January 25, 2024

EMI Project No. 22-161

Mark Thomas & Company, Inc.
2121 Alton Parkway, Suite 210
Irvine, CA 92606

Attention: Mr. Pat Somerville, PE

Subject: **Foundation Report**
Cannon Street Widening Improvement Project
Orange, California

Dear Mr. Somerville:

Attached is our Foundation Report for the proposed Cannon Street Widening Improvement Project in the City of Orange, California. This report presents the findings, conclusions and recommendations for the design and construction of the bridge foundations and roadway improvements.

Please submit this report to City of Orange and any other participating agencies for their review. Responses to their review comments, as well as your comments, will be incorporated into a revised report.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions, please do not hesitate to contact us.

Sincerely,

EARTH MECHANICS, INC.

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FOUNDATION REPORT
CANNON STREET WIDENING IMPROVEMENT PROJECT
ORANGE, CALIFORNIA

Prepared for:

Mark Thomas & Company, Inc.
2121 Alton Parkway, Suite 210
Irvine, CA 92606

Prepared by:

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EMI Project No. 22-161

January 25, 2024

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

1.0 SCOPE OF WORK

This Foundation Report presents the findings and conclusions of a geotechnical study conducted by Earth Mechanics, Inc. (EMI) for the Cannon Street Widening Improvement Project in the City of Orange. The purpose of the geotechnical study was to obtain information on subsurface soils and conditions, and develop design and construction recommendations to assist Mark Thomas & Company, Inc. (MTC) in preparing the project Plans, Specifications, and Estimates (PS&E) for the project.

The geotechnical services provided for this project included the following tasks:

- Collection and review of existing geologic and groundwater information;
- Geotechnical field exploration including drilling and logging of exploratory borings;
- Laboratory testing of selected subsurface soil samples;
- Engineering analysis to develop design and construction recommendations for bridge foundations and pavement structural sections; and
- Preparation of this report presenting our findings, conclusions, and recommendations.

2.0 PROJECT DESCRIPTION

The City of Orange proposes to widen Cannon Street between Santiago Canyon Road and Serrano Avenue. The proposed project will construct a new pedestrian bridge over Santiago Creek as well as the roadway north of the bridge until just south of Serrano Avenue. The project location is shown in Figure 1.

Cannon Street Pedestrian Bridge: The proposed single-span bridge will be approximately 200 feet long and 14.3 feet wide. This prefabricated steel truss bridge will be supported on seat-type abutments.

Earth Mechanics, Inc.
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Cannon Street Widening Improvement Project

Project No. 22-161 | Date: January 2024

SITE LOCATION MAP

Figure 1

3.0 GEOTECHNICAL INVESTIGATION

In August 2023, EMI conducted a site-specific geotechnical field investigation consisting of four shallow hand auger borings and two rotary-wash borings. Out of the six borings, two (R-23-002 and R-23-003) were drilled to collect subsurface information for the proposed bridge widening and others (HA-23-001 and HA-23-004 through HA-23-006) were drilled to collect subsurface information for the proposed roadway widening. The approximate locations of these borings are shown in Figure 2. Soil exploration information is summarized in Table 1 and the Log-Of-Test-Borings (LOTB) sheet and boring logs are included in Appendix A.

Table 1. Site-Specific Soil Exploration Information

Boring No.	Boring Type	Approx. Northing	Approx. Easting	Approx. Station	Station Line	Approx. Offset (feet)	Approx. Ground Surface El. (feet)	Approx. Bottom of Hole El. (feet)	Approx. Groundwater El. During Drilling (feet)
HA-23-001	HA	2243316.0	6092328.0	67+63	"C"	31.3 Rt.	+372.3	+367.3	NE
R-23-002	RW	2243458.6	6092224.9	169+03	"C1"	24.3 Lt.	+371.3	+300.8	+346.3
R-23-003	RW	2243667.1	6092236.3	171+12	"C1"	15.2 Lt.	+376.6	+305.6	+360.6
HA-23-004	HA	2244103.0	6092273.0	75+51	"C"	41.0 Lt.	+410.7	+405.7	NE
HA-23-005	HA	2244736.0	6092100.0	82+10	"C"	45.9 Lt.	+433.0	+428.0	NE
HA-23-006	HA	2245191.0	6092086.0	86+69	"C"	44.0 Lt.	+450.2	+445.2	NE

Notes:
(1) Ground Surface Elevations were estimated from topographic civil plans.
(2) HA = Hand Auger, R = Rotary-Wash.

The rotary-wash borings were drilled using a truck-mounted drill rigs equipped with 5-inch diameter drill rods. Subsurface soil conditions were logged and samples of soils were collected for laboratory testing. Smaller disturbed and relatively undisturbed soil samples were collected from soil borings generally at 5-foot intervals using the Standard Penetration Test (SPT) sampler and the Modified California Drive (MCD) sampler, respectively. The SPT sampler is unlined and has an inside diameter of 1.4 inches and the MCD sampler is lined with a series of 1-inch tall brass rings with an inside diameter of 2.4 inches.

Blowcounts from the SPT and MCD samplers were recorded during the exploration. The samplers were driven using a 140-lb hammer falling 30 inches down a total depth of 18 inches or until refusal, whichever occurs first. The drill rigs were equipped with auto-trip safety hammer with rated efficiencies of 74% (hammer efficiency provided by the drilling contractor). The blowcounts for the last 12 inches or less of penetration were recorded and are shown in the LOTB sheet included in Appendix A.

HA-23-001
Depth (ft)

Proposed Location of Geotechnical Boring
"HA" - Hand Auger Boring
"R" - Rotary Wash

0 80 160 FEET
SCALE

N

Sheet 1 of 3

Earth Mechanics, Inc.
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Cannon Street Widening Improvement Project
Boring Location Plan

HA-23-001 Depth (ft) Proposed Location of Geotechnical Boring
"HA" - Hand Auger Boring
"R" - Rotary Wash

0 80 160 FEET
SCALE

N

Sheet 2 of 3

Earth Mechanics, Inc.
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Cannon Street Widening Improvement Project
Boring Location Plan

HA-23-001
Depth (ft)

Proposed Location of Geotechnical Boring
"HA" - Hand Auger Boring
"R" - Rotary Wash

0 80 160 FEET
SCALE

N

Sheet 3 of 3

Earth Mechanics, Inc.
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Cannon Street Widening Improvement Project
Boring Location Plan

4.0 LABORATORY TESTING PROGRAM

Selected soil samples were tested to determine soil classification and physical and engineering properties. A list of soil tests performed, the corresponding test methods, and purpose of testing is presented in Table 2.

The laboratory soil tests were conducted in general accordance with California Test (CT) methods or American Society for Testing and Materials (ASTM) standards. The test results are presented in Appendix B. The locations where tests were performed are shown on the boring logs and LOTB sheet included in Appendix A.

Table 2. Explanation of Laboratory Tests Performed

Type of Test	Applicable Test Method	Purpose
Dry Density	ASTM D 2937	Estimate in-situ soil density
Moisture Content	ASTM D 2216	Estimate in-situ soil moisture content
No. 200 Wash	ASTM D 1140	Estimate percentage of gravel, sand, and fines content
Atterberg Limits	ASTM D 4318	Evaluate plasticity of fine-grained particles
Direct Shear	ASTM D 3080	Estimate strength parameters
R-Value	ASTM D 2844	Measure strength of subgrade for use in pavements
Soil pH	CT 643	Determine corrosion potential
Minimum Resistivity	CT 643	
Sulfate Content	CT 417	
Chloride Content	CT 422	

5.0 SITE GEOLOGY

5.1 Physiography and Topography

The project area is located within the northwestern part of the Peninsular Ranges geomorphic province in the central block of the Los Angeles Basin. The Los Angeles Basin is a large, relatively flat, alluvium filled, low-lying coastal plain surrounded by mountains on the north, east, and southeast, and the Pacific Ocean and Palos Verdes Hills along the western margin of the basin. The basin floor gradually slopes southwesterly along the margins of the surrounding hills to sea level along the coastline. The basin floor is disrupted by an alignment of northwest-southeast trending, low-elevation hills along the Newport-Inglewood Structural Zone (NISZ). The areas on either side of the NISZ are essentially flat and comprise the Downey-Tustin plain on the northeast and the Torrance Plain on the southwest. The project site is located within the Downey plain. Major rivers within the basin are the Los Angeles, San Gabriel, and Santa Ana Rivers which enter the basin through gaps in the surrounding mountains, which drain southerly across the basin floor to the Pacific Ocean in the west. The Santa Ana River extends just north of the northern limit of the project and the Santiago Creek channel crosses the project corridor just north of the SR-22 freeway.

The Rancho Cucamonga creek passes directly through the site and the closest major landmark is the Chino Airport which is one mile to the west. The onsite ground elevation is relatively flat except where it dips down in the Creek. The ground elevation varies between +608 (creek bottom near pedestrian and bicycle bridge) and +635 feet.

5.2 Stratigraphy

Onsite materials consist of young alluvial soils underlain by bedrock of the middle Miocene-age El Modeno Volcanics. The El Modeno Volcanics at the bridge site consists of bedded palagonite tuff (Schoellhamer and others, 1981).

The alluvial deposits generally consist of moist, brown, fine- to coarse-grained clayey to silty sand. The bedrock materials generally consist of highly weathered, olive brown to yellowish brown, friable, poorly indurated palagonite tuff volcanics. The tuff is mapped to be bedded according to regional maps (Schoellhamer and others, 1981) with bedding generally dipping between 20-30 degrees to the east.

5.3 Geologic Structure

The Los Angeles Basin is a deep structural basin comprising two major downward folds (synclines) separated by the NISZ uplift; the Paramount syncline is east of the NISZ under the Downey-Tustin Plain, and the Gardena syncline is west of the NISZ under the Torrance Plain. The Paramount syncline is the larger and deeper syncline with basement rocks as deep as about 30,000 feet (Yerkes et al, 1965). As described in the physiography section, the basin is rimmed by marginal elevated plains that rise about a hundred feet to as much as a couple hundred feet above the general level of the basin floor. These elevated plains and the Coyote Hills are underlain by fault-bounded, upward folds (anticlines) that are prolific oil fields.

The geologic structure at the site is quite simple and is characterized by relatively flat-lying Quaternary sediments overlying gently dipping Tertiary sedimentary rocks. The site is over the northeast limb of Anaheim nose, a faulted, west plunging anticlinal structure (upwarp). Pliocene-age and older Tertiary-age strata on the limb of Anaheim nose dip gently northwesterly.

5.4 Faulting

The nearest major active or potentially active surface faults within the project vicinity are the Puente Hills blind thrust fault, the Whittier-Elsinore Fault, the Newport Inglewood Structural Zone, and El Modeno-Peralta Hills Faults.

The Newport-Inglewood Structural Zone. The NISZ comprises a northwest-southeast trending series of faults and folds in the western Los Angeles Basin. The zone lies along the coast and in the offshore area south-southwest of the project site (Figure 4). The nearest mapped trace along the NISZ is located approximately 12.4 miles southwest of the proposed bridge structure (USGS, 2023). The NISZ consists of several faults and folds over an area more than three miles wide. The structural zone extends southeasterly from the Santa Monica Mountains on the north to the San Joaquin Hills to the Newport Beach area on the south where it extends into the offshore area. In the offshore area, the fault zone is believed to continue to at least the Dana Point area. In the Newport Beach-San Joaquin Hills area, the structural zone widens to include faults such as the Bolsa, Fairview, and Pelican Hill faults. Offshore, in the San Onofre region, the fault is believed to connect with a similar zone of folding and faulting called the Offshore Zone of Deformation and together, the fault system may extend southerly to the Rose Canyon fault zone in the San Diego region. This larger trend of faults and folds is commonly referred to as the Santa Monica-Baja Zone of Deformation.

Whittier-Elsinore Fault Zone. The Elsinore fault extends northwesterly along the eastern flank of the Santa Ana Mountains and is located northeast of the project area. The Elsinore fault system extends from the Los Angeles basin area to Mexico, a distance of more than 160 miles. The fault zone comprises several interconnected fault segments. The northwest end of the zone is the Whittier fault which is along the southwest side of the Puente Hills. The Whittier fault connects to the Elsinore fault along the eastern side of the Santa Ana Mountains and may continue into Mexico connecting to faults such as the Laguna Salada fault. The nearest mapped trace along the Elsinore Fault (Whittier Section) is located approximately 6 miles northeast of the proposed bridge structure (USGS, 2023).

The Puente Hills Blind Thrust Fault. The Puente Hills blind thrust fault (Coyote Hills and Richfield segments) dips northerly under the San Gabriel Valley (Shaw et al., 2002). The fault extends for more than 25 miles along strike in the northern Los Angeles basin from downtown Los Angeles east to Brea in northern Orange County (Shaw et al., 2002). The blind thrust system consists of three segments, the Santa Fe Springs segment stepped to the right from the Los Angeles segment farther west and the Coyote Hills segment southeast of the Santa Fe Springs segment. The study area overlies the Santa Fe Springs segment of the Puente Hills thrust fault system, with the LA segment extending west of the study area and the Coyote Hills segment southeast of the study area. Based on projections from available published and unpublished oil field data, the fault is probably about 8 to 10 miles below the site. The projected trace of the

Coyote Hills segment of the Puente Hills blind thrust fault is located approximately 7 miles northwest of the proposed bridge structure.

El Modeno-Peralta Hills Faults. The El Modeno fault and Peralta Hills fault are two potentially active faults located along the southern margin of the Peralta Hills. The El Modeno fault is a southwest-dipping, north-south trending normal fault that extends from the Peralta Hills area south to the vicinity of Peters Canyon Wash. A portion of the fault is also mapped as an east-west westerly trace that extends west beyond SR-55 into the central lowland. The Peralta Hills fault is an east-west trending, north-dipping thrust fault that has a known sinuous trace extending around 6.2 miles. The fault is considered capable of a 6 to 7 magnitude earthquake based on the estimated fault length. El Modeno fault is located approximately 1200 feet northeast of the proposed bridge structure. The Peralta Hills fault crosses the project corridor approximately 1 mile north of the proposed bridge structure.

Yorba Linda Trend (Seismicity Zone). The Yorba Linda seismicity trend is northeast/southwest trending 5- to 10-mile long zone between latitude 33° 45' N and 33° 55' N. The seismicity zone is believed to be the source of the 2008 Chino Hills earthquake (M_W=5.4). The seismicity zone is located approximately 4.3 miles northwest of the proposed bridge structure.

5.5 Seismicity

The project site is in seismically active southern California. The present-day seismotectonic stress field in the Los Angeles region is one of north-northeasterly compression. This is indicated by the geologic structures, by earthquake focal-mechanism solutions, and by geodetic measurements. These data suggest compression rates of between 0.2 and 0.4 inch/year (5 and 9 mm/year) across the greater Los Angeles area.

Historical earthquake epicenter maps show widespread seismicity throughout the basin. Earthquakes in the region occur primarily as loose clusters along the NISZ, the southern margin of the Santa Monica Mountains, the southern margin of the Santa Susana and the San Gabriel Mountains, and in the Coyote Hills-Puente Hills area. Although historical earthquakes occur in proximity to known faults, they are difficult to directly associate with mapped faults. Part of this difficulty is due to the fact that the basin is underlain by several subsurface thrust faults (blind faults).

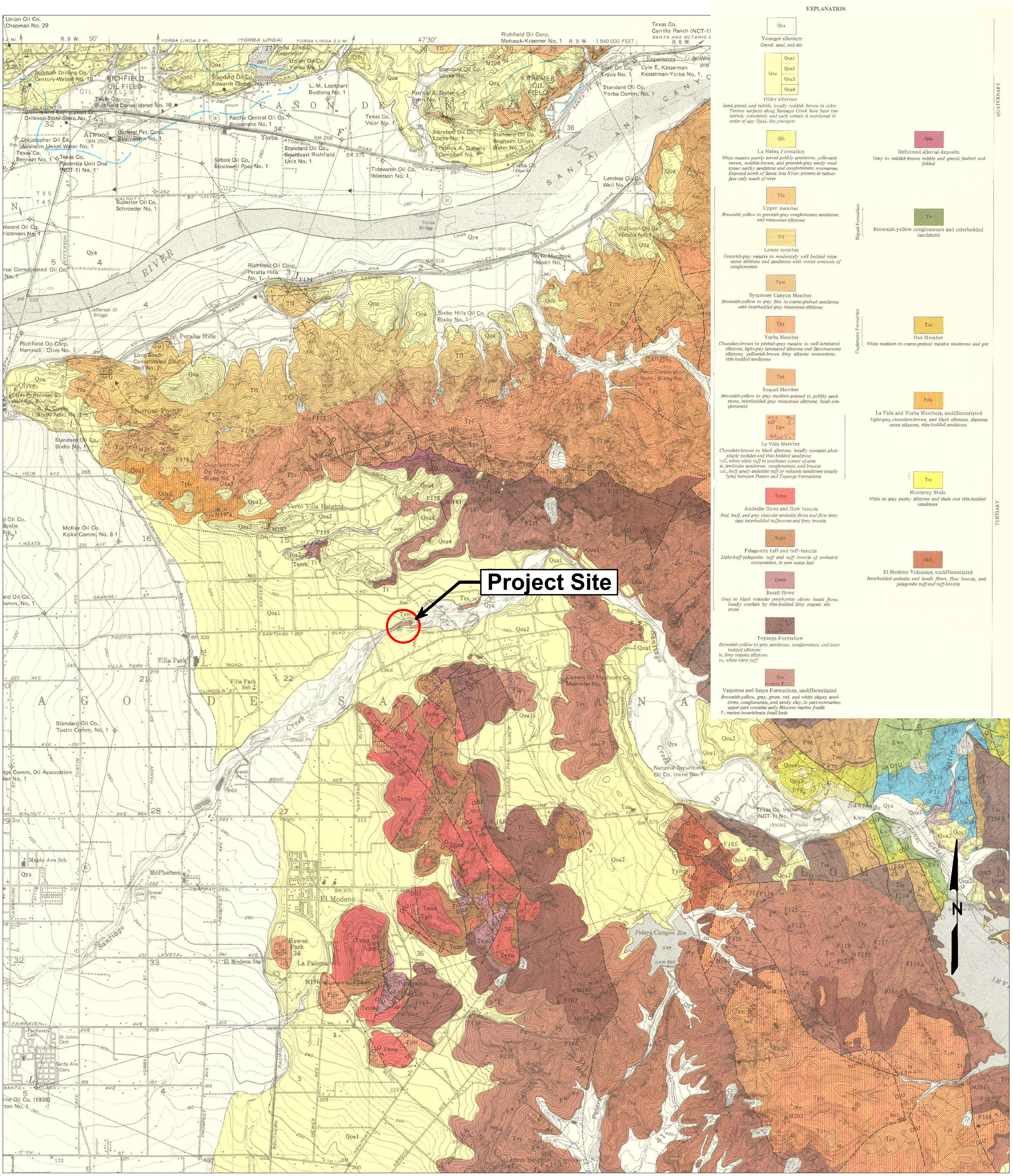
There is no clustering or alignment of earthquakes in proximity to the site. There are fewer earthquakes in the Tustin Plain-western Santa Ana Mountains region (i.e. the site area) than anywhere else in the Los Angeles Basin area. This apparent lack of earthquake activity suggests that the site area is relatively tectonically stable and suggests that there are no unrecognized active faults at the site.

The largest historical earthquake within the Los Angeles Basin was the 1933 Long Beach earthquake of $M_W = 6.4$ ($M_L = 6.3$) which is generally believed (e.g. Benioff, 1938) to have been associated with the Newport-Inglewood Structural Zone (NISZ). The association was based on abundant ground failures along the NISZ trend but no unequivocal surface rupture was identified. Hauksson and Gross (1991) reevaluated the seismicity data and relocated the 1933 earthquake hypocenter to a depth of about 6 miles below the Huntington Beach-Newport Beach city boundary.

A significant historical earthquake in the Orange County region was the 1812 earthquake which caused damage at the San Juan Capistrano Mission. The location and magnitude of the 1812 earthquake are unknown but geological studies (Jacoby et al, 1988; Fumal et al, 1993; Weldon et al., 2004) postulated that it did not occur in the Capistrano area but, rather, was a large (M> 7.0) distant event on the San Andreas Fault in the Wrightwood area of the San Gabriel Mountains.

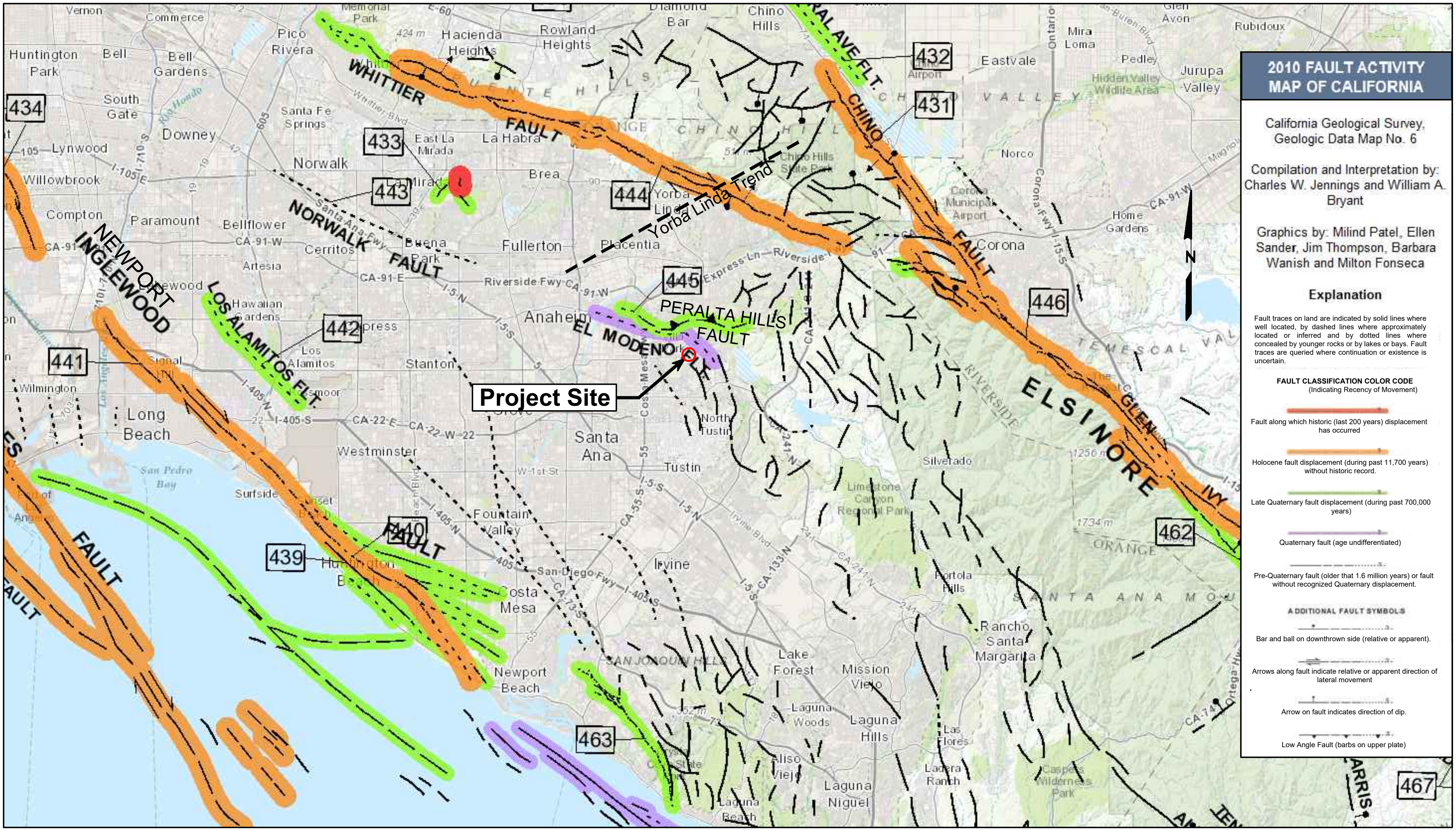
The earliest documented earthquake in the region was reported by the Portola expedition as they camped near the Santa Ana River in 1769. This event has been attributed by various geoscientists to just about every fault in the region but it could very well have been a distant event that shook a wide area as did the 1812 event, the 1971 San Fernando, the 1987 Whittier, and the 1994 Northridge events, as well as many other more-distant events (for example, 1992 Landers event).

A large earthquake occurred along the southern end of the Whittier-Elsinore fault system on April 4, 2010. The event, called the El Mayor-Cucapah earthquake, had a magnitude of 7.2. The epicenter occurred in northern Baja California, approximately 30 miles south of the Mexico-USA border at shallow depth. The aftershock zone extends from near the northern tip of the Gulf of California to 6 miles northwest of the Mexico-USA border and overlaps with the portion of the fault system that is thought to have ruptured in the Laguna Salada earthquake (M~7+) of 1892. The event was an oblique-slip event associated with surface fault ruptures as large as about 6 feet.



REFERENCE: Schoellhamer, J.E., Vedder, J.G., Yerkes, R.F., and Kinney, D.M., 1981, Geology of the Northern Santa Ana Mountains, California, Geological Survey Professional Paper 420-D, Plate 1.

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Cannon Street Widening Improvement Project	Regional Geologic Map
	Project No. 22-161 · Date: January 2024	Figure 3



Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Cannon Street Widening Improvement Project	Regional Fault Map
	Project No. 22-161 / Date: January 2024	Figure 4

6.0 SUBSURFACE CONDITIONS

6.1 Subsurface Soil Conditions

The available subsurface information indicates that the site is underlain by alluvial deposits and bedrock. The alluvial deposits generally consist of silty sand and clayey sand. The bedrock generally consists of highly weathered palagonite tuff volcanics. It should be noted that the above soil/rock description is general and is intended to describe the subsurface in very broad terms. The soil/rock description above should not be construed to mean that the subsurface profile is uniform and that soil is homogeneous within the project area. Details on stratigraphy at each borehole location are provided on the LOTB sheet and boring logs presented in Appendix A.

An idealized soil/rock profile and design strength parameters for geotechnical analyses and foundation design were developed using the available subsurface information, and are presented in Table 3. The shear strength parameters for sandy soils and bedrock (composed of sandy soils) were estimated using laboratory test data and correlations with field blowcounts (Lam and Martin, 1986). In locations where a discrepancy was observed between blowcount correlations and laboratory test results, the design strength parameters were selected using the blowcount correlations considering that the blow count correlations provide the best indication of in-situ soil strength. In Table 3, a factor of 0.65 was used to convert Modified California Drive (MCD) sampler blowcounts to Standard Penetration Test (SPT) sampler blowcounts.

Table 3. Idealized Soil/Rock Profile and Strength Parameters for Cannon Street Pedestrian Bridge

Approximate Elevation (feet)	Predominant Soil Type	Range of SPT N_{60} Blowcount (Blows/ft)	Friction Angle (degrees)	Cohesion or Undrained Shear Strength (psf)	Total Unit Weight (pcf)
Abutment 1					
+371 to +361	Clayey Sand	>50	38	100	125
+361 to +301	Bedrock	>50	38	100	125
Abutment 2					
+377 to +369	Silty Sand	7	30	0	115
+369 to +350	Bedrock	>50	38	100	125
+350 to +325	Bedrock	(37) to (>50) Average = 41	36	0	120
+325 to +306	Bedrock	>50	38	0	125
Note: Values in ( ) are MCD sampler blowcounts converted to equivalent SPT blowcounts by adjusting for sampler size.					

It should be noted that the idealized soil/rock profiles and shear strength parameters in Table 3 were developed primarily for the bridge foundation design addressed in this report. Direct application of the same idealized profiles and shear strength parameters for other design elements not specifically addressed in details in this report are likely to be invalid. This is because selecting an idealized soil/rock profile and shear strength parameters, to some extent, is influenced by the preferred design methodologies associated with bridge foundation. The same is true for the laboratory test results: the type and distribution of testing were tailored to bridge foundation design. Selective usage of one or multiple sets of test results for other design elements not specifically addressed in detail in this report will likely provide an erroneous interpretation of onsite soil/rock properties. For design elements not specifically addressed herein, we recommend supplemental field exploration and laboratory tests be performed to establish suitable and representative geotechnical design data for the specific design element.

6.2 Groundwater Conditions

Based on review of the as-built LOTB sheets of Santiago Creek Bridge Widening at Loma Street, groundwater was encountered between elevations +337 and +342 feet near the project site in 1995. During the field investigation performed by EMI for the project, groundwater was encountered between elevations +346 and +361 feet (approximately 16 to 25 feet below grade). The California Statewide Groundwater Elevation Monitoring (CASGEM) Online System was reviewed for additional groundwater level readings in the vicinity of the project site. A groundwater well located within 1 mile of the project site (State Well Number 04S09W23A004S) has recorded highest groundwater elevation of +347 feet. Ground surface elevation at this well is about +383 feet.

Groundwater depth affects liquefaction assessment and foundation design. Caltrans Geotechnical Manual on Liquefaction Evaluation (Caltrans, 2020) does not recommend using an abnormally higher groundwater elevation without clear evidence for seasonal or long-term fluctuations. This is because using abnormally high groundwater level would result in costly and unnecessary overdesign.

Based on the above data, a design groundwater table was placed at an elevation of +361 feet (about 16 to 25 feet below proposed bridge grade) for liquefaction analysis and foundation design. It should be noted that the groundwater elevation is subject to seasonal rainfall fluctuation and runoff amount, local irrigation practices, extraction and recharge of local and regional aquifers, and other manmade conditions. Therefore, the groundwater elevation during construction may be different from the design groundwater elevation provided above.

7.0 SCOUR EVALUATION

It is our understanding that Q3 Consulting is working on the scour study. Based on our discussion with the bridge designer, we understand that scour will not impact the proposed bridge foundations.

8.0 CORROSION EVALUATION

Two soil samples were tested to determine corrosivity including minimum resistivity, pH, soluble sulfate content, and soluble chloride content, and the results are summarized in Table 4.

Table 4. Soil Corrosion Test Results

Boring No.	Sample No.	Sample Depth (feet)	USCS Soil Type	Minimum Resistivity (ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
R-23-002	D-2	10	Silty Sand (SM)	790	7.5	79	276
R-23-003	S-2	10	Silty Sand with Gravel (SM)	850	7.8	95	652

According to the Caltrans Corrosion Guidelines V3.2 (Caltrans, 2021), soils are considered corrosive if the pH is 5.5 or less, or chloride concentration is 500 parts per million (ppm) or greater, or sulfate concentration is 1,500 ppm or greater. Based on the above corrosion test results and the Caltrans criteria, the on-site soil samples are not considered to be corrosive.

9.0 SEISMIC DESIGN INFORMATION AND RECOMMENDATIONS

9.1 Seismic Design

Following the procedures described in Caltrans Seismic Design Criteria Version 2.0 (SDC 2.0) (2019a) and October 2019 Interim Revisions to SDC 2.0 (2019b), the design ARS curve for a 975-year Return Period was determined using the Caltrans ARS Online V3.1.0 (2023a) and utilizing the small-strain shear wave velocity for the upper 100 feet (V_{s30}). This V_{s30} was estimated from the information presented in the LOTB sheet included in Appendix A and the SPT correlations provided in the Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (Caltrans, 2012). The key parameters for determining the design ARS curves are provided in Table 5.

Table 5. Key Parameters for Determining Design ARS Curves

Site Latitude	33.8147°
Site Longitude	-117.7955°
Shear Wave Velocities, V_{s30}	1,017 feet/sec (310 m/sec)

The design ARS curve is presented in Figure 5. The design magnitude (M) is 6.61, the design Peak Ground Acceleration (PGA) is 0.55g, and the mean site-to-fault distance at 1.0 second period is 13.6 miles. Based on the subsurface information and per Sections 6.1.2 and 6.1.3 of SDC 2.0 (2019b), the onsite soils are classified as "Class S1" soils.

9.2 Liquefaction Potential

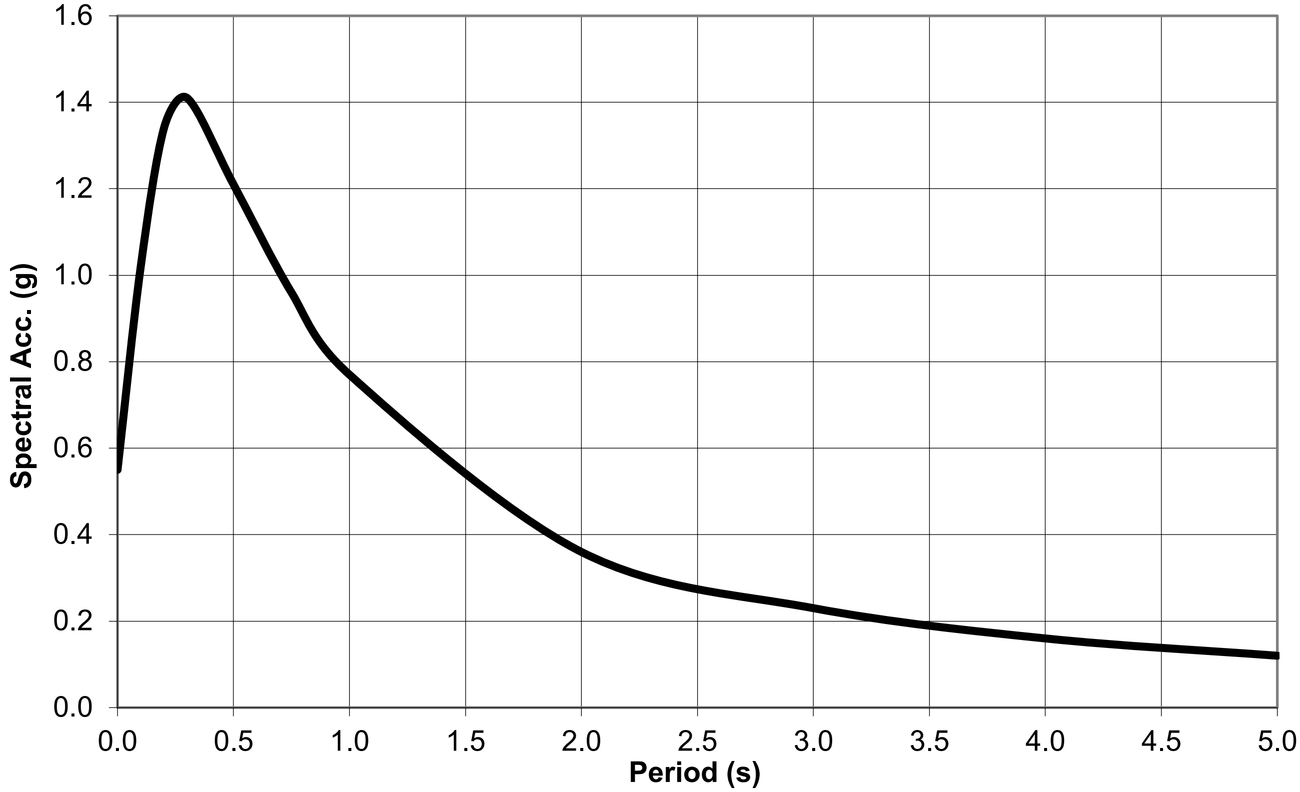
As stated in Section 6.2, the design groundwater table was placed at an elevation of +361 feet (about 16 to 25 feet below proposed bridge grade). Liquefaction analysis was performed using the site-specific data collected from the boreholes done by EMI. The liquefaction potential of saturated, coarse-grained soils below the design groundwater elevation was evaluated using the procedures outlined by Youd, et al. (2001). Per the Caltrans Geotechnical Manual for Liquefaction Evaluation (2020), the evaluation was limited to 70 feet below the ground surface. Results of the analyses indicate that liquefaction potential does not exist at the wall site.

9.3 Seismically-Induced Settlement

Since liquefaction potential does not exist at the project site, seismically-induced settlement is expected to be negligible, and therefore, not expected to impact the proposed bridge foundations.

9.4 Lateral Spreading

Since liquefaction potential does not exist, lateral spreading is not considered a design issue. Results of the pseudo-static slope stability analyses of abutment end-slopes are presented in Section 10.5.



Design ARS Curve

Latitude: 33.8147 Longitude: -117.7955

Shear Wave Velocity (V_{s30}) = 310 m/s

Design Magnitude (M) = 6.61

Peak Ground Acceleration (PGA) = 0.55g

5% Damping

Spectral Coordinates:

Period (sec)	**Acc.** (g)	**Period** (sec)	**Acc.** (g)
0.00	0.55	1.00	0.77
0.10	1.02	2.00	0.36
0.20	1.34	3.00	0.23
0.30	1.41	4.00	0.16
0.50	1.21	5.00	0.12
0.75	0.96		

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Project Number : 22-161		Date:	Jan. 2024
CANNON STREET PEDESTRIAN BRIDGE			
Design Acceleration Response Spectrum		Figure 6	

9.5 Ground Rupture

No major faults traverse through the project site. The California Geological Survey has not identified Alquist-Priolo Fault Zones through the site. Therefore, the risk of ground surface rupture and related hazards at the project site are expected to be low. According to Caltrans Memo To Designers 20-10 (Caltrans, 2013), since the project site does not fall within an Alquist-Priolo Earthquakes Fault Zone or within 1,000 feet of an unzoned fault that is Holocene or younger in age, further fault studies will not be needed.

10.0 FOUNDATION RECOMMENDATIONS

10.1 Foundation Type

Based on the information provided by the bridge designer, 24-inch diameter Cast-In-Drilled-Hole (CIDH) piles are proposed at the bridge supports.

10.2 Axial Pile Capacity

Per Caltrans policy, Load Resistance Factor Design (LRFD) is used for foundation design. The foundation design data sheet and foundation factored design loads were provided by the bridge designer following the latest Caltrans Memo to Designers 3-1 (Caltrans, 2014b), and are shown in Table 6 and Table 7, respectively.

Table 6. Foundation Design Data Sheet

Support No.	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inch)	Number of Piles per Support
				B	L		
Abut 1	24-inch CIDH	365.50	365.25	NA	NA	2	3
Abut 2		376.25	376.00	NA	NA	2	3

Based on the information provided by the bridge designer, the on-center spacing between two piles is at-least 3 pile diameters. Based on California Amendments to AASHTO LRFD Bridge Design Specifications – Eighth Edition (Caltrans, 2019c) and the pile spacing provided by the bridge designer, a group reduction factor is not required in the axial pile capacity calculations.

The axial capacities were estimated using the computer program SHAFT v2017 (Ensoft, 2017). The axial pile capacities are based on soil resistance only and may be further limited by the pile-head connection details and structural material strength. The calculated pile tip elevations are presented in Table 8. The pile data table is presented in Table 9.

Table 7. 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	320	260	510	180	-	-	-	-	-	-
Abut 2	210	250	500	170	-	-	-	-	-	-

Table 8. Foundation Design Recommendations

<table>
<tr><th rowspan="3">Sup. No.</th><th rowspan="3">Pile Type</th><th rowspan="3">Cut-off El. (feet)</th><th colspan="2" rowspan="2">Service-I Limit State Load per Support (kips)</th><th rowspan="3">Total Permissible Support Settlement (in.)</th><th colspan="4">Nominal Resistance (kips)</th><th rowspan="3">Design Tip El. (feet)</th><th rowspan="3">Spec. Tip El. (feet)</th></tr>
<tr><th colspan="2">Strength / Construction</th><th colspan="2">Extreme Event</th></tr>
<tr><th>Total</th><th>Perm</th><th>Comp φ=.7</th><th>Tens φ=.7</th><th>Comp φ=1</th><th>Tens φ=1</th></tr>
<tr><td>Abut 1</td><td rowspan="2">24-inch CIDH</td><td>+365.25</td><td>320</td><td>260</td><td>2</td><td>260</td><td>0</td><td>0</td><td>0</td><td>+326(a-I)
+345(c)
+335(d)</td><td>+326</td></tr>
<tr><td>Abut 2</td><td>+376.00</td><td>210</td><td>250</td><td>2</td><td>250</td><td>0</td><td>0</td><td>0</td><td>+344(a-I)
+356(c)
+348(d)</td><td>+344</td></tr>
<tr><td colspan="12">Notes:
(1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load.
(2) The Specified Tip Elevation shall not be raised.
(3) Column heading modified from “Required Factored Nominal Resistance” to “Nominal Resistance”.</td></tr>
</table>

Table 9. Pile Data Table

<table>
<tr><th rowspan="2">Support No.</th><th rowspan="2">Pile Type</th><th colspan="2">Nominal Resistance (kips)</th><th rowspan="2">Design Tip Elevation (feet)</th><th rowspan="2">Specified Tip Elevation (feet)</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>Abut 1</td><td rowspan="2">24-inch CIDH</td><td>260</td><td>0</td><td>+326(a)
+345(c)
+335(d)</td><td>+326</td></tr>
<tr><td>Abut 2</td><td>250</td><td>0</td><td>+344(a)
+356(c)
+348(d)</td><td>+344</td></tr>
<tr><td colspan="6">Notes:
(1) Design Tip Elevations are controlled by the following demands: (a) Compression, (b) Tension, (c) Settlement, and (d) Lateral Loads.
(2) The Specified Tip Elevation shall not be raised.</td></tr>
</table>

10.3 Lateral Pile Solutions

Lateral single-pile analyses were performed for a fixed-head loading condition using the idealized soil profiles provided in Table 3 and computer program LPILE v2019 (Ensoft, 2019). The LPILE generated p-y curves for sandy soils were estimated using the API criteria (API, 2000). Group Efficiency Factor was determined in accordance with the procedures outlined in California Amendments to AASHTO LRFD Bridge Design Specifications – Eighth Edition (Caltrans, 2019c) and the pile layouts provided by the bridge designer. A Group Efficiency Factor of 0.9 was used at each abutment for loadings along the longitudinal direction. The resulting pile-head shear capacity and maximum bending moment caused by lateral pile-head deflections are provided in Table 10 along with the location of maximum bending moment.

Table 10. Lateral Pile Solutions for Fixed-Head Loading Condition

Support Location (Direction)	Pile Type	Pile Head Deflection (inch)	Pile Head Shear (kip)	Max Moment (kip-in)	Depth to Max. Moment from Pile Top (feet)
Abutment 1 (Longitudinal)	24-inch CIDH	0.25	110	3,912	0
		0.5	161	6,697	0
		1	229	11,075	0
		2	301	17,666	0
Abutment 2 (Longitudinal)	24-inch CIDH	0.25	66	3,040	0
		0.5	113	5,624	0
		1	173	9,650	0
		2	245	15,649	0

The solutions presented in Table 10 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. Lateral pile solutions are provided for pile-head deflections from 0.25 to 2 inches, and linear interpolation can be used for intermediate pile-head deflections.

10.4 Bridge Abutment Wall Earth Pressures

If abutment walls are free to move laterally at the top, a static active lateral earth pressure of 36 psf per foot of depth is recommended for a free draining, level and compacted backfill. If lateral movement at the top of abutment walls is restrained, the lateral earth pressure for a free draining, level and compacted backfill should follow Section 5.5.5.11 of the Caltrans Bridge Design Specification (Caltrans, 2004). For this condition, we recommend a coefficient of active lateral earth pressure of 0.3, a coefficient of at-rest lateral earth pressure of 0.47, and a soil unit weight of 120 pcf.

In accordance with AASHTO LRFD Bridge Design Specifications – Eighth Edition (2017), Section 3.11.6.4, a uniform lateral pressure due to traffic loading should be applied. Based on the abutment height, the vertical pressure shall be produced by an equivalent height of earth with a soil unit weight of 120 pcf. For abutment walls that are free to move laterally at the top, a coefficient of active lateral earth pressure of 0.3 is recommended; for abutment walls where lateral movement at the top of the abutment walls is restrained, a coefficient of at-rest lateral earth pressure of 0.47 is recommended.

Section 6.3.1 of the Caltrans SDC 2.0 (2019b) can be used to estimate the abutment longitudinal stiffness and idealized ultimate passive backfill capacity under seismic loading. This section also describes the procedure used to determine the effective abutment longitudinal displacement to reach the idealized ultimate passive earth capacity for both seat-type and diaphragm abutments. It should be noted that the equations given in Section 6.3.1 were developed by backfitting experimental data obtained for backwall heights equal to and between 2 and 10 feet, and retaining structural backfill with a relative compaction of at least 95 percent. Therefore, there are uncertainties when applying these same equations for abutment wall heights greater than 10 feet or for backfill compacted to a lesser relative compaction.

10.5 Approach Embankments

Sliver fills will be required to construct the approaches for the proposed bridge. Based on the cross-sections provided by MTC, up to 7 feet and 10 feet of fill will be placed at Abutment 1 and Abutment 2, respectively, to bring the existing grade to the proposed grade.

Settlement and Settlement Period. Based on the settlement calculations, the maximum ground settlement due to fill placement is less than an inch at Abutment 1 and about 1.5 inches at Abutment 2. A settlement period of 14 days is recommended for Abutment 2 pile construction.

Global Stability. Global stability analyses were conducted for both static and pseudo-static conditions for the bridge approach embankments for potential deep-seated failures below the abutment footing. The analysis was performed using the computer program Slide 2 (Rocscience, 2020). The material used for the proposed embankment fill was modeled with a friction angle of 34 degrees and a cohesion of 100 psf.

Slope stability analyses were conducted for the static condition including a 2-foot soil surcharge to represent traffic loading. In accordance with Caltrans guidelines (2014a), stability analysis for the seismic condition was performed using the pseudo-static approach with a seismic coefficient of 0.183 for Cannon Street Pedestrian Bridge, which is equal to one-third PGA.

According to the results of the analyses, the approach embankments meet the minimum required factor-of-safety for deep-seated failure of 1.5 for the static condition and 1.1 for the pseudo-static condition per Caltrans guidelines (2014a).

11.0 PAVEMENT STRUCTURAL SECTION DESIGN

Four R-value tests were performed from the surface samples collected from borings HA-23-001 and HA-23-004 through HA-23-006 and the resulting R-values are 50, 7, 4, and 8, respectively. We recommend using a R-value of 4 for pavement design.

Flexible pavement sections were designed using the CalME V3.0 (2022) computer program developed by Caltrans and the methodology given in Chapter 630 of Caltrans Highway Design Manual (2023b). The flexible pavement sections were determined for a 20-year design life and Traffic Indices (TIs) between 5 and 10. The recommended flexible pavement sections are presented in Table 11.

Table 11. Recommended Flexible Pavement Structural Sections

Location	Design R-Value	Design Life (Years)	TI	Recommended Pavement Sections
Cannon Street	4	20	5.0	0.30’ HMA-A / 0.50’ AB
			6.0	0.40’ HMA-A / 0.50’ AB
			7.0	0.45’ HMA-A / 0.60’ AB
			8.0	0.50’ HMA-A / 0.60’ AB
			9.0	0.60’ HMA-A / 0.70’ AB
			10.0	0.70’ HMA-A / 0.75’ AB
Note: HMA-A = Hot Mix Asphalt (Type A); AB = Aggregate Base (Class 2).				

In locations where structural pavement sections will be constructed atop import fill (Select) Material, the Select Material placed within four feet of the grading plane should have a Plasticity Index of less than or equal to 12%. Also, minimum R-value should be equal to or greater than the design R-value. Otherwise, remedial removals will need to be performed to replace the subgrade soils with materials possessing Plasticity Index less than or equal to 12% and R-value equal to or greater than the design R-value.

12.0 CONSTRUCTION CONSIDERATIONS

12.1 Earthwork

Earthwork should be performed in accordance with Section 19 of the Caltrans Standard Specifications (2023a). Appropriate measures should be taken to prevent damage to adjacent existing structures and utilities.

In areas where compacted fill will be placed, complete removal of compressible surficial materials including vegetation, topsoil, loose or soft alluvium, dry or saturated soil, wet, unstable, or otherwise unsuitable material is required prior to fill placement. A minimum overexcavation and recompaction of 12 inches is recommended within all areas to receive compacted fill, and the overexcavation depth is measured from existing grade. The overexcavation should extend horizontally a minimum distance of 24 inches from edges of new fills. In cut areas, the minimum overexcavation and recompaction depth is 12 inches if the difference between the finished and existing grade is 2 feet or less, and overexcavation is not required if the difference between the finished and existing grade is greater than 2 feet. In cut areas, the overexcavation depth is measured from the grading plane. Unless specified on the contract plans or specifications, the excavated soils (in both fill and cut areas) may be reused as compacted fill. Actual depths and extent of remedial removals should be determined in the field by qualified geotechnical personnel during earthwork activities. 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 density determined by California Test (CT) 216.

Remedial earthwork beneath pavement structural sections should follow Section 11.0.

12.2 Temporary Excavations

Design of temporary construction slopes and shoring is the contractor's responsibility during construction. Heavy construction equipment should not be used immediately adjacent to shoring due to large lateral pressures induced by such equipment unless the shoring is designed to accommodate resulting pressures. Excavated soil or construction materials should not be stockpiled adjacent to shoring or open excavations. Stockpiled soil and construction materials should be set back a distance at least equal to the height of the excavation. 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 safety and health standards, rules, regulations, and orders established by the State of California. If a trench shoring design and safety plan is required, the geotechnical consultant should review the plan to confirm that recommendations presented in this report have been applied to the design.

The contractor is responsible for evaluating the ease/difficulty of installing and extracting structural elements for temporary shoring walls in contact with the ground.

12.3 Groundwater Control

During the field investigation performed by EMI for this project, groundwater was encountered between elevations +346 and +361 feet (approximately 16 to 25 feet below grade). Therefore, groundwater is expected to be encountered during construction at shallow depth. Should groundwater be encountered during footing construction, it should be controlled in accordance with Section 19-3.03B(5) of the Caltrans Standard Specification (2023c). Any seepage or groundwater removed from an excavation should be tested and disposed of in compliance with all applicable local, state, and federal requirements.

It shall be made the contractor's responsibility to control subsurface and surface water. The contractor should dewater the site as necessary, if groundwater is encountered. Contractor should also be cognizant that any dewatering activities could induce ground subsidence which affects adjacent surface and subsurface structures and utilities. Water should not be allowed to stand in any excavations. If excavations become flooded, at-least the bottom 8 inches of soil should be removed and replaced, and re-compacted to a minimum 90 percent relative compaction. Additional removals may be required at the discretion of the resident engineer or geotechnical personnel.

12.4 Preload and Settlement Period

As discussed in Section 10.5, preloading and a minimum settlement period of 14 days is specified for Abutment 2 pile construction. This settlement period involves placing earthen embankments per Caltrans Standard Plan Sheet A62B (2023d), with no surcharge, to preload the approach area. The settlement period starts after completion of the preload embankment. Once the settlement period is complete, the earthen embankment will be completely removed and the approach areas will be available for abutment pile construction.

12.5 CIDH Pile Construction

Construction of CIDH piles should follow Section 49-3.02 of the Caltrans Standard Specifications (2023c). Per Caltrans Standard Plan B2-3 (2023d), a minimum of 3-inch of concrete cover over reinforcement should be provided to improve the construction of 24-inch diameter CIDH piles.

Exploratory borings performed for this project encountered difficult drilling conditions due to bedrock. The contractor should anticipate that penetration will be slow; casing (if used) installation into these materials will also be difficult. Hard drilling should be anticipated.

Loose soils should be cleaned from the bottom of the drilled excavations. Pile borings should be inspected and approved by the geotechnical engineer prior to the 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 will be required. Sufficient space should be provided in the pile reinforcing cage during fabrication to allow the insertion of a tremie tube for concrete placement.

Onsite soils are susceptible to caving. Contractor is responsible for evaluating the use of casing, drilling fluid or other means to control caving. Casings, when used, shall conform to Section 49-

3.02 of Caltrans Standard Specifications (2023c). A full-length temporary casing can be allowed for controlling caving. The contractor is responsible for evaluating the soil conditions to determine the minimum plug inside the temporary casing necessary to prevent migration of material from outside the casing into the shaft excavation.

For wet pile construction, the contractor should be required to maintain a minimum 10 feet head of slurry over the piezometric surface at all times during CIDH pile construction. The minimum 10 feet head of slurry should be required during shaft excavation to prevent a “quick” condition at the bottom of the CIDH pile excavation. Water should not be allowed as slurry, even if full length casing is used during shaft excavation.

In the event that any boring becomes bell-shaped and cannot be advanced, all loose material should be removed from the bottom of the boring and the caved region filled with a low strength sand-cement slurry. Drilling may continue when the slurry has reached its initial set.

The above information is not meant to direct the pile contractor to excavate and build the CIDH piles; any construction means and methods remain the responsibility of the pile contractor.

12.6 Backdrain and Backfill Requirements for Abutment Walls

Caltrans Structure Backfill should be used as backfill material behind the bridge abutment walls (see Figure 6). Backfill should be compacted in accordance with Section 19-5 of the Caltrans Standard Specifications (2023c). 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 percent 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 in order to avoid damaging the walls due to large lateral earth pressures.

Backdrains should be installed behind abutment walls to relieve hydrostatic pressure. Backdrains should be constructed in accordance with Sheet B9-6 per Caltrans Standard Plans (2023d) or the bridge plans.

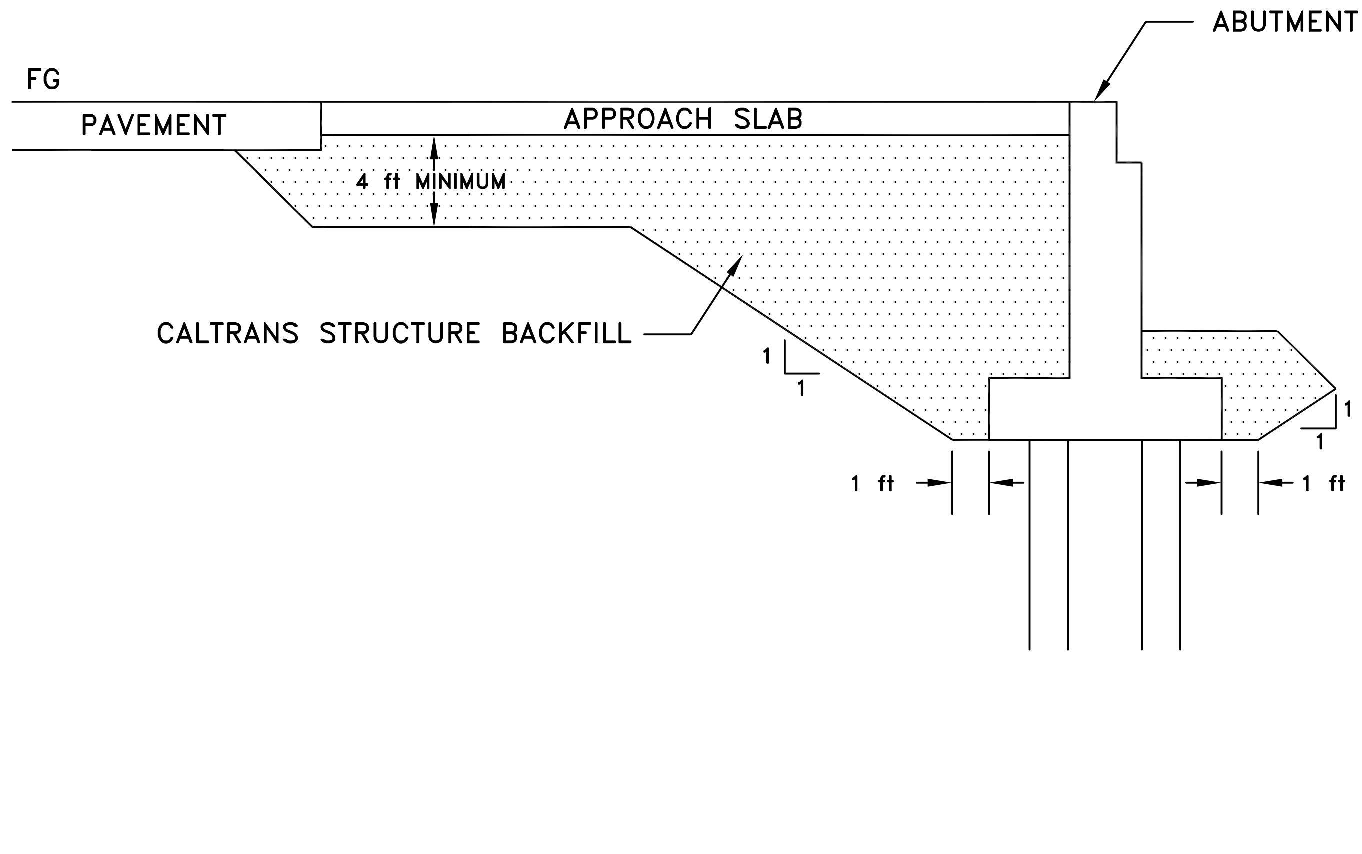
12.7 Review of Construction Plans and Specifications

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

12.8 Geotechnical Observation and Testing

Qualified geotechnical personnel should perform inspections and testing during the following stages of construction:

- Grading operations, including temporary and permanent excavations and placement of compacted fill.
- Placement of structure backfill behind retaining walls.
- Placement of subdrain pipes and prefabricated geocomposite drains.
- Shoring installation, if necessary.
- Footing excavations.
- Preparation of foundation subgrades.
- Construction of CIDH piles.
- Backdrain installation and backfilling of bridge abutment walls.
- Removal of existing pavement structural sections, curb and gutter, and concrete sidewalk.
- Preparation of pavement subgrade.
- Placement of aggregate base and surface course.
- Removal or installation of support of buried utilities or structures.
- When any unusual subsurface conditions are encountered.



Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Cannon Street Pedestrian Bridge

Project No. 22-161

Date: January, 2024

BRIDGE ABUTMENT WALL BACKFILL

FIGURE 6

13.0 LIMITATIONS

This report is intended for use by Mark Thomas & Company, Inc. and City of Orange for the design and construction of the Cannon Street Widening Improvement Project. This report is based on the project as described herein and the information obtained from the exploratory borings at the approximate locations indicated on the attached figure and LOTB sheet. The findings and recommendations contained in this report are based on the results of the field investigation, laboratory tests, and engineering analyses. Also, 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. Findings reflect an interpretation of the direct evidence obtained. Recommendations presented herein are based on the assumption that an appropriate level of quality control and quality assurance (inspections and tests) will be provided during construction. EMI has no responsibility for errors and incompleteness of available design drawings and assumptions made by EMI due to these errors and incomplete information. EMI should be notified of any pertinent changes in the project plans or if subsurface conditions are found to vary from those described herein. Modifications to the project plans or variations in subsurface conditions may require 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, or 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.

14.0 REFERENCES

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APPENDIX A

LOG-OF-TEST-BORING SHEET AND BORING LOGS

LOG-OF-TEST-BORINGS SHEET

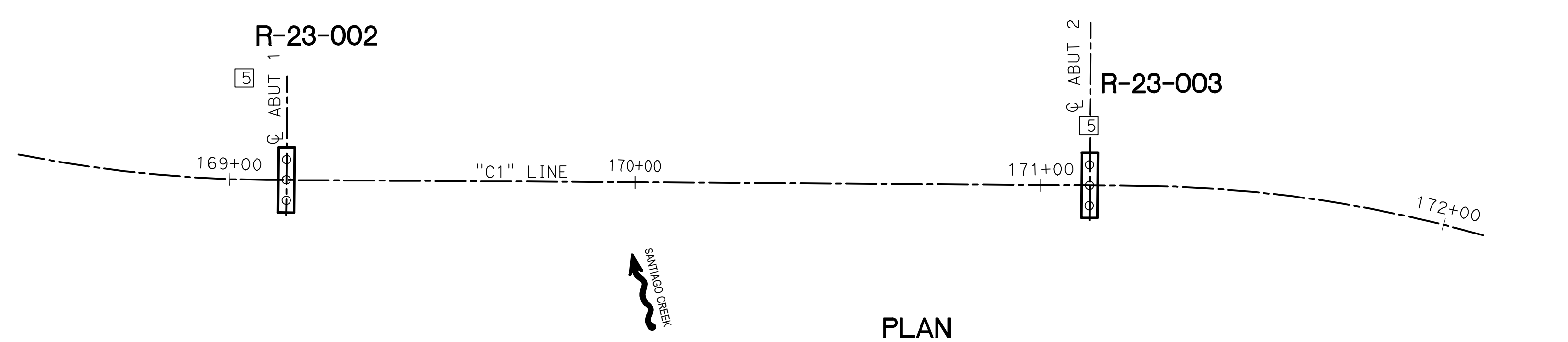
NOTES:

(1) This LOTB sheet (Boring Record) was prepared in accordance with Caltrans Soil and Rock Logging, lassification and Presentation Manual (2022).

(2) 1.4" samples were taken using a Standard Penetration Sampler and 2.4" samples were taken using a California Modified Sampler; multiply California Modified Sampler blowcounts by 0.65 to convert to equivalent SPT blowcounts.

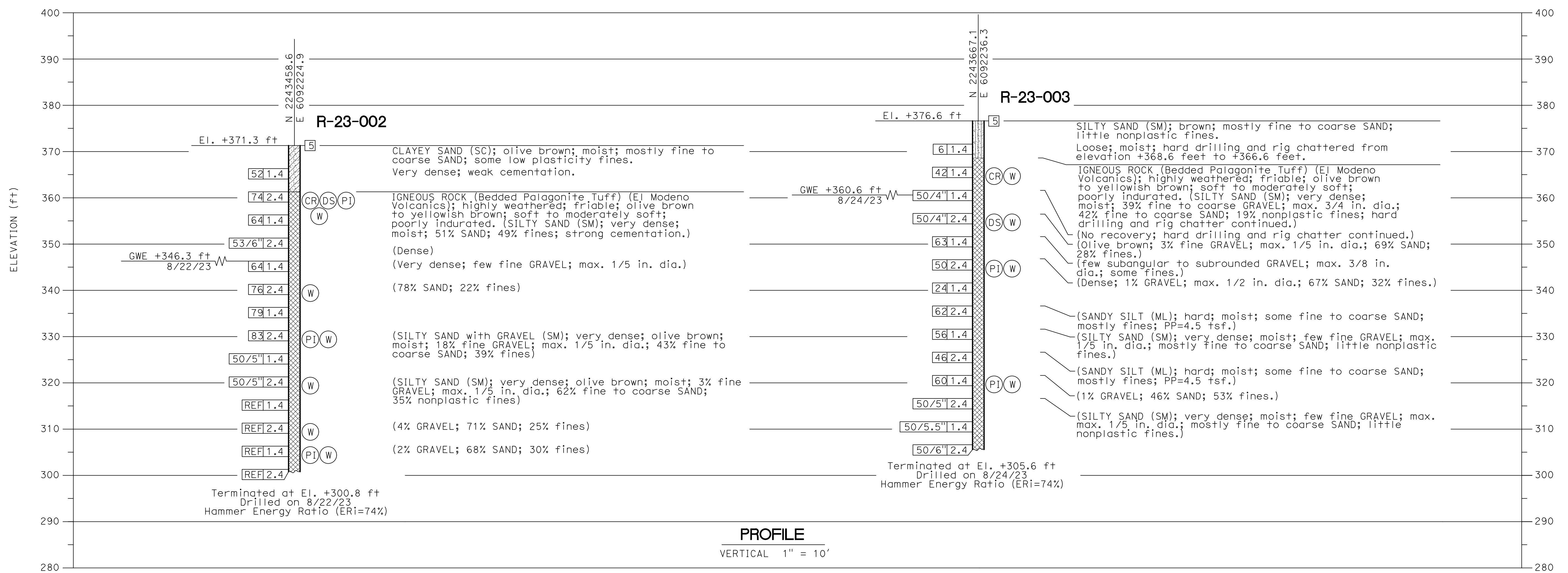
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" drop was used to advance the samplers.

(4) For Soil Legend, see A10F and A10G of the Caltrans Standard Plans.



PLAN

1" = 20'



PROFILE

VERTICAL 1" = 10'

BOREHOLE LOCATION TABLE

Hole ID	Alignment Name	Station and Offset
R-23-002	"C1" Line	169+03 24.3 Lt.
R-23-003	"C1" Line	171+12 15.2 Lt.

PRELIMINARY ENGINEERING

CITY OF ORANGE

OFFICE OF THE CITY ENGINEER

IMPROVEMENT PLANS

SANTIAGO CREEK BIKE TRAIL NORTH

FROM CANNON STREET TO SANTIAGO OAKS REGIONAL PARK

CANNON STREET PEDESTRIAN BRIDGE

LOG-OF-TEST-BORINGS 1 OF 1

REGISTERED PROFESSIONAL ENGINEER ALAHESWARAN THURAIRAJAH NO. GE 3123 EXP. 9-30-2025 GEOTECHNICAL STATE OF CALIFORNIA

	NAME	DATE
DRAWN	TN	
DESIGNED		
CHECKED	PR	

APPROVED: DATE CITY ENGINEER

APPROVED: DATE PUBLIC WORKS DIRECTOR

SCALE: HORIZ. AS NOTED VERT. AS NOTED F.B.____ PG.____ SHEET### OF # SHEETS

22-00092

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

X:\Projects\2022\22-161 - MTC, Cannon St Widening\LOTB\Cannon St LOTB.dwg
By: Taylor Nguyen on 1/25/2024 10:19 AM

BORING LOGS

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)

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BORING RECORD LEGEND

Cannon Street Widening Improvement Project

Project Number: 22-161	Date: 1-25-24	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.

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
KK	8-23-23	8-23-23	N 2243316.0 E 6092328.0	HA-23-001

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
EMI	31.30' Rt Sta 67+63 "C"	372.3 ft
DRILLING METHOD	**DRILL RIG**	**BOREHOLE DIAMETER**
Hand Auger	Hand Auger	4 in
SAMPLER TYPE(S) AND SIZE(S) (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
Bulk	N/A	N/A
BOREHOLE BACKFILL AND COMPLETION	**GROUNDWATER READINGS / GROUNDWATER DEPTH**	**TOTAL DEPTH OF BORING**
Cuttings	NE	5.0 ft

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		SILTY SAND (SM); brown; moist; 14% fine to coarse GRAVEL, max. 1.5 in. dia.; 57% fine to coarse SAND; 29% nonplastic fines.		0			100		20					PI, W
	1														
370.30	2														
	3														
368.30	4														
	5		Bottom of borehole at 5.0 ft bgs												
366.30	6														
	7														
364.30	8														
	9														
362.30	10														
	11														
360.30	12														
	13														
358.30	14														
	15														
356.30	16														
	17														
354.30	18														
	19														
352.30	20														
	21														
350.30	22														
	23														
348.30	24														
	25														

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
KK	**8-23-23**	**8-23-23**	**N 2244103.0 E 6092273.0**	**HA-23-004**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
EMI	**41.00' Lt Sta 75+51 "C"**	**410.7 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hand Auger	**Hand Auger**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk	**N/A**	**N/A**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS / GROUNDWATER DEPTH	TOTAL DEPTH OF BORING
Cuttings	**NE**	**5.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		SANDY lean CLAY (CL); dark brown; moist; 7% fine to coarse GRAVEL, max. 3/4 in. dia.; 33% fine to coarse SAND; 60% medium plasticity fines.		0			100		13					PI, W
	1														
408.65	2														
	3														
406.65	4														
	5		Bottom of borehole at 5.0 ft bgs												
404.65	6														
	7														
402.65	8														
	9														
400.65	10														
	11														
398.65	12														
	13														
396.65	14														
	15														
394.65	16														
	17														
392.65	18														
	19														
390.65	20														
	21														
388.65	22														
	23														
386.65	24														
	25														

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REPORT TITLE	HOLE ID
BORING RECORD	**HA-23-004** Project No. **22-161**
PROJECT NAME	
Cannon Street Widening Improvement Project	
PREPARED BY	DATE **1-25-24** / SHEET **1 of 1**

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
KK	**8-25-23**	**8-25-23**	**N 2244736.0 E 6092100.0**	**HA-23-005**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
EMI	**45.90' Lt Sta 82+10 "C"**	**433.0 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hand Auger	**Hand Auger**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk	**N/A**	**N/A**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS / GROUNDWATER DEPTH	TOTAL DEPTH OF BORING
Cuttings	**NE**	**5.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		SANDY lean CLAY (CL); reddish brown; moist; 1% fine to coarse GRAVEL, max. 3/4 in. dia.; 37% fine to coarse SAND; 62% medium plasticity fines.		0			100		13					PI, W
	1														
431.00	2														
	3														
429.00	4														
	5		Bottom of borehole at 5.0 ft bgs												
427.00	6														
	7														
425.00	8														
	9														
423.00	10														
	11														
421.00	12														
	13														
419.00	14														
	15														
417.00	16														
	17														
415.00	18														
	19														
413.00	20														
	21														
411.00	22														
	23														
409.00	24														
	25														

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REPORT TITLE	HOLE ID
BORING RECORD	**HA-23-005** Project No. **22-161**

PROJECT NAME		
Cannon Street Widening Improvement Project		
PREPARED BY	DATE **1-25-24**	SHEET **1 of 1**

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
KK	**8-25-23**	**8-25-23**	**N 2245191.0 E 6092086.0**	**HA-23-006**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
EMI	**44.00' Lt Sta 86+69 "C"**	**450.2 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Hand Auger	**Hand Auger**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Bulk	**N/A**	**N/A**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS / GROUNDWATER DEPTH	TOTAL DEPTH OF BORING
Cuttings	**NE**	**5.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		CLAYEY SAND (SC); reddish brown; moist; 3% fine to coarse GRAVEL, max. 3/4 in. dia.; 49% fine to coarse SAND; 48% low plasticity fines.		0			100		9					PI, W
	1														
448.20	2														
	3														
446.20	4														
	5		Bottom of borehole at 5.0 ft bgs												
444.20	6														
	7														
442.20	8														
	9														
440.20	10														
	11														
438.20	12														
	13														
436.20	14														
	15														
434.20	16														
	17														
432.20	18														
	19														
430.20	20														
	21														
428.20	22														
	23														
426.20	24														
	25														

APPENDIX B

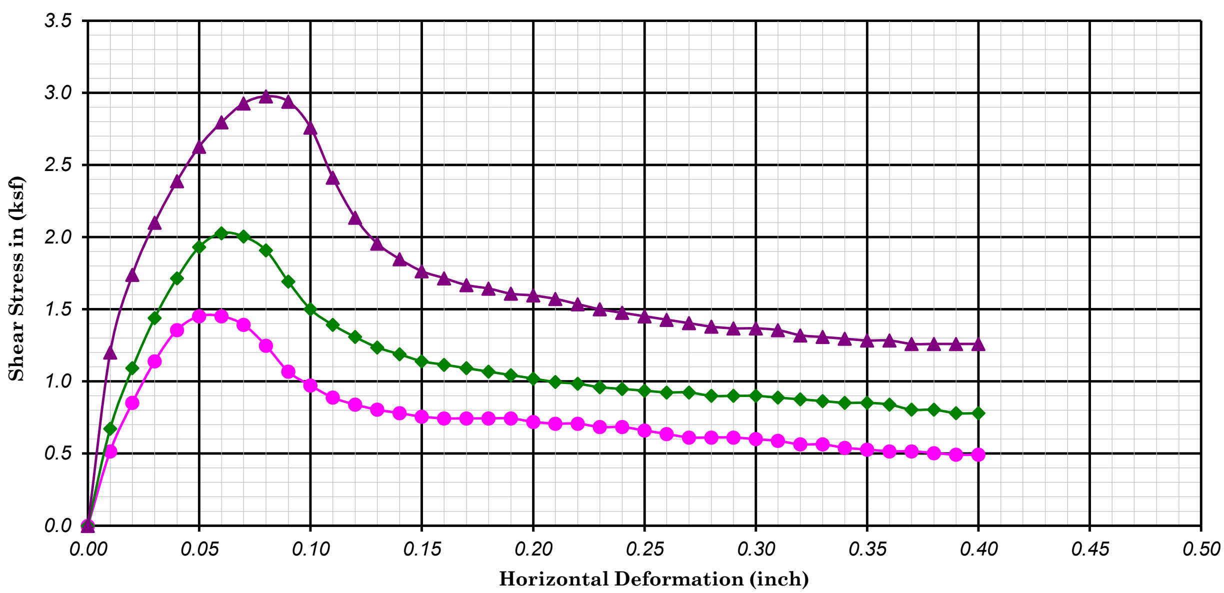
LABORATORY TEST RESULTS

TABLE B-1 SUMMARY OF LABORATORY TEST RESULTS

Project No. : *22-161* **Project Name :** MTC, Cannon St Widening

Boring No .	Sample No.	Sample Depth (ft)	Soil Identification (group symbol) ASTM D2488/D2487	Moisture Content ASTM D2216 (%)	Total Unit Weight ASTM D2937 (pcf)	Pocket Penetrometer (tsf)	Torvane Shear (tsf)	Grain Size Distribution GR:SA:FI (%)	Sand Equivalent (CT-217)	Atterberg Limits ASTM D4318 (LL/PL/PI)	Soil-Minimum Resistivity CT-643 (ohm-cm)	Soil-pH CT-643	Soil-Soluble Sulfate Content CT-417 (ppm)	Soil-Moisture Free Chloride Content CT-422 (ppm)
HA-23-001	*B-0*	0	SM	19.7				14:57:29		36/28/8				
R-23-002	*D-2*	10	SM	36.7	108.8	>4.5		0:51:49		38/36/2	790	7.5	276	79
R-23-002	*D-4*	20	SM	25.8	110.1									
R-23-002	*D-6*	30	SM	33.1	107.8			0:78:22						
R-23-002	*D-8*	40	SM	22.7	121.2	>4.5		18:43:39		37/30/7				
R-23-002	*D-10*	50	SM	31.8	110.6	3.9		3:62:35						
R-23-002	*D-12*	60	SM	26.5	131.6			4:71:25						
R-23-002	*S-13*	65	SM	25.6				2:68:30		39/38/1				
HA-23-006	*B-0*	0	SC	8.8				3:49:48		33/16/17				
R-23-003	*S-2*	10	SM	21.1				39:42:19			850	7.8	652	95
R-23-003	*D-4*	20	SM	35.2	116.4	>4.5		3:69:28						
R-23-003	*D-6*	30	SM	34.4	116.0	>4.5		1:67:32		33/31/2				
R-23-003	*D-8*	40	ML	45.3	113.4	4.5								
R-23-003	*D-10*	50	ML	51.7	107.6	4.5								
R-23-003	*S-11*	55	ML	46.6				1:46:53		39/31/8				
R-23-003	*D-12*	60	SM	46.7	112.7									
R-23-003	*D-14*	70	SM	36.1	117.0									
HA-23-004	*B-0*	0	CL	13.3				7:33:60		35/16/19				
HA-23-005	*B-0*	0	CL	13.5				1:37:62		39/15/24				

Ultimate : ○	Shear Type :	Field Moisiture	Undisturbed	Peak : ●



Boring No.	: R-23-002		Strength Intercept (C) :	0.98	(ksf)	Peak	0.25	(ksf)	Ultimate
Sample No.	: D-2			46.83	(kPa)		12.07	(kPa)	
Depth (ft/m)	: 10.0	0.00	Friction Angle (ϕ) :	45.18	Degree		26.90	Degree	
Description	: Dark olive yellow, SILTY SAND (SM)						Shear Rate (inch/minute) :	0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	36.71	79.59	12.53	1.12	0.50	23.94	1.45	69.52	0.49	23.56
◆	37.76	75.08	11.82	1.25	1.00	47.88	2.03	97.10	0.78	37.35
▲	35.28	80.98	12.75	1.08	2.00	95.76	2.98	142.49	1.26	60.33

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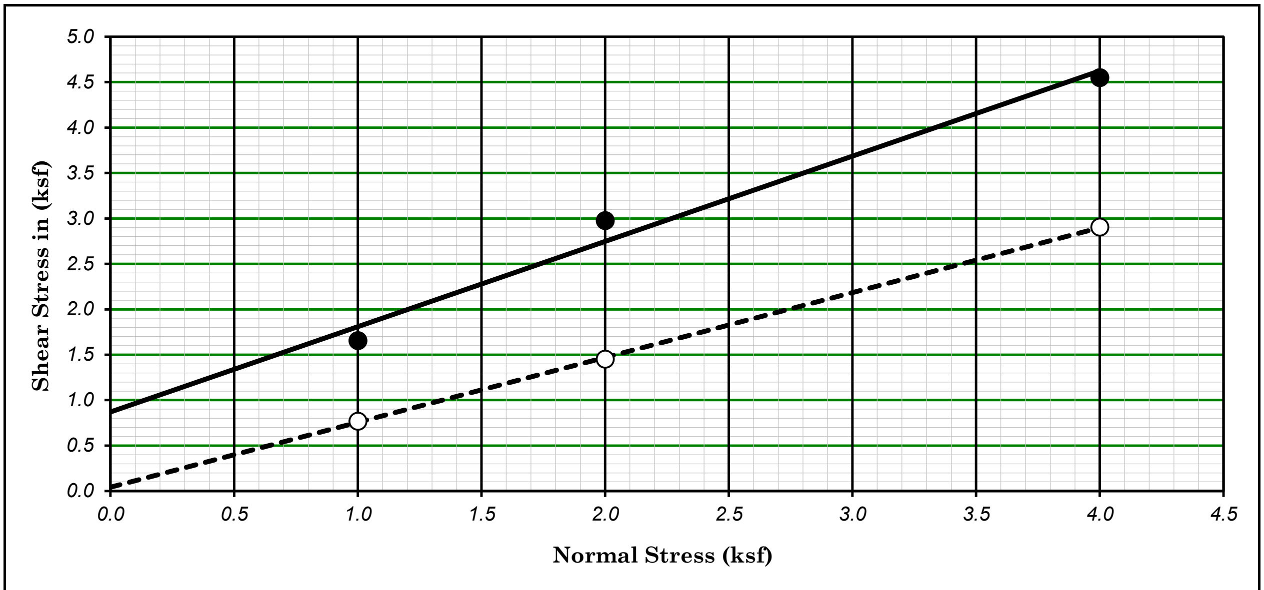
MTC, Cannon St Widening

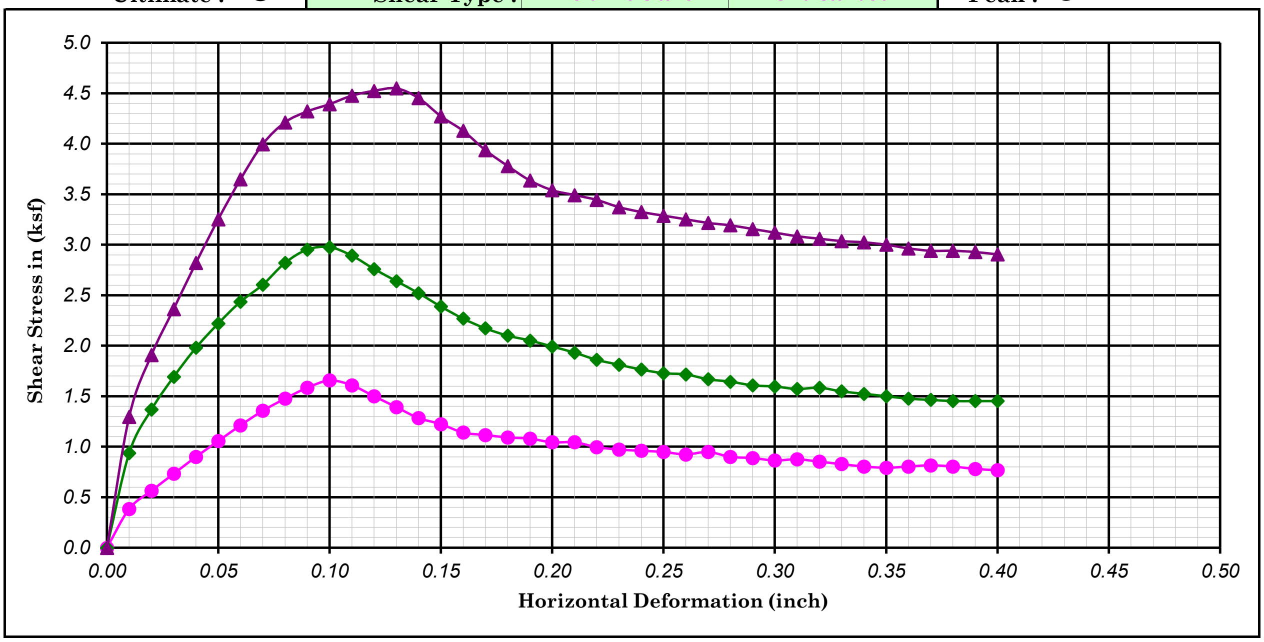
DIRECT SHEAR TEST
(ASTM D-3080)

Project No. : 22-161 **Date :** 09/11/23

Figure No.



Ultimate : ○ | Shear Type : *Field Moisture* | *Undisturbed* | Peak : ●



Boring No.	: R-23-003	Strength Intercept (C) :	0.87	(ksf)	Peak	0.04	(ksf)	Ultimate
Sample No.	: D-4		41.66	(kPa)		2.01	(kPa)	
Depth (ft/m)	: 20.0 / 0.00	Friction Angle (ϕ) :	43.19	Degree		35.53	Degree	
Description	: Dark olive yellow, SILTY SAND (SM)					Shear Rate (inch/minute) :	0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	34.83	86.32	13.59	0.95	1.00	47.88	1.66	79.29	0.77	36.77
◆	35.21	84.69	13.33	0.99	2.00	95.76	2.98	142.49	1.45	69.52
▲	33.57	83.11	13.08	1.03	4.00	191.52	4.55	217.76	2.90	139.04

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MTC, Cannon St Widening

DIRECT SHEAR TEST

(ASTM D-3080)

Project No. : 22-161 | Date : 09/11/23

Figure No.

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: Cannon St Widening
Project Number: 22-161
Boring No.: HA-23-001
Sample No.: B-0   Depth (ft.): 0-5
Location: N/A
Soil Description: Clayey Sand w/gravel

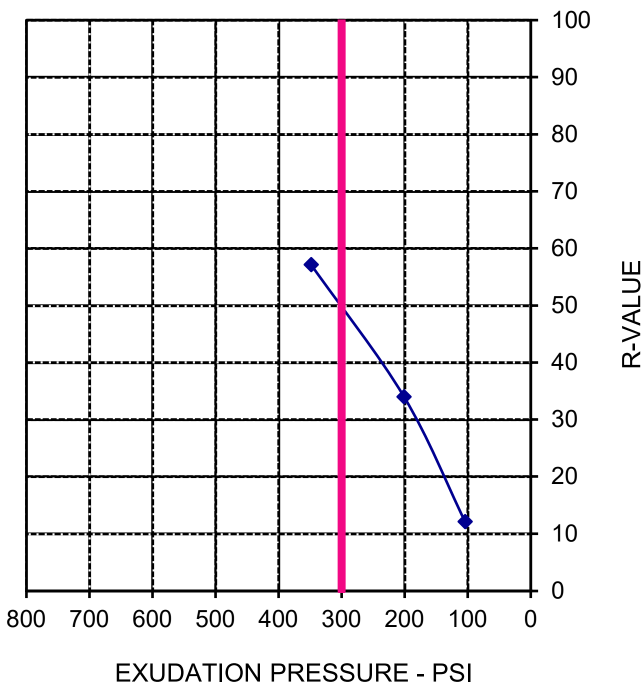
Tested By: ST   Date: 09/06/23
Computed By: KM   Date: 09/07/23
Checked By: AP   Date: 09/08/23

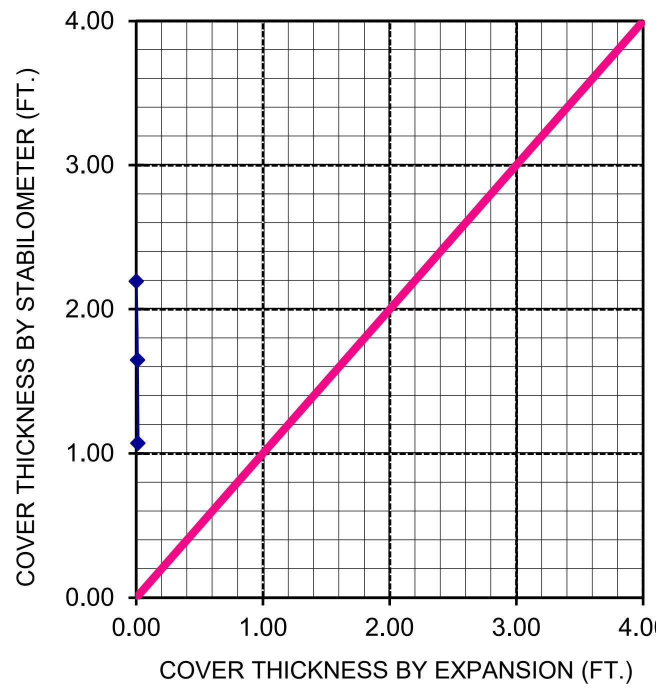
Mold Number	A	B	C	
Water Added, g	18	0	-16	
Compact Moisture(%)	27.8	25.7	23.8	
Compaction Gage Pressure, psi	80	210	280	
Exudation Pressure, psi	104	201	348	
Sample Height, Inches	2.7	2.7	2.7	
Gross Weight Mold, g	3053	3053	3044	
Tare Weight Mold, g	1966	1965	1967	
Net Sample Weight, g	1087	1088	1077	
Expansion, inchesx10^{-4}	0	3	4	
Stability 2,000 (160 psi)	48/126	30/86	18/50	
Turns Displacement	5.28	5.03	5.00	
R-Value Uncorrected	11	30	52	
R-Value Corrected	12	34	57	
Dry Density, pcf	95.5	97.1	97.6	
Traffic Index	10.0	10.0	10.0	
G.E. by Stability	2.19	1.65	1.07	
G.E. by Expansion	0.00	0.01	0.01	

R-VALUE		
	By Exudation:	50
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	50

Remarks

Gf = 1.28, and 10.1 % Retained on the ¾"
*Not Applicable





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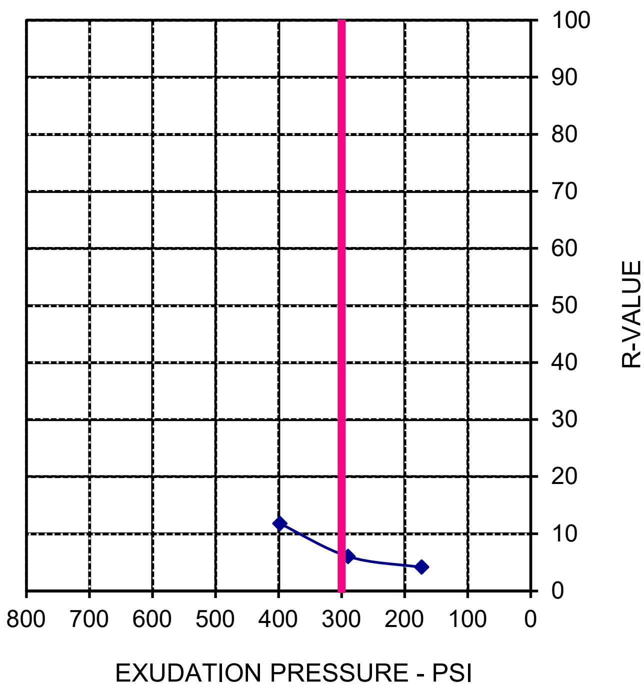
R-VALUE TEST DATA

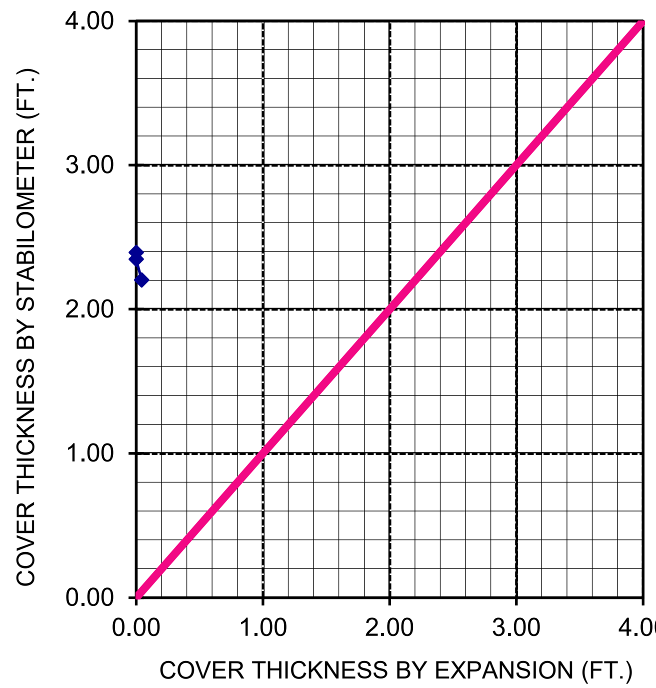
ASTM D2844

Project Name: Cannon St Widening | Tested By: ST | Date: 09/05/23
Project Number: 22-161 | Computed By: KM | Date: 09/06/23
Boring No.: HA-23-004 | Checked By: AP | Date: 09/08/23
Sample No.: B-0 | Depth (ft.): 0-5
Location: N/A
Soil Description: Lean Clay

Mold Number	D	E	F	
Water Added, g	10	21	35	
Compact Moisture(%)	17.6	18.8	20.2	
Compaction Gage Pressure, psi	150	100	50	
Exudation Pressure, psi	398	290	173	
Sample Height, Inches	2.5	2.5	2.5	
Gross Weight Mold, g	2990	3002	2893	
Tare Weight Mold, g	1954	1964	1868	
Net Sample Weight, g	1036	1038	1026	
Expansion, inchesx10^{-4}	13	0	0	
Stability 2,000 (160 psi)	51/131	60/144	67/148	
Turns Displacement	4.14	4.33	4.70	
R-Value Uncorrected	12	6	4	
R-Value Corrected	12	6	4	
Dry Density, pcf	106.8	105.9	103.4	
Traffic Index	10.0	10.0	10.0	
G.E. by Stability	2.20	2.35	2.39	
G.E. by Expansion	0.04	0.00	0.00	

R-VALUE	By Exudation:	7
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	7
Remarks	Gf = 1.28, and 5.3 % Retained on the ¾" *Not Applicable	





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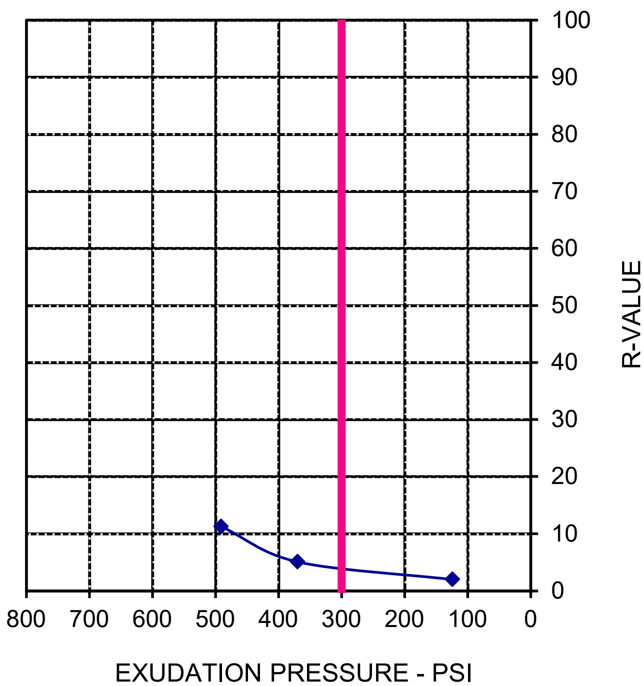
R-VALUE TEST DATA

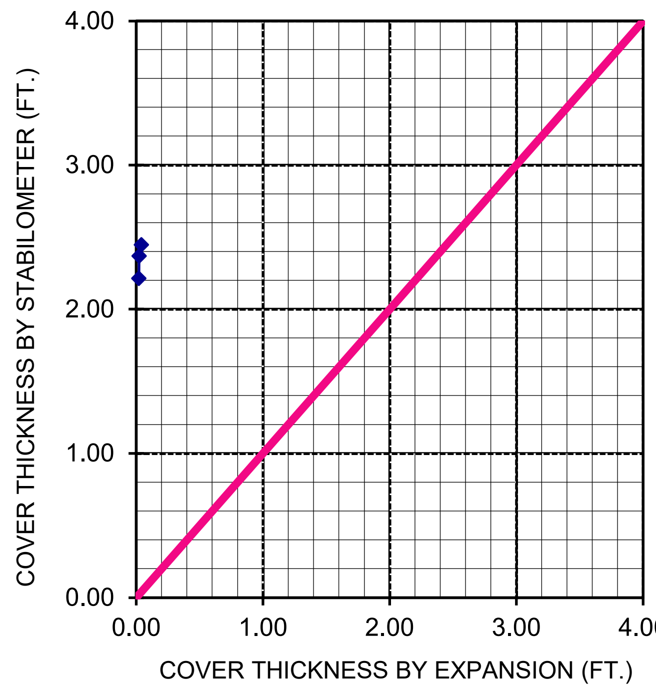
ASTM D2844

Project Name:	Cannon St Widening	Tested By:	ST	Date:	09/05/23
Project Number:	22-161	Computed By:	KM	Date:	09/06/23
Boring No.:	HA-23-005	Checked By:	AP	Date:	09/08/23
Sample No.:	B-0 Depth (ft.): 0-5				
Location:	N/A				
Soil Description:	Lean Clay				

Mold Number	G	H	I	
Water Added, g	40	58	68	
Compact Moisture(%)	19.3	21.1	22.2	
Compaction Gage Pressure, psi	100	75	50	
Exudation Pressure, psi	491	370	125	
Sample Height, Inches	2.5	2.5	2.6	
Gross Weight Mold, g	2868	2868	2860	
Tare Weight Mold, g	1826	1836	1818	
Net Sample Weight, g	1042	1032	1042	
Expansion, inchesx10^{-4}	6	7	12	
Stability 2,000 (160 psi)	57/132	66/146	66/154	
Turns Displacement	4.17	4.47	5.20	
R-Value Uncorrected	11	5	2	
R-Value Corrected	11	5	2	
Dry Density, pcf	105.8	103.2	99.3	
Traffic Index	10.0	10.0	10.0	
G.E. by Stability	2.21	2.37	2.45	
G.E. by Expansion	0.02	0.02	0.04	

R-VALUE	By Exudation:	4
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	4
Remarks	Gf = 1.28, and 0.5 % Retained on the ¾" *Not Applicable	





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R-VALUE TEST DATA

ASTM D2844

Project Name:	Cannon St Widening	Tested By:	ST	Date:	09/05/23
Project Number:	22-161	Computed By:	KM	Date:	09/06/23
Boring No.:	HA-23-006	Checked By:	AP	Date:	09/08/23
Sample No.:	B-0	Depth (ft.):	0-5		
Location:	N/A				
Soil Description:	Lean Clay				

Mold Number	C	A	B	
Water Added, g	12	21	43	
Compact Moisture(%)	15.9	16.9	19.2	
Compaction Gage Pressure, psi	250	200	100	
Exudation Pressure, psi	533	365	269	
Sample Height, Inches	2.5	2.5	2.5	
Gross Weight Mold, g	3004	3011	2989	
Tare Weight Mold, g	1967	1966	1965	
Net Sample Weight, g	1037	1045	1024	
Expansion, inchesx10^{-4}	45	14	3	
Stability 2,000 (160 psi)	36/93	50/126	62/142	
Turns Displacement	3.93	4.94	5.03	
R-Value Uncorrected	31	12	6	
R-Value Corrected	31	12	6	
Dry Density, pcf	108.4	108.4	104.1	
Traffic Index	10.0	10.0	10.0	
G.E. by Stability	1.71	2.20	2.35	
G.E. by Expansion	0.15	0.05	0.01	

R-VALUE		
	By Exudation:	8
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	8

Remarks

Gf = 1.28, and 0.4 % Retained on the ¾"
*Not Applicable