INFORMATION HANDOUT

For Contract No. 12-0M4904
At 12-Ora-5-1.2/2.2

Identified by
Project ID 1212000090

MATERIALS INFORMATION

2nd Revised Foundation Report for El Camino Real UC Widening (Br. No. 55-0203) dated February 4, 2016

3rd Revised Foundation Report for El Camino Real UC Retaining Wall 90 dated February 4, 2016

Revised Foundation Report for El Camino Real UC Curtain Wall 100 dated January 22, 2016

2nd Revised Foundation Report for El Camino Real UC Retaining Wall 103 dated February 4, 2016

Overhead Signs Foundation Report

Aerially Deposited Lead Memorandum

PERMITS

California Coastal Commission Permit Waiver

State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

Memorandum

Serious drought.
Help save water!

To: Mr. MATT HOLM, Branch Chief
Senior Transportation Engineer
Structure Design

Date: February 4, 2016

File: 12-ORA-5, PM 1.2
EA: 12-0M4901
Project ID: 1212000090
El Camino Real UC
Widening (Bridge No. 55-0203)

Attn: Mr. Jinrong Wang

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1
Branch C

Subject: 2nd Revised Foundation Report For El Camino Real UC Widening

Per your e-mail request dated February 10, 2015, a 2nd Revised Foundation Report (FR) has been prepared for the proposed El Camino Real UC Bridge (Bridge no. 55-0203) Widening located on Interstate 5 Freeway (I-5) in the City of San Clemente (see the Site Vicinity Map Figure 1). This Revised Report replaces the previous Revised Foundation Report dated January 22, 2016. The data in this FR include recommendations for foundation design of the bridge widening located at Postmile 1.2 (Approximately 96+49 to 99+43 per 'Alt2' Mainline Centerline stationing of the I-5). The recommendations provided below are based on a subsurface investigation for the proposed El Camino Real UC Bridge Widening per the latest general plans (Dated August 21, 2015). Retaining Walls 90 and 103 and Curtain Wall 100, as part of the project plans, will be addressed in a separate FR.

The recommendations provided below are based on observations of site conditions, including the adjacent retaining wall structure, review of soil borings drilled for this project, and the layout plans dated August 2015. The revisions in this report include reference to 2010 Standard Plans and Specifications and a modification on the pile cutoff elevation for Abutment 1.

1.1 Scope of Work

The following tasks were prepared for the preparation of this Report:

- Review of archive data and Preliminary Foundation Report
- Review of soil borings and laboratory test results
- Geotechnical Analysis
- Preparation of this FR

2.0 PROJECT DESCRIPTION

2.1 Existing Site Conditions and Proposed Improvement

The existing El Camino Real Bridge on I-5 at Postmile 1.2 is a 286 foot long and 150 feet wide three-span bridge supported on 16-inch Cast-in-Drilled-Hole (CIDH) 45-ton piles. Based on as-built foundation plans, existing embedded pile lengths are approximately 40 feet on the average. According to as-built plans dated 1958, the bridge was built as a 4 span bridge with a width of about 102 feet. Per As-builts dated 1983, the bridge was widened by about 9.25 feet on both sides. Existing Retaining walls adjacent to the bridge structure are founded on Type 1 spread footings.

Per the latest plans provided to our office, the present proposed improvement involves widening the northbound side of the bridge by about 9 to 18 feet. In addition, new Retaining walls and Curtain Walls would be built about 10 feet from the existing edge of travel way (ETW) on the east side of the bridge (addressed in separate reports).

The Project Plan, Figure 2, also shows the location of the walls and bridge widening.

3.0 GEOLOGY

3.1 Site Geology

The project site is located along the inland side of a broad gently rolling portion of a non-marine terrace. The existing freeway is located adjacent to the southern flank of the base of the slope of the Santa Ana Mountains. The material under the site is mapped as Capistrano Formation (Blank and Cleveland 1968). The Capistrano Formation consists of siltstone, mudstone and silty shales with interbedded sandstones. The fill and alluvium encountered consists of soft to medium stiff clays and medium dense to dense sand and silty sand/sandy silt. The majority of the clays encountered outside the footprint of the existing embankment fill were soft, fat clays. Most of the clay under the existing embankment fills was more stiff most likely due to the presence of the overlying fill material having consolidated this material over time.

The closest fault to the site is the Newport-Inglewood (Offshore) fault oriented in a northwest-southeast direction approximately 4.5 miles southwest of the proposed project.

4.0 SUBSURFACE CONDITIONS

Eight total soil borings were drilled for the bridge undercrossing supports and proposed retaining and curtain walls to depths of 36.5 to 75.2 feet between June 30 and July 9 2015. Borings R-15-001 to 007 were drilled using the Rotary Wash method of drilling except A-15-004A, which was drilled using a 6-inch Hollow Stem Auger. This boring was drilled in order to obtain representative groundwater elevation conditions for the project site. The Boring locations are shown on Figure 3 of this report. Stationing, offsets and elevations of the soil borings are approximate and are summarized in the Table below. This information, to be updated, will also be provided on Log of Test Boring Sheets to be provided at a later date. Subsurface conditions are summarized below for each proposed structure. These summaries are based on the geotechnical investigation and laboratory results for this project.

Table 1 – Soil Borings Summary

Borings	Date Drilled	Station	Offset, ft	Surface Elevation, ft	Location
R-15-001	7/1/15	91+50	103R	216.1	N/B Offramp
R-15-002	7/7/15	94+75	50.4R	241.5	N/B Main right shoulder
R-15-003	7/1/15	94+88	126R	214.1	N/B Offramp
R-15-004	7/8/15	97+30	90R	216.8	N/B Onramp
A-15-004A	6/30/15	97+40	99R	217.2	N/B Onramp
R-15-005	7/8/15	98+30	113.4R	222.4	N/B Onramp
R-15-006	6/30/15	99+60	115.5R	234	N/B Onramp
R-15-007	7/7/15	104+00	55R	258.7	N/B Mainline right shoulder

Notes: (1) Approximate Stationing per layout plans dated March 2013. Per ‘A Line’ alignment.

<u>El Camino Real UC</u>

Based on soil borings drilled within the proposed UC widening, the native soils near Abutment 1 are composed of stiff sandy clays and medium dense clayey sands between elevations 217 and 177 feet Mean Sea Level (MSL). These soils are underlain by the Capistrano Bedrock formation. Based on a projection from R-15-003, soils near Pier 2 consisted of soft clay between elevation 214 and 202 feet MSL which is underlain by weathered bedrock. The soils near Pier 3 are composed of stiff sandy clays and medium dense to dense sands and clayey sands with increasing bedrock and cobble fragments interspersed to an elevation of about 180 feet MSL. While soils near Abutment 4 were found to be stiff clays to about an elevation of 205 feet MSL underlain by about 13 feet of dense to very dense sands further underlain by bedrock materials.

During the geotechnical investigation for this project, groundwater was not encountered to the full depth explored (+156.7 feet MSL). Based on historical data from 1955 groundwater was encountered at an elevation of 184 feet MSL (California Department of Transportation, 1958). A groundwater of +170 feet MSL was assumed for design.

5.0 SEISMICITY

The site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey. Based on Caltrans ARS Online Version 2.3.06, the Newport-Inglewood (Offshore) fault is the nearest active seismic source from the proposed project site.

Table No. 2 summarizes the faults identified by Caltrans ARS Online Version 2.3.06.

Table 2 – Fault and Design Ground Motion Parameters.

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (mi)	R_{JB} (mi)	R_X (mi)
Newport Inglewood (Offshore)	381	6.9	SS	90	V	4.5	4.5	4.5
San Joaquin Hills	376	7.0	REV	23	W	12.1	11.5	6.1
Elsinore (Temecula)	378	7.7	SS	90	V	21.2	21.2	21.2

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{UP} = Closest distance to the fault rupture plane

The design ARS curve is controlled by the probabilistic acceleration response spectrum curve. The probabilistic ARS curve is developed with a ground motion return period of 975 year which is corresponding with 5% probability to be exceeded in 50 years and the Next Generation Attenuation (NGA) is used for the deterministic ARS curve. The PGA was determined to be 0.4g with an average shear wave velocity (Vs) of 1300 fps in the top 100 feet of subsurface material. The design ARS Curve is provided on Figure 4.

6.0 LIQUEFACTION EVALUATION

Based on the absence of groundwater in the borings drilled for this project well within formational material, liquefaction potential is considered negligible for this site. Furthermore, according to the California Geological Society (CGS) the site is outside of potentially liquefiable zones (Caltrans 2012).

7.0 LABORATORY TESTING

Laboratory testing was performed on selected soil samples from the investigation program by the Sacramento Geotechnical Laboratory. The materials tested are representative of the native soils and bedrock materials. Laboratory testing included Grain size analysis, Atterberg limits, direct shear strength, unconfined compression, and corrosivity tests. Geotechnical testing was performed in accordance with California Test Methods and/or ASTM procedures as indicted by Table 3. A summary of the laboratory results is included in Appendix A.

Table No. 3 – Laboratory Test Methods

Test	Standard
Gradation Analysis	ASTM D 422
Atterberg Limits	AASHTO T 90 & 89
Direct shear	ASTM D 3080
Unconfined Compression Strength	ASTM D 2166
Corrosion	CTM 643, CTM 422, CTM 417

8.0 CORROSIVITY

Composite bulk samples from Boring A-15-004A, were tested for corrosion potential. The results show that the fill soils in these areas are corrosive to buried concrete and metal. It is recommended that personnel from the Corrosion Technology Branch be consulted for corrosion protection recommendations. The results of the Corrosion Test Results are summarized in Table No. 4 below.

Table No. 4 – Corrosion Test Results

Boring	Depth (ft)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
A-15-004A	0-20	1294	9.12	N/A	N/A
A-15-004A	20-40	418	7.98	770	200
A-15-004A	40-60	363	7.07	720	1600

Note: Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

9.0 SOIL PARAMTERS

Embankment fill, native soil and bedrock parameters summarized in the table below were derived from in-situ field tests of soil and rock samples derived during the field exploration, correlations, and laboratory test results. Soil profiles and parameters were chosen based on worst case conservative conditions for design purposes only. The LOTB's, as part of the construction plans,

should be referred to for pile construction issues. A summary of the soil and rock parameters used for analysis of the piles and spread footings of the foundations is provided in the table below.

Table No. 5 – Summary of Soil Parameters

Location	Depth, ft	USCS	Unit Weight, pcf	ϕ, degrees	Su, psf
Abutment 1	0-8	SM (Embankment Fill)	120	32	
	8-12	CL (Embankment Fill)	120		450
	12-22	SC (Embankment Fill)	120	29	
	22-38	CL	130		1000
	38-62	SC	120	35	
	62-76	Formation	122	38	
Pier 2	0-12	CL	120		250
	12-16	SP	120	34	
	16-24	Claystone (Formational)	122	30	
	24-46	Sandstone (Formational)	122	34	
	46-52	Sandstone (Formational)	122	38	
Pier 3	0-20	CL	130		1000
	20-28	SP	120	37	
	28-38	CL/ML	120	38	
	38-60	CL/ML (Formational)	122	38	
Abutment 4	0-15	CL	130		1000
	15-28	SP	120	37	
	28-38	CL/ML	120	38	
	38-60	CL/ML (Formational)	122	38	

Notes: (1) for stations 98+37 to 99+00.
(2) for stations 99+00 to 103+13

10.0 FOUNDATION RECOMMENDATIONS

Foundation recommendations were based on subsurface soil conditions derived from the results of the geotechnical investigation for this project and subsequent soil and rock parameters derived from the samples. In brief, 24-inch diameter Cast-in-Drilled Hole (CIDH) piles are a feasible foundation type for the El Camino Real undercrossing abutment and pier locations. The CIDH pile design is based on the Load Resistance Factor (LRFD) Method. As such, guidelines from Section 3 of the 2007 AASHTO LRFD Bridge Design Specifications are followed in this Report. The design was further based on available topography, cross sections developed from soil borings drilled for this project. The design was also based on soil parameters summarized in Section 9 and Table 5 of this Report.

10.1 El Camino Real Undercrossing

24-inch diameter CIDH piles are proposed for Abutments 1 and 4 and Piers 2 and 3 of the EL Camino Bridge UC widening. In addition, Per Structure Design request, 24-inch diameter CIDH piles are also recommended for Beams A and B between Abutment 1 and Pier 2 and between Pier 3 and Abutment 4. Per the Load Resistance Factor Design (LRFD), Design Loads sent from Structure Design on July 31, 2015 have been summarized in Table 6. Foundation recommendations for the Abutments and Bents with pile cutoff elevations and Specified Tip Elevations (STE) are provided in Table 7. The Pile Data Table is summarized in Table 8.

Table No. 6 – Summary of Design Loads

Support No.	Service I Limit State (kips)			Strength Limit State (kips)				Extreme Event Limit State (kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max Per Pile	Per Support	Per Support	Max per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Abut 1	520	173	400	730	280			1000	400		
Beam A – Pile 1	120		70	240	240			343	343		
Beam A – Pile 2	120		70	240	240			343	343		
Beam B – Pile 1	120		70	240	240			343	343		
Beam B Pile 2	120		70	240	240			343	343		
Curtain Wall											
Pile 1	110		60	280	280			400	400		
Pile 2	110		70	280	280			400	400		
Pile 3	110		80	280	280			400	400		
Pier 2	1200	200	800	1680	280	336	112	2400	400	600	200
Pier 3	1200	200	800	1680	280	336	112	2400	400	600	200
Beam A – Pile 1	120		70	240	240			343	343		
Beam A – Pile 2	120		70	240	240			343	343		
Beam A – Pile 3	120		70	240	240			343	343		
Beam A Pile 4	120		70	240	240			343	343		
Curtain Wall											
Pile 1	120		70	240	240			343	343		
Pile 2	120		70	240	240			344	343		
Pile 3	120		70	240	240			343	343		
Pile 4	120		70	240	240			343	343		
Abut 4	400	133	230	680	280			1000	400		

Table No. 7 - Pile Foundation Recommendations for Abutments and Bents

Support	Pile (1)	Cut-off Elevation (ft)	LRFD Service-I Limit State Load (kips) per Support	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevation (ft) (2)	Specified Tip Elevation (ft)
					Strength Limit		Extreme Event			
					Comp (ϕ=0.7)	Tension (ϕ=0.7)	Comp (ϕ=1)	Tension (ϕ=1)		
Abut 1	24" CIDH	222.25	520	1	280		400		172 (a-I) 172 (a-II) 182 (c) 187 (d)	172
Beam A – Pile 1	24" CIDH Ext.	225.5	120	1	240		343		175.5 (a-I) 175.5 (a-II)	175.5
Beam A – Pile 2	24" CIDH Ext.	220.5	120	1	240		343		170.5 (a-I) 170.5 (a-II)	170.5
Beam B – Pile 1	24" CIDH Ext.	225.5	120	1	240		343		175.5 (a-I) 175.5 (a-II)	175.5
Beam B – Pile 2	24" CIDH Ext.	220.5	120	1	240		343		170.5 (a-I) 170.5 (a-II)	170.5
Pier 2	24" CIDH	211.5	1200	1	280	112	400	200	151.5 (a-I) 181.5 (b-I) 151.5 (a-II) 171.5 (b-II) 180.5 (c) 181.5 (d)	151.5
Pier 3	24" CIDH	211.5	1200	1	280	112	400	200	161.5 (a-I) 181.5 (b-I) 171.5 (a-II) 181.5 (b-II) 181.5 (c) 181.5 (d)	161.5
Beam A – Pile 1	24" CIDH Ext.	216.5	120	1	240		343		176.5 (a-I) 176.5 (a-II)	176.5
Beam A – Pile 2	24" CIDH Ext.	220.5	120	1	240		343		180.5 (a-I) 180.5 (a-II)	180.5
Beam A – Pile 3	24" CIDH Ext.	223.5	120	1	240		343		183 (a-I) 183 (a-II)	183
Beam A – Pile 4	24" CIDH Ext.	226.5	120	1	240		343		186.5 (a-I) 186.5 (a-II)	186.5
Abut 4	24" CIDH	223.75	400	1	280		400		183.75 (a-I) 183.75 (a-II) 185.75 (c) 191.75 (d)	183.75

Notes: (1) Piles types for Beam are 24-inch CIDH Extensions per SD plans and Pile data table (July 2015).
(2) (a-I) Compression Strength Limit, (a-II) Compression Extreme Event, (b-I) Tension Strength Limit, (b-II) Tension Extreme Event, (c) Settlement, (d) Lateral Load

Table No. 8 - Pile Data Table

Support	Pile Type (1)	Required Factored Nominal Resistance (kips)				Design Tip Elevation (ft) (2)	Specified Tip Elevation (ft)
		Strength Limit		Extreme Event			
		Comp (ϕ=0.7)	Tension (ϕ=0.7)	Comp (ϕ=1)	Tension (ϕ=1)		
Abut 1	24" CIDH	280		400		172 (a-I) 172 (a-II) 182 (c) 187 (d)	172
Beam A – Pile 1	24" CIDH Ext.	240		343		175.5 (a-I) 175.5 (a-II)	175.5
Beam A – Pile 2	24" CIDH Ext.	240		343		170.5 (a-I) 170.5 (a-II)	170.5
Beam B – Pile 1	24" CIDH Ext.	240		343		175.5 (a-I) 175.5 (a-II)	175.5
Beam B – Pile 2	24" CIDH Ext.	240		343		170.5 (a-I) 170.5 (a-II)	170.5
Pier 2	24" CIDH	280	112	400	200	151.5 (a-I) 181.5 (b-I) 151.5 (a-II) 171.5 (b-II) 180.5 (c) 181.5 (d)	151.5
Pier 3	24" CIDH	280	112	400	200	161.5 (a-I) 181.5 (b-I) 171.5 (a-II) 181.5 (b-II) 181.5 (c) 181.5 (d)	161.5
Beam A – Pile 1	24" CIDH Ext.	240		343		176.5 (a-I) 176.5 (a-II)	176.5
Beam A – Pile 2	24" CIDH Ext.	240		343		180.5 (a-I) 180.5 (a-II)	180.5
Beam A – Pile 3	24" CIDH Ext.	240		343		183 (a-I) 183 (a-II)	183
Beam A – Pile 4	24" CIDH Ext.	240		343		186.5 (a-I) 186.5 (a-II)	186.5
Abut 4	24" CIDH	280		400		183.75 (a-I) 183.75 (a-II) 185.75 (c) 191.75 (d)	183.75

Notes:
(1) Piles types for Beam are 24-inch CIDH Extensions per SD plans and Pile data table (July 2015).
(2) (a-I) Compression Strength Limit, (a-II) Compression Extreme Event, (b-I) Tension Strength Limit, (b-II) Tension Extreme Event (c) Settlement, (d) Lateral Load

As summarized in Table 7 above, compression loads from the Strength Limit case generally govern pile lengths. Specified pile tip elevations are based on skin frictional capacity of axial compressive loads only. End bearing is not considered. The capacities are based on the general FHWA Method for Axial pile capacities of CIDH piles in combination with the LRFD Methodology. Pile lengths based on settlement analysis were based on a settlement amount of 1-inch. The method used to calculate the pile settlement was based on the Semi-Empirical Method (Vesic, 1977). Based on calculations of the Abutments and Piers and using the same subsurface profiles with less pile loading, compression is assumed to control for the Beam extension piles and thus pile lengths for settlement and lateral load were not calculated.

<u>For Curtain Walls, 24-inch CIDH pile lengths of 60 feet should be used between Abutment 1 and Pier 2. 24-inch diameter CIDH 50 foot length piles should be used between Pier 3 and Abutment 4. These lengths should be subtracted by the pile cutoff elevations to obtain specified tip elevations.</u>

Lateral loads on abutment and pier piles were analyzed using the computer program LPILE, Version 6 by Ensoft. Lateral loading was based on a deflection of ¼-inch with fixed head condition assumed at the abutments and pinned-head conditions assumed for the piers. The maximum lateral load allowed with the ¼-inch deflection along with maximum bending moment is summarized in Table 9 below. The lateral load analysis was based on subsurface conditions developed for this project and the associated input parameters summarized in Table 5 of this report.

Table No. 9 – Summary of Maximum Shear Force and Maximum Bending Moment

Location	Depth of Pile, ft (1)	Deflection at Pile Head, in	Maximum Shear Force, kips	Maximum Bending Moment, in-kips	Depth of Maximum Bending Moment below top of pile, ft
Abutment 1	50	0.25	95	3400	19
Pier 2	50	0.25	31	3500	13
Pier 3	50	0.25	44	3625	13
Abutment 4	50	0.25	102	4000	19

Note: (1) Depth below cutoff elevation.

11.0 CONSTRUCTION ISSUES

- Sandstone and Claystone bedrock of the Capistrano formation will be encountered during drilling for drilled holes at the El Camino Real UC Widening Location. Hard to very hard cobbles should also be expected to be encountered in soils above the bedrock formation and within the formation itself.

- Although groundwater was not encountered during our investigation, historical data show the groundwater to be about +184 feet MSL. We therefore recommend the contractor be prepared to encounter groundwater and thus use the wet method during drilling.

- Caving is anticipated within the drilled holes for the CIDH piles, specifically within the native soil zones (approximately +215-180 feet elevation). Therefore, the contractor should devise a method to keep the drilled holes open. Any steel casing to be used should be considered temporary.

- It is highly recommended that draft construction plans and specifications be submitted to this office for review before finalization.

12.0 REFERENCES

- California Department of Transportation, "El Camino Real UC" Log of Test Borings, As-Built Plans, September 17, 1958 (signed).

- Blanc, Robert P., 1930; Cleveland, George B. "Natural slope stability as related to geology, San Clemente area, Orange and San Diego Counties, California", California. Division of Mines and Geology. -- San Francisco: California Geological Survey, 1968

- California Geological Survey. California Geological Survey, "Seismic Hazard Map, San Clemente 7.5-minute quadrangle", Orange County, California, 2002.

- Vesic, A. S. 1977. "ASCE-Design of Pile Foundations No. 1: Design of Pile Foundations, "National Cooperative Highway Research Program, Synthesis of Highway Practice No. 42", Transportation Research Board, Washington, DC.

If you have any questions, please contact Sam Sukiasian at (213) 620-2135 or Christopher Harris at (213) 620-2147.

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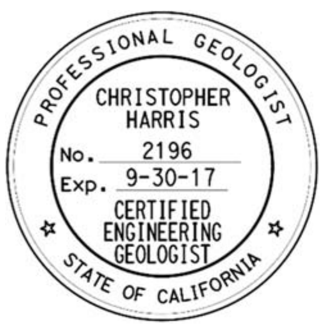


cc. Ahmed Abou-Abdou – District 12 Project Manager

Attachments:

Figures 1-4
Appendix A – Laboratory Results

Figures

Vicinity Map

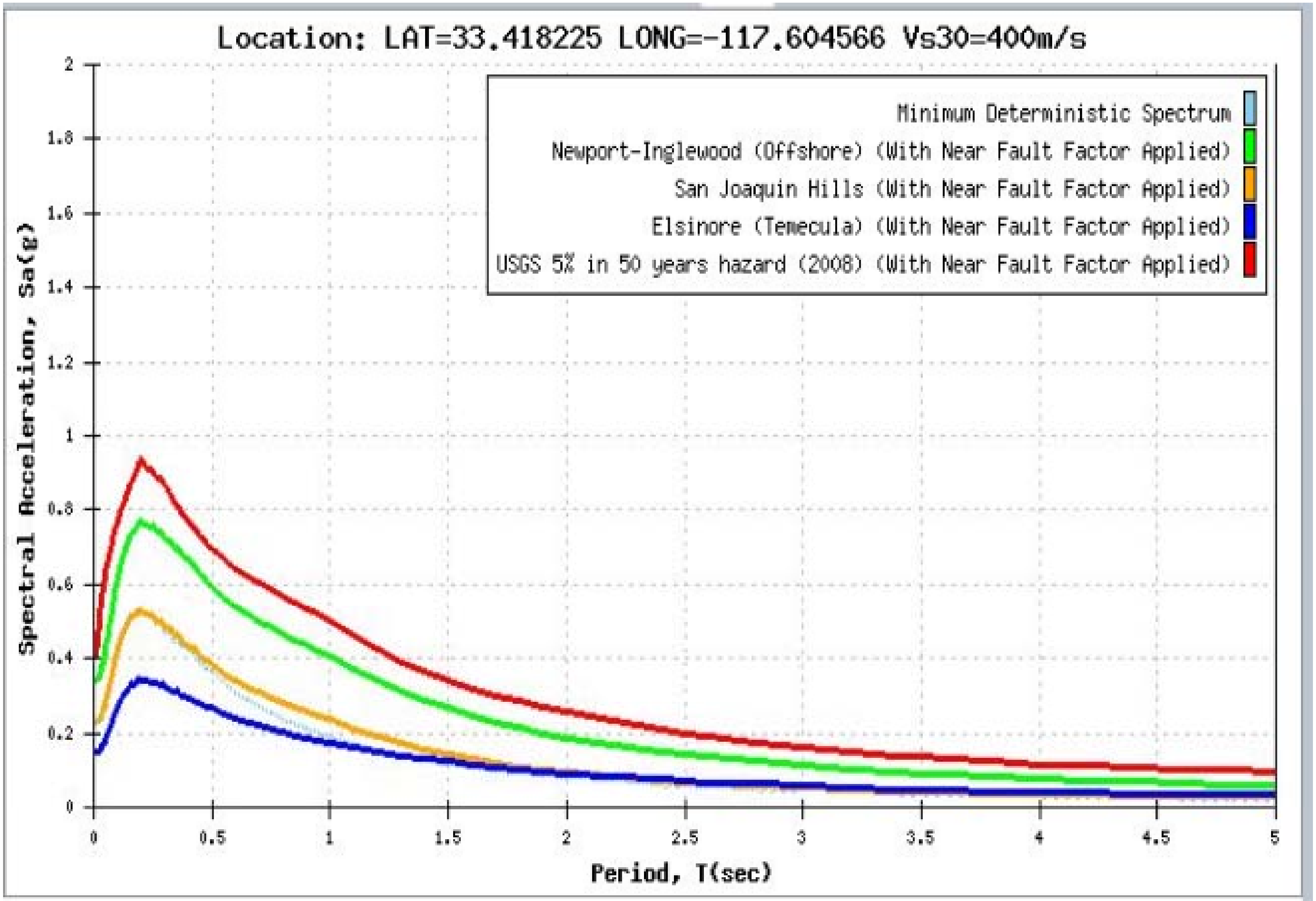
Figure 1

El Camino Real UC - Project Plan

Figure 2

El Camino Real UC - Boring Location Plan

Figure 3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Minimum Deterministic Spectrum
Newport-Inglewood (Offshore) (With Near Fault Factor Applied)
San Joaquin Hills (With Near Fault Factor Applied)
Elsinore (Temecula) (With Near Fault Factor Applied)
USGS 5% in 50 years hazard (2008) (With Near Fault Factor Applied)
Spectral Acceleration, Sa(g)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period, T(sec)

Figure 4

Appendix A – Laboratory Results

Direct Shear Tests (ASTM D3080)

Boring/ Sample No.	Depth, ft	Soil Description	Initial Dry Density, pcf (1)	Initial Water Content, % (1)	Normal Stress Range, psf	Ultimate Shear Strength Values	
						Friction Angle	Undrained Shear, psf
R-15-002/4	20	Clayey Sand (fill)	112	18.2	1000-4000	28	776
R-15-002/6	30	Sandy Clay (Native)	109	16.8	2000-6000	36	497
R-15-004/3	15	Clay (Native)	114	16.7	1000-4000	22	1320
R-15-004/5	25	Sand Clay	101	23.2	2000-6000	36	689

Note: (1) Average of three samples.

Unconfined Compression Test Results

(ASTM D2166)

Boring/ Sample No.	Depth, ft	Soil Description	Initial Dry Density,	Initial Water Content, %	Unconfined Compressive Strength, psi	Shear Strength, psf
R-15-003/6	30	Formational	114	16.5	48.7	3500
R-15-005/9	45	Formational	101	22.5	44.2	3180

Notes: Based on Strain Rate of 1%/min

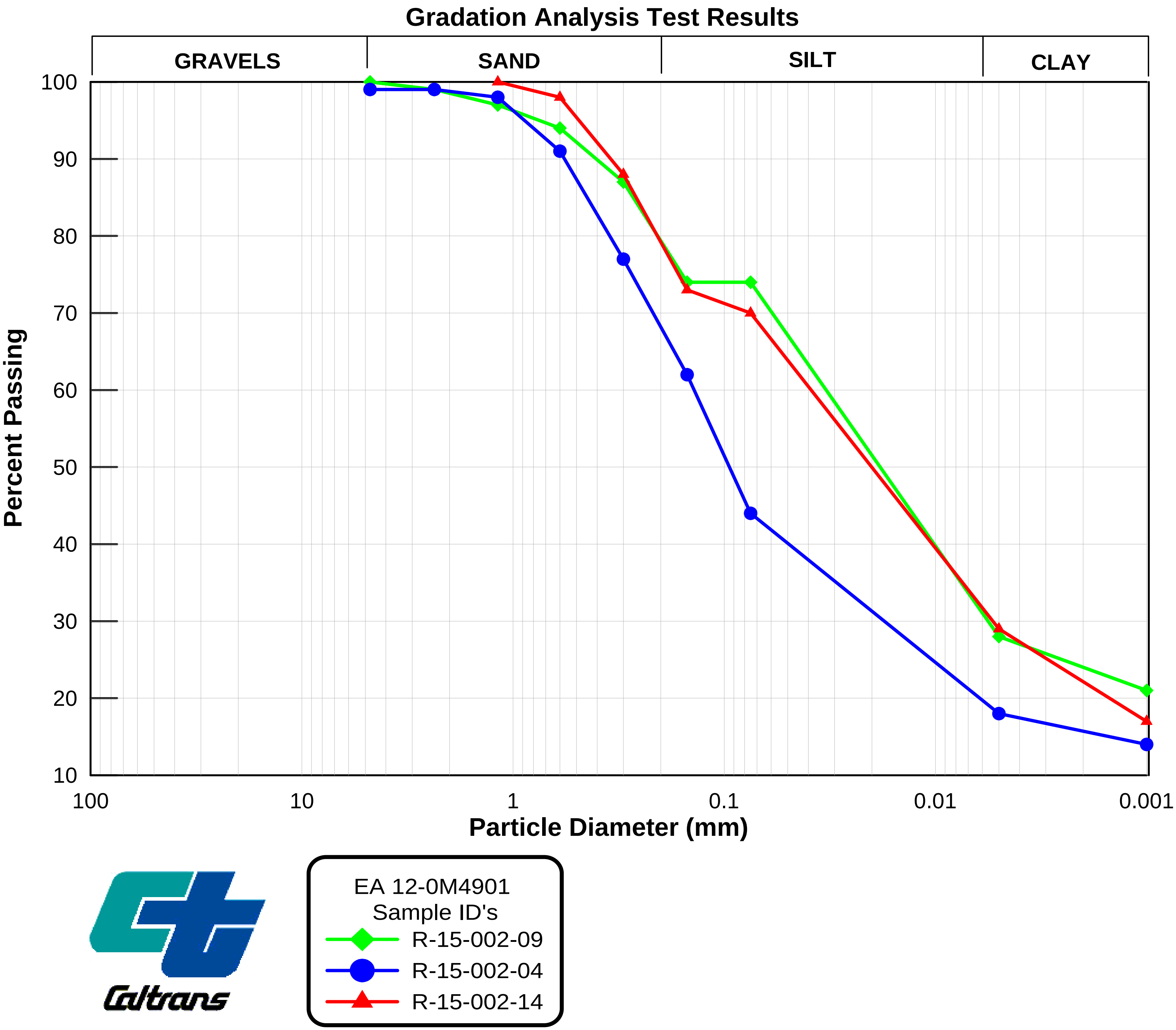
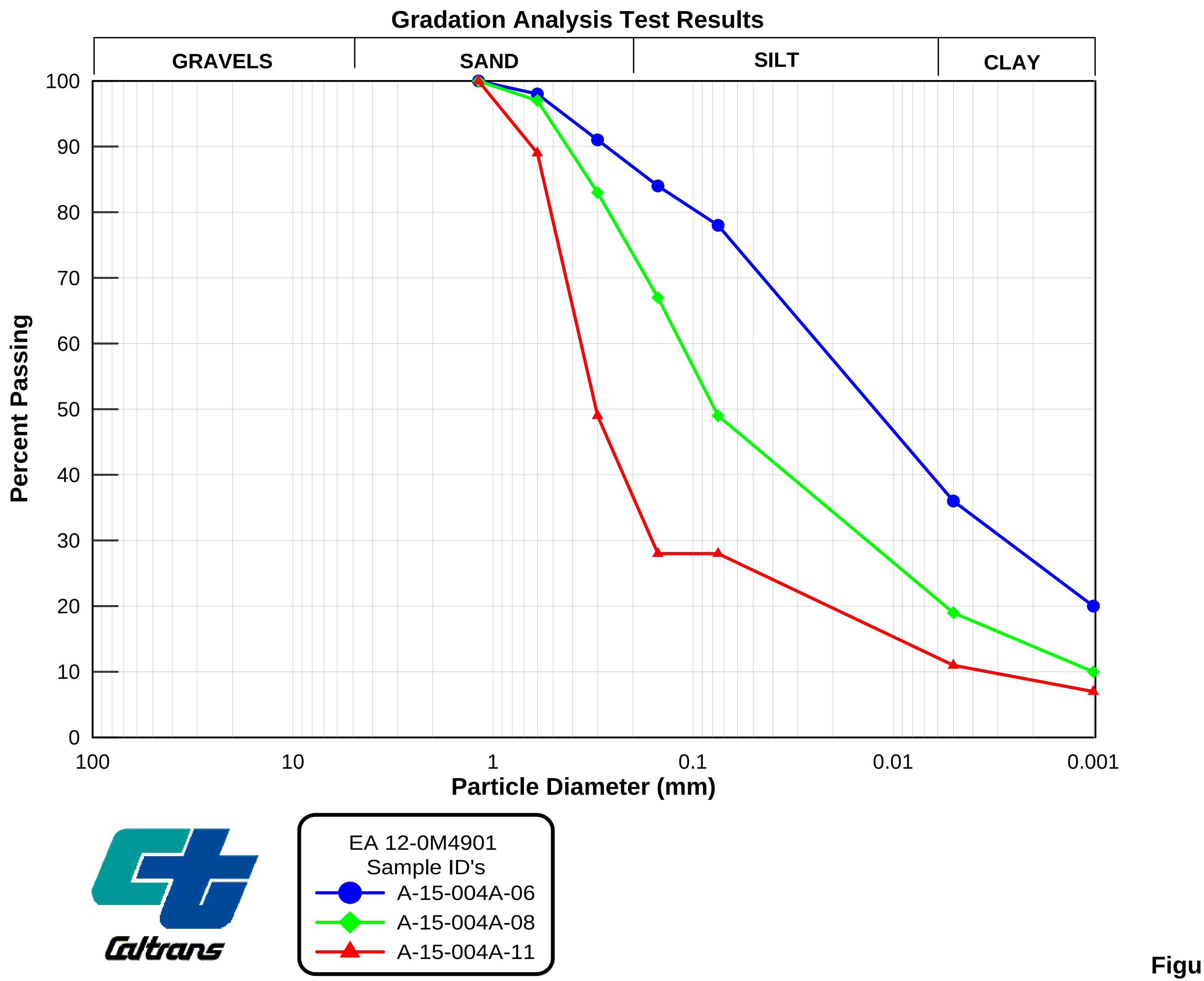
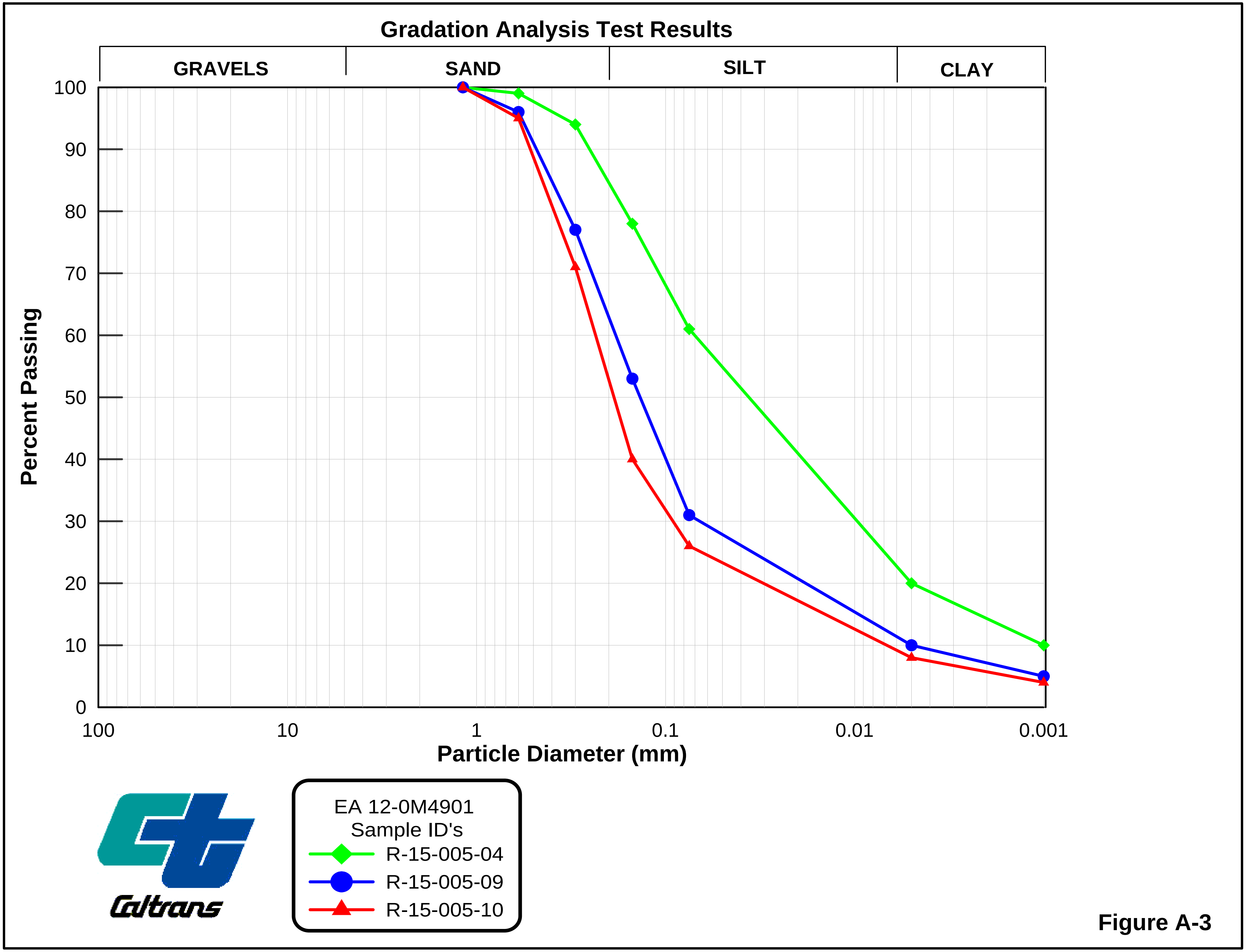

Gradation Analysis Test Results
GRAVELS
SAND
SILT
CLAY
Percent Passing
100
90
80
70
60
50
40
30
20
10
100
10
1
0.1
0.01
0.001
Particle Diameter (mm)
EA 12-0M4901
Sample ID's
R-15-002-09
R-15-002-04
R-15-002-14

Figure A-1



Gradation Analysis Test Results
GRAVELS
SAND
SILT
CLAY
Percent Passing
100
90
80
70
60
50
40
30
20
10
0
100
10
1
0.1
0.01
0.001
Particle Diameter (mm)
Caltrans
EA 12-0M4901
Sample ID's
A-15-004A-06
A-15-004A-08
A-15-004A-11

Figure A-2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GRAVELS
SAND
SILT
CLAY
Percent Passing
100
90
80
70
60
50
40
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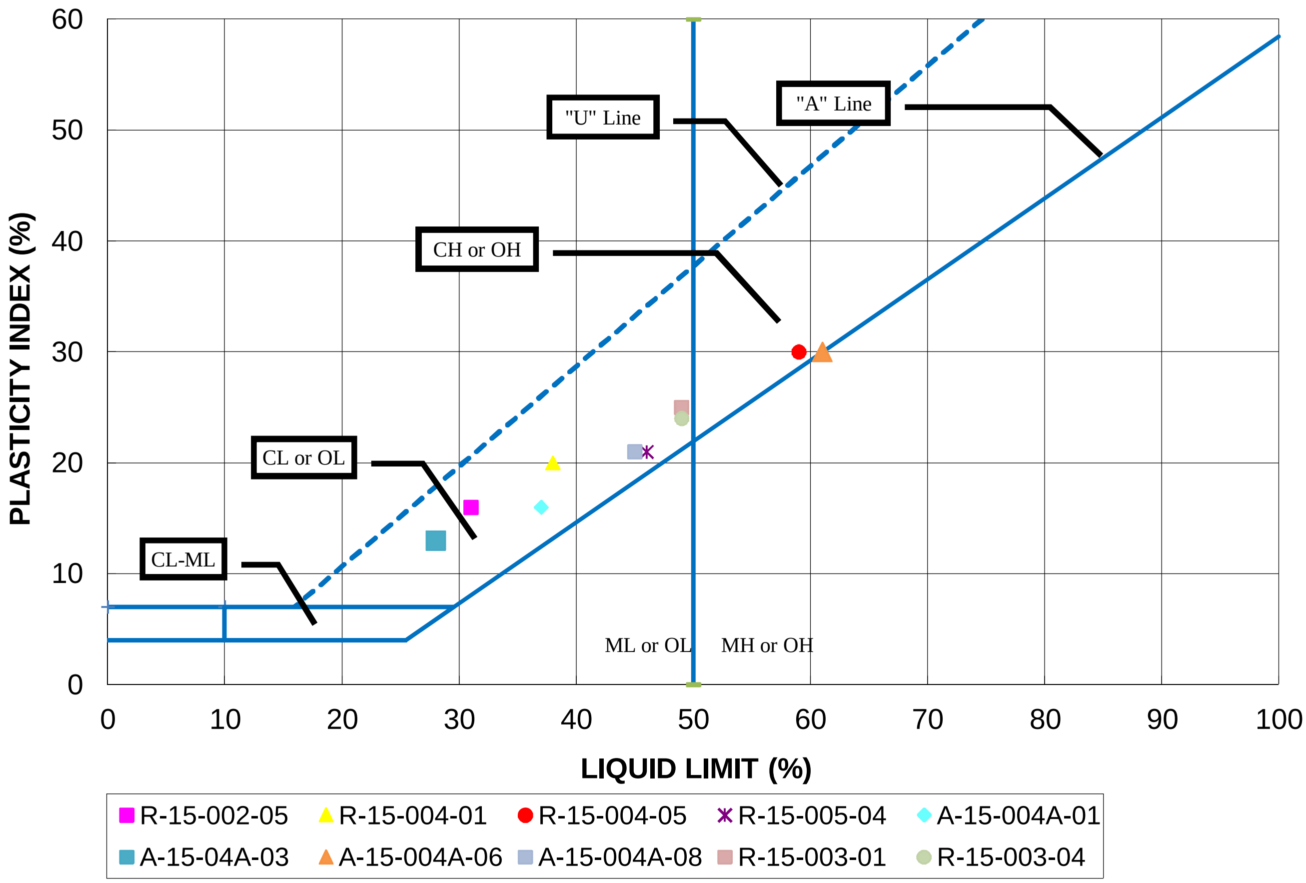
20
10
0
100
10
1
0.1
0.01
0.001
Particle Diameter (mm)
EA 12-0M4901
Sample ID's
R-15-005-04
R-15-005-09
R-15-005-10
Caltrans

Figure A-3

Atterberg Limits Test Results



Boring No.	Sample No.	Depth (ft)	LL	PI	PL	Classification
R-15-002	5	25	31	16	15	CL
R-15-004	1	5'	38	20	18	CL
R-15-004	5	25'	59	30	29	CH
R-15-005	4	10'	46	21	25	CL
A-15-004A	1	5	37	16	21	CL
A-15-004A	3	15	28	13	15	CL
A-15-004A	6	30	61	30	31	CH
A-15-004A	8	40	45	21	24	CL
R-15-003	1	15	49	25	24	CL
R-15-003	4	20	49	24	25	CL

Caltrans

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1
(Location in parenthesis)

Project:	El Camino Real
EA:	12-0M4901
Dist-Co-Rte-PM:	12-ORA-5-1.2

M e m o r a n d u m

To: Mr. MATT HOLM, Branch Chief
Senior Transportation Engineer
Structure Design

Date: February 4, 2016

File: 12-ORA-5, PM 1.2
EA: 12-0M4901
Project ID: 1212000090
Retaining Wall 90

Attn: Mr. Jinrong Wang

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1
Branch C

Subject: 3rd Revised Foundation Report For El Camino Real UC Retaining Wall 90

A 3rd Revised Foundation Report (FR) has been prepared for the proposed Retaining Wall 90 which is part of the El Camino Real UC Bridge (Bridge no. 55-0203) Widening located on Interstate 5 Freeway (I-5) in the City of San Clemente (see the Site Vicinity Map Figure 1). This 3rd Revised Report replaces the previous Revised FR dated January 22, 2016. The data in this 3rd Revised FR includes recommendations for foundation design of Retaining Wall 90 (Approximately 92+08 to 95+30 per the 'A-Line' Alignment). The recommendations provided below are based on a subsurface investigation for the proposed Retaining Wall 90 per the latest general plans (Dated August 21, 2015). El Camino Real UC, Retaining Wall 103 and Curtain Wall 100, as part of the project plans, will be addressed in separate FR's. This 3rd Revised Report presents updated pile design loads provided by Structure Design. The updated pile design loads are summarized in Table 7. Pile Design is based on these updated loads.

The recommendations provided below are based on observations of site conditions, including the adjacent retaining wall structure, review of soil borings drilled for this project, and the layout plans dated August 2015. This Revised Report changes reference back to the 2010 Standard Plans and Specifications, fixes minor edits and makes clarifications in Sections 10.1.2 and 11.0.

1.1 Scope of Work

The following tasks were prepared for the preparation of this Report:

- Review of archive data and Preliminary Foundation Report
- Review of soil borings and laboratory test results
- Geotechnical Analysis
- Preparation of this 2nd Revised FR

2.0 PROJECT DESCRIPTION

2.1 Existing Site Conditions and Proposed Improvement

The existing El Camino Real Bridge on I-5 at Postmile 1.2 is a 286 foot long and 150 feet wide three-span bridge supported on 16-inch Cast-in-Drilled-Hole (CIDH) 45-ton piles. Based on as-built foundation plans, existing embedded pile lengths are approximately 40 feet on the average. According to as-built plans dated 1958, the bridge was built as a 4 span bridge with a width of about 102 feet. Per As-builts dated 1983, the bridge was widened by about 9.25 feet on both sides. Existing Retaining walls adjacent to the bridge structure are founded on Type 1 spread footings. The new Retaining Wall 90 would be built about 10 feet from the existing edge of travel way (ETW) on the east side of the bridge. Retaining Wall 90 details along with the adjacent proposed Curtain Wall 100 and Retaining Wall 103 are summarized in Table No. 1 below:

Table 1 – Proposed Retaining Wall Summary

Retaining Wall (RW) /Curtain Wall (CW)	Approximate Station (A-Line)	Wall Height, ft	Proposed Foundation Type
RW 90	92+08 to 93+60	8-10	Type 1 Spread Footing
RW 90	93+60 to 95+53	12-20	Type 1 Spread Footing or Type 1 on CIDH piles
CW 100	98+37 to 103+13	18-6	On CIDH piles
RW 103	103+13 to 104+09	8-6	Type 1 on Spread Footings

The Project Plan, Figure 2, also shows the location of the walls and bridge widening.

3.0 GEOLOGY

3.1 Site Geology

The project site is located along the inland side of a broad gently rolling portion of a non-marine terrace. The existing freeway is located adjacent to the southern flank of the base of the slope of the Santa Ana Mountains. The material under the site is mapped as Capistrano Formation (Blank and Cleveland 1968). The Capistrano Formation consists of siltstone, mudstone and silty shales with interbedded sandstones. The fill and alluvium encountered consists of soft to medium stiff clays and medium dense to dense sand and silty sand/sandy silt. The majority of the clays encountered outside the footprint of the existing embankment fill were soft, fat clays. Most of the clay under the existing embankment fills was more stiff most likely due to the presence of the overlying fill material having consolidated this material over time.

The closest fault to the site is the Newport-Inglewood (Offshore) fault oriented in a northwest-southeast direction approximately 4.5 miles southwest of the proposed project.

4.0 SUBSURFACE CONDITIONS

Eight total soil borings were drilled for the bridge undercrossing supports and proposed retaining and curtain walls to depths of 36.5 to 75.2 feet between June 30 and July 9 2015. Borings R-15-001 to 007 were drilled using the Rotary Wash method of drilling except A-15-004A, which was drilled using a 6-inch Hollow Stem Auger. This boring was drilled in order to obtain representative groundwater elevation conditions for the project site. The Boring locations are shown on Figure 3 of this report. Stationing, offsets and elevations of the soil borings are approximate and are summarized in the Table below. This information, to be updated, will also be provided on Log of Test Boring Sheets to be provided at a later date. Subsurface conditions are summarized below for each proposed structure. These summaries are based on the geotechnical investigation and laboratory results for this project.

Table 2 – Soil Borings Summary

Borings	Date Drilled	Station	Offset, ft	Surface Elevation, ft	Location
R-15-001	7/1/15	91+50	103R	216.1	N/B Offramp
R-15-002	7/7/15	94+75	50.4R	241.5	N/B Main right shoulder
R-15-003	7/1/15	94+88	126R	214.1	N/B Offramp
R-15-004	7/8/15	97+30	90R	216.8	N/B Onramp
A-15-004A	6/30/15	97+40	99R	217.2	N/B Onramp
R-15-005	7/8/15	98+30	113.4R	222.4	N/B Onramp
R-15-006	6/30/15	99+60	115.5R	234	N/B Onramp
R-15-007	7/7/15	104+00	55R	258.7	N/B Mainline right shoulder

Notes: (1) Approximate Stationing per layout plans dated March 2013. Per ‘A Line’ alignment.

<u>Retaining Wall 90</u>

Soil borings along the proposed wall revealed 20-40 feet of native soils (Elevation 216 to about 176 MSL) composed of 10 feet medium dense sands and clayey sands overlying 10 feet of medium stiff clays near the south end of the proposed wall and transition to 12 feet of soft clays overlying 4 feet of medium dense sands toward the north end (Elevation 214 to 198 feet MSL). These soils were found to be underlain by interbedded Claystone and Sandstone of the Capistrano formation.

5.0 SEISMICITY

The site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey. Based on Caltrans ARS Online Version 2.3.06, the Newport-Inglewood (Offshore) fault is the nearest active seismic source from the proposed project site.

Table No. 3 summarizes the faults identified by Caltrans ARS Online Version 2.3.06.

Table 3 – Fault and Design Ground Motion Parameters.

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (mi)	R_{JB} (mi)	R_X (mi)
Newport Inglewood (Offshore)	381	6.9	SS	90	V	4.5	4.5	4.5
San Joaquin Hills	376	7.0	REV	23	W	12.1	11.5	6.1
Elsinore (Temecula)	378	7.7	SS	90	V	21.2	21.2	21.2

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
$_{RUP}$ = Closest distance to the fault rupture plane

The design ARS curve is controlled by the probabilistic acceleration response spectrum curve. The probabilistic ARS curve is developed with a ground motion return period of 975 year which is corresponding with 5% probability to be exceeded in 50 years and the Next Generation Attenuation (NGA) is used for the deterministic ARS curve. The PGA was determined to be 0.4g with an average shear wave velocity (Vs) of 1300 fps in the top 100 feet of subsurface material. The design ARS Curve is provided on Figure 4.

6.0 LIQUEFACTION EVALUATION

Based on the absence of groundwater in the borings drilled for this project well within formational material, liquefaction potential is considered negligible for this site. Furthermore, according to the California Geological Society (CGS) the site is outside of potentially liquefiable zones (Caltrans 2012).

7.0 LABORATORY TESTING

Laboratory testing was performed on selected soil samples from the investigation program by the Sacramento Geotechnical Laboratory. The materials tested are representative of the native soils and bedrock materials. Laboratory testing included Grain size analysis, Atterberg limits, direct shear strength, unconfined compression, and corrosivity tests. Geotechnical testing was performed in accordance with California Test Methods and/or ASTM procedures as indicted by Table 4. A summary of the laboratory results is included in Appendix A.

Table No. 4 – Laboratory Test Methods

Test	Standard
Gradation Analysis	ASTM D 422
Atterberg Limits	AASHTO T 90 & 89
Direct shear	ASTM D 3080
Unconfined Compression Strength	ASTM D 2166
Corrosion	CTM 643, CTM 422, CTM 417

8.0 CORROSIVITY

Composite bulk samples from Boring A-15-004A, were tested for corrosion potential. The results show that the fill soils in these areas are corrosive to buried concrete and metal. It is recommended that personnel from the Corrosion Technology Branch be consulted for corrosion protection recommendations. The results of the Corrosion Test Results are summarized in Table No. 5 below.

Table No. 5 – Corrosion Test Results

Boring	Depth (ft)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
A-15-004A	0-20	1294	9.12	N/A	N/A
A-15-004A	20-40	418	7.98	770	200
A-15-004A	40-60	363	7.07	720	1600

Note: Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

9.0 SOIL PARAMTERS

Embankment fill, native soil and bedrock parameters summarized in the table below were derived from in-situ field tests of soil and rock samples derived during the field exploration, correlations, and laboratory test results. Soil profiles and parameters were chosen based on worst case conservative conditions for design purposes only. The LOTB's, as part of the construction plans,

should be referred to for pile construction issues. A summary of the soil and rock parameters used for analysis of the piles and spread footings of the foundations is provided in the table below.

Table No. 6 – Summary of Soil Parameters

Location	Depth, ft	USCS	Unit Weight, pcf	ϕ, degrees	Su, psf
RW 90 (1)	0-20	SP/SC	120	34	
	20-37	Clay	122		1500
	37-60	Claystone/Siltstone (Formational)	122	38	
RW 90 (2)	0-12	CL	120		250
	12-16	SP	120	34	
	16-24	Claystone (Formational)	122	30	
	24-46	Sandstone (Formational)	122	34	
	46-52	Sandstone (Formational)	122	38	

Notes: (1) for stations 92+08 to 93+60.
(2) for stations 93+60 to 95+53.

10.0 FOUNDATION RECOMMENDATIONS

Foundation recommendations were based on subsurface soil conditions derived from the results of the geotechnical investigation for this project and subsequent soil and rock parameters derived from the samples. In brief, Type 1 spread footings or 24-inch diameter Cast-in-Drilled Hole (CIDH) piles, between stations 93+60 to 95+53, are a feasible foundation type for the Retaining Wall 90. The CIDH pile design is based on the Load Resistance Factor (LRFD) Method. As such, guidelines from Section 3 of the 2007 AASHTO LRFD Bridge Design Specifications are followed in this Report. The design was further based on available topography, cross sections developed from soil borings drilled for this project. The design was also based on soil parameters summarized in Section 9 and Table 5 of this Report.

10.1 Standard Type 1 Spread Footings

The proposed Retaining Wall 90 may be supported on Standard Type 1 spread footings (between stations 92+08 and 95+53 for a wall height of 8-20 feet). However, from stations 93+60 to 95+53, 12 feet of overexcavation and replacement will need to be performed to mitigate the amount of potential settlement in this area, see Section 10.2.3. Section 10.1.1 provides detailed recommendations on the spread footings for Retaining Wall 90. Potential settlement for the retaining wall is discussed in Section 10.1.2.

10.1.1 Bearing Capacity

Standard Type 1 Retaining Walls supported on spread footings are based on soil conditions of 34 degree friction angle and 120 pcf unit weight, per Sheet B3-1A of the 2015 Standard Plans. Based on in-situ soil conditions the on-site soil conditions meet these criteria between stations 92+08 and 93+60. Therefore, the Type 1 Wall design may be used for the wall heights required on the plans. Between Stations 93+60 and 95+53 12 feet overexcavation and replacement with structural backfill material would have to be performed to achieve the same minimum soil parameter criteria. It should be noted that the minimum soil cover over the front the spread footing should be 1’-6” per page B3-3 of the Standard Plans.

10.1.2 Anticipated Settlement of Spread Footings

Total and differential settlements were calculated for the proposed retaining wall footings. Settlement was based on an allowable bearing capacity calculated for the retaining walls. The settlement parameters were estimated from generalized soil profiles for the proposed retaining wall footings. Based on a potentially compressible 20 foot thick clay layer (elevation +195 to 175 ft MSL) near station 92+00 to 93+60, the total and differential settlements for the proposed type 1 retaining walls are expected to be 1-inch and ½-inch, respectively. For stations between 93+60 to 95+53 with overexcavation and replacement, total settlement is expected to be less than 1-inch and differential settlement should therefore be less than ½-inch.

10.2 Retaining Wall on CIDH Piles

As an alternate, Retaining Wall No. 90 may be founded on 24-inch CIDH piles between Stations 93+60 and 95+53. The CIDH piles would be 24-inches in diameter and have a length of about 40-42 feet. Table No. 7 summarizes the pile design recommendations based on the LRFD Method for Retaining Wall 90. Wall height will vary along the limits and topography of the alignment, with the design height given as the maximum. The Pile Summary Table 7 provides design parameters for the soldier pile wall. Pile embedment is based on the three LRFD limit states, Service I, Strength I and Extreme Event I. Table number 8 summarizes the Pile Data Table information.

Pile lengths based on settlement analysis were based on a settlement amount of 1-inch. The method used to calculate the pile settlement was based on the Semi-Empirical Method (Vesic, 1977).

Lateral loads on the piles were analyzed using the computer program LPILE, Version 6 by Ensoft. Lateral loading was based on a deflection of 1-inch with a free head condition assumed for the retaining wall. The maximum lateral load allowed for the 1-inch deflection determined to be 70 kips. The maximum bending moment was determined to be 1.08 x10^4 in-kips acting at a depth of 17 feet below the top of pile. The lateral load analysis was based on subsurface conditions developed for this project and the associated input parameters summarized in Table 6 of this report.

Table No. 7 - Pile Design Recommendations

ERS station limits (ft)	Pile Type	Cut-off Elevation (ft)	Maximum Wall Height, (ft)	Service-I Limit State Load per Pile (kips)		Total Permissible Segment Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevations (ft)	Specified Tip Elevation (ft)
							Strength Limit		Extreme Event			
				Total	Permanent		Comp. (φ=0.70)	Tension (φ=0.70)	Comp. (φ=1)	Tension (φ=1)		
93+60 to 95+53	24" CIDH	215-217	20	101	-	1	133	67	154	77	175(a-I) 185(b-I) 175(a-II) 185 (b-II) 185(c) 177(d)	175

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, and (d) Lateral Load, respectively.*

Table No. 8 – Pile Data Table

ERS station limits	Pile Type	Maximum Wall Height, (ft)	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)
			Compression	Tension		
93+60 to 95+53	24"CIDH	*20*	190	95	175(a-I) 185(b-I) 175(a-II) 185 (b-II) 185(c) 177(d)	175

Notes:

1) *Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement, (d) Lateral Load*
2) *The specified tip elevation shall not be raised above the design tip elevations for Tension, Settlement, and Lateral Load.*

11.0 EARTHWORK

Excavation bottoms of the Retaining Wall 90 footprint for stations 92+08 to 93+60, where spread footing will be proposed, should be inspected for loose or soft areas. Any loose, soft, oversized or deleterious materials should be removed. The bottom should be prepared, compacted, and structural backfill placed per Section 19-3 of the 2010 Standard Specifications. The structure backfill material composition should comply with Section 19-3.02B of the Standard Specifications. On-site material, within the area to be excavated, is generally not suitable for structural backfill.

Side slopes should be excavated at no steeper than 1:1 temporary slopes with minimum bench widths of 3 feet. Excavation widths should equal the footing width plus 3-5 feet on either side of the footprint. Due to space constraints, a temporary shoring system will likely be needed on the freeway embankment side of the excavation. Soil parameters give in Table No. 6 of this report in combination with the earth pressure diagram shown on Figure 3.11.5.6-7 of the LRFD Bridge Design Specifications (2007) may be used for shoring design.

12.0 CONSTRUCTION ISSUES

- Sandstone and Claystone bedrock of the Capistrano formation will be encountered during drilling for drilled holes at the El Camino Real UC Widening Location. Hard to very hard cobbles should also be expected to be encountered in soils above the bedrock formation and within the formation itself.

- Although groundwater was not encountered during our investigation, historical data show the groundwater to be about +184 feet MSL. We therefore recommend the contractor be prepared to encounter groundwater and thus use the wet method during drilling.

- Caving is anticipated within the drilled holes for the CIDH piles, specifically within the native soil zones (approximately +215-180 feet elevation). Therefore, the contractor should devise a method to keep the drilled holes open.

- All temporary shoring must be removed upon completion.

- It is highly recommended that draft construction plans and specifications be submitted to this office for review before finalization.

12.0 REFERENCES

- Blanc, Robert P., 1930; Cleveland, George B. “Natural slope stability as related to geology, San Clemente area, Orange and San Diego Counties, California”, California. Division of Mines and Geology. -- San Francisco: California Geological Survey, 1968

- California Geological Survey. California Geological Survey, “Seismic Hazard Map, San Clemente 7.5-minute quadrangle”, Orange County, California, 2002.

- Vesic, A. S. 1977. “ASCE-Design of Pile Foundations No. 1: Design of Pile Foundations, “National Cooperative Highway Research Program, Synthesis of Highway Practice No. 42”, Transportation Research Board, Washington, DC.

If you have any questions, please contact Sam Sukiasian at (213) 620-2135 or Christopher Harris at (213) 620-2147.

Prepared by: Date: 2/4/16

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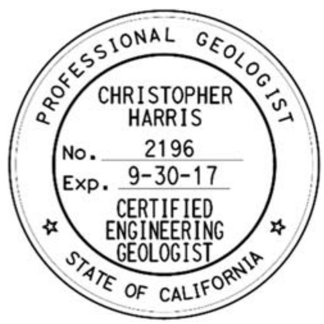

CHRISTOPHER HARRIS, C.E.G.
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Materials Engineer – behdad.baseghi@dot.ca.gov
Geotechnical Archive – http://svgcgeodog.dot.ca.gov/

Attachments:

Figures 1-4
Appendix A – Laboratory Results

Figures

Vicinity Map

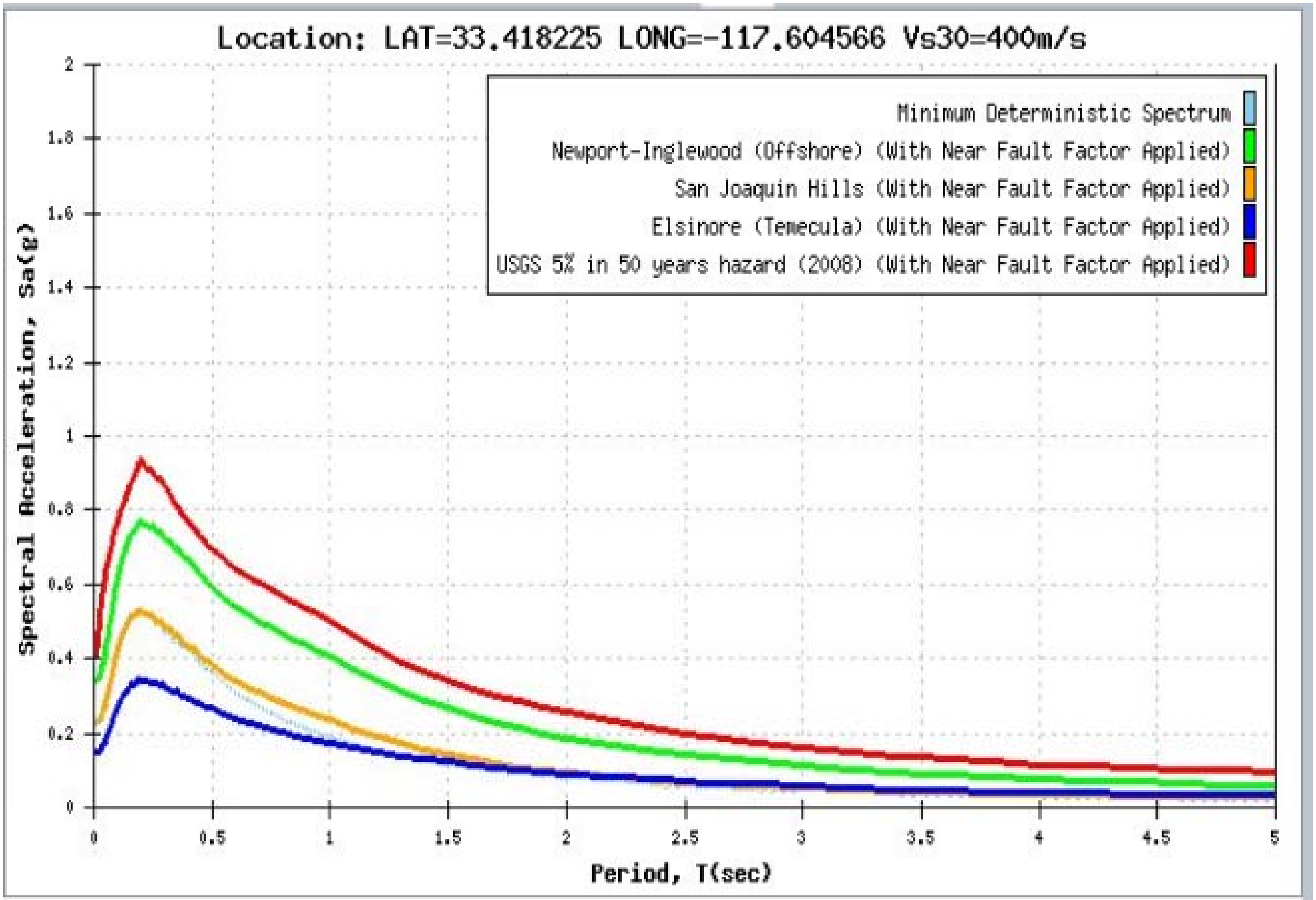
Figure 1

El Camino Real UC - Project Plan

Figure 2

El Camino Real UC - Boring Location Plan

Figure 3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Minimum Deterministic Spectrum
Newport-Inglewood (Offshore) (With Near Fault Factor Applied)
San Joaquin Hills (With Near Fault Factor Applied)
Elsinore (Temecula) (With Near Fault Factor Applied)
USGS 5% in 50 years hazard (2008) (With Near Fault Factor Applied)
Spectral Acceleration, Sa(g)
Period, T(sec)
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0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5

Figure 4

Appendix A – Laboratory Results

Direct Shear Tests (ASTM D3080)

Boring/ Sample No.	Depth , ft	Soil Description	Initial Dry Density, pcf (1)	Initial Water Content, % (1)	Normal Stress Range, psf	Ultimate Shear Strength Values	
						Friction Angle	Undrained Shear, psf
R-15-001/6	30	Sandy Clay	108.3	19.9	2000-6000	17	1780
R-15-002/4	20	Sandy Clay (Native)	112.5	18.2	1000-4000	28	776
R-15-002/6	30	Sandy	109	16.8	2000-6000	36	497

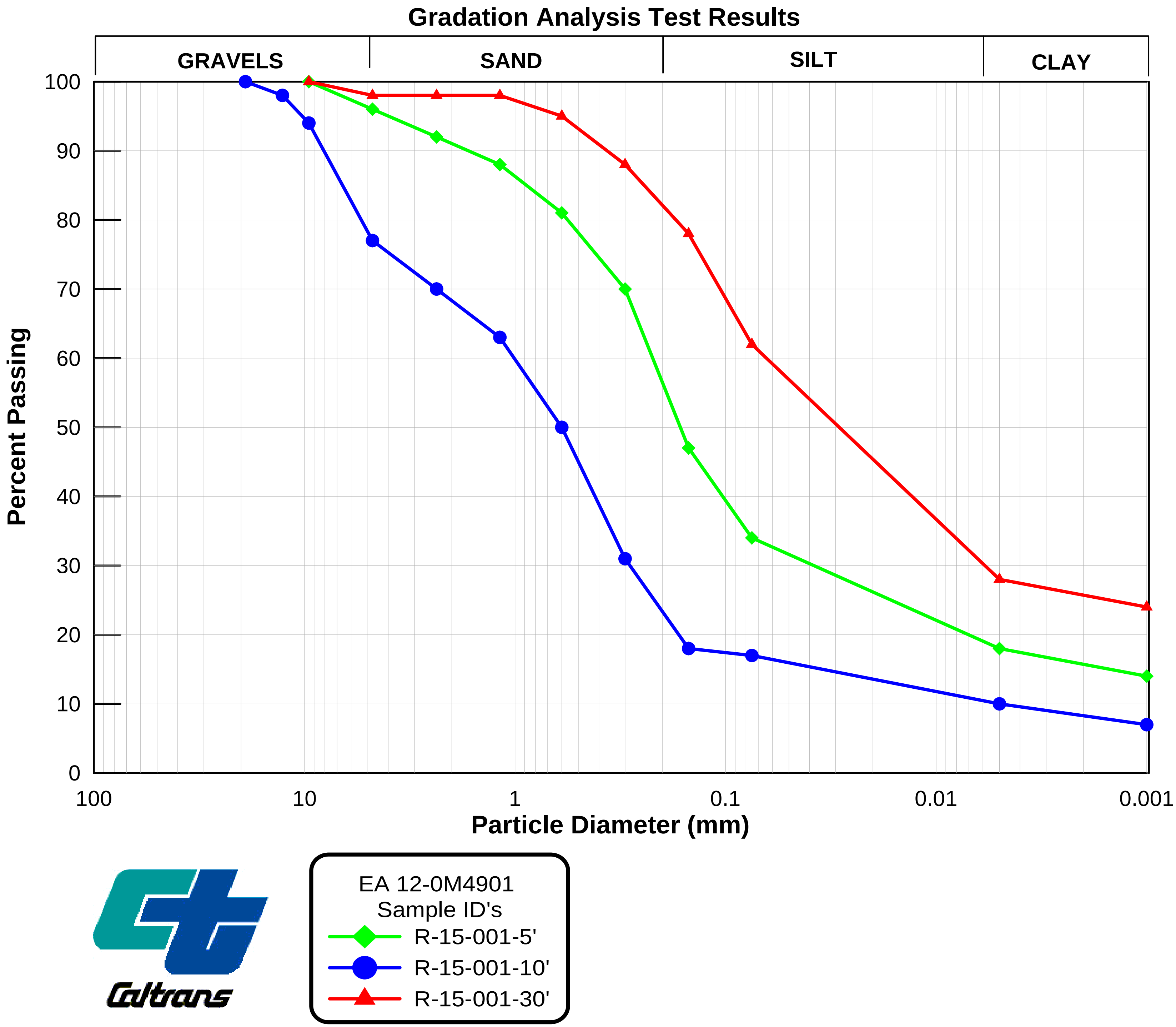
Note: (1) Average of three samples.

Unconfined Compression Test Results

(ASTM D2166)

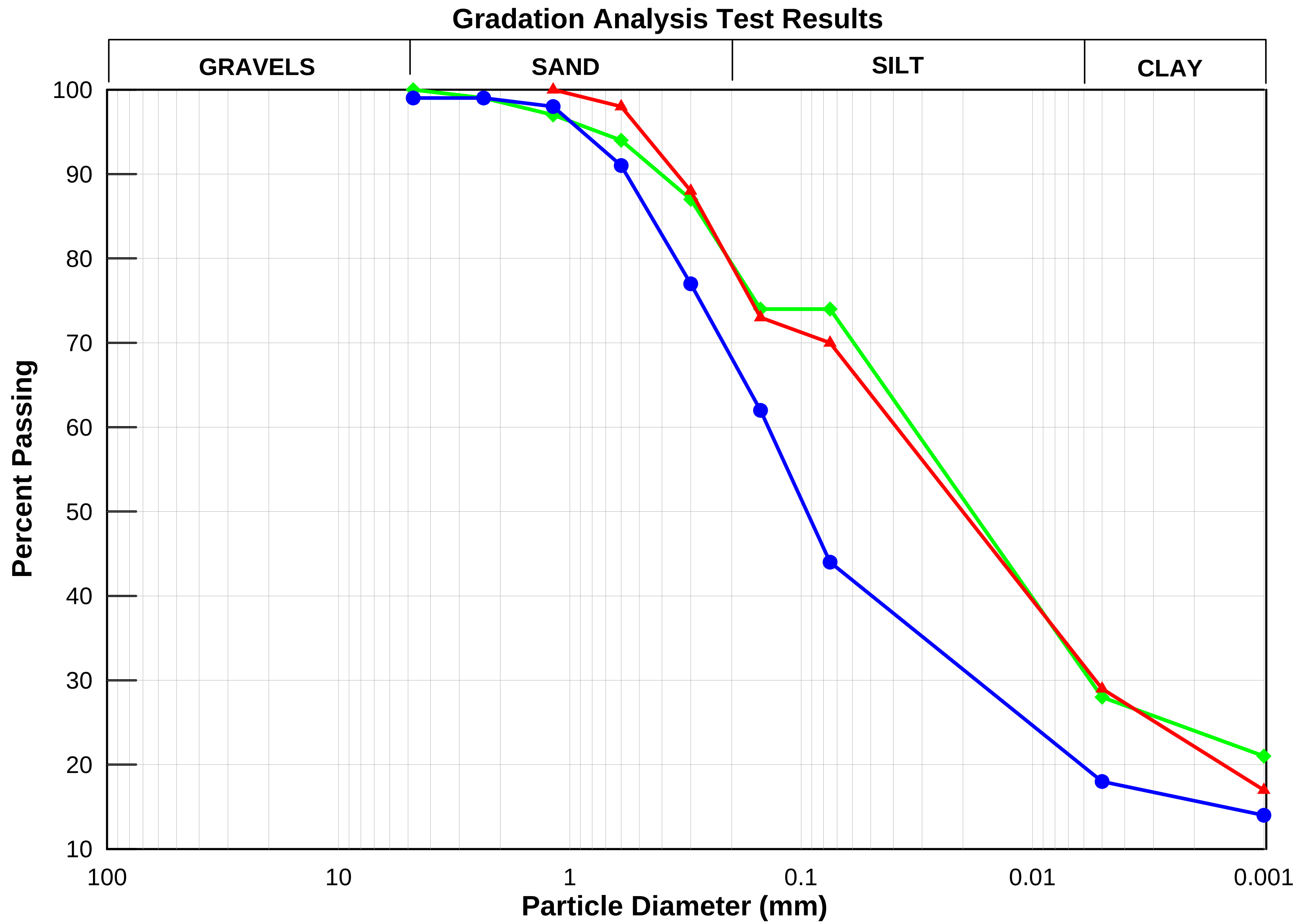
Boring/ Sample No.	Depth, ft	Soil Description	Initial Dry Density,	Initial Water Content, %	Unconfined Compressive Strength, psi	Shear Strength, psf
R-15-003/6	30	Formational	114	16.5	48.7	3500

Notes: Based on Strain Rate of 1%/min



Caltrans

Figure A-1


Gradation Analysis Test Results
GRAVELS
SAND
SILT
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Percent Passing
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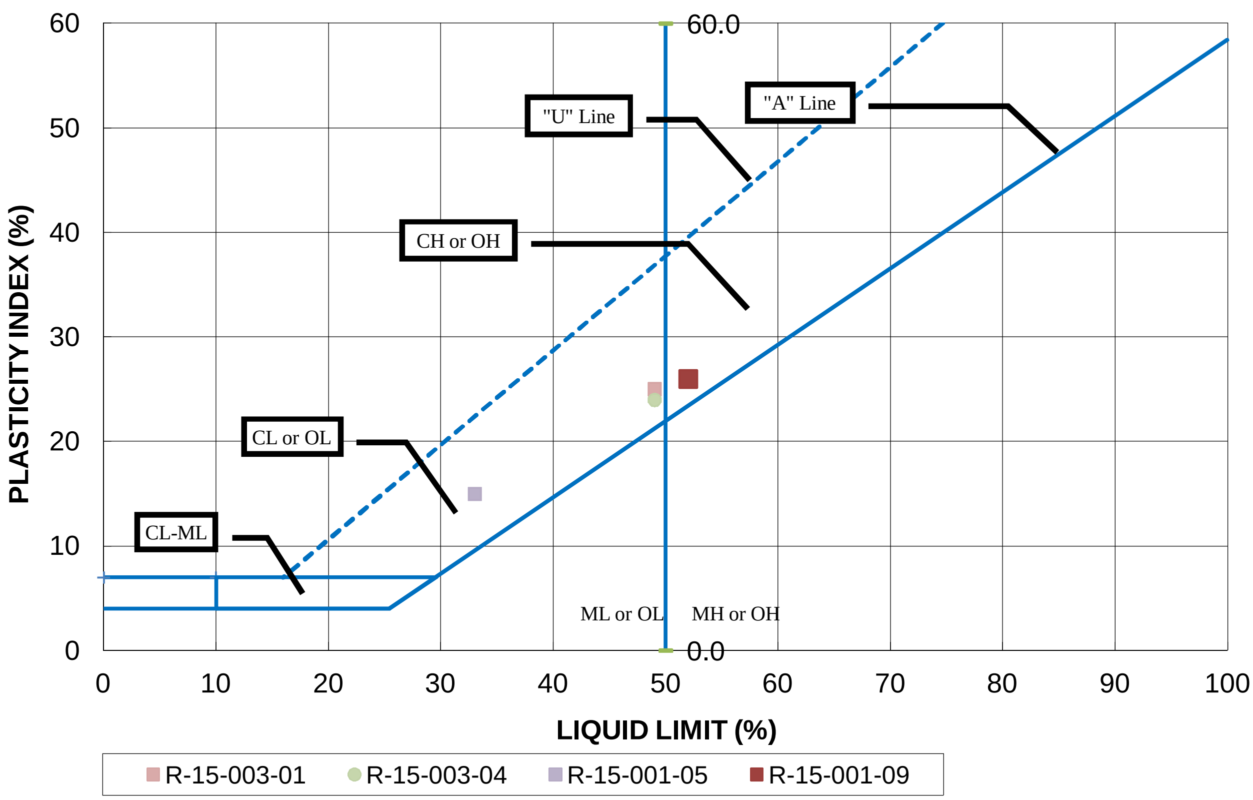
0.01
0.001
Particle Diameter (mm)
EA 12-0M4901
Sample ID's
R-15-002-09
R-15-002-04
R-15-002-14

Figure A-2

Atterberg Limits Test Results



Boring No.	Sample No.	Depth (ft)	LL	PI	PL	Classification
R-15-001	5	25	33	15	18	CL
R-15-001	9	45	52	26	26	CH
R-15-003	1	15	49	25	24	CL
R-15-003	4	20	49	24	25	CL

Caltrans

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1
(Location in parenthesis)

Project:	El Camino Real
EA	12-0M4901
Dist-Co-Route-PM	12-ORA-5-1.2

State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

M e m o r a n d u m

Serious drought. Help save water!

To: Mr. MATT HOLM, Branch Chief
Senior Transportation Engineer
Structure Design

Date: January 22, 2016

File: 12-ORA-5, PM 1.2
EA: 12-0M4901
Project ID: 1212000090
Curtain Wall 100

Attn: Mr. Jinrong Wang

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1
Branch C

Subject: Revised Foundation Report For El Camino Real UC Curtain Wall 100

Per your e-mail request dated February 10, 2015, a Revised Foundation Report (FR) has been prepared for the proposed El Camino Real UC Bridge (Bridge no. 55-0203) Widening located on Interstate 5 Freeway (I-5) in the City of San Clemente (see the Site Vicinity Map Figure 1). This Revised Report replaces the original FR dated September 24, 2015. The data in this FR include recommendations for foundation design of Curtain Wall 100 as part of the bridge widening located at Postmile 1.2 (Approximately 98+37 to 103+13 per ‘Alt2’ Mainline Centerline stationing of the I-5). The recommendations provided below are based on a subsurface investigation for the proposed Retaining Wall 90 per the latest general plans (Dated August 21, 2015). El Camino Real UC, Retaining Wall 103 and Retaining Wall 90, as part of the project plans, will be addressed in separate FR’s.

The recommendations provided below are based on observations of site conditions, including the adjacent retaining wall structure, review of soil borings drilled for this project, and the layout plans dated August 2015. This Revised Report updates the Seismicity Section and updates other minor edits.

1.1 Scope of Work

The following tasks were prepared for the preparation of this Report:

- Review of archive data and Preliminary Foundation Report
- Review of soil borings and laboratory test results
- Geotechnical Analysis
- Preparation of this FR

2.0 PROJECT DESCRIPTION

2.1 Existing Site Conditions and Proposed Improvement

The existing El Camino Real Bridge on I-5 at Postmile 1.2 is a 286 foot long and 150 feet wide three-span bridge supported on 16-inch Cast-in-Drilled-Hole (CIDH) 45-ton piles. Based on as-built foundation plans, existing embedded pile lengths are approximately 40 feet on the average. According to as-built plans dated 1958, the bridge was built as a 4 span bridge with a width of about 102 feet. Per As-builts dated 1983, the bridge was widened by about 9.25 feet on both sides. Existing Retaining walls adjacent to the bridge structure are founded on Type 1 spread footings. The new Curtain Wall 100 would be built about 10.5 feet from the existing edge of travel way (ETW) on the east side of the bridge. The Curtain wall 100 details along with details of the other proposed retaining walls are summarized in Table No. 1 below:

Table 1 – Proposed Retaining Wall Summary

Retaining Wall (RW) /Curtain Wall (CW)	Approximate Station (A-Line)	Wall Height, ft	Proposed Foundation Type
RW 90	92+08 to 93+60	8-10	Type 1 Spread Footing
RW 90	93+60 to 95+53	12-20	Type 1 Spread Footing or Type 1 on CIDH piles
CW 100	98+37 to 103+13	18-6	On CIDH piles
RW 103	103+13 to 104+09	8-6	Type 1 on Spread Footings

The Project Plan, Figure 2, also shows the location of the walls and bridge widening.

3.0 GEOLOGY

3.1 Site Geology

The project site is located along the inland side of a broad gently rolling portion of a non-marine terrace. The existing freeway is located adjacent to the southern flank of the base of the slope of the Santa Ana Mountains. The material under the site is mapped as Capistrano Formation (Blank and Cleveland 1968). The Capistrano Formation consists of siltstone, mudstone and silty shales with interbedded sandstones. The fill and alluvium encountered consists of soft to medium stiff clays and medium dense to dense sand and silty sand/sandy silt. The majority of the clays encountered outside the footprint of the existing embankment fill were soft, fat clays. Most of the clay under the existing embankment fills was more stiff most likely due to the presence of the overlying fill material having consolidated this material over time.

The closest fault to the site is the Newport-Inglewood (Offshore) fault oriented in a northwest-southeast direction approximately 4.5 miles southwest of the proposed project.

4.0 SUBSURFACE CONDITIONS

Eight total soil borings were drilled for the bridge undercrossing supports and proposed retaining and curtain walls to depths of 36.5 to 75.2 feet between June 30 and July 9 2015. Borings R-15-001 to 007 were drilled using the Rotary Wash method of drilling except A-15-004A, which was drilled using a 6-inch Hollow Stem Auger. This boring was drilled in order to obtain representative groundwater elevation conditions for the project site. The Boring locations are shown on Figure 3 of this report. Stationing, offsets and elevations of the soil borings are approximate and are summarized in the Table below. This information, to be updated, will also be provided on Log of Test Boring Sheets to be provided at a later date. Subsurface conditions are summarized below for each proposed structure. These summaries are based on the geotechnical investigation and laboratory results for this project.

Table 2 – Soil Borings Summary

Borings	Date Drilled	Station	Offset, ft	Surface Elevation, ft	Location
R-15-001	7/1/15	91+50	103R	216.1	N/B Offramp
R-15-002	7/7/15	94+75	50.4R	241.5	N/B Main right shoulder
R-15-003	7/1/15	94+88	126R	214.1	N/B Offramp
R-15-004	7/8/15	97+30	90R	216.8	N/B Onramp
A-15-004A	6/30/15	97+40	99R	217.2	N/B Onramp
R-15-005	7/8/15	98+30	113.4R	222.4	N/B Onramp
R-15-006	6/30/15	99+60	115.5R	234	N/B Onramp
R-15-007	7/7/15	104+00	55R	258.7	N/B Mainline right shoulder

Notes: (1) Approximate Stationing per layout plans dated March 2013. Per 'A Line' alignment.

<u>Curtain Wall 100</u>

Subsurface soils in the proposed Curtain Wall area consists of dense sands and stiff clays with increasing bedrock fragments and cobbles at an elevation of about 200 feet MSL near the south end, adjacent to the bridge. This material transitions to soft to medium stiff clays and loose to medium dense sands to an elevation of about 191 feet MSL going north along the proposed alignment. This material is underlain by the Capistrano Bedrock formation varying from 180 to 191 feet MSL.

5.0 SEISMICITY

The site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey. Based on Caltrans ARS Online Version 2.3.06, the Newport-Inglewood (Offshore) fault is the nearest active seismic source from the proposed project site.

Table No. 1 summarizes the faults identified by Caltrans ARS Online Version 2.3.06.

Table 3 – Fault and Design Ground Motion Parameters.

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (mi)	R_{JB} (mi)	R_X (mi)
Newport Inglewood (Offshore)	381	6.9	SS	90	V	4.5	4.5	4.5
San Joaquin Hills	376	7.0	REV	23	W	12.1	11.5	6.1
Elsinore (Temecula)	378	7.7	SS	90	V	21.2	21.2	21.2

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
$_{RUP}$ = Closest distance to the fault rupture plane

The design ARS curve is controlled by the probabilistic acceleration response spectrum curve. The probabilistic ARS curve is developed with a ground motion return period of 975 year which is corresponding with 5% probability to be exceeded in 50 years and the Next Generation Attenuation (NGA) is used for the deterministic ARS curve. The PGA was determined to be 0.4g with an average shear wave velocity (Vs) of 1300 fps in the top 100 feet of subsurface material. The design ARS Curve is provided on Figure 4.

6.0 LIQUEFACTION EVALUATION

Based on the absence of groundwater in the borings drilled for this project, with drilled depths reaching bedrock, liquefaction potential is considered negligible for this site. Furthermore, according to the California Geological Society (CGS) the site is outside of potentially liquefiable zones (Caltrans 2012).

7.0 LABORATORY TESTING

Laboratory testing was performed on selected soil samples from the investigation program by the Sacramento Geotechnical Laboratory. The materials tested are representative of the native soils and bedrock materials. Laboratory testing included Grain size analysis, Atterberg limits, direct shear strength, unconfined compression, and corrosivity tests. Geotechnical testing was performed in accordance with California Test Methods and/or ASTM procedures as indicted by Table 4. A summary of the laboratory results is included in Appendix A.

Table No. 4 – Laboratory Test Methods

Test	Standard
Gradation Analysis	ASTM D 422
Atterberg Limits	AASHTO T 90 & 89
Direct shear	ASTM D 3080
Unconfined Compression Strength	ASTM D 2166
Corrosion	CTM 643, CTM 422, CTM 417

8.0 CORROSIVITY

Composite bulk samples from Boring A-15-004A, were tested for corrosion potential. The results show that the fill soils in these areas are corrosive to buried concrete and metal. It is recommended that personnel from the Corrosion Technology Branch be consulted for corrosion protection recommendations. The results of the Corrosion Test Results are summarized in Table No. 5 below.

Table No. 5 – Corrosion Test Results

Boring	Depth (ft)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
A-15-004A	0-20	1294	9.12	N/A	N/A
A-15-004A	20-40	418	7.98	770	200
A-15-004A	40-60	363	7.07	720	1600

Note: Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

9.0 SOIL PARAMTERS

Embankment fill, native soil and bedrock parameters summarized in the table below were derived from in-situ field tests of soil and rock samples derived during the field exploration, correlations, and laboratory test results. Soil profiles and parameters were chosen based on worst case conservative conditions for design purposes only. The LOTB's, as part of the construction plans,

should be referred to for pile construction issues. A summary of the soil and rock parameters used for analysis of the piles and spread footings of the foundations is provided in the table below.

Table No. 6 – Summary of Soil Parameters

Location	Depth, ft	USCS	Unit Weight, pcf	ϕ, degrees	Su, psf
CW 100	0-6	SP	120	30	
	6-16	CL with fragments	120		400
	16-30	CL	120		530
	30-40	SP	120	30	
	40-60	Sandstone (Formational)	122	38	

10.0 FOUNDATION RECOMMENDATIONS

Foundation recommendations were based on subsurface soil conditions derived from the results of the geotechnical investigation for this project and subsequent soil and rock parameters derived from the samples. In brief, 24-inch diameter Cast-in-Drilled Hole (CIDH) piles are a feasible foundation type for the El Camino Real undercrossing abutment and pier locations. The CIDH pile design is based on the Load Resistance Factor (LRFD) Method. As such, guidelines from Section 3 of the 2007 AASHTO LRFD Bridge Design Specifications are followed in this Report. The design was further based on available topography, cross sections developed from soil borings drilled for this project. The design was also based on soil parameters summarized in Section 9 and Table 6 of this Report.

10.1 Retaining Wall on CIDH Piles

Curtain Wall No. 100 may be founded on 24-inch CIDH piles between Stations 98+37 and 103+13. The CIDH piles would be 24-inches in diameter and have a length of about 60 feet. Table No. 7 summarizes the pile design recommendations based on the LRFD Method for Curtain Wall 100. The wall height will vary along the limits and topography of the alignment, with the design height given as the maximum. The pile lengths should not vary with the height of the walls. Pile embedment is based on the three LRFD limit states, Service I, Strength I and Extreme Event I. Table Number 8 summarizes the Pile Data Table information. Cutoff elevations for Curtain Wall 100 range from 230 to 250.5 feet with 0.5 to 1 foot steps for each of the 30 piles as given by Structure Design. For brevity a range of cutoff elevations and specified pile tips is given in Tables 7 and 8.

Pile lengths based on settlement analysis were based on a settlement amount of 1-inch. The method used to calculate the pile settlement was based on the Semi-Empirical Method (Vesic, 1977).

Lateral loads on the piles were analyzed using the computer program LPILE, Version 6 by Ensoft. Lateral loading was based on a deflection of 1-inch with a free head condition assumed for the retaining wall. The maximum lateral load allowed for the 1-inch deflection determined to be 95 kips. The maximum bending moment was determined to be 7700 in-kips acting at a depth of 12 feet below the top of pile. The lateral load analysis was based on subsurface conditions developed for this project and the associated input parameters summarized in Table 6 of this report.

Table No. 7 - Pile Design Recommendations

ERS station limits (ft)	Pile Type	Cut-off Elevation (ft)	Maximum Wall Height, (ft)	Service-I Limit State Load per Pile (kips)		Total Permissible Segment Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevations (ft)	Specified Tip Elevation (ft)
							Strength Limit		Extreme Event			
				Total	Permanent		Comp. (φ=0.70)	Tension (φ=0.70)	Comp. (φ=1)	Tension (φ=1)		
98+37 to 103+13	24" CIDH	230-250	18.3	118	-	1	266	-	380	-	170-190(a-I) 170-190(a-II) 190-210(c) 190-210 (d)	170-190

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, and (d) Lateral Load, respectively.*

Table No. 8 – Pile Data Table

ERS station limits	Pile Type	Maximum Wall Height, (ft)	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)
			Compression	Tension		
98+37 to 103+13	24" CIDH	18.3	380	-	170-190(a-I) 170-190(a-II) 190-210 (c) 190-210(d)	170-190 (3)

Notes:

1) *Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement, (d) Lateral Load*
2) *The specified tip elevation shall not be raised above the design tip elevations for Tension, Settlement, and Lateral Load.*
3) *Specified pile tips range from 170 to 190 feet MSL based on cutoff elevations of 2330 to 250 feet as given by Structure Design. Pile lengths of 60 feet should not vary along the length of the wall.*

11.0 CONSTRUCTION ISSUES

- Sandstone and Claystone bedrock of the Capistrano formation will be encountered during drilling for drilled holes at the El Camino Real UC Widening Location. Hard to very hard cobbles should also be expected to be encountered in soils above the bedrock formation and within the formation itself.

- Although groundwater was not encountered during our investigation, historical data show the groundwater to be about +184 feet MSL. We therefore recommend the contractor be prepared to encounter groundwater and thus use the wet method during drilling.

- Caving is anticipated within the drilled holes for the CIDH piles, specifically within the native soil zones (approximately +235 to 190 feet elevation). Therefore, the contractor should devise a method to keep the drilled holes open.

- All temporary shoring must be removed upon completion.

- It is highly recommended that draft construction plans and specifications be submitted to this office for review before finalization.

12.0 REFERENCES

- Blanc, Robert P., 1930; Cleveland, George B. “Natural slope stability as related to geology, San Clemente area, Orange and San Diego Counties, California”, California. Division of Mines and Geology. -- San Francisco: California Geological Survey, 1968

- California Geological Survey. California Geological Survey, “Seismic Hazard Map, San Clemente 7.5-minute quadrangle”, Orange County, California, 2002.

- Vesic, A. S. 1977. “ASCE-Design of Pile Foundations No. 1: Design of Pile Foundations, “National Cooperative Highway Research Program, Synthesis of Highway Practice No. 42”, Transportation Research Board, Washington, DC.

If you have any questions, please contact Sam Sukiasian at (213) 620-2135 or Christopher Harris at (213) 620-2147.

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Geotechnical Archive – http://svgcgeodog.dot.ca.gov/

Attachments:

Figures 1-4
Appendix A – Laboratory Results

Figures

Vicinity Map

Figure 1

El Camino Real UC - Project Plan

Figure 2

El Camino Real UC - Boring Location Plan

Figure 3

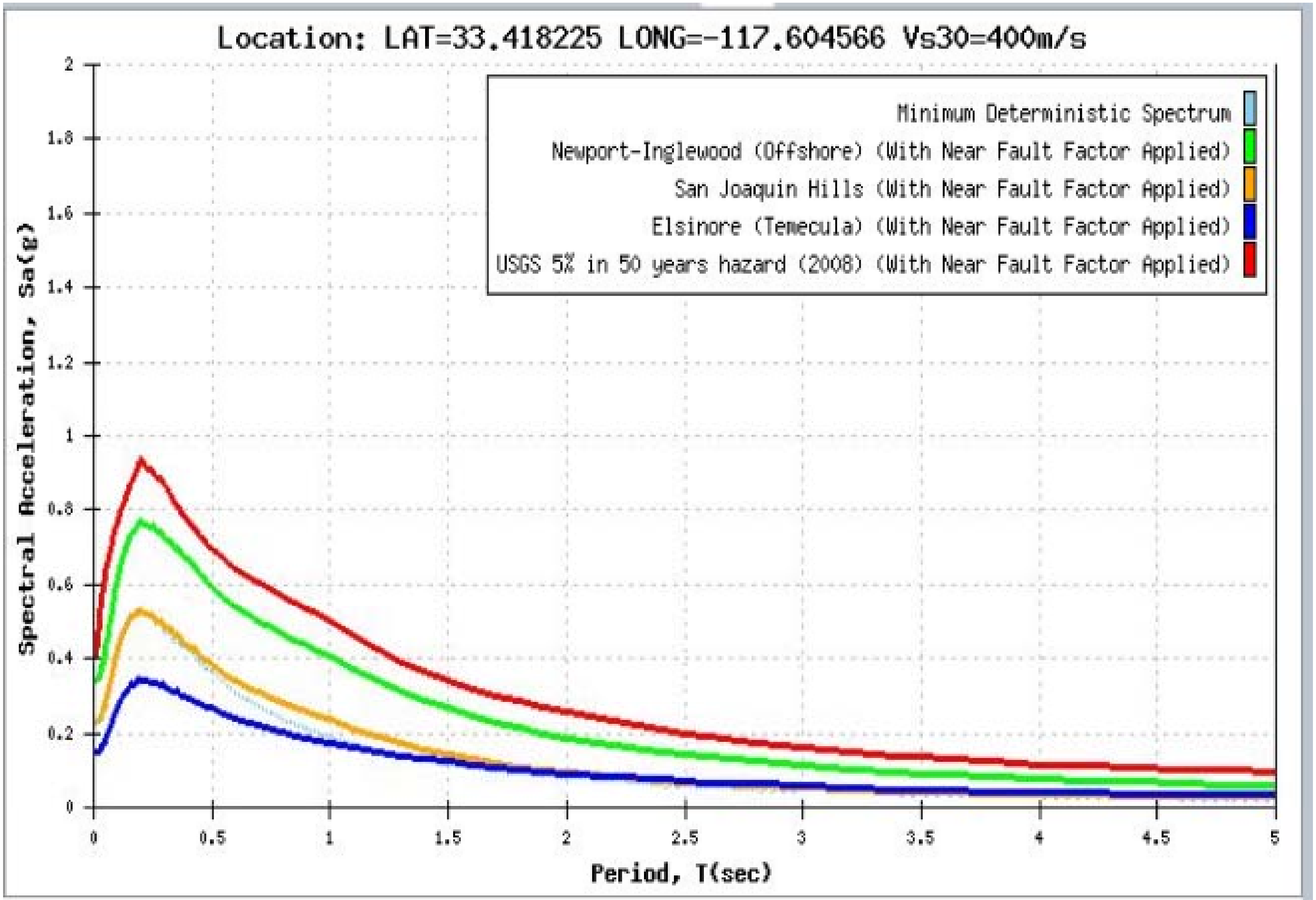

Location: LAT=33.418225 LONG=-117.604566 Vs30=400m/s
Minimum Deterministic Spectrum
Newport-Inglewood (Offshore) (With Near Fault Factor Applied)
San Joaquin Hills (With Near Fault Factor Applied)
Elsinore (Temecula) (With Near Fault Factor Applied)
USGS 5% in 50 years hazard (2008) (With Near Fault Factor Applied)
Spectral Acceleration, Sa(g)
Period, T(sec)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5

Figure 4

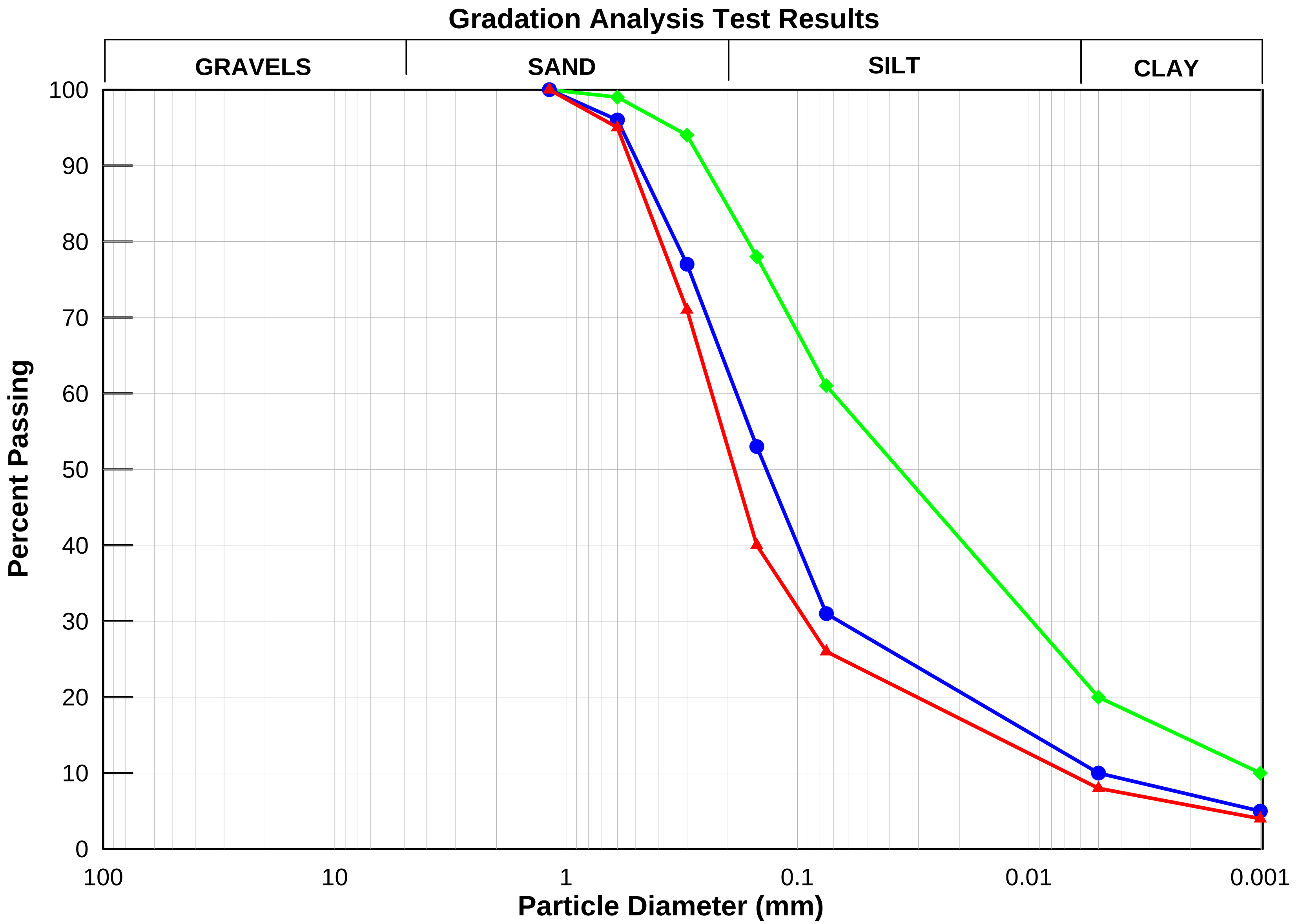
Appendix A – Laboratory Results

Unconfined Compression Test Results

(ASTM D2166)

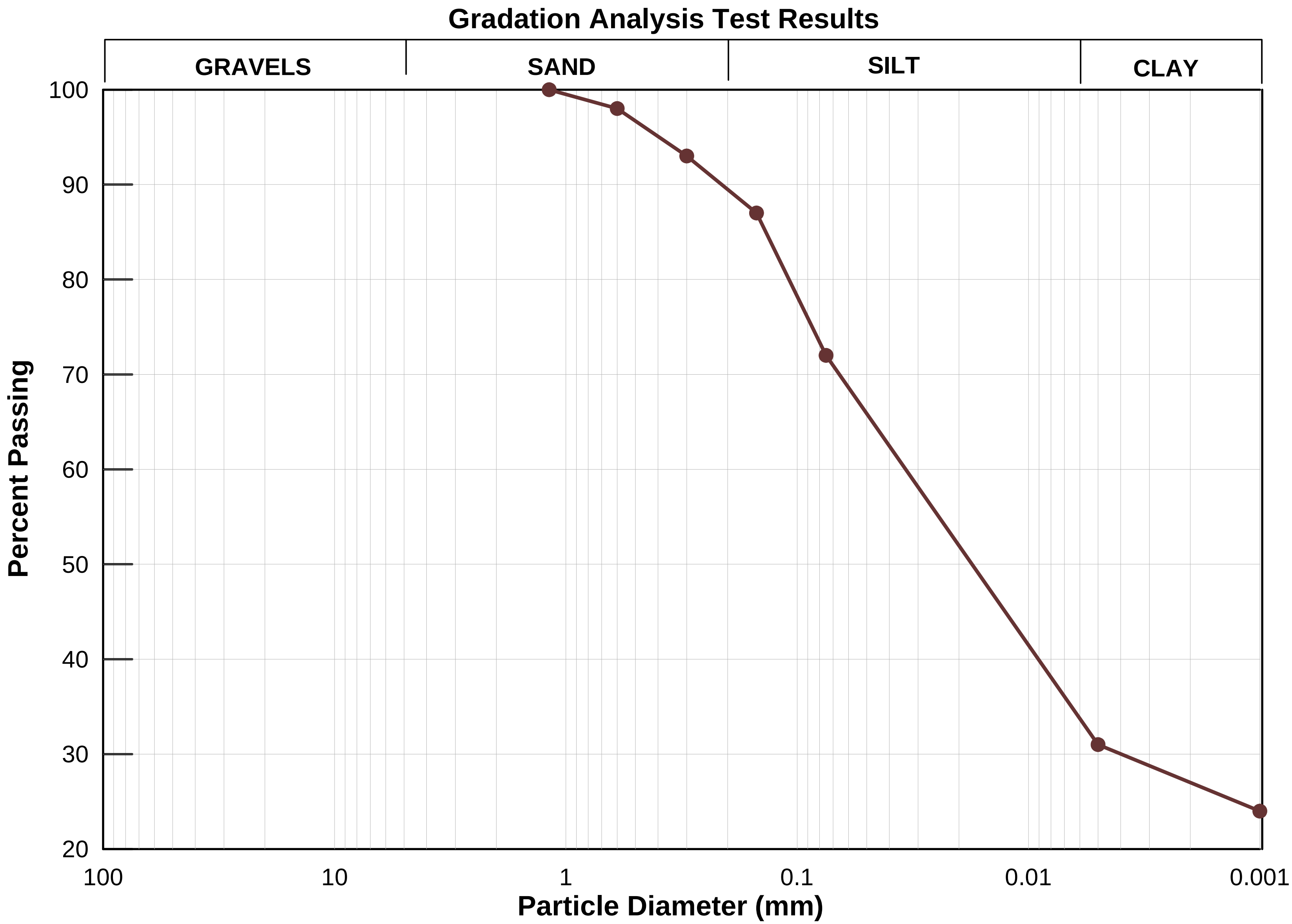
Boring/ Sample No.	Depth, ft	Soil Description	Initial Dry Density,	Initial Water Content, %	Unconfined Compressive Strength, psi	Shear Strength, psf
R-15-005/5	25	Formational	101	22.5	44.2	3180

Notes: Based on Strain Rate of 1%/min



EA 12-0M4901
Sample ID's
R-15-005-04
R-15-005-09
R-15-005-10

Figure A-1

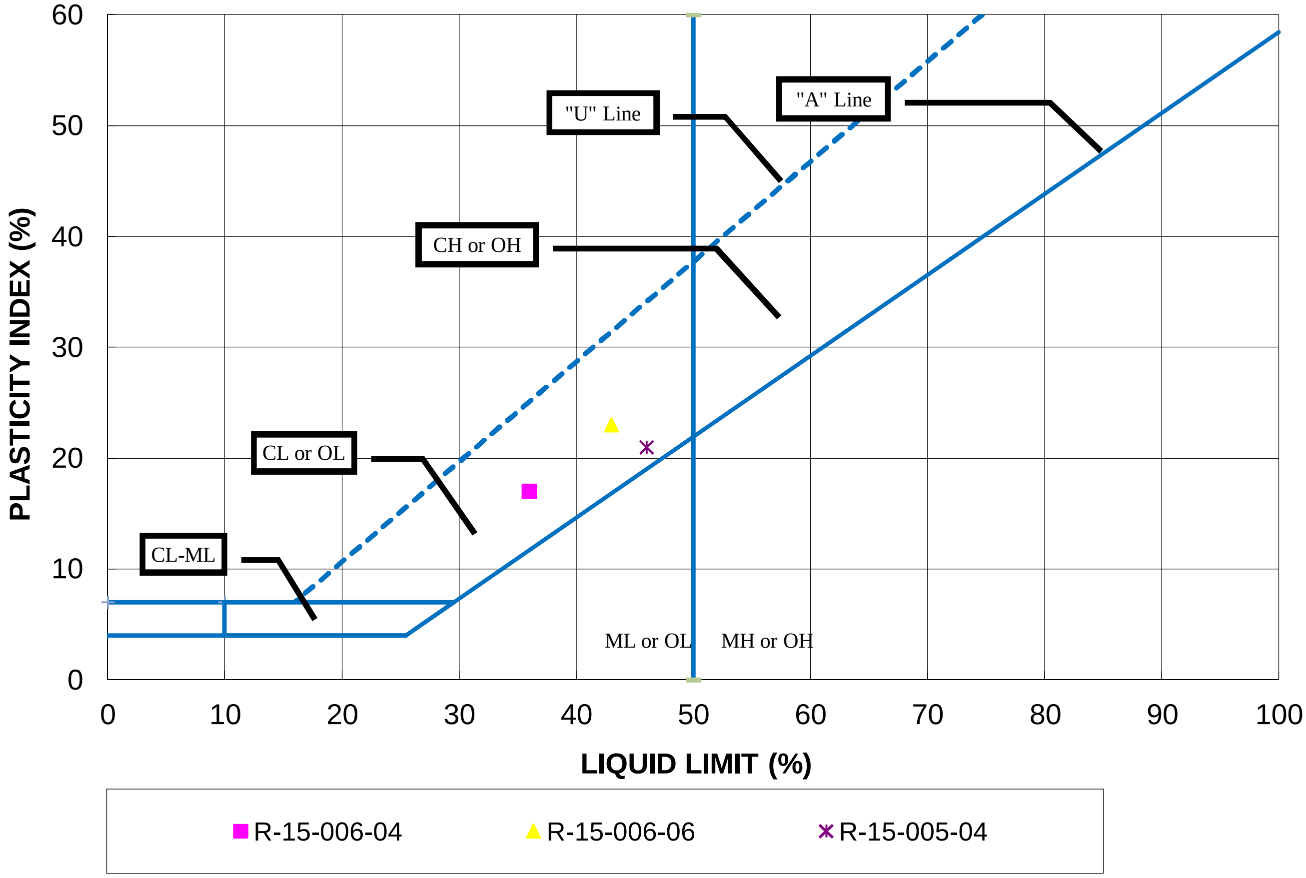


Caltrans

EA 12-0M4901
Sample ID's
R-15-006-5

Figure A-2

Atterberg Limits Test Results



Boring No.	Sample No.	Depth (ft)	LL	PI	PL	Classification
R-15-006	4	20'	36	17	19	CL
R-15-006	6	30'	43	23	20	CL
R-15-005	4	10'	46	21	25	CL

Caltrans

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1

Project:	El Camino Real
EA:	12-0M4901
Dist-Co-Rte-PM:	12-ORA-5-1.2

State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

M e m o r a n d u m

Serious drought.
Help save water!

To: Mr. MATT HOLM, Branch Chief
Senior Transportation Engineer
Structure Design

Date: February 4, 2016

File: 12-ORA-5, PM 1.2
EA: 12-0M4901
Project ID: 1212000090
Retaining Wall 103

Attn: Mr. Jinrong Wang

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1
Branch C

Subject: 2nd Revised Foundation Report For El Camino Real UC Retaining Wall 103

Per your e-mail request dated February 10, 2015, this 2nd Revised Foundation Report (FR) has been prepared for the proposed El Camino Real UC Bridge (Bridge no. 55-0203) Widening located on Interstate 5 Freeway (I-5) in the City of San Clemente (see the Site Vicinity Map Figure 1). This Revised Report replaces the Revised FR dated January 22, 2016. The data in this FR includes recommendations for foundation design of Retaining Wall 103 as part of the bridge widening located at Postmile 1.2 (Approximately 103+13 to 104+09 per the ‘A-Line’ Alignment). The recommendations provided below are based on a subsurface investigation for the proposed Retaining Wall 103 per the latest general plans (Dated August 21, 2015). El Camino Real UC, Retaining Wall 90 and Curtain Wall 100, as part of the project plans, will be addressed in separate FR’s.

The recommendations provided below are based on observations of site conditions, including the adjacent retaining wall structure, review of soil borings drilled for this project, and the layout plans dated August 2015. The revisions in this report include reference back to the 2010 Standard Plans with 2012 Plan Updates applied to Tables 6A through 6C.

1.1 Scope of Work

The following tasks were prepared for the preparation of this Report:

- Review of archive data and Preliminary Foundation Report
- Review of soil borings and laboratory test results
- Geotechnical Analysis
- Preparation of this FR

2.0 PROJECT DESCRIPTION

2.1 Existing Site Conditions and Proposed Improvement

The existing El Camino Real Bridge on I-5 at Postmile 1.2 is a 286 foot long and 150 feet wide three-span bridge supported on 16-inch Cast-in-Drilled-Hole (CIDH) 45-ton piles. Based on as-built foundation plans, existing embedded pile lengths are approximately 40 feet on the average. According to as-built plans dated 1958, the bridge was built as a 4 span bridge with a width of about 102 feet. Per As-builts dated 1983, the bridge was widened by about 9.25 feet on both sides. Existing Retaining walls adjacent to the bridge structure are founded on Type 1 spread footings. The new Retaining Wall 103 would be built about 10.5 feet from the existing edge of travel way (ETW) on the east side of the bridge. Retaining Wall 103 details along with the adjacent proposed Retaining wall 90 and Curtain Wall 100 are summarized in Table No. 1 below:

Table 1 – Proposed Retaining Wall Summary

Retaining Wall (RW) /Curtain Wall (CW)	Approximate Station (A-Line)	Wall Height, ft	Proposed Foundation Type
RW 90	92+08 to 93+60	8-10	Type 1 Spread Footing
RW 90	93+60 to 95+53	12-20	Type 1 Spread Footing or Type 1 on CIDH piles
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RW 103	103+13 to 104+09	8-6	Type 1 on Spread Footings

The Project Plan, Figure 2, also shows the location of the walls and bridge widening.

3.0 GEOLOGY

3.1 Site Geology

The project site is located along the inland side of a broad gently rolling portion of a non-marine terrace. The existing freeway is located adjacent to the southern flank of the base of the slope of the Santa Ana Mountains. The material under the site is mapped as Capistrano Formation (Blank and Cleveland 1968). The Capistrano Formation consists of siltstone, mudstone and silty shales with interbedded sandstones. The fill and alluvium encountered consists of soft to medium stiff clays and medium dense to dense sand and silty sand/sandy silt. The majority of the clays encountered outside the footprint of the existing embankment fill were soft, fat clays. Most of the clay under the existing embankment fills was more stiff most likely due to the presence of the overlying fill material having consolidated this material over time.

The closest fault to the site is the Newport-Inglewood (Offshore) fault oriented in a northwest-southeast direction approximately 4.5 miles southwest of the proposed project.

4.0 SUBSURFACE CONDITIONS

Eight total soil borings were drilled for the bridge undercrossing supports and proposed retaining and curtain walls to depths of 36.5 to 75.2 feet between June 30 and July 9 2015. Borings R-15-001 to 007 were drilled using the Rotary Wash method of drilling except A-15-004A, which was drilled using a 6-inch Hollow Stem Auger. This boring was drilled in order to obtain representative groundwater elevation conditions for the project site. The Boring locations are shown on Figure 3 of this report. Stationing, offsets and elevations of the soil borings are approximate and are summarized in the Table below. This information, to be updated, will also be provided on Log of Test Boring Sheets to be provided at a later date. Subsurface conditions are summarized below for each proposed structure. These summaries are based on the geotechnical investigation and laboratory results for this project.

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R-15-004	7/8/15	97+30	90R	216.8	N/B Onramp
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Notes: (1) Approximate Stationing per layout plans dated March 2013. Per ‘A Line’ alignment.

<u>Retaining Wall 103</u>

Subsurface materials in this area were found to be composed of 6 feet of medium dense sands overlying stiff to very stiff sandy clays and silts to an elevation of about 221 feet MSL. This material was found to be underlain by sandstone material of the Capistrano Bedrock formation.

5.0 SEISMICITY

The site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey. Based on Caltrans ARS Online Version 2.3.06, the Newport-Inglewood (Offshore) fault is the nearest active seismic source from the proposed project site.

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Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{UP} = Closest distance to the fault rupture plane

The design ARS curve is controlled by the probabilistic acceleration response spectrum curve. The probabilistic ARS curve is developed with a ground motion return period of 975 year which is corresponding with 5% probability to be exceeded in 50 years and the Next Generation Attenuation (NGA) is used for the deterministic ARS curve. The PGA was determined to be 0.4g with an average shear wave velocity (Vs) of 1300 fps in the top 100 feet of subsurface material. The design ARS Curve is provided on Figure 4.

6.0 LIQUEFACTION EVALUATION

Based on the absence of groundwater in the borings drilled for this project, with drilled depths reaching bedrock, liquefaction potential is considered negligible for this site. Furthermore, according to the California Geological Society (CGS) the site is outside of potentially liquefiable zones (Caltrans 2012).

7.0 LABORATORY TESTING

Laboratory testing was not performed for this portion of the widening. However, test results from adjacent boring samples for Curtain Wall 100 were used to help develop design parameters for RW 103. Please refer to Section 7.0 and Appendix A of the CW 100 FR for Laboratory Test Results. In addition Corrosion test results from other portions of the improvement were applied to this wall, see Section 8.0.

8.0 CORROSIVITY

Composite bulk samples from Boring A-15-004A, were tested for corrosion potential. The results show that the fill soils in these areas are corrosive to buried concrete and metal. It is recommended that personnel from the Corrosion Technology Branch be consulted for corrosion protection recommendations. The results of the Corrosion Test Results are summarized in Table No. 4 below.

Table No. 4 – Corrosion Test Results

Boring	Depth (ft)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
A-15-004A	0-20	1294	9.12	N/A	N/A
A-15-004A	20-40	418	7.98	770	200
A-15-004A	40-60	363	7.07	720	1600

Note: Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

9.0 SOIL PARAMTERS

Embankment fill, native soil and bedrock parameters summarized in the table below were derived from in-situ field tests of soil and rock samples derived during the field exploration, correlations, and laboratory test results. Soil profiles and parameters were chosen based on worst case conservative conditions for design purposes only. The LOTB's, as part of the construction plans, should be referred to for pile construction issues. A summary of the soil and rock parameters used for analysis of the piles and spread footings of the foundations is provided in the table below.

Table No. 5 – Summary of Soil Parameters

Location	Depth, ft	USCS	Unit Weight, pcf	ϕ, degrees	Su, psf
RW 103	0-4	SP	120	30	
	4-35	CL	120	-	1200
	35-50	Sandstone (Formational)	122	38	

10.0 FOUNDATION RECOMMENDATIONS

Foundation recommendations were based on subsurface soil conditions derived from the results of the geotechnical investigation for this project and subsequent soil and rock parameters derived from the samples. In brief, Type 1 spread footings, between stations 103+13 to 104+09, are a feasible foundation type for the Retaining Wall 103. The design was based on available topography, cross sections developed from soil borings drilled for this project. The design was also based on soil parameters summarized in Section 9 and Table 5 of this Report.

10.1 Standard Type 1 Spread Footings

The proposed Retaining Wall 103 may be supported on Standard Type 1 spread footings (between stations 103+13 and 104+09 for a wall height of 6-8 feet). It is recommended 4 feet of overexcavation and replacement be performed within the footprint area to remove the upper medium dense sands. Section 10.1.1 provides detailed recommendations on the spread footings for Retaining Wall 103. Potential settlement for the retaining wall is discussed in Section 10.1.2.

10.1.1 Bearing Capacity and Settlement

Standard Type 1 Retaining Walls (Case I) supported on spread footings may be used for Retaining Wall 103. Using the basic equation for nominal bearing capacity, (10.6.3.1.2a-1, pg 10-61 of the LRFD Manual) service, strength, and extreme bearing stresses provided in the 2010 Standard Plans (2012 Updated Plan B3-1A) are satisfied, as summarized in Tables 6A, 6B, and 6C. In Table 6A Bearing Resistance was based on an undrained shear strength of 1,200 psf and a resistance factor of 0.5 for wall heights of 6 and 8 feet. In Table 6B, the extreme limit state II bearing stress was used with the 54 kip collision load in the force effect load combination equation, shown in B3-1A. Table 6C shows calculated total settlement of the proposed spread footings to be within the total permissible settlement of 1-inch for the retaining wall based on Service Limit State Loads. Settlement is based on a 20 foot thick compressible sandy clay layer with an estimated compression index (C_c) of 0.2 and a Boussinesq influence factor of 0.3 for rectangular footings. The Type 1 Spread Footing data table is shown on Table 6D.

Table 6A - Strength Limit State Foundation Data for Type 1

ERS Stationing (ft)	Design Height (ft)	Bottom of Footing Elevation (ft)	Footing Width (ft)	Minimum Footing Embedment Depth (ft)	Effective Footing Width for Strength Limit State (ft)	Strength Limit State Gross Uniform Bearing Stress (psf)	Strength Limit State Factored Bearing Resistance (psf) ϕ_b (q_N) ϕ_b =0.5
103+13 to 103+61	8.0	250.5	7.25	3.3	3.6	2300	4000
103+61 to 104+09	6.0	248.5	7.0	3.3	5.0	1800	4000

Table 6B - Extreme Limit State II Foundation Data Table for Type 1

ERS Stationing	Design Height (ft)	Bottom of Footing Elevation (ft)	Footing Width (ft)	Minimum Footing Embedment Depth (ft)	Effective Footing Width for Extreme Limit State (ft)	Extreme Limit State Gross Uniform Bearing Stress (psf)	Extreme Limit State Factored Bearing Resistance (psf) ϕ_b (q_N) ϕ_b =1.0
103+13 to 103+61	8.0	250.5	7.25	3.3	2.8	2800	8000
103+61 to 104+09	6.0	248.5	7.0	3.3	2.7	2600	8000

Table 6C - Service Limit State Foundation Data Table

ERS Statio ning (ft)	Design Height (ft)	Bottom of Footing Elevation (ft)	Footing Width (ft)	Minimum Footing Embedment Depth (ft)	Effective Footing Width for Service Limit State (ft)	Service Limit State Net Bearing Stress (psf)	Calculated Settlement at Net Bearing Pressure (inches)	Total Permissible Settlement (inches)
103+13 to 103+61	8.0	250.5	7.25	3.3	6.2	1300	1.0	1.0
103+61 to 104+09	6.0	248.5	7.0	3.3	6.5	1000	0.6	1.0

Table 6D - Spread Footing Data Table

ERS Stationing (ft)	Design Height (ft)	Service Limit State Permissible Net Contact Stress (ksf)	Strength Factored Gross Nominal Bearing Resistance for Controlling Load Case $\phi_b = 0.5$ (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance for Controlling Load Case
103+13 to 103+61	8.0	1.2	4.0	8.0
103+61 to 104+09	6.0	0.9	4.0	8.0

10.1.2 Sliding Resistance

It is anticipated that the proposed footings will be resting on sandy clay. Therefore, the sliding resistance of the footings should be taken as the 1,200 psf (estimated cohesion of the sandy clay material). Per the LRFD Manual Section 10.6.3.4, the resistance factor, for concrete on sandy clay, should be taken as 0.85.

11.0 EARTHWORK

Excavation bottoms of the Retaining Wall 103 footprint between stations 103+13 and 104+09 should be inspected for loose or soft areas. Any loose, soft, oversized or deleterious materials should be removed. The bottom of the excavation should be prepared, compacted, and structural backfill placed per Section 19-3 of the 2010 Standard Specifications. The structure backfill material composition should comply with Section 19-3.02B of the Standard Specifications. On-site material, within the area to be excavated, is generally not suitable for structural backfill.

All temporary slopes should be excavated at no steeper than 1:1 with minimum bench widths of 3 feet. Excavation widths should equal the footing width plus 3-5 feet on either side of the footprint. Due to space constraints, a temporary shoring system will likely be needed on the freeway embankment side of the excavation. Soil parameters give in Table No. 5 of this report in combination with the earth pressure diagram shown on Figure 3.11.5.6-7 of the LRFD Bridge Design Specifications (2007) may be used for shoring design.

12.0 CONSTRUCTION ISSUES

- All temporary shoring must be removed upon completion.
- It is highly recommended that draft construction plans and specifications be submitted to this office for review before finalization.

13.0 REFERENCES

- Blanc, Robert P., 1930; Cleveland, George B. “Natural slope stability as related to geology, San Clemente area, Orange and San Diego Counties, California”, California. Division of Mines and Geology. -- San Francisco: California Geological Survey, 1968
- California Geological Survey. California Geological Survey, “Seismic Hazard Map, San Clemente 7.5-minute quadrangle”, Orange County, California, 2002.
- Vesic, A. S. 1977. “ASCE-Design of Pile Foundations No. 1: Design of Pile Foundations, “National Cooperative Highway Research Program, Synthesis of Highway Practice No. 42”, Transportation Research Board, Washington, DC.

If you have any questions, please contact Sam Sukiasian at (213) 620-2135 or Christopher Harris at (213) 620-2147.

Prepared by: Date: 2/4/16

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Reviewed By: Date: 2/4/16

CHI-TSENG LIU, PhD, G.E.
Senior Transportation Engineer
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Prepared By: Date: 2/4/16

CHRISTOPHER HARRIS, C.E.G.
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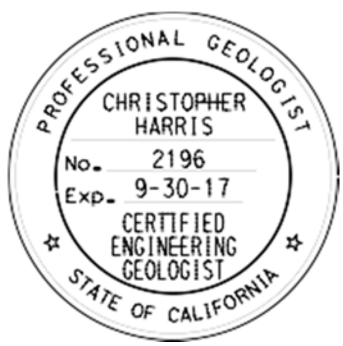


cc. Project Manager – ahmed.abou-abdou@dot.ca.gov
Materials Engineer – behdad.baseghi@dot.ca.gov
Geotechnical Archive – http://svgcgeodog.dot.ca.gov/

Attachments:

Figures 1-4

Figures

Vicinity Map

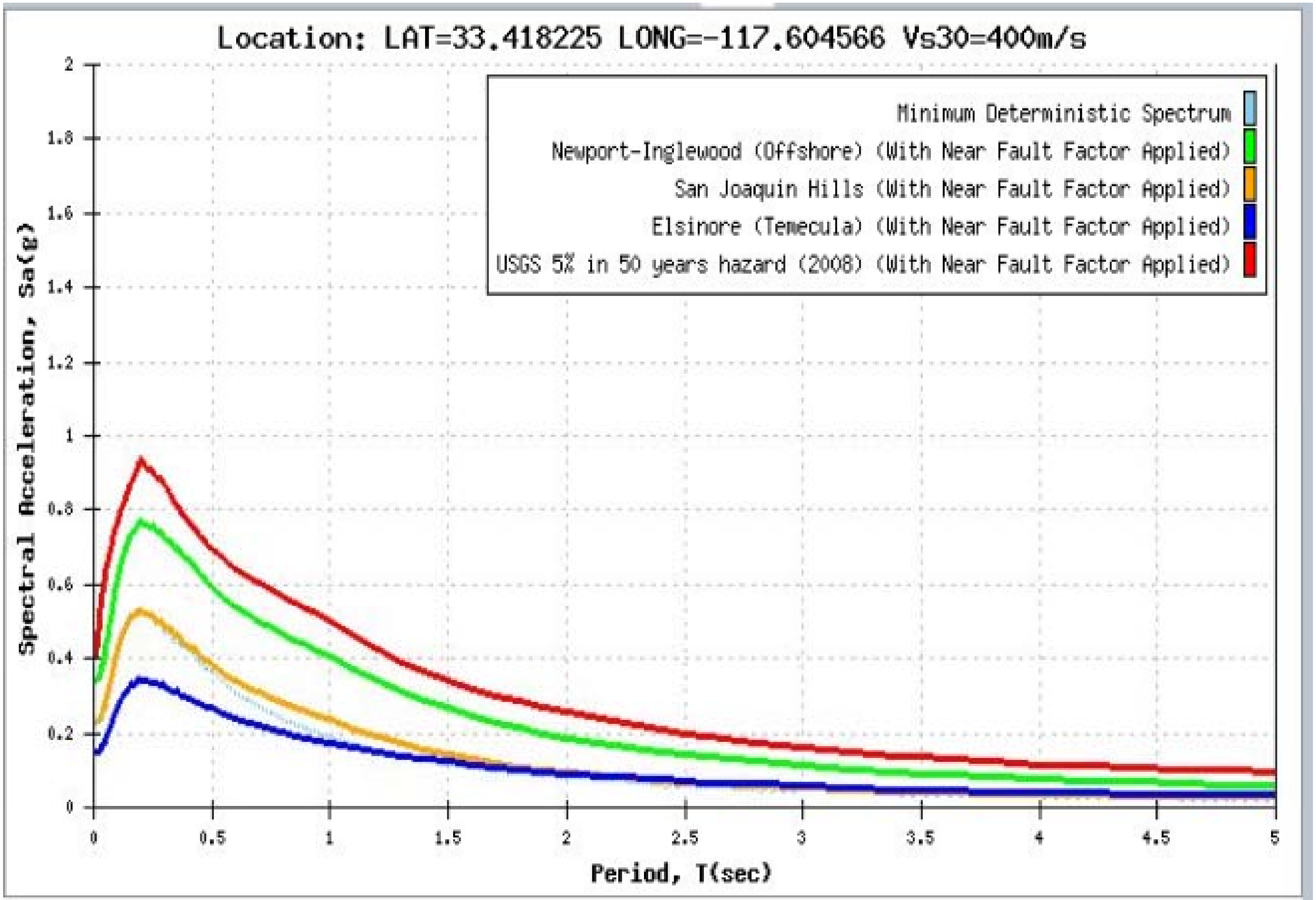
Figure 1

El Camino Real UC - Project Plan

Figure 2

El Camino Real UC - Boring Location Plan

Figure 3



Location: LAT=33.418225 LONG=-117.604566 Vs30=400m/s
Minimum Deterministic Spectrum
Newport-Inglewood (Offshore) (With Near Fault Factor Applied)
San Joaquin Hills (With Near Fault Factor Applied)
Elsinore (Temecula) (With Near Fault Factor Applied)
USGS 5% in 50 years hazard (2008) (With Near Fault Factor Applied)
Spectral Acceleration, Sa(g)
Period, T(sec)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5

Figure 4

M e m o r a n d u m

Serious drought. Help save water!

To: MR. KAMRAN MAZHAR, CHIEF
Design Branch "F"

Attn: Mr. Bang Hua

Date: August 27, 2015
File: 07-ORA-5-PM 1.2/2.2

Proj. ID: 1212000090
EA: 12-0M4901

OVERHEAD SIGN NOS.
203, 207, 312 & 405

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1 MS # 18

Subject: Foundation Report For Overhead Signs-Truss Single Post

A foundation investigation was conducted in July 2015 for the proposed Type VII (signs 203, 312 & 405) and Type V (Sign 207) truss single post type overhead signs. These signs are proposed as part of the El Camino Real UC widening and improvement project. Two 6" auger borings on the I-5NB (Signs 203) and I-5SB (Sign 312) shoulders, and two 4.5" mud rotary borings on the I-5NB (Sign 207) and I-5SB (Sign 405) shoulders, with SPT were drilled at the sign sites as shown on the road plans. Results will be shown on the forthcoming separate Log of Test Borings for each sign which must be included in the contract plans.

SITE GELOGY AND SUBSURFACE CONDITIONS

The proposed signs are located on a relatively flat marine/non-marine terrace deposits of early and mid-Pleistocene consisting of silty sand with clay and gravel, and gravelly sand with clay and silt. The areas including signs 203 and 207 are mostly underlain by Capistrano Formation (upper Miocene to lower Pliocene) consisting of marine, poorly consolidated and bedded siltstone, mudstone, silty and diatomaceous shale with minor weakly cemented and poorly bedded sandstone. The areas including Signs 312 and 405 are covered with late Pleistocene to early Holocene landslide deposits of broken up and weathered material, mapped to be overlying the Capistrano Formation (Geologic Map of the San Clemente 7.5' Quadrangle, California Geologic Survey 1999). The landslide material since has been graded relatively flat for city development. Based on recent boring logs, signs 203, 207, 312 & 405 are underlain by about 20-25 ft. of embankment fill consisting of mostly clayey sand and sandy clay overlain the sandstone/siltstone/mudstone of Capistrano Formation. Depth to rock-like material is estimated to be about 30 to 60 ft. (signs 203 & 207), and about 30 ft. (signs 312 & 405) below original ground.

Groundwater was not encountered during 2015 filed investigation to approximate elevation of 163.3 similar to 1966 borings. However, groundwater was encountered at elevation 184.0 ft. during 1954 field investigation. All elevations shown in this report are based on the NGVD 1929 Vertical Datum. It should be noted that ground water levels can fluctuate with the change of season and other factors including local irrigation.

FOUNDATION RECOMMENDATIONS

In order to verify that Standard Design foundation for cast-in-drilled hole (CIDH) piles of 60" diameter x 23'-0" foundation depth for Type VII, and 54" dia. x 22'-0" foundation depth for Type V, are sufficient to support the proposed signs, lateral load-deformation response was analyzed, utilizing LPILE v2012.0. The unfactored boundary condition loads at the pile head are listed in Table 2. Maximum bending moment and shear force with corresponding depths and pile-head deflection are presented in Table 3.

Table 2 – Boundary Conditions

Sign No.	Post Type No.	Axial Force (kips)	Shear Force (kips)	Bending Moment (kips-ft.)
203	VII	20.7	14.1	417
207	V	13.4	9.3	186
312	VII	18.9	11.5	320
405	VII	20.7	14.1	417

Table 3 – Maximum Bending Moment and Shear Force

Sign No.	Max. Bending Moment (in-lbs.)	Max. Shear Force (lbs.)	Depth of Max. Bending Moment Below Pile Head	Depth of Max. Shear Force Below Pile Head	Pile Head Deflection
203	5874537	65284	99 inch	212 inch	0.18 inch
207	2813992	26793	90 inch	148 inch	0.12 inch
312	4474503	56645	88 inch	218 inch	0.17 inch
405	5248903	37149	36 inch	148 inch	0.02 inch

It should be noted that depths to maximum Bending Moment and Shear Force are considered below pile head. We also assumed that pile head coincides with ground surface.

Based on information from recent borings logs and LPILE analysis, the subsurface soil within the 23'-0" and 22'-0" depths of overhead sign foundations appear to meet the minimum 30 degree angle of internal friction and 120 lb. $/ft^3$ unit weight, as required in "standard plan overhead and changeable message signs – Design Procedure" section of the October 2014 Caltrans Geotechnical Manual, and 2006 standard plans which are still applicable in 2010 standard plans, according to Mr. KC LIU. Therefore, pile depths given in the standard plans (S8, page 341) are sufficient to support the proposed overhead signs – truss, single post Types VII & V without exceeding the maximum allowable pile-head deflection. The proposed Types VII and V overhead signs may be supported by a standard plan S8 for SPT blow counts of 15. Pile diameter is 5'-0", 23'-0" length for Type VII, and 4'-6", 22'-0" length for Type V.

The recommendations and comments contained in this report are based on specific project information that has been provided by District 12 Design Branch I. If any conceptual changes are made during final project design, our office should review the change to determine if the foundation recommendations are still applicable. It should be noted that Sign No. 205 will be supported by Retaining Wall 90 and will be included in the Foundation Report of the mentioned retaining wall.

CONSTRUCTION CONSIDERATIONS

1. The bottom of CIDH pile boring should be cleaned of loose debris before placing concrete.

2. The contractor may experience caving conditions within the sandy zones, and should be prepared to utilize stabilizing methods, such as temporary casing to keep the hole open during CIDH pile construction.

3. It is recommended that drilling of the pile boring, placement of rebar cage and concrete take place in the same day.

Any questions regarding this report should be directed to Faramarz Gerami at 213-620-2149.

Prepared by:

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Office of Geotechnical Design - South 1
Branch C

Supervised by: Date: 8/27/2015

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Reviewed by:

SAMUEL SUKIASIAN, P.E., G.E
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c: Project Manager – ahmed.abou-abdou@dot.ca.gov
Project Engineer – trang.luong@dot.ca.gov; bang.hua@dot.ca.gov
Materials Engineer – behdad.baseghi@dot.ca.gov
Geotechnical Archive – http://svgcgeodog.dot.ca.gov/

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: Kamran Mazhar, Chief
Design Branch F

Date:
November 12, 2015

File:
Dist-County Route 12-Ora-5
PM: 1.2/2.2
EA No. 0M4901
1212000090

Reza Aurasteh

From: Reza Aurasteh, Chief
Environmental Engineering Branch

Subject: **Aerial Deposited Lead (ADL) SSP, Realignment and Increase of Existing Mainline Horizontal Curvature at El Camino Real along Route I-5, City of San Clemente, Orange County, CA**

We have received the project description/layout and request for the preparation of ADL SSP from your branch. Accordingly, we have conducted extending research of all projects and found out the historical ADL sampling data (EA No: 0C8701) that could be applied to this project.

Based on the previous testing results shown at the borehole numbers BH24 and BH26, ADL is present in the soil and the average lead concentrations are below 1,000 mg/kg total lead and below 5 mg/L soluble lead that is considered to be non-hazardous waste.

Therefore, the excavated soil contained ADL at the project site considers to be non-hazardous waste and can be reused onsite without any restriction. The attached is SSP for the reuse of ADL contained soil onsite (no offsite disposal). If the offsite disposal is needed, please let us know so SSP could be revised.

According to SSP, the contractor needs to prepare a lead compliance plan. Please include a bid item for this plan. In addition, please submit a layout plan to show the contractor how to reuse ADL contained soil onsite for our Branch's review.

If you have any question regarding this review, feel free to call Wayne Chiou at 949-724-2221.

Cc: Wayne Chiou, Environmental Engineering Branch
Trang Luong, Design Branch F

Enclosure: ADL SSP 7-1.02K(6)(j)(iii)_A01-18-13

STATE OF CALIFORNIA - NATURAL RESOURCES AGENCY

EDMUND G. BROWN, JR., *GOVERNOR*

CALIFORNIA COASTAL COMMISSION
SOUTH COAST DISTRICT OFFICE
200 OCEANGATE, 10TH FLOOR
LONG BEACH, CALIFORNIA 90802-4416
PH (562) 590-5071 FAX (562) 590-5084
WWW.COASTAL.CA.GOV

February 1, 2016

Coastal Development Permit De Minimis Waiver
Coastal Act Section 30624.7

Based on the project plans and information provided in your permit application for the development described below, the Executive Director of the Coastal Commission hereby waives the requirement for a Coastal Development Permit pursuant to Section 13238.1, Title 14, California Code of Regulations. If, at a later date, this information is found to be incorrect or the plans revised, this decision will become invalid; and, any development occurring must cease until a coastal development permit is obtained or any discrepancy is resolved in writing.

Waiver: 5-16-0064-W

Applicant: California Department of Transportation (CalTrans) - District 12

Location: I-5 freeway between postmile 1.2 and postmile 2.2 (just north of Avenida Menocino on ramp to just south of Avenida Presidio undercrossing), San Clemente, Orange County

Proposed Development: Re-align and re-stripe I-5 freeway between postmile 1.2 and postmile 2.2, including construction of new approach and departure slabs and replacement of retaining walls and sound walls at east side of freeway near El Camino Real overcrossing, replacement of five overhead directional signs, replacement of three trees, and replacement of 4,000 square feet of groundcover vegetation.

Rationale: The proposed freeway improvement project is entirely within CalTrans right-of-way, approximately 1/3 mile inland of the public beach. The majority of the proposed development is in the vicinity of the El Camino Real overcrossing, which is outside the Coastal Zone and therefore exempt from coastal development permit requirements. Proposed development within the coastal zone consists of lane re-striping and re-construction of the center median between postmile 1.2 and postmile 1.5. Construction will begin after Labor Day and cease prior to Memorial Day in order to avoid peak beach use periods. Tree replacement will occur outside of bird nesting season. CalTrans determined that the proposed project was categorically exempt from CEQA requirements on March 21, 2013. The proposed development will not adversely impact coastal resources, public access, or public recreation opportunities, and is consistent with past Commission actions in the area and Chapter Three policies of the Coastal Act.

This waiver will not become effective until reported to the Commission at their February 10-12, 2016 meeting and the site of the proposed development has been appropriately noticed, pursuant to 13054(b) of the California Code of Regulations. The Notice of Pending Permit shall remain posted at the site until the waiver has been validated and no less than seven days prior to the Commission hearing. If four (4) Commissioners object to this waiver of permit requirements, a coastal development permit will be required.

Charles Lester,
Executive Director

ZRehm
Zach Rehm
Coastal Program Analyst

cc: File

CALIFORNIA COASTAL COMMISSION
SOUTH COAST DISTRICT OFFICE
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NOTICE OF PERMIT WAIVER EFFECTIVENESS

February 26, 2016

To: California Department of Transportation (CalTrans) - District 12

From: Teresa Henry, District Manager
Zach Rehm, Coastal Program Analyst

Subject: Coastal Development Permit (CDP) Waiver 5-16-0064-W

Please note that CDP Waiver 5-16-0064-W was reported to the California Coastal Commission on February 11, 2016 and became effective as of that date. CDP Waiver 5-16-0064-W allows for:

Re-align and re-stripe I-5 freeway between postmile 1.2 and postmile 2.2, including construction of new approach and departure slabs and replacement of retaining walls and sound walls at east side of freeway near El Camino Real overcrossing, replacement of five overhead directional signs; replacement of three trees, and replacement of 4,000 square feet of groundcover vegetation.

At: I-5 freeway between postmile .12 and postmile 2.2 (just north of Avenida Menocino on ramp to just south of Avenida Presidio undercrossing), San Clemente, Orange County (APN(s):)

Please be advised that CDP Waiver 5-16-0064-W only authorizes the development as proposed and described in the Commission's files; any changes to the proposed and described project may require a CDP to account for the changes or a CDP for the entire project. If you have any questions, please contact Zach Rehm in the South Coast District Office at the address and phone number above.

Sincerely,
John Ainsworth
Senior Deputy Director

ZRehm
Zach Rehm
Coastal Program Analyst

cc: City of San Clemente, CCC file, Agent