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(39) – Integral Abutment Recommendations

1.0 INTRODUCTION

As requested, we have completed our geotechnical and geological analyses for the subject project involving the replacement of Bridge 48 located on Stowe TH 43 (Nebraska Valley Road). Bridge 48 is located over the Miller Brook just west of the intersection with Stowe TH 47(Sugar Bush Ln) in Stowe, Vermont. The subject project consists of replacing the existing structure with a new bridge founded on pile supported integral abutments. Contained herein are the results of our geotechnical and geological analysis and recommendations for pile supported integral abutments as determined using the 2020 AASHTO *LRFD Bridge Design Specifications.*

2.0 FIELD INVESTIGATION

A field investigation was conducted between May 17 and 19, 2021. A report submitted by Sanborn Head & Associates Inc. (Sanborn), dated August 2021 summarizes the subsurface investigation and findings. Information taken from that report was used to estimate the soil and rock material parameters used in these analyses.

3.0 ANALYSIS

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: Final unfactored loads were provided by the Structures Section as part of the Geotechnical Request Form, submitted by Cory Burrall, dated February 16, 2024. The unfactored loads used in the analysis can be found in Table 3.1.1 and 3.1.2 for Abutments 1 and 2, respectively.

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-MultiPier 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 the braking forces, acting longitudinally, are to be resisted by the stiffness of the integral structure that is not accounted for in design.

Table 3.1.1 Unfactored Loads Abutment 1

Load Type	Load	Value	Elevation(ft)	Direction
Superstructure Dead Load*	DC	372.90 kips	741.00	Vertical
Superstructure Super Dead	DW	36.40 kips	741.00	Vertical
Vehicular Live Load	LL	158.24 kips	741.00	Vertical
Vehicular Centrifugal	CE	11.03 kips	758.96	Transverse
Wind on Structure	WS	1.59 kips	752.96	Longitudinal
		6.37 kips		Transverse
Wind on Live Load	WL	0.98 kips	758.96	Longitudinal
		2.46 kips		Transverse
Thermal Contraction	TU, Δ_t	0.225 in	741.00	Longitudinal

**Includes pile cap self-weight*

Table 3.1.2 Unfactored Loads Abutment 2

Load Type	Load	Value	Elevation(ft)	Direction
Superstructure Dead Load*	DC	344.20 kips	742.00	Vertical
Superstructure Super Dead	DW	36.40 kips	742.00	Vertical
Vehicular Live Load	LL	158.24 kips	742.00	Vertical
Vehicular Centrifugal	CE	11.03 kips	759.60	Transverse
Wind on Structure	WS	1.59 kips	753.60	Longitudinal
		6.37 kips		Transverse
Wind on Live Load	WL	0.98 kips	759.60	Longitudinal
		2.46 kips		Transverse
Thermal Contraction	TU, Δ_t	0.225 in	742.00	Longitudinal

**Includes pile cap self-weight*

The abutments were analyzed for both the scour and non-scour conditions. A scour elevation of 736.5 feet(ft) for both Abutments was provided in the Load and Deformations document provided by the Structures Section dated February 16, 2024.

According to the loads provided in Tables 3.1.1 and 3.1.2 and AASHTO LRFD Bridge Design Specifications Table 3.4.1-1, Limit State Strength I, Strength III, and Strength V were analyzed. It was determined that conditions from Strength V governed for both non-scour and scour conditions. The maximum factored axial load was determined from Strength I, with values of 797.6 kips and 761.8 kips which would be distributed over Abutment 1 and Abutment 2, respectively, resulting in maximum factored axial loads equal to 199.4 and 190.4 kips per pile for 4 pile layouts at Abutment 1 and 2, respectively.

3.2 Soil Profile: Results from the subsurface investigation completed in May 2021 were used to develop the soil profile for each abutment and corresponding models in

FB-MultiPier. B-102 was used for Abutment 1, and B-103 and B-104 were used for Abutment 2.

In boring B-102, bedrock was encountered at a depth of 54 ft below the ground surface (bgs) corresponding to an approximate elevation of 698 ft. Groundwater was measured after drilling operations on May 17, 2021, at a depth of 8.0 ft bgs, corresponding to an approximate elevation of 744 ft.

In boring B-104, bedrock was encountered at a depth of 44 ft bgs, corresponding to an elevation of 709.69 ft. Bedrock was not confirmed with a core in B-103. Based on the notes from the drillers, bedrock was presumed at a depth of 43.5 ft bgs for B-103 corresponding to an elevation of 710.19 ft, which was used in the models for Abutment 2. Groundwater was encountered in B-103 and B-104 at depths of 4 and 8 ft bgs, corresponding to approximate elevations of 749.69 and 745.69 ft, respectively.

The soil parameters used in the analysis for Abutment 1 and Abutment 2 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. It should be noted that the torsional shear stress values are skin friction values for that given layer of soil or rock.

Table 3.2.1 FB-MultiPier Analysis Soil Parameters – Abutment 1 (B-102)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)	Shear Modulus (ksi)	Torsional Shear Stress (psf)
741 – 735	V. Dense GRAVEL, some Sa, some Si	38	130	125	3.48	961.0
735– 725	Dense SAND, some Gr, little Si	38	120	125	3.04	1515.3
725-698	SAND, trace Gr, trace Si	38	135	125	3.48	2539.0
< 698	Bedrock	30	169	-	3053.3	-

Table 3.2.2 FB-MultiPier Analysis Soil Parameters – Abutment 2 (B-103, B-104)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)	Shear Modulus (ksi)	Torsional Shear Stress (psf)
742 – 741	Loose SAND and Gravel, tr Si	32	110	20	0.75	273.3
741– 736	Very Dense SAND, some Gr, trace Si	38	130	125	3.48	758.8
736-726	Medium Dense SILT and Sand	33	110	60	1.24	649.6
726-710	Dense SAND and Gravel, trace Silt	38	135	125	3.48	1659.4
< 710	Bedrock	30	169	-	3053.3	-

Table 3.2.3 FB-MultiPier Analysis Rock Parameters

Parameter	Value
Unconfined Compressive Strength (ksf)	606.7
Modulus of Elasticity (ksi)	6839.4
Poisson's Ratio	0.12
Shear Modulus (ksi)	3053.3

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 8.35 ft high, 3.0 ft wide, and 37.5 ft long pile cap, with 4 HP 12x63 piles spaced at 8 ft 3 ¼ in. on center. Abutment 2 was modeled as having a 7.99 ft high, 3.0 ft wide, and 35.9 ft long pile cap, with 4 HP 12x63 piles spaced at 8 ft 3 ¼ in. on center. Bottom of pile cap elevations of 741.0 ft and 742.0 ft were used in the analysis for Abutments 1 and 2, respectively. Dimensions and elevations for the pile caps and pile spacing were taken from the Final Plans dated February 2024.

All piles were assumed to be driven plumb and oriented for weak axis bending in a single row for each abutment. The piles were modeled as 42.6 ft long and 31.8 ft long driven piles extending to and bearing on competent bedrock for Abutments 1 and 2, respectively. Figures 3.3.1 and 3.3.2 below show the pile layouts for Abutments 1 and 2, respectively.

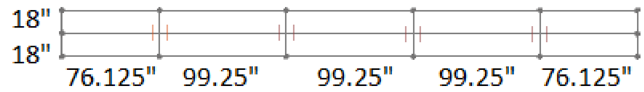

Figure 3.3.1 Abutment 1 Pile Layout

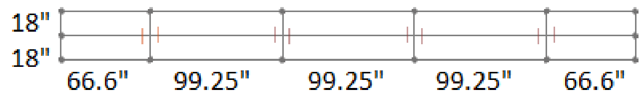

Figure 3.3.2 Abutment 2 Pile Layout

A scour elevation of 736.5 ft was provided by the Structures Section resulting in the piles being modeled as having 4.5 ft and 5.5 ft of free-standing length in the scour condition for Abutments 1 and 2, respectively. 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 Pile Stresses: Four HP 12x63 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 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 and 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-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 the VTrans 2008 Integral Abutment Bridge Design Guidelines. FB-MultiPier outputs as well as calculated values are displayed below in Table 4.1.1.

Table 4.1.1 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	34.4	0.33	11.6	10.0	16.0
1	Scour	19.0	0.37	4.1	14.9	14.7
2	Non-Scour	32.3	0.33	13.1	10.5	15.9
2	Scour	20.6	0.36	4.0	18.2	17.8

* 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 for both abutments; 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 4 piles spaced at 8 ft 3 ¼ in on-center as shown in Figures 3.3.1 and 3.3.2 for Abutments 1 and 2, respectively. The pile size needed to satisfy design requirements was found to be HP 12x63 piles. 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 0.47 and 0.56 in in the scour condition for Abutments 1 and 2, respectively.

4.2 Driving Resistances: Soil conditions were evaluated in all of the borings to determine the drivability of H-piles through the materials to reach minimum tip elevation. Past experience and information collected suggest that the HP 12x63 piles can be driven through the soils encountered at both abutments by pile-driving equipment commonly used by contractors in the region. Difficult drilling and possible weathered rock was encountered approximately 10 ft above bedrock in B-102 corresponding to an approximate elevation of 708.44. This elevation corresponds to approximately 32.5 ft below the bottom of footing for Abutment 1. <u>The minimum pile embedment for both Abutments was determined to be 25 ft below the bottom of footing corresponding to an approximate elevation of 716.0 and 717.0 ft for Abutments 1 and 2, respectively.</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 $\sigma = 0.9 * \phi_{da} * f_y$. 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.

4.3 Nominal Axial Pile Resistance: The piles are assumed to be driven to and seated on very dense material or bedrock. All of the required axial capacity will be generated from the end bearing of the piles likely 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, with a minimum of one per abutment, 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. Given the loads provided in Tables 3.1.1 and 3.1.2, the nominal axial pile resistance, or resistance the piles should be driven to, is 320 kips at both abutments.

4.4 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 an average distance of 8.35 feet from the bottom of the bridge seat to the bottom of the pile cap for Abutment 1 and a distance of 7.99 ft for Abutment 2, and the abutments 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 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 17.3 k/ft for Abutment 1 and 15.8 k/ft for Abutment 2.

4.5 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.

4.6 Settlement Analysis: Settlement of the abutment is anticipated to be negligible due to the piles being driven to very dense material or 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 The following table provides a summary of the requirements for the piles at Abutment 1 and 2.

Table 2. Summary of requirements of H-piles at each abutment

Requirement	Abutment 1 and 2
Pile Size	HP 12x63
Number of Piles per Abutment	4
Pile Spacing	99.25 in OC
Minimum Pile Embedment*	25 ft
Method of Installation	Driven pile
Nominal Axial Pile Resistance	320 kips

*Length of pile below bottom of pile cap.

Pile lengths below the bottom of pile cap for <u>estimating purposes</u> should be assumed to be <u>45 ft for Abutment 1 and 35 ft for Abutment 2.</u>

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 5.3.1. 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	3.54
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 us by email. Final FB-MultiPier input files used in the analyses are located in the *M:\Projects\12j658\MaterialsResearch\FB-Multipier* folder on the M/drive.

Abutment 1 STR I.in
Abutment 1 STR III.in
Abutment 1 STR V.in
Abutment 1 STR I Scour.in
Abutment 1 STR III Scour.in
Abutment 1 STR V Scour.in
Abutment 1 SER I.in
Abutment 1 SER II.in
Abutment 1 SER I Scour.in
Abutment 1 SER II Scour.in

Abutment 2 STR I.in
Abutment 2 STR III.in
Abutment 2 STR V.in
Abutment 2 STR I Scour.in
Abutment 2 STR III Scour.in
Abutment 2 STR V Scour.in
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