To: Danny Landry, Structures Project Manager

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From: Eric Denardo, Geotechnical Engineer via Marcy Montague P.E. Senior Geotechnical Engineer

Date: June 30, 2017 **- Revised: July 11, 2017**

Subject: West Haven BO 1443(51) – Integral Abutment Addendum-Updated Pile Layout

1.0 INTRODUCTION

We have completed additional geotechnical analyses for the replacement of Bridge 10 over the Poultney River located on TH 3 (Brook Rd)/NY Route 10 on the border between Vermont and New York in West Haven, Vermont. Previous geotechnical recommendations for integral abutments supported on piles were provided to the Structures Section by Terracon in a Geotechnical Report dated October 11, 2016. This memo documents our geotechnical analysis and recommendations based on updated loads and pile layout according to the 2016 AASHTO LRFD Bridge Design Specifications.

2.0 ANALYSIS

Updated unfactored final loads were provided by Mike Chenette of Stantec in an email dated May 19, 2017. The superstructure type was modified from prefabricated bridge units (PBU) to steel girders with a cast in place deck during design, therefore the loads were revised and required additional analysis. The unfactored loads provided in Table 2.1 were applied in these analyses. The loads are distributed over a 25.33 foot long abutment.

Table 2.1 Unfactored Loads/Rotations Provided by Stantec

Type	Load Type	Value	Load
Dead Loads	DC	18.4 kips/ft	253.3 k
Wearing Loads	DW	2 kips/ft	50.7 k
Live Loads	LL	8.5 kips/ft	215.3 k
Vehicular Braking	BR	0.9 kips/ft	22.8 k
Live Load Surcharge	LS	1.2 kips/ft	30.4 k
Horizontal Earth Pressure	EH	2 kips/ft	50.7 k
Thermal Movement	TU	0.6”	
Live Load Rotation	LL	-0.01 Radians*	

*The rotation was assumed per VTrans’ typical practice

It was requested by Stantec to determine if a 4 pile layout was appropriate for the updated loads to correspond with the reduction in the number of girders from 6 to 4. According to loads provided in Table 2.1 and AASHTO LRFD Table 3.4.1-1 Limit State Strength I, a load of 1035 kips would be distributed over each abutment resulting in a maximum axial load equal to 259 kips per pile for a four pile layout.

The piles were analyzed for both the scour and non-scour conditions. In the scour condition, it was assumed that there would be 23 feet of freestanding pile and thus no live loading on the bridge. This assumption has been made for past projects where the pile freestanding length during scour conditions is similar to that of this project. As a result, only dead load (DC), wearing load (DW), thermal loads (TU), and rotations were considered in the scour analysis.

The analyses were performed with a software program called FB-Multipier, developed by the Florida Bridge Software Institute. FB-Multipier (version 5.1.1) is a multi-aspect software that factors in the subsurface strata, the response of the pile group including cap, and the structural capabilities of the abutment system.

Models were created to analyze various load combinations for the abutment. The models used soil strata information as provided in the Geotechnical Engineering Report, dated October 11, 2016 provided by Terracon, as determined from borings B-101 and B-102A. Due to the similarity in the soils, one soil profile was used for both abutments. A bottom of pile cap elevation of 111 feet was used in the analyses. The models were analyzed for strength and service loading combinations in both non-scour and scour conditions.

The abutment was modeled with 4 HP 12 x 84 piles with the piles spaced at 7'2" on center. All piles are assumed to be driven plumb and oriented for strong axis bending. Figure 2.1 below shows the pile layout.

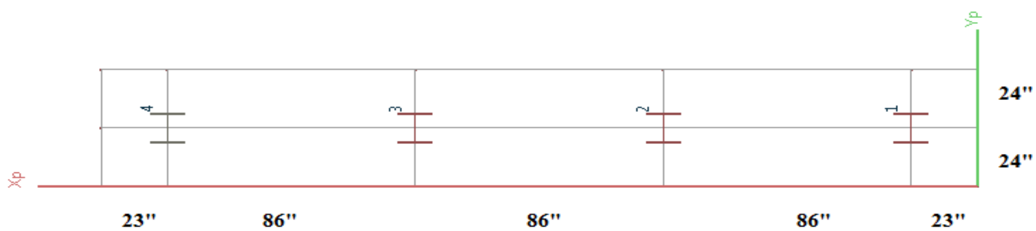

Figure 2.1: Abutment Pile Layout

3.0 RESULTS & RECOMMENDATIONS

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 the 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 3.1 for assumed 100 foot piles.

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	553.6	387.8	0.55	101.4	8.3	43.8
Scour	183.2	422.3	0.35	14.2	21.5	48.7

* 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 conditions. 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.

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 259 kips and a resistance factor, ϕdyn = 0.65, a nominal axial pile resistance of 399 kips is required.

Using the Nordlund method for cohesionless soils, unit skin friction values were calculated for each soil layer. For the cohesive soils, the α method described in AASHTO Section 10.7.3.8.6b was used. Based on these values, the lengths of piles needed to resist the 399 kip load was calculated to be 85 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 100 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.

4 HP 12x84 piles organized in a single row spaced at 7 feet 2 inches 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 399 kips at both Abutments. The minimum required embedment for the piles is 50 feet below bottom of footing for both abutments. Pile lengths for estimating purposes should be assumed to be 100 feet below the bottom of footing.

4.0 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 files are located in the M:\Projects\13j198\MaterialsResearch\FB-Pier folder:

STR I 4 Piles.in
STR I 4 Piles Scour.in
SER II 4 Piles.in
SER II 4 Piles Scour.in

cc: Mike Chanette, P.E., Stantec
Project File/CEE
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