AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Nick Wark, P.E., P.I.I.T. Program Manager

SPM CEE

From: Stephen Madden, Geotechnical Engineer via Callie Ewald, P.E., Geotechnical Engineering Manager

Date: November 18th, 2019

Subject: Chester BF 0134(50) – Geotechnical Design Parameters Memo

1.0 INTRODUCTION

As requested, we have completed an additional geotechnical and geological subsurface assessment for the Chester BF 0134(50) project. This project consists of the replacement of Bridge No. 51 located on VT Route 11 approximately 1.6 miles east of the intersection of VT Route 11 and VT Route 103 in the town of Chester, VT. The project consists of the replacement of the existing corrugated galvanized metal plate pipe (CGMPP) culvert with a precast concrete box culvert with new headwalls and wingwalls. Contained herein are design parameter recommendations for use in the design of the proposed replaced structure, as determined using the 2017 AASHTO LRFD Bridge Design Specifications.

2.0 FIELD INVESTIGATION

An initial field investigation was completed between September 9th and September 15th, 2016 to evaluate the subsurface profile for design and construction of the proposed replacement structure. The findings of this investigation are detailed in the Geotechnical Data Report dated October 18th, 2016 and submitted by Stephen Madden to Jennifer Fitch, Structures Project Manager. 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 SOIL PROFILE

Review of laboratory data and boring logs revealed the following information pertaining to the soil strata. It should be noted that groundwater elevations are subject to change given the fact that boreholes were generally left open for a short period of time. Because groundwater elevations can fluctuate seasonally and are affected by temperature and precipitation, groundwater may be encountered during construction when not previously noted on the logs.

3.1 Inlet: B-102 and B-104 (Wingwalls No. 2 and 4)

The ground surface elevation at B-102 was approximately 714.3 feet (ft). Groundwater was measured after drilling on September 15th, 2016 at a depth of 11.7 ft below ground surface (bgs) corresponding to an approximate elevation of 702.6 ft. The ground surface elevation at B-104 was approximately 713.9 ft. Groundwater was measured before drilling on September 12th, 2016 at a depth of 12.4 ft bgs corresponding to an approximate elevation of 701.5 ft.

Depth (Below Ground Surface Elevation)	Soil Profile
0 – 12 feet	Loose Silty Gravelly Sand/Sandy Gravel
12 – 18 feet	Very Dense Sandy Silt/Sandy Gravel
>18 feet*	Bedrock

**Note: Top of Bedrock was encountered between depths of 18 and 24 feet.*

3.2 Outlet: B-101 and B-103 (Wingwalls No. 1 and 3)

The ground surface elevation at B-101 was approximately 712.5 ft. Groundwater was measured during drilling on September 13th, 2016 at a depth of 11.2 ft bgs corresponding to an approximate elevation of 701.3 ft. The ground surface elevation at B-103 was approximately 713.0 ft. Groundwater was measured during drilling on September 14th, 2016 at a depth of 8.1 ft bgs corresponding to an approximate elevation of 704.9 ft.

Depth (Below Ground Surface Elevation)	Soil Profile
0 – 4 feet	Loose Gravelly Sand/Sandy Gravel
4 – 12 feet	Medium Dense Gravelly Silty Sand/Sandy Silt/Sandy Gravel
12 – 18 feet	Very Dense Gravelly Sand/Sandy Gravel
>18 feet*	Bedrock

**Note: Top of Bedrock was encountered between depths of 18 and 20 feet.*

Results of the rock coring from the above referenced borings are available in the previously provided boring logs. Information from the cores indicated Gneiss to be present at the boring locations. The bedrock had an average rock mass rating (RMR) of 56, indicating fair rock.

4.0 ANALYSIS

4.1 Shallow Foundation Analysis

AASHTO's LRFD Bridge Design Specifications Manual (2017) was used as the reference for settlement and bearing resistance equations. Section 10.6.3.1.2 contains the equation used for bearing resistance. Neither depth factors nor load inclination factors were used in the analysis as they were not considered pertinent due to the designed embedment of the structure, per Section C.10.6.3.1.2a. Hough's Method, used to calculate settlement in normally consolidated cohesionless soils, can be found in Section 10.6.2.4.2.

It is recommended that the bottom of the wingwall footings be at least 4 ft below the ground surface based on frost susceptibility and bearing stratum at the site. An embedment value of 4 ft was used for the strength limit state analysis and an embedment value of 0 ft was used for

the service limit state analysis, which tends to control the design, to account for potential scour conditions at the design flood per Section 2.6.4.4.2. A conservative groundwater elevation above the bottom of footing elevation was used in design.

As per section 10.5.5.1 of the 2017 AASHTO LRFD Bridge Design Specifications, a resistance factor of 1.0 should be applied to the unfactored bearing resistance for use in service limit state design. Service limit state design includes, but is not limited to, settlement and scour. Section 10.5.5.2.2 specifies that a resistance factor of 0.45 should be applied to the unfactored bearing resistance for use in strength limit state design for spread footings on rock and soil. Strength limit state design includes, but is not limited to, checks for bearing resistance, sliding and constructability. Potential for overturning is limited by controlling the location of the resultant of the reaction forces (eccentricity). Eccentricity, e, shall be limited as follows:

Foundations on soil:	$\lvert e \rvert < b/3$
Foundations on rock:	$\lvert e \rvert < 0.45b$

Eccentricity should be considered for settlement and bearing resistance design of spread footings by using effective footing widths based on AASHTO Section 10.6.1.3. All footing widths presented in this report are *effective* footing widths.

4.1.1 Bearing Resistance (Inlet - Wingwalls 2 and 4, B-102 and B-104)

The maximum length of wingwalls No. 2 and 4 used in the analysis was 6.6 ft, based on the Layout Sheet from the Preliminary Plans dated February 5th, 2019. The Preliminary Plans specify a bottom of footing elevation of 698.25 ft at the inlet of the proposed culvert. Based on the geometry and elevations shown in the plans, it is assumed that the footings will bear on the very dense sandy silt/sandy gravel layer. Based on boring information and subsequent calculations the soil in this layer was assigned a friction angle, $\phi = 38^{o}$ and density, $\gamma = 130$ lb/ft^3. The embedment was assumed to be 4 ft below the ground surface.

For effective footing widths of 2, 4, 6, and 8 ft, the maximum factored bearing resistances for the strength and service limit state are given in Table 4.1.1. Bedrock was encountered in Boring B-102 within 2 ft of the proposed bottom of footing elevation and considering the granular nature of the foundation soils settlement is expected to occur during or immediately after construction. The service limit state design was found to be controlled by the calculated bearing resistance of the foundation soils.

Table 4.1.1 Factored Bearing Resistances at Various Effective Footing Widths at the Inlet

Maximum Wingwall Length (ft)	Effective Footing Width (ft)	Factored Bearing Resistance, Strength Limit State (ksf)	Factored Bearing Resistance, Service Limit State (ksf)
6.6	2	9.1	4.5
	4	11.9	7.7
	6	14.1	9.7
	8	15.8	10.5

4.1.2 Bearing Resistance (Outlet - Wingwalls 1 and 3, B-101 and B-103)

The maximum length of wingwalls No. 1 and 3 used in the analysis was 9.8 ft, based on the Layout Sheet from the Preliminary Plans dated February 5th, 2019. The Preliminary Plans specify a bottom of footing elevation of 696.00 ft at the outlet of the proposed culvert. Based on the geometry and elevations shown in the plans it is assumed that the footings will bear on the very dense gravelly sand/sandy gravel layer. Based on boring information and subsequent calculations the soil in this layer was assigned a friction angle, $\phi = 38^{o}$ and density, $\gamma = 135$ lb/ft^3. The embedment was assumed to be 4 ft below the ground surface.

For effective footing widths of 2, 4, 6, and 8 ft, the maximum factored bearing resistances for the strength and service limit state and the are given in Table 4.1.2. Bedrock was encountered in Boring B-103 within 1 ft of the proposed bottom of footing elevation and considering the granular nature of the foundation soils settlement is expected to occur during or immediately after construction. The service limit state design was found to be controlled by the calculated bearing resistance of the foundation soils.

Table 4.1.2 Factored Bearing Resistances at Various Effective Footing Widths at the Outlet

Maximum Wingwall Length (ft)	Effective Footing Width (ft)	Factored Bearing Resistance, Strength Limit State (ksf)	Factored Bearing Resistance, Service Limit State (ksf)
9.8	2	9.1	4.8
	4	11.8	8.8
	6	14.2	11.9
	8	16.1	14.2

5.0 RECOMMENDATIONS

Shallow foundations appear to be feasible for the proposed precast box and wingwalls as detailed in the Preliminary Plans dated February 5th, 2019. Factored bearing resistances for various footing widths were calculated for the wingwalls and can be found in Tables 4.1.1 and 4.1.2. These calculations are based on the geometric and geotechnical assumptions outlined in Section 4.0 of this report. The bearing resistances presented in this report are controlled by the service limit state and were calculated assuming a conservative scour condition (0 ft embedment). Sections 10.5.2 and 10.5.3 of AASHTO outline all design states relevant to spread footing design and their respective resistance factors. Eccentricity should be considered for settlement and bearing resistance design of spread footings by using effective footing widths based on AASHTO Section 10.6.1.3. Table 5.1 shows the appropriate resistance factors for various design states.

Table 5.1: Summary of Resistance Factors

Design State	Resistance Factor, φ
Settlement	1.0
Scour	1.0
Bearing Resistance	0.45
Sliding	0.80

5.1 Design Parameters

Table 5.1.1 highlights engineering properties assigned to the in-situ soils. Engineering properties of common construction materials are shown in Table 5.1.2. These values should be used when designing any 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 5.1.1: Engineering Properties of In-Situ Soils

	V. Dense Sandy Silt/Sandy Gravel (Inlet Bearing Stratum)	**V. Dense Gravelly Sand/Sandy Gravel (Outlet Bearing Stratum)**
Unit Weight, γ (lbs/ft^3):	130	135
Internal Friction Angle, ϕ (degrees):	38	38
Coefficient of Friction, f		
- mass concrete cast against soil:	0.34	0.45
- soil against precast/formed concrete:	0.31	0.31
Active Earth Pressure Coef., K_a:	0.24	0.24
Passive Earth Pressure Coef., K_p:	4.20	4.20
At-Rest Earth Pressure Coefficient, K_o:	0.38	0.38

Table 5.1.2: 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

5.2 Construction Considerations

5.2.1 Cofferdams/Temporary Earthwork Support

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 operations for each cofferdam specified in the Contract. Construction drawings shall be submitted in accordance with Section 105 (Control of the Work)".

Based on the presence of bedrock at depths of approximately 18 to 24 ft below the roadway surface, as well as the presence of cobbles, boulders, and broken rock as described in the boring logs, the subsurface conditions are not ideal for driving sheets for phased construction and we recommend that other methods of supporting the excavation and roadway during construction be considered.

5.2.2 Construction Dewatering

The bottom of footing elevations for the culverts are estimated to be beneath the water table based on where water was encountered during the subsurface investigation therefore temporary construction dewatering will likely be required to construct the foundations. Temporary dewatering will 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 water, or discharge, should comply with all applicable permits and regulations.

Sumps and trenches should lie outside a 1V:1H line extending downward and outward from the edge of footing. Installation and operation of the Contractor's dewatering system should be integrated with other earthwork operations and sequence of cutting, filling, foundation construction, and backfilling.

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 equal to 95% of the maximum dry density as determined in accordance with AASHTO T-99.

5.2.4 Excavation of Bedrock

Bedrock was encountered within 2 ft of the proposed bottom of footing elevation at the inlet (B-102) and within 1 ft of the proposed bottom of footing elevation at the outlet (B-103). As the bedrock elevation varies across the footprint of the proposed structure, based on the elevations where bedrock was encountered in the borings, we recommend carrying a small quantity for Solid Rock Excavation in the contract to account for this variability and the potential to encounter bedrock that will need to be removed prior to placement of a precast structure.

Based on the logged rock cores and mapped geology, bedrock at the site is a combination of two competent and hard Gneisses—a quartz rich Gneiss and a biotite rich Gneiss. We anticipate the biotite rich Gneiss to be slightly weaker than the quartz rich Gneiss. The rock is fresh, but a weathering rind of a few inches may weaken the top of the bedrock near the interface of the soil horizon. In terms of ability to remove the rock, we expect both types of Gneiss to be difficult to scrape away with an excavator bucket. A hydraulic hammer may be able to remove material, but the actual jointing, geologic structure, and amount of material needed to be removed will determine how effective the operation will be therefore we anticipate rippability of the rock will be difficult.

In order to minimize risk during construction and better quantify the volume of Solid Rock Excavation that may be required, we can perform a series of probes advanced to bedrock within the footprint of the proposed box and/or geophysical methods to better assess the variability in the top of bedrock elevation prior to the development of final plans. If this is something the project team would like to pursue, we can assist with either providing additional probe locations for review and then performing the field work or coordinating the use of geophysical techniques.

6.0 CONCLUSION

If you have any questions or would like to discuss this report, please contact us by phone at (802) 828-2561.

cc: Gary Laroche, P.E., Structures Project Engineer
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