AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Mark Sargent, Project Manager, Structures
MLM

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

Date: June 5th, 2015

Subject: Calais BHF 037-2(11) – Bridge No. 82 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 Kinsbury Branch, the proposed project includes the removal of the existing Bridge No. 82 and replacing it with a 57.7 foot single span NEXT Beam Bridge with associated roadway and channel work. A previous report by Terracon dated June 13th, 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 2014 AASHTO *LRFD Bridge Design Specifications.*

2.0 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.

2.1 Loads: Unfactored loads and their respective offsets and elevations were provided in an email by Adam Stockin, P.E., of Parsons Brinckerhoff (PB) on April 2nd, 2015 and can be found in Table 2.1. Loads were provided per pile assuming a four pile layout.

Table 2.1: Unfactored Loads

Type	Unfactored Loads/Pile	Load Orientation
Dead Loads, DC	99.08 kips	Vertical
Wearing Loads, DW	11.13 kips	Vertical
Live Loads, LL	95.02 kips	Vertical
Wind Lateral to Bridge, WS1	2.50 kips	Lateral
Wind at Skew to Bridge, WS2	1.27 kips	Lateral
Wind at Skew to Bridge, WS2	1.84 kips	Longitudinal
Wind on Vehicles, WL	0.71 kips	Lateral
Wind on Vehicles, WL	0.32 kips	Longitudinal
Braking Force, BR	6.11 kips	Longitudinal
Water Loads, WA	5.37 kips	Lateral

The DC load includes the weight of the pile cap. The wind on structure (WS) loads were provided as two non-concurrent conditions as the wind applied laterally to the structure (WS1), and as the wind applied at a skew to the structure (WS2).

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 were factored according to AASHTO's 2014 LRFD Bridge Design Specifications, Section 3.4.1 and the Strength I governing load case was used in the final analysis. A passive pressure of 14.9 kips/ft along the abutment length as well as the stiffness provided by the entire bridge structure were assumed to resist the longitudinal braking force imposed on the NEXT Beams. As a result, the longitudinal braking loads were neglected in the design of the piles, meaning only the vertical, and lateral loads listed above in Table 2.1, factored according to Strength I, were modeled in the final design. This resulted in a factored axial load of 306.8 kips/pile assuming a four pile layout.

A total expected thermal movement of 0.23 inches per pile was provided as a combination of both thermal movement and shrinkage movement in an email from PB dated February 19th, 2015. A live load rotation of 0.01 radians per abutment was assumed for the analysis.

2.2 Modeling: Due to the similarity of soil conditions in Borings B-1 and B-2, one soil profile was developed and modeled in FB-Pier. The piles were analyzed at both the non-scour and scour condition. A bottom of pile elevation of 893.43 feet, provided in the email with the loads, was used in the analysis. A contraction scour depth of 4 feet below the thalweg at the Q100 storm was provided in the final Hydraulics memo dated August 4, 2014. However, per general recommendations from the VTrans Hyrdaulics Section, a minimum scour depth of 6 feet below the thalweg was used, which resulted in a scour elevation of 885.5 feet used in our analysis.

The abutments were modeled as having a 7.89 foot high, 3.5 foot thick, and 42'-11.5" long pile cap, with 4 HP 12x84 piles spaced at 9.094 feet on center, as provided in an email from PB dated May 4th, 2015. All piles are assumed to be driven plumb and oriented for weak axis bending. Figure 2.1 below shows the pile layout for both abutments.

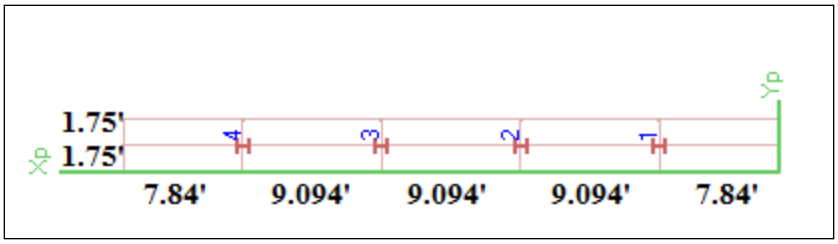

Figure 2.1: Abutment Pile Layout

3.0 RESULTS

3.1 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, lateral load and corresponding moment, 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 3.1 for an assumed 100 foot pile.

Table 3.1: 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	167.4	151.2	0.57	29.5	8.3	25.0
Scour	105.7	132.9	0.57	11.8	12.4	32.5

** 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 3.1, 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.

3.2 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 a factored load of 307 kips, and a resistance factor, ϕ_{dyn} = 0.65, a nominal axial pile resistance of 473 kips is required.

Using the Nordlund method for cohesionless soils, unit skin friction values were calculated for each soil layer. Based on these values, the length of pile needed to resist the 473 kip load was calculated to be 62 feet, measured from the bottom of the pile cap. However, based on past experience with piles tending to run in similar soil conditions, we recommend pile lengths of 85 feet 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.

3.3 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 7.89 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 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 14.9 k/ft.

3.4 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 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.

3.5 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.

4.0 RECOMMENDATIONS

4.1 Integral Abutment Foundations: 4 HP 12x84 piles organized in a single row spaced at 9.094 feet center to center spacing will satisfy the requirements for design. The piles are anticipated to be driven to a nominal axial resistance of 473 kips. The minimum required embedment for the piles is 25 feet below bottom of footing for both abutments. Pile lengths for estimating purposes should be assumed to be 85 feet below the bottom of footing for both abutments.

4.2 Construction Considerations:

4.2.1 Cofferdams/Temporary Earthwork Support: With the bottom of pile cap (Elevation 893.4 ft) estimated to be located below ordinary high water (Elevation 894.5 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)."

4.2.2 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.

4.2.3 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.

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

4.3 Design Parameters: Table 4.1 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 4.1 Engineering Properties for Construction and In-Situ Materials

	703.01A - Granular Borrow	**704.08 - Granular Backfill for Structures**	**In-Situ Loose SiGrSa**
Density (lb/ft^3):	130	135	105
Internal Friction Angle, φ (degrees)	32	34	31
Coefficient of Friction, f			
- concrete cast against soil:	0.50	0.55	0.35
- 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

5.0 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\12b146\MaterialsResearch\FB-Pier folder:

Non-scour.in

Scour.in

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

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