AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Bruce Martin, P.E. Roadway Design Project Manager

JAC SPM

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

CEE

Date: February 23rd, 2017

Subject: Mount Holly ER STP 0133(8) Geotechnical Report (Culvert Recommendations)

1.0 INTRODUCTION

As requested, we have completed our geotechnical investigation for the Mount Holly ER STP 0133(8) project located on Vermont Route 155 near MM 5.57 in Mount Holly, Vermont. This project consists of the replacement of two existing culverts with precast concrete box culverts with new headwalls and wingwalls, located at Sta. 292+21.53 and Sta. 296+33.35. Contained herein are the results of our geotechnical analysis and design parameter recommendations as determined using the 2014 AASHTO LRFD *Bridge Design Specifications.*

2.0 FIELD INVESTIGATION

An initial field investigation was completed in October, 2013 in support of a slope instability investigation along the eastern slope adjacent to the Mill River. The borings advanced as part of this investigation were in close proximity to the locations of the proposed culverts and were therefore used to develop geotechnical design parameters for use by the designer. The findings of this investigation are detailed in the report dated March 10th, 2014 and submitted by Marcy Meyers to Paul Libby, Highway Safety and Design Project Manager. Refer to this report for a detailed description of the field investigation and field and laboratory testing.

3.0 SOIL PROFILE

Soil parameter information from the report dated March 10th, 2014 were used in this analysis and are summarized below in Table 3.1. Due to the similar soil strata found in the borings at the proposed bottom of footing elevations, one conservative profile was used in the analysis.

Table 3.1: Soil Profile

Depth (Below Ground Surface Elevation)	Soil Profile
0 – 5 feet	M. Dense to Dense Gravelly Sand
5 – 28 feet	M. Dense to Dense Silty Sand
28 – 45 feet	Dense to V. Dense Gravelly Sand
45 – 70 feet	V. Dense Silt

Groundwater was encountered during drilling operations at depths of 11.7 feet and 41.2 feet below the ground surface in Borings B-101 and B-102, respectively. A groundwater depth of 11.7 feet was used to develop the above soil profile. Because groundwater elevations can fluctuate seasonally and are affected by temperature and precipitation, groundwater may be encountered during construction when not previously noted in the logs.

4.0 ANALYSIS

4.1 Shallow Foundation Analysis

AASHTO's LRFD Bridge Design Specifications Manual (2014) 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.

We recommend the bottom of the wingwall footings to be at least 4 feet below the ground surface based on frost susceptibility and bearing stratum at the site. 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.

As per section 10.5.5.1 of the 2014 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: $|e| < b/3$

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 effective footing widths.

4.1.1 Bearing Resistance (Culvert @ Station 292+21.53)

The maximum wingwall length used in the analysis was 17.5 feet (Wingwall 1) based on Culvert Layout Sheet 1 from the plans dated January 6th, 2017. Bottom of footing elevations of 1440.6 feet and 1439.9 feet were assumed at the inlet wingwalls and outlet wingwalls, respectively, based on Wingwall Elevation Sheet 1. Based on the geometry and elevations shown in the plans it is assumed that the footings will bear on the Medium Dense to Dense Silty Sand. It was determined that the soil in this layer has a friction angle, $\phi = 34^{o}$ and density, $\gamma = 120$ lb/ft^3. The embedment was assumed to be 4 feet below the ground surface. Figure 4.1 displays the minimum effective footing width per maximum bearing resistance, factored due to LRFD strength and service limit states.

For effective footing widths of 2, 3, 4, and 6 ft, the maximum factored bearing resistances for the controlling service limit state are 2.4, 3.4, 4.5, and 6.4 kips/ft^2 (ksf), respectively, as seen in Figure 4.1.

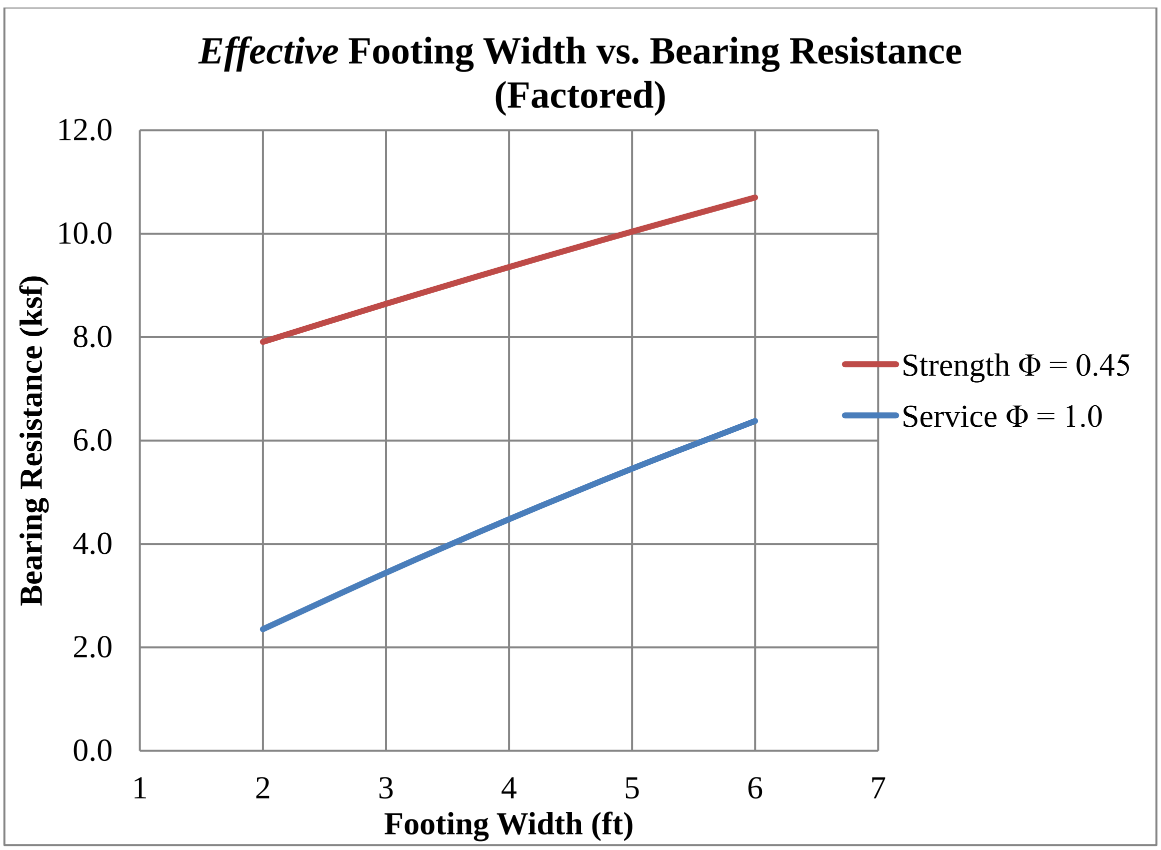

Figure 4.1: *Effective* Footing Width vs. Bearing Resistance for Culvert Wingwalls @ Station 292+21.53

Soil settlement values were calculated for various footing widths based on the nominal bearing pressure. Found in Figure 4.2 are the settlement values for effective footing widths of 2 to 7 feet for the wingwalls. Due to the granular nature of the foundation soils, settlement is expected to occur during or immediately after construction.

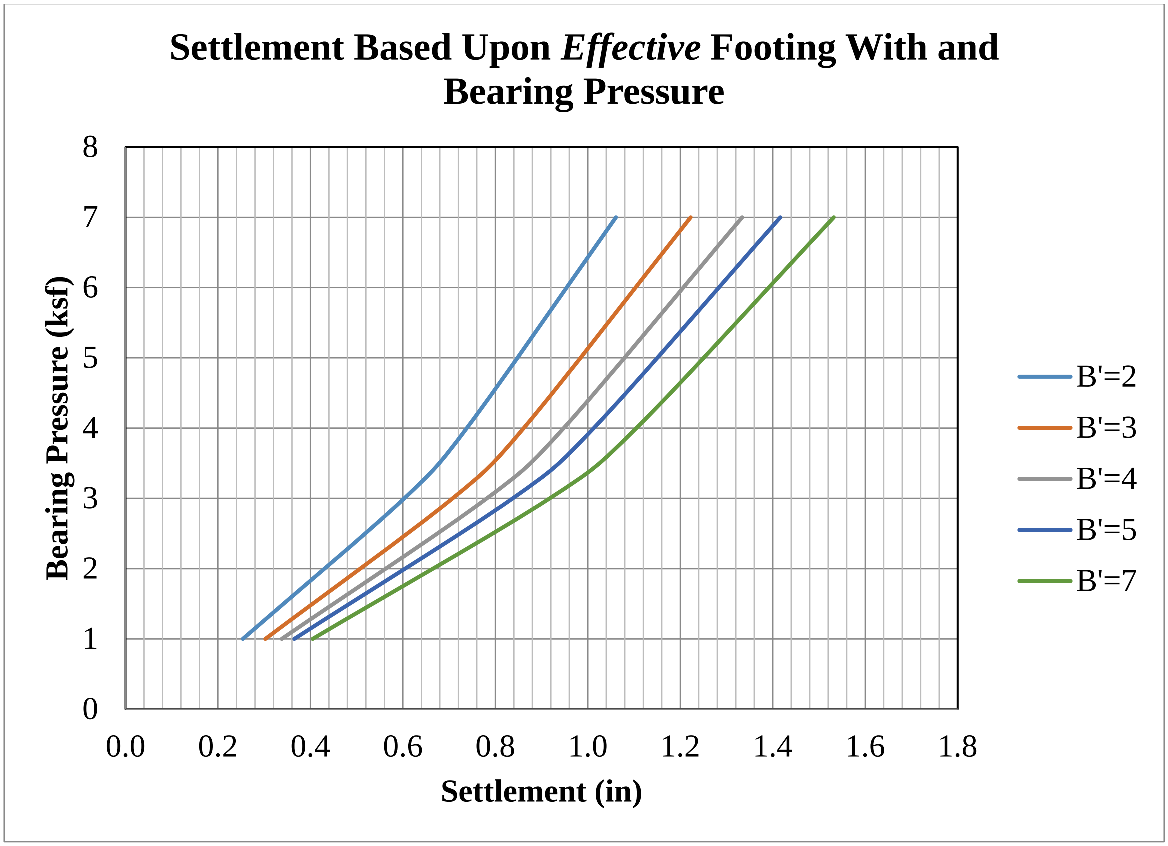

Figure 4.2: Settlement vs. Nominal Bearing Pressure (Culvert Wingwalls @ Station 292+21.53)

4.1.2 Bearing Resistance (Culvert @ Station 296+33.35)

The maximum wingwall length used in the analysis was 14.5 feet (Wingwall 3) based on Culvert Layout Sheet 2 dated January 6th, 2017. Bottom of footing elevations of 1428.0 feet and 1425.0 feet were assumed at the inlet wingwalls and outlet wingwalls, respectively, based on Wingwall Elevation Sheet 2. Based on the geometry and elevations shown in the plans it is assumed that the footings will bear on the Medium Dense to Dense Silty Sand. It was determined that the soil in this layer has a friction angle, $\phi = 34^{o}$ and density, $\gamma = 120$ lb/ft^3. The embedment was assumed to be 4 feet below the ground surface. Figure 4.3 displays the minimum effective footing width per maximum bearing resistance, factored due to LRFD strength and service limit states.

For effective footing widths of 2, 3, 4, and 6 feet, the maximum factored bearing resistances for the controlling service limit state are 2.3, 3.4, 4.4, and 6.2 kips/ft^2 (ksf), respectively, as seen in Figure 4.3.

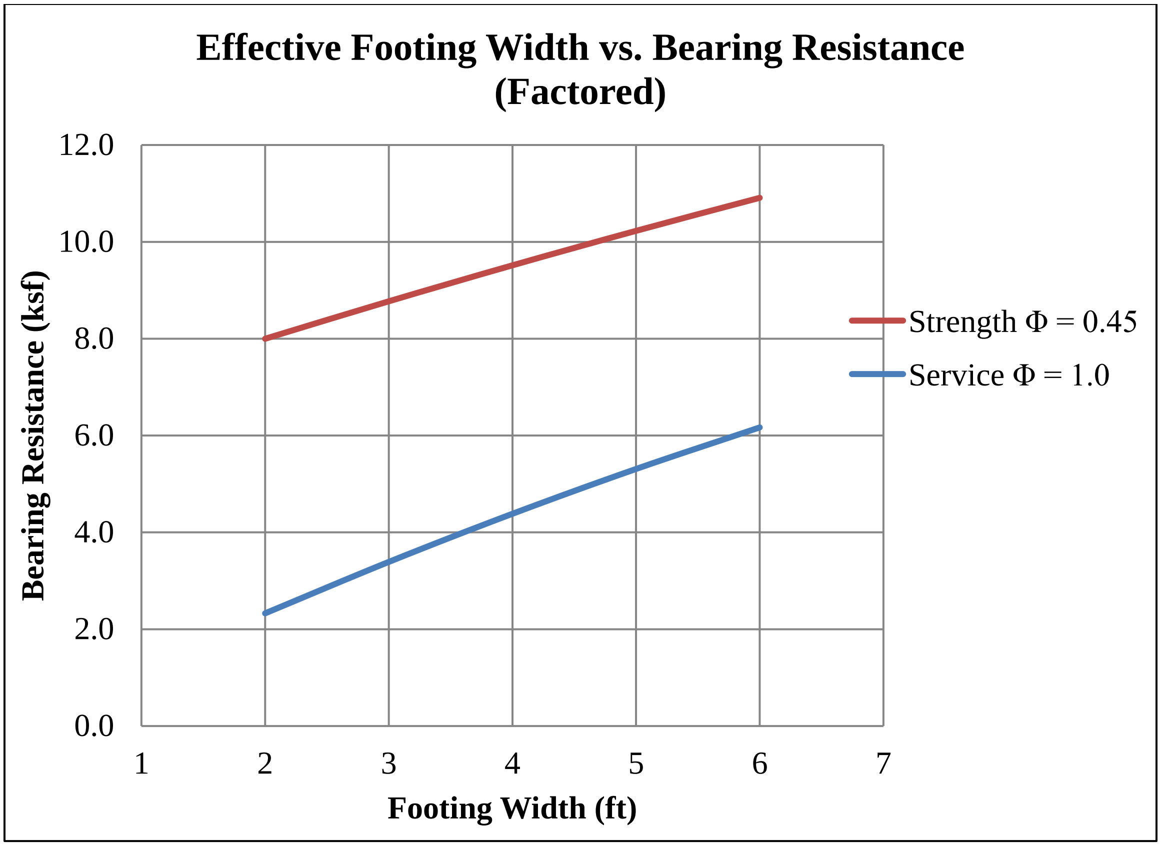

Figure 4.3: *Effective* Footing Width vs. Bearing Resistance for Culvert Wingwalls @ Station 296+33.35

Soil settlement values were calculated for various footing widths based on the nominal bearing pressure. Found in Figure 4.4 are the settlement values for effective footing widths of 2 to 7 feet for the wingwalls. Due to the granular nature of the foundation soils, settlement is expected to occur during or immediately after construction.

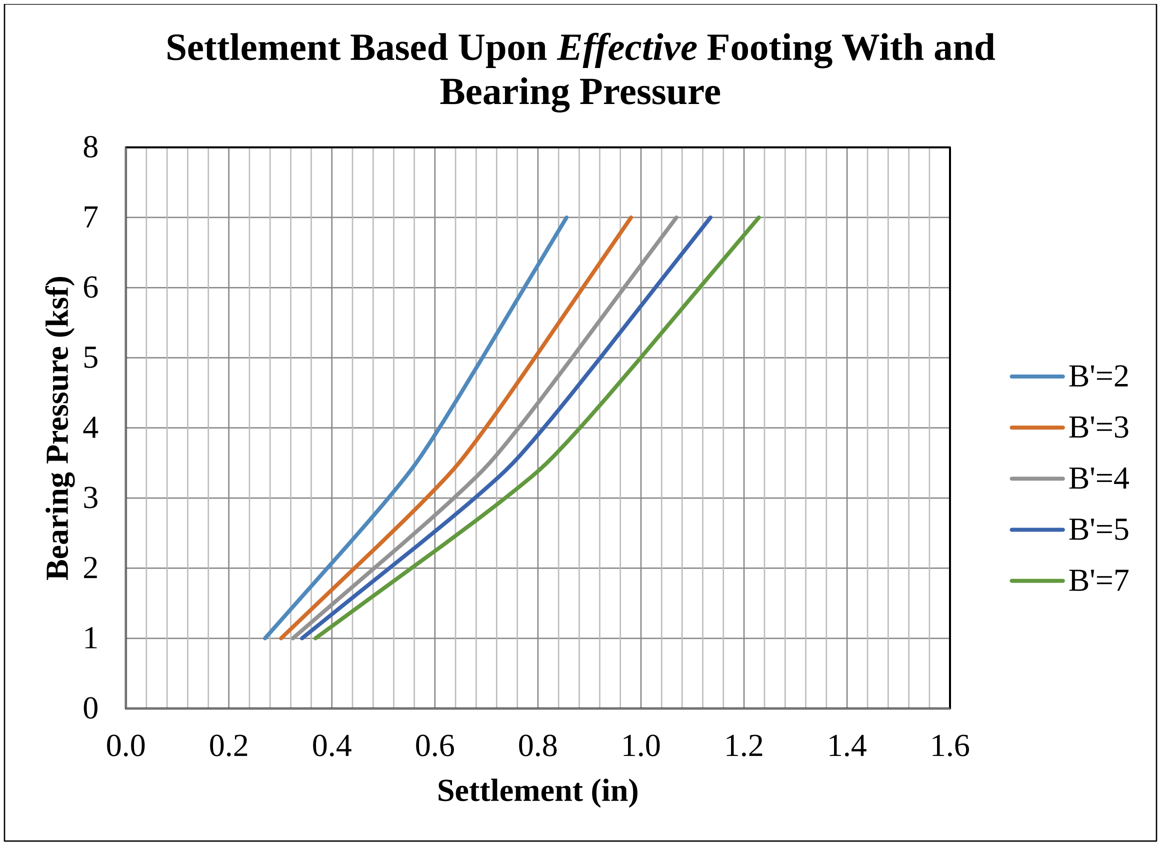

Figure 4.4: Settlement vs. Nominal Bearing Pressure (Culvert Wingwalls @ Station 296+33.35)

5.0 RECOMMENDATIONS

Shallow foundations appear to be feasible for the bottom of box elevation as well as for the wingwalls at the inlet and outlet at each culvert. Factored bearing resistances were calculated for various footing widths of the wingwalls for the culverts and can be found in Figures 4.1 and 4.3. Depending on the bearing pressure and effective footing width used in the analysis, the limiting factor for design is either maximum bearing resistance for the service limit state or settlement. <u>As a result, we recommend the allowable bearing pressure be limited to 4 ksf in design.</u> 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. 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 Construction Considerations

5.1.1 Cofferdams/Temporary Earthwork Support

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 its 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)."

5.1.2 Construction Dewatering

The bottom of footing elevations for the culverts are estimated to be near the water table. Therefore, temporary construction dewatering may 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.

5.1.3 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 at least 95% of the maximum dry density determined in accordance with AASHTO T-99. The current specification calls for 90%, however we are in the process of revising it to be 95% as recommended above.

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 Design Parameters

Table 5.2 highlights the geotechnical design parameters. 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 5.2: Soil Design Parameters

	M. Dense to Dense Silty Sand(Wingwall Bearing Stratum)	**703.04 – Granular Borrow**	**704.08 – Granular Backfill for Structures**
Density (lb/ft^3):	120	130	140
Internal Friction Angle, ϕ (degrees)	34	32	34
Coefficient of Friction			
-soil against mass concrete	0.45	0.45	0.55
-soil against formed concrete	0.30	0.40	0.48
Active Earth Pressure Coefficient, K_a:	0.28	0.31	0.28
Passive Earth Pressure Coefficient, K_p:	3.57	3.26	3.57
At-Rest Earth Pressure Coefficient, K_o:	0.44	0.47	0.44

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: Electronic Read File/DJH
Project File/CEE
JAC
SPM

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