GEOTECHNICAL INVESTIGATION

B-478 REPLACEMENT
EUREKA COUNTY, NEVADA

EA 74121

January 2019

| **NEVADA DEPARTMENT OF TRANSPORTATION** | MATERIALS DIVISION |
| GEOTECHNICAL SECTION | 1263 STEWART ST, CARSON CITY, NEVADA 89712 |

STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION
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EA 74135

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Introduction

The Nevada Department of Transportation (NDOT) plans to replace Bridge Structure B-478 at approximately Mile Post (MP) 4.3 on SR 278 between Eureka and Carlin. This report presents the findings and recommendations developed from our geotechnical engineering investigation for the proposed culvert replacement. The investigation was conducted in accordance with American Association of State Highway and Traffic Administration (AASHTO) and Federal Highway Administration (FHWA) guidelines.

1.1 Project Description

It is our understanding that the project consists of replacing the existing 20- by 35-foot double barrel reinforced concrete box culvert with an approximate 24- by 90-foot triple barrel reinforced concrete box culvert.

The project Vicinity Map and Exploration Map are shown in Appendix A on Figures A-1 and A-2, respectively.

1.2 Purpose and Scope of Work

The purpose of this investigation was to evaluate the suitability of the project site from a geotechnical perspective, for the proposed culvert replacement. The main objectives of the investigation were to characterize the subsurface materials, perform engineering analyses, develop geotechnical recommendations for design and construction, and document our findings, and recommendations in this report.

The scope of our geotechnical investigation includes the following:

- A review of published geologic and geotechnical information pertaining to the site vicinity;
- A field exploration consisting of drilling one boring to a maximum depth of 36½ feet below ground surface (bgs) to obtain information to evaluate the subsurface conditions;
- Perform geotechnical laboratory testing on select soil samples collected from the borings;
- Perform engineering analyses to develop geotechnical design criteria and recommendations for the proposed project; and
- Preparation of this report.

1.3 Limitations

This report has been prepared by Nevada Department of Transportation (NDOT) Geotechnical Section under the supervision of those whose signatures appear herein. The interpretation of data, findings, and recommendations presented in this report were developed from our geotechnical investigation.

If the proposed project is modified or relocated, or if the subsurface conditions found during construction differ from those described in this report, NDOT Geotechnical Section should be contacted immediately to assess the new information or changed conditions and determine if additional recommendations are required.

2. Field Exploration and Laboratory Testing

2.1 Field Exploration

One boring was drilled on September 26, 2018 at the approximate location shown on Figure A-2. The boring was advanced to a maximum depth of approximately 36½ feet below ground surface (bgs) utilizing a truck-mounted Diedrich D-120 (NDOT 1082) drill rig equipped with 6-inch hollow stem augers. Samples were collected using Modified California (3-inch outer diameter) and Standard Penetration Test samplers driven by an automatic hammer with a weight of 140 pounds and a drop of 30 inches.

The number of blows required to drive the sampler 6-inches were recorded for the 18-inch drive, and the cumulative blow count for the bottom 12 inches of drive is presented in the logs of borings. The blow counts presented in the logs are uncorrected and are shown as they were recorded in the field. Normalizing the blow counts for use in analysis was performed utilizing corrections for sampler type, rod length, auger diameter, hammer efficiency, and overburden stress. Both the samples and drill cuttings were visually classified in the field based on the Unified Soil Classification System (USCS) in general accordance with ASTM D2488.

Logs of the borings were prepared based on the field logging and the results of laboratory testing in general accordance with ASTM D2487. The boring logs and key are presented in Appendix B.

2.2 Geotechnical Laboratory Testing

Laboratory testing was conducted on select soil samples recovered during the field exploration. Tests conducted include the following:

- Method of Test Sieve Analysis of Coarse and Fine Aggregate (Nev. T206);
- Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil (AASHTO T265);
- Method of Test For Determination of The Resistance R-Value of Treated and Untreated Bases, Subbases and Basement Soils by The Stabilometer (Nev. T115);
- Standard Test Methods for Laboratory Determination of Density (ASTM D7263);
- Method of Test For Determining The Liquid Limit, Plastic Limit, and Plasticity Index of Soil (Nev. T210, T211, and T212);
- Standard Method of Test for Determining Minimum Laboratory Soil Resistivity (AASHTO T288);
- Standard Method of Test for Determining pH of Soil (AASHTO T289);
- Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil (AASHTO T290);
- Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil (AASHTO T291);

Geotechnical laboratory test results are presented in Appendix C.

3. Site and Subsurface Conditions

3.1 Site Conditions

Structure B-478 is located on SR 278 in Eureka County, Nevada. The site consists of an existing double-barrel box culvert at the intersection of Slough Creek and the two-lane highway. The site topography is generally flat except for the approximately 7-foot tall embankment fill. At the time of our exploration, Slough Creek was dry, and the surrounding ground surface consisted of sand with sparse grass and brush. The double-barrel reinforced concrete box culvert was damaged, steel reinforcement was exposed, and the concrete was spalling. The Vicinity Map is presented on Figure A-1.

3.2 Subsurface Conditions

3.2.1 General Geology and Faulting

The site is located in the Basin and Range geomorphic province, at the southern end of Diamond Valley. The site area is mapped as being comprised of primarily Quaternary alluvium. The nearest active fault with historic movement (last 150 years) is the Pleasant Valley fault zone, Pearce section located approximately 95 miles to the west. Other active faults nearby include the Fairview fault zone, and the Dixie fault zone, 1954 section, located approximately 110 miles to the southwest from the site. The nearest Quaternary fault is the Western Diamonds Mountains fault zone, located approximately 3 miles to the southeast.

3.2.2 Subsurface Materials

The results of our field exploration and laboratory analyses indicate approximately 6 inches of asphalt underlain by 10 inches of aggregate base comprise the highway road section. Beneath the base, 7 feet of loose clayey sand (SC) embankment fill was encountered. Native soil beneath the embankment fill consists of medium dense sand (SW) with gravel to a depth of approximately 13 feet bgs. Below the gravel an approximate 5-foot thick layer of soft fat clay (CH) was encountered. Below the clay, medium dense to dense silty sand (SM) was encountered to the maximum depth explored, approximately 36½ feet bgs.

3.2.3 Groundwater Conditions

Groundwater was not encountered in the boring during our exploration. Based on review of published well logs from the general vicinity, groundwater is expected to be much deeper than the depths explored and is not anticipated to be encountered during construction excavations.

4. Recommendations

4.1 Site Preparation and Earthwork

4.1.1 Site Preparation

General site preparation should include stripping of any surface vegetation in accordance with Silver Book Section 201. The removal of any existing utilities or obstructions should follow Silver Book Section 202.

4.1.2 Subgrade Preparation

Any soft or loose areas at the base of excavations should be stabilized prior to the placement of the box culvert. Stabilization may be accomplished by excavating to firm, native material and replacing with granular backfill or using geotextiles in accordance with Silver Book 203.03.18. Proof-rolling and final verification of stabilization should be observed by NDOT Geotechnical Section. Upon completion of subgrade preparation, granular backfill should be placed as described below.

4.1.3 Temporary Excavations

Temporary excavations and shoring should conform to OSHA standards. Based on the subsurface materials encountered in our exploration, the clayey sand embankment soils can be classified as Type B (OSHA 1926). Vertical excavations should not exceed 4 vertical feet. Excavations greater than 4 vertical feet should be sloped in accordance with OSHA 1926 or shored. Protection of workers and adjacent structures, shoring design, and the stability of all temporary slopes are the sole responsibility of the contractor.

4.1.4 Cut and Fill Slopes

Permanent fill slopes should have a maximum slope of 2:1 (H:V) and should be overbuilt and trimmed to limits on the staking. Slopes should be constructed in accordance to Silver Book 203.03.06. All slopes should be stabilized from wind and rain erosion in accordance with Silver Book Section 211.

4.2 Foundations

The soil parameters used for foundation analysis are presented in Table 1 below. The parameters are based on the subsurface boring and laboratory testing of collected samples.

Table 1 Foundation Soil Parameters

Parameter	Recommended Value
Unit weight, γ_t (pcf)	120
Cohesion, c (psf)	0
Internal friction angle, φ (degrees)	34
Active Earth Pressure Coefficient K_a	0.283
At-Rest Earth Pressure Coefficient K_0	0.441
Passive Earth Pressure Coefficient K_p	3.537

4.2.1 Bearing Resistance

A soft fat clay layer was encountered at an approximate depth of 6 feet below the proposed foundation. The weaker layer resulted in the utilization of a two-layer bearing resistance analysis and subsequently a reduced capacity. The proposed culvert and wingwalls may be supported on shallow foundations. The foundation can be designed with service limit bearing resistance of 1,800 pounds per square foot (psf).

Table 2 Bearing Resistance

Service Limit	Factored Strength Limit	Nominal Strength Limit
1,800 psf	1,800 psf	4,000 psf

4.2.2 Settlement

For foundations and subgrade designed and prepared as recommended in this report, total settlements of less than 1 inch are anticipated at the service limit. Settlement within the footprint of the existing culvert should be negligible. Where the footprint of the proposed culvert extends past the existing culvert footprint 30-feet to either side, differential settlements are expected to be on the order of 1 inch over a horizontal distance of 30 feet.

4.2.3 Passive Resistance

A nominal passive earth pressure of 420 pounds per cubic foot (pcf) may be used if the upper foot of soil is neglected. The passive resistance may be taken as the passive earth pressure multiplied by the depth of the footing.

The shear resistance between the foundation and the supporting soil is taken as the friction coefficient multiplied by the total load at the interface. A nominal sliding resistance of 0.67V is recommended for the soils described above in Table 1, where V is the total vertical force.

Both the passive and shear resistance should be factored by 0.5 and 0.8 respectively for the Strength Limit State. Resistance factors of 1.0 should be used for the Service Limit and Extreme Limit States.

4.3 Retaining Wall Earth Pressure

The wingwalls may be evaluated for a drained active earth pressure of 35 pcf for level backfill, and 50 pcf for 2:1 (H:V) sloping backfill. The walls of the culvert may be designed for a drained at-rest earth pressure of 55 pcf.

4.4 Corrosion

Soils corrosivity analysis is important for estimating and mitigating the deterioration of buried ferrous metals and concrete. We performed corrosion testing on representative samples from boring B-1 at a depth of 2.5, and 10 feet bgs as an indicator of the corrosive properties of the soil. Test results are summarized below in Table 3 and presented in Appendix C.

Table 3 Soil Corrosion Results

Boring No.	Depth (ft.)	pH	Minimum Resistivity (ohm-cm)	Water Soluble Sulfates (ppm)	Water Soluble Chlorides (ppm)
B-1	2.5	8.50	2,701	5	51
B-1	10	8.80		5	51

According to ACI 318, water soluble sulfates less than 1,000 parts per million is considered "not applicable". A water-soluble chloride content of less than 500 ppm is generally non-corrosive to reinforced concrete.

The provided corrosion test results are only an indicator of potential soil corrosivity for the sample tested at the selected depth interval. It is possible that corrosion potential can vary by sample location and depth. It is recommended that Type II Portland cement is used for the concrete foundations, box culvert, and wingwalls.

4.5 Seismic Design

The seismic design criteria for the site (39.6130°N, 116.0163°W) were developed utilizing the USGS seismic hazards tool in accordance with AASHTO 2017, considering the site location, and the subsurface information obtained from our geotechnical investigation. Minimum seismic parameters for use in design are listed by county in the NDOT Structures Manual and supersede the USCS mapped values presented below.

Table 4 Seismic Design Criteria

Parameter	USCS Mapped Value	NDOT Structures Manual Value
Site Class	D	D
Peak ground acceleration (PGA)	0.139 g	0.25g
Mapped horizontal response spectral response at short period (S_S)	0.334 g	0.60g
Mapped horizontal response spectral response at 1sec period (S_1)	0.113 g	0.20g
Peak ground acceleration coefficient (F_{PGA})	1.522	1.3
Site coefficient (Fa)	1.533	1.32
Site coefficient (F_V)	2.348	2.0
Mapped MCE peak ground acceleration (A_S)	0.212 g	0.325g
Design Spectral Acceleration for short period (S_{DS})	0.512 g	0.792g
Design Spectral Acceleration for 1 sec period (S_{D1})	0.265 g	0.4g

5. References

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Appendix A
Figures

Appendix B
Logs of Borings

Appendix C

Laboratory Test Results

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