. If required, the Contractor should be reminded that Section 208.07 of VTrans' *2011 Standard Specifications for Construction* indicates that "The Contractor shall prepare detailed plans and a schedule of operation for each cofferdam specified in the Contract. The design and structural details of the cofferdam shall be signed, stamped, and dated by a Professional Engineer (Structural or Civil)."

7.2.2 Construction Dewatering: Temporary construction dewatering may be required to construct the abutments. Temporary dewatering may also be necessary to limit disturbance to and maintain the integrity of the bearing surface. 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, or discharge, should comply with all applicable permits and regulations.

7.2.3 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, such as a small vibratory plate compactor, is used, 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 at least 95% of the maximum dry density determined in accordance with AASHTO T-99. The current specification calls for 90%, however we are in the process of revising it to be 95% as recommended above. 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.

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

7.3 Design Parameters: Table 7.1 highlights the geotechnical design parameters of the foundation bearing soil as well as regularly specified aggregates. 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 7.1 Engineering Properties for Construction and In-Situ Materials

	703.01A - Granular Borrow	**704.08 - Granular Backfill for Structures**	**B-209 In-Situ M. Dense SiSa**	**B-207 In-Situ Loose SiSa**
Density (lb/ft^3):	130	135	115	110
Internal Friction Angle, φ (degrees)	32	34	35	31
Coefficient of Friction, f				
- concrete cast against soil:	0.50	0.55	0.40	0.40
- soil against formed concrete	0.40	0.45	0.31	0.31
Active Earth Pressure Coefficient, K_a:	0.31	0.28	0.27	0.32
Passive Earth Pressure Coefficient, K_p:	3.25	3.53	3.69	3.12
At-Rest Earth Pressure Coefficient, K_o:	0.47	0.44	0.43	0.48

8.0 CONCLUSION

If any further analysis is needed or you would like to discuss this report, please contact us at (802) 828-2561. Computer generated boring logs are attached and available in the *M:\Projects\78d347\MaterialsResearch* folder. FB-Pier input file are located in the *M:\Projects\78d347\MaterialsResearch\FB-Pier* folder:

Abutment 1 STR I 7 Piles.IN
Abutment 1 STR I Scour 7 Piles.IN
Abutment 1 SER II 7 Piles.IN
Abutment 1 SER II Scour 7 Piles.IN

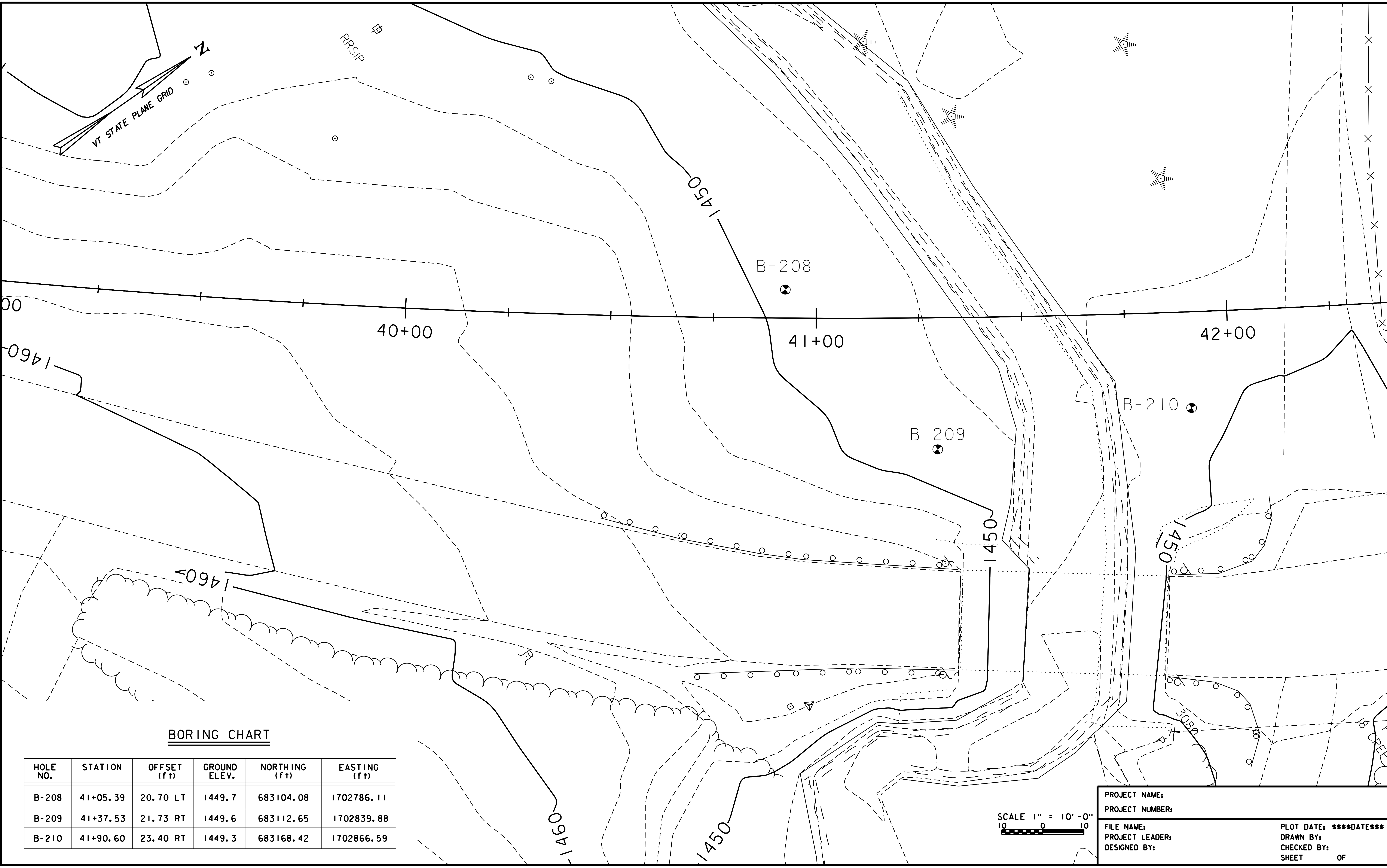
Abutment 2 STR I 7 Piles.IN
Abutment 2 STR I 7 Piles Scour.IN
Abutment 2 SER II 7 Piles.IN
Abutment 2 SER II 7 Piles Scour.IN

Attachments: Boring Location Plan (1 pages)
Boring Logs (5 pages)

cc: Erik Atkins, Green International
Mike Cruz, Green International

Electronic Read File/DJH
Project File/CEE
END

Z:\Highways\ConstructionMaterials\GeotechEngineering\Projects\Cabot-Danville FEGC-F 028-3(36) C2\REPORTS\Cabot-Danville FEGC-F 028-3(36) Integral Abutments BR 88.docx



BORING CHART

HOLE NO.	STATION	OFFSET (ft)	GROUND ELEV.	NORTHING (ft)	EASTING (ft)
B-208	41+05.39	20.70 LT	1449.7	683104.08	1702786.11
B-209	41+37.53	21.73 RT	1449.6	683112.65	1702839.88
B-210	41+90.60	23.40 RT	1449.3	683168.42	1702866.59

PROJECT NAME:
PROJECT NUMBER:

FILE NAME:
PROJECT LEADER:
DESIGNED BY:

PLOT DATE: $$$$DATE$$$
DRAWN BY:
CHECKED BY:
SHEET OF

STATE OF VERMONT AGENCY OF TRANSPORTATION CONSTRUCTION AND MATERIALS BUREAU CENTRAL LABORATORY	BORING LOG **CABOT-DANVILLE** **FEGC-F 028-3(26) C/2** **US-2 BR-87**	Boring No.: **B-209** Page No.: 1 of 2 Pin No.: 78d347 Checked By: END

Boring Crew: JUDKINS, GARROW, NIETO
Date Started: 3/02/16 Date Finished: 3/08/16
VTSPG NAD83: N 683112.65 ft E 1702839.88 ft
Station: 41+37.5 Offset: 21.73
Ground Elevation: 1449.6 ft

	Casing	Sampler
Type:	WB	SS
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/AWJ	
Rig:	CME 55 TRACK	C_E = 1.41

Groundwater Observations

Date	Depth (ft)	Notes
03/08/16	2.0	W.T. After Drilling
03/08/16	0.2	W.T. Before Drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		A-2-4, SiSa, brn, Moist, Rec. = 1.2 ft, Lab Note: Wood fibers and pieces of wood were within sample.	1-1-2-2 (3)	25.5	14.6	62.1	23.3
		A-2-4, SiSa, brn, Moist, Rec. = 1.4 ft	3-5-6-4 (11)	21.6	16.0	53.0	31.0
		A-2-4, GrSiSa, brn-blk, MTW, Rec. = 2.0 ft	7-5-6-11 (11)	22.4	20.4	54.4	25.2
		Visual Description:, GrSiSa, brn, Moist, Rec. = 0.3 ft, Lab Note: Insufficient sample size to perform gradation. Broken rock and plant material was within sample.	10-6-8-9 (14)	61.1			
		A-4, SaSi, gry, MTW, Rec. = 0.8 ft	4-11-13-14 (24)	19.7	13.3	22.2	64.5
10		A-4, GrSaSi, gry, MTW, Rec. = 0.6 ft	9-8-8-9 (16)	19.5	22.0	30.7	47.3
		A-4, SaSi, gry, Moist, Rec. = 1.0 ft	5-11-11-5 (22)	25.1	2.6	29.6	67.8
		A-4, Si, gry, MTW, Rec. = 1.3 ft	5-8-9-9 (17)	25.1	7.8	10.1	82.1
		A-4, Si, gry, MTW, Rec. = 1.3 ft	5-8-11-12 (19)	26.7	4.1	7.1	88.8
		A-4, Si, gry, MTW, Rec. = 1.5 ft	8-10-10-10 (20)	26.3	4.1	4.7	91.2
20		A-4, Si, gry, MTW, Rec. = 1.5 ft	4-8-9-9 (17)	28.0	5.7	5.8	88.5
		A-4, Si, gry, MTW, Rec. = 0.6 ft	9-10-10-12 (20)	28.1	11.3	15.4	73.3
		A-4, Si, gry, Moist, Rec. = 1.4 ft, Lab Note: A very small amount of clay was within sample. Sample tested non-plastic.	7-7-11-12 (18)	25.0	11.3	5.1	83.6
30		A-4, Si, gry, Moist, Rec. = 0.3 ft	5-18-23-21 (41)	15.8	6.2	16.3	77.5
		A-4, Si, gry-brn, MTW, Rec. = 1.8 ft	3-11-13-14 (24)	32.5	0.4	3.6	96.0
40		A-4, Si, gry, MTW, Rec. = 1.1 ft	3-12-13-15 (25)	34.5	3.0	4.7	92.3
		A-4, Si, gry, Moist, Rec. = 1.8 ft	3-13-15-16 (28)	30.6	1.5	5.5	93.0
50		A-4, Si, gry, Moist, Rec. = 1.8 ft	3-8-13-15 (21)	30.1	1.0	5.5	93.5

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 **CABOT-DANVILLE** **FEGC-F 028-3(26) C/2** **US-2 BR-87**	Boring No.: **B-209** Page No.: 2 of 2 Pin No.: 78d347 Checked By: END

Boring Crew: JUDKINS, GARROW, NIETO
Date Started: 3/02/16 Date Finished: 3/08/16
VTSPG NAD83: N 683112.65 ft E 1702839.88 ft
Station: 41+37.5 Offset: 21.73
Ground Elevation: 1449.6 ft

	Casing	Sampler
Type:	WB	SS
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/AWJ	
Rig:	CME 55 TRACK	C_E = 1.41

Groundwater Observations

Date	Depth (ft)	Notes
03/08/16	2.0	W.T. After Drilling
03/08/16	0.2	W.T. Before Drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		A-4, Si, gry, Moist, Rec. = 2.0 ft	5-9-13-16 (22)	31.1	0.4	3.6	96.0
60		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: Thin layers of clay were within sample. Sample tested non-plastic.	5-11-16-23 (27)	31.9		5.7	94.3
		A-4, Si, gry, Moist, Rec. = 1.8 ft	W.R.-4-12-16 (16)	33.7		4.8	95.2
70		A-4, Si, gry, Moist, Rec. = 1.8 ft, Lab Note: A small amount of clay was within sample. Sample tested non-plastic.	W.R.-4-11-15 (15)	32.5	0.1	4.0	95.9
80		A-2-4, SiSa, gry, Moist, Rec. = 2.0 ft	5-18-30-30 (48)	25.4		65.2	34.8
90		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: Sample tested non-plastic.	5-4-13-19 (17)	30.8	0.1	16.7	83.2
100		A-4, Si, gry, Moist, Rec. = 2.0 ft	5-10-14-19 (24)	30.6	0.1	4.2	95.7

Hole stopped @ 102.0 ft

Remarks:
Hole filled with bentonite and portland cement grout.

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
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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

CABOT-DANVILLE
FEGC-F 028-3(26) C/2
US-2 BR-87

Boring No.: **B-210**

Pin No.: 78d347
Checked By: END

Boring Crew: GARROW, NIETO
Date Started: 3/09/16 Date Finished: 3/22/16
VTSPG NAD83: N 683168.42 ft E 1702866.59 ft
Station: 41+90.6 Offset: 23.40
Ground Elevation: 1449.3 ft

	Casing	Sampler
Type:	WB	SS
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/AWJ	
Rig:	CME 55 TRACK	C_E = 1.41

Groundwater Observations

Date	Depth (ft)	Notes
03/14/16	3.2	W.T. Before Drilling
03/16/16	10.2	W.T. Before Drilling
03/22/16	5.5	W.T. Before Drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		A-1-b, GrSa, brn-gry, Moist, Rec. = 0.8 ft, Lab Note: Grass, grass roots, and broken rock were within sample.	32-R@3.5" (R)	16.2	39.6	44.5	15.9
		A-2-4, SiSa, gry, Moist, Rec. = 1.3 ft	2-3-3-2 (6)	33.3	3.2	61.7	35.1
		A-2-4, SiSa, gry-brn, Moist, Rec. = 1.4 ft, Lab Note: Pieces of wood and wood fibers were within sample.	1-1-1-1 (2)	43.2	10.5	61.2	28.3
		A-1-a, SaGr, gry-brn, Moist, Rec. = 0.5 ft, Lab Note: Pieces of wood and wood fibers were within sample.	3-12-9-5 (21)	21.4	49.8	43.4	6.8
		Field Note:, Cleaned out with NXDC					
10		Field Note:, Cobbles and Stones					
		Field Note:, Cleaned out with NXDC					
		A-4, GrSiSa, gry, Moist, Rec. = 1.0 ft	5-7-5-7 (12)	16.1	20.5	37.4	42.1
		A-4, SaSi, gry, Moist, Rec. = 1.5 ft	4-5-5-6 (10)	17.6	7.6	36.6	55.8
		A-4, SaSi, gry, Moist, Rec. = 1.8 ft	7-6-7-8 (13)	21.5	2.2	25.0	72.8
		A-4, SaSi, gry, Moist, Rec. = 1.6 ft	6-7-9-12 (16)	17.9	11.8	30.2	58.0
		A-4, SaSi, gry, Moist, Rec. = 1.5 ft	7-7-8-7 (15)	22.6	0.3	33.0	66.7
20		A-4, Si, gry, Moist, Rec. = 1.6 ft	7-8-11-15 (19)	25.9	0.6	4.6	94.8
		A-4, Si, gry, Moist, Rec. = 1.6 ft	14-12-11-18 (23)	23.2	0.1	15.0	84.9
		A-4, Si, gry, Moist, Rec. = 1.4 ft	12-14-14-21 (28)	26.1		9.8	90.2
30		A-4, Si, gry, Moist, Rec. = 1.5 ft	8-11-13-15 (24)	27.4	0.5	6.1	93.4
		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: A very small amount of clay was within sample. Sample tested non-plastic.	6-9-13-15 (22)	31.4	0.1	2.1	97.8
40		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: Sample tested non-plastic.	9-13-14-16 (27)	26.5	0.2	10.2	89.6
		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: Sample tested non-plastic.	7-12-15-23 (27)	30.1	0.3	5.9	93.8
50		A-4, Si, gry, Moist, Rec. = 2.0 ft	6-9-13-23 (22)	29.5	0.1	6.4	93.5
		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: A very small amount of clay was within sample. Sample tested non-plastic.	7-10-14-21 (24)	30.0	0.1	5.9	94.0
60		A-4, Si, gry, Moist, Rec. = 2.0 ft	8-14-20-27 (34)	31.4	0.2	4.5	95.3

Notes:
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VTrans Working to Get You There (Vermont Agency of Transportation)	STATE OF VERMONT AGENCY OF TRANSPORTATION CONSTRUCTION AND MATERIALS BUREAU CENTRAL LABORATORY	**BORING LOG** **CABOT-DANVILLE** **FEGC-F 028-3(26) C/2** **US-2 BR-87**	Boring No.: **B-210** Page No.: 2 of 3 Pin No.: 78d347 Checked By: END

Boring Crew: GARROW, NIETO
Date Started: 3/09/16 Date Finished: 3/22/16
VTSPG NAD83: N 683168.42 ft E 1702866.59 ft
Station: 41+90.6 Offset: 23.40
Ground Elevation: 1449.3 ft

	Casing	Sampler
Type:	WB	SS
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/AWJ	
Rig:	CME 55 TRACK	C_E = 1.41

Groundwater Observations

Date	Depth (ft)	Notes
03/14/16	3.2	W.T. Before Drilling
03/16/16	10.2	W.T. Before Drilling
03/22/16	5.5	W.T. Before Drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: Sample tested non-plastic.	8-13-15-19 (28)	31.0	0.1	5.7	94.2
70		A-4, Si, gry, Moist, Rec. = 1.9 ft, Lab Note: A very small amount of clay was within sample. Sample tested non-plastic.	9-13-19-24 (32)	30.3		3.8	96.2
80		A-4, Si, gry, Moist, Rec. = 1.7 ft	17-20-24-28 (44)	27.8	0.2	6.8	93.0
90		A-4, Si, gry, Moist, Rec. = 2.0 ft	13-14-23-32 (37)	29.4	0.1	10.3	89.6
100		A-4, Si, gry, Moist, Rec. = 1.9 ft, Lab Note: A very small amount of clay was within sample. Sample tested non-plastic.	13-14-18-24 (32)	29.5		4.4	95.6
110		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: Several thin layers of clay were within sample. Sample tested non-plastic.	10-15-19-29 (34)	29.8	0.1	2.5	97.4
120		A-4, Si, gry, Moist, Rec. = 1.9 ft, Lab Note: A very small amount of clay was within sample. Sample tested non-plastic.	9-13-17-24 (30)	29.4		3.2	96.8

Notes:
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STATE OF VERMONT AGENCY OF TRANSPORTATION CONSTRUCTION AND MATERIALS BUREAU CENTRAL LABORATORY	**BORING LOG** **CABOT-DANVILLE** **FEGC-F 028-3(26) C/2** **US-2 BR-87**

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Groundwater Observations

Date	Depth (ft)	Notes
03/14/16	3.2	W.T. Before Drilling
03/16/16	10.2	W.T. Before Drilling
03/22/16	5.5	W.T. Before Drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		A-4, Si, gry, Moist, Rec. = 2.0 ft, Lab Note: A very small amount of clay was within sample. Sample tested non-plastic.	9-12-14-19 (26)	31.5		3.8	96.2
140		A-4, Si, gry, Moist, Rec. = 1.9 ft, Lab Note: Sample tested non-plastic.	9-12-19-33 (31)	30.2		2.6	97.4
		Field Note:, No Recovery, Rec. = 0.0 ft	R@1.0"				
150		Field Note:, Attempted core runs for assumed bed rock 148.1-158.1, 3.0' recovery of boulder					
		Field Note:, Cleaned out with BXDC					
160		A-2-4, SiSa, gry, Moist, Rec. = 1.8 ft	2-1-3-2 (4)	22.7	0.5	67.7	31.8
170		A-4, Si, brn, Moist, Rec. = 0.8 ft	18-29-32-R@1.0" (61)	20.6	14.7	12.7	72.6
		Field Note:, Cleaned out with BXDC					
180		A-1-b, SaGr, brn, Moist, Rec. = 1.0 ft	9-34-35-R@1.0" (69)	14.8	53.2	28.1	18.7
		Hole stopped @ 181.6 ft					
190		Remarks: Hole Filled with bentonite and portland cement grout.					

Notes:
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