AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Robert Young, P.E., Structures Project Manager

SPM CEE

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

Date: June 3rd, 2019

Subject: Moretown BF 0167(16) – Design Parameters Memo

1.0 INTRODUCTION

As requested, we have completed an additional geotechnical and geological subsurface assessment for the Moretown BF 0167(16) project. This project consists of the replacement of Bridge No. 2 on VT Route 100B over the Mad River in the town of Moretown, VT approximately 0.55 miles north of the intersection of VT 100B with VT Route 100, and includes construction of new abutments supported by spread footings founded on bedrock and a retaining wall adjacent to the southwest corner of the bridge. Contained herein are design parameters for use in the design of the proposed abutments and retaining wall, as determined using them 2017 *AASHTO LRFD Bridge Design Specifications*.

2.0 FIELD INVESTIGATION

An initial field investigation was completed between October 2nd and October 16th, 2017 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 November 8th, 2017 and submitted by Stephen Madden to Robert Young, 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. Borings where soil samples were collected were used to develop the soil profiles below. 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.

Abutment No. 1

3.1 B-102: The ground surface elevation at B-102 was approximately 623.8 feet (ft). Groundwater was not encountered during drilling operations.

Depth (Below Ground Surface Elevation)	**Soil Profile**
0 – 0.65 feet	Asphalt Pavement
0.65 – 11 feet	V. Dense Sandy Gravel/Gravelly Sand
11 – 17.3 feet	Dense Sandy Silt/Silty Sand/Silty Gravel
>17.3 feet	Bedrock

3.2 B-103: The ground surface elevation at B-103 was approximately 624.3 ft. Groundwater was not encountered during drilling operations.

Depth (Below Ground Surface Elevation)	**Soil Profile**
0 – 0.65 feet	Asphalt Pavement
0.65 – 9.4 feet	V. Dense Silty Sandy Gravel/ Gravelly Silty Sand (Boulder 9.4 ft – 13 ft)
9.4 – 13 feet	Boulder
13 – 15.4 feet	Dense to V. Dense Sandy Silt/Gravelly Silty Sand
>15.4 feet	Bedrock

Abutment No. 2

3.3 B-104: The ground surface elevation at B-104 was approximately 627.5 ft. Groundwater was encountered during drilling operations on October 4th, 2017 at a depth of 8.1 ft below ground surface (bgs) corresponding to an approximate elevation of 619.4 ft.

Depth (Below Ground Surface Elevation)	**Soil Profile**
0 – 8 feet	M. Dense Sandy Gravel/Silty Gravelly Sand
8 – 8.7 feet	No Recovery*
>8.7 feet	Bedrock

** N value indicates very dense material*

3.4 B-106: The ground surface elevation at B-106 was approximately 626.0 feet. Groundwater was encountered during drilling operations on October 2nd, 2017 at a depth of 14.5 ft bgs corresponding to an approximate elevation of 611.5 ft.

Depth (Below Ground Surface Elevation)	Soil Profile
0 – 15.3 feet	M. Dense Gravelly Silty Sand/Sandy Silt
>15.3 feet	Bedrock

Wingwall No. 2 (Retaining Wall)

3.5 B-101: The ground surface elevation at B-101 was approximately 622.1 ft. Groundwater was encountered during drilling operations on October 11th, 2017 at a depth of 19.2 ft bgs corresponding to an approximate elevation of 602.9 ft.

Depth (Below Ground Surface Elevation)	Soil Profile
0 – 7 feet	Asphalt Pavement
0.5 – 7 feet	M. Dense Sandy Gravel/Sand
7 – 25 feet	Dense Gravelly Sandy Silt/ Gravelly Silty Sand
25 – 27.3 feet	V. Dense Silty Sandy Gravel
>27.3 feet	Bedrock

Results of the rock coring from the above referenced borings are available in the previously provided boring logs. Information from the cores indicated Greenstone and Greenschist to be present at the boring locations. The bedrock had an average rock mass rating (RMR) of 52, indicating fair rock.

4.0 DESIGN PARAMETERS

Table 4.1 below highlights the geotechnical design parameters assigned to the in-situ soil at the proposed retaining wall bearing stratum and bedrock encountered at the site based on the soil profiles above and laboratory testing. Table 4.1 also includes the parameters assigned to regularly specified aggregates. These values should be used in the analysis of the proposed abutments and retaining wall at this location. 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 of In-Situ Soils and Bedrock and Construction Materials

	Retaining Wall Bearing Stratum (Dense GrSaSi/ GrSiSa)	**In-Situ Bedrock**	**703.01A – Granular Borrow**	**704.08 – Granular Backfill for Structures**
Density, γ (lbs/ft^3):	125	177	130	140
Internal Friction Angle, ϕ (degrees):	35	30	32	34
Coefficient of Friction, f				
- soil/rock against mass concrete:	0.45	0.70	0.50	0.55
- soil against precast/formed concrete:	0.31	N/A	0.40	0.45
Active Earth Pressure Coefficient., K_a:	0.27	0.33	0.31	0.28
Passive Earth Pressure Coefficient., K_p:	3.69	3.00	3.25	3.54
At-Rest Earth Pressure Coefficient., K_o:	0.43	0.50	0.47	0.44

5.0 ANALYSIS

5.1 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. Hough's Method, used to calculate settlement in normally consolidated cohesionless soils, can be found in section 10.6.2.4.2.

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 soil and rock. Strength limit state design includes, but is not limited to, checks for bearing resistance, sliding and constructability. Table 5.1 summarizes the appropriate resistance factors for various design states.

Table 5.1: Summary of Resistance Factors for Design States

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

Additional sliding resistance can be accomplished by doweling the abutment footings into the bedrock. Potential for overturning is limited by controlling the location of the resultant of the reaction forces (eccentricity). Per AASHTO Section 10.6.3.3, eccentricity, e, shall be limited as follows:

Foundations on soil: $|e| < b/3$
Foundations on rock: $|e| < 0.45b$

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 <u>effective</u> footing widths.

5.1.1 Abutments No. 1 and No. 2

Due to shallow bedrock encountered at both abutments it is proposed that spread footings on bedrock will be used at the foundations for the replacement structure. Due to the variations in bedrock across each abutment, we anticipate a subfooting will be necessary to facilitate construction and to evenly distribute the footing pressure onto the bedrock.

The bedrock at Abutment No. 1 has poor to good rock mass rating and is classified as moderately hard to hard, very slightly to moderately weathered, Greenstone. The bedrock at Abutment No. 2 has fair to good rock mass rating and is classified as moderately hard to hard, slightly weathered to unweathered Greenstone and Greenschist with quartz veins.

For both abutments, AASHTO recommends a presumptive bearing resistance of 70 ksf per Table C10.6.2.6.1-1. Taken as the nominal bearing resistance, in combination with a resistance factor of 0.45 for spread footings on rock, per AASHTO 10.5.5.2.2-1, this yields a factored bearing resistance of 31.5 ksf. These values are summarized in Table 5.2.

Uniaxial compressive testing was performed on select rock cores recovered during boring operation to verify that the presumptive bearing resistance was valid for bedrock at the project site. The testing was performed in general accordance with ASTM D7012, *Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures*. One to three samples of NX sized (2.125 inch) intact rock core specimens per core tested were cut to approximately 4.5 inches in length and machined flat on the ends.

Table 5.2: Recommended Bearing Resistance Values for Bedrock

Nominal Bearing Resistance (ksf)	Resistance Factor, φ	Factored Bearing Resistance (ksf)
70	0.45	31.5

Due to the footings bearing directly on competent bedrock, settlement and scour are anticipated to be negligible.

5.1.2 Wingwall No. 2 (Retaining Wall)

We recommend the bottom of the retaining wall footing to be at least 4 feet below the ground surface based on frost susceptibility and bearing stratum at the site. Based on the Preliminary Plans dated January 3rd, 2019, a bottom of footing elevation of 605.50 feet is proposed at wingwall No. 2. Based on the soil profile at this location the bearing material at this elevation has been assigned a friction angle, $\phi = 35^{o}$ and density, $\gamma = 125$ lb/ft^3. An embedment value of 4 feet was used for the strength limit state analysis and an embedment value of 0 feet was used for the service limit state analysis to account for potential scour conditions. A conservative groundwater elevation at the bottom of footing elevation was used in design.

The Preliminary Plans provide a retaining wall length of 67 feet, confirmed by Structures via email on May 15th, 2019. For effective footing widths of between 4 and 6 feet, and greater than 6 feet to 10 feet, the maximum bearing resistances factored due to LRFD strength and service limit states are given in Table 5.3. For the service limit state design, a factored bearing resistance is provided to limit settlement to less than 1 inch. Due to the granular nature of the foundation soils, settlement is expected to occur during or immediately after construction.

Once a final retaining wall length, bottom of footing elevation, and footing width have been established we recommend reevaluating the results of our analyses if the final design geometry of the wingwalls differs from our stated assumptions.

Table 5.3: Bearing resistance values for various effective footing widths

Retaining Wall Length (ft)	Effective Footing Width (ft)	Factored Bearing Resistance, Strength Limit State (ksf)	Factored Bearing Resistance, Service Limit State (ksf)
67	4 - 6	10.0	6.0
	>6 - 10	12.0	8.5

6.0 RECOMMENDATIONS

A maximum factored bearing resistance of 31.5 ksf is recommended for design purposes for the spread footings on rock. It is recommended that any incompetent, weathered, and fractured bedrock be removed until competent bedrock is reached. Based on the average unconfined compressive strength and the low degree of weathering described in the rock cores, we anticipate the rock to be moderately difficult to excavate with conventional equipment, however bedrock removal can likely be achieved using mechanical methods such as hoe-ramming.

A note should be included in the plans to require the Agency Geologist time for inspection of the exposed bedrock prior to any pouring of concrete. If necessary, a concrete subfooting should be poured to provide continuity along the bearing surface.

Shallow foundations appear to be feasible for the proposed retaining wall. Factored bearing resistances were calculated for a range of footing widths and can be found in Table 5.3. The settlement is expected to occur during or immediately after construction. These calculations are based on the geometric and geotechnical assumptions outlined in Section 5.0.

6.1 Construction Considerations

6.1.1 Cofferdams/Temporary Earthwork Support: Cofferdams or temporary shoring may be necessary during construction of the abutments and retaining wall depending on the final location of the bottom of footing. If required, 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 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) registered in the State of Vermont.

6.1.2 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 water, or discharge, should comply with all applicable permits and regulations.

6.1.3 Placement and Compaction of Soils: Fills should be placed systematically in horizontal layers no more than 12 inches thick 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 at least 95% of the maximum dry density determined in accordance with AASHTO T-99 per section 203.11 of the *2018 VTrans Standard Specifications for Construction*. 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.

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

7.0 CONCLUSION

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

cc: Fianna Barrows, P.E., Structures Project Engineer
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