AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Gary Laroche, P.E., Structures Project Manager

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

Date: June 22nd, 2020

Subject: Springfield BF 0134(43) – Pile Foundation Recommendations

1.0 INTRODUCTION

As requested, we have completed our geotechnical and geological evaluation for the Springfield BF 0143(43) project located on VT Route 11 approximately 425 feet east of the intersection of VT Route 11 and TH-86 (Breezy Hill Rd) in the town of Springfield, Vermont. The proposed project consists of replacing the existing bridge (Bridge No. 57) over an unnamed brook with a three-sided frame supported on piles at each abutment. Contained herein are our results for our geotechnical analysis and recommendations for pile supported foundations, as determined using the 2017 AASHTO *LRFD Bridge Design Specifications.*

2.0 FIELD INVESTIGATION

A previous Subsurface Investigation Report summarizing our field investigation was submitted by Eric Denardo, dated September 26th, 2016. Refer to this report for a detailed description of the field sampling and testing, laboratory analysis of soil and rock samples, and boring logs.

3.0 ANALYSIS

Developed by the Florida Bridge Software Institute, FB-MultiPier, version 5.4, 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 analysis, only the piles and cap were modeled in FB-MultiPier.

3.1 Loads: Final unfactored loads were provided by the Structures Section in an email from Geoffrey Dargan dated June 27th, 2019. These loads were calculated for a three-sided rigid frame structure under both scour and non-scour conditions. The loads for are highlighted below in Table 3.1.1.

Following discussions with the Structures Section it was determined that the scour condition would not include any earth pressures acting on the structure, the assumption being that under the anticipated full scour condition no soil material would remain

around the structure and foundations. As a result, only dead load (DC) was considered in the scour analysis.

Table 3.1.1 Unfactored Loads per Abutment – Rigid Frame Structure

Load Type	Direction	Value
Dead Load, DC*	Vertical (z)	8.0 kip/ft
Wearing Load, DW	Vertical (z)	2.0 kip/ft
Vertical Earth Pressure, EV	Vertical (z)	11.0 kip/ft
Live Load, LL	Vertical (z)	5.0 kip/ft
Live Load Surcharge, LS	Horizontal (y)	1.0 kip/ft
Horizontal Earth Pressure, EH	Horizontal (y)	3.0 kip/ft

**Includes pile cap self-weight*

According to the loads provided in Table 3.1.1, and AASHTO LRFD Table 3.4.1-1, Limit State Strength I was determined to be the governing load case for both the non-scour and scour conditions. As a result, maximum factored axial loads, maximum factored lateral loads, and maximum factored moments distributed at each abutment were calculated. Maximum factored loads per pile at each abutment were then calculated based on a 12 pile layout per abutment. These loads are presented in Table 3.1.2.

Table 3.1.2 Maximum Factored Loads per Abutment and per Pile (Non-Scour Condition)

Abutments (East & West)	Total Distributed Load		Load Distributed per Pile (12 Pile Layout)	
	Axial (kips)	Lateral (kips)	Axial (kips)	Lateral (kips)
Strength I	3258.0	557.0	271.5	46.4
Service II	2448.0	383.0	204.0	31.9

3.2 Soil Profile: A subsurface investigation was performed between July 25 and July 27, 2016 and results from that investigation are included in the previous Geotechnical Report dated September 26, 2016. Information from that report was used to develop soil profiles for the pile foundations. Borings B-101, B-101A, and B-102 were used to develop the soil profiles and corresponding models in FB-MultiPier.

In boring B-101A, bedrock was encountered at a depth of 30.1 feet (ft) below the ground surface (bgs) corresponding to an approximate elevation of 537.8 ft. Groundwater was measured before drilling on July 26th at a depth of 16.6 ft bgs corresponding to an approximate elevation of 551.3 ft. In boring B-102, bedrock was encountered at a depth of 42.0 ft bgs corresponding to an approximate elevation of 527.9 ft. Groundwater was measured during drilling on July 27th at a depth of 12.1 ft bgs corresponding to an approximate elevation of 557.8 ft. The soil parameters used in the analysis for the East and West Abutments are displayed below in Tables 3.2.1 and

3.2.2, respectively. Rock parameters used for both abutments are summarized in Table 3.2.3.

Table 3.2.1 FB-MultiPier Analysis Soil Parameters – East Abutment (B-102)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)	Shear Modulus (ksi)	Torsional Shear Stress (psf)
553 - 554	Dense GrSa	36	130	125	2.50	169.0
554 - 528	V. Dense SaGr/GrSa	38	135	125	3.48	843.6
<528	Bedrock (Gneiss)	27	160	-	2552.8	-

Table 3.2.2 FB-MultiPier Analysis Soil Parameters – West Abutment 2 (B-101/101A)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)	Shear Modulus (ksi)	Torsional Shear Stress (psf)
553 - 548	Dense SaGr	36	130	125	1.38	93.9
548 - 544	Loose SiSa	31	115	20	0.672	155.3
544 - 538	V. Dense SaGr	38	135	125	3.48	531.4
<538	Bedrock (Schist)	27	160	-	2552.8	-

Table 3.2.3 FB-MultiPier Analysis Rock Parameters

Parameter	Value
Unconfined Compressive Strength (ksf)	669.6
Modulus of Elasticity (ksi)	4970
Poisson's Ratio	0.12
Shear Modulus (ksi)	2552.8

3.3 Modeling: The models were analyzed for strength and service loading combinations in both the scour and non-scour conditions. Both Abutments were modeled as having a 3 ft high, 3 ft wide, and 89 ft long pile cap, with 12 HP14x89 piles spaced at 7.25 ft on center. A bottom of pile cap elevation of 553.0 ft at both abutments was used in the analysis. Dimensions and elevations for the pile caps were provided by the Structures Section as part of the Geotechnical Services Request form submitted on June 27th, 2019.

All piles were assumed to be driven plumb and oriented for strong axis bending in a single row for each abutment. As the bedrock varies across the site between 15 and 25 feet below the pile cap elevation, several models were developed with varying pile lengths to check design requirements. Figure 3.3.1 below shows the pile layout for both Abutments.

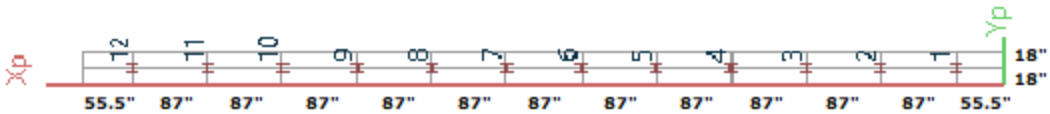

Figure 3.3.1 East and West Abutment Pile Layout

A scour depth of 3.15 feet during the Check Flood Event was provided in the hydraulics memo, corresponding to a scour elevation of 546.8 feet. This resulted in the piles being modeled as having 6.2 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 for either structure type.

4.0 RESULTS

Twelve (12) HP 14x89 piles were modeled for both the non-scour and scour conditions at each abutment. The models were designed for strength limit state and then evaluated for deflection in the service limit state.

The piles were analyzed similarly to how integral abutments are evaluated and were checked for combined axial compression and flexure under both the non-scour and scour conditions using the requirements of AASHTO LRFD 6.9.2.2. FB-Pier analyses were performed by applying loads and moments 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 2010 Integral Abutment Bridge Design Guidelines. An analysis was 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. Strength Case I was found to be the governing load case. For the strength limit state, the piles were found to be within acceptable stress limits. A summarized output of these analyses is available upon request.

For the non-scour service limit state, maximum deflections at the top of the piles at both Abutments were found to be 0.21 inches.

4.1 Nominal Axial Pile Resistance: The piles are assumed to be driven to and seated on bedrock. 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, Φ_{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 $R_R = \Phi_{dyn} * R_n$. The resistance factor, Φ_{dyn}, which should be applied to those piles bearing in either soil or on rock to attain the factored resistance, is 0.65. The use of 0.65 requires a minimum of 2 dynamic tests performed during installation in accordance with Table 10.5.5.2.3-1 of the AASHTO LRFD BDS. The remaining piles should be calibrated by wave equation analysis. No less than 1 test shall be performed at each abutment. Given the loads provided in Table 3.1.1, in order to accommodate either structure option, the

nominal axial pile resistance, or resistance the piles should be driven to, is 418 kips at both abutments.

4.2 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.

4.3 Driving Resistances: Past experience suggests that the HP 14x89 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 BDS 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.

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

5.0 RECOMMENDATIONS

5.1 Pile Cap Foundations: 12 HP14x89 piles organized in a single row spaced at 7.25 ft center to center spacing at each abutment will satisfy the requirements for design. The piles are anticipated to be driven to a nominal axial resistance of 418 kips at both abutments. The minimum required embedment for the piles is 15 ft for piles at both Abutments. Pile lengths are expected to range between 15 and 25 feet, therefore, pile quantities should be calculated assuming pile lengths of 25 feet for both abutments, which will ensure enough piling is planned for the project.

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' *2018 Standard Specifications for Construction* indicates that "The Contractor shall prepare detailed plans and a schedule of operation for each cofferdam specified in the Contract. Construction drawings shall be submitted in accordance with Section 105 (Control of the Work)".

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 5.3.1. 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 5.3.1: Engineering Properties of Construction Materials

	703.04 – Granular Borrow	**704.08 – Granular Backfill for Structures**
Unit Weight, γ (lbs/ft^3):	130	140
Internal Friction Angle, ϕ (degrees):	32	34
Coefficient of Friction, f		
- mass concrete cast against soil:	0.45	0.55
- soil against precast/formed concrete:	0.40	0.48
Active Earth Pressure Coef., K_a:	0.31	0.28
Passive Earth Pressure Coef., K_p:	3.26	3.57
At-Rest Earth Pressure Coefficient, K_o:	0.47	0.44

6.0 CONCLUSION

If any further analysis is needed or if you would like to discuss this report, please contact me at (802) 595-4589.

cc: Electronic Read File/MG
Project File/CEE

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