FINAL FOUNDATION REPORT

Overhead Sign Structures
Interstate 5 HOV Improvement Project (Segment 3)
PCH to San Juan Creek Road
Orange County, California
12-ORA-5, PM 6.2/8.7
Caltrans Project No. 1200020279 (EA 12-0F96E1)
EMI Project No. 11-137
Date: November 9, 2012

EARTH MECHANICS, INC.

Geotechnical and Earthquake Engineering

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

November 9, 2012

EMI Project No. 11-137

TRC Solutions, Inc.
123 Technology Drive West
Irvine, California 92618

Attention: Dr. Ayman Salama, P.E.

Subject: Final Foundation Report for Overhead Sign Structures
Interstate 5 HOV Improvement Project (Segment 3)
PCH to San Juan Creek Road
Orange County, California
12-ORA-5, PM 6.2/8.7
Caltrans Project No. 1200020279 (EA 12-0F96E1)

Dear Dr. Salama:

Attached please find the Final Foundation Report for the Overhead Sign Structures. This report contains the findings and conclusions of our field investigation and laboratory testing program. This report also contains our recommendations for the design and construction of the overhead sign structure foundations.

Please submit this report to Caltrans and other participating agencies for review. All review comments and approved responses will be incorporated into a final report later.

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

Sincerely,

EARTH MECHANICS, INC.

4/9/13

(Raja) S. Pirathiviraj, GE 2693
Senior Staff Engineer

11-9-12

Lino Cheang, GE 2345
Project Manager

SP/sp,lcc

FINAL FOUNDATION REPORT

OVERHEAD SIGN STRUCTURES
INTERSTATE 5 (HOV) IMPROVEMENT PROJECT (SEGMENT 3)
PCH TO SAN JUAN CREEK ROAD
ORANGE COUNTY, CALIFORNIA
12-ORA-5, PM 6.2/8.7
CALTRANS PROJECT NO. 1200020279 (EA 12-0F96E1)

Prepared for:

TRC Solutions, Inc.
123 Technology Drive West
Irvine, California 92618

Prepared By:

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

EMI Project No. 11-137

November 9, 2012

TABLE OF CONTENTS

Section Page

1.0 INTRODUCTION 1

1.1 Purpose and Scope of Study 1
1.2 Project Description 1

2.0 FIELD INVESTIGATION AND LABORATORY TESTING 3

2.1 Field Investigation 3
2.2 Laboratory Testing 4

3.0 GEOLOGY AND SEISMICITY 5

3.1 Physiography 5
3.2 Geologic Structure 5
3.3 Geologic Hazard 5
3.4 Seismicity 8

4.0 SUBSURFACE CONDITIONS 12

4.1 Soil Conditions 12
4.2 Groundwater Conditions 13

5.0 CONCLUSIONS AND RECOMMENDATIONS 14

5.1 Seismic Design 14
5.2 Liquefaction 14
5.3 Soil Corrosivity 15
5.4 Foundation Design 15
5.4.1 Pile Foundation Demand 15
5.4.2 Axial Pile Analysis 17
5.4.3 Lateral Pile Analysis 17
5.4.4 Torsional Capacity 18
5.4.5 Settlement 18
5.4.6 Global Slope Stability 18

6.0 CONSTRUCTION RECOMMENDATIONS 19

6.1 Earthwork 19
6.2 CIDH Pile Construction 19
6.3 Review of Construction Plans 20
6.4 Geotechnical Observation and Testing 20

7.0 LIMITATIONS 21

8.0 REFERENCES 22

LIST OF TABLES

Table 2-1. Geotechnical Exploration Information 3
Table 4-1. Idealized Soil Profile and Strength Parameters 12
Table 5-1. Summary of Liquefaction Analyses 14
Table 5-2. Soil Corrosion Test Results 15
Table 5-3. Foundation Data for Sign Structures 16
Table 5-4. Foundation Demands for Sign Structures 17
Table 5-5. Results of Lateral Pile Analysis 18

LIST OF FIGURES

Figure 1-1. Site Location Map 2
Figure 3-1. Regional Map of Active Faults and Physiography 6
Figure 3-2. Map of Liquefaction Potential 7
Figure 3-3. Geological Map 9

APPENDICES

Appendix A. Log of Test Boring Sheets
Appendix B. Laboratory Test Results
Appendix C. Design Calculations

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF STUDY

This Foundation Report presents the findings and conclusions of a geotechnical investigation conducted by Earth Mechanics, Inc. (EMI). It also presents foundation evaluation, design and construction recommendations for the proposed Overhead Sign Structures of the Interstate 5 (I-5) High-Occupancy Vehicle Lanes (HOV) Improvement Project from Pacific Coast Highway (PCH) to San Juan Creek Road located in County of Orange, California. A site location map is presented in Figure 1-1.

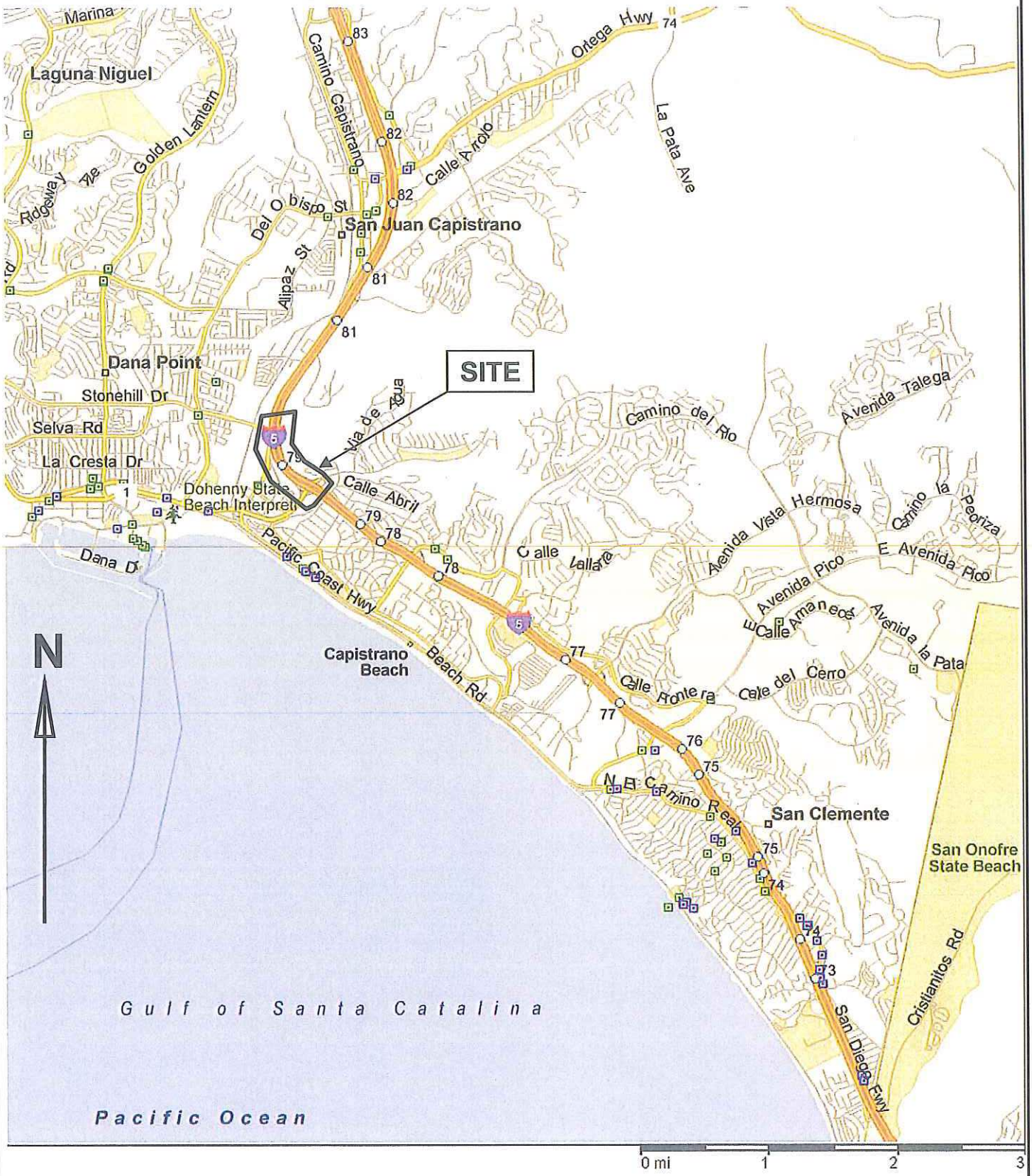
EMI is a subconsultant to TRC Solutions, Inc. (TRC). The geotechnical services provided for this project included the following tasks:

- Field exploration consisting of drilling and logging exploratory borings and performing cone penetration test soundings;
- Laboratory testing of selected bulk and relatively undisturbed soil samples;
- Engineering calculations and analysis to develop foundation design and construction recommendations; and
- Preparation of this report presenting our findings, conclusions, and recommendations.

1.2 PROJECT DESCRIPTION

The California Department of Transportation (Caltrans), in cooperation with the Orange County Transportation Authority (OCTA) and the cities of Dana Point and San Juan Capistrano, proposes to improve the I-5 corridor. The project's southern terminus is approximately 0.6 mile south of the PCH/Camino Las Ramblas Interchange (I-5 Mainline Station 340+00) in the City of Dana Point. The northern terminus of the project is approximately 0.2 mile south of San Juan Creek Road (I-5 Mainline Station 465+00) in the City of San Juan Capistrano. The total project length along the I-5 corridor is approximately 2.5 miles. However, according to the Project Report prepared in October 2011, the proposed project improves the I-5 corridor from I-5 Mainline Stations 340+00 (southern project limit) to 407+50. From I-5 Mainline Stations 407+50 to 465+00 (northern project limit), the proposed project will consist primarily of restriping the existing pavements only.

This foundation report pertains to the proposed Overhead Sign Structures only. A total of six Overhead Sign Structures on isolated and individual foundations are proposed within the project limits.



I-5 HOV Improvement Project
PCH to San Juan Creek Road

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

SITE LOCATION MAP

Figure 1-1

Project No. 11-137

Date: 07-09-2012

2.0 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 FIELD INVESTIGATION

A geotechnical field investigation was conducted between September 26 and October 17, 2011. Four soil borings and four cone penetration test (CPT) soundings were performed for the proposed Overhead Sign Structures. Boring information, including surveyed locations and elevations, are summarized in Table 2-1. Locations of the borings and CPT soundings are shown on the LOTB sheets provided in Appendix A.

Table 2-1. Geotechnical Exploration Information

Boring/ CPT	Easting	Northing	Station (A-Line) (feet)	Offset (feet)	Top of Boring El. (feet)	Bottom of Boring El. (feet)	Ground Water El. (feet)	Drilling Method
A-11-303	6,130,944	2,114,821	340+56	92 Rt	+194.8	+163.3	+176.3	HSA
A-11-304	6,130,588	2,115,147	345+39	112 Rt	+196.9	+145.4	+159.4	HSA
CPT-11-306	6,130,115	2,115,554	351+62	119 Rt	+200.5	+186.3	NM	CPT
CPT-11-325	6,128,605	2,116,429	369+12	109 Lt	+148.7	+83.7	NM	CPT
CPT-11-332	6,130,323	2,115,101	347+11	93 Lt	+198.5	+190.8	NM	CPT
A-11-342	6,127,523	2,117,230	382+22	114 Lt	+147.6	+106.1	NE	HSA
A-11-350	6,127,160	2,119,046	400+46	75 Lt	+113.1	+12.5	+42.4	HSA
CPT-11-351	6,127,209	2,119,341	403+38	73 Lt	+110.4	+63.4	NM	CPT

Notes:
(1) A-Line = I-5 Mainline; NE = Not Encountered; NM = Not Measured.
(2) CPT = Cone Penetration Test; HSA = Hollow-Stem Auger.

The HSA borings were drilled using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Sampling was performed by alternating the Modified California Drive (MCD) sampler and Standard Penetration Test (SPT) sampler. The soil sampling interval was generally 5 feet.

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

The CPT sounding was performed using an electronic cone penetrometer in general accordance with current ASTM Standards (ASTM D5778 and ASTM D3441). The CPT equipment consisted of a cone penetrometer assembly mounted at the end of a series of hollow sounding rods. The cone penetrometer assembly consisted of a conical tip with a 60° apex angle and a projected cross sectional area of 1.55 in² (10 cm²) and a cylindrical friction sleeve with a surface area of 23.25 in² (150 cm²). The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurements of cone tip and friction sleeve resistance during penetration. The cone penetrometer assembly is continuously pushed into the soil by a set of hydraulic rams at a standard rate of 0.79 inch per second (20 mm per second) while the cone tip resistance and sleeve friction resistance are recorded every 1.967 inches (50 mm) and stored in digital form. A specially designed all-wheel drive 25-ton truck provides the required reaction weight for pushing the cone assembly and is also used to transport and house the testing equipment. The computer generated graphical logs include tip resistance, friction resistance, and friction ratio. Soil behavior type interpretations are based on guidelines by Robertson and Campanella (1989).

2.2 LABORATORY TESTING

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

- In-situ Moisture Content and Unit Weight
- Percent Passing No. 200 Sieve
- Direct Shear
- Unconsolidated Undrained Triaxial
- Consolidation
- Minimum Resistivity, pH, Sulfate Content and Chloride Content

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

3.0 GEOLOGY AND SEISMICITY

3.1 PHYSIOGRAPHY

The project area is in the northwestern part of the Peninsular Ranges physiographic province. The Peninsular Ranges comprise a northwest-southeast trending group of fault-bounded ranges between the Salton Trough and the Pacific Ocean. The Santa Ana Mountains, Puente Hills, and San Joaquin Hills are ranges within the Peninsular Ranges.

The site is located on the low lying rolling hills westerly of the Santa Ana Mountains at the southerly end of the San Joaquin Hills in an area referred to as the Capistrano Embayment.

3.2 GEOLOGIC STRUCTURE

The geological structure at the site consists of slightly to moderately folded bedrock of the Capistrano formation overlain by horizontally bedded Quaternary terrace deposits and alluvium without any notable geological structures such as faults, folds, or unconformities. The northerly portion of the project is underlain by deposits of the McCracken Hill landslide which is described in Section 3.3.

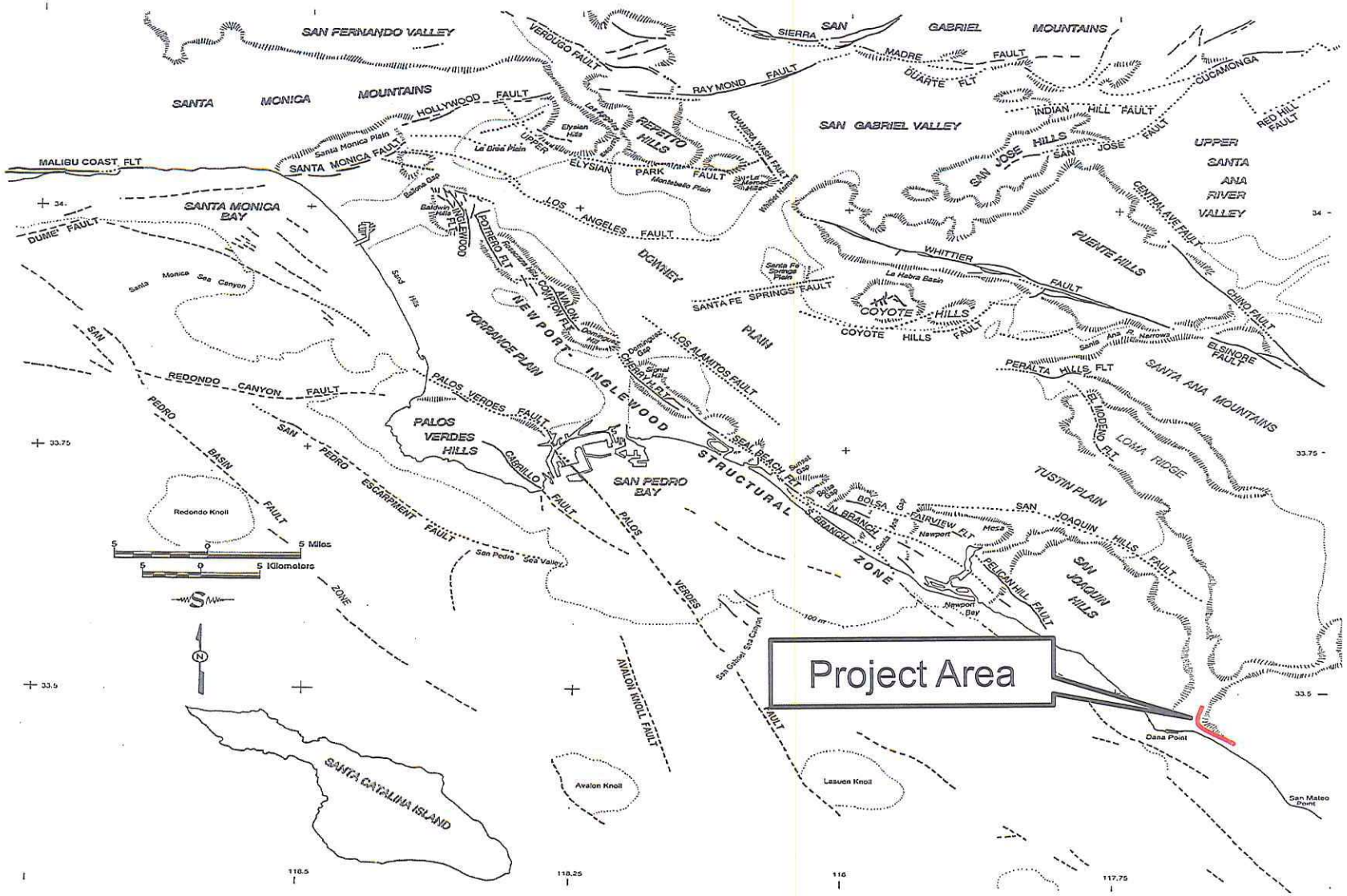
The Capistrano Formation is widespread throughout the southern part of Orange County, which is known geologically as the Capistrano Embayment. The Capistrano Embayment is the name given to the structural/stratigraphic block west of the Cristianitos Fault. Geologic faults in the region are shown on Figure 3-1. Geological structure in the Capistrano Embayment area consists primarily of a broad, gentle syncline of the Monterey and Capistrano Formations between the San Joaquin Hills and the Santa Ana Mountains. This structure originated as a deep submarine structural trough that has since been uplifted at least 3000 feet from the marine environment to its present position above sea level (Ehlig, 1989). Subsequent regional uplift during the late Pliocene and Pleistocene time resulted in folding of the bedrock units.

The bedrock of the Capistrano Formation underlying the site was found to be massive to poorly bedded. Where bedding was observed, the strikes were generally to the northeast with shallow dips less than about 10 degrees westerly. Throughout the project corridor, the dominant structural pattern is high-angle joints and fractures within the Capistrano bedrock.

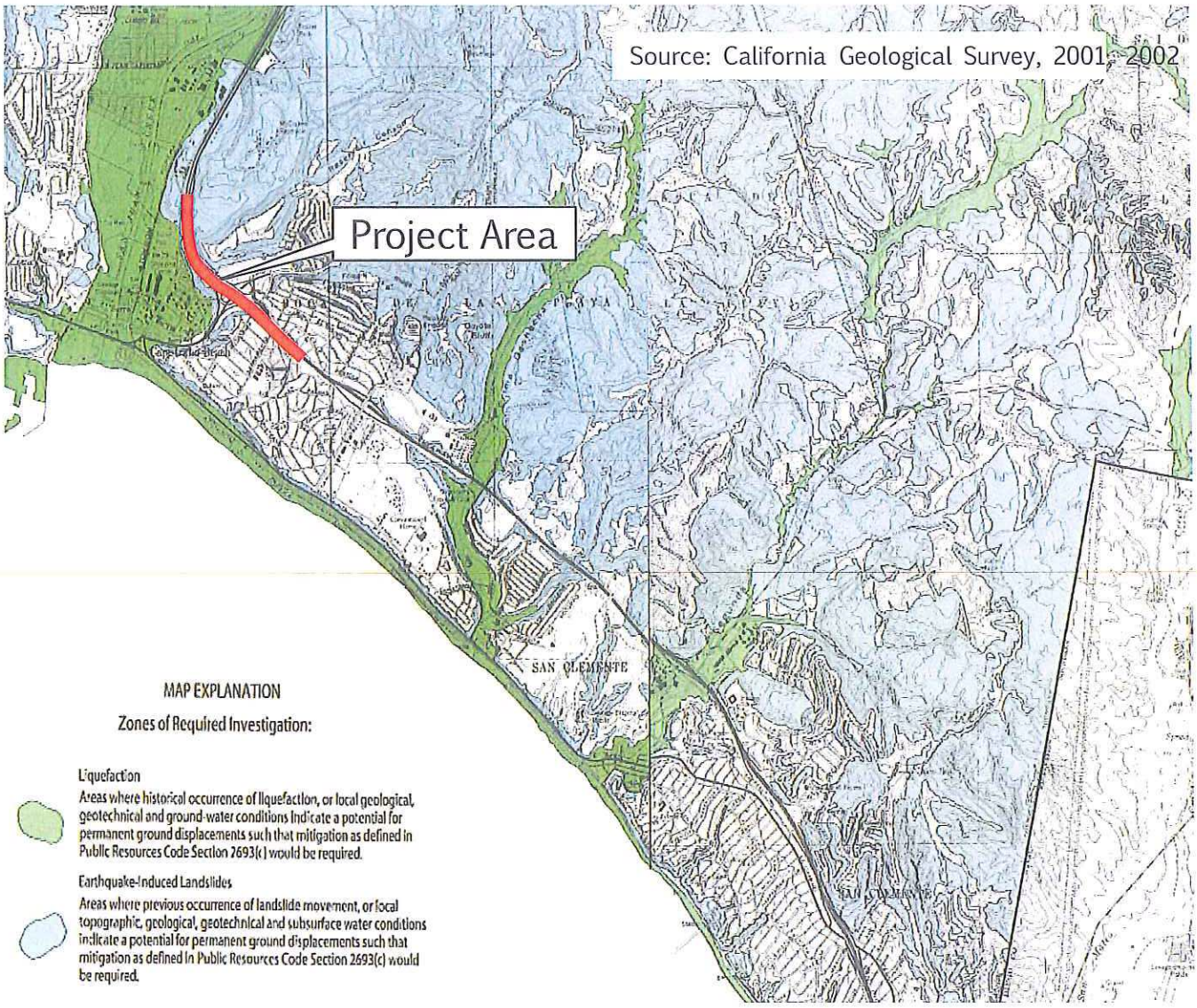
Pleistocene Terrace Deposits unconformaly overlie the Capistrano Formation. The contact is generally undulatory, with a slight overall dip seaward averaging about 2 degrees. This erosional contact is marked by cobble and boulder rich beds of varying thicknesses.

3.3 GEOLOGIC HAZARD

The geological hazards present at the site include earthquake shaking and landsliding. The site lies outside identified tsunami inundation zones (CGS, 2009), and there are no large bodies of water within the site area that could generate a seiche. There are no volcanos in the region and there are no known active surface faults within the project area so ground rupture is not a factor. As shown in Figure 3-2, the California Geological Survey (CGS, 2001a) has indicated that the project alignment has a low susceptibility to liquefaction during a strong earthquake. The potential for liquefaction is discussed in detail in Section 5.2.



I-5 HOV Improvement Project (Segment 3) PCH to San Juan Creek Road	REGIONAL MAP OF ACTIVE FAULTS AND PHYSIOGRAPHY Figure 3-1
Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Project No. 11-137 / Date: 07-09-12



I-5 HOV Improvement Project (Segment 3) PCH to San Juan Creek Road	MAP OF LIQUEFACTION POTENTIAL Figure 3-2
Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Project No. 11-137 / Date: 07-09-12

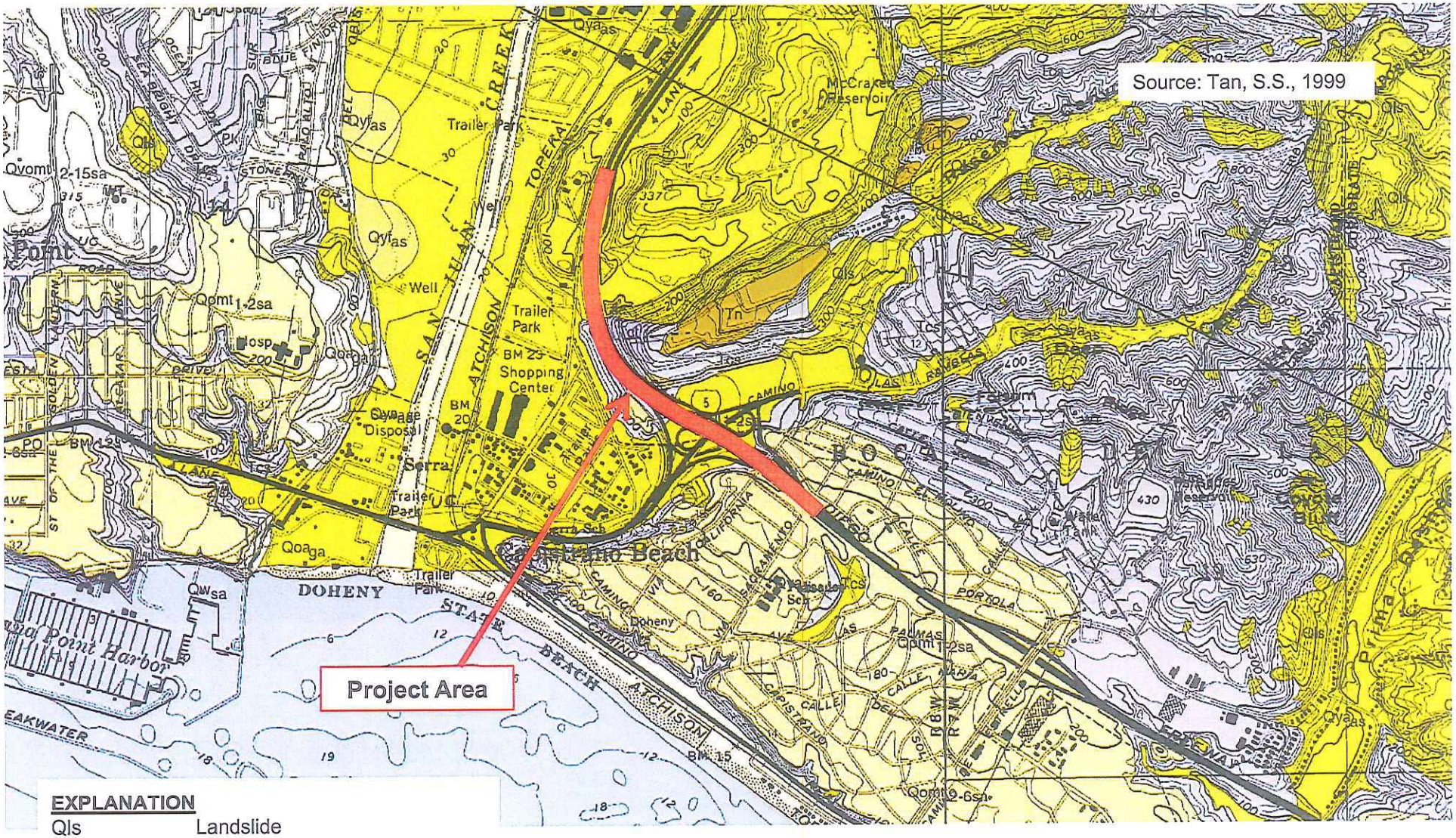
The Capistrano Formation bedrock underlying the majority of the site is notoriously susceptible to landsliding. The landslides are shown in Figure 3-3. At the north of PCH/Camino Las Ramblas Interchange, the I-5 alignment traverses the McCracken Hill Landslide. The remnant headscarp of this ancient landslide is defined by a west to northwest facing slope that extends for about 7000 feet on the eastern side of the I-5 Freeway (AMEC, 2006a and 2006b). It is believed that the landslide occurred between 11,000 to 30,000 years ago when the sea level was lower than present day and San Juan Creek carved a deep channel on its course to the Pacific Ocean. During the Pleistocene, the climate was considerably wetter than present day and this combined with the loss of support at the toe of the slope due to erosion by San Juan Creek are believed to have been causative factors in the landslide failure. Since the landslide failure, a considerable thickness of alluvium has infilled the ancestral San Juan Creek channel with deposits more than 130 feet thick forming a natural buttress for the landslide mass. Extensive investigations of the landslide performed by Leighton and Associates (Leighton, 2004) and AMEC (AMEC, 2006a, 2006b, and 2000a through 2000e) in conjunction with proposed residential developments in the area indicate that the landslide is stable.

In addition, a smaller landslide is present along the PCH NB on-ramp to I-5 SB. The slope ascending from the PCH NB on-ramp to I-5 SB is underlain by Quaternary Terrace Deposits overlying bedrock of the Capistrano Formation. A landslide measuring approximately 150 feet wide by 300 feet long was observed on this slope during field mapping. Research of files at the City of Dana Point did not reveal any geologic reports relating to the landslide or details of when it occurred. Based on the geomorphology of the slide it appears to have been a shallow failure involving the Terrace Deposits and possibly the weathered upper portion of the underlying bedrock. It is estimated the landslide is likely less than about 30 feet deep. Proposed grading performed in conjunction with the project is not considered to have an impact on this landslide.

Bedrock exposures along Via Canon (southwesterly of the landslide, outside the limits of the geologic map) indicate that the bedding is variable but generally dips south to southeast at angles ranging from 5 to 22 degrees. This bedding orientation is generally considered favorable to the gross stability of the slope.

3.4 SEISMICITY

The project is in seismically active Southern California. The present-day seismotectonic stress field in the Los Angeles region is one of north-northeasterly compression. This is indicated by the geologic structures, earthquake focal-mechanism solutions, and geodetic measurements. These data suggest crustal shortening of between 0.2 and 0.35 inch per year across the greater Los Angeles area (Argus et al., 1999).



EXPLANATION

Qls	Landslide
Qya, Qoa	Alluvium
Qomt	Marine Terrace Deposits
Tcs, Tct	Capistrano Formation

I-5 HOV Improvement Project (Segment 3)
PCH to San Juan Creek Road

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

GEOLOGICAL MAP

Figure 3-3

Project No. 11-137

Date: 07-09-12

Historical earthquake epicenter maps show widespread seismicity throughout the region. Although historical earthquakes occur in proximity to known faults, they are difficult to directly associate with mapped faults. Part of this difficulty is due to the fact that the basin is underlain by several poorly known subsurface thrust faults, generally referred to as blind thrust faults. Ward (1994) estimated that about 40 percent of seismic moment cannot be associated with known faults. Earthquakes occur primarily as loose clusters along the Newport-Inglewood Structural Zone (NISZ), the southern margin of the Santa Monica Mountains, the margin between the Santa Susana-San Fernando Valley and the southern margin of the San Gabriel Mountains, and in the Coyote Hills-Puente Hills area. There is no clustering or alignment of earthquakes in proximity to the site. There are fewer earthquakes in the site region than anywhere else in the Los Angeles Basin area. This apparent lack of earthquake activity suggests that the site area is tectonically stable and suggests that there are no unrecognized active faults at the site.

The largest historical earthquake within the Los Angeles Basin was the 1933 Long Beach event which had a moment magnitude (M_W) of about 6.4 (M_L = 6.3). This earthquake did not rupture the surface but is believed to have been associated with the NISZ (Benioff, 1938). The association was based on abundant ground failures along the NISZ trend but no unequivocal surface rupture was identified. Reevaluation of the seismicity data by Hauksson and Gross (1991) relocated the 1933 earthquake hypocenter to a depth of about 6 miles below the Huntington Beach-Newport Beach city boundary.

Other major earthquakes in the region include the 1994 Northridge and the 1971 San Fernando earthquake both of which occurred in the San Fernando Valley region. The 1994 earthquake had a M_W of about 6.7 (M_S = 6.8, M_L = 6.4), and occurred on a southerly dipping subsurface fault which was unknown prior to the earthquake. The main shock occurred at a depth of about 12 miles. Earthquake aftershocks clearly defined the rupture surface dipping about 35 degrees southerly from a depth of about 1.2 or 1.9 miles to 14 miles (Hauksson et al, 1995). The causative fault was never identified with certainty. The event may have occurred on an eastern extension of the Oakridge fault (Yeats and Huftile, 1995), a southerly dipping feature fault bounding the Ventura Basin and the Santa Susana Mountains.

The 1971 San Fernando earthquake was of similar size (M_W = 6.7, M_S = 6.4, M_L = 6.4) to the 1994 event but did involve surface rupture. The 1971 event occurred on a northerly dipping thrust fault that dips from the northern side of the San Fernando Valley to a depth of about 9 miles under the San Gabriel Mountains. Several mapped surface faults were involved such as the Sylmar fault, Tujunga fault, and Lakeview fault. These faults are commonly considered to be part of the Sierra Madre fault system which extends easterly from the San Fernando Valley, along the base of the San Gabriel Mountains on the north side of the San Gabriel Valley, and to the Cucamonga fault in the San Bernardino area.

The 1987 Whittier earthquake (M_L = 5.9, M_W = 5.9) occurred on a subsurface fault dipping under the Puente Hills to about 10 miles beneath the San Gabriel Basin (Shaw and Shearer, 1999; Shaw et al., 2002). This event did not rupture the ground surface.

A magnitude 5.4 earthquake occurred at a depth of about 9 miles on 29 July 2008. The epicenter was in the Chino Hills area between the Chino fault and the Whittier fault. Preliminary data were inconclusive with regard to the causative fault. Detailed analysis by Shao and Hauksson (2009)

indicated a rupture plane striking N71°W, dipping 62 degrees northeast. They suggested a preference for the Whittier fault being the causative fault but were uncertain, primarily because the Whittier fault is supposed to dip at about 80 degrees. The aftershock pattern formed a subhorizontal alignment indicating the possibility that the event could have been associated with a subsurface thrust fault such as one of the blind faults of the Puente Hills Blind Thrust Fault System or with a blind fault under the Peralta Hills.

Another significant earthquake was the 1812 earthquake which caused damage at the San Juan Capistrano Mission. The location and magnitude of the 1812 earthquake are unknown because of the sparse population at the time, but geological studies (Jacoby et al., 1987; Fumal et al., 1993; Weldon et al., 2004) postulate that it did not occur in the Capistrano area, but rather was a large ($M_W > 7.0$) distant event on the San Andreas fault in the Wrightwood area of the San Gabriel Mountains.

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

4.0 SUBSURFACE CONDITIONS

4.1 SOIL CONDITIONS

The idealized soil profile and design strength parameters for overhead sign foundation design are presented in Table 4-1. The strength parameters for the fine-grained soils in Table 4-1 were obtained from laboratory test results and CPT data (Robertson and Campanella, 1989). The strength parameters for the silty sand layer in Table 4-1 were based on correlations with SPT blowcounts (Lam and Martin, 1986).

Table 4-1. Idealized Soil Profile and Strength Parameters

Approximate Elevation (feet)	Predominant Soil Type	Equivalent SPT Blowcount* (blows/foot)	Total Unit Weight (pcf)	Friction Angle (degree)	Cohesion / Undrained Shear Strength (psf)
Overhead Sign Structure SS1-1 (Borings A-11-303 and A-11-304)					
+198 to +188	Sand with Gravel	15 to (40) Average = 31	120	34	100
+188 to +165	Lean Clay	(6) to (29) Average = 11	120	-	2000
Overhead Sign Structure SS1-3 (CPT-11-332)					
+200 to +190	Sand	-	120	34	100
+190 to +165	Bedrock	-	120	-	3000
Overhead Sign Structure SS1-4 (CPT-11-306)					
+200 to +186	Sand	-	120	34	100
+186 to +165	Bedrock	-	120	-	3000
Overhead Sign Structure SS3-1 (CPT-11-325)					
+150 to +84	Lean Clay	-	120	-	2000
Overhead Sign Structure SS5-4 (Boring A-11-342)					
+142 to +132	Lean Clay	13 to 27 Average = 20	120	-	2500
+132 to +122	Sand	21 to 27 Average = 24	120	33	100
+122 to +115	Lean Clay	12	120	-	2000
Overhead Sign Structure SS7-1 (Boring A-11-350 and CPT-11-351)					
+110 to +45	Lean Clay	5 to (16) Average = 10	120	-	1700

* Values in ( ) are converted SPT blowcounts corrected for sampler size; correction factor from Modified California Drive sampler blowcounts to SPT blowcounts is 0.5.

Within the project limits, undrained shear strengths of the bedrock obtained from all the laboratory UU-tests range from 7,900 to 10,300 psf. Undrained shear strengths of the bedrock obtained from all the CPT correlations vary from 2,000 to over 10,000 psf. Based on this, it appears the laboratory measured shear strength is close to the average shear strength obtained from the CPT correlation. A closer examination of the CPT interpreted logs show a majority of the undrained shear strength values immediately below the bedrock contact varies from 2,000 to 6,000 psf. Conservatively, an undrained shear strength value of 3,000 psf was selected for the bedrock.

4.2 GROUNDWATER CONDITIONS

The groundwater encountered during the recent field investigation is presented in Table 2-1. Groundwater was encountered near two overhead sign structures.

Based on the recent and past field investigations within the project limit, groundwater data appears to indicate that the groundwater depth is deeper (with respect to the existing freeway surface) in the north end as compared to the south end of the project limits. The groundwater encountered during EMI investigation in September and October of 2011 appears to be perched water because at several locations, where side-by-side borings are less than 50 feet apart, groundwater was only encountered in one of the borings. Nevertheless, groundwater during construction will likely be different from those reported above because groundwater level can fluctuate due to variations in seasonal precipitation, irrigation, groundwater injection or extraction, or numerous other man-made and natural influences.

Based on California Geological Survey, Division of Mines and Geology (CGS, 2001b), the highest historical groundwater at the project site is 5 to 10 feet below the ground surface. So, there is a major discrepancy between the CGS data and measured groundwater data described above. Based on our past experience, the CGS historical high groundwater data is often based on limited data and may or may not reflect actual site conditions. We believe the historical high groundwater data reported by CGS is likely to be either a perched water zone or the original ground surface is significantly lower than the current ground surface. In both cases, it does not appear to be representative of the site conditions and should not be used for liquefaction assessment for this project.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 SEISMIC DESIGN

We determined the design ARS curve based on the 2010 Caltrans Seismic Design Criteria (SDC) (Caltrans, 2010a) and Geotechnical Services Design Manual (Caltrans, 2009) procedures. The peak ground acceleration (PGA) is the zero-period spectral acceleration in the ARS curve. A PGA of 0.4g is obtained from the design ARS curve.

5.2 LIQUEFACTION

Liquefaction analysis was performed using the available site-specific subsurface information. As discussed in Section 4.2, there is a major discrepancy between CGS groundwater data and measured groundwater data. Therefore, conservatively, a groundwater depth of 5 feet below the top of borings and CPTs is used for the liquefaction analyses.

The liquefaction potential of saturated, granular materials below the groundwater table was evaluated using the procedures outlined by Seed et al. (1983), Seed and Harder (1990), and updated by NCEER (1997). Results of the liquefaction analyses are included in Appendix C and summarized in Table 5-1. Table 5-1 presents the approximate elevations of liquefiable layers, thickness of liquefiable layers, and approximate seismically-induced settlement. All liquefiable layers thinner than 0.2 feet have been omitted from Table 5-1.

Table 5-1. Summary of Liquefaction Analyses

Boring/ CPT	Top of Boring Elevation (feet)	Approximate Elevations of Liquefiable Layers (feet)	Thickness of Liquefiable Layer (feet)	Approximate Seismically Induced Settlement (inches)
A-11-303	+194.8	No Liquefaction	-	-
A-11-304	+196.9	No Liquefaction	-	-
CPT-11-325	+148.7	+137.9 to +136.2	1.7	0.3
A-11-342	+147.6	+117.6 to +112.6	5.0	1.3
A-11-350	+113.1	No Liquefaction	-	-
CPT-11-351	+110.4	No Liquefaction	-	-

The 5-foot thick liquefiable layer shown above coincides with the soil sampling interval; actual thickness of the liquefiable layer can be less than 5 feet.

Based on the above observations, the liquefiable layers are relatively thin, isolated and discontinuous, and are not anticipated to impact the proposed overhead sign structures. Therefore, the liquefaction potential is concluded to be low. Furthermore, as discussed in Section 3.3, the California Geological Survey (CGS, 2001a) has also indicated that the project alignment has a low susceptibility to liquefaction during a strong earthquake.

5.3 SOIL CORROSIVITY

Three soil samples were tested for pH, minimum resistivity, soluble chloride content and soluble sulfate content. The test results are summarized in Table 5-2. Minimum resistivities were between 480 and 1,796 ohm-cm. The pH values were between 7.3 and 8.1. The soluble sulfate measurements were between 256 and 1,720 parts per million (ppm), and the soluble chloride measurements were between 120 and 644 ppm.

Table 5-2. Soil Corrosion Test Results

Boring	Location (A-Line Stations)		Sample Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Soluble Sulfate Content (ppm)	Soluble Chloride Content (ppm)
	Station (feet)	Offset (feet)						
A-11-303	340+56	92 Rt	0-5.0	SM	1,796	7.9	256	240
A-11-304	345+39	112 Rt	2.5	CL	480	8.1	800	644
A-11-342	382+22	114 Lt	0-5.0	SM	1,055	7.3	1,720	120

In addition to the above test results, 19 soil corrosion tests were performed on similar soils and those test results are included in the Materials Report for this project (EMI, 2012).

Based on the Caltrans Corrosion Guidelines (2003b), soils are considered corrosive if the pH is 5.5 or less, or the sulfate concentration is 2,000 ppm or greater, or the chloride concentration is 500 ppm or greater. Based on the test results and the Caltrans criteria, the on-site soils are considered to be corrosive to bare metals and concrete.

For the above measurements and the test results included in the Materials Report (EMI, 2012), minimum concrete cover over reinforcement should be in accordance with Table 8.22.1 of the Caltrans BDS (Caltrans, 2003a) for "Corrosive soil above MLLW level with chloride concentration between 500 and 5,000 ppm". Cement type should be in accordance with Table 8.22.2 of the Caltrans BDS (Caltrans, 2003a) for "Sulfate Concentration from 1,500 to 1,999". Additional corrosion protection requirements for concrete structural members are presented in Section 8.22 of the Caltrans BDS (Caltrans, 2003a).

5.4 FOUNDATION DESIGN

Total of six overhead sign structures on isolated foundations are proposed within the project limit. The overhead sign structure pertinent data, as provided by the structural engineer, are presented in Table 5-3. Foundation design recommendations for the proposed overhead sign structures are provided in the following sections.

5.4.1 Pile Foundation Demand

The proposed overhead sign structures are supported on a single Cast-in-Drilled-Hole (CIDH) pile. The diameters and depths of the CIDH piles are selected from Caltrans Standard Plan sheets (2010b) based on the type of sign posts and the shape of the pedestals. The sign post types and foundation informations are provided in Table 5-3.

Table 5-3. Foundation Data for Sign Structures

Sign Structure ID	Mainline Location	Station (feet)	Sign Structure Type	Sign Post Type	CIDH Pile Diameter (feet)	Foundation Depth (feet)	2010 Standard Plan Sheet No.
SS1-1	I-5 Northbound	342+08 (A-Line)	Two-Post Sign Bridge with Cantilever	VI-S	5.0	27'-10"	S14
SS1-3	I-5 Southbound	346+80 (A-Line)	Single-Post Cantilever	VIII	5.0	25'-0"	S7
SS1-4	I-5 Northbound Off-Ramp to PCH	351+38 (PCH-5 Line)	Single-Post Cantilever	IX	5.0	25'-0"	S7
SS3-1	PCH Northbound	29+05 (CLR-Line)	Single-Post Cantilever	VII	5.0	23'-0"	S7
SS5-4	I-5 Southbound	384+22 (A-Line)	Two-Post Sign Bridge	VI-S	5.0	27'-10"	S14
SS7-1	I-5 Southbound	401+50 (A-Line)	Single-Post Cantilever	IX	5.0	25'-0"	S7

All the sign structures have square pedestal pile foundation.

As noted in Table 5-3, the foundation diameters and depths were obtained from the 2010 Caltrans Standard Plan Sheets. These foundation diameters and depths are the same as those shown on the 2006 Caltrans Standard Plan Sheets. However, based on the comparison between the 2006 and 2010 Caltrans Standard Plan Sheets, a foundation design note was removed from the 2010 Caltrans Standard Plan Sheets.

The 2006 Caltrans Standard Plan Design Note "Foundation design is based on 2001 AASHTO article 13.6 Broms' approximate procedure assuming a cohesionless material. The angle of internal friction used is 30 degrees and unit weight of soil used is 120 lb/ft^3" is missing from the 2010 Caltrans Standard Plan Sheets. Although this Design Note is missing, the 2010 Caltrans Standard Plan Sheets do indicate that this note was transferred to the Caltrans Specifications. Therefore, the Design Note should still be valid. However, as a precaution, the structural engineer should compare the 2006 and 2010 Caltrans Standard Plan Sheets to confirm that there are no changes in the structural information, details, and design notes.

The site-specific field investigation reveals that the subsurface earth materials within the "foundation depth" zone generally consist of clays and silts. These soil types do not meet the soil property requirements as described in the above Design Note. As a result, pile foundation design should be performed using the site-specific soil parameters listed in Table 4-1, the pile diameter and depth shown on the Caltrans Standard Plan Sheets S7 and S14, and foundation demands provided by the structural engineer. Single pile loads provided by the structural engineer are presented in Table 5-4.

Table 5-4. Foundation Demands for Sign Structures

Sign Structure ID	Mainline Location	Station (feet)	Pile Head Shear (kips)	Pile Head Moment (kip-ft)	Axial Compression (kips)
SS1-1	I-5 Northbound	342+08 (A-Line)	36.0	752.8	37.8
SS1-3	I-5 Southbound	346+80 (A-Line)	13.4	350.3	12.0
SS1-4	I-5 Northbound Off-Ramp to PCH	351+38 (PCH-5 Line)	16.9	488.3	12.9
SS3-1	PCH Northbound	29+05 (CLR-Line)	10.0	237.8	11.3
SS5-4	I-5 Southbound	384+22 (A-Line)	32.8	852.9	38.7
SS7-1	I-5 Southbound	401+50 (A-Line)	14.7	423.2	11.6

5.4.2 Axial Pile Analysis

Axial capacity of a single CIDH pile is estimated using the soil strength parameters presented in Table 4-1. Per the structural engineer, the maximum axial compression demand is 39 kips. Results of the axial pile analysis are included in Appendix C. Based on the results of the analysis, the foundation depths in Table 5-3 are sufficient to resist the axial demands provided by the structural engineer.

5.4.3 Lateral Pile Analysis

Since the axial demand of overhead sign structures is not significant, the design of the pile foundations is primarily controlled by lateral demand. Lateral pile analysis was performed using the computer program LPILE (Ensoft, 2010). A cracked section modulus of the CIDH pile equal to one-half the gross was used in the analysis.

The lateral pile analysis for a 5-foot diameter and 23-foot long CIDH pile was conducted using a combined maximum pile-head shear (V) of 36 kips and overturning moment (M) of 853 kip-feet under two different subsurface conditions: (1) the 2006 Caltrans Standard Plan soil condition (ϕ =30 degrees and γ=120 pcf); (2) subsurface condition at Sign "SS7-1"; this subsurface condition was selected because it has the lowest soil strength as compared to other sign structure locations. Results of the lateral pile analysis are summarized in Table 5-5 and included in Appendix C.

Based on the results of the analysis, the pile-head lateral deflections in the in-situ materials were determined to be less than deflections in the Caltrans Standard Plan soil profile (ϕ =30 degrees). This analysis was performed using the maximum combined shear and moment in Table 5-4, and the lowest in-situ soil strength in Table 4-1. As a result, the designated Caltrans Standard Plan sheets are applicable for all six of the proposed overhead sign structures.

Table 5-5. Results of Lateral Pile Analysis

Selected Subsurface Soil Condition	Maximum Pile-Head Deflection (inch)	Maximum Pile-Head Slope (rad)	Maximum Moment (kip-ft)	Depth to Maximum Moment from Pile Top (feet)	Maximum Shear (kip)	Depth to Maximum Shear from Pile Top (feet)
Caltrans Standard Plan Soil Profile (ϕ =30°)	0.40	0.003148	976	5.3	85.5	15.5
Existing Soil Profile at Sign SS7-1	0.36	0.002973	924	3.9	85.4	14.5

5.4.4 Torsional Capacity

A maximum torsional moment of 355 kip-feet for a single-post overhead sign structure was provided by the structural engineer. Based on the results of the analysis, the CIDH piles under site-specific soil conditions have adequate torsional capacities. Torsional capacity calculations are included in Appendix C.

5.4.5 Settlement

The total static settlement of the CIDH piles under static load is expected to be less than 0.25 inch and the seismically-induced settlement under the design earthquake is estimated to be negligible as discussed in Section 5.2.

5.4.6 Global Slope Stability

Based on plans provided by the structural engineer, the proposed overhead sign structures are founded on relatively flat grades except for overhead sign structure SS7-1.

The overhead sign structure SS7-1 is located on an existing slope with an inclination of about 2H:1V. Based on the global stability analyses performed for a cross-section selected near the sign structure, the calculated factor-of-safety under static and seismic conditions is greater than the required minimum values; therefore, the global slope stability of the ground supporting this overhead sign structure under static and seismic conditions are not considered to be a design issue. Results of the slope stability analyses, performed using the computer program SLIDE (Rocscience, 2006), are included in Appendix C.

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 EARTHWORK

Earthwork should be performed in accordance with Caltrans Standard Specifications, Section 19 (Caltrans, 2010c). Appropriate measures should be taken to prevent damage to adjacent structures and utilities. Any design and construction of temporary sloping, sheeting, or shoring should be made the contractor's responsibility. It should be noted that it is the responsibility of the contractor to oversee the safety of the workers in the field during construction. The contractor shall conform to all applicable occupational and health standards, rules, regulations, and orders established by the State of California. In addition, other State, County, or Municipal regulations may supersede the recommendations presented in this section. If a trench shoring design and safety plan is required, the geotechnical consultant should review the plan to confirm that recommendations presented in this report have been applied to the design.

Based on the recent and past field investigations within the project limit, groundwater data appears to indicate that the groundwater depth is deeper (with respect to the existing freeway surface) in the north end as compared to the south end of the project limits. The groundwater encountered during EMI investigation in September and October of 2011 appears to be perched water because at several locations, where side-by-side borings are less than 50 feet apart, groundwater was only encountered in one of the borings. Perched groundwater might be encountered during CIDH pile construction at the proposed overhead sign structures. Therefore, contractor should be prepared for a wet construction.

6.2 CIDH PILE CONSTRUCTION

Difficult pile installation is anticipated due to the presence of localized dense and hard soil layers, above and below the bedrock contact, caving soils and high groundwater. Construction of CIDH piles should follow Section 49-3 of the Caltrans Standard Specifications (2010c). Loose soils should be cleaned from the bottom of the borings. Pile borings should be inspected and approved by the geotechnical engineer prior to the installation of reinforcement. Extreme care in drilling, placement of steel, and the pouring of concrete is essential to avoid excessive disturbance of pile boring walls. Concrete placement by pumping or tremie tube to the bottom of the pile borings is recommended. Specifications should require that sufficient space be provided in the pile reinforcing cage during fabrication to allow the insertion of a tremie tube for concrete placement. The pile reinforcing cage should be installed and the concrete pumped immediately after drilling is completed. Per Caltrans Amendments to AASHTO LRFD Bridge Design Specification Section 10.8.1.3 (Caltrans, 2011), 5-inch of concrete cover over reinforcement should be provided to improve construction of the CIDH piles.

Based on recent groundwater measurements, a wet construction is anticipated for the CIDH piles construction. Contractor should be fully prepared to implement the Caltrans Standard Specifications and Special Provisions for a wet construction. Localized dewatering is not recommended because the operation could induce soil settlement which could damage buried utilities and nearby structures.

On-site earth materials are generally fine-grained and contain a few thin layers of granular soil. Granular soils are susceptible to caving. The use of temporary casing is left to the contractor's discretion. If temporary casing is used, vibratory and oversized predrilling techniques for casing installation are not allowed. Temporary casing should be placed tight in the borehole. The casing should be pulled as the concrete is being poured while always maintaining at least a 5-foot head of concrete inside the casing. If any boring becomes bell-shaped and cannot be advanced due to severe caving, all loose material should be removed from the bottom of the boring and the caved region filled with low strength sand-cement slurry. Drilling may continue when the slurry has reached its initial set.

6.3 REVIEW OF CONSTRUCTION PLANS

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

6.4 GEOTECHNICAL OBSERVATION AND TESTING

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

- Earthwork operations, including excavations
- CIDH pile excavations
- Casing or shoring installation if required
- Steel reinforcement installation and concrete placement
- Removal or installation of support for buried utilities or structures
- When any unusual subsurface conditions are encountered

7.0 LIMITATIONS

This report is intended for the use of OCTA, TRC and Caltrans for design and construction of the Overhead Sign Structures for I-5 HOV Improvement Project (Segment 3) between I-5 Mainline Stations 340+00 and 407+50. This report is based on the project as described and the information obtained from the exploratory borings at the approximate locations indicated on the attached LOTB sheets. The findings and recommendations contained in this report are based on the results of the field investigation, laboratory tests, and engineering analyses. In addition, soils and subsurface conditions encountered in the exploratory borings are presumed to be representative of the project site. However, subsurface conditions and characteristics of soils between exploratory borings can vary. The findings reflect an interpretation of the direct evidence obtained. The recommendations presented in this report are based on the assumption that an appropriate level of quality control and quality assurance (inspections and tests) will be provided during construction. EMI should be notified of any pertinent changes in the project plans or if subsurface conditions are found to vary from those described herein. Such changes or variations may require a re-evaluation of the recommendations contained in this report.

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

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

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

8.0 REFERENCES

American Society for Testing and Materials (ASTM), 2007, Annual Book of Standards. Soil and Rock; Dimension Stone; Geosynthetics. Vol. 04.08.

Argus, D.F., Heflin, M.B., Donnellan, A., Webb, F.H., Dong, D., Hurst, K.J., Jefferson, D.C., Lyzenga, G.A., Watkins, M.M., and Zumberge, J.F., 1999, Shortening and thickening of metropolitan Los Angeles measured and inferred by using geodesy: Geology, v. 27, p. 703-706.

Amec Earth and Environmental, Inc. (AMEC), 2006a, Geotechnical Report of Rough Grading, Lot 94 (Garden Lots area), TTM 14196, Window Hill Area, Pacific Point Project, San Juan Capistrano, California, June 23.

AMEC, 2006b, Geotechnical Report of Rough Grading, Lots 105 through 150, TTM 15609, Harbor Vista Lots – West Valley, Pacific Point Project, San Juan Capistrano, California, August 24.

AMEC, 2000a, Geotechnical Evaluation and Response to Review Comments, La Mirada/Calita Slope Area, "AL" Category Perimeter Slope, Pacific Point Project, San Juan Capistrano, California, October 6.

AMEC, 2000b, Previous Review Letters, Pacific Point Project, San Juan Capistrano, California, July 27.

AMEC, 2000c, Second Geotechnical Review Response, La Mirada/Calita Slope Area, "AL" Category Perimeter Slope, Pacific Point Project, San Juan Capistrano, California, December 6.

AMEC, 2000d, Settlement Monitoring, Pacific Point Project, San Juan Capistrano, California, September 26.

AMEC, 2000e, Transfer of Responsibility of Geotechnical Consultant of Record, Pacific Point Project, San Juan Capistrano, California, December 7.

Benioff, H., 1938, The Determination of The Extent of Faulting With Application To The Long Beach Earthquake, Bulletin of The Seismological Society of America, v. 28, p. 77-84.

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

Caltrans, 2011, Amendments to AASHTO LRFD Bridge Design Specifications, Fourth Edition, November.

Caltrans, 2010a, Seismic Design Criteria, Version 1.6, November.

Caltrans, 2010b, Standard Plans.

Caltrans, 2010c, Standard Specifications.

Caltrans, 2009, Geotechnical Services Design Manual, Division of Engineering Services, Geotechnical Services, Version 1.0, August.

Caltrans, 2003a, Bridge Design Specifications, Section 8, September.

Caltrans, 2003b, Corrosion Guidelines, Version 1.0, September.

California Geological Survey, 2009, Tsunami Inundation Map For Emergency Planning, Dana Point and San Juan Capistrano Quadrangles, Division of Mines and Geology, March 15.

California Geological Survey, 2001a, Seismic Hazard Zones, Dana Point Quadrangle, Division of Mines and Geology, Official Map, December 21.

California Geological Survey, 2001b, Seismic Hazard Zone Report For The Dana Point 7.5-Minute Quadrangle, Orange County, California, Department of Conservation, Division of Mines and Geology.

EMI, 2012, Materials Report, I-5 HOV Improvement Project (Segment 3), PCH To San Juan Creek Road, Orange County, California, 12-ORA-5, PM 6.2/8.7, Caltrans Project No. 1200020279 (EA 12-0F96E1), In Progress.

Ensoft, 2010, LPILE Plus Version 6.0, A Program for Analyzing Stress and Deformation of a Pile or Drilled Shaft under Lateral Loading, Austin, Texas.

Ehlig, P., 1989, The Late Cenozoic Evolution of The Capistrano Embayment in Engineering Geology Along Coastal Orange County, Association of Engineering Geologists, Southern California Section, Annual Field Trip guide book, p. 79-87, September 23.

Fumal, T.E., Pezzopane, S.K., Weldon, R.J. II, and Schwartz D.P., 1993, A 100-year Average Recurrence Interval for the San Andreas Fault at Wrightwood, California, Science, v. 259, p. 199-203.

Hauksson, E., Jones, L.M., and Hutton, K., 1995, The 1994 Northridge Earthquake Sequence in California, Seismological and Tectonic Aspects, Journal of Geophysical Research, v. 100, p. 12,335.

Hauksson, E. and Gross, S., 1991, Source Parameters of The 1933 Long Beach Earthquake, Bulletin of The Seismological Society of American, v. 81, p. 81-98.

Jacoby, G., Sheppard, P., and Sieh, K., 1987, Was the 8 December 1812 California Earthquake Produced By The San Andreas Fault, Evidence From Trees Near Wrightwood, Seismological Research Letters, v. 58, p. 14.

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

Leighton and Associates, Inc., 2004, Report of Partial Rough Grading for the North and South Valley and North Estate Lots Area, Pacific Point Project, City of San Juan Capistrano, California, March 12.

NCEER, 1997, "Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", T. L. Youd and I. M. Idriss Editors, Technical Report NCEER-97-0022, NCEER, Buffalo, NY.

Robertson, P.K. and Campanella, R.G., 1989, "Guidelines for Geotechnical Design Using CPT and CPTU", Civil Engineering Department, University of British Columbia, Vancouver, Canada, November.

Rocscience, 2006, "SLIDE 5.0 – 2D Limit Equilibrium Slope Stability Program", Toronto, Canada.

Seed, R.B. and Harder, L.F., Jr., 1990, "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength", in Proceedings, H. Bolton Seed Memorial Symposium, BiTech Publishers, Ltd., P. 351-376.

Seed, H. B., Idriss, I. M., and Arango, I., 1983,"Evaluation of Liquefaction Potential Using Field Performance Data", Journal of Geotechnical Engineering, ASCE, Vol. 109, No. 3, pp. 458-482.

Shao, G., and Hauksson, E., 2009, A Finite Fault Study of the 29 July 2008 M_W = 5.4 Chino Hills Earthquake, Seismological Research Letters, v. 80, p.320-321.

Shaw, J.H., Plesch, A., Dolan, J.F., Pratt, T.L., and Fiore, P., 2002, Puente Hills Blind-Thrust System, Los Angeles, California, Bulletin of the Seismological Society of America, v. 92, p. 2946-2960.

Shaw, J.H., and Shearer, P.M., 1999, An Elusive Blind-Thrust Fault Beneath Metropolitan Los Angeles, Science, v. 283, p.1516, March 5.

Ward, S.N., 1994, A Multidisciplinary Approach to Seismic Hazard in Southern California, Bulletin of the Seismological Society of America, v. 84, p. 1293-1309.

Weldon, R., Scharer, K., Fumal, T., and Biasi, G., 2004, Wrightwood and the Earthquake Cycle, What a Long Recurrence Record Tells Us About How Faults Work, Geological Society of America, GSA Today, v. 14, no. 9, p. 4-10.

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

Yeats, R.S., and Huftile, G.J., 1995, The Oak Ridge Fault System and the 1994 Northridge Earthquake, Nature, v. 373, p. 418-420.

Appendix A

LOG OF TEST BORING SHEETS

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
12	Ora	5	6.2/8.7		

REGISTERED ENGINEER — DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — S. PIRATHIVIRAJ — NO. GE 2963 — EXP. 12-31-13 — GEOTECHNICAL — STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

EARTH MECHANICS, INC.
17800 NEWHOPE STREET,
SUITE B
FOUNTAIN VALLEY, CA 92708

ORANGE COUNTY
TRANSPORTATION AUTHORITY
550 S. MAIN STREET
ORANGE, CA 92863-1584

BENCH MARK

Designation: F-785 Elev = 75.717 feet (NAVD 88); 73.413 feet (NGVD29) Described by OCS 2003 - Found 3¾" US Coast and Geodectic Survey Bronzed Disk Stamped "F 785 1946", Set in the Top of a Concrete Bridge Abutment. Monument is Located in the Southwesterly Corner of the Atchinso\Topeka and Santa Fe Railway Overcrossing of San Mateo Creek, 69 ft. Northerly Along the Railway from the Centerline of the Interstate 5 Freeway Southbound Onramp at Camino Capistrano Prolonged to the West and 150± Westerly from the Centerline of Camino Capistrano. Monument is Set 2.0 ft. below the Tracks.

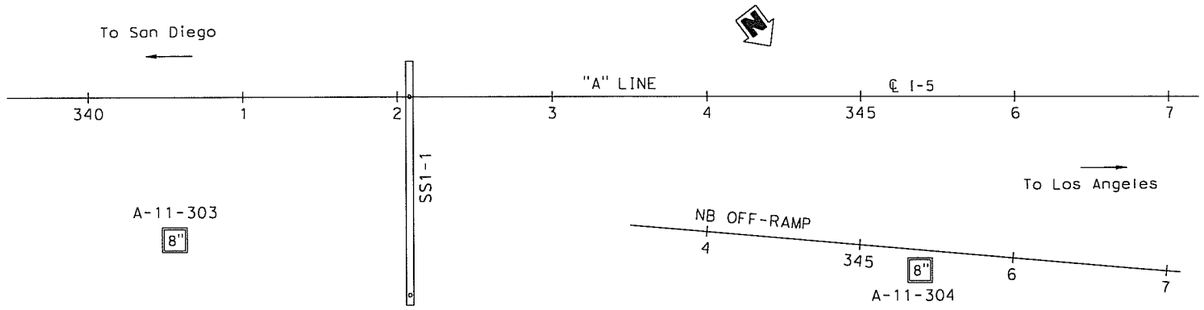
NOTES:

(1) This LOTB sheet was prepared in accordance with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2010).

(2) 2.4" samples were taken using a California Modified Sampler.

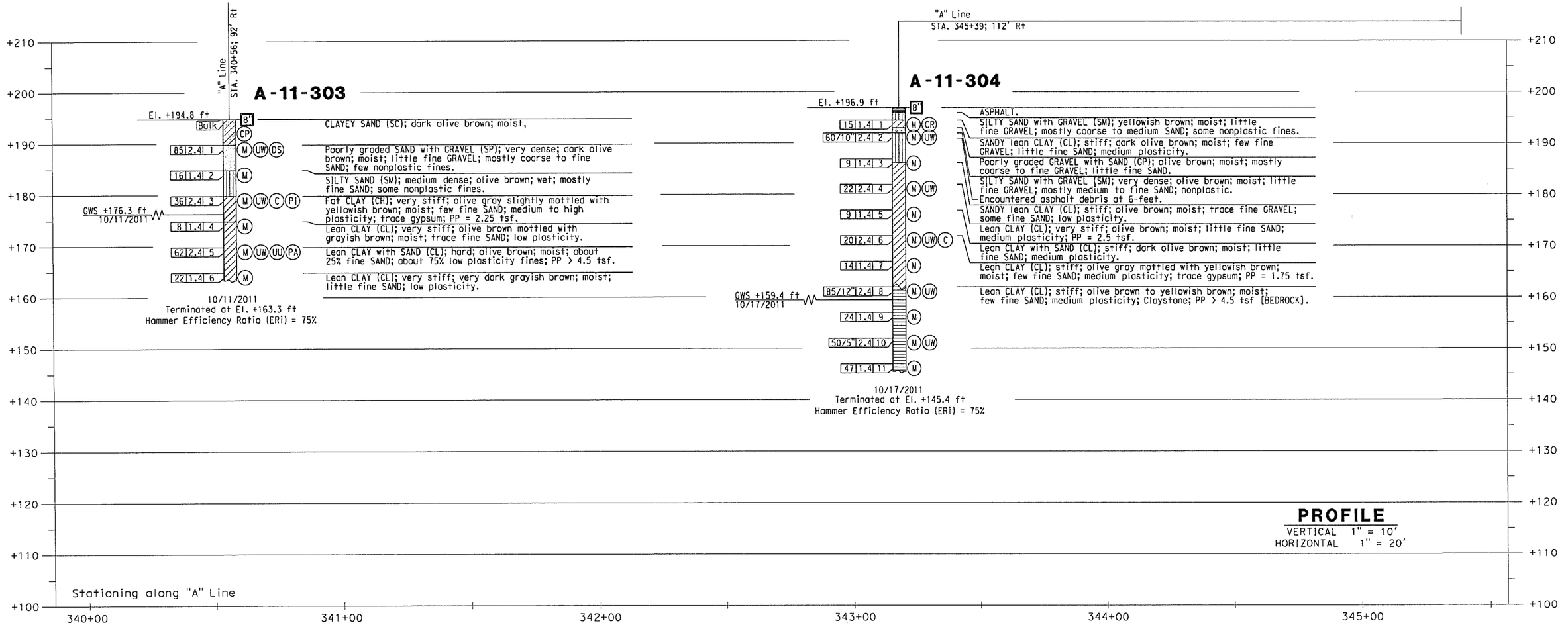
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance the drive sampler.

(4) Conversion factor from 2.4" Modified California Ring Sampler blowcounts to Standard Penetration Test (SPT) blowcounts is 0.5.



PLAN

1" = 50'



PROFILE

VERTICAL 1" = 10'
HORIZONTAL 1" = 20'

LOG OF TEST BORINGS SIGN STRUCTURE SS1-1

SCALE AS SHOWN

SD-11

REVISED BY	DATE REVISED

CALCULATED-DESIGNED BY: J. FANG
CHECKED BY: S. PIRATHIVIRAJ
CONSULTANT FUNCTIONAL SUPERVISOR: LINO CHEANG

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans

LAST REVISION | DATE PLOTTED = > DATE

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
12	Ora	5	6.2/8.7		

REGISTERED ENGINEER — DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — S. PIRATHIVIRAJ — NO. GE 2963 — EXP. 12-31-13 — GEOTECHNICAL — STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

EARTH MECHANICS, INC.
17800 NEWHOPE STREET,
SUITE B
FOUNTAIN VALLEY, CA 92708

ORANGE COUNTY
TRANSPORTATION AUTHORITY
550 S. MAIN STREET
ORANGE, CA 92863-1584

BENCH MARK

Designation: F-785 Elev = 75.717 feet (NAVD 88); 73.413 feet (NGVD29)
Described by OCS 2003 - Found 3¼" US Coast and Geodetic Survey Bronzed Disk Stamped "F 785 1946", Set in the Top of a Concrete Bridge Abutment. Monument is Located in the Southwesterly Corner of the Atchinso\Topeka and Santa Fe Railway Overcrossing of San Mateo Creek, 69 ft. Northerly Along the Railway from the Centerline of the Interstate 5 Freeway Southbound Onramp at Camino Capistrano Prolonged to the West and 150± Westerly from the Centerline of Camino Capistrano. Monument is Set 2.0 ft. below the Tracks.

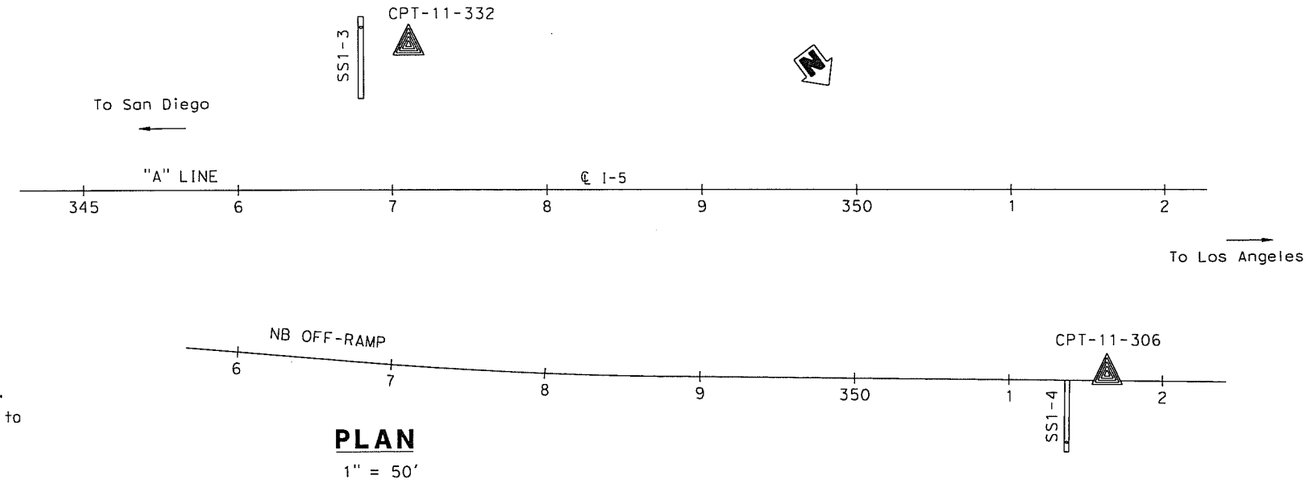
NOTES:

(1) This LOTB sheet was prepared in accordance with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2010).

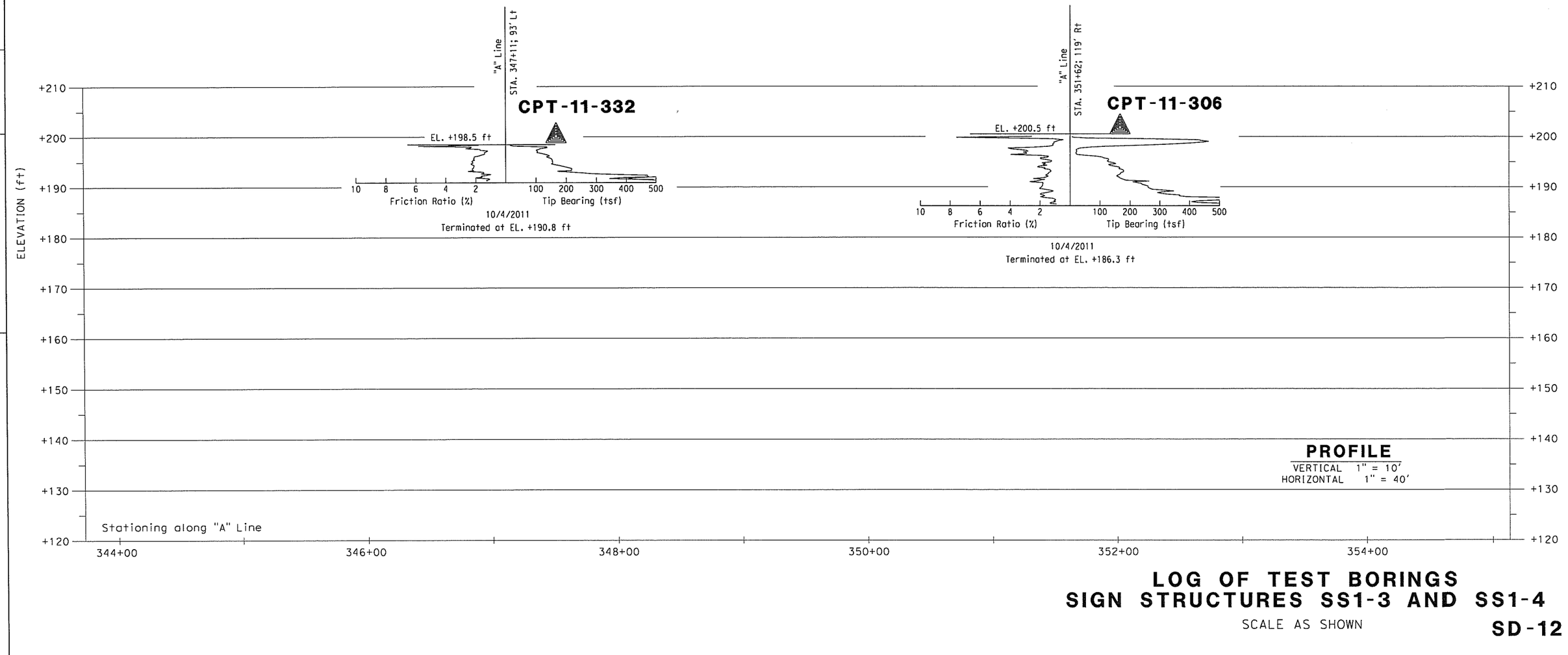
(2) 2.4" samples were taken using a California Modified Sampler.

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

(4) Conversion factor from 2.4" Modified California Ring Sampler blowcounts to Standard Penetration Test (SPT) blowcounts is 0.5.



PLAN
1" = 50'



PROFILE
VERTICAL 1" = 10'
HORIZONTAL 1" = 40'

LOG OF TEST BORINGS
SIGN STRUCTURES SS1-3 AND SS1-4

SCALE AS SHOWN

SD-12

CALCULATED-DESIGNED BY	J. FANG
CHECKED BY	S. PIRATHIVIRAJ
CONSULTANT FUNCTIONAL SUPERVISOR	LINO CHEANG
STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION	Caltrans

REVISED BY	DATE REVISED

LAST REVISION | DATE PLOTTED =>

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
12	Ora	5	6.2/8.7		

REGISTERED ENGINEER _____ DATE _____

PLANS APPROVAL DATE _____

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER — S. PIRATHIVIRAJ — NO. GE 2963 — EXP. 12-31-13 — GEOTECHNICAL — STATE OF CALIFORNIA

EARTH MECHANICS, INC.
17800 NEWHOPE STREET,
SUITE B
FOUNTAIN VALLEY, CA 92708

ORANGE COUNTY
TRANSPORTATION AUTHORITY
550 S. MAIN STREET
ORANGE, CA 92863-1584

BENCH MARK

Designation: F-785 Elev = 75.717 feet (NAVD 88); 73.413 feet (NGVD29)
Described by OCS 2003 - Found 3¼" US Coast and Geodetic Survey Bronzed Disk Stamped "F 785 1946", Set in the Top of a Concrete Bridge Abutment. Monument is Located in the Southwesterly Corner of the Atchinso\Topeka and Santa Fe Railway Overcrossing of San Mateo Creek, 69 ft. Northerly Along the Railway from the Centerline of the Interstate 5 Freeway Southbound Onramp at Camino Capistrano Prolonged to the West and 150± Westerly from the Centerline of Camino Capistrano. Monument is Set 2.0 ft. below the Tracks.

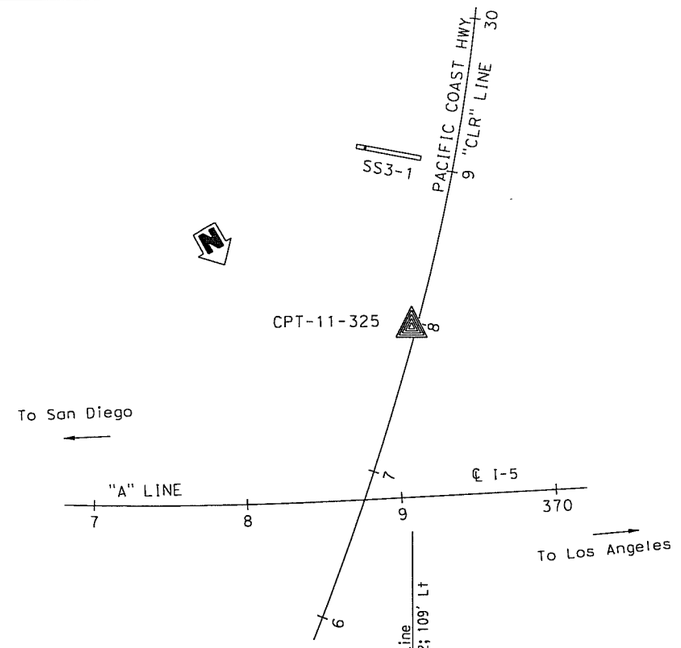
NOTES:

(1) This LOTB sheet was prepared in accordance with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2010).

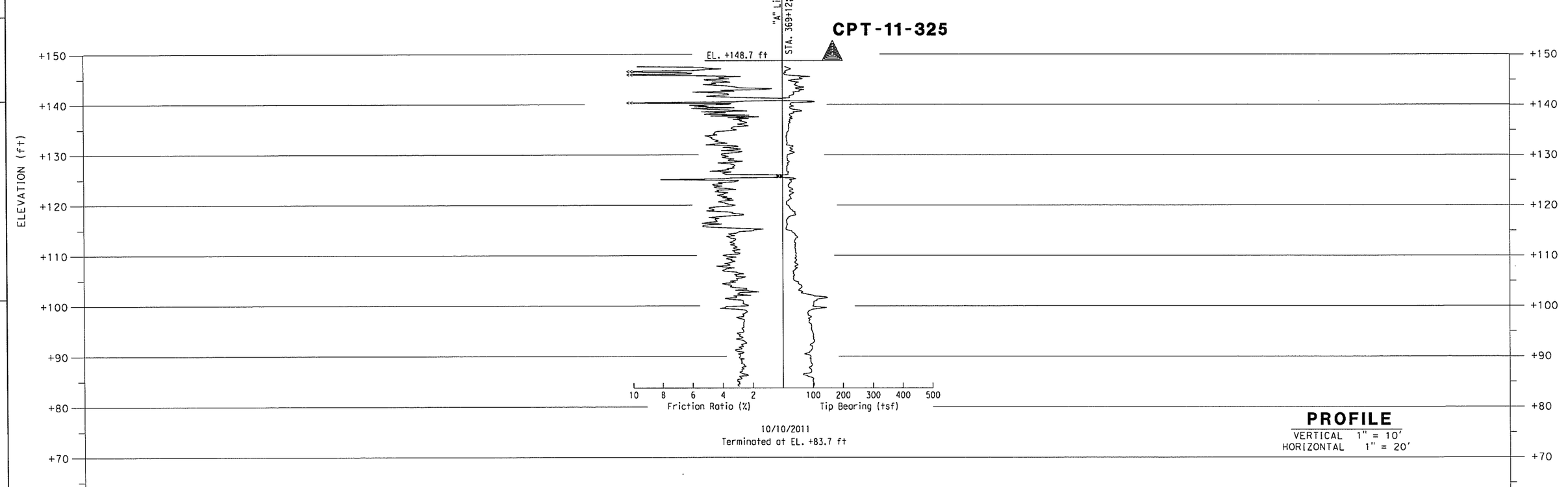
(2) 2.4" samples were taken using a California Modified Sampler.

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

(4) Conversion factor from 2.4" Modified California Ring Sampler blowcounts to Standard Penetration Test (SPT) blowcounts is 0.5.



PLAN
1" = 50'



PROFILE
VERTICAL 1" = 10'
HORIZONTAL 1" = 20'

LOG OF TEST BORING
SIGN STRUCTURE SS3-1

SCALE AS SHOWN

SD-13

REVISED BY	DATE REVISED

CALCULATED-DESIGNED BY: J. FANG
CHECKED BY: S. PIRATHIVIRAJ
DRAWN BY
CONSULTANT FUNCTIONAL SUPERVISOR: LINO CHEANG
STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans
LAST REVISION | DATE PLOTTED = 3 SEP 13

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
12	Ora	5	6.2/8.7		

REGISTERED ENGINEER — DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — S. PIRATHIVIRAJ — NO. GE 2963 — EXP. 12-31-13 — GEOTECHNICAL — STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

EARTH MECHANICS, INC.
17800 NEWHOPE STREET,
SUITE B
FOUNTAIN VALLEY, CA 92708

ORANGE COUNTY
TRANSPORTATION AUTHORITY
550 S. MAIN STREET
ORANGE, CA 92863-1584

BENCH MARK

Designation: F-785 Elev = 75.717 feet (NAVD 88); 73.413 feet (NGVD29) Described by OCS 2003 - Found 3¼" US Coast and Geodectic Survey Bronzed Disk Stamped "F 785 1946", Set in the Top of a Concrete Bridge Abutment. Monument is Located in the Southwesterly Corner of the Atchinso\Topeka and Santa Fe Railway Overcrossing of San Mateo Creek, 69 ft. Northerly Along the Railway from the Centerline of the Interstate 5 Freeway Southbound Onramp at Camino Capistrano Prolonged to the West and 150± Westerly from the Centerline of Camino Capistrano. Monument is Set 2.0 ft. below the Tracks.

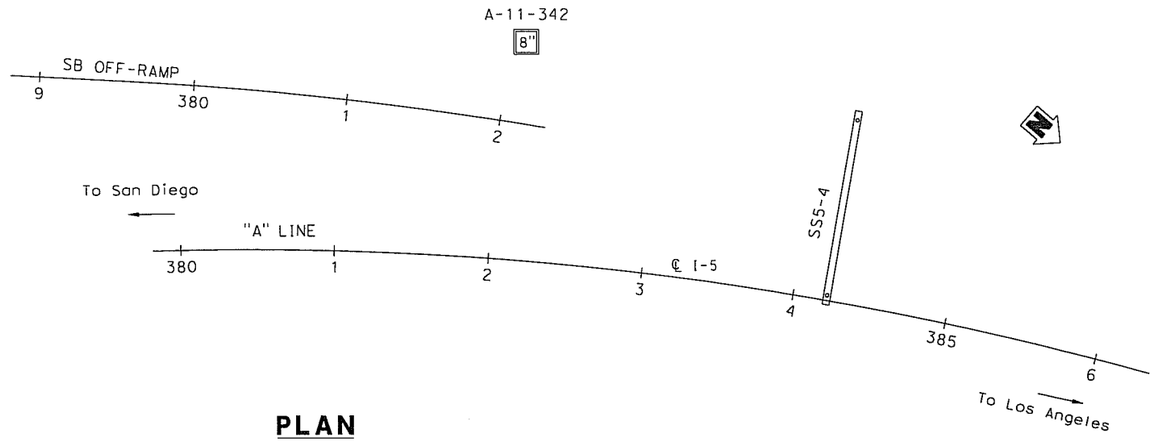
NOTES:

(1) This LOTB sheet was prepared in accordance with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2010).

(2) 2.4" samples were taken using a California Modified Sampler.

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

(4) Conversion factor from 2.4" Modified California Ring Sampler blowcounts to Standard Penetration Test (SPT) blowcounts is 0.5.



PLAN

1" = 50'

A-11-342

"A" Line STA. 382+22; 114' Lt

El. +147.6 ft

8"

Bulk

13	1.4	1	M	CLAYEY SAND with GRAVEL (SC); olive brown; dry; little fine GRAVEL, max. ¼in. dia.; coarse to fine SAND; some nonplastic fines.
42	2.4	2	M UW	SANDY lean CLAY (CL); very stiff; yellowish brown; moist; trace fine GRAVEL, max.¾in. dia.; some fine SAND; low plasticity fines.
27	1.7	3	M PA	Lean CLAY (CL); hard; dark yellowish brown; moist; trace fine GRAVEL, max. ⅜in. dia.; few medium to fine SAND; low plasticity fines; PP > 4.5 tsf.
42	2.4	4	M UW UU	Lean CLAY with SAND (CL); hard; dark yellowish brown; moist; 2% fine GRAVEL, max.½in. dia.; 19% fine SAND; 79% low plasticity fines.
12	1.4	5	M	SILTY SAND with GRAVEL (SM); medium dense to dense; dark gray; moist; little fine GRAVEL, max.¾in. dia.; medium to fine SAND; low plasticity fines.
17	2.4	6	M UW	Lean CLAY with SAND (CL); stiff; olive brown; moist; few medium to fine SAND; medium plasticity fines; PP = 1.5 tsf.
2	1.4	7	M	CLAYEY SAND (SC); medium dense; yellowish brown; moist; few fine GRAVEL, max.⅜in. dia.; medium to fine SAND; low plasticity fines.
11	1.4	8	M	SANDY lean CLAY (CL); stiff; yellowish brown; moist; medium to fine SAND; low plasticity fines.
				CLAYEY SAND (SC); stiff; yellowish brown; moist; fine SAND; low plasticity fines.
				SANDY SILT (ML); soft; yellowish brown; moist; fine SAND; low plasticity fines.
				stiff

09-26-2011
Terminated at EL. + 106.1 ft
Hammer Efficiency Ratio (ERi) = 75%

ELEVATION (ft)

+150 +140 +130 +120 +110 +100 +90 +80 +70 +60

Stationing along "A" Line

380+00 381+00 382+00 383+00 384+00

PROFILE

VERTICAL 1" = 10'
HORIZONTAL 1" = 20'

LOG OF TEST BORING SIGN STRUCTURE SS5-4

SCALE AS SHOWN

SD-14

REVISED BY	DATE REVISED

CALCULATED-DESIGNED BY: J. FANG
CHECKED BY: S. PIRATHIVIRAJ
CONSULTANT FUNCTIONAL SUPERVISOR: LINO CHEANG

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans®

LAST REVISION | DATE PLOTTED => 9/SEP/12

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
12	Ora	5	6.2/8.7		

REGISTERED ENGINEER — DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — S. PIRATHIVIRAJ — NO. GE 2963 — EXP. 12-31-13 — GEOTECHNICAL — STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

EARTH MECHANICS, INC.
17800 NEWHOPE STREET,
SUITE B
FOUNTAIN VALLEY, CA 92708

ORANGE COUNTY
TRANSPORTATION AUTHORITY
550 S. MAIN STREET
ORANGE, CA 92863-1584

REVISED BY	DATE REVISED

CALCULATED-DESIGNED BY	CHECKED BY
J. FANG	S. PIRATHIVIRAJ

CONSULTANT FUNCTIONAL SUPERVISOR: LINO CHEANG

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

BENCH MARK

Designation: F-785 Elev = 75.717 feet (NAVD 88); 73.413 feet (NGVD29)
Described by OCS 2003 - Found 3¾" US Coast and Geodectic Survey Bronzed Disk Stamped "F 785 1946", Set in the Top of a Concrete Bridge Abutment. Monument is Located in the Southwesterly Corner of the Atchinso\Topeka and Santa Fe Railway Overcrossing of San Mateo Creek, 69 ft. Northerly Along the Railway from the Centerline of the Interstate 5 Freeway Southbound Onramp at Camino Capistrano Prolonged to the West and 150± Westerly from the Centerline of Camino Capistrano. Monument is Set 2.0 ft. below the Tracks.

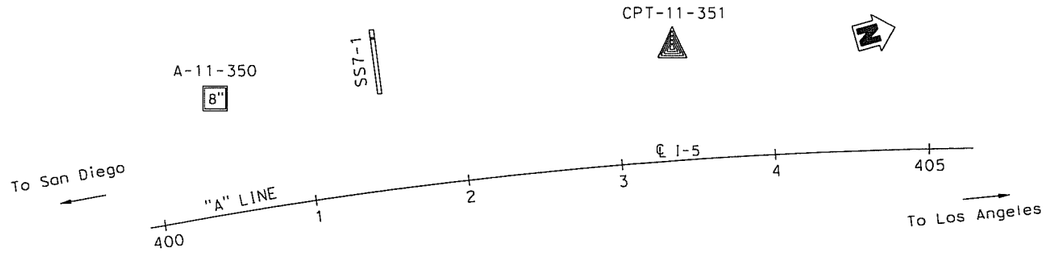
NOTES:

(1) This LOTB sheet was prepared in accordance with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2010).

(2) 2.4" samples were taken using a California Modified Sampler.

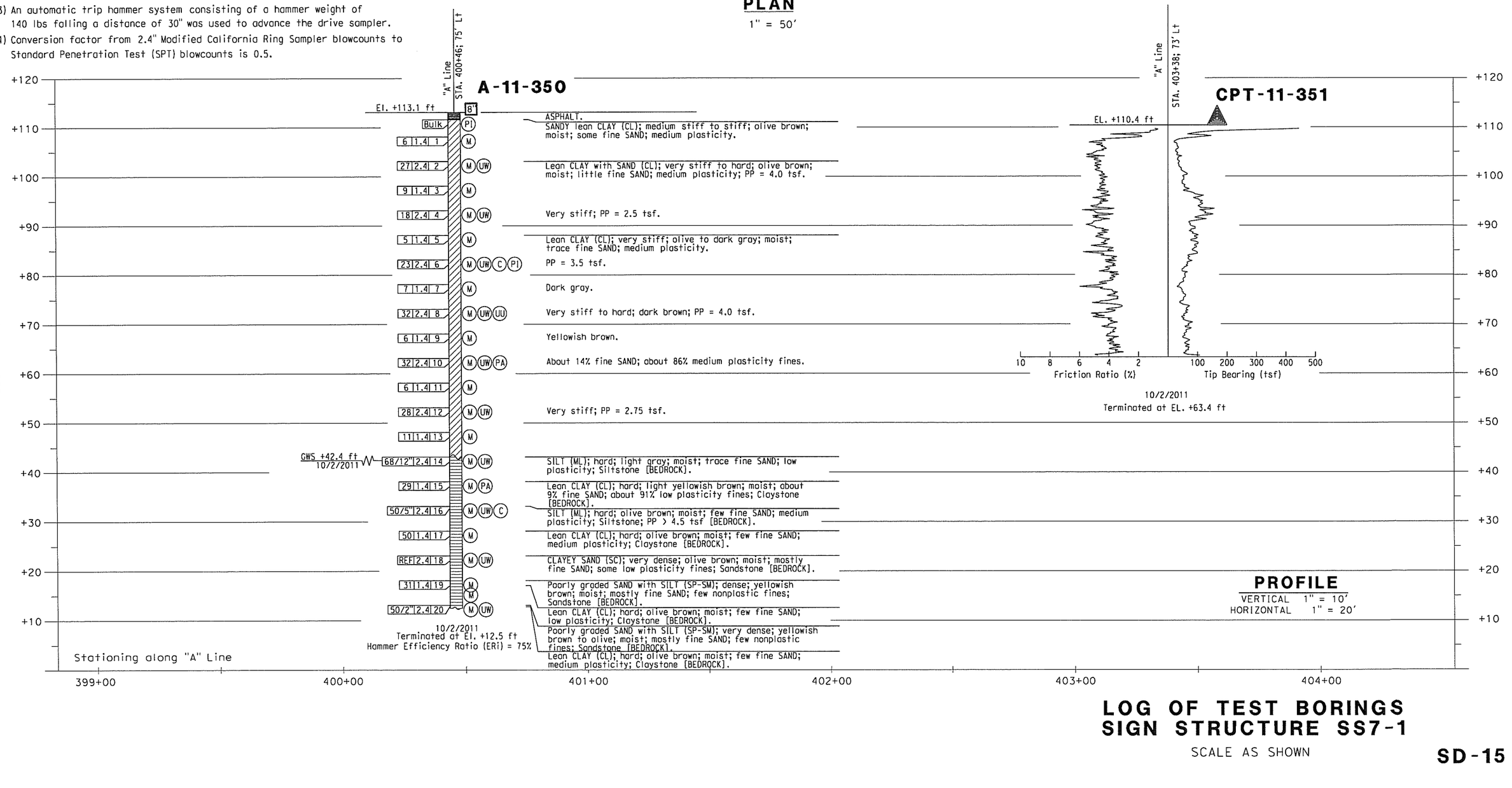
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance the drive sampler.

(4) Conversion factor from 2.4" Modified California Ring Sampler blowcounts to Standard Penetration Test (SPT) blowcounts is 0.5.



PLAN

1" = 50'



PROFILE

VERTICAL 1" = 10'
HORIZONTAL 1" = 20'

LOG OF TEST BORINGS
SIGN STRUCTURE SS7-1

SCALE AS SHOWN

SD-15

LAST REVISION | DATE PLOTTED =>

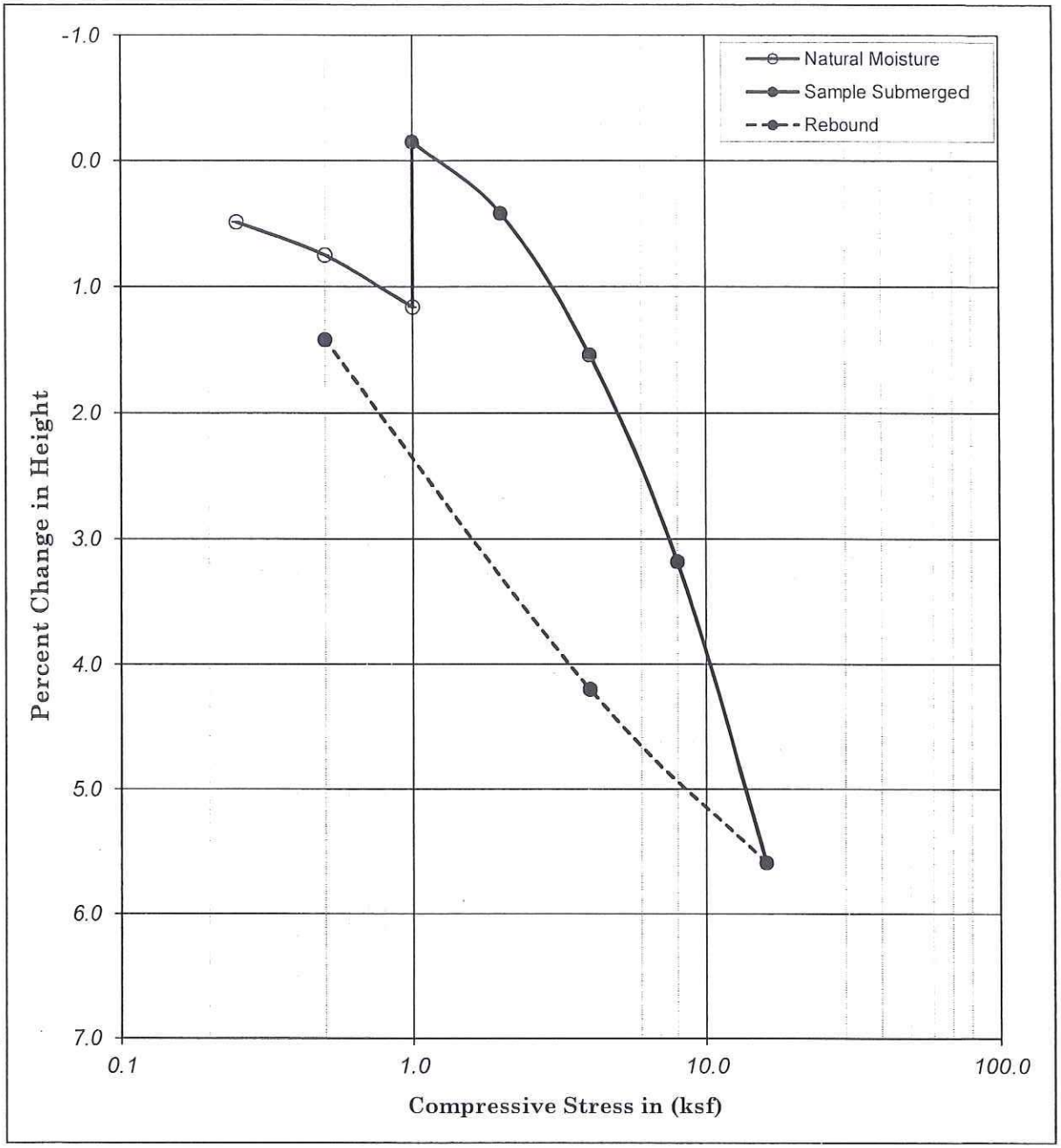
Appendix B

LABORATORY TEST RESULTS

TABLE B-1 SUMMARY OF LABORATORY TEST RESULTS

Project No. : 11-137 Project Name : I-5 HOV Improvement Project - PCH to San Juan Creek Road

Boring No.	Sample No.	Sample Depth (ft)	Soil Identification (group symbol) ASTM D2488/D2487	Moisture Content ASTM D2216 (%)	Total Unit Weight ASTM D2937 (pcf)	Pocket Penetrometer (tsf)	Torvane Shear (tsf)	Grain Size Distribution GR:SA:FI (%)	Sand Equivalent (CT-217)	Atterberg Limits ASTM D4318 (LL/PL/PI)	Soil-Minimum Resisivity CT-532 (ohm-cm)	Soil-pH CT-532	Soil-Soluble Sulfate Content CT-417 (ppm)	Soil-Moisture Free Chloride Content CT-422 (ppm)
A-11-303	B-0	0-5	SC					0:69:31		22/13/9				
A-11-303	D-1	5	SP	4.8	121.8									
A-11-303	S-2	10	SM	24.3										
A-11-303	D-3	15	CH	26.1	121.4	2.25				55/26/29				
A-11-303	S-4	20	CL	32.8										
A-11-303	D-5	25	CL	23.4	126.5	>4.5		0:25:75						
A-11-303	S-6	30	CL	24.3										
A-11-304	S-1	2.5	CL	9.9							480	8.1	800	644
A-11-304	D-2	5	SM	7.7	119.1									
A-11-304	S-3	10	SM to CL	15.2										
A-11-304	D-4	15	CL	20.3	124.1	2.5								
A-11-304	S-5	20	CL	17.8										
A-11-304	D-6	25	CL	31.9	117.7	1.75								
A-11-304	S-7	30	CL	31.4										
A-11-304	D-8	35	CL	28.4	119.6	>4.5								
A-11-304	S-9	40	CL	32.0										
A-11-304	D-10	45	CL	26.3	120.4	>4.5								
A-11-304	S-11	50	CL	29.0										
A-11-342	B-0	0-5	SC					0:77:23		30/18/12				
A-11-342	S-1	5	CL	23.1										
A-11-342	D-2	10	CL	19.6	127.6	>4.5								
A-11-342	S-3	15	CL	17.5				2:19:79						
A-11-342	D-4T	20	CL	24.5	121.6	1.5								
A-11-342	D-4B	21	CL to SC	7.3	129.3									
A-11-342	S-5	25	CL	23.7										
A-11-342	D-6	30	SC	21.6	120.6									
A-11-342	S-7	35	ML	22.6										
A-11-342	S-8	40	ML	22.6										
A-11-350	B-0	0-5	CL					0:50:50		34/16/18				
A-11-350	S-1	5	CL	20.7										
A-11-350	D-2	10	CL	21.7	123.6	4.0								
A-11-350	S-3	15	CL	22.0										
A-11-350	D-4	20	CL	24.8	117.1	2.5								
A-11-350	S-5	25	CL	19.9										
A-11-350	D-6	30	CL	20.2	123.3	3.5				38/19/19				
A-11-350	S-7	35	CL	23.2										
A-11-350	D-8	40	CL	19.4	126.4	4.0								
A-11-350	S-9	45	CL	26.4										
A-11-350	D-10	50	CL	28.5	120.6			0:14:86						
A-11-350	S-11	55	CL	29.1										
A-11-350	D-12	60	CL	29.4	120.0	2.75								
A-11-350	S-13	65	CL	27.4										
A-11-350	D-14	70	ML	27.4	119.4									
A-11-350	S-15	75	CL	30.5				0:9:91						
A-11-350	D-16	80	ML	28.5	121.5	>4.5								
A-11-350	S-17	85	CL	25.1										
A-11-350	D-18	90	SC	28.1	121.5									
A-11-350	S-19T	95	SP-SM	20.2										
A-11-350	S-19B	96	CL	27.5										
A-11-350	D-20	100	SP to CL	20.0	122.6									



Boring No.	: A-11-303		Liquid Limit :	-		Moisture Content (%)	Dry Density (pcf)	Dry Density (kN/m³)	Percent Saturation	Void Ratio
Sample No.	: D-3		Plastic Limit :	-						
Depth (ft)	: 15.0	16.5	Plastic Index :	-	Initial	29.03	94.41	14.86	99.79	0.79
Depth (m)	: 4.58	5.03	Specific Gravity :	2.70	Final	31.21	95.78	15.08	110.90	0.76

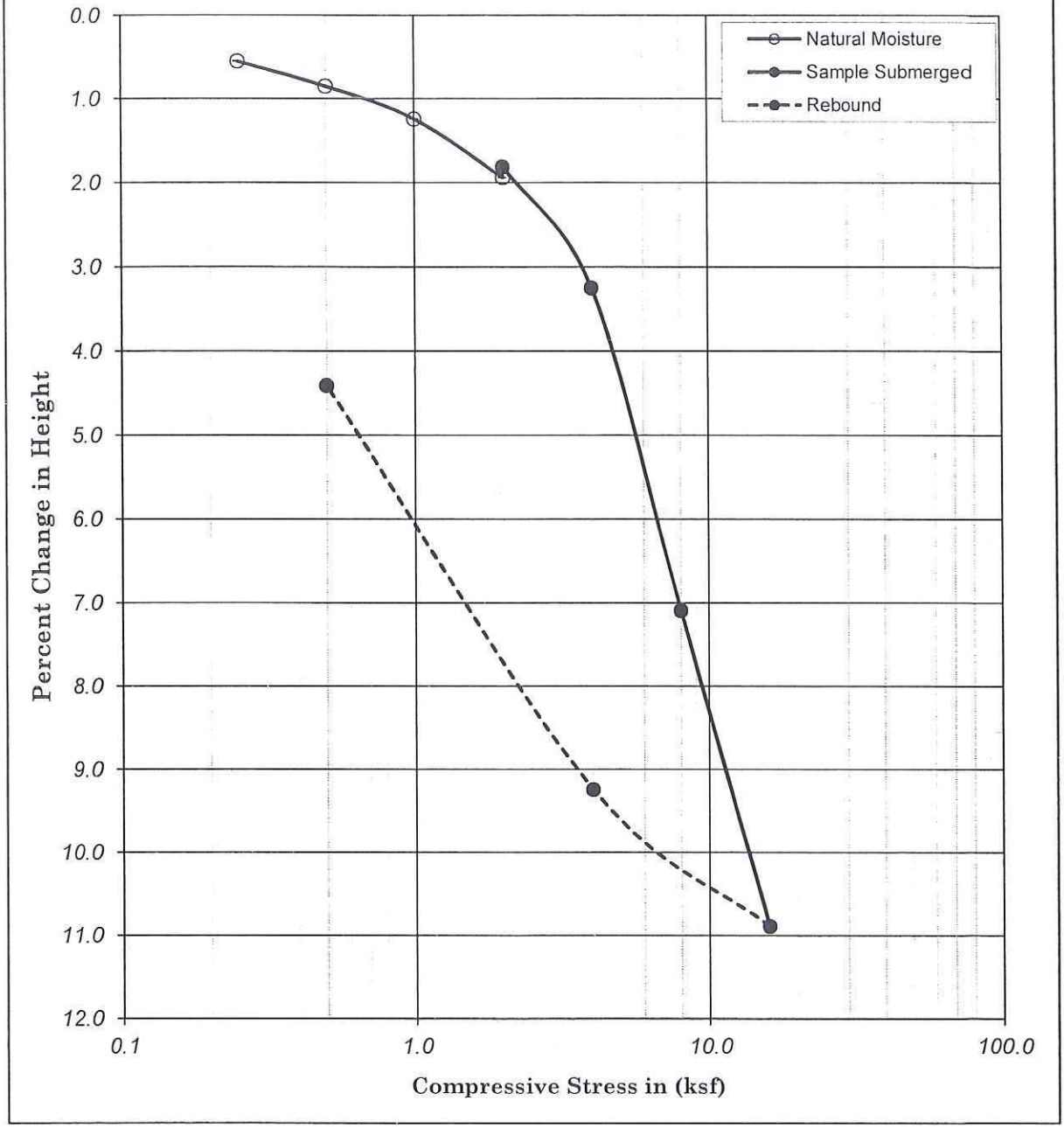
Description : *Olive-gray with yellowish brown , Lean CLAY (CL)*

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

I-5 HOV Improvement Project
PCH to San Juan Creek Road

Project No. : 11-137 12/04/11

CONSOLIDATION TEST
(ASTM D-2435 / CT-219)



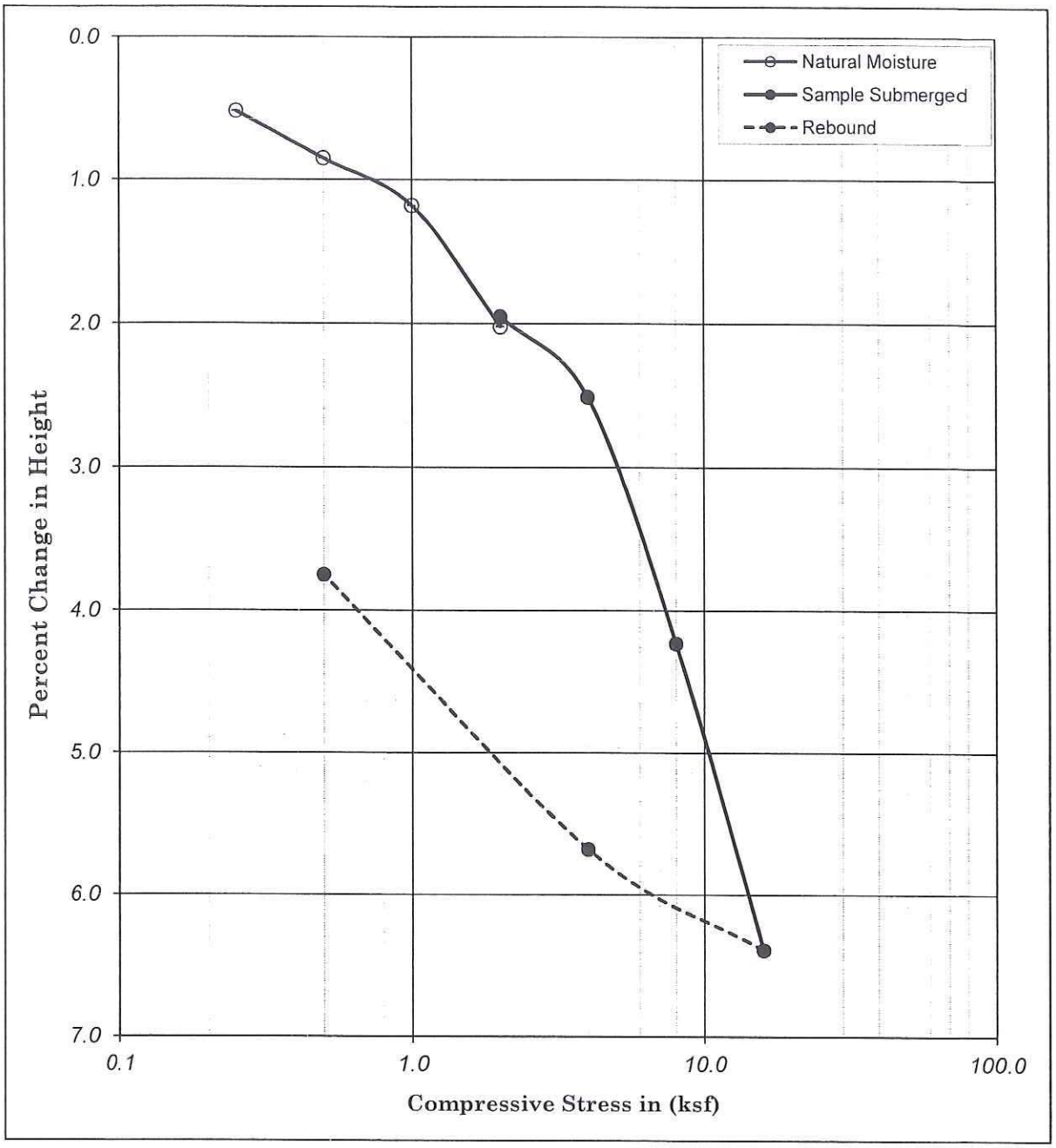
Boring No. : A-11-304				Liquid Limit :	-		Moisture Content (%)	Dry Density (pcf)	Dry Density (kN/m^3)	Percent Saturation	Void Ratio
Sample No. : D-6				Plastic Limit :	-						
Depth	(ft) :	15.0	16.5	Plastic Index :	-	Initial	33.36	87.44	13.76	97.09	0.93
	(m) :	4.58	5.03	Specific Gravity :	2.70	Final	32.43	91.49	14.40	103.94	0.84
Description : Olive-gray with yellowish brown , Lean CLAY (CL)											

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

I-5 HOV Improvement Project
PCH to San Juan Creek Road

Project No. : 11-137 12/04/11

CONSOLIDATION TEST
(ASTM D-2435 / CT-219)



Boring No.	: A-11-350		Liquid Limit :	-		Moisture Content (%)	Dry Density (pcf)	Dry Density (kN/m³)	Percent Saturation	Void Ratio
Sample No.	: D-6		Plastic Limit :	-						
Depth (ft)	: 30.0	31.5	Plastic Index :	-	Initial	34.14	85.74	13.50	95.42	0.97
Depth (m)	: 9.15	9.61	Specific Gravity :	2.70	Final	36.96	89.09	14.02	111.86	0.89

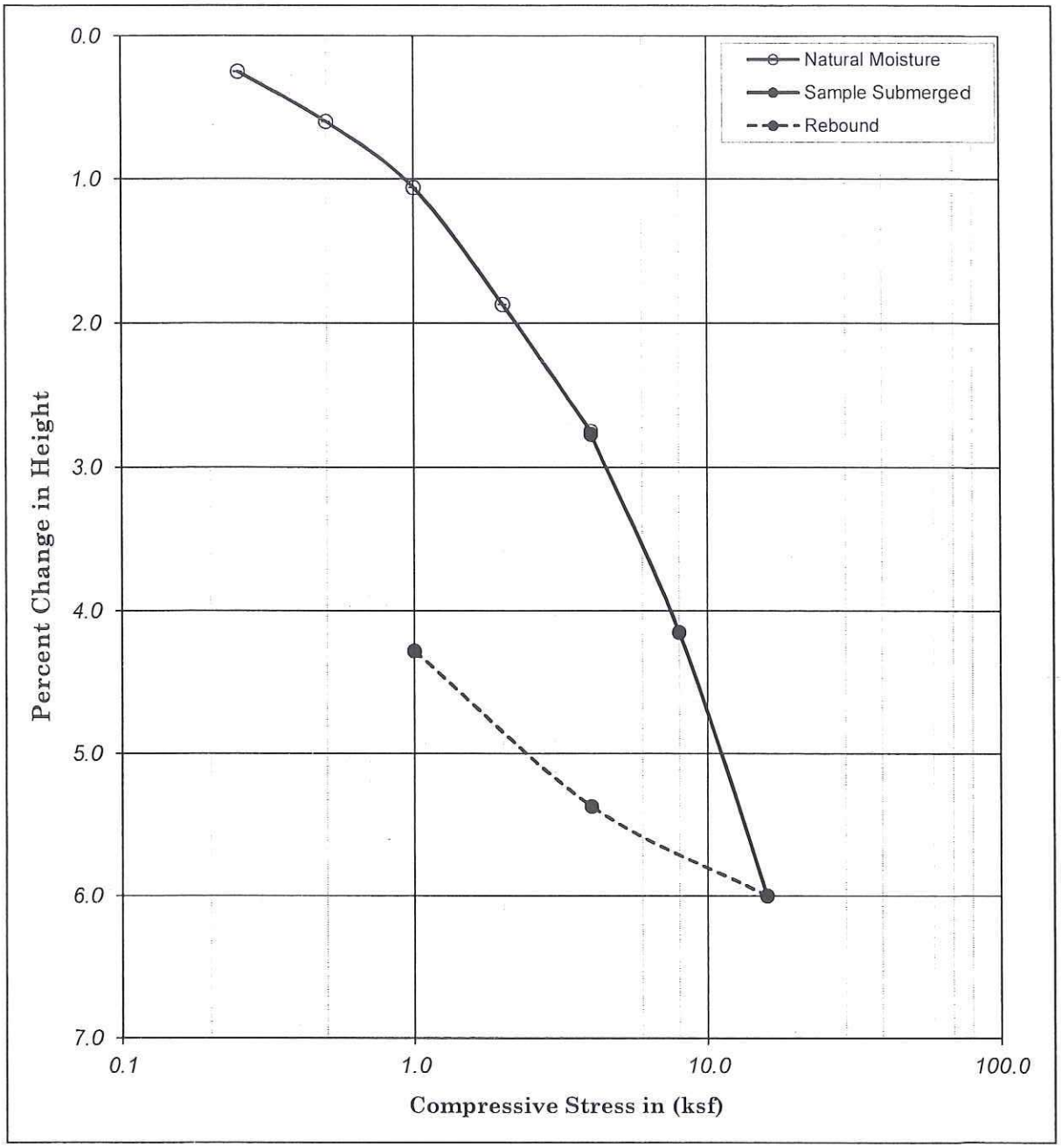
Description : *Olive-brown with yellowish brown, Lean CLAY (CL)*

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

I-5 HOV Improvement Project
PCH to San Juan Creek Road

Project No. : 11-137 12/04/11

CONSOLIDATION TEST
(ASTM D-2435 / CT-219)



Boring No.	: A-11-350		Liquid Limit :	-		Moisture Content (%)	Dry Density (pcf)	Dry Density (kN/m^3)	Percent Saturation	Void Ratio
Sample No.	: D-16		Plastic Limit :	-						
Depth (ft)	: 80.0	81.5	Plastic Index :	-	Initial	31.66	93.70	14.75	107.00	0.80
Depth (m)	: 24.40	24.86	Specific Gravity :	2.70	Final	29.97	97.90	15.41	112.12	0.72

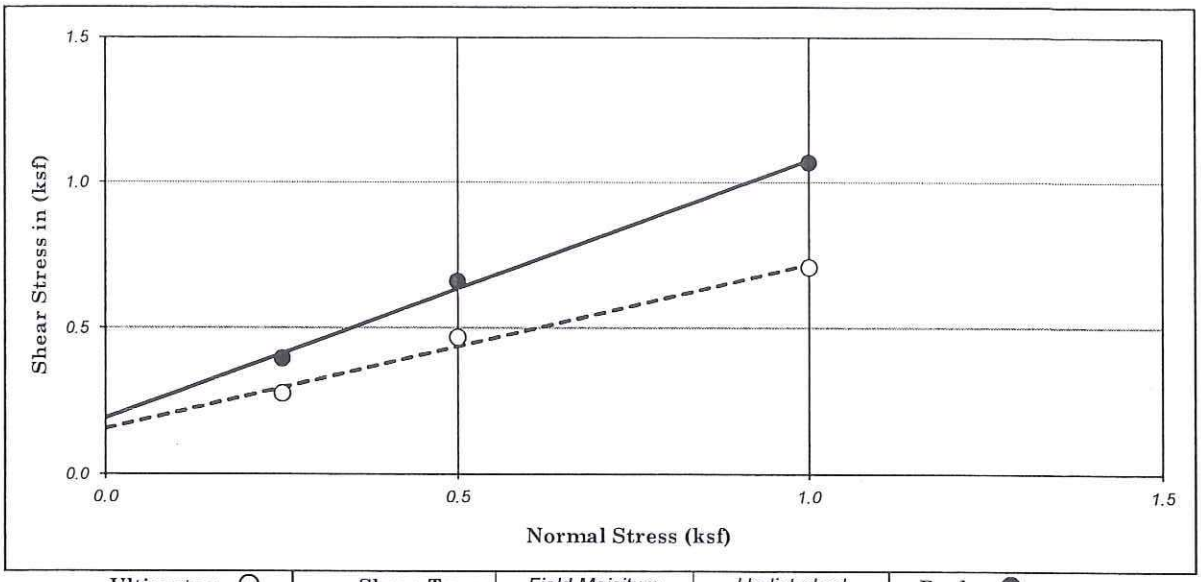
Description : *Very dark grayish brown, Elastic SILT (MH)*

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

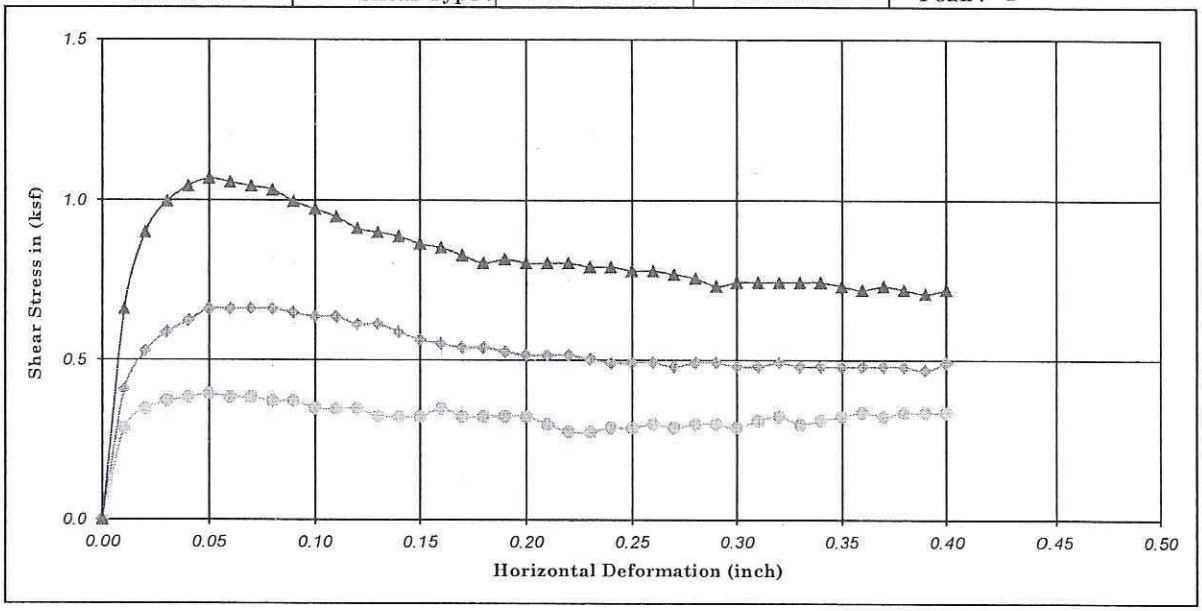
I-5 HOV Improvement Project
PCH to San Juan Creek Road

Project No. : *11-137* *12/04/11*

CONSOLIDATION TEST
(ASTM D-2435 / CT-219)



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



Boring No. : A-11-303		Strength Intercept (C) :	0.19	(ksf)	Peak	0.16	(ksf)	Ultimate
Sample No. : D-1			9.19	(kPa)		7.47	(kPa)	
Depth (ft/m) : 5.0	0.00	Friction Angle (ϕ) :	41.50	Degree		29.35	Degree	
Description : Dark olive-brown, Poorly graded SAND with GRAVEL (SP)					Shear Rate (inch/minute) :			0.02

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	5.34	113.52	17.87	0.48	0.25	11.97	0.40	18.96	0.28	13.21
◆	7.05	108.95	17.15	0.55	0.50	23.94	0.66	31.60	0.47	22.41
▲	7.77	113.27	17.83	0.49	1.00	47.88	1.07	51.14	0.71	33.90

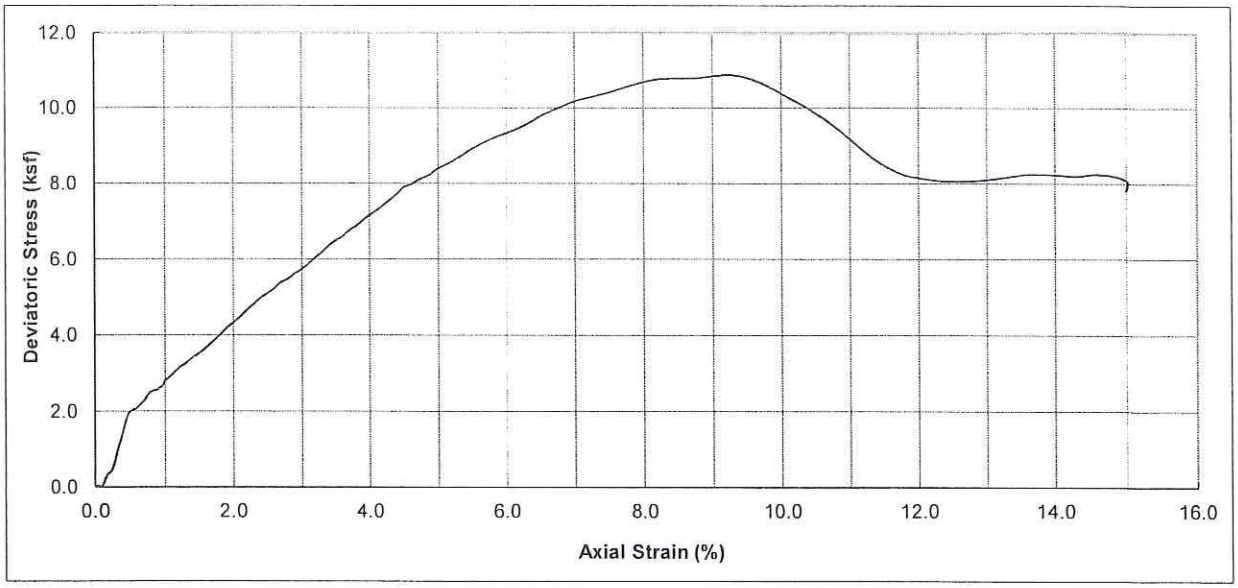
Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

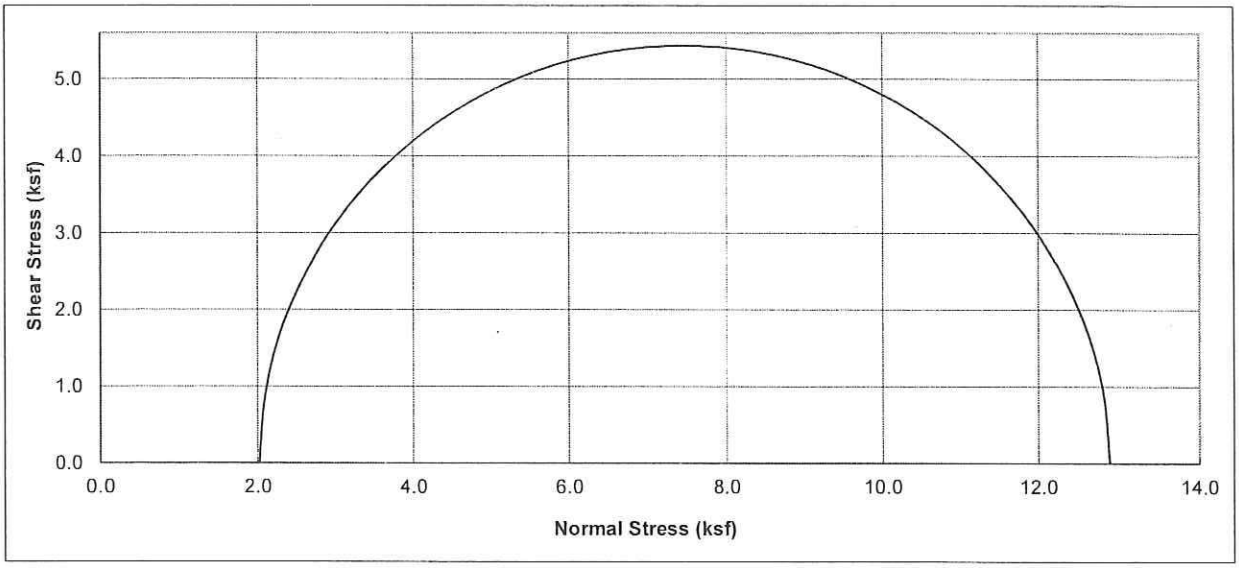
I-5 HOV Improvement Project
PCH to San Juan Creek Road

Soundwall 340

DIRECT SHEAR TEST (ASTM D-3080)

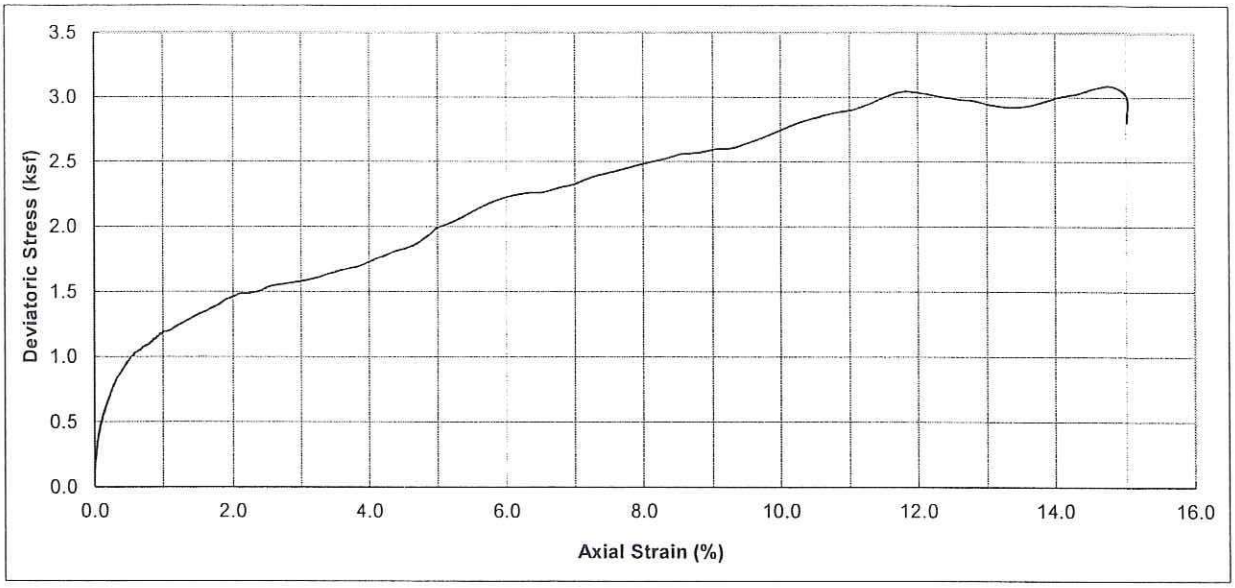
Project No. : *11-137* | Date : *11/05/11* | Figure No. :

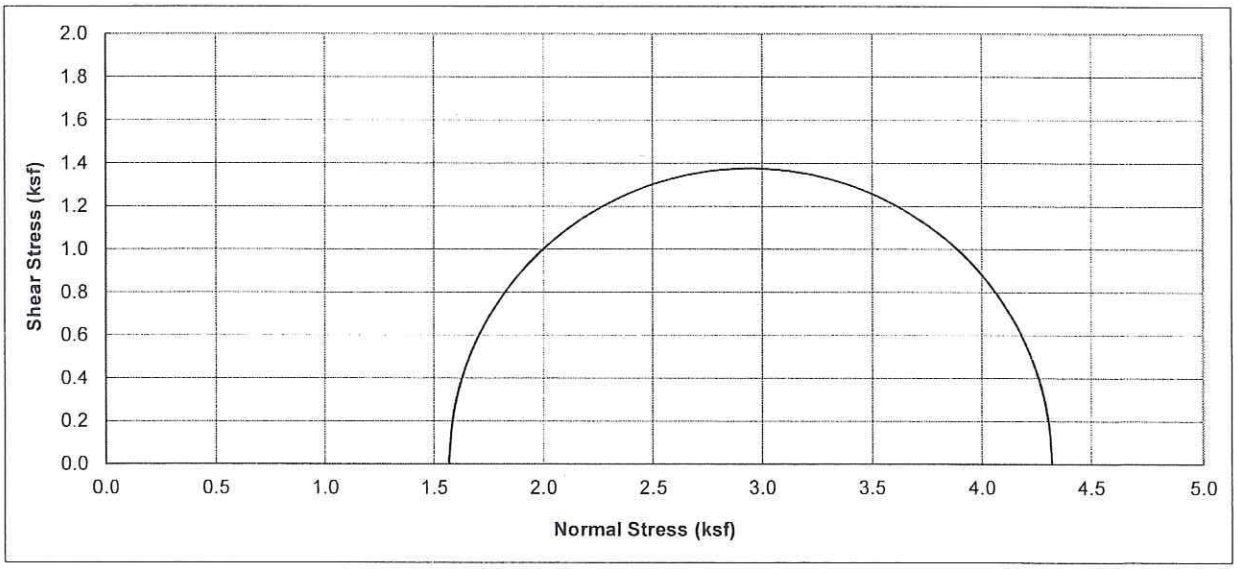




Boring No.	Sample No.	Depth (ft)	Soil Type	Dry Density (pcf)	Moisture Content (%)	Conf. Stress (ksf)	10% Axial Strain Dev. Stress (ksf)	Initial Saturation (%)
A-11-303	D-5	25	Olive brown , Lean CLAY with SAND (CL)	102.1	23.44	2.02	10.87	97.5

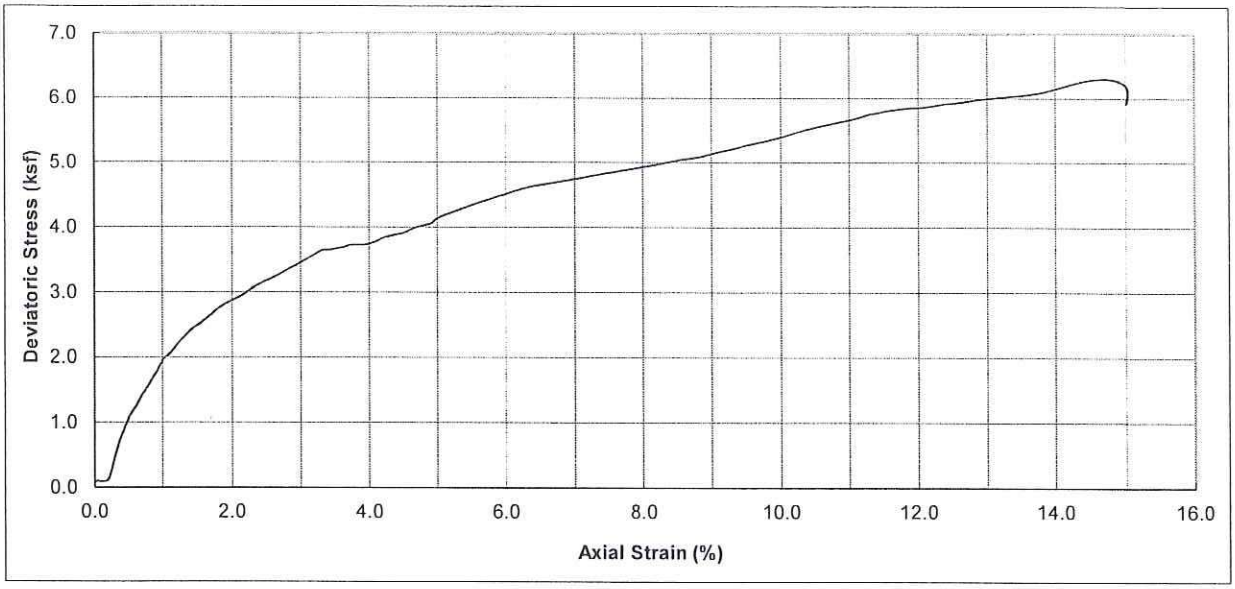
Earth Mechanics, Inc. Geotechnical and Earthquake Engineering		I-5 HOV Improvement Project PCH to San Juan Creek Road
		UNCONSOLIDATED UNDRAINED TEST
Project No. : 11-137	Date : 10/27/11	(ASTM D2850) Figure No. :

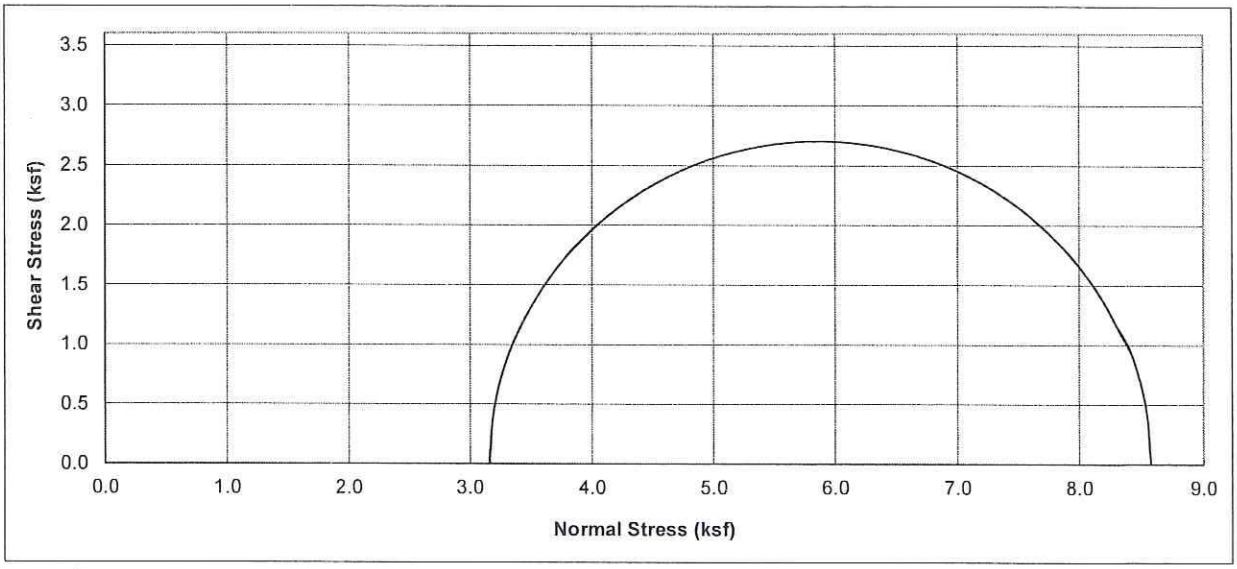




Boring No.	Sample No.	Depth (ft)	Soil Type	Dry Density (pcf)	Moisture Content (%)	Conf. Stress (ksf)	10% Axial Strain Dev. Stress (ksf)	Initial Saturation (%)
A-11-342	D-4	20	Olive brown , Lean CLAY with SAND (CL)	96.5	24.51	1.57	2.75	88.8

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering		I-5 HOV Improvement Project PCH to San Juan Creek Road
		UNCONSOLIDATED UNDRAINED TEST
Project No. : 11-137	Date : 11/01/11	(ASTM D2850) Figure No. :





Boring No.	Sample No.	Depth (ft)	Soil Type	Dry Density (pcf)	Moisture Content (%)	Conf. Stress (ksf)	10% Axial Strain Dev. Stress (ksf)	Initial Saturation (%)
A-11-350	D-8	40	Dark brown , Lean CLAY (CL)	107.1	19.44	3.16	5.41	91.6

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering		I-5 HOV Improvement Project PCH to San Juan Creek Road
		UNCONSOLIDATED UNDRAINED TEST
Project No. : 11-137	Date : 11/02/11	(ASTM D2850) Figure No. :

Appendix C

DESIGN CALCULATIONS

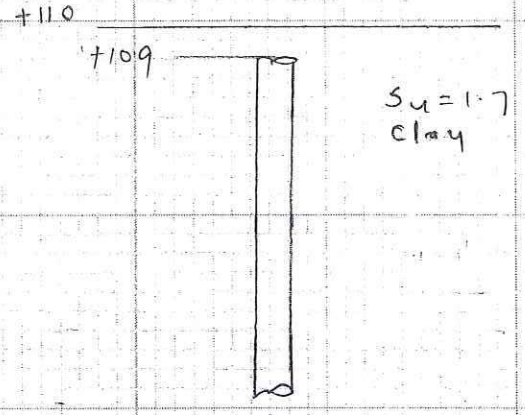
Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

17800 Newhope Street, Suite B, Fountain Valley, CA 92708
(714) 751-3826, Fax (714) 751-3928

Project I-5 HOV segment 3 - Sign Structures Project No. 11-137

By SP Date 11/5/12 Checked By LCC Date 11/8/12 Sheet ___ of ___

Sign structure SS7-11



foundation demands Axial = 38.7 kips
pile head shear = 36 kips
pile head moment = 853 kip-ft

for axial capacity

axial demand = 38.7 kips
Nominal resistance = 38.7 x 2 = 77.4 kips
consider 80 kips

from analysis
at depth of 11 feet, axial capacity = 88 kips
Required pile length = 11 feet

for lateral capacity

using pile head shear = 36 kips
pile head moment = 853 kip-ft

from analysis
pile head deflection = 0.36 inches
maximum moment = 924 kip-ft @ 3.9 feet
maximum shear = 65.4 kips @ 14.5 feet

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

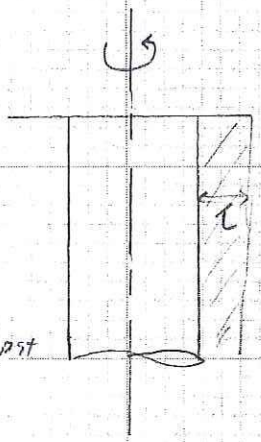
17800 Newhope Street, Suite B, Fountain Valley, CA 92708
(714) 751-3826, Fax (714) 751-3928

Project I-5 HOV Segment 3 - Sign Structures Project No. 11-137

By SP Date 11/6/12 Checked By LCC Date 11/8/12 Sheet ____ of ____

In order to use caltrans standard plans
use $\phi = 30°$ with foundation demand
pile head deflection = 0.4 inch
maximum moment = 976 kip-ft @ 5.3 feet
maximum shear = 85.5 kip @ 15.5 feet

Torsional Capacity



Resisting torsional
moment $M_T = (\pi d) L \tau_{all} r$

from NAVFAC
$\tau = 890$ psf for $S_u = 1700$ psf

$M_T = \pi \times 5 \times 23 \times 890 \times 5/2$
$= 804$ kip-ft

Required $M_u = 355 \times 2 = 710$

$M_T > M_u$

SKIN FRICTION FOR DRILLED SHAFT IN BOTH COHESIONLESS AND COHESIVE SOILS USING REESE AND O'NEILL'S METHOD (1999)

PROJECT: I-5 HOV Segment 3 — ITEM: 5-feet Diameter CIDH Pile
PROJECT NO.: 11-137 — Overhead Sign Structure SS7-1
DONE BY: SP — DATE: 11/06/12
CHECKED BY: — DATE:

D_s => 5.00 ft — Hw => 0.0 ft — Hg => 0.0 ft
p = 15.71 ft — Ho => 0.0 ft — Hf => 0.0 ft

Hw is the depth to water table.
Hg is the depth to ground surface used in calculation.
Ho is the depth to location starting initial overburden pressu
Hf is the depth to location starting accumulation of skin fricti

Soil Layer =>	from Top	to Bottom =>	γ (pcf)=>	γ' (pcf)=	c' (ksf) =>	ϕ' =>	
(1)	0	5	120	57.6	0.00	0	Clay
(2)	5	50	120	57.6	1.70	0	Clay

Sublayer No.	Depth, z (ft)	z_i (ft)	New Depth (ft)	Soil Unit Weight (pcf)	Overburden Pressure (psf)	c' (ksf)	β	f_i^s (ksf)	ΔQ_s^s (kips)	Q_s^s (kips)	f_i^c (ksf)	ΔQ_s^c (kips)	Q_s^c (kips)	Q_s (kips)
0	0.0	0.0	0.0	57.6	0									
1	0.0	0.0	0.0	57.6	0	0.0				0.0			0.0	0.0
2	1.0	0.5	0.5	57.6	29	0.0	1.20	0.00	0.0	0.0	0.00	0.0	0.0	0.0
3	2.0	1.5	1.5	57.6	86	0.0	1.20	0.00	0.0	0.0	0.00	0.0	0.0	0.0
4	3.0	2.5	2.5	57.6	144	0.0	1.20	0.00	0.0	0.0	0.00	0.0	0.0	0.0
5	4.0	3.5	3.5	57.6	202	0.0	1.20	0.00	0.0	0.0	0.00	0.0	0.0	0.0
6	5.0	4.5	4.5	57.6	259	0.0	1.20	0.00	0.0	0.0	0.00	0.0	0.0	0.0
7	6.0	5.5	5.5	57.6	317	1.7	1.18	0.00	0.0	0.0	0.94	14.7	14.7	14.7
8	7.0	6.5	6.5	57.6	374	1.7	1.16	0.00	0.0	0.0	0.94	14.7	29.4	29.4
9	8.0	7.5	7.5	57.6	432	1.7	1.13	0.00	0.0	0.0	0.94	14.7	44.1	44.1
10	9.0	8.5	8.5	57.6	490	1.7	1.11	0.00	0.0	0.0	0.94	14.7	58.7	58.7
11	10.0	9.5	9.5	57.6	547	1.7	1.08	0.00	0.0	0.0	0.94	14.7	73.4	73.4
12	11.0	10.5	10.5	57.6	605	1.7	1.06	0.00	0.0	0.0	0.94	14.7	88.1	88.1
13	12.0	11.5	11.5	57.6	662	1.7	1.04	0.00	0.0	0.0	0.94	14.7	102.8	102.8
14	13.0	12.5	12.5	57.6	720	1.7	1.02	0.00	0.0	0.0	0.94	14.7	117.5	117.5
15	14.0	13.5	13.5	57.6	778	1.7	1.00	0.00	0.0	0.0	0.94	14.7	132.2	132.2
16	15.0	14.5	14.5	57.6	835	1.7	0.99	0.00	0.0	0.0	0.94	14.7	146.9	146.9
17	16.0	15.5	15.5	57.6	893	1.7	0.97	0.00	0.0	0.0	0.94	14.7	161.6	161.6
18	17.0	16.5	16.5	57.6	950	1.7	0.95	0.00	0.0	0.0	0.94	14.7	176.2	176.2
19	18.0	17.5	17.5	57.6	1008	1.7	0.94	0.00	0.0	0.0	0.94	14.7	190.9	190.9
20	19.0	18.5	18.5	57.6	1066	1.7	0.92	0.00	0.0	0.0	0.94	14.7	205.6	205.6
21	20.0	19.5	19.5	57.6	1123	1.7	0.90	0.00	0.0	0.0	0.94	14.7	220.3	220.3

Caltrans Standard Plan Soil Profile

```
================================================================================

                    LPile Plus for Windows, Version 6.0 (6.0.09)

                  Analysis of Individual Piles and Drilled Shafts
                  Subjected to Lateral Loading Using the p-y Method

                            (c) 1985-2010 by Ensoft, Inc.
                                All Rights Reserved

================================================================================

This program is licensed to:

Earth Mechanics, Inc.
Earth Mechanics, Inc.

--------------------------------------------------------------------------------
                              Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations:      X:\Projects\2011\11-137 - TRC, I-5 HOV Widening - Segment 3\Reports\08 Sign Structures\Analysis\
Name of input data file:     Lateral Soil Profile from CT Standard Plans.lp6d
Name of output file:         Lateral Soil Profile from CT Standard Plans.lp6o
Name of plot output file:    Lateral Soil Profile from CT Standard Plans.lp6p
Name of runtime file:        Lateral Soil Profile from CT Standard Plans.lp6r

--------------------------------------------------------------------------------
                           Date and Time of Analysis
--------------------------------------------------------------------------------

               Date:  November 6, 2012      Time:  12:05:52

--------------------------------------------------------------------------------
                                 Problem Title
--------------------------------------------------------------------------------

Project Name:
Job Number:
Client:
Engineer:
Description:


--------------------------------------------------------------------------------
                                 Program Options
--------------------------------------------------------------------------------

Units Used - US Customary Units: pounds, inches, feet

Basic Program Options:

This analysis computes nonlinear bending stiffness and nominal Moment
  Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No p-y curves to be computed and output for user-specified depths

Solution Control Parameters:
- Number of pile increments                      =          100
- Maximum number of iterations allowed           =          100
- Deflection tolerance for convergence           =   1.0000E-05  in
- Maximum allowable deflection                   =     100.0000  in

Pile Response Output Options:
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points)  = 1

--------------------------------------------------------------------------------
                      Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Total Number of Sections                        =         1

Total Pile Length                               =      23.00 ft

Depth of ground surface below top of pile       =       0.00 ft
```

Slope angle of ground surface = 0.00 deg.

Pile dimensions used for p-y curve computations defined using 2 points.
p-y curves are computed using values of pile diameter interpolated over the length of the pile.

Point	Depth X ft	Pile Diameter in
1	0.00000	60.0000000
2	23.000000	60.0000000

Input Structural Properties:

Section No. 1:

Section Type	=	Elastic Pile
Cross-sectional Shape	=	Circular
Section Length	=	23.000 in
Top Width	=	60.000 in
Bottom Width	=	60.000 in
Top Area	=	2827.000000 sq. in
Bottom Area	=	2827.000000 sq. in
Moment of Inertia at Top	=	3.181E+05 in^4
Moment of Inertia at Bottom	=	3.181E+05 in^4
Elastic Modulus	=	3200000.000 lbs/in

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.000 ft
Distance from top of pile to bottom of layer = 50.000 ft
p-y subgrade modulus k for top of soil layer = 70.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 70.000 lbs/in**3

(Depth of lowest layer extends 27.00 ft below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 2 points

Point No.	Depth X ft	Eff. Unit Weight pcf
1	0.00	120.00000
2	50.00	120.00000

Summary of Soil Properties

Layer Num.	Soil Type (p-y Curve Criteria)	Depth ft	Eff. Unit Wt., pcf	Cohesion psf	Friction Ang., deg.	qu psi	RQD percent	Epsilon 50	kpy pci	Rock Emass psi	krm	Test Type	Test Prop.	Elas. Subgr. pci
1	Sand (Reese, et al.)	0.00	120.000	--	30.000	--	--	--	70.000	--	--	--	--	--
		50.000	120.000	--	30.000	--	--	--	70.000	--	--	--	--	--

Loading Type

---

p-y criteria for static loading was used for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs
1	1	V = 36000.000 lbs	M = 10234800.000 in-lbs	80000.000

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust values were determined from pile-head loading conditions

Number of Sections = 1

Section No. 1:

Moment-Curvature properties derived from elastic section properties

---

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (BC Type 1)

Horizontal shear force at pile head = 36000.000 lbs
Applied moment at pile head = 10234800.000 in-lbs
Axial thrust load on pile head = 80000.000 lbs

Depth X inches	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/in	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3986	10234800.	36000.	-0.003148	993.5847	1.018E+12	0.000	0.000	0.000
2.760	0.3900	10334852.	35928.	-0.003120	1003.9210	1.018E+12	-52.3599	370.5822	0.000
5.520	0.3814	10434499.	35710.	-0.003092	1012.4191	1.018E+12	-105.7698	765.4209	0.000
8.280	0.3729	10533334.	35343.	-0.003063	1021.7406	1.018E+12	-160.0249	1184.4251	0.000
11.040	0.3645	10630943.	34825.	-0.003035	1030.9466	1.018E+12	-214.9219	1627.4717	0.000
13.800	0.3561	10726910.	34156.	-0.003006	1039.9975	1.018E+12	-270.2587	2094.4019	0.000
16.560	0.3478	10820810.	33333.	-0.002976	1048.8537	1.018E+12	-325.8353	2585.0180	0.000
19.320	0.3397	10912223.	32357.	-0.002947	1057.4752	1.018E+12	-381.4530	3099.0793	0.000
22.080	0.3316	11000723.	31228.	-0.002917	1065.8220	1.018E+12	-436.9151	3636.2982	0.000
24.840	0.3236	11085888.	29946.	-0.002887	1073.8543	1.018E+12	-492.0268	4196.3354	0.000
27.600	0.3157	11167299.	28512.	-0.002857	1081.5325	1.018E+12	-546.5954	4778.7954	0.000
30.360	0.3078	11244539.	26929.	-0.002827	1088.8173	1.018E+12	-600.6846	5385.4996	0.000
33.120	0.3001	11317196.	25195.	-0.002796	1095.6699	1.018E+12	-655.8194	6031.8571	0.000
35.880	0.2924	11384851.	23310.	-0.002765	1102.0508	1.018E+12	-710.3534	6704.9137	0.000
38.640	0.2848	11447089.	21275.	-0.002734	1107.9206	1.018E+12	-764.1363	7404.7573	0.000
41.400	0.2773	11503498.	19112.	-0.002703	1113.2408	1.018E+12	-803.6590	7998.4800	0.000
44.160	0.2698	11553778.	16851.	-0.002672	1117.9829	1.018E+12	-834.3060	8531.7120	0.000
46.920	0.2626	11597697.	14510.	-0.002641	1122.1250	1.018E+12	-862.3707	9064.9440	0.000
49.680	0.2553	11635039.	12094.	-0.002609	1125.6469	1.018E+12	-887.9044	9598.1760	0.000
52.440	0.2482	11665610.	9611.9887	-0.002578	1128.5302	1.018E+12	-910.9583	10131.	0.000
55.200	0.2411	11689235.	7069.2803	-0.002546	1130.7584	1.018E+12	-931.5840	10665.	0.000
57.960	0.2341	11705757.	4472.9255	-0.002514	1132.3166	1.018E+12	-949.8325	11198.	0.000
60.720	0.2272	11715036.	1829.4143	-0.002482	1133.1918	1.018E+12	-965.7553	11731.	0.000
63.480	0.2204	11716951.	-854.9047	-0.002451	1133.3724	1.018E+12	-979.4034	12264.	0.000
66.240	0.2137	11711399.	-3573.8235	-0.002419	1132.8488	1.018E+12	-990.8276	12798.	0.000
69.000	0.2071	11698292.	-6321.2740	-0.002387	1131.6126	1.018E+12	-1000.0786	13331.	0.000
71.760	0.2005	11677560.	-9091.3278	-0.002355	1129.6573	1.018E+12	-1007.2067	13864.	0.000
74.520	0.1941	11649148.	-11878.	-0.002324	1126.9776	1.018E+12	-1012.2618	14397.	0.000
77.280	0.1877	11613018.	-14676.	-0.002292	1123.5701	1.018E+12	-1015.2935	14930.	0.000
80.040	0.1814	11569147.	-17480.	-0.002261	1119.4325	1.018E+12	-1016.3507	15464.	0.000
82.800	0.1752	11517528.	-20284.	-0.002230	1114.5640	1.018E+12	-1015.4819	15997.	0.000
85.560	0.1691	11458165.	-23083.	-0.002198	1108.9653	1.018E+12	-1012.7350	16530.	0.000
88.320	0.1631	11391081.	-25872.	-0.002167	1102.6383	1.018E+12	-1008.1572	17063.	0.000
91.080	0.1571	11316311.	-28645.	-0.002137	1095.5864	1.018E+12	-1001.7950	17597.	0.000
93.840	0.1513	11233903.	-31399.	-0.002106	1087.8142	1.018E+12	-993.6941	18130.	0.000
96.600	0.1455	11143916.	-34128.	-0.002076	1079.3273	1.018E+12	-983.8995	18663.	0.000
99.360	0.1398	11046432.	-36828.	-0.002046	1070.1310	1.018E+12	-972.4551	19196.	0.000
102.120	0.1342	10941531.	-39496.	-0.002016	1060.2394	1.018E+12	-959.4042	19730.	0.000
104.880	0.1287	10829316.	-42122.	-0.001986	1049.6559	1.018E+12	-944.7888	20263.	0.000
107.640	0.1232	10709897.	-44707.	-0.001957	1038.3930	1.018E+12	-928.6501	20796.	0.000
110.400	0.1179	10583397.	-47246.	-0.001928	1026.4623	1.018E+12	-911.0282	21329.	0.000

113.160	0.1126	10449951.	-49734.	-0.001900	1013.8765	1.018E+12	-891.9620	21863.	0.000
115.920	0.1074	10309705.	-52167.	-0.001872	1000.6492	1.018E+12	-871.4894	22396.	0.000
118.680	0.1023	10162813.	-54543.	-0.001844	986.7953	1.018E+12	-849.6468	22929.	0.000
121.440	0.0972	10009443.	-56856.	-0.001816	972.3304	1.018E+12	-826.4698	23462.	0.000
124.200	0.0922	9849772.	-59103.	-0.001790	957.2711	1.018E+12	-801.9924	23995.	0.000
126.960	0.0873	9683985.	-61281.	-0.001763	941.6351	1.018E+12	-776.2474	24529.	0.000
129.720	0.0825	9512280.	-63386.	-0.001737	925.4408	1.018E+12	-749.2662	25062.	0.000
132.480	0.0778	9334861.	-65415.	-0.001711	908.7077	1.018E+12	-721.0790	25595.	0.000
135.240	0.0731	9151943.	-67365.	-0.001686	891.4560	1.018E+12	-691.7145	26128.	0.000
138.000	0.0684	8963751.	-69232.	-0.001662	873.7068	1.018E+12	-661.1998	26662.	0.000
140.760	0.0639	8770517.	-71013.	-0.001638	855.4821	1.018E+12	-629.5610	27195.	0.000
143.520	0.0594	8572482.	-72706.	-0.001614	836.8046	1.018E+12	-596.8221	27728.	0.000
146.280	0.0550	8369895.	-74306.	-0.001591	817.6978	1.018E+12	-563.0063	28261.	0.000
149.040	0.0506	8163015.	-75812.	-0.001569	798.1861	1.018E+12	-528.1347	28795.	0.000
151.800	0.0463	7952106.	-77220.	-0.001547	778.2944	1.018E+12	-492.2274	29328.	0.000
154.560	0.0421	7737444.	-78528.	-0.001526	758.0487	1.018E+12	-455.3025	29861.	0.000
157.320	0.0379	7519308.	-79732.	-0.001505	737.4754	1.018E+12	-417.3770	30394.	0.000
160.080	0.0338	7297988.	-80830.	-0.001485	716.6018	1.018E+12	-378.4660	30927.	0.000
162.840	0.0297	7073781.	-81820.	-0.001466	695.4560	1.018E+12	-338.5833	31461.	0.000
165.600	0.0257	6846981.	-82698.	-0.001447	674.0664	1.018E+12	-297.7412	31994.	0.000
168.360	0.0217	6617928.	-83462.	-0.001428	652.4626	1.018E+12	-255.9503	32527.	0.000
171.120	0.0178	6386912.	-84109.	-0.001411	630.6745	1.018E+12	-213.2197	33060.	0.000
173.880	0.0139	6154267.	-84638.	-0.001394	608.7328	1.018E+12	-169.5571	33594.	0.000
176.640	0.0101	5920328.	-85044.	-0.001377	586.6690	1.018E+12	-124.9688	34127.	0.000
179.400	0.006327	5685432.	-85326.	-0.001362	564.5151	1.018E+12	-79.4593	34660.	0.000
182.160	0.002591	5449929.	-85481.	-0.001347	542.3037	1.018E+12	-33.0321	35193.	0.000
184.920	-0.001106	5214170.	-85507.	-0.001332	520.0684	1.018E+12	14.3112	35727.	0.000
187.680	-0.004763	4978517.	-85401.	-0.001318	497.8430	1.018E+12	62.5699	36260.	0.000
190.440	-0.008382	4743338.	-85161.	-0.001305	475.6623	1.018E+12	111.7451	36793.	0.000
193.200	-0.0120	4509007.	-84783.	-0.001293	453.5616	1.018E+12	161.8389	37326.	0.000
195.960	-0.0155	4275906.	-84266.	-0.001281	431.5769	1.018E+12	212.8547	37859.	0.000
198.720	-0.0190	4044424.	-83607.	-0.001269	409.7449	1.018E+12	264.7974	38393.	0.000
201.480	-0.0225	3814957.	-82803.	-0.001259	388.1029	1.018E+12	317.6728	38926.	0.000
204.240	-0.0260	3587908.	-81852.	-0.001249	366.6889	1.018E+12	371.4879	39459.	0.000
207.000	-0.0294	3363686.	-80751.	-0.001239	345.5416	1.018E+12	426.2507	39992.	0.000
209.760	-0.0328	3142709.	-79498.	-0.001230	324.7004	1.018E+12	481.9704	40526.	0.000
212.520	-0.0362	2925402.	-78089.	-0.001222	304.2053	1.018E+12	538.6570	41059.	0.000
215.280	-0.0396	2712196.	-76523.	-0.001215	284.0969	1.018E+12	596.3213	41592.	0.000
218.040	-0.0429	2503531.	-74796.	-0.001207	264.4169	1.018E+12	654.9750	42125.	0.000
220.800	-0.0462	2299455.	-72906.	-0.001201	245.2073	1.018E+12	714.6304	42659.	0.000
223.560	-0.0495	2101620.	-70850.	-0.001195	226.5110	1.018E+12	775.3007	43192.	0.000
226.320	-0.0528	1909290.	-68625.	-0.001190	208.3716	1.018E+12	836.9992	43725.	0.000
229.080	-0.0561	1723335.	-66228.	-0.001185	190.8334	1.018E+12	899.7400	44258.	0.000
231.840	-0.0594	1544233.	-63657.	-0.001180	173.9415	1.018E+12	963.5373	44791.	0.000
234.600	-0.0626	1372469.	-60908.	-0.001176	157.7418	1.018E+12	1028.4058	45325.	0.000
237.360	-0.0659	1208539.	-57979.	-0.001173	142.2808	1.018E+12	1094.3600	45858.	0.000
240.120	-0.0691	1052945.	-54866.	-0.001170	127.6061	1.018E+12	1161.4146	46391.	0.000
242.880	-0.0723	906197.	-51566.	-0.001167	113.7657	1.018E+12	1229.5842	46924.	0.000
245.640	-0.0755	768815.	-48077.	-0.001165	100.8086	1.018E+12	1298.8831	47458.	0.000
248.400	-0.0788	641326.	-44395.	-0.001163	88.7847	1.018E+12	1369.3253	47991.	0.000
251.160	-0.0820	524269.	-40550.	-0.001161	77.7445	1.018E+12	1416.6486	47707.	0.000
253.920	-0.0852	418003.	-36575.	-0.001160	67.7221	1.018E+12	1463.9306	47445.	0.000
256.680	-0.0884	322888.	-32469.	-0.001159	58.7514	1.018E+12	1511.4583	47211.	0.000
259.440	-0.0916	239286.	-28231.	-0.001158	50.8666	1.018E+12	1559.2476	47003.	0.000
262.200	-0.0948	167562.	-23861.	-0.001158	44.1020	1.018E+12	1607.3118	46817.	0.000
264.960	-0.0979	108082.	-19359.	-0.001157	38.4922	1.018E+12	1655.6625	46653.	0.000
267.720	-0.1011	61214.	-14722.	-0.001157	34.0719	1.018E+12	1704.3095	46507.	0.000
270.480	-0.1043	27329.	-9950.3574	-0.001157	30.8760	1.018E+12	1753.2606	46379.	0.000
273.240	-0.1075	6798.8317	-5043.3771	-0.001157	28.9398	1.018E+12	1802.5222	46266.	0.000
276.000	-0.1107	0.000	0.000	-0.001157	28.2985	1.018E+12	1852.0989	23084.	0.000

* This analysis makes computations of pile response using nonlinear moment-curvature relationships.
  The above values of total stress are computed for combined axial stress and do not equal the
  actual stresses in concrete and steel in the range of nonlinear bending.

Output Verification: Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =      0.3986124 inches
Computed slope at pile head      =     -0.0031477 radians
Maximum bending moment           =      11716951. inch-lbs
Maximum shear force              =        -85507. lbs
Depth of maximum bending moment  =     63.4800000 inches below pile head
Depth of maximum shear force     =    184.9200000 inches below pile head
Number of iterations             =              8
Number of zero deflection points =              1
```

Summary of Pile Response(s)

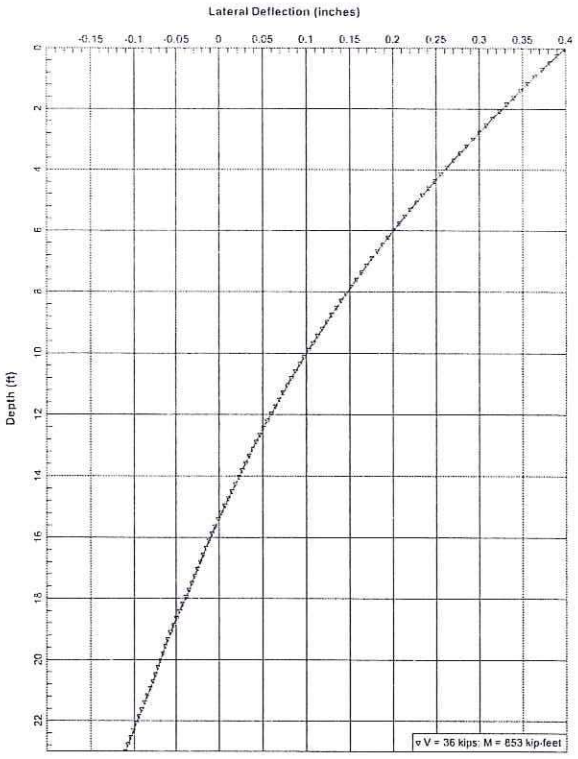
Definitions of Pile-Head Loading Conditions:

Load Type 1: Load 1 = Shear, lbs, and Load 2 = Moment, in-lbs

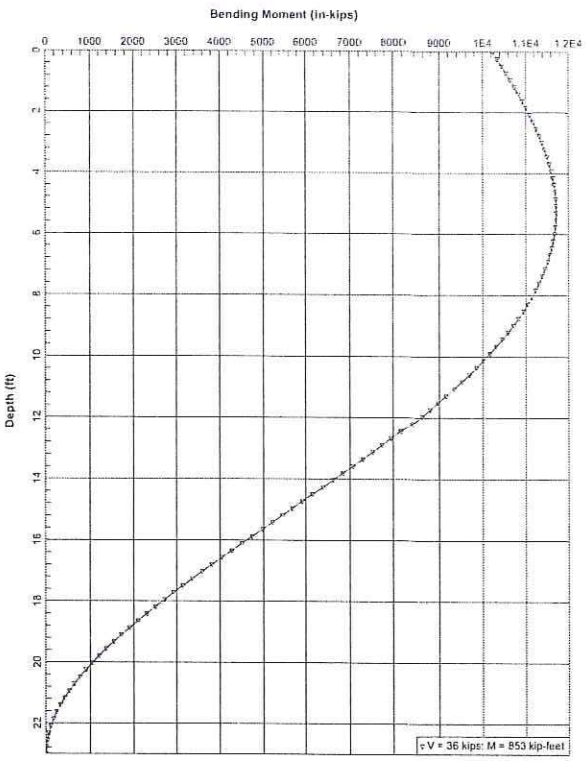
Load Type 2: Load 1 = Shear, lbs, and Load 2 = Slope, radians
Load Type 3: Load 1 = Shear, lbs, and Load 2 = Rotational Stiffness, in-lbs/radian
Load Type 4: Load 1 = Top Deflection, inches, and Load 2 = Moment, in-lbs
Load Type 5: Load 1 = Top Deflection, inches, and Load 2 = Slope, radians

Load Case No.	Load Type No.	Condition 1 V(lbs) or y(inches)	Condition 2 in-lb, rad., or in-lb/rad.	Axial Load lbs	Pile-Head Deflection inches	Maximum Moment in-lbs	Maximum Shear lbs	Pile-Head Rotation radians
1	1	V = 36000.	M = 10234800.	80000.	0.39861245	11716951.	-85507.	0.00000000

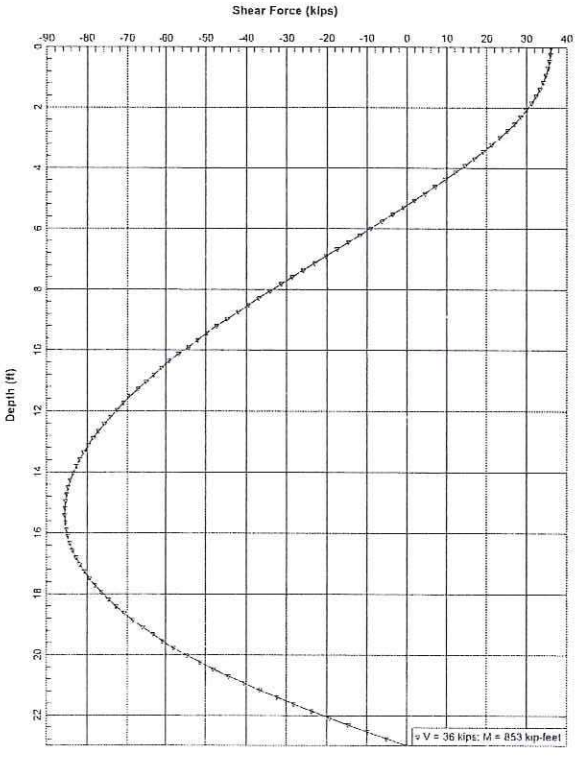
The analysis ended normally.



I-5 HOV Segment 3 Sign Structures - Using Caltrans Standard Plan Soil Profile



I-5 HOV Segment 3 Sign Structures - Using Caltrans Standard Plan Soil Profile



I-5 HOV Segment 3 Sign Structures - Using Caltrans Standard Plan Soil Profile

Existing Soil Profile at Sign SS7-1

LPile Plus for Windows, Version 6.0 (6.0.09)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Earth Mechanics, Inc.
Earth Mechanics, Inc.

Files Used for Analysis

Path to file locations: X:\Projects\2011\11-137 - TRC, I-5 HOV Widening - Segment 3\Reports\08 Sign Structures\Analysis\
Name of input data file: Lateral Soil Profile at SS7-1.lp6d
Name of output file: Lateral Soil Profile at SS7-1.lp6o
Name of plot output file: Lateral Soil Profile at SS7-1.lp6p
Name of runtime file: Lateral Soil Profile at SS7-1.lp6r

Date and Time of Analysis

Date: November 6, 2012 Time: 14:44:08

Problem Title

Project Name:
Job Number:
Client:
Engineer:
Description:

Program Options

Units Used - US Customary Units: pounds, inches, feet

Basic Program Options:

This analysis computes nonlinear bending stiffness and nominal Moment
Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No p-y curves to be computed and output for user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in

Pile Response Output Options:
- Values of pile-head deflection, bending moment, shear force, and
soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1

Pile Structural Properties and Geometry

Total Number of Sections = 1

Total Pile Length = 23.00 ft

Depth of ground surface below top of pile = 0.00 ft

Slope angle of ground surface = 0.00 deg.

Pile dimensions used for p-y curve computations defined using 2 points.
p-y curves are computed using values of pile diameter interpolated over the length of the pile.

Point	Depth X ft	Pile Diameter in
1	0.00000	60.0000000
2	23.000000	60.0000000

Input Structural Properties:

Section No. 1:

Section Type	=	Elastic Pile
Cross-sectional Shape	=	Circular
Section Length	=	23.000 in
Top Width	=	60.000 in
Bottom Width	=	60.000 in
Top Area	=	2827.000000 sq. in
Bottom Area	=	2827.000000 sq. in
Moment of Inertia at Top	=	3.181E+05 in^4
Moment of Inertia at Bottom	=	3.181E+05 in^4
Elastic Modulus	=	3200000.000 lbs/in

Ground Slope and Pile Batter Angles

Ground Slope Angle	=	0.000 degrees
	=	0.000 radians
Pile Batter Angle	=	0.000 degrees
	=	0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	0.000 ft
Distance from top of pile to bottom of layer	=	50.000 ft

(Depth of lowest layer extends 27.00 ft below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 2 points

Point No.	Depth X ft	Eff. Unit Weight pcf
1	0.00	120.00000
2	50.00	120.00000

Summary of Soil Properties

Layer Num.	Soil Type (p-y Curve Criteria)	Depth ft	Eff. Unit Wt., pcf	Cohesion psf	Friction Ang., deg.	qu psi	RQD percent	Epsilon 50	kpy pci	Rock Emass psi	krm	Test Type	Test Prop.	Elas. Subgr. pci
1	Soft Clay	0.00	120.000	1700.000	--	--	--	0.00700	--	--	--	--	--	--
		50.000	120.000	1700.000	--	--	--	0.00700	--	--	--	--	--	--

Loading Type

p-y criteria for static loading was used for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs
1	1	V = 36000.000 lbs	M = 10234800.000 in-lbs	80000.000

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust values were determined from pile-head loading conditions

Number of Sections = 1

Section No. 1:

Moment-Curvature properties derived from elastic section properties

---

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (BC Type 1)

Horizontal shear force at pile head = 36000.000 lbs
Applied moment at pile head = 10234800.000 in-lbs
Axial thrust load on pile head = 80000.000 lbs

Depth X inches	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/in	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3615	10234800.	36000.	-0.002973	993.5847	1.018E+12	-744.6589	2843.0171	0.000
2.760	0.3533	10331977.	33939.	-0.002945	1002.7498	1.018E+12	-748.6724	5848.8432	0.000
5.520	0.3452	10423445.	31868.	-0.002917	1011.3766	1.018E+12	-752.5041	6016.5569	0.000
8.280	0.3372	10509174.	29786.	-0.002888	1019.4620	1.018E+12	-756.1524	6189.3867	0.000
11.040	0.3293	10589137.	27694.	-0.002860	1027.0037	1.018E+12	-759.6155	6367.5560	0.000
13.800	0.3214	10663308.	25593.	-0.002831	1033.9990	1.018E+12	-762.8917	6551.3018	0.000
16.560	0.3136	10731660.	23483.	-0.002802	1040.4456	1.018E+12	-765.9789	6740.8750	0.000
19.320	0.3059	10794171.	21365.	-0.002773	1046.3413	1.018E+12	-768.8753	6936.5422	0.000
22.080	0.2983	10850819.	19239.	-0.002744	1051.6839	1.018E+12	-771.5788	7138.5865	0.000
24.840	0.2908	10901582.	17106.	-0.002714	1056.4717	1.018E+12	-774.0871	7347.3092	0.000
27.600	0.2833	10946443.	14966.	-0.002685	1060.7026	1.018E+12	-776.3982	7563.0313	0.000
30.360	0.2760	10985382.	12821.	-0.002655	1064.3752	1.018E+12	-778.5094	7786.0952	0.000
33.120	0.2687	11018385.	10669.	-0.002625	1067.4878	1.018E+12	-780.4184	8016.8663	0.000
35.880	0.2615	11045436.	8512.9585	-0.002595	1070.0391	1.018E+12	-782.1226	8255.7357	0.000
38.640	0.2544	11066522.	6352.2351	-0.002565	1072.0278	1.018E+12	-783.6190	8503.1221	0.000
41.400	0.2473	11081633.	4187.6722	-0.002535	1073.4530	1.018E+12	-784.9048	8759.4746	0.000
44.160	0.2404	11090758.	2019.8555	-0.002505	1074.3136	1.018E+12	-785.9769	9025.2758	0.000
46.920	0.2335	11093889.	-150.6205	-0.002475	1074.6089	1.018E+12	-786.8318	9301.0447	0.000
49.680	0.2267	11091019.	-2323.1516	-0.002445	1074.3382	1.018E+12	-787.4661	9587.3411	0.000
52.440	0.2200	11082145.	-4497.1238	-0.002415	1073.5012	1.018E+12	-787.8760	9884.7690	0.000
55.200	0.2134	11067262.	-6671.9122	-0.002385	1072.0976	1.018E+12	-788.0575	10194.	0.000
57.960	0.2068	11046369.	-8846.8802	-0.002355	1070.1271	1.018E+12	-788.0063	10516.	0.000
60.720	0.2004	11019467.	-11021.	-0.002325	1067.5898	1.018E+12	-787.7176	10851.	0.000
63.480	0.1940	10986558.	-13195.	-0.002295	1064.4860	1.018E+12	-787.1867	11200.	0.000
66.240	0.1877	10947645.	-15366.	-0.002265	1060.8161	1.018E+12	-786.4081	11564.	0.000
69.000	0.1815	10902716.	-17535.	-0.002236	1056.5805	1.018E+12	-785.3762	11944.	0.000
71.760	0.1754	10851838.	-19701.	-0.002206	1051.7800	1.018E+12	-784.0846	12341.	0.000
74.520	0.1693	10794960.	-21863.	-0.002177	1046.4156	1.018E+12	-782.5268	12757.	0.000
77.280	0.1633	10732114.	-24020.	-0.002148	1040.4884	1.018E+12	-780.6955	13192.	0.000
80.040	0.1575	10663316.	-26172.	-0.002119	1033.9997	1.018E+12	-778.5828	13648.	0.000
82.800	0.1516	10588579.	-28318.	-0.002090	1026.9511	1.018E+12	-776.1801	14127.	0.000
85.560	0.1459	10507924.	-30456.	-0.002061	1019.3441	1.018E+12	-773.4780	14631.	0.000
88.320	0.1403	10421371.	-32587.	-0.002033	1011.1809	1.018E+12	-770.4665	15161.	0.000
91.080	0.1347	10328942.	-34709.	-0.002005	1002.4636	1.018E+12	-767.1342	15720.	0.000
93.840	0.1292	10230664.	-36821.	-0.001977	993.1945	1.018E+12	-763.4690	16310.	0.000
96.600	0.1238	10126563.	-38923.	-0.001949	983.3764	1.018E+12	-759.4572	16934.	0.000
99.360	0.1184	10016671.	-41013.	-0.001922	973.0120	1.018E+12	-755.0840	17596.	0.000
102.120	0.1132	9901021.	-43090.	-0.001895	962.1046	1.018E+12	-750.3329	18299.	0.000
104.880	0.1080	9779650.	-45154.	-0.001868	950.6576	1.018E+12	-745.1855	19048.	0.000
107.640	0.1029	9652596.	-47203.	-0.001842	938.6746	1.018E+12	-739.6212	19847.	0.000
110.400	0.0978	9519902.	-49236.	-0.001816	926.1597	1.018E+12	-733.6173	20702.	0.000
113.160	0.0928	9381614.	-51252.	-0.001790	913.1172	1.018E+12	-727.1479	21619.	0.000
115.920	0.0879	9237782.	-53249.	-0.001765	899.5518	1.018E+12	-720.1840	22607.	0.000

118.680	0.0831	9088457.	-55227.	-0.001740	885.4084	1.018E+12	-712.6925	23674.	0.000
121.440	0.0783	8933699.	-57183.	-0.001716	870.8724	1.018E+12	-704.6359	24832.	0.000
124.200	0.0736	8773567.	-59115.	-0.001692	855.7698	1.018E+12	-695.9709	26094.	0.000
126.960	0.0690	8608129.	-61023.	-0.001668	840.1666	1.018E+12	-686.6476	27475.	0.000
129.720	0.0644	8437454.	-62905.	-0.001645	824.0696	1.018E+12	-676.6078	28995.	0.000
132.480	0.0599	8261621.	-64757.	-0.001623	807.4860	1.018E+12	-665.7832	30679.	0.000
135.240	0.0554	8080711.	-66579.	-0.001600	790.4236	1.018E+12	-654.0927	32558.	0.000
138.000	0.0511	7894813.	-68367.	-0.001579	772.8909	1.018E+12	-641.4391	34671.	0.000
140.760	0.0467	7704025.	-70118.	-0.001558	754.8968	1.018E+12	-627.7043	37071.	0.000
143.520	0.0425	7508450.	-71830.	-0.001537	736.4514	1.018E+12	-612.7423	39826.	0.000
146.280	0.0382	7308203.	-73498.	-0.001517	717.5653	1.018E+12	-596.3700	43032.	0.000
149.040	0.0341	7103409.	-75119.	-0.001497	698.2503	1.018E+12	-578.3522	46824.	0.000
151.800	0.0300	6894205.	-76688.	-0.001478	678.5194	1.018E+12	-558.3793	51397.	0.000
154.560	0.0259	6680743.	-78198.	-0.001460	658.3869	1.018E+12	-536.0206	57055.	0.000
157.320	0.0219	6463194.	-79643.	-0.001442	637.8690	1.018E+12	-510.7056	64288.	0.000
160.080	0.0180	6241751.	-81012.	-0.001425	616.9838	1.018E+12	-481.5115	73958.	0.000
162.840	0.0141	6016636.	-82294.	-0.001408	595.7523	1.018E+12	-447.6002	87749.	0.000
165.600	0.0102	5788113.	-83469.	-0.001392	574.1903	1.018E+12	-404.5543	109518.	0.000
168.360	0.006174	5556504.	-84508.	-0.001377	552.3553	1.018E+12	-348.4633	150884.	0.000
171.120	0.002395	5322238.	-85348.	-0.001362	530.2607	1.018E+12	-260.1320	276714.	0.000
173.880	-0.001145	5085986.	-85440.	-0.001348	507.9789	1.018E+12	193.3461	466013.	0.000
176.640	-0.004847	4851205.	-84725.	-0.001335	485.8357	1.018E+12	324.9879	185065.	0.000
179.400	-0.008512	4618896.	-83731.	-0.001322	463.9257	1.018E+12	394.8926	128042.	0.000
182.160	-0.0121	4389593.	-82548.	-0.001310	442.2991	1.018E+12	447.6798	101755.	0.000
184.920	-0.0157	4163697.	-81272.	-0.001298	420.9940	1.018E+12	491.5574	86190.	0.000
187.680	-0.0193	3941543.	-79863.	-0.001287	400.0417	1.018E+12	529.8565	75742.	0.000
190.440	-0.0228	3723423.	-78353.	-0.001277	379.4699	1.018E+12	564.2972	68176.	0.000
193.200	-0.0264	3509600.	-76752.	-0.001267	359.3034	1.018E+12	595.8967	62406.	0.000
195.960	-0.0298	3300314.	-75066.	-0.001258	339.5647	1.018E+12	625.3114	57842.	0.000
198.720	-0.0333	3095789.	-73302.	-0.001249	320.2751	1.018E+12	652.9922	54129.	0.000
201.480	-0.0367	2896236.	-71464.	-0.001241	301.4545	1.018E+12	679.2635	51040.	0.000
204.240	-0.0401	2701856.	-69554.	-0.001233	283.1217	1.018E+12	704.3673	48426.	0.000
207.000	-0.0435	2512840.	-67577.	-0.001226	265.2948	1.018E+12	728.4892	46181.	0.000
209.760	-0.0468	2329372.	-65534.	-0.001220	247.9912	1.018E+12	751.7753	44228.	0.000
212.520	-0.0503	2151629.	-63428.	-0.001213	231.2276	1.018E+12	774.3424	42514.	0.000
215.280	-0.0536	1979784.	-61261.	-0.001208	215.0201	1.018E+12	796.2860	40995.	0.000
218.040	-0.0569	1814003.	-59034.	-0.001203	199.3846	1.018E+12	817.6847	39637.	0.000
220.800	-0.0602	1654450.	-56748.	-0.001198	184.3365	1.018E+12	838.6045	38416.	0.000
223.560	-0.0635	1501284.	-54405.	-0.001194	169.8908	1.018E+12	859.1011	37311.	0.000
226.320	-0.0668	1354661.	-52006.	-0.001190	156.0622	1.018E+12	879.2220	36306.	0.000
229.080	-0.0701	1214735.	-49552.	-0.001186	142.8652	1.018E+12	899.0081	35387.	0.000
231.840	-0.0734	1081657.	-47044.	-0.001183	130.3141	1.018E+12	918.4950	34543.	0.000
234.600	-0.0766	955575.	-44482.	-0.001180	118.4228	1.018E+12	937.7134	33765.	0.000
237.360	-0.0798	836635.	-41868.	-0.001178	107.2051	1.018E+12	956.6905	33046.	0.000
240.120	-0.0832	724983.	-39202.	-0.001176	96.6747	1.018E+12	975.4502	32377.	0.000
242.880	-0.0864	620761.	-36484.	-0.001174	86.8450	1.018E+12	994.0135	31755.	0.000
245.640	-0.0896	524110.	-33715.	-0.001173	77.7295	1.018E+12	1012.3992	31174.	0.000
248.400	-0.0929	435171.	-30896.	-0.001171	69.3413	1.018E+12	1030.6241	30630.	0.000
251.160	-0.0961	354083.	-28026.	-0.001170	61.6935	1.018E+12	1048.7029	30119.	0.000
253.920	-0.0993	280983.	-25107.	-0.001169	54.7992	1.018E+12	1066.6491	29639.	0.000
256.680	-0.1026	216008.	-22139.	-0.001169	48.6711	1.018E+12	1084.4747	29186.	0.000
259.440	-0.1058	159294.	-19121.	-0.001168	43.3222	1.018E+12	1102.1902	28759.	0.000
262.200	-0.1090	110976.	-16055.	-0.001168	38.7652	1.018E+12	1119.8054	28354.	0.000
264.960	-0.1122	71188.	-12940.	-0.001168	35.0126	1.018E+12	1137.3289	27971.	0.000
267.720	-0.1154	40064.	-9776.6422	-0.001167	32.0772	1.018E+12	1154.7684	27607.	0.000
270.480	-0.1187	17737.	-6565.5213	-0.001167	29.9714	1.018E+12	1172.1308	27262.	0.000
273.240	-0.1219	4338.1512	-3306.5779	-0.001167	28.7077	1.018E+12	1189.4224	26933.	0.000
276.000	-0.1251	0.000	0.000	-0.001167	28.2985	1.018E+12	1206.6486	13310.	0.000

* This analysis makes computations of pile response using nonlinear moment-curvature relationships.
  The above values of total stress are computed for combined axial stress and do not equal the
  actual stresses in concrete and steel in the range of nonlinear bending.

Output Verification: Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	0.3614573 inches
Computed slope at pile head	=	-0.0029732 radians
Maximum bending moment	=	11093889. inch-lbs
Maximum shear force	=	-85440. lbs
Depth of maximum bending moment	=	46.9200000 inches below pile head
Depth of maximum shear force	=	173.8800000 inches below pile head
Number of iterations	=	28
Number of zero deflection points	=	1

Summary of Pile Response(s)

Definitions of Pile-Head Loading Conditions:

Load Type 1: Load 1 = Shear, lbs, and Load 2 = Moment, in-lbs
Load Type 2: Load 1 = Shear, lbs, and Load 2 = Slope, radians
Load Type 3: Load 1 = Shear, lbs, and Load 2 = Rotational Stiffness, in-lbs/radian

Load Type 4: Load 1 = Top Deflection, inches, and Load 2 = Moment, in-lbs
Load Type 5: Load 1 = Top Deflection, inches, and Load 2 = Slope, radians

Load Case No.	Load Type No.	Condition 1 y(lbs) or y(inches)	Condition 2 in-lb, rad., or in-lb/rad.	Axial Load lbs	Pile-Head Deflection inches	Maximum Moment in-lbs	Maximum Shear lbs	Pile-Head Rotation radians
1	1	Y = 16000.	M = 10234800.	80000.	0.36145730	11093889.	-85440.	0.00000000

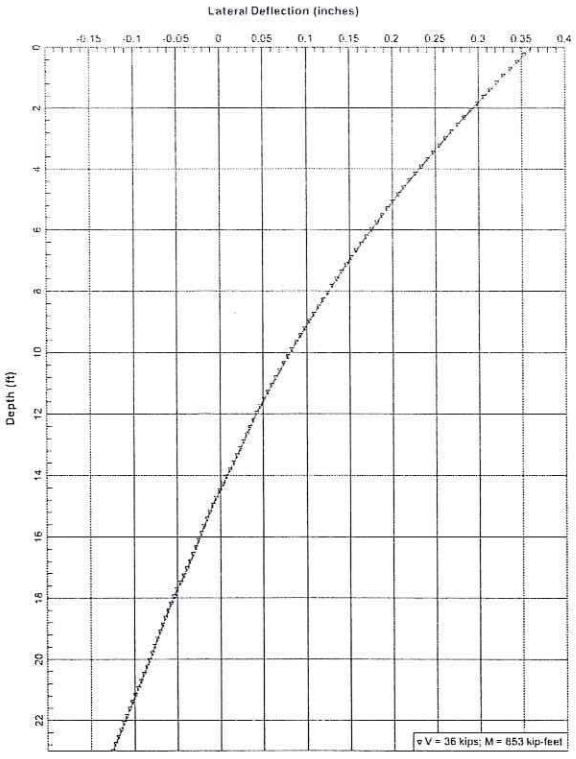
Summary of Warning Messages

---

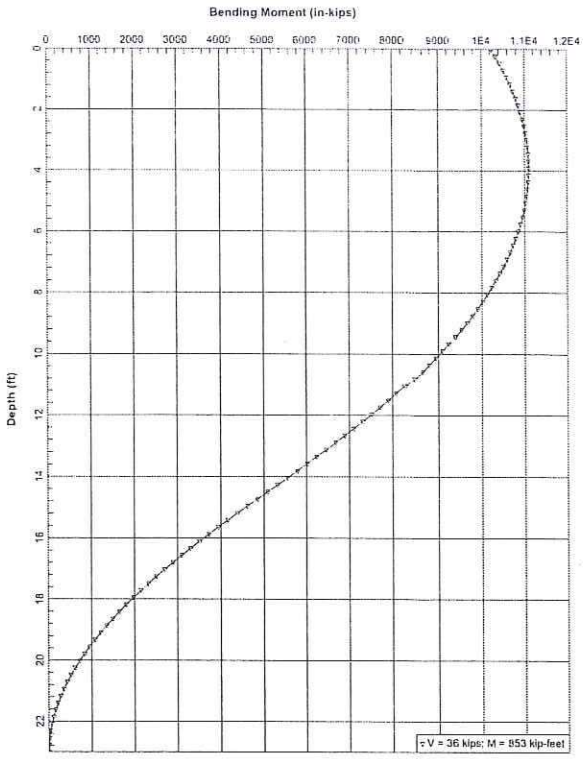
the following warning was reported 2727 times

**** Warning ****

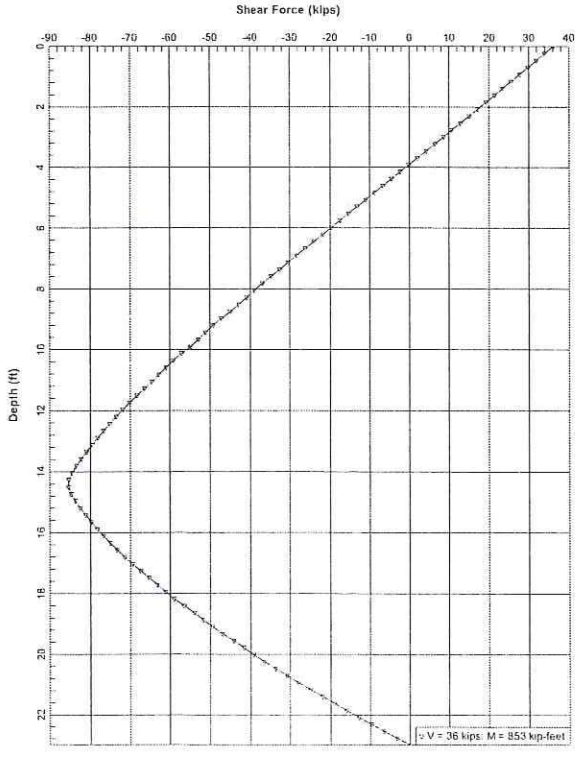
An unreasonable input value for shear strength has been specified for a soil defined using the soft clay criteria. The input value is greater than 8.68 psi (1,250 psf). You should check your input data for correctness.



I-5 HOV Segment 3 Sign Structures - Using Sign Structure SS7-1 Soil Profile

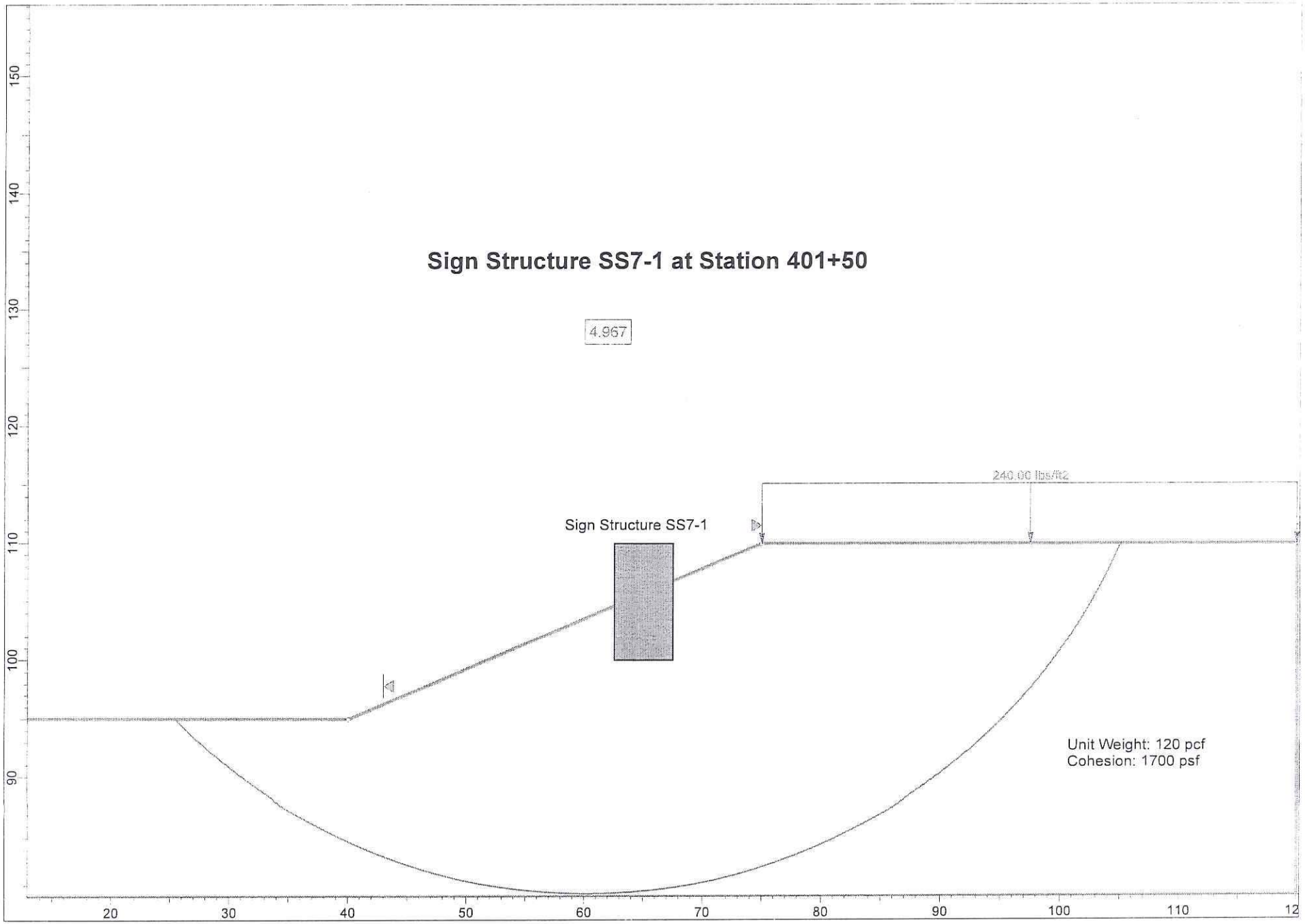


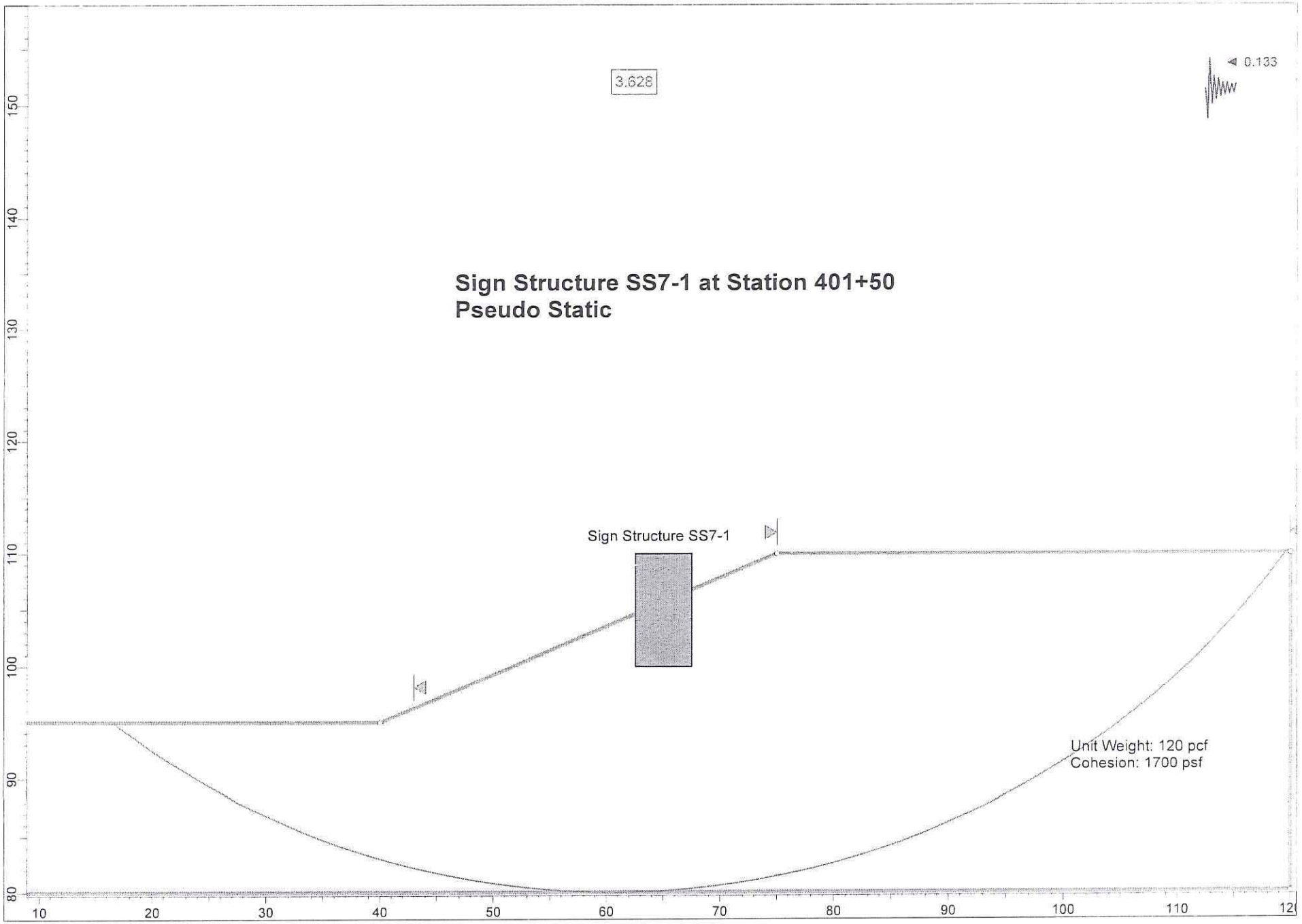
I-5 HOV Segment 3 Sign Structures - Using Sign Structure SS7-1 Soil Profile



I-5 HOV Segment 3 Sign Structures - Using Sign Structure SS7-1 Soil Profile

Sign Structure SS7-1 at Station 401+50




3.628
0.133
Sign Structure SS7-1 at Station 401+50
Pseudo Static
Sign Structure SS7-1
Unit Weight: 120 pcf
Cohesion: 1700 psf
150
140
130
120
110
100
90
80
10
20
30
40
50
60
70
80
90
100
110