FOUNDATION REPORT

Ranchero Road Bridge (Replace) Over California Aqueduct
Bridge No. 54C-0449
City Project Number: C.O. No. 7094
Encroachment Permit #1519
Hesperia, California

EMI Project No. 15-111
Date: Jan. 29, 2018

EARTH MECHANICS, INC.

Geotechnical and Earthquake Engineering

Earth Mechanics, Inc.

Geotechnical & Earthquake Engineering

Jan. 29, 2018

EMI Project No. 15-111

TranSystems
6 Hutton Centre Drive, Suite 1250
Santa Ana, CA 92707

Attention: S. Andy Cheah

Subject: Foundation Report for Ranchero Road Bridge (Replace) Over California Aqueduct, Hesperia, California

Dear Mr. Cheah:

Attached please find the Foundation Report for the subject bridge replacement project. This report contains the findings and conclusions of our field investigation and laboratory testing program. It also contains our recommendations for the design and construction of the bridge and wall foundations and the roadway pavement.

All review comments received were incorporated into this report.

We appreciate the opportunity to provide geotechnical design services for this project. If you have any questions, please call us.

Sincerely,

EARTH MECHANICS, INC.

REGISTERED PROFESSIONAL ENGINEER
MIKE KAPUSKAR
No. GE2564
Expires 12-31-18
GEOTECHNICAL
STATE OF CALIFORNIA

Mike Kapuskar, GE 2564
Sr. Engineer

Lino Cheang, GE 2345
Project Manager

REGISTERED PROFESSIONAL ENGINEER
LINO C. CHEANG
NO. GE 2345
EXP. 9/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

MK/lcc:mh

17800 Newhope Street, Suite B, Fountain Valley, California 92708 • Tel: (714) 751-3826 • Fax: (714) 751-3928

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

FOUNDATION REPORT

RANCHERO ROAD BRIDGE (REPLACE) OVER CALIFORNIA AQUEDUCT

BRIDGE NO. 54C-0449
CITY PROJECT NUMBER: C.O. NO. 7094
ENCROACHMENT PERMIT #1519

HESPERIA, CALIFORNIA

Prepared for:

TranSystems
6 Hutton Centre Drive, Suite 1250
Santa Ana, CA 92707

Prepared By:

Earth Mechanics, Inc.
17800 Newhope Street, Suite B
Fountain Valley, California 92708

EMI Project No. 15-111

January 29, 2018

TABLE OF CONTENTS

17800 Newhope Street, Suite B, Fountain Valley, California 92708 • Tel: (714) 751-3826 • Fax: (714) 751-3928

LIST OF TABLES

LIST OF FIGURES

APPENDICES

17800 Newhope Street, Suite B, Fountain Valley, California 92708 • Tel: (714) 751-3826 • Fax: (714) 751-3928

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF STUDY

This Foundation Report presents the findings and conclusions of a geotechnical investigation conducted by Earth Mechanics, Inc. (EMI). The report presents foundation design and construction recommendations for replacement of the existing Ranchero Road Bridge (Bridge No. 54C-0449) in the City of Hesperia, California. A site location map is presented in Figure 1-1.

EMI is a subconsultant to TranSystems. The geotechnical services provided for this project included the following tasks:

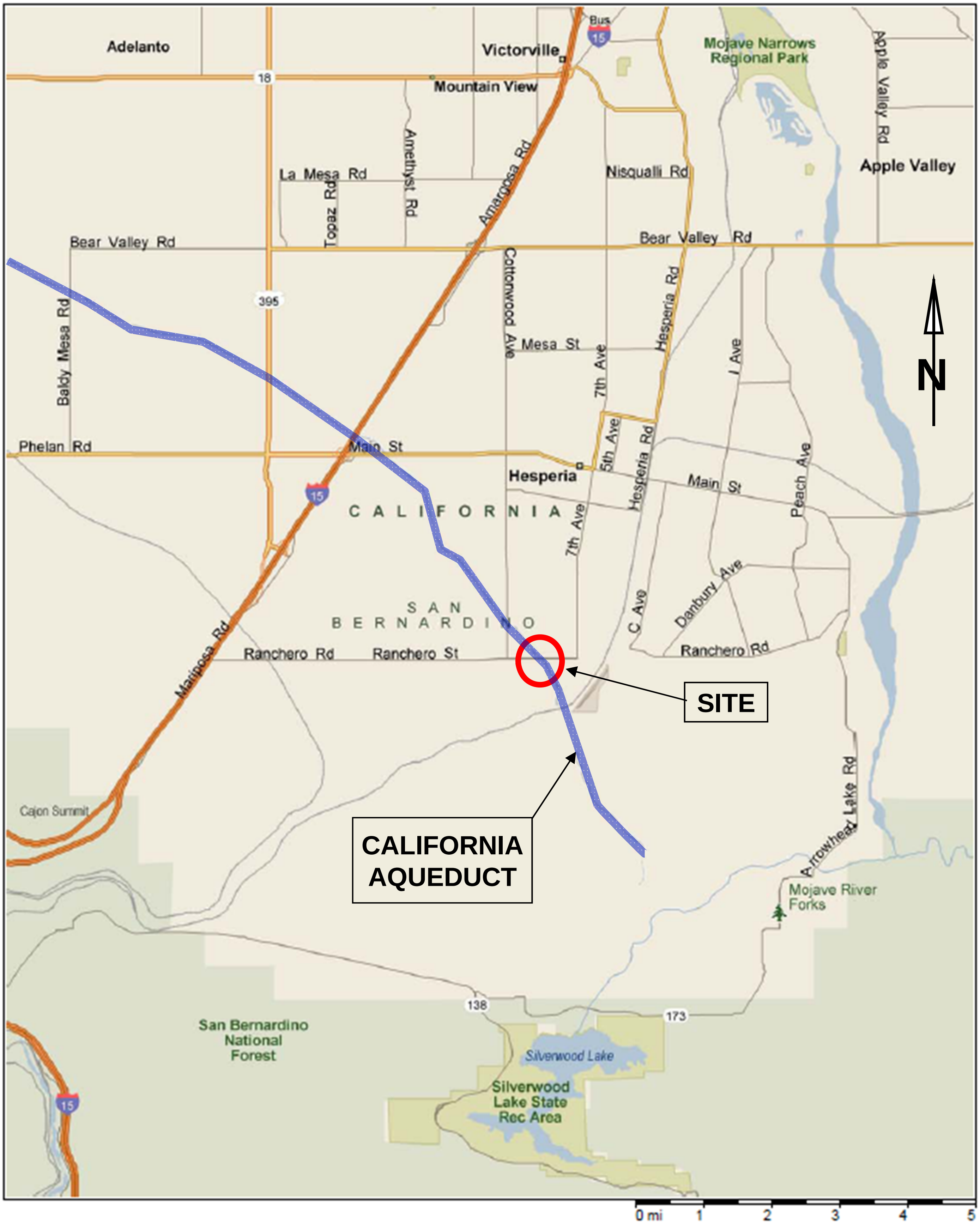
- Collection and review of existing geotechnical information;
- Field exploration consisting of drilling and logging exploratory borings;
- Laboratory testing of selected bulk and relatively undisturbed soil samples;
- Engineering calculations and analysis to develop foundation design and construction recommendations; and
- Preparation of this report presenting our findings, conclusions, and recommendations.

1.2 PROJECT DESCRIPTION

The City of Hesperia, in coordination with the County of San Bernardino and California Department of Water Resources (DWR), proposes to construct a new bridge to replace the existing Ranchero Road Bridge (Bridge No. 54C-0449). The bridge crosses over the existing California Aqueduct which is a concrete-lined trapezoidal channel approximately 19 ft deep with 2H:1V side slopes.

The existing two-span bridge was constructed in 1980 and is approximately 136 ft long and 33 ft wide. Abutments 1 and 3 and Pier 2 are supported on shallow foundations.

The new bridge structure is proposed to be a skewed single-span precast, prestressed wide-flange girder bridge with a length of approximately 155 ft and a width of approximately 137 ft. The structure is proposed to be supported on 24-inch diameter Cast-In-Drilled-Hole (CIDH) pile foundations. The west abutment will have two attached retaining walls on spread footings. Further improvements include widening of the bridge approach roadways which include construction of a new retaining wall on spread footing.



Ranchero Rd Bridge	SITE LOCATION MAP
Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Figure 1-1
Project No. 15-111	Date: June 2016

2.0 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 FIELD INVESTIGATION

No As-built Log-of-Test-Borings (LOTB) sheets exist for the present structure.

To obtain subsurface information, a field investigation consisting of five soil borings was performed by EMI on June 9 and 10, 2016 for the bridge and roadway along Ranchero Road. In addition, a surface grab sample was taken from the east shoulder of 11th Avenue for pavement design. Pertinent location information is summarized in Table 2-1. The log-of-test-borings (LOTB) sheets for the bridge and retaining walls are presented in Appendix A. Locations and letter-size logs of the pavement boring are also provided in Appendix A. The locations were scaled from plans provided.

Table 2-1. Geotechnical Exploration Information

<table>
<tr><th rowspan="2">Boring</th><th colspan="3">Approx. Location</th><th rowspan="2">Approx. Top of Boring Elevation (ft)</th><th rowspan="2">Borehole Depth (ft)</th><th rowspan="2">Ground Water Depth (ft)</th><th rowspan="2">Drilling Method</th></tr>
<tr><th>Station (ft)</th><th>Station Line</th><th>Offset (ft)</th></tr>
<tr><td>A-16-201</td><td>307+56</td><td rowspan="2">Ranchero Road</td><td>34 Lt.</td><td>3,457</td><td>30.5</td><td>NE</td><td>HSA</td></tr>
<tr><td>A-16-202</td><td>304+70</td><td>28 Lt.</td><td>3,462</td><td>31.5</td><td>NE</td><td>HSA</td></tr>
<tr><td>HA-16-203</td><td>22+50</td><td>11th Avenue</td><td>17 Rt.</td><td>3,464</td><td>5</td><td>NE</td><td>HA</td></tr>
<tr><td>A-16-204</td><td>302+18</td><td rowspan="3">Ranchero Road</td><td>22 Lt.</td><td>3,467</td><td>70.3</td><td>NE</td><td>HSA</td></tr>
<tr><td>A-16-205</td><td>299+42</td><td>26 Lt.</td><td>3,471</td><td>70.9</td><td>NE</td><td>HSA</td></tr>
<tr><td>A-16-206</td><td>296+60</td><td>8 Rt.</td><td>3,474</td><td>31.5</td><td>NE</td><td>HSA</td></tr>
</table>

Notes:
NE = Not Encountered; HSA = Hollow-Stem Auger; HA = Hand-Auger.

The hand-auger (HA) borings were drilled using a 3-inch diameter stainless steel hand-auger. The auger (A) borings were drilled using a truck-mounted CME 75 drill rig equipped with 8-inch diameter hollow-stem auger. Sampling was performed by alternating the Modified California Drive (MCD) sampler and Standard Penetration Test (SPT) sampler at 5 ft depth intervals. Drilling the on-site soils with this equipment required moderate effort.

Relatively undisturbed soil and bedrock samples were obtained using a 3.25-inch outer diameter MCD sampler lined with brass rings. Each of these brass rings is 1-inch long with a 2.5-inch inside diameter. The SPT sampler (1.4-inch inside diameter) was also used to obtain samples. The sampling interval was generally 5 ft alternating between MCD and SPT samplers. The MCD and SPT samplers were driven 18 inches into the ground or until refusal was encountered using a 140-lb automatic trip hammer free falling from a height of 30 inches. The number of blows to advance the sampler each 6 inches of penetration were recorded. The number of blows for the final 12 inches or shorter of driving was recorded on the LOTB sheet. Charts published by Winterkorn and Fang (1975) can be used to determine a reduction factor used to convert blowcounts recorded using the MCD sampler into SPT blowcounts. Using those charts, we obtained a reduction factor of 0.5 for coarse-grained soils.

2.2 LABORATORY TESTING

Soil samples considered representative of the subsurface conditions were tested to obtain or derive relevant physical and engineering properties. The following laboratory tests were conducted to supplement the observations recorded during the field investigation:

- In-situ Moisture Content and Unit Weight,
- Percent Passing No. 200 Sieve,
- Grain Size Analysis,
- Direct Shear,
- R-Value, and
- Soil Corrosivity (Minimum Resistivity, pH, Sulfate Content and Chloride Content).

The laboratory tests were conducted in general accordance with California Test Methods or American Society for Testing and Materials (ASTM) Standards. Laboratory test results are included in Appendix B.

3.0 GEOLOGY AND SEISMICITY

3.1 PHYSIOGRAPHY AND TOPOGRAPHY

The project area is located in the northwestern part of San Bernardino County in the city of Hesperia, California. Hesperia is located in the western part of the Mojave Desert physiographic province. The Mojave Desert province is a region of flat desert plains that is enclosed by surrounding mountain ranges, wide valleys, and two main fault trends that are oriented in a northwest-southeast and east-west direction. Dry lakes, also known as playas, are located in the low parts of the plains and valleys. The area between the playas and the mountains are composed of alluvial fans and coalesced fans (bajadas), which overlie rock shelves (pediments). The surrounding mountain ranges near the project area are the San Gabriel Mountains and San Bernardino Mountains, which are part of the Transverse Ranges. The project area is located on the northwestern side of the San Bernardino Mountains and northeastern side of the San Gabriel Mountains. Tectonic forces along the San Andreas Fault system border the San Gabriel Mountains and the San Bernardino Mountains.

Elevations vary from 2,500 to 3,500 ft on the plains and reach maximum elevation of approximately 4,500 ft in the hills. The highest point in the site region is within the San Bernardino Mountains, where elevations exceed 6,000 ft, and San Gabriel Mountains, where elevations exceed 8,000 ft. The elevations in the site vicinity range from 3,470 to 3,485 ft.

3.2 STRATIGRAPHY

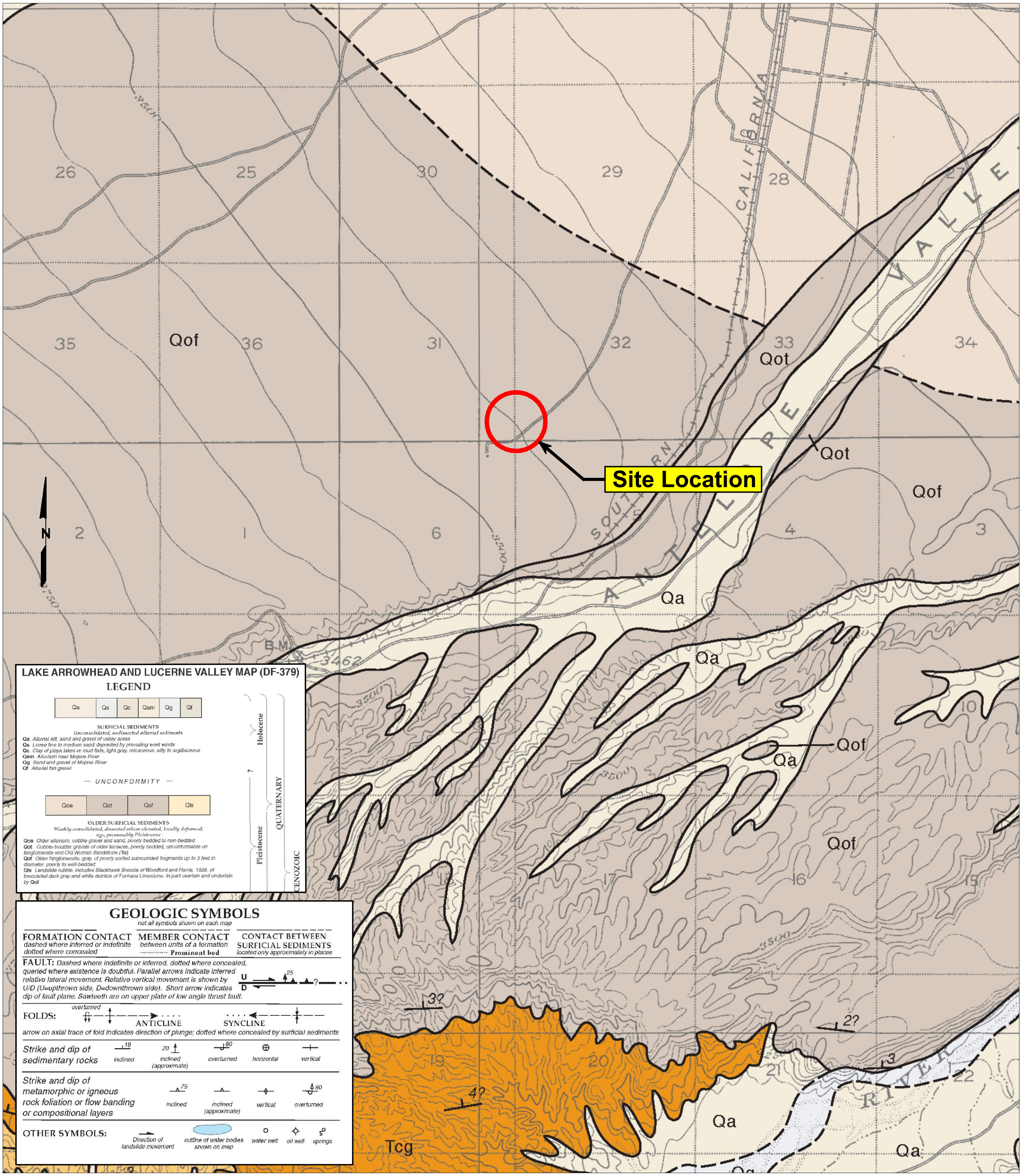
Figure 3-1 is a geological map that shows the distribution of geological formations and geological structure (Dibblee, 2008). The geologic units encountered within the project area are described as follows:

- Shoemaker Gravel (Qof): The Shoemaker Gravel Formation consists of older alluvial fan deposits that extended down from the San Gabriel Mountains. The soils are generally gravelly, gray to brown, vaguely bedded, poorly-sorted and composed of subrounded detritus of gneissic and plutonic rocks from the San Gabriel mountain terrane to the south.

- Alluvium (Qa, Qoa): The site is immediately underlain by alluvial fan and wash deposits, which includes brown to gray gravels, sands, and silty sands. These Holocene- and late Pleistocene age alluvial deposits are generally derived from nearby tributaries, including the Oro Grande Wash to the northwest.

- Artificial Fill (Af): Artificial fill was placed as part of the construction of the California Aqueduct. The fill materials were most likely derived from the nearby alluvial channel and fan deposits.

The site is underlain primarily by older alluvial deposits. These alluvial deposits generally consist of silty sand, sand with silt, sand, and sand with gravel.



REFERENCE: Dibblee T.W., 2008, Geologic Map of the 7.5 Minute Hesperia Quadrangle

0 4,000 8,000 FEET

SCALE 1" = 4,000'

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Ranchero Road Bridge Over California Aqueduct

REGIONAL GEOLOGIC MAP

Project No. 15-111

Date: June 2016

Figure 3-1

3.3 GEOLOGIC STRUCTURE

The Mojave Desert province consists of mountain ranges and desert plains physiography. The northern, western, and southern boundaries of the Mojave province are characterized by continuous mountain fronts uplifted along major faults (see Figure 3-2). The western boundary is defined by the San Andreas Fault, the northern boundary by the Garlock Fault, and the southern boundary by the North Frontal Fault system of the San Bernardino Mountains. The central-eastern boundary of the Mojave Desert consists of a series of sub-parallel dextral strike-slip faults and hill ranges. The eastern boundary is underlain by subcrustal discontinuities (Fuis, 1982), which suggest crustal structure or crustal weakness (Schell, 1994).

The major distinguishing structural aspect of the Mojave province is the prominence of long, northwesterly trending, right-lateral, strike-slip faults, most of which have experienced surface rupture during the Quaternary (Jennings, 2010). These faults have recently been referred to as the Eastern California Shear Zone (Dokka and Travis, 1990). Although these faults are believed to have originated as normal faults, they have experienced strike-slip motions since about Pliocene time. These strike-slip faults are relatively long and they are generally terminated at both the north and south margins by east-west trending faults (Schell, 1994).

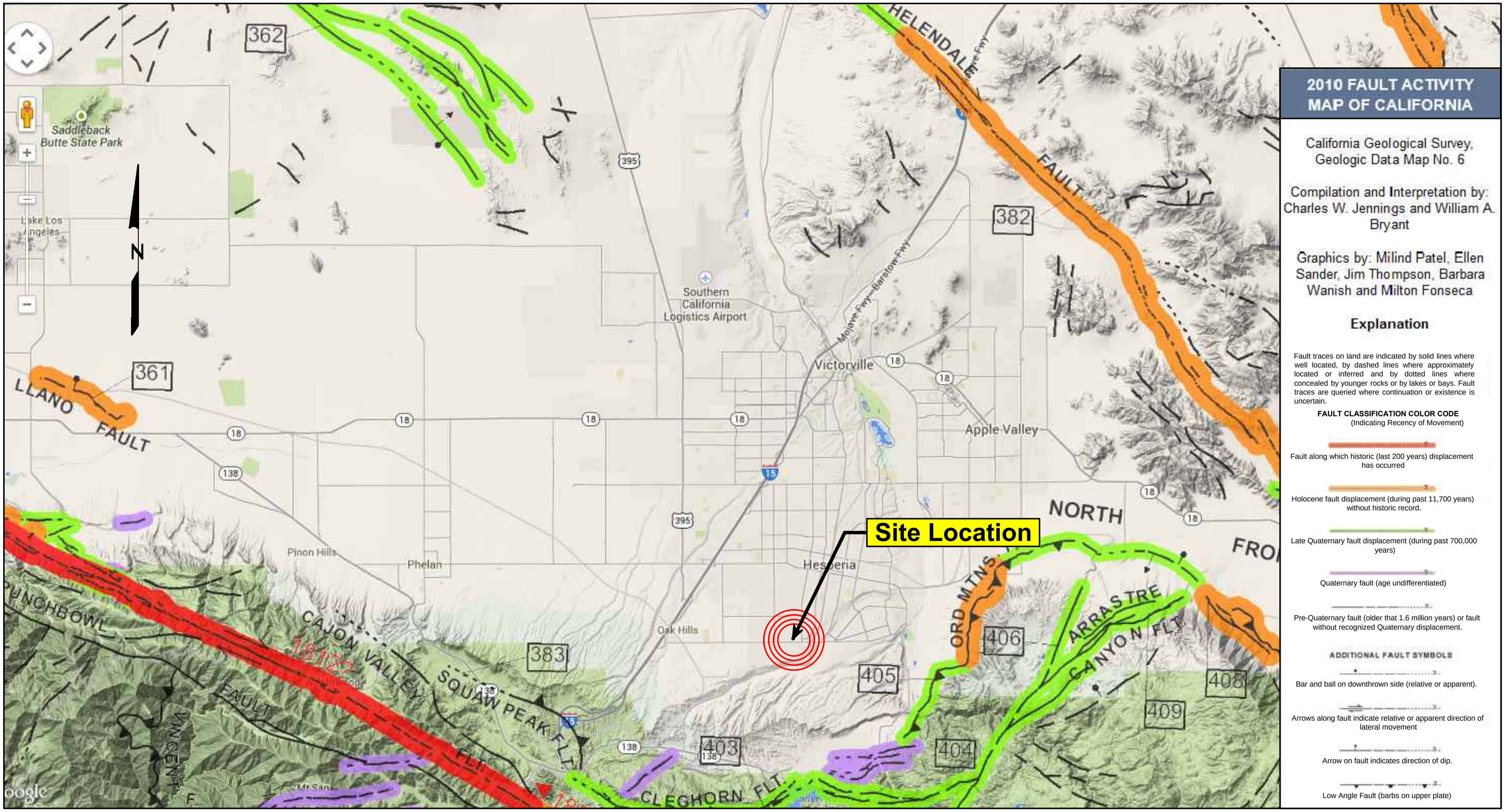
There are no active faults that are mapped or known near the project site, and the site is not located within an Earthquake Fault Zone. The most significant seismic hazard that could affect the project site is ground shaking resulting from distant earthquakes from major active faults, such as the San Andreas Fault, North Frontal Thrust Fault Zone, the Eastern California Shear Zone, the Garlock Fault Zone, or the Cleghorn Fault Zone.

3.3.1 Faulting

The project site does not cross any active surface faults. The closest known active faults to the project site are the Cleghorn Fault, the North Frontal Fault Zone, the San Andreas Fault, and the San Jacinto Fault Zone (see Figure 3-2).

The San Andreas Fault is a major dextral strike-slip fault, which is the boundary between the Pacific and the North American Plate. It is the dominant active fault throughout California, which extends approximately 680 miles from San Francisco to the northern Gulf of California in Mexico. The displacement across this fault zone is right-lateral with an estimated slip rate of about 24 mm per year. The San Andreas Fault is capable, but not limited to producing a magnitude of 7.9 earthquakes. Based on historical events on the San Andreas Fault, larger magnitudes of 7.7, in 1906, to 7.8, in 1857, are plausible (Ellsworth, 1990). The project site is located approximately 10.7 miles north of the fault zone.

The Cleghorn Fault zone is a sinistral strike-slip fault that is divided into two different sections, the Southern Cleghorn and the Northern Cleghorn. The fault is part of the San Andreas Fault system and is the principal fault in the northwestern part of the San Bernardino Mountains. The fault length is approximately 15.5 miles with a left lateral displacement and a slip rate of 1.0 to 5.0 mm per year. The project site is located approximately 5.6 miles north of the Cleghorn Fault.



Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Ranchero Road Bridge Over California Aqueduct

Project No. 15-111 | Date: July 2016

Regional Fault Map

Figure 3-2

The North Frontal Thrust Fault system is a complex zone of thrust, reverse, and dextral strike-slip fault splays that generally trend in an east-west direction. The fault is an approximate 50 miles long fault that extends along the northeastern flank of the San Bernardino Mountains. The average slip rate is between 0.2 and 1.0 mm per year and according to the probabilistic earthquake analysis of California Geological Survey (Cao et al., 2003) the fault is capable of producing a magnitude 7.2 earthquake. The project site is located approximately 5.8 miles west of the North Frontal Thrust system.

The San Jacinto Fault Zone is historically the most seismically active fault zone in California and passes as close as about 20 miles from the proposed project location. Segments of the San Jacinto Fault Zone extend from near San Bernardino on southeast more than 190 miles through the Imperial Valley and into northern Baja California, Mexico (Ziony and Yerkes, 1985). At its northern end, this right-lateral strike-slip fault appears to merge with the San Andreas Fault. Over the past century, the San Jacinto Fault Zone has produced at least 10 earthquakes of about magnitude 6 or greater. Geologic, geodetic and seismologic observations generally point to an average slip rate of 0.3 to 0.5 inches (8 to 12 millimeters) per year during Quaternary time. This fault zone is generally considered capable of generating earthquakes up to a magnitude of 7.7. The project site is located approximately 11.8 miles south of the San Jacinto Fault Zone.

3.4 SEISMICITY

The Mojave province is characterized by moderate to high seismicity. Seismicity levels are highest in the east-central part of the province, due to the series of northwest-trending sub-parallel dextral strike-slip faults. The length of the major faults in this province indicate that earthquakes of magnitudes larger than 7 are possible. The 1992 Landers earthquake with a magnitude of 7.3 ruptured about 70 km along the Camp Rock, Emerson, Homestead, and Johnson Valley faults. The 1999 Hector Mine earthquake had a magnitude of 7.1 and ruptured about 40 km along the Lavic Lake and Bullion faults. Another notable event was the 1947 Manix earthquake (M=6.2) which was associated with surface rupture on an east-west trending short fault east of Barstow. The 1857 Fort Tejon Earthquake of moment magnitude 7.8 ruptured a long section of San Andreas Fault from the Parkfield area in central California to the Wrightwood-San Bernardino area in the San Gabriel Mountains southwest of the project area. The southern end of this rupture was about 11.7 miles from the site.

3.5 SEISMIC HAZARDS

3.5.1 Surface Fault Rupture

Since no active faults are mapped at the project site, the potential for surface fault rupture at this bridge site is anticipated to be low. The proposed bridge does not fall within an Alquist-Priolo Earthquake Fault Zone or within 1,000 ft of an unzoned fault that is Holocene or younger in age, additional fault studies are not needed (Caltrans Memo to Designer 20-10, 2013a).

3.5.2 Liquefaction

There is no Seismic Hazard Zone map by the California Geological Survey for the site. Soils most susceptible to liquefaction are saturated low-density granular sands within 50 ft of the ground surface. Based on Section 4.2, groundwater levels are much deeper than 50 ft depth. Due to the lack of loose saturated granular soils, the site is found to have a low liquefaction potential.

3.5.3 Seismic Slope Stability

The site is relatively flat and there are no significant existing slopes adjacent to the bridge structure. The channel slopes are concrete-lined. Graded embankments will be constructed at the approaches and retained by new walls. Seismic slope stability is discussed in Section 5.7.2.

4.0 SUBSURFACE CONDITIONS

4.1 SOIL CONDITIONS

The proposed roadway surface elevation rises gradually from about El. 3,456 ft in the east to 3,477 ft in the west. Finished roadway grades will be about El. 3,480 ft at the west abutment and 3,477 ft at the east abutment. Based on the soil borings, the site is underlain by predominantly granular soils consisting of sands with varying amounts of fines and gravels. The gravels were observed to be angular to subrounded and may be fragments of cobbles.

The idealized soil profile and design strength parameters used for foundation design are presented in Table 4-1. These design parameters are equivalent design shear strengths based on SPT blowcount correlations (Lam and Martin, 1986) and direct shear test results. Engineering judgement was used to interpret the test results considering soil sample disturbance and that fact that direct shear tests were conducted on inundated soil samples.

Table 4-1. Idealized Soil Profile and Strength Parameters

Approx. Elevations At Abutments (ft)	Predominant Soil Type	Equivalent SPT Blowcount* (blows/ft)	Total Unit Weight (pcf)	Friction Angle (degree)	Cohesion (psf)
3,480 to 3,469	New Approach Fills	N/A	120	34	50
3,469 to 3,464	Medium Dense Silty Sand and Sand With Silt	10 to 19	120	32	50
3,464 to 3,435	Dense Silty Sand and Sand With Silt	14 to 50	120	33	50
3,435 to 3,415	Dense to Very Dense Silty Sand and Sand With Silt and few Gravel	32 to >50	125	34	50
3,415 to 3,397	Very Dense Silty Sand and Poorly Graded Sand with Gravel	44 to >100	120	36	50

Note:
* A correction factor of 0.5 was used to convert Modified California Drive sampler blowcounts to SPT blowcounts.

4.2 GROUNDWATER CONDITIONS

Groundwater was not encountered in the soil borings within 70.9 ft of depth (approximately El. 3,397 ft) explored (see LOTB sheets in Appendix A). Based on data from the California Department of Water Resources Water Data Library (2016), historical groundwater in the closest well (State Well No. 04N04W04F001S) located about 1.3 miles east of the site ranged between

407 ft and 420 ft below existing grade from 2005 to 2015. In addition, State Well No. 04N05W36R003S located about 1.1 miles west of the site showed groundwater levels ranging between 735 ft and 748 ft below existing grade from 1984 to 1996. These levels are below the proposed foundations, and groundwater was not considered in foundation design.

Groundwater depths during the construction phase may differ from those reported above because groundwater levels can fluctuate due to variations in seasonal precipitation, irrigation, groundwater injection or extraction, or numerous other man-made and natural influences.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 SEISMIC DESIGN CRITERIA

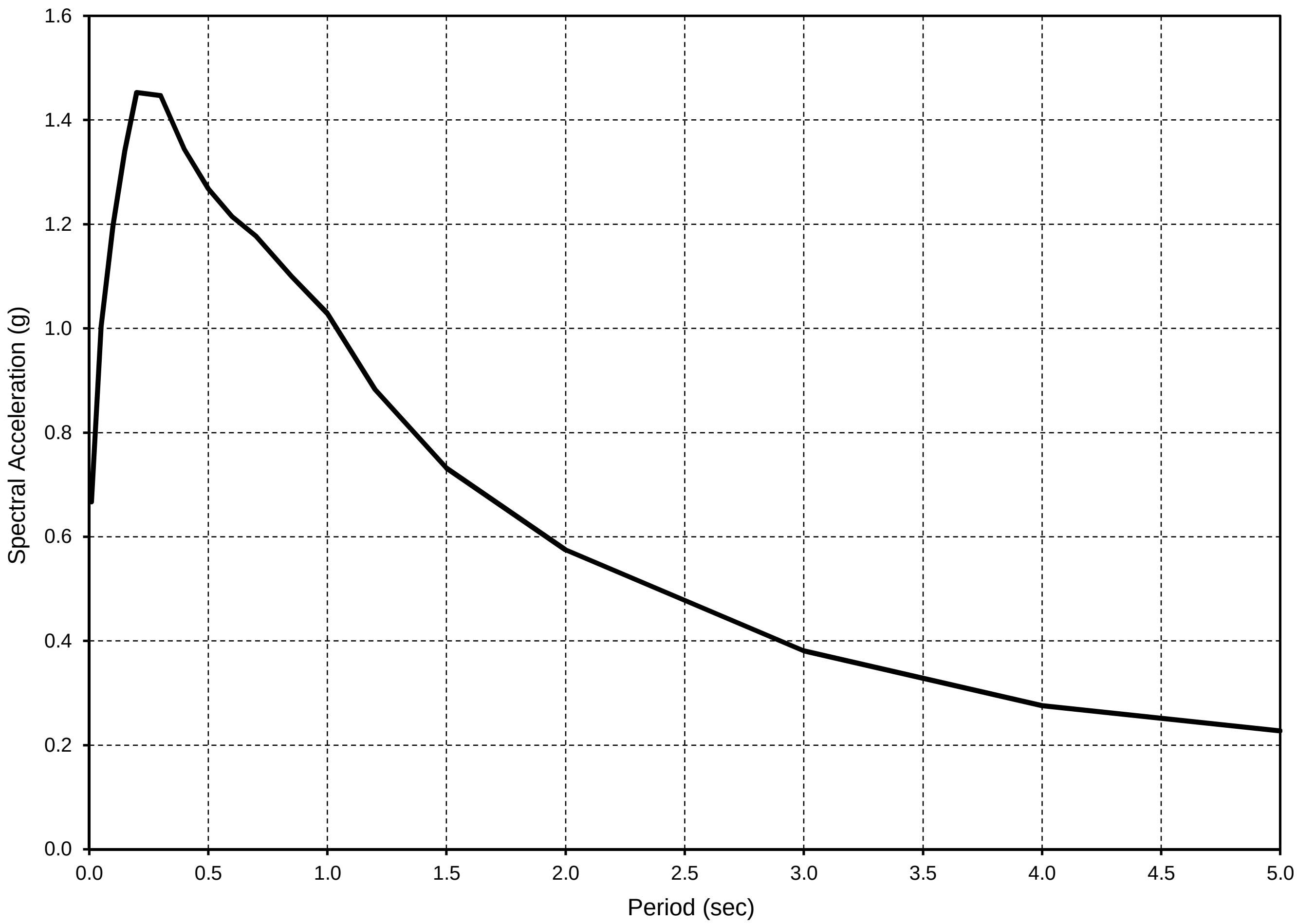
The design ARS curve for the new bridge was developed in accordance with the current Caltrans Seismic Design Criteria (SDC, 2013b) and methodology (2012d). The methodology consist of developing deterministic spectra for late-Quaternary faults in the 2012 fault database (Caltrans, 2012b; Merriam, 2012) and a probabilistic spectrum based on 5% probability of exceedance ground motion in 50 years.

Design deterministic spectra were developed for the three closest faults shown in Table 5-1. The table shows key fault parameters including their peak ground accelerations (PGA). These spectra were based on a small-strain shear wave velocity (V_s^{30}) of 300 m/sec (984 ft/sec) which was calculated using correlations with SPT blowcounts (Caltrans, 2012d) from the bridge LOTB in Appendix A.

Spectral acceleration values for the probabilistic response spectrum were calculated using the Caltrans tool based on USGS Interactive Deaggregation Tool (USGS, 2008). The final design ARS curve is the envelope of the deterministic and probabilistic spectra. The resulting design ARS spectrum is presented in Figure 5-1 together with the digitized coordinates. The design PGA is 0.667g.

Table 5-1. Fault Information Table

Fault ID	Fault Name	Maximum Magnitude	Fault Type	Distance (miles)	PGA (g)
279	North Frontal (West)	7.2	Reverse	5.8	0.329
301	Cleghorn Fault Zone (Northern Cleghorn Section)	6.7	Strike-Slip	5.6	0.286
325	San Andreas (San Bernardino Section)	7.9	Strike-Slip	10.7	0.263



Latitude = 34.3832 °
Longitude = -117.3294 °
Damping Ratio = 5%

Spectral Coordinates

Period (sec)	Acc. (g)	Period (sec)	Acc. (g)
0.010	0.667	0.700	1.190
0.050	0.999	0.850	1.115
0.100	1.188	1.000	1.045
0.150	1.331	1.200	0.899
0.200	1.442	1.500	0.748
0.250	1.443	2.000	0.590
0.300	1.443	3.000	0.391
0.400	1.346	4.000	0.283
0.500	1.274	5.000	0.233
0.600	1.225		

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Ranchero Road Bridge	Design ARS Curve
	Project: 15-111 / Date: 06/21/16	Figure 5-1

5.2 SOIL CORROSIVITY

Two representative soil samples were tested for pH, minimum resistivity, soluble chloride content and soluble sulfate content. The test results are summarized in Table 5-2.

Table 5-2. Soil Corrosion Test Results

Boring	Sample Depth (ft)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Soluble Sulfate Content (ppm)	Soluble Chloride Content (ppm)
A-16-201	0-5	SC	9,319	7.4	53	39
A-16-205	20	SM	12,225	8.2	55	40

The Caltrans Corrosion Guidelines (2012c) classify soil as corrosive if the soluble chloride content is higher than 500 ppm, or if the soluble sulfate content is more than 2,000 ppm, or if the pH value is less than 5.5. Based on the existing test results found and the Caltrans criteria, the on-site soils are not expected to be corrosive to bare metals and concrete.

Corrosion-resistant Type II modified cement for sulfate concentration from 0 to 1,499 ppm (Caltrans Corrosion Guidelines, 2012c) should be sufficient for concrete foundations in contact with soil. The minimum concrete cover over reinforcement for structural elements in contact with onsite soils should be in accordance with Table 5.12.3-1 of the Caltrans Amendments (2014a) to AASHTO (2012) for "Non-Corrosive Atmosphere/Soil/Water." The concrete cover for drilled concrete piles should be at least 3 inches per Section 10.8.1.3-1 of the Caltrans Amendments to AASHTO.

5.3 SCOUR EVALUATION

The channel is presently concrete lined and will remain lined after bridge construction. Therefore, scour potential is not a design issue.

5.4 FOUNDATION DESIGN

5.4.1 As-Built Foundation Data

The existing bridge is supported on shallow foundations. The as-built foundation data from the as-built plans are summarized in Table 5-3.

Table 5-3. As-Built Foundation Data

Support Location	Bottom of Footing Elevations (ft)	Allowable Footing Pressure (ksf)
Abutment 1	3,458±	6.0
Pier 2	3,441.1	8.0
Abutment 3	3,458±	6.0

5.4.2 Foundation Type

Large axial and lateral loads are anticipated for the new bridge. As a result, pile foundations are recommended to support both abutments. It is our understanding that the Department of Water Resources does not allow pile driving at this site. Therefore, CIDH piles with a 24" diameter are recommended.

Two wing walls will cantilever off Abutment 2. Foundation design for other retaining walls is addressed in Section 5.6.

5.4.3 Foundation Data

LRFD Service-I Limit State, Strength Limit State, and Construction Limit State load combinations should be used for design of the new abutment foundations following Caltrans State Amendments (2014a) to AASHTO LRFD Bridge Design Specifications (2012).

The foundation design data sheet provided by the structural designers is presented in Table 5-4. Foundation factored design loads were provided by the structural designers and they are presented in Table 5-5. The information presented in Table 5-4 and Table 5-5 follows Caltrans Memo To Designers 3-1 format (2014b).

Table 5-4. Foundation Design Data Sheet

Support No.	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (inch)	Number of Piles per Support
				B	L		
Abut 1	24-inch CIDH	3,465.30	3,458.50	14	195.93	1	80
Abut 2	24-inch CIDH	3,465.00	3,458.50	14	192.80	1	65

Table 5-5. Foundation Factored Design Loads

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	10,610	10,025	14,450	280	0	20	N/A	N/A	N/A	N/A
Abut 2	9,740	9,100	13,360	280	0	0	N/A	N/A	N/A	N/A

5.4.4 Axial Pile Capacity

Based on the abutment pile layout sheets provided by the structural designer and the Caltrans Amendments (2014a) to AASHTO (2012), an axial group reduction factor of 0.67 was used at both abutments.

Foundation design recommendations are presented in Table 5-6. The Pile Data Table for the contract plans is presented in Table 5-7. The nominal resistances for the piles are controlled by the Construction Limit State "Compression Maximum Per Pile". The pile capacity is also based on soil resistance only and may be further limited by the pile-head connection details and the strength of the pile materials.

Table 5-6. Foundation Design Recommendations

Support No.	Pile Type	Cut-Off Elev. (ft)	Service-I Limit State Load per Support (kips)		Total Perm. Support Settle. (inch)	Required Factored Nominal Resistance (kips)				Design Tip Elev. (ft)	Spec. Tip Elev. (ft)
						Strength/ Construction		Extreme Event			
			Total	Perm.		Comp. (ϕ=0.7)	Tension (ϕ=0.7)	Comp. (ϕ=1.0)	Tension (ϕ=1.0)		
Abut 1	24-inch CIDH	3,458.50	10,875	10,025	1	400	50	0	0	3,414 (a-I) 3,448 (b-I) 3,440 (c) 3,430 (d)	3,414
Abut 2	24-inch CIDH	3,458.50	9,875	9,100	1	400	0	0	0	3,414 (a-I) 3,440 (c) 3,430 (d)	3,414

Notes:

1. Design Tip Elevations are controlled by the following demands: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (c) Settlement, and (d) Lateral Load.
2. The Specified Tip Elevations shall not be raised.

Table 5-7. Pile Data Table

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)
		Compression	Tension		
Abut 1	24-inch CIDH	400	0	3,414 (a) 3,448 (b) 3,440 (c) 3,430 (d)	3,414
Abut 2	24-inch CIDH	400	0	3,414 (a) 3,440 (c) 3,430 (d)	3,414

Notes:
1. Design Tip Elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement, and (d) Lateral Load.
2. The Specified Tip Elevation shall not be raised.

5.4.5 Lateral Pile Capacity

Lateral pile analysis of the abutment piles was performed using the computer program LPILE (Ensoft, 2015). Based on the pile layout sheets provided by structural designers, an average group efficiency factor of 0.60 can be used at Abutment 1 and 0.68 at Abutment 2 for any lateral loading direction.

Pile-head deflection, shear, and bending moment are summarized in Table 5-8. Lateral solutions are provided for pile-head deflections for 0.25 and 1 inch. The solutions presented in Table 5-8 are entirely based on soil resistance and linear pile properties. Therefore, these values may be limited by the flexural strength (plastic moment) of the piles and pile-head connection details.

Table 5-8. Lateral Pile Solutions for Abutment Piles

Pile Head Condition	Pile Head Deflection (inch)	Pile Head Shear (kips)	Maximum Moment (kip-in)	Depth to the Maximum Moment from Pile Top (ft)
Abut 1 Free-Head Condition	¼	23	820	5.7
	1	56	2,800	6.6
Abut 1 Fixed-Head Condition	¼	53	2,475	0.0
	1	130	7,700	0.0
Abut 2 Free-Head Condition	¼	25	865	5.7
	1	61	2,950	6.6
Abut 2 Fixed-Head Condition	¼	57	2,610	0.0
	1	140	8,100	0.0

5.5 BRIDGE ABUTMENT WALL DESIGN

5.5.1 Abutment Earth Pressures

If the abutment walls and wing walls are free to move laterally at the top, a static active lateral earth pressure of 36 pcf equivalent fluid pressure is recommended for a free-draining, level and compacted backfill. If lateral movement at the top of walls is restrained, a static lateral earth pressure for this backfill of 55 pcf can be used. Uniform lateral pressure due to traffic loading, equivalent to a vertical pressure produced by at least 2 ft of earth with a soil unit weight of 120 pcf, should be added to the above lateral earth pressure. Using an active earth pressure coefficient of 0.3, the recommended uniform lateral earth pressure due to traffic loading is 72 psf.

Seismic lateral active earth pressure was estimated using the trial wedge method and horizontal seismic coefficient equal to one-half of the design PGA from Section 5.1. For a level and free-draining backfill at the abutments, an incremental seismic earth pressure of 30 pcf equivalent fluid pressure is recommended. This incremental earth pressure should be added to the static active earth pressure to obtain the total lateral earth pressure acting on the wall under seismic loading.

5.5.2 Passive Resistance of Abutment Backfill

Under seismic loading, an ultimate passive earth pressure of 5 ksf may be used for the approach backfill and abutment walls with a height equal to or greater than 5.5 ft. For abutment walls with heights less than 5.5 ft, the passive pressure may be calculated proportionally (e.g., for a 4-ft high wall, the passive pressure is [4/5.5] x 5 ksf = 3.64 ksf). The horizontal movement at which the maximum passive pressure is expected to be fully mobilized can be determined following the procedure outlined in Section 7.8.1 of the Caltrans SDC (2013b).

5.6 RETAINING WALLS

Two cantilevered retaining walls are proposed to retain the west bridge approach behind the west abutment. RW No. 1 at the north side is 29 ft long and heights vary from 4 to 14 ft. RW No. 2 at the south side is 21 ft long and heights vary from 4 to 12 ft.

A third cantilevered retaining wall is proposed to retain the south side of the western approach. RW No. 3 is 240 ft long and has heights of 6 and 8 ft. The wall is proposed to have a soundwall on top following Retaining Wall Type 1SWB-Details per Caltrans Standard Drawing XS 14-220 (2014c).

5.6.1 Lateral Earth Pressures

A static active lateral earth pressure of 36 psf per foot of depth can be used for free-draining, level and compacted backfill behind the cantilevered walls. If applicable, a uniform lateral pressure due to traffic loading, equivalent to a vertical pressure produced by at least 2 ft of earth with a soil unit weight of 120 pcf, should be added to the above lateral earth pressure. Using an active earth pressure of 0.3, the recommended uniform lateral earth pressure due to traffic loading is 72 psf. For incremental seismic lateral earth pressure, a lateral pressure of 30 psf is recommended for a free draining, level and compacted backfill behind the cantilevered wall.

5.6.2 Lateral Resistance

Lateral active loads can be resisted by frictional resistance acting along the base of the footing and passive lateral earth pressure. To determine frictional resistance along the base, a sliding coefficient of 0.55 is recommended. Passive earth pressure against the embedded wall stem and footing below level ground in front of the wall can be based on 400 pcf equivalent fluid pressure. Resistance factors conforming to Section 11 of the California Amendments (2014a) to AASHTO (2012) should be applied to the frictional and passive resistances.

5.6.3 Spread Footing Design

The site peak ground acceleration of 0.667g exceeds the maximum design PGA of 0.6g used for Caltrans Standard Plan (2015a) Type 1 retaining walls. As a result, all four walls are proposed to be Caltrans Modified Type 1 walls. RW No. 3 will also follow Type 1SWB-Details per Caltrans Standard Drawing XS 14-220 (2014c). Per Caltrans policy, the LRFD Service-I Limit State, Strength Limit State, and Extreme Event Limit States load combinations should be used for the design of the retaining walls (Caltrans 2014a Amendments to AASHTO 2012). The foundation design data was provided by the structural designers and is presented in Table 5-9. The foundation design loads are provided by the structural designers and presented in Table 5-10.

Table 5-9. Spread Footing Foundation Data for Walls

Retaining Wall No.	Design Height (ft)	Finished Grade Elevation (ft)	Bottom of Footing Elevation (ft)	Footing Dimensions		Permissible Settlement under Service Load (inch)
				Width	Length (ft)	
1	4	3,479.15	3,476.42	6’-10”	5	1
	6	3,479.10	3,472.42	7’-0”	8	1
	10	3,477.41	3,468.42	7’-7”	8	1
	14	3,474.75	3,464.08	9’-7”	8	1
2	4	3,478.10	3,473.17	6’-10”	5	1
	6	3,478.03	3,469.17	7’-0”	8	1
	12	3,476.75	3,464.42	8’-4”	8	1
3	6	3,476.00	3,472.50	6’-9”	100	1
	8	3,476.00	3,472.50	7’-3”	140	1

Table 5-10. Spread Footing Foundation Data for Walls

Retaining Wall No.	Design Height (ft)	Service 1 Limit State		Strength/Construction Limit State		Extreme Event Limit State 1, 2	
		Effective Foundation Width (ft)	Permissible Net Contact Stress (Settlement) (ksf)	Effective Foundation Width (ft)	Factored Gross Nominal Bearing Resistance (ϕ=0.55) (ksf)	Effective Foundation Width (ft)	Factored Gross Nominal Bearing Resistance (ϕ=1.0) (ksf)
1	4	6.8	0.7	6.6	1.6	5.2, 2.6	1.1, 2.2
	6	6.5	1.0	5.0	1.8	4.7, 2.7	1.5, 2.6
	10	6.0	1.6	3.0	3.3	3.1, 2.9	3.4, 3.6
	14	7.5	2.1	4.3	3.8	3.2, 5.2	5.3, 3.3
2	4	6.8	0.7	6.6	1.6	5.2, 2.6	1.1, 2.2
	6	6.5	1.0	5.0	1.8	4.7, 2.7	1.5, 2.6
	12	6.3	2.0	3.2	4.0	2.8, 3.7	4.8, 3.6
3	6	6.5	1.0	5.0	1.8	4.7, 2.7	1.5, 2.6
	8	6.2	1.3	3.6	2.3	3.9, 2.8	2.2, 3.1

Soil settlement calculations were performed for these walls using the Service Limit State bearing stress and effective footing widths to determine net contact stresses for 1 inch of permissible footing settlement. Overexcavation is recommended below the bottom of footing as shown in Table 5-11 to limit settlements to 1 inch. Construction recommendations are provided in Section 6.1.

Table 5-11. Minimum Overexcavation Recommendations for Walls

Retaining Wall No.	Design Height (ft)	Minimum Overexcavation Depth Below Footing Bottom (ft)
1, 2	All	1.0
3	6	1.5
	8	2.0

Soil bearing capacity calculations were also performed using the effective footing widths for the Strength Limit and Extreme Event states. The corresponding Gross Nominal Bearing Resistances of the foundation soils are provided in Table 5-12. The structural designer should check the Net Uniform Bearing Stresses do not exceed the Service Permissible Net Contact Stress, and the Gross Uniform Bearing Stresses do not exceed the Gross Nominal Bearing Resistances for the Strength/Construction Limit and Extreme Event states.

Table 5-12. Spread Footing Bearing Capacities for Walls

Retaining Wall No.	Design Height (ft)	Service 1 Limit State		Strength/Construction Limit State		Extreme Event Limit State 1, 2	
		Effective Foundation Width (ft)	Service Permissible Net Contact Stress (Settlement) (ksf)	Effective Foundation Width (ft)	Strength/ Construction Factored Gross Nominal Bearing Resistance (ϕ=0.55) (ksf)	Effective Foundation Width (ft)	Extreme Event Factored Gross Nominal Bearing Resistance (ϕ=1.0) (ksf)
1	4	6.8	2.3	6.6	5.0	5.2, 2.6	7.0, 3.5
	6	6.5	1.8	5.0	3.8	4.7, 2.7	6.5, 3.7
	10	6.0	2.1	3.0	5.0	3.1, 2.9	7.0, 6.7
	14	7.5	2.2	4.3	6.0	3.2, 5.2	7.1, 9.0
2	4	6.8	2.3	6.6	5.0	5.2, 2.6	7.1, 3.5
	6	6.5	1.8	5.0	3.8	4.7, 2.7	6.5, 3.7
	12	6.3	2.4	3.2	5.2	2.8, 3.7	6.6, 7.8
3	6	6.5	1.5	5.0	3.8	4.7, 2.7	6.4, 5.3
	8	6.2	1.4	3.6	2.8	3.9, 2.8	5.3, 3.8

5.7 EMBANKMENT SETTLEMENT AND SLOPE STABILITY

5.7.1 Settlement and Settlement Period

Based on the design information provided by the structural designer, embankment fills will need to be placed to construct the roadway approaches (Ranchero Rd, 11th Avenue, and maintenance road). The main approach embankments along Ranchero Road will be partially retained by Walls No. 1 to 3. Per the civil designer, the open finished slopes will have a 2H:1V or flatter side slopes and a maximum height of 10 ft. The endslopes at both abutments are the channel concrete liners which have a 2H:1V gradient and 19 ft height.

Because the subsurface soils are granular, ground settlements due to new fills are expected to be small and to occur during embankment construction. Based on the data available, maximum calculated ground subsidence due to new fills is estimated to be about 1/2 inch and no settlement period is required for pile construction.

5.7.2 Slope Stability

Global stability analyses were conducted for both static and pseudo-static conditions for the bridge approach embankment for potential deep-seated failures below the abutment footing. The soil strength parameters in Table 4-1 were used in the static and pseudo-static analysis.

For the static condition with a 2-foot traffic surcharge, the calculated factor of safety against a deep-seated global failure exceeded the minimum required factor of 1.5. The seismic condition

17800 Newhope Street, Suite B, Fountain Valley, California 92708 • Tel: (714) 751-3826 • Fax: (714) 751-3928

was modeled using pseudo-static loading with a seismic coefficient equal to 0.222g (which is one-third of the horizontal PGA from Section 5.1 following Caltrans Guidelines for Structure Foundation Reports, 2009). Analysis indicates that the calculated factor of safety is greater than the required minimum factor of 1.1.

Surficial stability is not a design concern for embankment slopes with a gradient of 2H:1V or flatter. To promote surficial stability, proper surface drainage devices and erosion control should be implemented. Erosion control in accordance with Section 21 of Caltrans Standard Specifications (2015b) is recommended.

5.8 PAVEMENT STRUCTURAL SECTION

The new roadway may be constructed using a flexible (asphalt concrete) composite pavement structural sections or rigid sections underlain by compacted subgrade.

Two bulk soil samples of shallow existing soils in the area of proposed improvements were tested to determine their R-value. The locations are shown in Appendix A. The measured R-values are shown in Table 5-13.

Table 5-13. R-Value Test Results

Boring No.	Approx. GSE (ft)	Sampling Depth (ft)	Soil Type	R-Value
A-16-201	3,457	0 to 5	Clayey Sand	24
A-16-205	3,471	0 to 5	Clayey Sand	25

New flexible and rigid structural pavement sections were determined in accordance with Chapter 630 and 620 of the Caltrans Highway Design Manual (2015c), respectively. A design R-value of 20 was used for design of new structural pavement sections. The civil designer specified a traffic index (TI) of 12.0 for Ranchero Road, 8.0 for 11th Avenue, and 6.0 for Kern Avenue and Cul-De-Sac (see bore location plan in Appendix A). The resulting recommended pavement structural sections are given in Table 5-14.

Table 5-14. Recommended Pavement Structural Sections

Traffic Index	Undrained Pavement Structural Sections: Minimum Design R-Value	Undrained Pavement Structural Sections: Flexible Section Thicknesses	Rigid Section Thicknesses
12.0	20	1.00’ HMA-A / 1.05’ AB	0.90’ JPCP / BB / 0.35’ LCB / 0.60’ AS
8.0	20	0.45’ HMA-A / 1.05’ AB	0.75’ JPCP / BB / 0.35’ LCB / 0.60’ AS
6.0	20	0.30’ HMA-A / 0.80’ AB or 0.35’ HMA-A / 0.70’ AB	0.75’ JPCP / BB / 0.35’ LCB / 0.60’ AS

Notes:
HMA-A = Hot-Mix Asphalt Type A
AB = Class-2 Aggregate Base
BB = Bond Breaker
AS = Aggregate Subbase
LCB = Lean Concrete Base
JPCP = Jointed Plain Concrete Pavement

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 EARTHWORK

Earthwork should be performed in accordance with applicable Sections of the Greenbook (2015) and Section 19 of the Caltrans Standard Specifications (2015b). Appropriate measures should be taken to prevent damage to adjacent structures and utilities. Any design and construction of temporary sloping, sheeting, or shoring should be made the contractor's responsibility. It should be noted that it is the responsibility of the contractor to oversee the safety of the workers in the field during construction. The contractor shall conform to all applicable occupational and health standards, rules, regulations, and orders established by the State of California. In addition, other State, County, or Municipal regulations may supersede the recommendations presented in this section. If a trench shoring design plan is required, the geotechnical consultant should review the plan to confirm that recommendations presented in this report have been applied to the design.

Groundwater was not encountered above approximately El. 3,404 ft explored during the field investigation conducted for this project. Groundwater is not expected to be encountered during foundation construction. However, groundwater level can fluctuate due to seasonal rainfall, groundwater recharge and man-made influences. Should groundwater be encountered, it should be controlled in accordance with Section 19-3.03B(5) of the Caltrans Standard Specifications (2015b).

In fill areas, compressible surficial materials including topsoil, loose alluvium, dry or saturated soils, uncertified fill, and otherwise unsuitable materials should be completely removed prior to placing compacted fill. Overexcavations should be observed by qualified geotechnical personnel to confirm that unsuitable materials are removed and that firm and unyielding subgrade are exposed. Actual depths and extent of the remedial removals should be determined in the field by qualified geotechnical personnel.

The horizontal limits of the overexcavation below the retaining wall footing bottom should begin one foot from each edge of the footing bottom and extending downward at a 45^{o} imaginary plane until the plane intersects the recommended minimum overexcavation depth. Caltrans Structure Backfill should be used to backfill the overexcavation. Caltrans Structure Backfill should be compacted to a minimum relative compaction of 95% of maximum density as determined by Caltrans Test Method 216. The excavation bottom should be proof rolled prior to backfilling.

Excavation bottoms should be cleaned of loose soils and debris and should be observed to be competent and unyielding prior to placing compacted fill. Prior to placing backfill, bottoms of overexcavations should be scarified to a minimum depth of 8 inches, moisture-conditioned to near optimum moisture content, and compacted in place to at least 90% relative compaction based on maximum densities determined using CT 216.

Subgrade soil for roadway pavement should have a minimum R-value of 20. Materials and construction methods (compaction and placement) of subgrade soil should follow Section 19 of the Caltrans Standard Specifications (2015b). Subgrade should be inspected and tested by qualified geotechnical personnel during grading to verify the minimum design R-value and the required minimum relative compaction. For rigid sections, a bond breaker should be placed between the JPCP and the LCB following Section 36-2 of the Caltrans Standard Specifications.

6.2 PILE CONSTRUCTION

Construction of CIDH piles should follow Section 49-3 of the Caltrans Standard Specifications (2015b). To improve CIDH pile construction, a 3 inch concrete cover over reinforcement should be provided per Caltrans State Amendments Table 10.8.1.3-1 (2014a) to AASHTO (2012).

Groundwater was not encountered above approximately El. 3,397 ft during the field investigation conducted for this project. Groundwater is not expected to be encountered during pile construction. However, should groundwater be encountered within the pile length, “wet” construction such as using slurry displacement method would be necessary. Caltrans standard practice for “wet” construction includes PVC tubings installed within the reinforcement cage for gamma gamma (GGL) testing to detect anomalies in concrete for CIDH piles installed below water (Caltrans, 2014b).

Caving soils were encountered at Borings A-16-204 and A-16-205 below approximately El. 3,426 ft. The on-site earth materials include coarse-grained soils such as poorly-graded sands and gravels. The gravels may be fragments of cobbles. In addition, loose soils were encountered at shallow depth. These materials are also susceptible to caving. The Contractor should be experienced in dealing with caving soils. Soil caving should be controlled expeditiously. The Contractor may elect to use temporary casing with smooth walls to control soil caving. Vibratory and oversized predrilling techniques for casing installation are not allowed. Temporary casing should be placed tight in the borehole. The casing should be pulled as the concrete is being poured while always maintaining at least a 5-foot head of concrete inside the casing. If any boring becomes bell-shaped and cannot be advanced due to severe caving, all loose material should be removed from the bottom of the boring and the caved region filled with low strength sand-cement slurry. Drilling may continue when the slurry has reached its initial set.

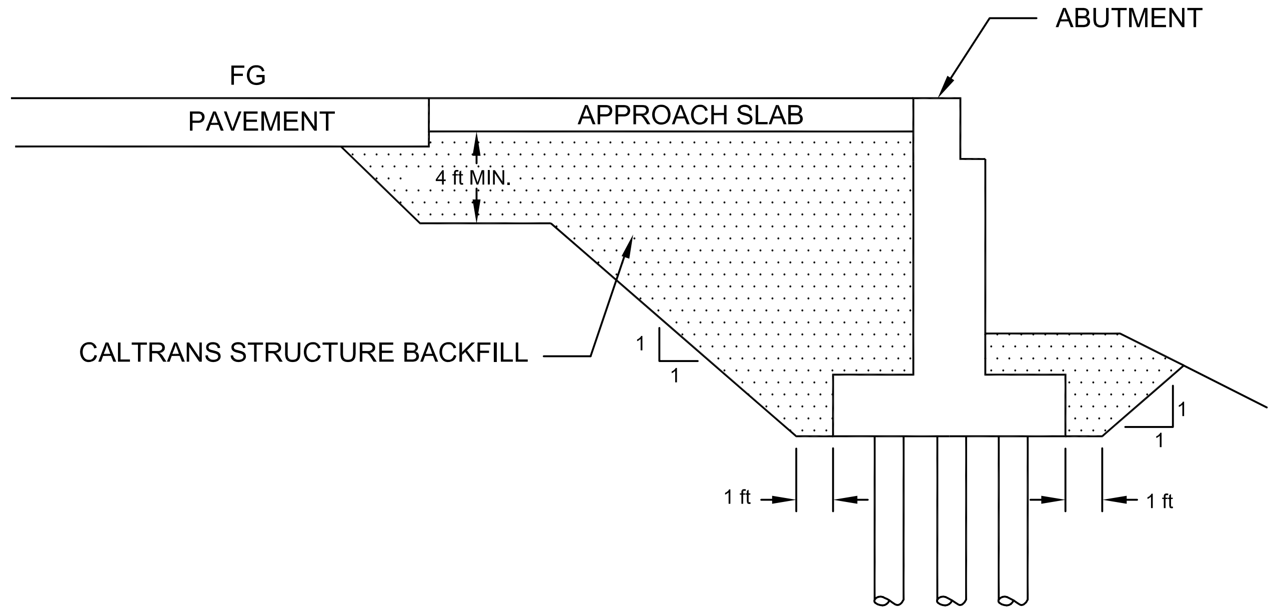
Loose soils should be cleaned from the bottom of the borings. The bottom of the drilled hole should be inspected and approved by a qualified person prior to installation of reinforcement. Extreme care in drilling, placement of steel, and the pouring of concrete is essential to avoid excessive disturbance of pile boring walls. Concrete placement by pumping or tremie tube to the bottom of the pile borings is recommended.

Pile construction should proceed expeditiously. Pile borings should not be left open unsupported between work shifts. The pile-reinforcing cage should be installed and the concrete pumped immediately after drilling is completed. No boring should be drilled until concrete in an adjacent pile has attained its initial set.

6.3 BACKDRAIN AND BACKFILL REQUIREMENTS FOR WALLS

Caltrans Structure Backfill should be used as backfill material behind the bridge abutment walls. The backfill materials should be placed as shown in Figure 6-1. Backfill should be compacted in accordance with Section 19-5 of the Caltrans Standard Specifications (2015b). Backfill should be placed in loose lifts not exceeding 8 inches in thickness, moisture-conditioned to near-optimum moisture content, and compacted to at least 95% relative compaction. The relative compaction should be based on the maximum density determined by California Test Method 216. Jetting or flooding to compact backfill is not recommended. Heavy compaction equipment, such as vibratory rollers, dozers, or loaders, should not be used adjacent to the abutment walls and retaining walls in order to avoid damaging the walls due to large lateral earth pressures.

Backdrains should be installed behind the abutment walls and retaining walls to relieve hydrostatic pressure. Backdrains are recommended to be in accordance with Bridge Detail 3-1 on Sheet B0-3 per Caltrans Standard Plans (2015a) or the geocomposite drain alternative per Section 6 of the Caltrans Bridge Design Details (1992).



NO SCALE

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	RANCHERO RD BRIDGE	BRIDGE ABUTMENT WALL BACKFILL
	Project No. 15-111 / Date: June 23, 2016	FIGURE 6-1

6.4 REVIEW OF CONSTRUCTION PLANS

Recommendations contained in this report are based on draft plans. The geotechnical consultant should review the final construction plans and specifications in order to confirm that the general intent of the recommendations contained in this report have been incorporated into the final construction documents. Recommendations contained in this report may require modification or additional recommendations may be necessary based on the final design.

6.5 GEOTECHNICAL OBSERVATION AND TESTING

It is recommended that inspections and testing be performed by qualified geotechnical personnel during the following stages of construction:

- Grading operations, including excavations and placement of compacted fill,
- Shoring installation,
- Removal or installation of support of buried utilities or structures,
- Pile drilling prior to placement of steel reinforcement,
- Footing excavations,
- Backdrain installation and wall backfill,
- Placement of pavement, and
- When any unusual subsurface conditions are encountered.

7.0 LIMITATIONS

This report is intended for use by the City of Hesperia and Athalye Consulting Engineering Services for design and construction of the Ranchero Road Bridge. This report is based on the project as described and the information obtained from the exploratory borings at the approximate locations indicated on the attached LOTB sheet. The findings and recommendations contained in this report are based on the results of the field investigation, laboratory tests, and engineering analyses. In addition, soils and subsurface conditions encountered in the exploratory borings are presumed to be representative of the project site. However, subsurface conditions and characteristics of soils between exploratory borings can vary. The findings reflect an interpretation of the direct evidence obtained. The recommendations presented in this report are based on the assumption that an appropriate level of quality control and quality assurance (inspections and tests) will be provided during construction. EMI should be notified of any pertinent changes in the project plans or if subsurface conditions are found to vary from those described herein. Such changes or variations may require a re-evaluation of the recommendations contained in this report.

The data, opinions, and recommendations contained in this report are applicable to the specific design element(s) and location(s) which is (are) the subject of this report. They have no applicability to any other design elements or to any other locations and any and all subsequent users accept any and all liability resulting from any use or reuse of the data, opinions, and recommendations without the prior written consent of EMI.

EMI has no responsibility for construction means, methods, techniques, sequences, or procedures; for safety precautions or programs in connection with the construction; for the acts or omissions of the CONTRACTOR or any other person performing any of the construction; or for the failure of any worker to carry out the construction in accordance with the Final Construction Drawings and Specifications.

Services performed by EMI have been conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions. No other representation, expressed or implied, and no warranty or guarantee is included or intended.

8.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 2012, AASHTO LRFD Bridge Design Specification, 6th Edition, Washington, DC: AASHTO.

American Society for Testing and Materials (ASTM), 2015, Annual Book of Standards, Soil and Rock, Vol. 04.08.

California Department of Transportation (Caltrans), California Test Methods.

_______, 2015a, Standard Plans.

_______, 2015b, Standard Specifications.

_______, 2015c, Highway Design Manual.

_______, 2014a, California Amendments to AASHTO LRFD Bridge Design Specifications, Sixth Edition.

_______, 2014b, Memo to Designers 3-1.

_______, 2014c, Standard Drawing.

_______, 2013a, Memo To Designer 20-10.

_______, 2013b, Seismic Design Criteria, Version 1.7.

_______, 2012a, Caltrans ARS Online Version 2, Website http://dap3.dot.ca.gov/ARS_Online/index.php.

_______, 2012b, Caltrans Fault Database V2a for ARS Online.

_______, 2012c, Corrosion Guidelines, Version 2.0.

_______, 2012d, Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, Division of Engineering Services Geotechnical Services, November.

_______, 2009, Guidelines For Structure Foundation Reports, V. 2.0.

_______, 1992, Bridge Design Details, Section 6.

California Department of Water Resources (DWR), 2016, Water Data Library, Groundwater Data, Website http:www.water.ca.gov/waterdatalibrary.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps June 2003: California Geological Survey Web Page, http://www.consrv.ca.gov/cgs/rghm/psha/fault_parameters/pdf/2002_CA_Hazard_Maps.pdf

Dibblee, T.W., Jr., 2008, Geologic Map of the Hesperia 15 Minute Quadrangle, San Bernardino County, California: Dibblee Geology Center Map #DF-382.

Dokka, R.K., and Travis, C.J., 1990, Late Cenozoic strike-slip faulting in the Mojave Desert, California: Tectonics, v. 9, p. 311-340.

Ellsworth, W.L., 1990, Earthquake History, 1769-1989, in The San Andreas fault system, California: U.S. Geological Survey, Professional Paper 1515, p. 153-181.

Ensoft Inc., 2015, LPILE Plus Version 6.0, A program for analyzing Stress and Deformation of a Pile or Drilled Shaft under Lateral Loading, Austin, Texas.

Fuis, G.S., 1982, Crustal Structure of the Mojave Desert, California: Geological Society of America, Abstracts with Programs, v. 14, p. 164.

Greenbook, 2015, Standard Specifications for Public Works Construction.

Jennings, C.W. and Bryant, W.A., 2010, 2010 Fault activity map of California: California Geological Survey, California Geologic Data Map Series, Map No. 6.

Lam, I.P., and Martin, G.R., 1986, Seismic Design of Highway Bridge Foundations, FHWA Report Nos. FHWA A/RD-86/102.

Merriam, M., 2012, Caltrans Fault Database (V2b) for ARS Online, California Department of Transportation, Sacramento, CA.

Schell, B.A., 1994, Newly Discovered Faults Along the Northwest Extension of the Lockhart Fault Zone, Mojave Desert, Kern County, California, in Mojave Desert: South Coast Geologic Society, Annual Field Trip Guidebook # 22, p. 239-252.

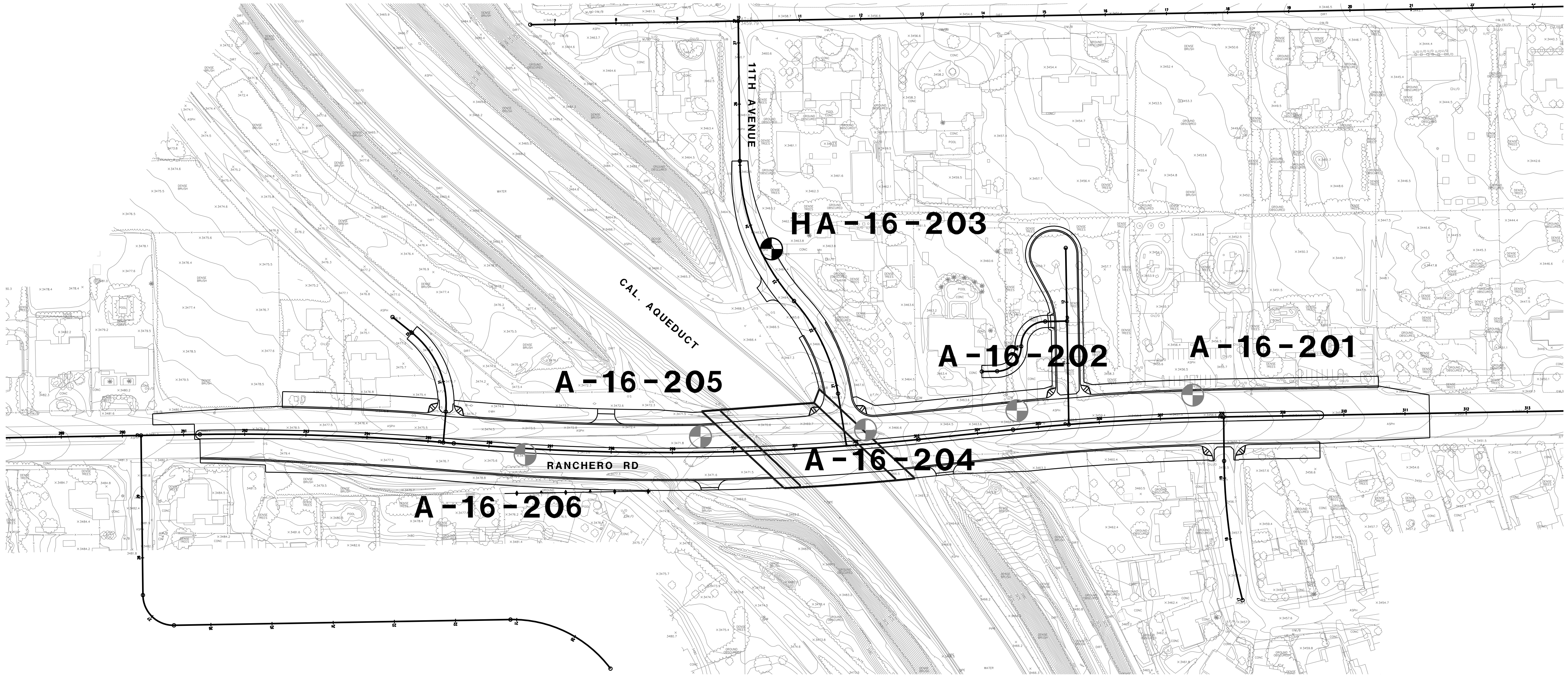
USGS, 2008, USGS 2008 Interactive Deaggregations (Beta), Website https://geohazards.usgs.gov/deaggint/2008

Winterkorn, H.F. and Fang, H.Y., 1975, Foundation Design Handbook, Van Nostrand Reinhold Company, New York, NY.

Ziony, J.I., and Yerkes, R.F., 1985, Evaluating earthquake and surface faulting potential, in Ziony, J.I., ed., Evaluating earthquake hazards in the Los Angeles region--An earth-science perspective: U.S. Geological Survey Professional Paper 1360, p. 43-91.

APPENDIX A

LOG OF TEST BORINGS SHEETS AND LETTER-SIZE BORING LOGS



LEGEND

Grab Sample

Auger Boring

0 100 200 ft

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

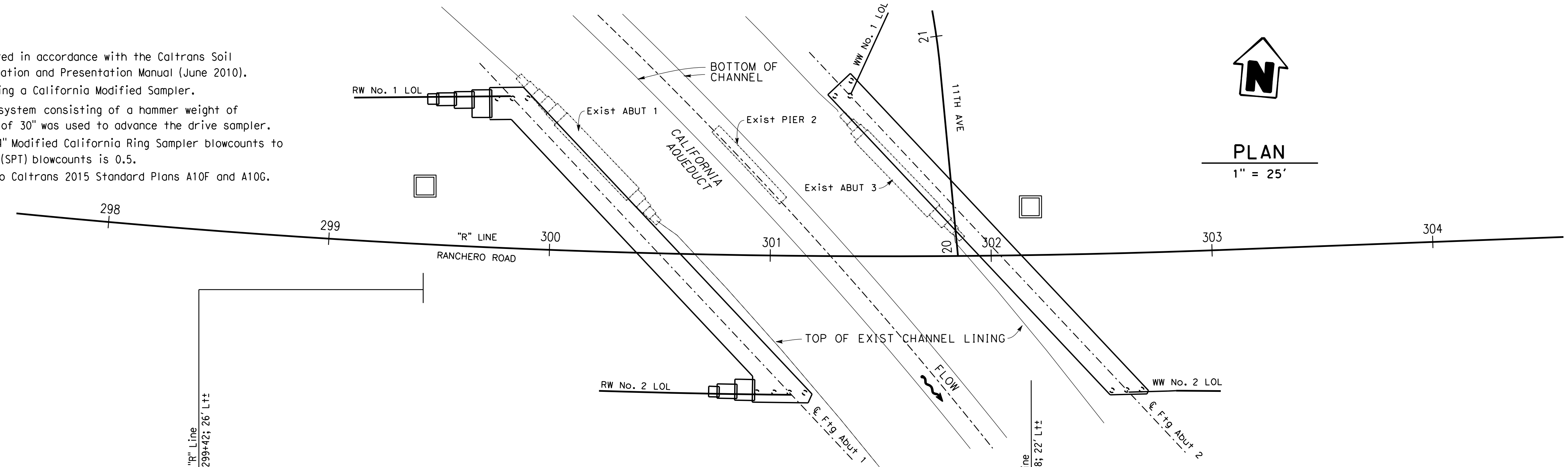
RANCHERO ROAD AQUEDUCT CROSSING

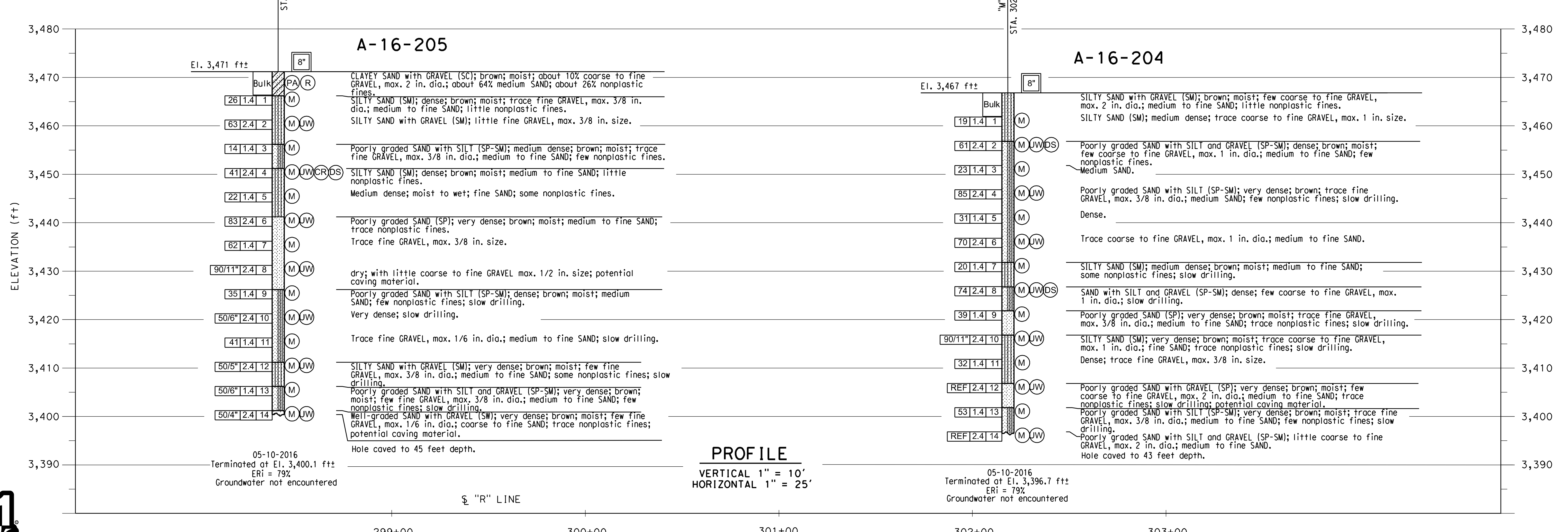
Project No.: 15-111 | Date: 01-29-2018

Bore Location Plan

NOTES

(1) This LOTB sheet was prepared in accordance with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2010).
(2) 2.4" samples were taken using a California Modified Sampler.
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance the drive sampler.
(4) Conversion factor from 2.4" Modified California Ring Sampler blowcounts to Standard Penetration Test (SPT) blowcounts is 0.5.
(5) For Legend Sheets, refer to Caltrans 2015 Standard Plans A10F and A10G.





Know what's **below.**
Call before you dig.

REV.	DESCRIPTION	DATE	BY

BENCHMARK: CITY OF HESPERIA H-21
LOCATION:
SW CORNER OF RANCHERO ROAD AND FUENTE AVENUE
DESCRIPTION:
TOP OF BRASS DISK STAMPED
ELEVATION = 3640.317 H-21

DESIGNED BY: M. Kapuskar
DRAWN BY: M. Kapuskar
CHECKED BY: L. Cheang
SUBMITTED BY:
L. Cheang GE 2345 01-29-18
NAME RCE No. DATE:

REGISTERED PROFESSIONAL ENGINEER
LINO CHEANG
No. GE 2345
Exp. 9-30-19
GEOTECHNICAL
STATE OF CALIFORNIA

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering
EARTH MECHANICS, INC.
17800 NEWHOPE STREET, SUITE B
FOUNTAIN VALLEY, CA 92708

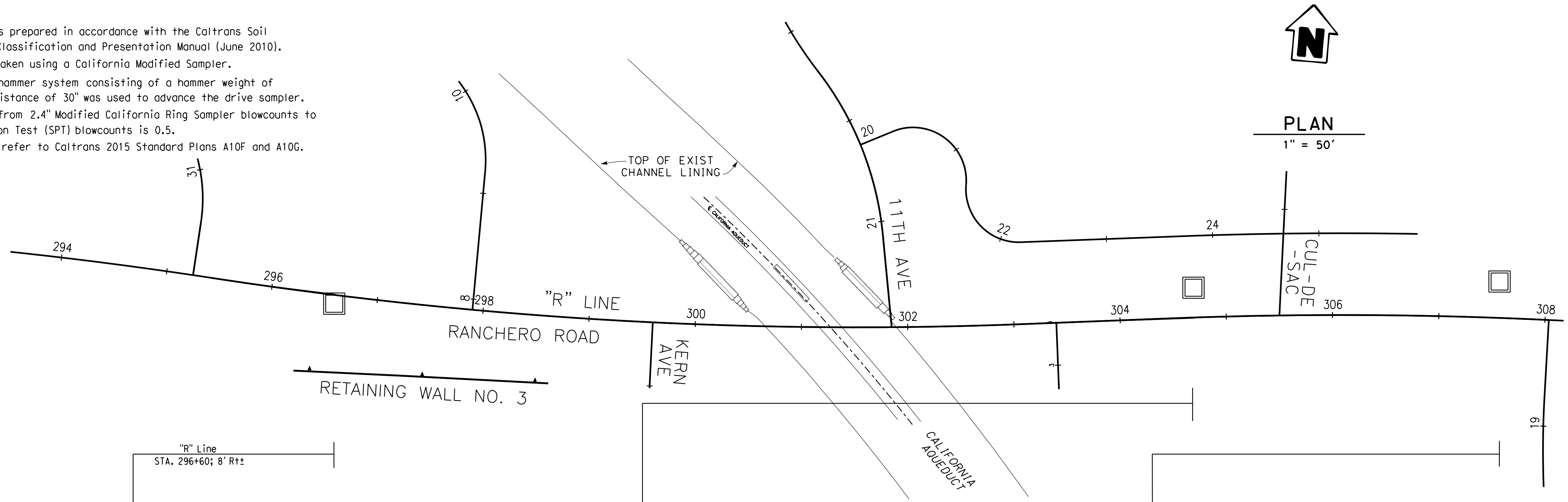
CITY OF HESPERIA
ENGINEERING DEPARTMENT
RECOMMENDED FOR APPROVAL BY:
DATE
AUTHORIZED SIGNATURE
APPROVED BY:
MICHAEL THORNTON
R.C.E. 44226 EXP. DATE 6/30/19
CITY ENGINEER

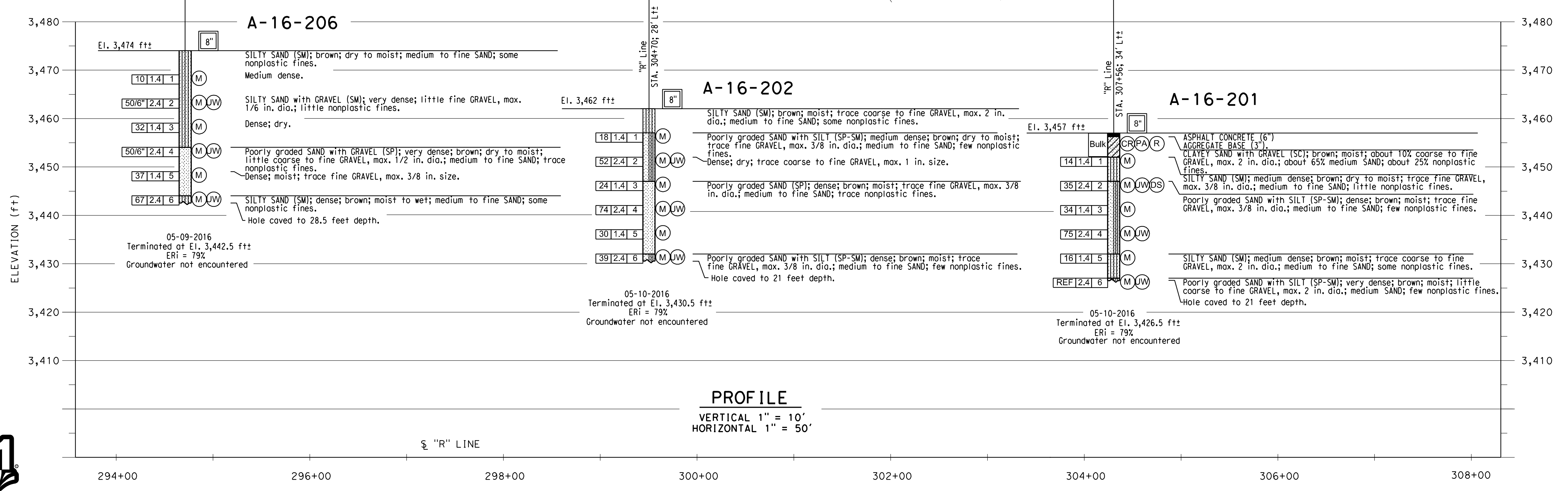
CITY OF HESPERIA
RANCHERO ROAD OVERCROSSING
LOG OF TEST BORINGS NO. 1

JOB No.
SHEET
OF

NOTES

(1) This LOTB sheet was prepared in accordance with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2010).
(2) 2.4" samples were taken using a California Modified Sampler.
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance the drive sampler.
(4) Conversion factor from 2.4" Modified California Ring Sampler blowcounts to Standard Penetration Test (SPT) blowcounts is 0.5.
(5) For Legend Sheets, refer to Caltrans 2015 Standard Plans A10F and A10G.





Know what's **below.**
Call before you dig.

REV.	DESCRIPTION	DATE	BY

BENCHMARK: CITY OF HESPERIA H-21
LOCATION:
SW CORNER OF RANCHERO ROAD AND FUENTE AVENUE
DESCRIPTION:
TOP OF BRASS DISK STAMPED
ELEVATION = 3640.317 H-21

DESIGNED BY: M. Kapuskar
DRAWN BY: M. Kapuskar
CHECKED BY: L. Cheang
SUBMITTED BY:
L. Cheang GE 2345 01-29-18
NAME RCE No. DATE:

REGISTERED PROFESSIONAL ENGINEER
LINO CHEANG
No. GE 2345
Exp. 9-30-19
GEOTECHNICAL
STATE OF CALIFORNIA

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering
EARTH MECHANICS, INC.
17800 NEWHOPE STREET, SUITE B
FOUNTAIN VALLEY, CA 92708

CITY OF HESPERIA
ENGINEERING DEPARTMENT
RECOMMENDED FOR APPROVAL BY:
DATE
AUTHORIZED SIGNATURE
APPROVED BY:
MICHAEL THORNTON
R.C.E. 44226 EXP. DATE 6/30/19
CITY ENGINEER

CITY OF HESPERIA
RANCHERO ROAD OVERCROSSING
LOG OF TEST BORINGS NO. 2

JOB No.
SHEET
OF

USER: $USER $TIME $REQUEST $DATE $PLTDRV

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names	Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])
WA	Wash Analysis (ASTM D 1140-97)

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

BORING RECORD LEGEND

Ranchero Road Aqueduct Crossing

Project Number: 15-111	Date: 5-10-16	SHEET 1 of 2

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE:
This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010 Edition), Section 2, for tables of additional soil description components and discussion of soil description and identification.

REF = Refusal; During drilling seating interval (first 6-inch interval) is not achieved.

LOG OF BORING NO. HA-16-203

Grade Elevation:	**3464± ft**				
Boring Depth:	**5.0 ft**	Driller:	**Hand auger**		SHEET **1 of 1**
Borehole Diameter:	**3"**	Type of Rig:	**Hand auger**	Comments:	
Date Drilled:	**5-10-16**	Hammer Data:	**Hand auger**		
Logged By:	**CP**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings and Cement	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Moisture (%)	Dry Density (pcf)	Test/ Results
0					Poorly graded SAND (SP); brown; dry; trace coarse to fine GRAVEL, max. 1 in. dia.; medium to fine SAND; trace nonplastic fines.			
		0			Grading to SILTY SAND (SM); brown; moist; trace fine GRAVEL, max. 3/8 in. dia.; medium to fine SAND; some nonplastic fines.			
5					Bottom of borehole at 5.0 ft bgs Groundwater not encountered			
10								
15								
20								
25								

EMI BORING LOG RANCHERO ROAD BORING LOGS.GPJ EMI CALTRANS 2013.GLB 6/24/16

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Ranchero Road Aqueduct Crossing

Project Number: 15-111 | Date: 6-24-16

APPENDIX B

LABORATORY TEST RESULTS

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

MOISTURE AND DENSITY TEST RESULTS

Client: Earth Mechanics, Inc.
Project Name: Ranchero Road & Main Street Widening
Project No.: 15-111

Laboratory No.: 16-0523
Date: 05/20/16

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
A-16-201	S-1	5	9.8	NA
A-16-201	D-2	10	7.4	115.4
A-16-201	S-3	15	4.8	NA
A-16-201	D-4	20	4.0	120.6
A-16-201	S-5	25	15.3	NA
A-16-201	D-6	30	4.0	117.2

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: Earth Mechanics, Inc.
AP No.: 16-0523

Project Name: Ranchero Road & Main Street Widening
Date: 05/20/16

Project Number: 15-111

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
A-16-201	B-0	0-5	25.2

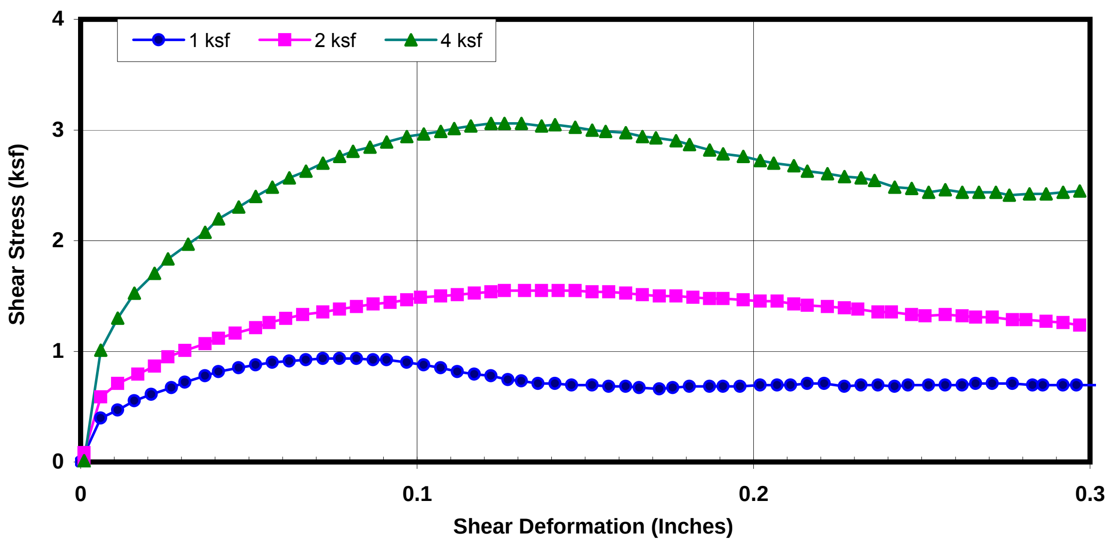
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

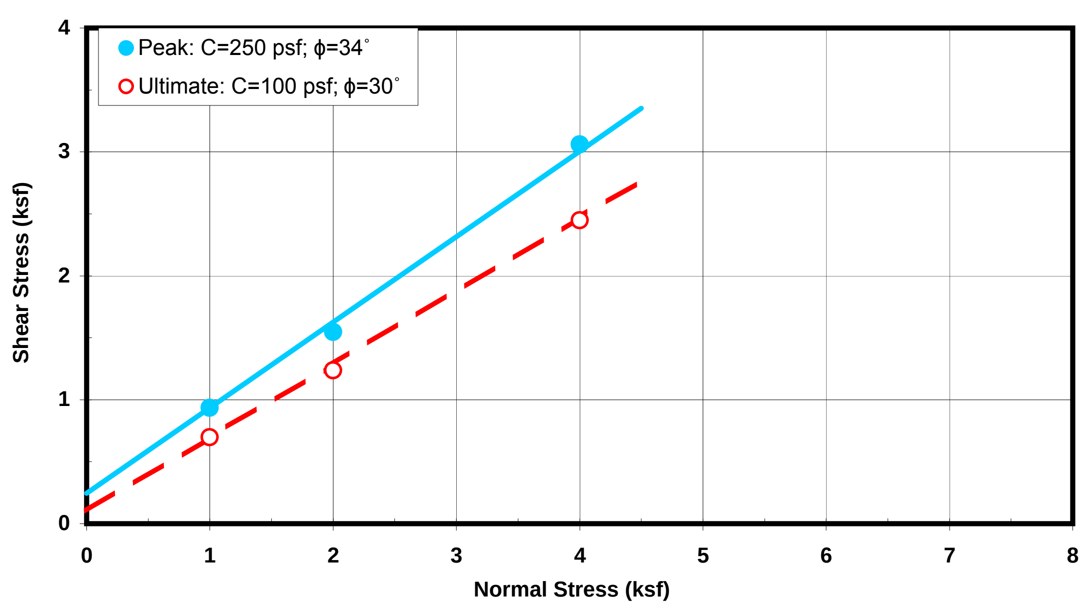
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Ranchero Road & Main Street Widening
Project No.: 15-111
Boring No.: A-16-201
Sample No.: D-2 **Depth (ft):** 10
Sample Type: Mod. Cal.
Soil Description: Sand w/silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: NG **Date:** 05/23/16
Computed By: NN **Date:** 05/26/16
Checked by: AP **Date:** 05/27/16

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
123.6	115.1	7.4	15.6	43	90	1	0.936	0.696
						2	1.548	1.236
						4	3.060	2.448





AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

Client Name: Earth Mechanics, Inc. AP Job No.: 16-0523
Project Name: Ranchero Road & Main Street Widening Date: 05/20/16
Project No.: 15-111

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
A-16-201	B-0	0-5	SC	9319	7.4	53	39

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

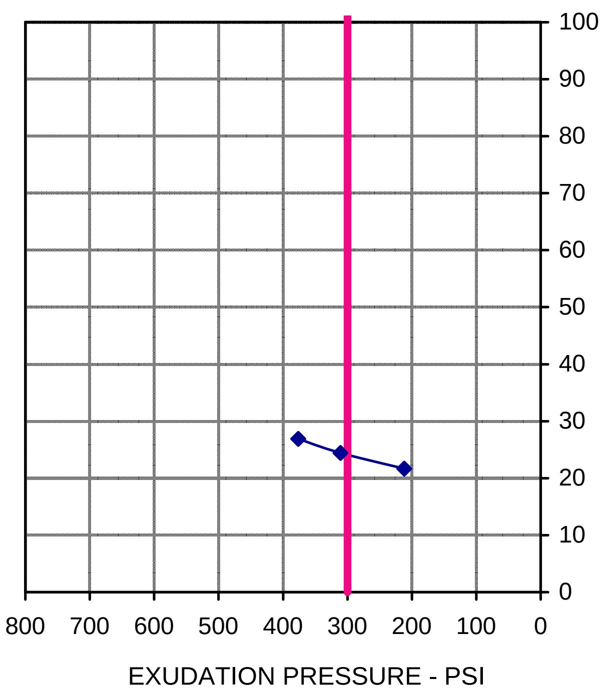
R-VALUE TEST DATA

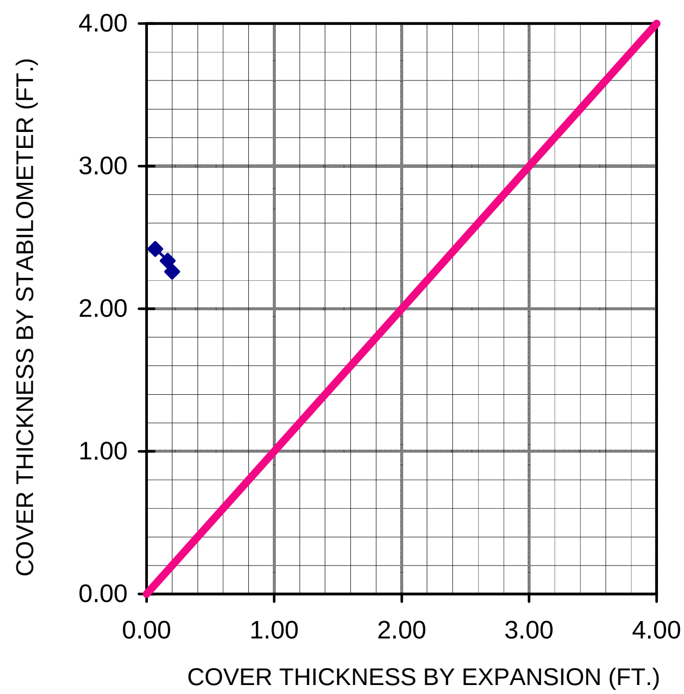
ASTM D2844

Project Name:	Ranchero Road & Main Street Widening	Tested By:	ST	Date:	05/20/16
Project Number:	15-111	Computed By:	KM	Date:	05/21/16
Boring No.:	A-16-201	Checked By:	AP	Date:	05/27/16
Sample Type:	B-0	Depth (ft.):	0-5		
Location:	N/A				
Soil Description:	Clayey Sand				

Mold Number	G	I	H	
Water Added, g	10	15	20	
Compact Moisture(%)	9.5	10.0	10.6	
Compaction Gage Pressure, psi	250	100	50	
Exudation Pressure, psi	376	311	212	
Sample Height, Inches	2.3	2.3	2.4	
Gross Weight Mold, g	2884	2902	2887	
Tare Weight Mold, g	1828	1837	1819	
Net Sample Weight, g	1056	1065	1067	
Expansion, inchesx10^{-4}	6	5	2	
Stability 2,000 (160 psi)	36/90	38/95	39/104	
Turns Displacement	4.28	4.37	4.48	
R-Value Uncorrected	31	28	23	
R-Value Corrected	27	24	22	
Dry Density, pcf	127.0	127.5	121.9	
Traffic Index	12.0	12.0	12.0	
G.E. by Stability	2.26	2.34	2.42	
G.E. by Expansion	0.20	0.17	0.07	

R-VALUE	By Exudation:	24
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	24
Remarks	Gf = 1.24, and 0.6 % Retained on the ¾" *Not Applicable	





AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

MOISTURE AND DENSITY TEST RESULTS

Client: Earth Mechanics, Inc.
Project Name: Ranchero Road & Main Street Widening
Project No.: 15-111

Laboratory No.: 16-0523
Date: 05/20/16

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
A-16-202	S-1	5	5.5	NA
A-16-202	D-2	10	2.3	116.0
A-16-202	S-3	15	3.3	NA
A-16-202	D-4	20	3.7	113.8
A-16-202	S-5	25	4.4	NA
A-16-202	D-6	30	7.9	127.0

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

MOISTURE AND DENSITY TEST RESULTS

Client: Earth Mechanics, Inc.
Laboratory No.: 16-0523
Project Name: Ranchero Road & Main Street Widening
Date: 05/20/16
Project No.: 15-111

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
A-16-204	S-1	5	9.8	NA
A-16-204	D-2	10	6.1	122.8
A-16-204	S-3	15	5.7	NA
A-16-204	D-4	20	7.1	111.7
A-16-204	S-5	25	8.8	NA
A-16-204	D-6	30	7.2	111.3
A-16-204	S-7	35	10.8	NA
A-16-204	D-8	40	5.7	120.9
A-16-204	S-9	45	5.2	NA
A-16-204	D-10	50	9.3	126.5
A-16-204	S-11	55	6.8	NA
A-16-204	D-12	60	4.6	114.8
A-16-204	S-13	65	6.1	NA
A-16-204	D-14	70	2.9	110.6

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: Earth Mechanics, Inc. AP No.: 16-0523

Project Name: Ranchero Road & Main Street Widening Date: 05/20/16

Project Number: 15-111

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
A-16-205	B-0	0-5	25.5

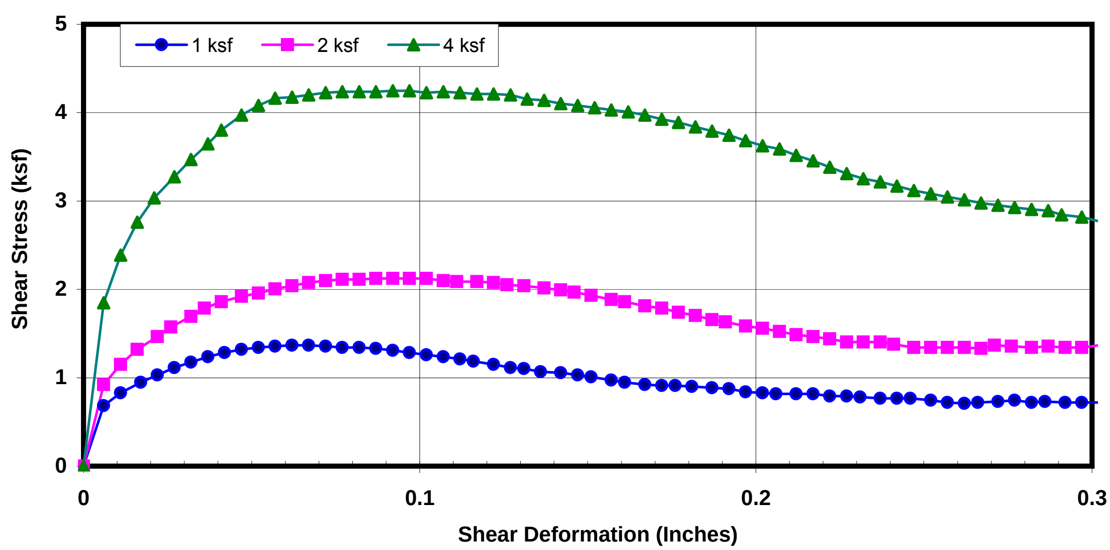
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

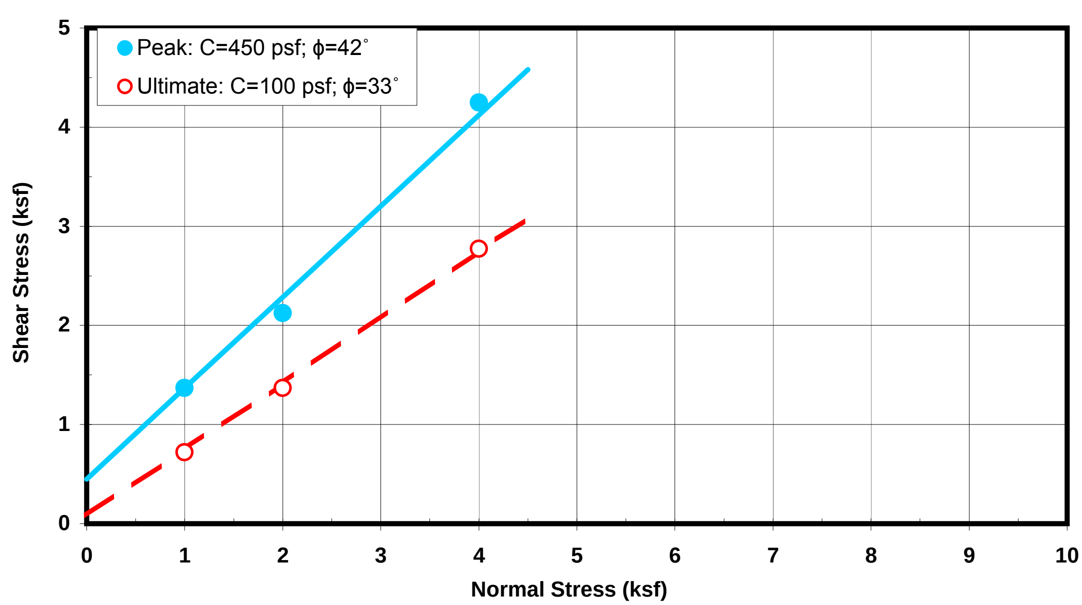
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Ranchero Road & Main Street Widening
Project No.: 15-111
Boring No.: A-16-204
Sample No.: D-2 **Depth (ft):** 10
Sample Type: Mod. Cal.
Soil Description: Clayey Sand w/gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: NG **Date:** 05/23/16
Computed By: NN **Date:** 05/26/16
Checked by: AP **Date:** 05/27/16

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.5	122.1	6.1	13.9	43	99	1	1.368	0.720
						2	2.124	1.368
						4	4.248	2.772



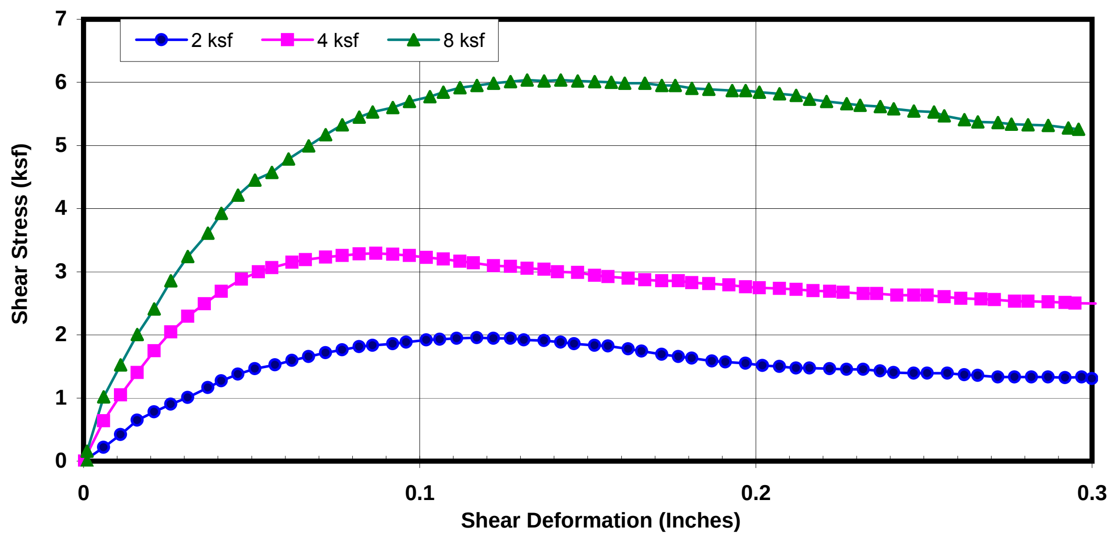


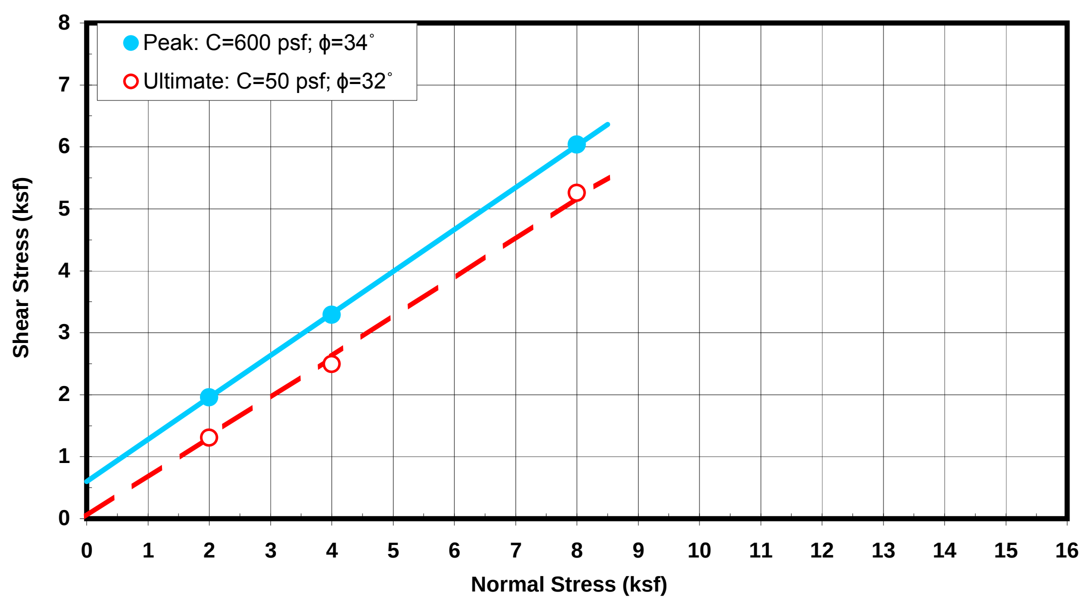
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Ranchero Road & Main Street Widening	**Tested By:** NG	**Date:** 05/24/16
Project No.:	15-111	**Computed By:** NN	**Date:** 05/26/16
Boring No.:	A-16-204	**Checked by:** AP	**Date:** 05/27/16
Sample No.:	D-8 **Depth (ft):** 40		
Sample Type:	Mod. Cal.		
Soil Description:	Sand w/silt & gravel		
Test Condition:	Inundated **Shear Type:** Regular		

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
127.1	120.2	5.7	14.2	38	95	2	1.956	1.308
						4	3.291	2.496
						8	6.036	5.256





MOISTURE AND DENSITY TEST RESULTS

Client: Earth Mechanics, Inc.
Laboratory No.: 16-0523
Project Name: Ranchero Road & Main Street Widening
Date: 05/20/16
Project No.: 15-111

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
A-16-205	S-1	5	5.7	NA
A-16-205	D-2	10	7.3	127.2
A-16-205	S-3	15	7.0	NA
A-16-205	D-4	20	5.5	124.4
A-16-205	S-5	25	9.6	NA
A-16-205	D-6	30	5.1	110.3
A-16-205	S-7	35	3.3	NA
A-16-205	D-8	40	3.0	116.3
A-16-205	S-9	45	6.3	NA
A-16-205	D-10	50	4.9	108.2
A-16-205	S-11	55	6.2	NA
A-16-205	D-12	60	6.6	126.3
A-16-205	S-13	65	5.2	NA
A-16-205	D-14	70	3.6	112.9

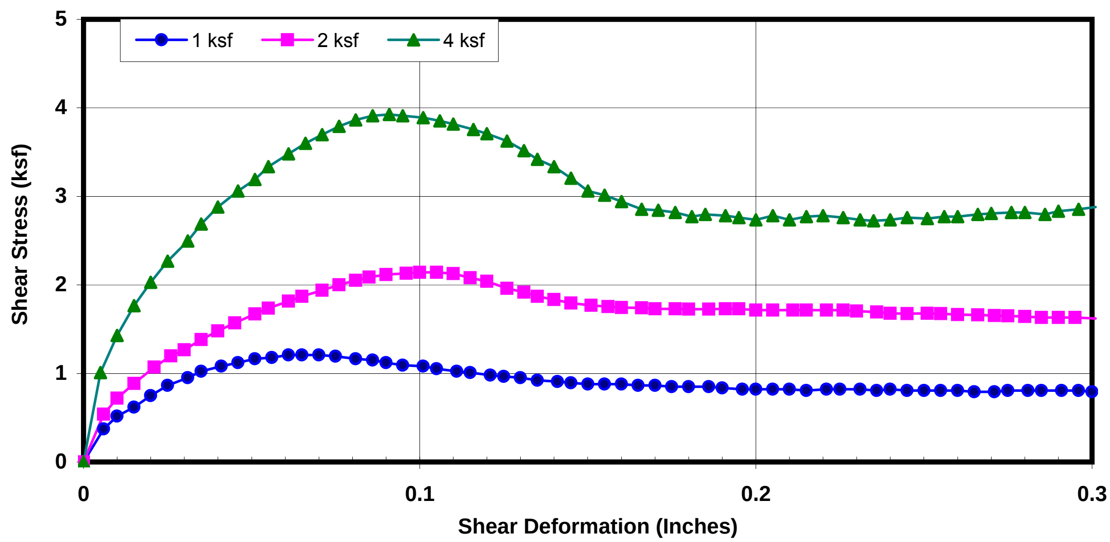
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

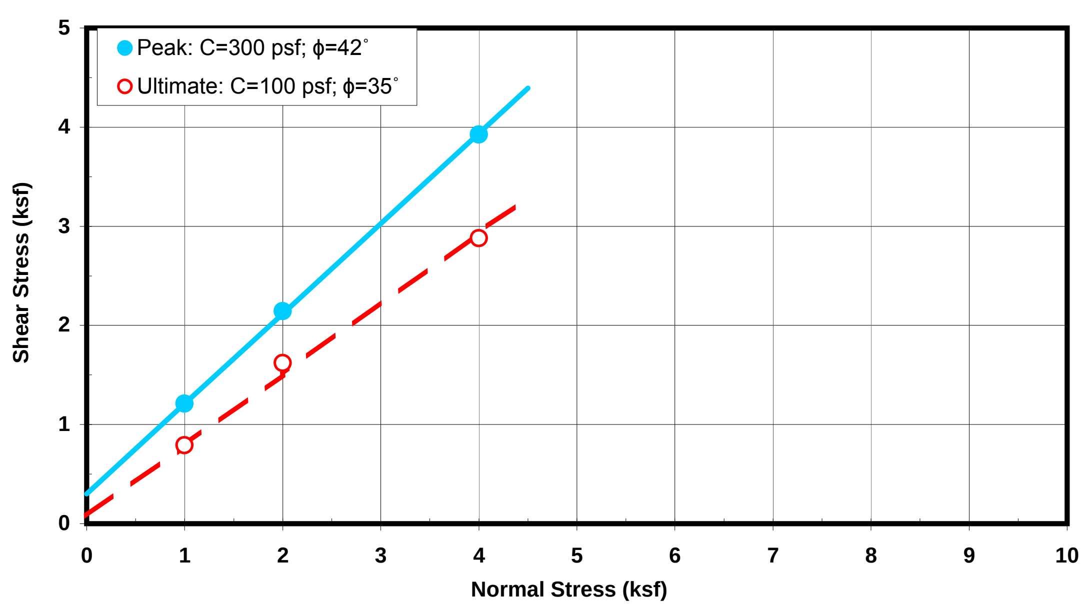
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Ranchero Road & Main Street Widening
Project No.: 15-111
Boring No.: A-16-205
Sample No.: D-4 **Depth (ft):** 20
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: NG **Date:** 05/23/16
Computed By: NN **Date:** 05/26/16
Checked by: AP **Date:** 05/27/16

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
130.3	123.4	5.5	15.7	41	116	1	1.210	0.792
						2	2.142	1.620
						4	3.924	2.880





AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

Client Name: Earth Mechanics, Inc.
Project Name: Ranchero Road & Main Street Widening
Project No.: 15-111

AP Job No.: 16-0523
Date: 05/20/16

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
A-16-205	D-4	20	SM	12225	8.2	55	40

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

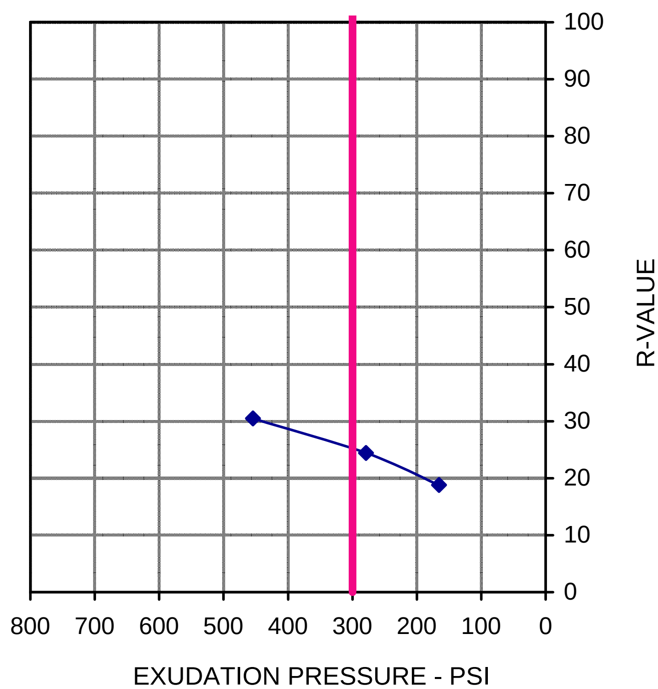
R-VALUE TEST DATA

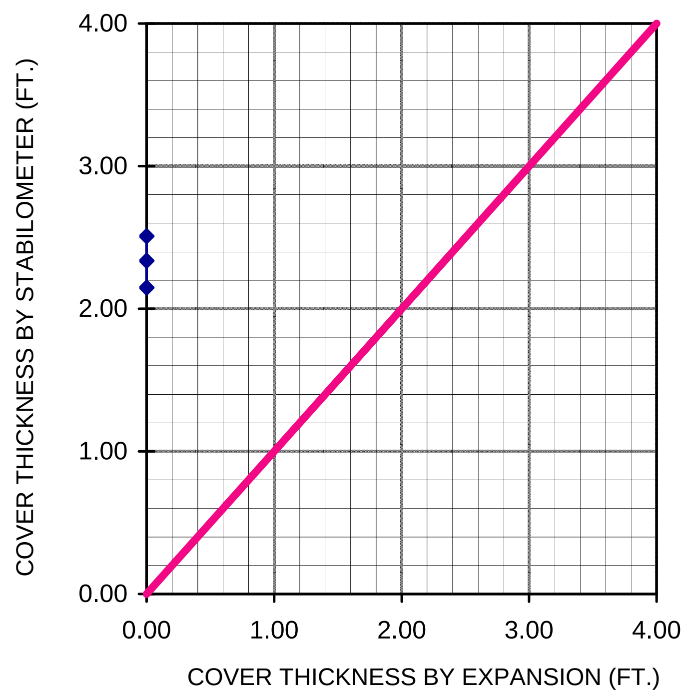
ASTM D2844

Project Name:	Ranchero Road & Main Street Widening	Tested By:	ST	Date:	05/20/16
Project Number:	15-111	Computed By:	KM	Date:	05/21/16
Boring No.:	A-16-205	Checked By:	AP	Date:	05/27/16
Sample Type:	B-0	Depth (ft.):	0-5		
Location:	N/A				
Soil Description:	Clayey Sand				

Mold Number	A	C	B	
Water Added, g	22	26	32	
Compact Moisture(%)	9.9	10.4	11.0	
Compaction Gage Pressure, psi	250	100	60	
Exudation Pressure, psi	454	279	166	
Sample Height, Inches	2.3	2.3	2.3	
Gross Weight Mold, g	3029	3034	3037	
Tare Weight Mold, g	1968	1965	1967	
Net Sample Weight, g	1062	1069	1070	
Expansion, inchesx10^{-4}	0	0	0	
Stability 2,000 (160 psi)	38/86	40/96	44/108	
Turns Displacement	3.97	4.38	4.62	
R-Value Uncorrected	35	28	21	
R-Value Corrected	31	24	19	
Dry Density, pcf	127.2	127.5	127.0	
Traffic Index	12.0	12.0	12.0	
G.E. by Stability	2.15	2.34	2.51	
G.E. by Expansion	0.00	0.00	0.00	

R-VALUE	By Exudation:	25
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	25
Remarks	Gf = 1.24, and 2.2 % Retained on the ¾" *Not Applicable	





AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

MOISTURE AND DENSITY TEST RESULTS

Client: Earth Mechanics, Inc.
Laboratory No.: 16-0523
Project Name: Ranchero Road & Main Street Widening
Date: 05/20/16
Project No.: 15-111

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
A-16-206	S-1	5	2.9	NA
A-16-206	D-2	10	6.3	126.5
A-16-206	S-3	15	4.8	NA
A-16-206	D-4	20	4.4	110.0
A-16-206	S-5	25	7.0	NA
A-16-206	D-6	30	14.9	113.4