

Earth Mechanics, Inc. Geotechnical & Earthquake Engineering

# **TECHNICAL MEMORANDUM**

# **EMI PROJECT NO:** 13-116

DATE:	October 29, 2013
PREPARED FOR:	Mr. Todd W. Dudley / AECOM
PREPARED BY:	(Raja) S. Pirathiviraj and Lino Cheang / Earth Mechanics, Inc. (EMI)
SUBJECT:	Preliminary Foundation Report (PFR) Mt. Vernon Avenue Overhead (Replace), Bridge No. 54C-0066 City of San Bernardino, California

#### 1.0 Scope of Work

This memorandum has been prepared to provide the necessary geotechnical information to assist the structural designers in the type selection process for the replacement of Mount (Mt.) Vernon Avenue Overhead (OH) (Bridge No. 54C-0066). The content of this memorandum follows Caltrans Foundation Report Preparation for Bridge Foundations (Caltrans, 2009). It includes preliminary geotechnical, seismic, and foundation recommendations for the subject structure.

The recommendations provided in this memorandum are based on soil boring data collected in September 1997 for seismic retrofit of the subject structure. An additional supplemental geotechnical investigation is on-going; therefore, the following recommendations may change when this additional site-specific information becomes available.

#### 2.0 **Project Description**

The City of San Bernardino, in cooperation with the California Department of Transportation (Caltrans), as assigned by the Federal Highway Administration (FHWA), proposes to replace Mt. Vernon Avenue OH (Bridge No. 54C-0066). Mt. Vernon Avenue OH spans over Burlington Northern Santa Fe (BNSF) railroad mainlines, storage tracks and intermodal yard, as well as regional commuter tracks operated by Southern California Regional Rail Authority (Metrolink) and rail tracks used by Amtrak. A Site Location Map is presented in Figure 1.

<u>Existing Structure</u>. The existing Mt. Vernon Avenue OH follows a generally north-south alignment along Mt. Vernon Avenue. The existing structure is approximately 1,016 feet long and 49 feet wide with two traffic lanes in each direction.

<u>Proposed Structure</u>. The replacement structure is proposed to be a nine-span structure.

#### **3.0** Exception to Policy

There is no exception that deviates from Caltrans policy related to the preparation of this report.

# 4.0 Field Investigation and Testing Program

Supplemental field investigation consisting of soil borings and Standard Penetration Tests (SPT's) are on-going. Conclusions and preliminary recommendations provided in this report are based on the 1997 LOTB sheets.

### 5.0 Laboratory Testing Program

Soil laboratory testing will be performed when supplemental field investigation is completed after approval of bridge type selection. Conclusions and preliminary recommendations provided in this report are based on the 1997 LOTB sheets.

# 6.0 Site Geology, Drainage and Subsurface Conditions

<u>Site Geology</u>: The site is located in the Upper Santa Ana River (USAR) Valley in San Bernardino County. The USAR Valley is a broad alluvial plain between the San Gabriel-San Bernardino Mountain blocks on the north, and the high lands of the Perris Block and Crafton Hills on the south. The Santa Ana Mountains and Puente Hills lie to the west. The Perris Block, Puente Hills, Santa Ana Mountains, and San Jacinto Mountains are within the Peninsular Ranges physiographic province, and the San Gabriel and San Bernardino mountains are within the Transverse Ranges province. The low-lying physiography of the USAR Valley suggests it is a transition zone between the Peninsular Ranges and Transverse Ranges province.

The formation at the site is a young alluvial deposit of late Holocene age. These surficial deposits are probably underlain by older but similarly non-indurated (unconsolidated) alluvial deposits to a depth of up to about 1,000 feet. The nature and type of bedrock below the alluvial sediments is not well known but is believed to consist of Mesozoic-age crystalline igneous and metamorphic rocks similar to those exposed in the nearby San Gabriel and San Bernardino Mountains, and in small hills such as the Perris Hills a short distance northeast of the site.

<u>Drainage</u>: The major drainages in the immediate project area are the Santa Ana River to the south and Lytle Creek and Warm Creek to the east. In the vicinity of the project site, Lytle Creek drains into two concrete-lined channels. All of these drainages merge with the Santa Ana River south of the site.

<u>Subsurface Condition</u>: Elevation of the existing ground surface inside the rail facilities is about 1,082 feet. The subject site is underlain by four main soil units. The site's existing ground surface and underlying soil stratigraphy is uniform and nearly horizontal along the entire length of the bridge. As a result, a single idealized soil profile representative of the entire site was used for all abutments and bents for the purpose of foundation design. The table below summarizes the four soil units and the groundwater conditions encountered in our borings.



Unit	Top Elevation of Unit (feet)	Design Top Elevation (feet)	Soil Unit Classification			
Ι	1,082	1,082	Loose to Compact Silty Sand and Sandy Silt (SM/ML)			
II	1,054 to 1,057	1,056	Dense to Very Dense Sand with Gravel (SP)			
III	1,033 to 1,037	1,036	Stiff to Very Stiff Sandy Silt (ML)			
IV	1,018 to 1,022	1,020	Dense to Very Dense Sand and Silty Sand with Gravel (SP/SM)			
Note: Groundwater ranges from elevation 1,019 to 1,037 feet. Design groundwater is elevation 1,037 feet.						

In September 1997, groundwater was encountered in four borings at depths of 45 and 63 feet below the ground surface. For design, groundwater was assumed to be at the top of Soil Unit III which is at a depth of 46 feet below the existing ground surface.

# 7.0 Scour Evaluation

The existing structure does not cross a channel or basin that conveys water; therefore, scour potential should not be a design issue.

#### 8.0 Corrosion Evaluation

Based on the soil samples obtained from the 1997 soil borings, laboratory test results indicate a minimum soil resistivity of 2,400 to 4,700 ohm-cm, pH values of 7.2 to 8.2, chloride content of 104 to 143 parts per million (ppm) and sulfate content of 100 to 160 ppm.

Based on the Caltrans Corrosion Guidelines (2012), soils are considered corrosive if the pH is 5.5 or less, or the sulfate concentration is 2,000 ppm or greater, or the chloride concentration is 500 ppm or greater. Based on the test results and the Caltrans criteria, the on-site soils are considered to be non-corrosive to bare metals and concrete.

Additional soil corrosion tests will be performed during the final design phase.

#### 9.0 Preliminary Seismic Recommendations

The preliminary design ARS curve was determined using the Caltrans ARS Online website. The table below summarizes the key parameters for determining the preliminary design ARS curve. The peak ground acceleration (PGA) is the zero-period spectral acceleration shown on the ARS curve. Preliminary design ARS curve is presented in Figure 2.

Site Coordinates	Latitude = $34.1037$ degrees	Longitude = $-117.3129$ degrees
Shear Wave Velocity, $V_{s(30)}$	892 feet/sec (272 m/s)	
Peak Ground Acceleration (PGA)	0.862 g	

<u>Liquefaction Evaluation</u>: Based on the soil boring data, the saturated onsite soils consist of either dense to very dense sand or stiff to very stiff sandy silt. The relative density of the sandy soil is high enough to be resistant to liquefaction and the silty soil layer has sufficient fines and high



enough Standard Penetration Test (SPT) blowcounts to resist widespread liquefaction. Therefore, liquefaction potential is not considered to be a design issue.

However, a more detailed liquefaction assessment will be conducted during the final design phase.

<u>Seismic Settlement:</u> Since the liquefaction potential at the site is anticipated to be low, seismic settlement of onsite soils is small. However, during the final design phase, we will evaluate seismic settlement.

<u>Seismic Slope Stability</u>: The project area is composed of relatively flat terrains. Graded embankment consisting of retaining walls and fill slopes will be constructed at the approaches. These graded embankments, if properly constructed, should be stable at a gradient of 2:1 (H:V). We will evaluate slope stability during the final design phase when layout and profile sheets are available.

<u>Surface Fault Rupture:</u> Aerial photographs were analyzed to evaluate the potential for active surface faults at the sites. These lineaments can be caused by many natural and cultural features. Some may be caused by fault displacement that has resulted in differential uplifting or lateral shifting of landforms causing cracks in the ground. Differentiating between natural and manmade lineaments is imperative and not always possible. Many, if not most, lineaments identified on the photographs will turn out to be something other than an earthquake fault.

The aerial-photograph analysis for this project used black and white, stereographic photographs from 1930 to 1960. More-recent (post-1960) photographs are also available but these commonly are not very useful because the natural landforms (geomorphology) are obscured by 19th and 20th century agriculture and urban development.

The aerial-photographs show strong lineaments along the mapped trace of the San Jacinto Fault and the Rialto Bench to the southwest of the site. In addition numerous discontinuous northwesterly trending lineaments were observed in the vicinity of the site. The lineaments were generally weakly defined and discontinuous over long distances. However, no lineaments were observed to directly pass through the project site. Two lineaments that were classified as extremely weak project northwesterly toward the Mt. Vernon Avenue Overhead however they were not visible in later photos and are not considered through-going.

In summary, obvious lineaments to the west and southwest of the site correlate with mapped traces of the San Jacinto Fault as shown on the Alquist-Priolo map. Other lineaments sub-parallel to the San Jacinto fault were observed to be subtle and weak to very weak. These lineaments could be the result of faulting or the remnants of old stream channels. No lineaments were observed passing through or projecting into the immediate site area, and the fault rupture hazard at the site appears low.



#### **10.0 As-Built Foundation Data**

The existing Mt. Vernon Avenue OH is supported on pile foundations. Piles are creosoted wooden piles, but the pile size is unknown. As shown in the as-built plans, there are 2 abutments and 21 bents. The maximum pile load is 20 tons and the estimated pile length is 25 feet.

# **11.0 Preliminary Foundation Recommendation**

Based on the information provided by the structural engineers, both Class 140 steel driven piles and 30-inch diameter Cast-In-Drilled-Hole (CIDH) piles are considered for the abutments. For the two-column bents, both 72-inch diameter Cast-In-Steel-Shell (CISS) concrete driven piles and 96-inch diameter CIDH piles are considered. The nominal resistances for each pile types are provided by the structural engineers. For cost estimating purposes, preliminary pile lengths are provided below for each pile type.

The nominal resistances are 210 kips and 260 kips for Class 140 steel driven piles for Abutments 1 and 10, respectively. Caltrans standard Class 140 pile is HP 10x57. The preliminary pile lengths are 55 feet and 60 feet for Abutments 1 and 10, respectively.

The nominal resistances are 380 kips and 520 kips for 30-inch diameter CIDH piles for Abutments 1 and 10, respectively. The preliminary pile lengths are 35 feet and 40 feet for Abutments 1 and 10, respectively.

The nominal resistances vary between 3,800 kips and 7,500 kips for Bents 2 to 9. Preliminary pile lengths are estimated for nominal resistance of 3,800 kips, 5,470 kips and 7,500 kips. Pile length for intermediate nominal resistances can be obtained by linear interpolation. The preliminary pile lengths are 85 feet, 115 feet and 146 feet for nominal resistances of 3,800 kips, 5,470 kips and 7,500 kips, respectively, for 96-inch diameter CIDH piles. For the 72-inch diameter CISS piles, the preliminary pile lengths are 99 feet, 117 feet and 135 feet for nominal resistances of 3,800 kips, 5,470 kips and 7,500 kips, respectively.

<u>Settlement Period</u>: Approach embankments will be raised for the replacement bridge. Since the subsurface material is predominantly granular, settlements are anticipated to be small. The settlement magnitude and settlement period will need to be evaluated using supplemental soil borings and laboratory test results during the final design phase. For cost estimating purposes, settlement period is estimated to be about 14 days.

# 12.0 Pile Load Testing

Pile load testing is not required but may be considered. Per Caltrans Memo to Designer 3-1 (2008), pile load test for CIDH piles is required when piles are located in "unproven" soil formations. In our opinion, there is no unusual and uncertainty in the subsurface conditions that will affect pile capacity calculations; there is sufficient soil information on the 1997 LOTB sheets and supplemental borings are planned as discussed in Section 13.0.

The pile load testing, if performed, should be conducted using the same pile dimension, pile material, and construction procedure and equipment as those planned for the production piles.



The pile test should be performed at the locations with representative soil condition. The load test should be performed in accordance with Caltrans Amendments (Caltrans, 2011) to AASHTO LRFD Bridge Design Specification Section 10.8.3.5.6 (AASHTO 2007).

# **13.0** Additional Field Work and Laboratory Testing

We recommend drilling three supplemental soil borings for the proposed bents of the replacement bridge. These supplemental borings will be extended below the termination depths of the 1997 borings. The maximum boring depth is expected to be near 150 feet. Supplemental borings are also proposed at the abutments; one boring at each abutment down to a depth of 90 feet.

Samples recovered during the field investigation will be transported to the laboratory for testing. All of the soil samples will be visually classified and moisture content/density tests will be performed. Additional samples will be selected for sieve analysis, #200 wash, corrosion, and direct shear tests. Other laboratory tests may be required depending upon the nature of the soils encountered during the investigation.

# 14.0 References

- American Association of State Highway and Transportation Officials (AASHTO), 2007, AASHTO LRFD Bridge Design Specification, 4th Edition. Washington, DC: AASHTO.
- Caltrans, 2012, Corrosion Guidelines, Materials Engineering and Testing Services, Corrosion Technology Branch, Version 2.0, November.
- Caltrans, 2011, California Amendments to AASHTO LRFD Bridge Design Specifications, Fourth Edition, November.
- Caltrans, 2009, Foundation Report Preparation for Bridge Foundations, Division of Engineering Services, Geotechnical Services, December.

Caltrans, 2008, Memo to Designer 3-1, July.







# ATTACHMENT 1

Log-Of-Test-Boring (LOTB) Sheets



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