AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Mark Sargent, Project Manager, Structures

MLM CEE

From: Marcy Meyers, Geotechnical Engineer via Callie Ewald, P.E., Senior Geotechnical Engineer

Date: September 11th, 2014

Subject: Calais BHF 037-2(10) – Bridge No. 74 Integral Abutments

1.0 INTRODUCTION

Per your request we have completed our geotechnical evaluation for the proposed project in Calais, VT. Located on VT Route 14 crossing over the Pekin Brook, the proposed project includes the removal of the existing Bridge No. 74 and replacing it with a 62.5 foot single span NEXT Beam Bridge with associated roadway and channel work. A previous report by Terracon dated May 1st, 2014, summarizes the results of the subsurface investigation as well as subsurface conditions. Contained herein are the results from our geotechnical analysis and recommendations for integral abutments supported on piles as determined using the 2012 AASHTO *LRFD Bridge Design Specifications.*

2.0 FIELD INVESTIGATION

Terracon conducted the initial field investigation from March 10th through March 19th, 2014 using Drilex Environmental of West Boylston, MA. Boring B-2 was then redrilled on April 9th and 10th, 2014 using Crawford Drilling of Gardner, MA to obtain a confirmatory bedrock core. Several samples from this investigation were brought back to the VTrans Materials Laboratory for classification and testing. A geotechnical report, authored by Anant Panwalkar, P.E., of Terracon dated May 1st, 2014 documents the field investigation. Information regarding the subsurface conditions and laboratory results can be found in that report.

3.0 ANALYSIS

Developed by the Florida Bridge Software Institute, FB-Multipier, version 4.18.1, 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.

3.1 Loads: Unfactored loads were provided by Adam Stockin, P.E. of Parsons Brinckerhoff (PB) in an email dated July 31st, 2014. 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, 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 standing alone. Axial loads, deflection, and rotation due to live load were applied in this analysis.

Table 3.1. Vertical Loads Provided by PB

Type	Vertical Unfactored Loads/Pile*
Dead Loads, DC	103 kips
Wearing Loads, DW	12.63 kips
Live Loads, LL+IM	54.1 kips

*Note: Loads were provided per pile assuming 4 piles per abutment.

According to loads provided in Table 3.1 and AASHTO LRFD Table 3.4.1-1 Strength I Limit State, a load of 242.37 kips (not including the abutment weight) would be distributed over each pile assuming a four pile layout. The self weight of the pile cap was accounted for in FB-Pier with a load factor of 1.25, which corresponds to the AASHTO LRFD Strength Case I, DC load factor.

A total expected one-way longitudinal movement of 0.479 inches per pile, provided by PB, was used in the analysis. A live load rotation was not provided, therefore a value of 0.01 radians was assumed for the analysis. This value was used based on previous recommendations from the VTrans Structures Section on bridges of similar dimensions.

3.2 Modeling: Because the soils encountered in Borings B-1 and B-2 were similar, one soil profile was developed and modeled in FB-Pier. The piles were analyzed at both non-scour and scour conditions. A bottom of pile cap elevation of 708 feet for both abutments was taken from the Revised Preliminary Plans dated June 2014. The final hydraulics memo dated April 3rd, 2014 noted piles should be freestanding to a depth of at least 11 feet below the streambed during scour. Based on a streambed elevation of 708 feet, a scour elevation of 697 feet was used in the scour analysis model. The soil and rock parameters used in the analysis are displayed below in Tables 3.2 and 3.3, respectively.

Table 3.2. FB-Pier Analysis Soil Parameters

Elevation (feet)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)	Shear Modulus (ksi)	Torsional Shear Stress (psf)
708 - 704	SiSa	31	105	20	0.91	358
704 – 598.5	Si	30	105	20	0.91	1080
< 598.5	Rock	27	150	---	750	---

Table 3.3. Parameters Used for Bedrock in FB-Pier Analyses

Parameter	Value
Unconfined Compressive Strength (ksf)	734
Modulus of Elasticity (ksi)	1,710
Poisson's Ratio	0.14
Shear modulus (ksi)	750

The abutments were modeled as four H-piles spaced 9.3 feet on center as shown in Figure 3.1. Dimensions and elevations for the pile cap were taken from the Revised Preliminary Plans. The bottom of cap elevation was estimated at 708 feet. The piles were modeled as 109.5 feet long seated on bedrock.

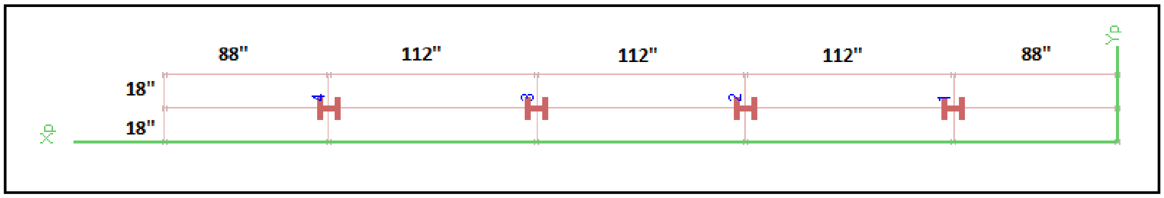

Figure 3.1. Pile Layout Analyzed (values in inches)

4.0 RESULTS

4.1 Pile Stresses: Four HP 12x84 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 4.1.

Table 4.1. FB-Pier Output for AASHTO Strength Case I

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	190.4	157.9	0.58	33.6	7.0	34.2
Scour	89.8	134.7	0.65	8.4	13.0	37.3

** 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 Table 4.1, 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.

4.2 Axial Capacity Analysis: These 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 pile on rock. Based upon the AASHTO code, the nominal structural resistance of the piles is calculated as $P_n = 0.66^{\lambda} * F_y * A_{pile}$. Per the Agency's Integral Abutment Design Guide this equation reduces to $P_n = C^* F_y * A_{pile}$, where C is assumed to be equal to 0.8.

4.3 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 a distance of 10 feet from the bottom of the approach slab to the bottom of the pile cap, 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 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 24.8 k/ft.

4.4 Settlement Analysis: Settlement of the abutment is not anticipated due to seating of piles on bedrock. Any settlement that does occur should be caused by the elasticity in the piles, which should occur as the piles are loaded. Due to the granular nature of the soil, any settlement occurring in the approaches due to the fill placement should occur during construction.

4.5 Downdrag Analysis: Negative skin friction, or down drag, is considered when the relative settlement between the pile and soil equals or exceeds 0.5 inches. The proposed roadway does not vary from the existing roadway. As a result, downdrag is not of concern and no additional forces were added onto the piles.

4.6 Driving Resistances: Past experience suggests that the HP 12x84 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.

4.7 Nominal Axial Pile Resistance: The nominal bearing resistance, R_R, should be factored using the resistance factors, ϕ_{dyn}, in Table 10.5.5.2.3-1 of the AASHTO LRFD code. 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 these 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 3 dynamic tests performed during installation in accordance with Table 10.5.5.2.3-3 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. Given the loads provided in Table 3.1 and the addition of the substructure self weight, the nominal axial pile resistance, or resistance the piles should be driven to, is 454 kips.

4.8 Roadway/Embankment Design: No geotechnical problems within the project limits are expected assuming standard Agency construction practices are utilized.

5.0 RECOMMENDATIONS

5.1 Integral Abutment Foundations: 4 HP 12x84 piles organized in a single row spaced at 9.3 feet center to center spacing will satisfy the requirements for design. The piles are anticipated to be driven to bedrock given the loose overburden soils, at an estimated length of 110 feet. The minimum required embedment for the piles is 40 feet below bottom of footing for both abutments.

5.2 Construction Considerations:

5.2.1 Cofferdams/Temporary Earthwork Support: With the bottom of pile cap (Elevation 708 ft) located below ordinary high water (Elevation 710.6 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)."

5.2.2 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.

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, 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 90% of the maximum dry density determined 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 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: Table 5.1 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 5.1 Engineering Properties for Construction and In-Situ Materials

	703.01A - Granular Borrow	**704.08 - Granular Backfill for Structures**	**In-Situ Loose SiSa**
Density (lb/ft^3):	130	140	105
Internal Friction Angle, φ (degrees)	32	34	31
Coefficient of Friction, f			
- concrete cast against soil:	0.50	0.55	0.40
- soil against formed concrete	0.40	0.45	0.31
Active Earth Pressure Coefficient, K_a:	0.31	0.28	0.32
Passive Earth Pressure Coefficient, K_p:	3.25	3.53	3.12
At-Rest Earth Pressure Coefficient, K_o:	0.47	0.44	0.48

6.0 CONCLUSION

If any further analysis is needed or you would like to discuss this report, please contact us at (802) 828-2561. FB-Pier input file are located in the M:\Projects\12b144\MaterialsResearch\FB-Pier folder:

Non-scour.in

Scour.in

c: Electronic Read File/DJH
Project File/CEE
MLM