FOUNDATION REPORT
MID-COAST MULTI-MODAL TRAIL PHASE 2
PEDESTRIAN OVERCROSSING AND RETAINING WALL
SAN MATEO COUNTY, CALIFORNIA

For

BKF ENGINEERS
255 Shoreline Drive, Suite 200
Redwood City, CA 94065

PARIKH CONSULTANTS, INC.
2360 Qume Drive, Suite A
San Jose, CA 95131

December 3, 2019

Job No. 2017-108-GEO

TABLE OF CONTENTS **PAGE**

<u>**LIST OF PLATES**</u>

FOUNDATION REPORT
MID-COAST MULTI-MODAL TRAIL PHASE 2
PEDESTRIAN OVERCROSSING AND RETAINING WALL
SAN MATEO COUNTY, CALIFORNIA

1.0 SCOPE OF WORK

This report presents the results of the geotechnical engineering investigation for the proposed Mid-Coast Multi-Modal Trail Phase 2 - Pedestrian Bridge and Retaining Wall (RW) (Project) to be constructed in San Mateo County, California. The Project site is adjacent to Highway 1 in an unincorporated area between the coastal villages of Miramar and El Granada, northwest of the City of Half Moon Bay. The approximate Project location is shown on the Project Location Map - Plate No. 1.

The purpose of this investigation was to evaluate the general soil and groundwater conditions at the Project site, to evaluate their engineering properties, and to provide foundation design recommendations for the proposed Project. The scope of work performed for this investigation included a review of the readily available geologic and geotechnical literature pertaining to the site, obtaining representative soil samples and logging materials encountered in the exploratory borings, laboratory testing of the collected samples, engineering analysis of the field and laboratory data, and preparation of this report.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used directly as specifications. These recommendations should not be used directly for bidding purposes or for construction cost estimates.

2.0 PROJECT DESCRIPTION

The County of San Mateo plans to construct a pedestrian and bicycle commuter trail adjacent to Highway 1 through the Mid-Coast. A pedestrian overcrossing and a retaining wall are required as part of the proposed trail. The overcrossing and retaining wall will be designed to meet current AASHTO and Caltrans standards. Based on available information, the pedestrian overcrossing will be a lightly loaded, prefabricated structure that measures 80 feet in length and 12 feet in width. Driven pile foundations for the bridge are planned. The pedestrian bridge is in the county's right-of-way and does not have a Caltrans Bridge Number.

The proposed retaining wall is in Caltrans jurisdiction and supports the trail along Highway 1. Originally, a conventional cast-in-place (CIP) concrete wall was proposed for the retaining wall using Caltrans standard plans or with modification. Due to the saturated and very soft surficial ground in the retaining wall area, shoring and over-excavation would be required. The CIP wall does not appear to be cost efficient. Therefore, a soldier pile wall is proposed to substitute for the CIP wall. In addition, a vehicular barrier rail is required between the shoulder of the highway and the pedestrian path, which requires soil parameters for rail foundation design.

Per information provided by the designer, the vertical datum of the project survey was based on NAVD 88. The horizontal bearings were based on NAD 83, California coordinate system of 1983, Epoch 2011.

3.0 EXCEPTIONS TO POLICIES AND PROCEDURES

None.

4.0 FIELD INVESTIGATION AND FIELD TESTING PROGRAM

Four borings (R-17-001 through R-17-004) were drilled to depths from approximately 31.5 to 51.5 feet below grade with a track-mounted drill rig on March 13 and 14, 2017. Hollow-stem augers and mud rotary wash drilling method were used. Selected soil samples were obtained from either a 2.5-inch I.D. Modified California (MC) or 1.4-inch I.D. Standard Penetration Test (SPT) sampler at various depths. The samplers were driven into subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Log of Test Borings (LOTB) in Appendix A.

The drilling subcontractor was Geo-Ex Subsurface Exploration from Dixon, California. Based on the hammer energy calibration information provided by Geo-Ex Subsurface Exploration, the hammer energy of the drill rig (CME 45) used is approximately 75 percent. Using a method suggested by Daniel, Howie, and Sy (2003), when correlating standard penetration data, the blow counts for the MC sampler may be converted to equivalent SPT blow counts by multiplying a conversion factor of 0.65. The soil samples were sealed and transported to our laboratory for further evaluation and testing. The field investigation was conducted under the supervision of the field engineer who logged the test borings and prepared the samples for subsequent laboratory testing and evaluation. The approximate boring locations are shown on the Site Plan - Plate No. 2 and on the Log of Test Boring sheets.

5.0 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected soil samples to evaluate the physical and engineering properties of soils. The test types performed for this study included:

- Moisture Content (ASTM D 2216)
- Atterberg Limits (ASTM D 4318)
- Grain Size (ASTM D 422)
- Unconfined Compression (ASTM D 2166)
- Corrosion (California Test Methods 643/417/422)

The corrosion tests were performed by Sunland Analytical in Rancho Cordova, California.

6.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

6.1 Site Geology

General geologic features pertaining to the Project site were evaluated by reference to Geologic Data Map No. 2 of the California Geologic Survey (CGS, 2010). Based on the publication, the Project site is mostly underlain by the following Quaternary geologic unit:

Qoa - Older Pleistocene to Holocene alluvium, lake, playa, and terrace deposits.

A portion of the published Geologic Map covering the Project site is attached as Plate No. 3.

6.2 Subsurface Conditions

The subsurface soil conditions are based on the field exploration. Based on an available topographic map, the top elevations of Borings R-17-001 and R-17-002 are around 72 feet, and Borings R-17-003 and R-17-004 are around 38 and 36 feet, respectively.

In general, the borings encountered alluvial materials consisting of interbedded lean clay, sandy lean clay, silty sand, and clayey sand to the maximum depths drilled, ranging from approximately 31.5 feet in the RW area to 51.5 feet in the Pedestrian Bridge location. The apparent densities of the sandy soils mostly vary from loose to dense, and the consistencies of the clayey soils mostly vary from soft to very stiff.

The boring logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs. The abrupt stratum changes shown on these logs may be gradual and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain a properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

7.0 GROUNDWATER

Groundwater was encountered at depths of about 13.5 and 12.5 feet respectively in Borings R-17-001 and R-17-002, and 12 feet in Borings R-17-003 and R-17-004 during drilling. Groundwater may vary with the passage of time due to seasonal groundwater fluctuation, local irrigation practice, water level in the stream, tide of the ocean, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of investigation.

Groundwater elevation could significantly vary in the event of a ‘normal’ rainfall period or following an El Nino event. Also, groundwater may take time to recharge or react to such changes and therefore seasonal fluctuations or the extreme conditions as noted above may or may not affect the groundwater immediately following such event. Therefore, it is all the more important to not rely on such transient measurements of groundwater for the design and construction of any underground improvements. It may be prudent to make conservative assumptions in the design and construction program.

8.0 AS-BUILT FOUNDATION DATA

The proposed Pedestrian Bridge and RW are new structures. No as-built foundation data is available.

9.0 SCOUR EVALUATION

The proposed Pedestrian Bridge crosses over Arroyo De En Medio, a small, intermittent coastal stream that discharges to the Pacific Ocean to the west. A hydraulic study, dated August 18, 2017, was conducted by BKF Engineers. Bridge abutments should be set back adequate distances to protect from potential scour along the stream banks. Stream bank protection measures may be required along the upstream and downstream ends of the abutments. The project hydraulic report indicates that the channel geometry will not be modified, and the new bridge will not result in any scouring of the existing channel. Therefore, scour should not be a design concern. No open water course passes by the retaining wall alignment.

10.0 CORROSION EVALUATION

The corrosion investigation for this Project was performed on selected soil samples in general accordance with the provisions of California Test Methods 643, 417 and 422. A summary of the corrosion test results is presented in Table 10.1. Caltrans (Version 3.0, March 2018) considers a site corrosive when one or more of the following conditions exist:

- The pH is 5.5 or less.
- The soil contains a chloride concentration of 500 ppm or greater.
- The soil contains a sulfate concentration of 1,500 ppm or greater.

TABLE 10.1 - CORROSION TEST RESULTS

Boring No.	Depth (ft)	pH	Minimum Resistivity (ohms-cm)	Chloride Content (ppm)	Sulfate Content (ppm)
R-17-001	3	6.27	3,750	9.5	6.0
R-17-002	11	6.45	3,220	30.0	14.3
R-17-004	6	6.86	2,220	31.8	44.4

Based on the corrosion test results, the on-site subsurface soils are considered non-corrosive. The guidelines presented in the California Amendments to the AASHTO LRFD Bridge Design Specifications (BDS, 2012), Article 5.12.3, for a minimum cement factor and cover thickness may be used for the substructure.

11.0 SEISMIC DESIGN INFORMATION AND RECOMMENDATIONS

11.1 Seismic Sources

The Project site is located in a seismically active part of northern California. Many faults in the region are capable of producing earthquakes, which may cause strong ground shaking at the Project site. The proposed bridge is located at coordinates of approximately 37.4955 degrees north latitude and 122.4558 degrees west longitude (Google Earth, 2015). The Caltrans Fault Database (V2b, 2012) and Acceleration Response Spectrum (ARS) Online Report (V2, 2012) contain known active faults (if there is evidence of surface displacement in the past 700,000 years) in the State. Based on the Caltrans ARS Online Report, the information of active faults in the region which would have more impact on the site is summarized in Table 11.1. The maximum magnitudes (M_{max}**)** represent the largest earthquake that a fault is capable of generating and are related to the seismic moment. The attached Caltrans ARS Online Map - Plate No. 4 presents the location of the fault system relative to the Project site.

TABLE 11.1 - CALTRANS ARS ONLINE INFORMATION

Fault	Fault ID	Maximum Magnitude, M_{max}	Fault Type	Approx. Distance R_{rup}/R_x (miles)
San Gregorio Fault (San Gregorio Section)	127	7.4	SS	1.82/1.82
San Andreas (Peninsula) 2011 CFM	134	8.0	SS	5.26/5.26
Hayward (South)	137	7.3	SS	23.60/23.56

R_{rup} = Closest distance to the fault rupture plane
R_x = Horizontal distance to the fault trace or surface projection of the top of rupture plane
SS = Strike-slip fault

11.2 Seismic Design Criteria

The Caltrans ARS Online program (V2, 2012) was used for producing acceleration response spectra. Development of ARS curves is based on several input parameters, including site location (longitude/latitude), average shear wave velocity for the top 100 feet of soils (V_{s30}), and other site parameters, such as fault characteristics and site-to-fault distances. The design methods incorporate both deterministic and probabilistic seismic hazards to produce the design response spectrum. The probabilistic method represents a 5 percent in 50 years probability of exceedance (975-year return period). The controlling spectrum (upper envelope) is adopted for the design response spectrum.

The shear wave velocities of the top 100 feet of soils at the Project site were estimated by using the established correlations and guidelines in Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (2012). An average shear wave velocity of 200 m/s was adopted. According to the Caltrans guidelines, the spectral acceleration values corresponding to periods of one second and greater have been increased by 20 percent to account for the near fault effect, and linearly tapered to zero at 0.5 sec. No adjustment is required for the basin effect. The Acceleration Response Spectrum Comparison Curves are presented on Plate No. 5A, and the Recommended ARS Curve is presented on Plate No. 5B.

Based on the boring data, the site soil is classified as “Marginal Soil” per Caltrans Seismic Design Criteria (SDC) 1.7. This has been updated and categorized as “S2” per SDC 2.0.

11.3 Seismic Hazards

Faulting

The site is located outside the designated State of California Alquist-Priolo Earthquake Fault Zones for active faulting and no mapped evidence of active or potentially active faulting was found for the site. The potential for fault rupture at the site appears to be low.

Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and low-plastic silts of low relative density are the type of soils that usually are susceptible to liquefaction. Clay is generally not susceptible to liquefaction. According to the AASHTO BDS guidelines (2012), sand and non-plastic silt with corrected SPT blow count $(N_1)_{60}$ less than or equal to 25 are susceptible to liquefaction. The liquefaction potential was evaluated according to procedures proposed by Youd, et al. (2001).

By using the Caltrans ARS Online program (V2, 2012), the peak ground acceleration (PGA) at the site was estimated to be 0.60g, and the earthquake magnitude was estimated to be 8.0, representing a hazardous level of 5 percent exceedance in 50 years. The above seismic parameters were incorporated into the liquefaction analysis. Based on the two soil borings drilled for the pedestrian bridge, the loose to medium dense, saturated sand encountered at depths from approximately 18 to 23 feet as well as from 43 to 48 feet in Boring R-17-001, and from 13 to 28 feet in Boring R-17-002 could be potentially liquefiable. The post-liquefaction settlement is estimated to be about 3 to 4 inches. The liquefaction analysis results are contained in Appendix C. Post-liquefaction settlement may induce down drag forces on deep foundations and should be accounted for during design. The potentially liquefiable soils encountered between 43 and 48 feet deep in R-17-001 is relatively thin (~5 feet thick) and overlain by about 20 feet thick of non-liquefiable materials (from 23 to 43 feet deep). The liquefaction impact on the foundation from the soils between 43 and 48 feet deep in R-17-001 is expected to be minor. Nevertheless, the frictional resistance of this layer of soil has been conservatively ignored under the liquefaction condition. Potentially liquefiable soils were generally not encountered in the two borings (R-17-003 and R-17-004) drilled in the retaining wall area.

Lateral Spreading

Liquefaction induced lateral spreading is a phenomenon in which the gently sloping ground displaces laterally as a result of pore pressure build-up or liquefaction in a shallow subsurface layer during an earthquake. This phenomenon is also observed when a flat ground with liquefiable underlying deposits is adjacent to an open face such as channel bank or approach embankment.

Based on a plan and profile provided (2017), the approach embankments of the bridge will generally match with the existing grade. The channel geometry will not be modified. The stream banks generally have a gradient of 6H:1V (horizontal to vertical).

Per Caltrans recently published Memo to Designers (MTD) 20-15 (May 2017), pseudo-static slope stability analysis should be performed under liquefied conditions. The liquefied soils should be assigned with residual shear strength, Sr, in combination of a horizontal seismic coefficient input, Kh (coupling). The site with a pseudo-static factor of safety (FS) equal to or greater than 1.10 shall be considered to have adequate stability. The search limits for the critical failure surface should be extended laterally to about 4 times the slope height from the slope crest, and vertically to about 1.2 times the slope height from the toe of the slope.

Slope stability analysis on the stream banks was performed using the Slope/W program by Geo-Slope International (2007). A horizontal seismic coefficient (k_h) of 0.2g, equal to 1/3 PGA (PGA = 0.6g) at the site in accordance with Caltrans Guidelines for Structure Foundation Reports (2009), was adopted for the pseudo-static slope stability analysis. The pseudo-static slope stability analysis produced factors of safety less than 1.0 with both circular and block searches, indicating that a flow liquefaction with relatively large ground deformation exists at the site (see Appendix C).

Mitigation of Lateral Spreading Impact

Either structure enhancement or ground improvement may be employed to address the lateral spreading issue. For project design, structure enhancement approach using CISS piles with 5/8-inch wall thickness was selected as the preferred option for limiting the effect of lateral spreading. Ground/soil improvement option was not selected for the project.

The pedestrian bridge is a relatively short, light-weight structure. During a seismic event, it is assumed that the soils at the two ends of the bridge will move in the same direction. The chance of a “squeezing” condition on the bridge is expected to be low. Based on engineering judgement and discussions with Caltrans, relative ground displacement of 1 foot under the liquefaction condition appears to be reasonable for the pedestrian bridge site with relatively gentle sloping ground and only two piles at each abutment. We have performed the LPILE analysis on a 30-inch cast-in-steel-shell (CISS) pile with wall thickness of 5/8-inch. A soil movement of 1 foot was input into the LPILE analysis. The pile stiffness was provided by the Designer. The resulting pile top deflection is approximately 1.6 inches, which is considered tolerable to the structure based on our discussions with the Designer. No additional shear force or moment are included. The LPILE analysis results are provided in Appendix C.

The soil crust would apply passive earth pressure on the abutment stem under the soil spreading. Once the liquefaction has occurred, cracks/fissures could develop in the crust overlying the liquefied soils due to shaking, settlement, tensile forces, etc. The crust soil shear strength could decrease considerably, and thus the “pushing” or passive load may also be reduced. Since the soil crust above the liquefied sand is relatively thin (about 8 to 9 feet thick), it is expected that the shear strength of the cracked crust could be reduced to the same residual shear strength of the underneath liquefied soils (about 250 psf). This residual shear strength of 250 psf may be used on the abutment stem width and depth for checking earth forces on the prefabricated structure. In our opinion, the soil crust earth pressure on the abutment stem may not occur at the same time as the soil flowing around the pile. The soil crust earth pressure may not need to be combined with soil movement when performing LPILE analysis under lateral liquefaction spreading condition (de-coupling). It is our understanding that the anticipated spread conditions are tolerable to the planned prefabricated structure.

Alternatively, ground improvement technique maybe considered to mitigate the lateral spreading. There are several liquefaction ground improvement methods such as dynamic compaction, stone column, deep soil mixing, compaction grouting, etc. In consideration of the location, size, and materials to be treated, compaction grouting appears to be workable for the site. The compaction grouting is a grouting technique, with which the low slump, low mobility aggregate grout is injected into the ground and columns of overlapping grout bulbs are created. The expansion of grout bulbs displaces surrounding soils and densifies the soils within the treatment zones. The soil treatment would focus on the potentially liquefiable soils only, approximately 20 feet thick. The preliminary proposed treatment area measures in width approximately 1 to 1.5 times the abutment width and in length 50 to 60 feet extending from the back of the abutment. For preliminary cost estimation, we have communicated with a grouting specialty contractor and the information has been provided to the Designer separately.

12.0 BRIDGE FOUNDATION RECOMMENDATIONS (COUNTY'S R/W)

12.1 General

This report was prepared specifically for the proposed Project as described earlier. Normal procedures were assumed for construction of the bridge structure throughout our analysis and represent one of the bases of recommendations presented herein. The design criteria have been based upon the materials encountered at the site. Therefore, this office should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

12.2 Foundations

Both driven cast-in-steel-shell (CISS) piles and cast-in-drilled-hole (CIDH) concrete piles were considered for the proposed pedestrian overcrossing. CISS piles were selected in consideration of better lateral resistance and convenient installation for the site with liquefaction and lateral spreading potential. Per Caltrans MTD 3-1 (2014), the design of deep foundations is performed using the LRFD method in accordance with the California Amendments to the AASHTO BDS (2012). Loads from the LRFD Strength and Extreme Event limit states are used for estimating the pile tip elevation. A minimum center-to-center pile spacing of three times the pile diameter is recommended. Both axial and lateral pile capacities should be analyzed during design. The pertinent foundation design information provided by the Designer, including Foundation Design Data, Foundation Design Loads, and scour data, is tabulated in Tables 12.1, 12.2, and 12.3.

TABLE 12.1 - FOUNDATION DESIGN DATA

Support No.	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	No. of Piles per Support
					B	L		
Abut 1	LRFD	30 x 0.625" CISS	71.3	64.3	3.5	15	1	2
Abut 2	LRFD	30 x 0.625" CISS	71.3	62.7	3.5	15	1	2

TABLE 12.2 - FOUNDATION FACTORED DESIGN LOADS

Support No.	Service-I Limit State (kips)			Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load		Perm. Loads	Compression (ϕ_{qs}=0.7)		Tension		Compression (ϕ_{qs}=1.0)		Tension	
	Per Support	Max. Per Pile	Per Support	Per Support*	Max. Per Pile*	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	180	90	120	320	160	n/a	n/a	120	60	n/a	n/a
Abut 2	180	90	130	340	170	n/a	n/a	130	65	n/a	n/a

*Per Designer, the values include a resistance factor of φ_{qs} = 0.7 at the strength/construction limit state.

TABLE 12.3 - SCOUR DATA

Support No.	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)[1]
Abut 1	No change	Not applicable
Abut 2	No change	Not applicable

12.3 Axial Pile Capacity

The axial pile capacity was estimated using computer program APILE by Ensoft, Inc. (2007) with the built-in Revised Recommended Practice 2A (RP 2A) method per American Petroleum Institute (API). The API method utilizes a K factor (K=0.8) for cohesionless soils and α factors for cohesive soils in combination with friction angles or undrained shear strengths of soils and effective overburden pressures to estimate skin friction. The pile capacity is primarily derived from skin friction on the pile surface and the end bearing capacity is not included. The soil resistance of the loose sands was ignored. Note that we have conservatively neglected the capacity contribution of the loose sands for both Extreme Event and Strength Limit States for the project. The APILE input soil data are presented in Tables 12.6 and 12.7.

The internal friction angles of sands were estimated by reference to the empirical correlation between the soil friction angle and the energy corrected SPT blow count (N_{60}) (Coduto, 1999). The undrained shear strengths of clay were estimated based on laboratory test results and correlation with N_{60} recommended by US Army Corps of Engineering (1992). Residual shear strengths were assigned to liquefied soils in accordance with MTD 20-15 (2017). Under the design service load, pile settlement is estimated to be less than 0.25 inches.

Due to the potentially liquefiable soils and anticipated settlement, down drag forces caused by the liquefied soils are considered in the upper ~20 feet of the piles assuming an adhesion of 250 psf (the residual soil strength, Sr) along the pile shafts. The liquefaction down drag load is counted in addition to the structural demand for the Extreme Event Limit State design. The pile tip should be extended below the bottom of potentially liquefied soils. The computer printouts of the APILE analysis are provided in Appendix C. The design tip elevations are determined by the Designer based on the lateral pile capacity analysis. The design recommendations are presented in Table 12.4. The Pile Data Table is shown in Table 12.5.

Note that in the Pile Data Table the Driving Resistance is estimated from normal soil condition without liquefaction and is intended for use in the field for pile installation. The Nominal Resistance for design is estimated conservatively assuming liquefaction condition.

TABLE 12.4 - FOUNDATION DESIGN RECOMMENDATIONS

Support No.	Pile Type	Cut-off Elev. (ft)	Service-I Limit State Load (kips) per Support		Total Permissible Support Settlement (in)	Nominal Resistance (kips)			
						Strength/Constr.		Extreme Event	
			Total	Perm.		Comp.* (φ_{qs}=0.7)	Tension (φ_{qs}=0.7)	Comp. (φ_{qs}=1.0)	Tension (φ_{qs}=1.0)
Abut 1	30 x 0.625" CISS	64.3	180	120	1	160	n/a	60	n/a
Abut 2	30 x 0.625" CISS	62.7	180	130	1	175	n/a	65	n/a

*The values include a resistance factor of φ_{qs} = 0.7 at the strength/construction limit state.

TABLE 12.5 – PILE DATA TABLE

Support No.	Pile Type	Nominal Resistance (kips)		Design Tip Elev. (ft)	Specified Tip Elev. (ft)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	30 x 0.625" CISS	160	0	27.0 (1); 20.5 (3)	20.5	240
Abut 2	30 x 0.625" CISS	175	0	23.0 (1); 18.0 (3)	18.0	225

Design tip elevation is controlled by the following demands: (1) Compression, (2) Tension, and (3) Lateral Load (determined by the Designer).

12.4 Lateral Pile Capacity

The lateral pile capacity analysis will be performed by the Designer using the LPILE program. The geotechnical soil parameters presented in Tables 12.5 and 12.6 are adopted for lateral pile capacity analysis. Per the California amendments to AASHTO BDS (2012) for group effect, *p*-multipliers of 0.8, 0.9 and 1.0 can be used for a single row of piles spacing of 2.5, 3 and 4 times the pile diameter, respectively, if the load direction is perpendicular to the row. The *y*-multiplier is taken as 1.0.

The minimum horizontal distance between the top near edge of the pile to the slope face should be 9 feet. The allowable pile top displacement is generally limited to be within 0.25 inches under service loads. However, the final allowable pile top movement maybe determined by the structure designer considering overall bridge behavior.

TABLE 12.6 - LPILE PARAMETERS Abutment 1 (Boring R-17-002, northwest)

Approx. Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	Soil Strength	K (pci)	E_{50} (in/in)	Effective Unit Wt. (pcf)
72 to 64	Lean Clay	Mod. Stiff Clay w/o Free Water	C = 800 psf	Default	Default	125
64 to 59	Clayey Sand	Sand (Reese) (no liquefaction)	$\phi = 30°$	Default	N/A	62
		Soft Clay (Matlock) (liquefied)	200 psf	N/A	0.05	62
59 to 54	Silty Sand	Sand (Reese)	$\phi = 30°$	Default	N/A	62
		Soft Clay (Matlock) (liquefied)	C = 275 psf	N/A	0.05	62
54 to 49	Clayey Sand	Sand (Reese) (no liquefaction)	$\phi = 28°$	Default	N/A	62

Approx. Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	Soil Strength	K (pci)	E_{50} (in/in)	Effective Unit Wt. (pcf)
		Soft Clay (Matlock) (liquefied)	200 psf	N/A	0.05	62
49 to 44	Silty Sand	Sand (Reese) (no liquefaction)	$\phi = 34°$	Default	N/A	62
		Mod. Stiff Clay w/o Free Water (liquefied)	700 psf	Default	Default	62
44 to 39	Lean Clay	Stiff Clay w/o Free Water (Reese)	C = 2,500 psf	N/A	Default	62
39 to 34	Lean Clay	Mod. Stiff Clay w/o Free Water	C = 650 psf	Default	Default	62
34 to 29	Lean Clay	Stiff Clay w/o Free Water (Reese)	C = 1,750 psf	N/A	Default	62
29 to 25.5	Lean Clay	Stiff Clay w/o Free Water (Reese)	C = 4,000 psf	N/A	Default	62
25.5 to 20.5	Clayey Sand	Sand (Reese)	$\phi = 38°$	Default	Default	62

TABLE 12.7 - LPILE PARAMETERS Abutment 2 (Boring R-17-001, southeast)

Approx. Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	Soil Strength	K (pci)	E_{50} (in/in)	Effective Unit Wt. (pcf)
72 to 63	Lean Clay	Mod. Stiff Clay w/o Free Water	C = 550 psf	Default	Default	125
63 to 58	Clayey Sand	Sand (Reese) (no liquefaction)	$\phi = 30°$	Default	N/A	62
		Soft Clay (Matlock) (liquefied)	250 psf	N/A	0.05	62
58 to 54	Sandy Lean Clay	Mod. Stiff Clay w/o Free Water	C = 750 psf	Default	Default	62
54 to 49	Silty Sand	Sand (Reese) (no liquefaction)	$\phi = 28°$	Default	N/A	62
		Soft Clay (Matlock) (liquefied)	225 psf	N/A	0.05	62
49 to 44	Lean Clay	Mod. Stiff Clay w/o Free Water	C = 650 psf	Default	Default	62
44 to 39	Lean Clay	Stiff Clay w/o Free Water (Reese)	C = 1,000 psf	N/A	Default	62
39 to 29	Silty Sand	Sand (Reese)	$\phi = 38°$	Default	N/A	62

Approx. Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	Soil Strength	K (pci)	E_{50} (in/in)	Effective Unit Wt. (pcf)
29 to 24	Clayey Sand	Sand (Reese)	$\phi = 30°$	Default	N/A	62
		Soft Clay (Matlock) (liquefied)	400 psf	N/A	0.05	62
24 to 20.5	Silty Sand	Sand (Reese)	$\phi = 36°$	Default	N/A	62

12.5 Lateral Earth Pressures

Abutment and wing walls should be designed to resist the following applied lateral earth pressures. As requested by the Designer, an incremental seismic active pressure is also provided. These values assume no hydrostatic pore pressure buildup behind the walls. The walls should be provided with permanent drains in accordance with Caltrans standards to prevent the buildup of hydrostatic pressures. Backfill materials should conform to the structure backfill requirements contained in Section 19 of the Caltrans Standard Specifications (2015).

- Active Condition — 36 pcf Equivalent Fluid Pressure (EFP).
- At-Rest Condition — 55 pcf EFP.
- Passive Resistance — 3 ksf (nominal) for seismic design of the abutment backwall (5.5 feet high or greater); for activated height less than 5.5 feet, modify proportionally, i.e. 3×(H/5.5) ksf, according to the Caltrans SDC V1.7 (2013). A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive pressure is required.
- Traffic Load — 120 psf (based on an H10 vehicle).
- Incremental Active Seismic Pressure — 30 pcf EFP (in a regular triangular shape).

In case that the wall has to be designed for hydrostatic pore pressure, the recommended total seismic lateral earth pressure is 90 pcf in regular triangular distribution. The above value includes static pore water pressure.

The site PGA is 0.6 g. We followed AASHTO LRFD specs using 50% PGA as the design Kh (= 0.3 g). Assuming structure backfill of 34-35 deg and conservatively no cohesion, the total Kae is 0.5 (AASHTO LRFD Appendix A11). With a little bit of cohesion (say 50-100 psf) for seismic case per AASHTO, the total Kae can be in the range of 0.4 to 0.5. The static Ka is ~0.28, so the ΔKae is ~0.12 to 0.22. For drained case, the incremental earth pressure is about 125 x 0.22 (max) = 27 pcf (say, 30 pcf EFP for design). For undrained case, (125-62.4) pcf x ~0.45 = 28 pcf. With water pressure, the total lateral seismic pressure is ~90 pcf. The normal triangular distribution is per AASHTO LRFD Specifications Appendix A11.3.1 for routine wall design.

Cantilever walls which are free to rotate at least 0.004 radian may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharges (dead or live loads) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional horizontal earth pressure resulting from the surcharge for active and at-rest conditions, respectively. The horizontal earth pressure in front of the abutment walls should be ignored.

13.0 SOLDIER PILE RETAINING WALL (CALTRANS R/W)

Originally, a cast-in-place (CIP) concrete retaining wall on spread footing was planned for the pedestrian/bicycle path. Due to saturated soft materials in the wet ground in the retaining wall area, footing subgrade over-excavation and replacement is required. Since the wall is close to Highway 1, shoring is required for structure excavation. Using a CIP wall does not appear to be cost efficient. A soldier pile wall consisting of steel beam in drilled hole with lagging is proposed to replace the CIP wall.

The minimum diameter of drilled holes should be 2 feet and the minimum pile embedment below the design grade should be 10 feet. The center-to-center pile spacing is generally between 6 to 8 feet. The spacing, structure feasibility, and dimension of the soldier pile wall should be evaluated by the Designer. It is expected that the wall will be designed in accordance with the guidelines provided in the California amendments to AASHTO BDS (2012). According to the AASHTO, Article 11.6.5, a 50 percent reduction ratio can be applied to the PGA to obtain the horizontal seismic acceleration coefficient k_h. The resulting k_h equals 0.30g which will be used to estimate incremental lateral seismic earth pressure on the wall. The passive earth pressure coefficient is obtained based on the log-spiral method. The passive pressure should not exceed 3 ksf.

The soil parameters presented in Table 13.1 should be incorporated into the soldier pile wall design assuming that Caltrans structure backfill is used and in drained condition. Structure backfill materials should conform to the requirements contained in Section 19 of the Caltrans

Standard Specifications (2015).

TABLE 13.1 – SOIL PARAMETERS FOR SOLDIER PILES

Design Height of Wall	6 feet
Effective Soil Unit Weight	120 pcf (retained soil) 60 pcf (below the design grade)
Active Earth Pressure Coefficient, ka	0.33
Passive Earth Pressure Coefficient, kp	2.5 (top 5 feet of the design grade) 4.5 (below the top 5 feet of the design grade)
Arching Capacity Factor	2 (top 5 feet of the design grade) 3 (below the top 5 feet of the design grade)
Incremental Active Seismic Pressure	30 pcf (in a regular triangular shape)
Incremental Traffic Load	120 psf (based on an H10 vehicle)

It should be noted that the traffic load may or may not be the controlling load on the soldier pile wall. Whether or not to include the traffic load into design should be determined by the Designer. Any other surcharges should be considered by the Designer.

A geocomposite drain system should be installed for the drainage behind the soldier pile wall. Please refer to the requirements contained in Section 68-7 “Geocomposite Drain System” of the Caltrans Standard Specifications (2015) for materials and construction of the geocomposite wall drain.

14.0 VEHICULAR BARRIER RAIL

A vehicular barrier rail is required between the highway shoulder and the pedestrian path. Spread footings of 8 feet wide are proposed as a moment slab to support the rail that is designed for traffic impact load. The footing design should follow the guidelines in the Caltrans amendments to the AASHTO LRFD BDS (2012), Article 10.6 “Spreading Footing.” Since the soils at the design footing level (~3 feet below the highway grade) are mostly medium stiff clay as encountered in the soil borings, footing subgrade over-excavation and replacement is required to provide uniform support. The footing subgrade should be over-excavated a minimum of 2 feet and replaced with compacted Class 2 aggregate base (AB). The over-excavation and replacement should be extended to a minimum of 1 foot beyond the footing footprint on the pedestrian path side and can be aligned with the footing footprint on the highway side. A layer of Caltrans subgrade enhancement geotextile (SEG) Class 2 should be placed between the AB and subgrade to prevent contamination from subgrade fines. The SEG should wrap the top of AB a minimum of 2 feet on the pedestrian path side and 1 foot on the highway side. With such subgrade treatment, the following soil parameters can be used for footing design based on the design footing width of 8 feet.

- Nominal bearing capacity 5 ksf
- Friction coefficient 0.45 (between footing and AB)
- Nominal passive earth pressure 500 pcf (100% available per log-spiral method)

According to the AASHTO, resistance factors of 0.55 and 1.0 should be applied to the nominal bearing capacity at the strength limit state and extreme event limit state, respectively. Passive lateral earth pressure, if combined with the friction resistance, should not exceed 50 percent of the available passive resistance.

In our opinion, the clayey soils with a vertical cut not more than 5 feet in height can generally stand for temporary purpose if no adverse soil conditions such as loose, wet and soft materials are exposed during construction. If deeper excavation is needed, shoring or sloped cut would be required. Caltrans may have a minimum safety distance requirement between the edge of excavation and the traffic. Adjustment of traffic may be required during construction.

15.0 GRADING

All grading and compaction operations should be performed in accordance with the project specifications and Section 19 “Earthwork” of the Caltrans Standard Specifications (2015). A representative from this office or regulating agency should observe all excavated areas during grading and perform moisture and density tests on prepared subgrade and compacted fill material.

Areas to receive fill should be clean of vegetation, shrubs, trees, and their roots greater than 1 inch in diameter. Zones of soft, organic or saturated soils could be encountered during site grading. Loose materials will be left after the removal of large trees. Where such conditions are encountered, deeper excavation may be required to expose firm soils. Deeper excavation may also be required in areas of demolition of existing structures.

Any fill materials imported to the Project site should be non-expansive, relatively granular material having a Plasticity Index (PI) of less than 15 and a minimum Sand Equivalent (SE) of 10. Caltrans standard specifications (Sect. 19-6.02) require that using material with minimum sand equivalent value of 10 within 2.5 feet of finished grade. The maximum particle size of fill material should not be greater than 4 inches in largest dimension. It should also be non-corrosive, free of deleterious material and should be reviewed by the Geotechnical Engineer.

16.0 NOTES TO DESIGNER

The foundation recommendations presented in the report are based on the loading demands at limit states. The lateral pile capacity analysis is conducted by the structure engineer. It is recommended that the structure engineer verify the pile tip elevations when finalizing the pile data table. Final specified pile tip elevations should be the lower of the design tip elevations resulting from the axial and lateral pile capacity analysis.

17.0 PLAN REVIEW

This report is prepared for the proposed Mid-Coast Multi-Modal Trail Project. It is recommended that the final foundation plans for the subject Project be reviewed by this office prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

18.0 CONSTRUCTION CONSIDERATIONS

18.1 General

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, observation of pile construction and grading operations should be carried out by the geotechnical engineer. If the encountered subsurface conditions differ from those forming the basis of our recommendations, this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

18.2 Construction of CISS Piles

Section 49-2 “Driven Piling” of the Caltrans Standard Specifications (2015) should be followed for construction of steel piles. Section 49-3.03 provides guidelines for driven steel shells filled with concrete and reinforcement. All piles installation should be observed by a geotechnical engineer or regulatory agency. The contractor should furnish specific data of pile driving equipment, operating hammer and energy information. The contractor should carefully examine the subsurface soil conditions and make their own interpretation and perform independent study on the constructability of the piles. If unanticipated pile driving conditions are encountered during production driving, further consultation may be required.

Nominal pile driving resistance can be estimated using the formula presented in the Caltrans Standard Specifications, Section 49-2.01A(4), for driving and capacity verification. Moderate to hard driving conditions should be anticipated below 20 feet deep of driving. Central relief drilling may be required during pile installation to attain specified tip elevation. Pile capacity is expected to develop after driving as a result of soil “freeze” and dissipation of excess pore water pressure. The gain of pile capacity after initial driving may be evaluated based on “re-driving” after a minimum of 48-hour set-up. In the event that unanticipated pile driving conditions are encountered, it is recommended that a Pile Driving Analyzer (PDA) be used to evaluate the pile capacity. Typical applications of the PDA include capacity evaluation during driving and re-striking. The PDA is not required but as an optional tool for the contractor to help confirm the pile capacity.

The inside of CISS piles will be drilled out and replaced with reinforcement and concrete to meet structural design requirements. The depth of interior cleaning and concrete replacement will depend on the structural demands. Per Caltrans MTD 3-1 (2014), a minimum of 2 times the pile diameter of the soil plug should be left in the bottom portion of shells for pile construction. In addition, a concrete seal course at least 4 feet thick should be placed on top of the soil plug. Thicker seal course may be required to prevent quick soil condition. Due to presence of sandy soils, it is imperative that the procedures for constructing the seal course be such that the tremied concrete does not get contaminated with native sand and fines (silt and clay) inside the steel piles. It would be necessary to take appropriate steps to allow sand particles and fines (silt and clay) to settle down in the water to reduce such contamination. In our previous experience of similar condition, the use of polymer slurry and rock/gravel bag may be considered to help construction of the seal course. During drilling and cleaning of the inside of the piles, the water head inside the piles should be maintained to counterbalance water pressure from the bottom of the shell.

18.3 Construction of Steel Soldier Piles

The Caltrans standard specifications (2015), Section 49-3 "Cast-In-Place Concrete Piling" and Section 49-4 "Steel Soldier Piling," can be referred to for excavation of concrete soldier piles. The contractor should carefully examine the subsurface conditions and make their own interpretation and perform independent study on the constructability of the piles. The testing for pile acceptance is not required.

Due to presence of granular material and groundwater, raveling or caving is expected, which may require additional drilling and cleaning effort and may increase the concrete volume for the piles. The use of temporary steel casing, tremie seals, and/or slurry displacement method should be anticipated at all times to maintain the integrity of the piles. It is prudent to make the contractor aware of these conditions so that they take appropriate steps to comply with the standards and maintain the integrity of the soldier piles. All pile excavations should be observed by a geotechnical engineer prior to the placement of reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

18.4 Waiting Period

Based on the Plan and Profile provided, the approach embankment will generally match with the existing grade. The new fill behind the abutments is less than 5 feet high. Settlement due to embankment fill is anticipated to be mostly within the over-consolidated range and should generally occur during construction. Post construction settlement less than 0.5 inches is considered tolerable for pavement. Waiting period is not required. However, it is recommended that structure construction should not start prior to completion of grading operation. The settlement estimates are presented in Appendix C.

18.5 Construction Dewatering

Groundwater may rise up to above the footing/pile cap excavation. Groundwater may cause instability of excavation walls and bottom (piping, erosion, blow-outs, etc.) and difficult working conditions. For excavation below the groundwater table, construction dewatering will be required.

The contractor should evaluate the subsurface conditions before selecting a dewatering method, which may include shoring, sumps or tremie slabs. Groundwater should be lowered to at least 2 feet below the bottom of excavation to prevent wet soil condition. Designing dewatering system should be the contractor's responsibility.

All dewatering systems should be properly designed to prevent pumping soil fines with the discharge water. The contractor should sample and test the groundwater for soil fines content from the discharge, as needed. If soil fines are pumped, the contractor should revise his dewatering operations. Otherwise, failure of shoring, partial instability of trench bottom resulting in intolerable ground settlement/ movement of existing utilities and unsafe working conditions may occur. The contractor should provide discharge sampling locations for each pump. The contractor is encouraged to perform their own investigation, test program, etc. prior to construction in order to satisfy their design requirements for an effective dewatering program. The contractor should confirm the design groundwater level (for shoring) prior to actual construction.

18.6 Working Platform

Soft and loose, saturated native soil deposits may be encountered at the bottom of excavation. In such case, working conditions at the bottom of excavation may become difficult; equipment used at the bottom of the excavation may lose mobility, etc. The contractor should take adequate measures to minimize the disturbance of the sensitive deposits at the excavation subgrade. The contractor may minimize the disturbance of sensitive deposits or mitigate existing soft ground conditions by constructing a working platform at the bottom of the excavation. The working platform may be installed by 1) over excavating about 2 feet below the planned subgrade; 2) placing a stabilizing subgrade enhancement geotextile at the bottom of the resulting excavation; 3) backfilling with 2-inch crushed rock, compacted AB, or other such approved bridging material. The contractor may use other methods of subgrade stabilization. The contractor's proposed method should be reviewed by the geotechnical engineer.

18.7 Temporary Excavation and Shoring

Excavation will be required for installation of foundations. It is possible that unknown old buried utilities are located at the site. It might require special equipment and additional efforts to remove these buried objects.

According to OSHA Safety Standards, temporary excavations with personnel working within the excavations should be sloped or shored if the excavations are deeper than 5 feet. All excavations for the project should be made and supported in accordance with OSHA standards. For excavations up to 20 feet deep in homogenous soils, OSHA guidelines state that the maximum allowable slope should be 1H:1V for clayey soils and 1.5H:1V for sandy soils. It should be noted that the slope ratio recommended by OSHA is for temporary, unsurcharged slopes and properly dewatered conditions. Construction equipment and surcharge loads should be set back at least 15 feet from the top of the excavations unless they are accounted for in the design. Flatter trench slopes may be required if seepage is encountered during construction or if exposed soils conditions differ from those encountered by test borings. The excavation should be closely monitored during construction to detect any evidence of instability, soil creep, settlement, etc. Appropriate mitigation measures should be implemented to correct such situations that may cause or lead to future damage to facilities, utilities and other improvements.

19.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. No warranty expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed Project as described earlier, to assist the engineer in the design of this Project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the Designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

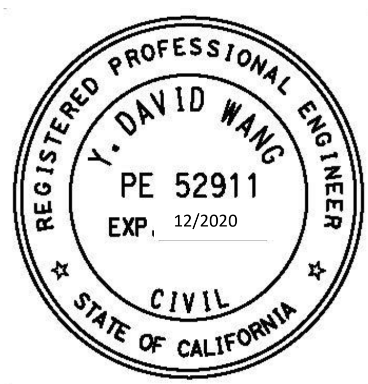
The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Y. David Wang

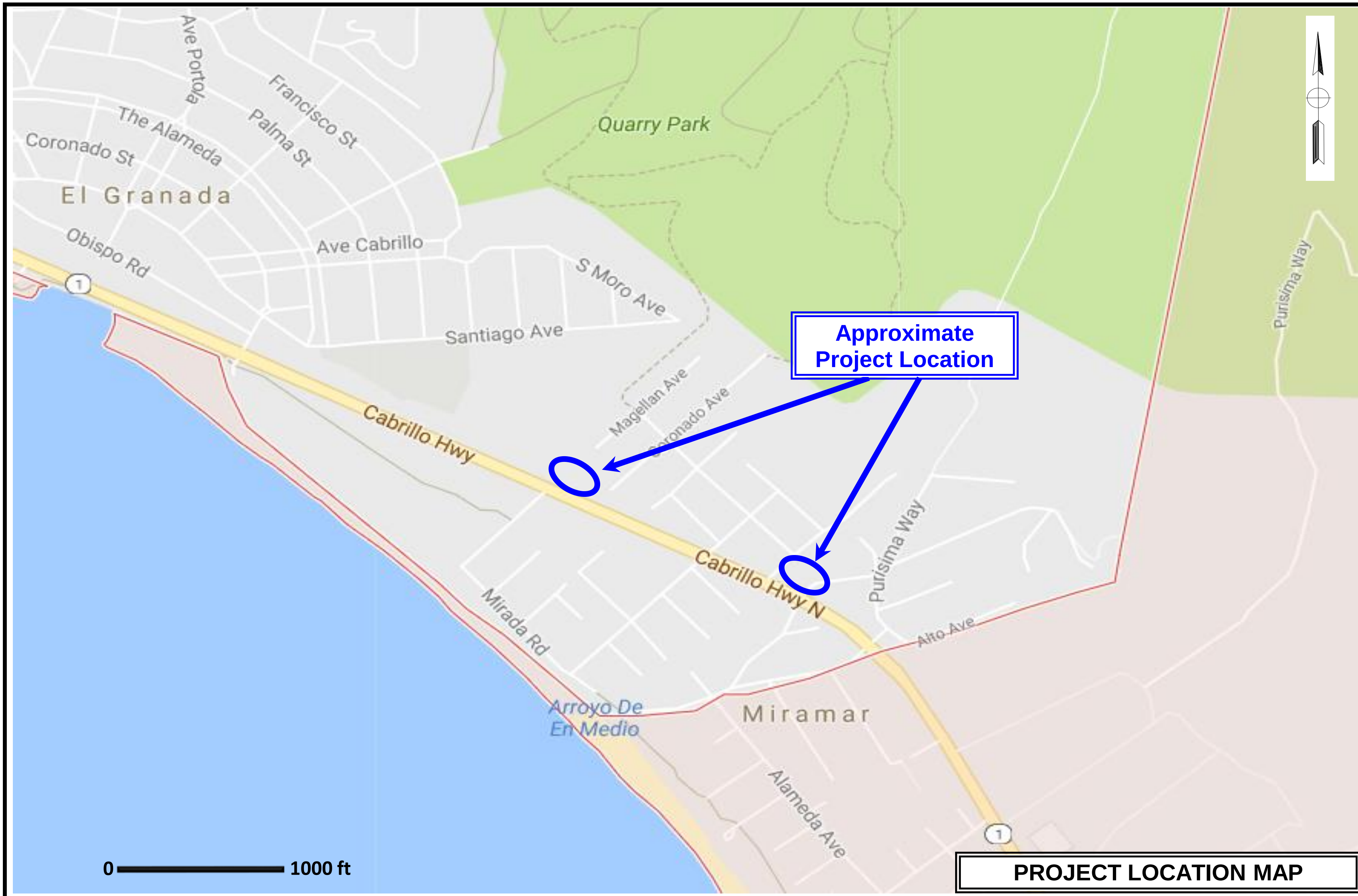
Peter Wei, PE, GE 2922
Sr. Project Engineer

Y. David Wang, PhD, PE 52911
Project Manager



REFERENCES

1. Caltrans, 2003, Bridge Design Specifications.
2. Caltrans, 2010, Soil & Rock Logging, Classification, and Presentation Manual, Office of Structural Foundations California Department of Transportation.
3. Caltrans, 2012, ARS Online V2, (http://dap3.dot.ca.gov/shake_stable/v2/index.php).
4. Caltrans, 2012, Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations.
5. Caltrans, 2013, Seismic Design Criteria, V1.7.
6. Caltrans, 2014, Memo To Designers 3-1.
7. Caltrans, 2015, Corrosion Guidelines, V2.1.
8. Caltrans, 2015, Highway Design Manual.
9. Caltrans, 2015, Standard Specifications.
10. Caltrans, 2017, Foundation Reports for Bridges.
11. California, 2014, Amendments to AASHTO LRFD Bridge Design Specifications, 6th Edition (2012).
12. California Geological Survey, 2010, Geologic Map of California, Geologic Data Map No. 2, Compilation and Interpretation by Jennings, C. W. (1977).
13. California Geological Survey, 2010, Fault Activity Map of California, Geologic Data Map No. 6, Compilation and Interpretation by Jennings, C. W. and Bryant, W. A.
14. Coduto, D.P., 1999, *Geotechnical Engineering: Principles and Practices*, Prentice-Hall, Inc., Upper Saddle River, NJ.
15. Daniel, C.R., Howie, J.A. and Sy, A., 2003, 'A Method for Correlating Large Penetration Test (LPT) to Standard Penetration Test (SPT) Blow Counts,' *Canadian Geotechnical Journal,* 40:66-77, 2003.
16. Ensoft Inc., 2014, APILE.
17. Geo-Slope International, 2014, Slope/W V8.
18. Youd, T. L. and Idriss, I. M., Co-Chairs, 2001, 'Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,' ASCE, *Journal of Geotechnical and Geoenvironmental Engineering,* V. 127, No. 4, p 297-313.



PROJECT LOCATION MAP

PARIKH
Practicing in the Geosciences

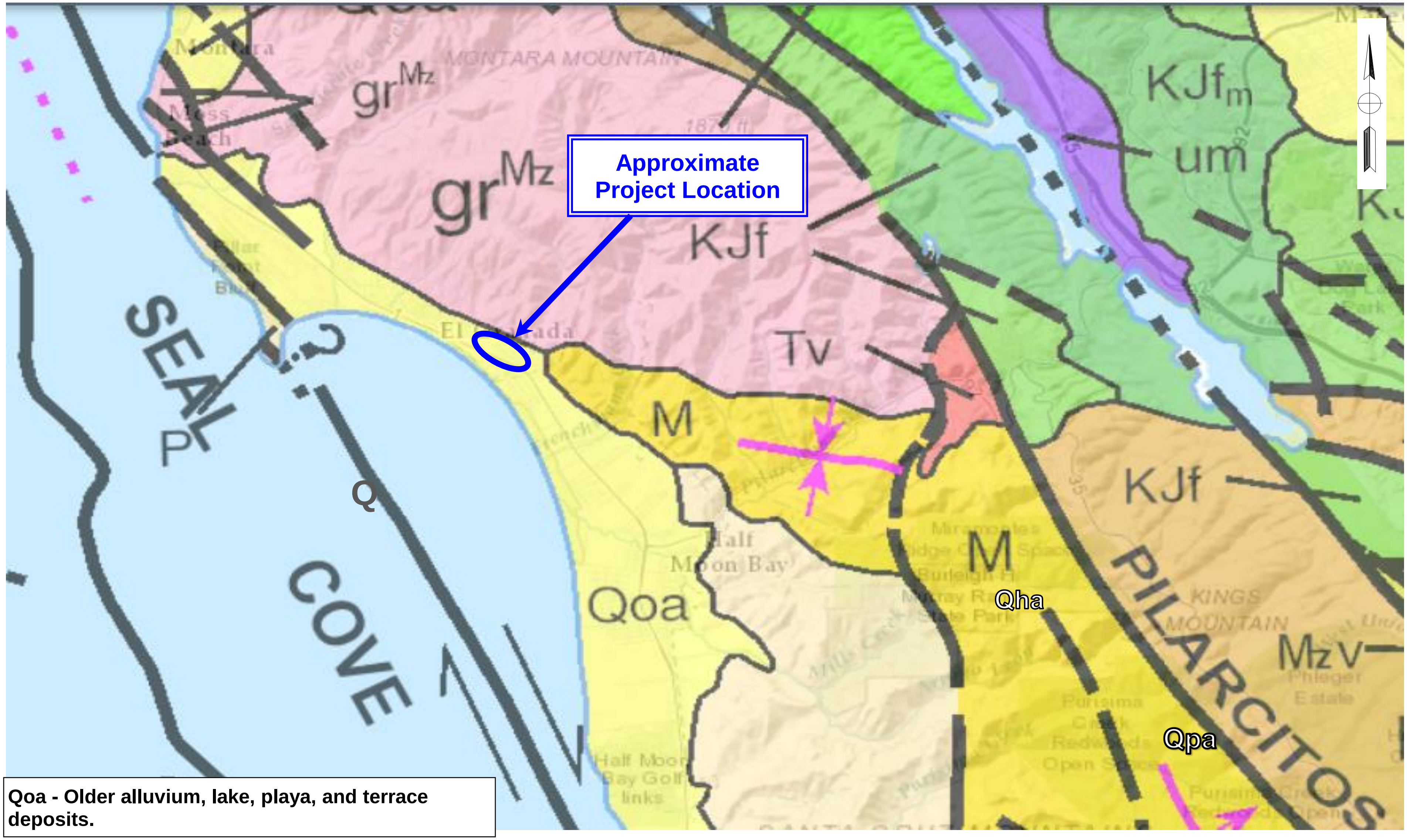
MID-COAST MULTI-MODAL TRAIL PHASE 2
SAN MATEO COUNTY, CALIFORNIA

JOB NO.: 2017-108-GEO	PLATE NO.: 1

PARIKH
Practicing in the Geosciences

MID-COAST MULTI-MODAL TRAIL PHASE 2
SAN MATEO COUNTY, CALIFORNIA

JOB NO.: 2017-108-GEO **PLATE NO.: 2**



Qoa - Older alluvium, lake, playa, and terrace deposits.

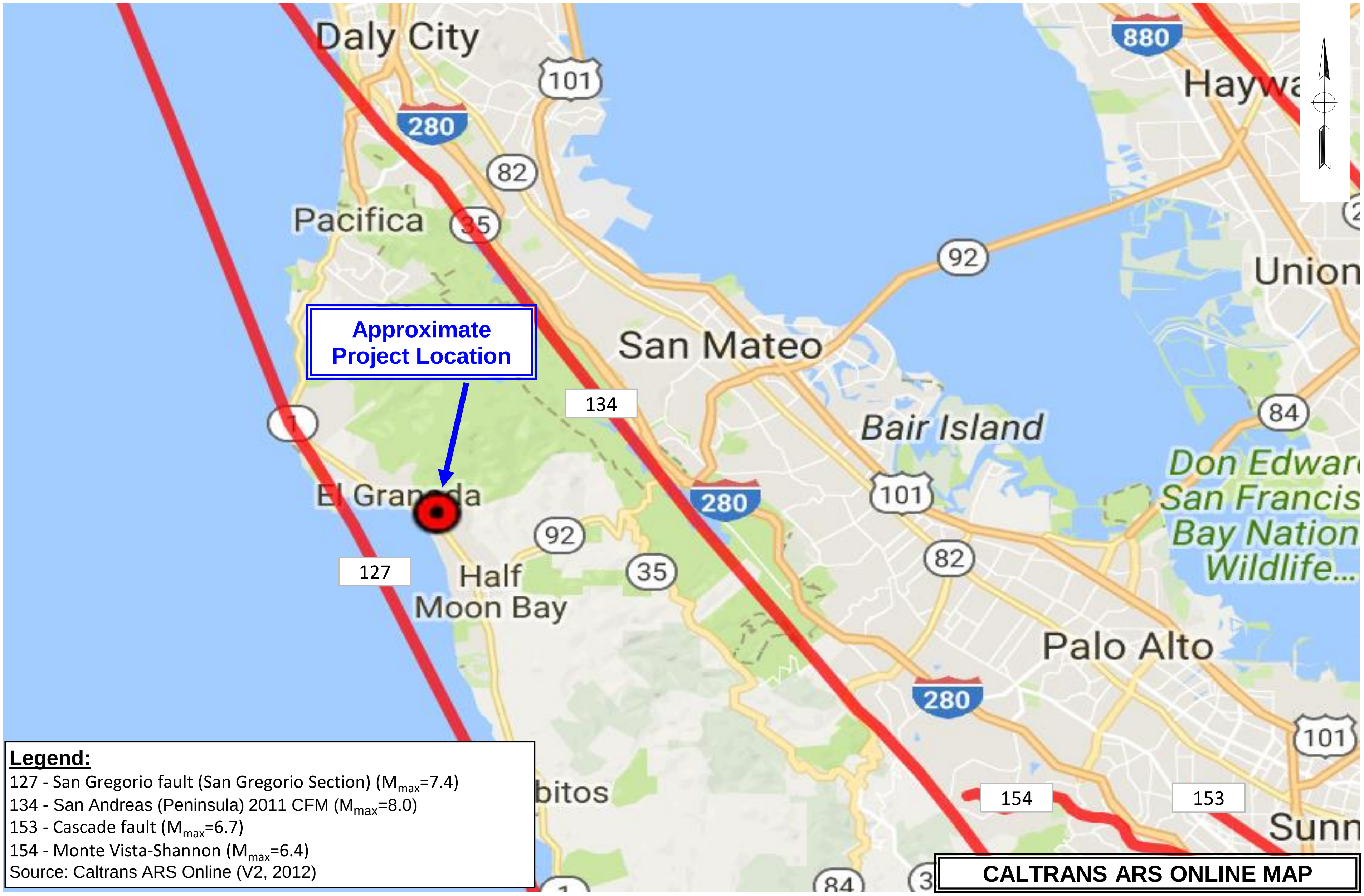
Source: California Geological Survey (2010), Geologic Data Map No. 2, by Jennings, C. W. (1977)

GEOLOGIC MAP

PARIKH
Practicing in the Geosciences

MID-COAST MULTI-MODAL TRAIL PHASE 2
SAN MATEO COUNTY, CALIFORNIA

JOB NO.: 2017-108-GEO	PLATE NO.: 3



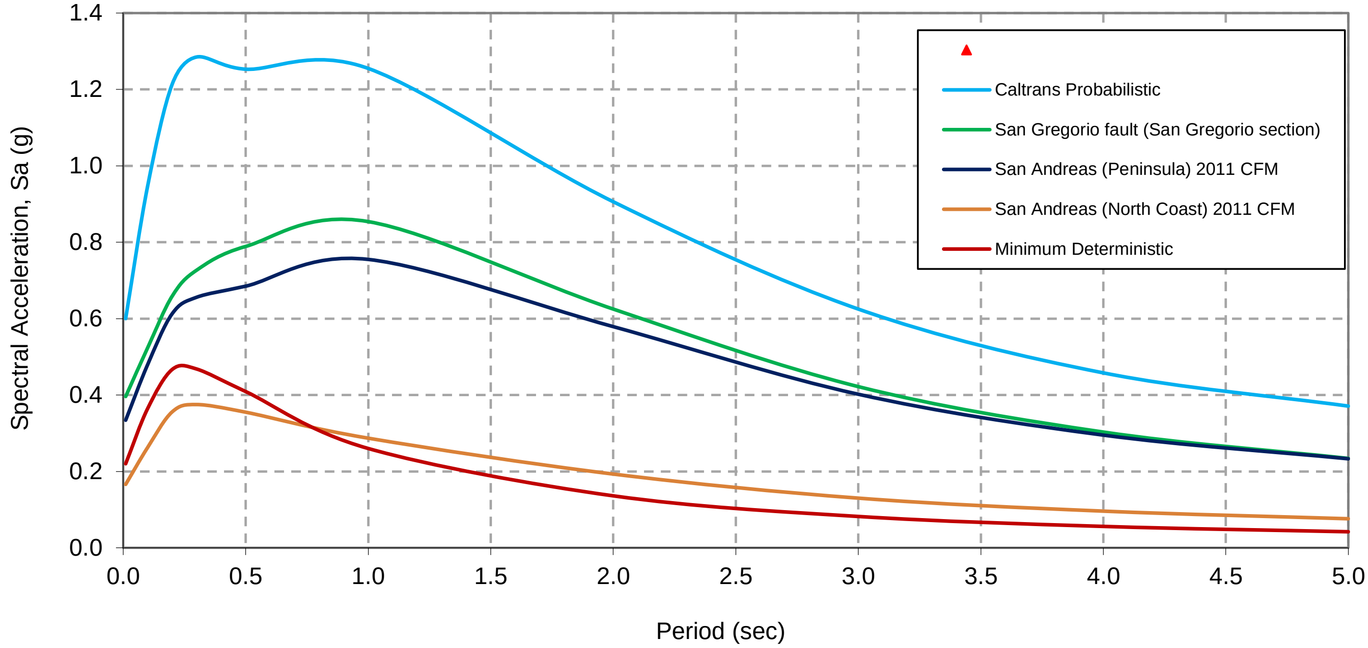
PARIKH
Practicing in the Geosciences

MID-COAST MULTI-MODAL TRAIL PHASE 2
SAN MATEO COUNTY, CALIFORNIA

JOB NO.: 2017-108-GEO	**PLATE NO.: 4**

ACCELERATION RESPONSE SPECTRUM COMPARISON

(Deterministic & Probablistic Curves)



Site Information

Latitude:	37.4955
Longitude	-122.4558
V_{S30} (m/s) =	200
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from Caltrans ARS Report Dist (km) =	2.94

Final Adjusted Spectral Accelerations (g)

Period (sec)	San Gregorio fault (San Gregorio section)	San Andreas (Peninsula) 2011 CFM	San Andreas (North Coast) 2011 CFM	Minimum Deterministic	Caltrans Probabilistic
0.0	0.396	0.334	0.166	0.220	0.600
0.1	0.524	0.480	0.263	0.365	0.948
0.2	0.659	0.613	0.356	0.467	1.214
0.3	0.727	0.656	0.375	0.468	1.285
0.5	0.789	0.685	0.355	0.409	1.253
1.0	0.854	0.755	0.287	0.260	1.255
2.0	0.625	0.579	0.193	0.136	0.906
3.0	0.422	0.402	0.130	0.082	0.625
4.0	0.303	0.295	0.096	0.056	0.458
5.0	0.234	0.233	0.076	0.042	0.371

Source:

1. Caltrans ARS Online tool (V2, http://dap3.dot.ca.gov/shake_stable/v2/index.php)
2. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

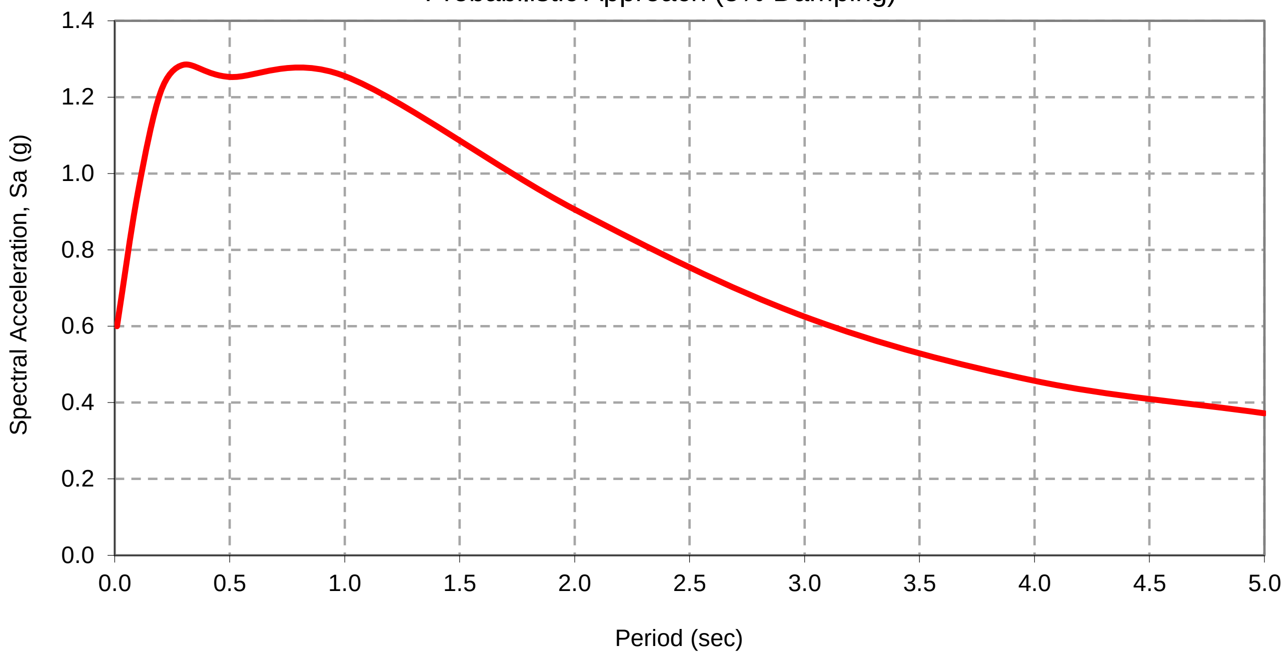
PARIKH
Practicing in the Geosciences

Mid-coast Multi-Modal Trail Phase 2
San Mateo County, California

Project No.: 2017-108-GEO | **Plate No.: 5A**

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

Probabilistic Approach (5% Damping)



Site Information

Latitude:	37.4955
Longitude	-122.4558
V_{S30} (m/s) =	200
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from Caltrans ARS Report Dist (km) =	2.94

Governing Curve:

Caltrans Online Probabilistic ARS

Recommended Response Spectrum

Period (sec)	Caltrans Online Probabilistic Spectral Acceleration (g)	Adjusted for Near Fault Effect	Adjusted For Basin Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.600	1	1	0.600
0.1	0.948	1	1	0.948
0.2	1.214	1	1	1.214
0.3	1.285	1	1	1.285
0.5	1.253	1	1	1.253
1.0	1.046	1.2	1	1.255
2.0	0.755	1.2	1	0.906
3.0	0.521	1.2	1	0.625
4.0	0.381	1.2	1	0.457
5.0	0.310	1.2	1	0.372

Note:

The curve has been modified to account for the proximity of the site to the fault. The spectral accelerations at periods of 1.0 sec. and greater have been increased by 20%. A linear interpolation is used between 0.5 and 1 sec.

Source:

1. Caltrans ARS Online tool (V2, http://dap3.dot.ca.gov/shake_stable/v2/index.php)
2. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

PARIKH
Practicing in the Geosciences

Mid-coast Multi-Modal Trail Phase 2
San Mateo County, California

Project No.: 2017-108-GEO	**Plate No.: 5B**

APPENDIX A

NOTES:

Standard Penetration Test Sampler: I.D. = 1.4"; O.D. = 2"
Modified California Sampler: I.D. = 2.5"; O.D. = 3"
Hammer Assembly: A 140 lb hammer with a 30" drop (Automatic Hammer)

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

See Caltrans 2018 Standard plans A10F, A10G, and A10H for Soil and Rock Legends.

All dimensions are in feet unless otherwise shown.

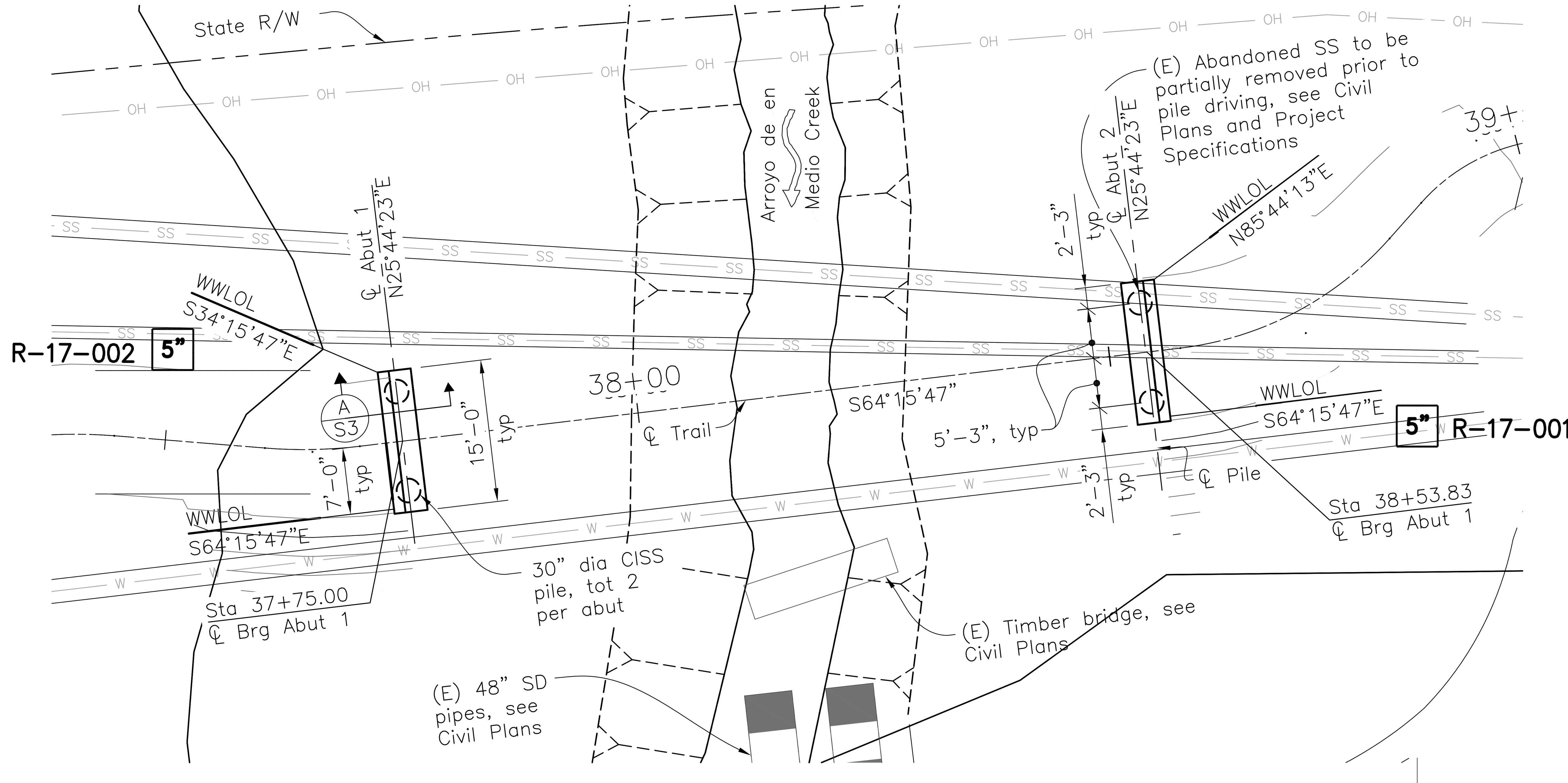
Basemap is prepared by Cornerstone 2017.

BENCHMARK STATEMENT:

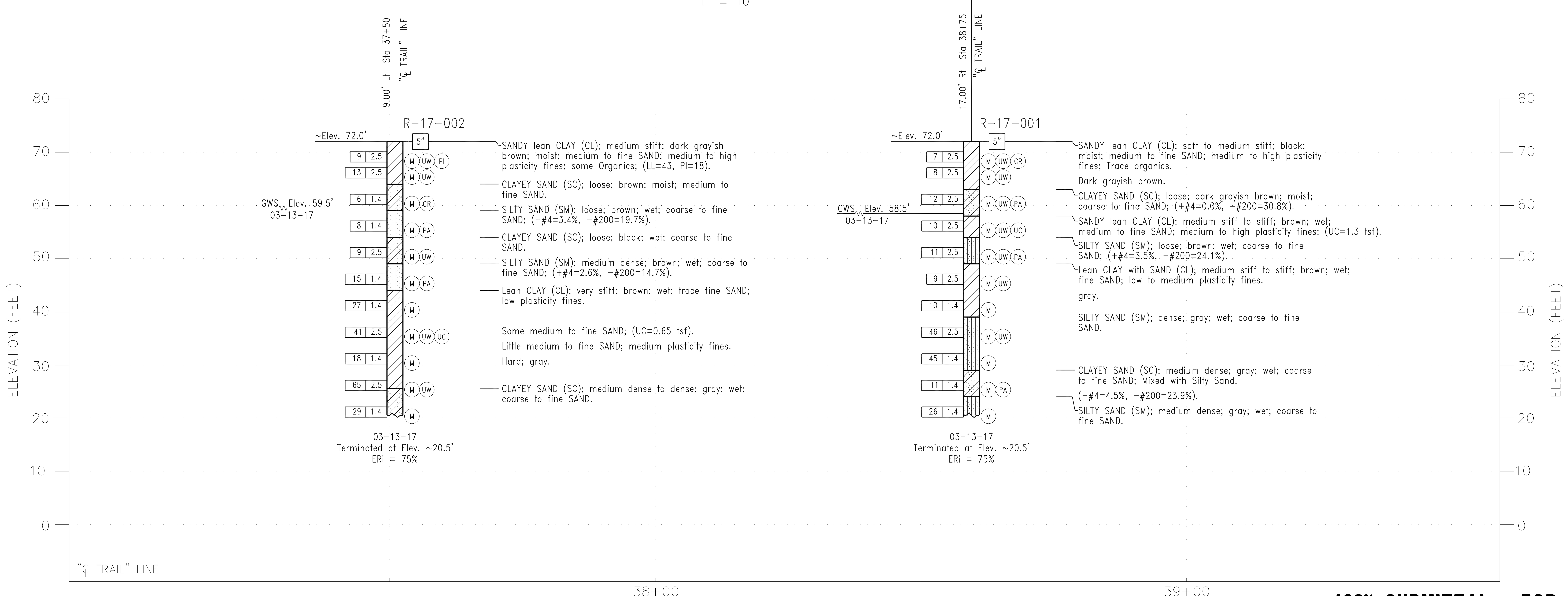
THE ELEVATIONS SHOWN ON THIS SURVEY WERE BASED ON NAVD-88.

THE SITE BENCHMARK IS MAG NAIL SET IN THE ASPHALT NEAR THE EDGE OF PAVEMENT ON THE EA NORTH OF MIRADA ROAD.

ELEVATION = 69.42.



PLAN
1" = 10'



PROFILE
Vert. : 1" = 10'
Hor. : 1" = 10'

100% SUBMITTAL - FOR REVIEW ONLY

APPROVED:

DATE:

JAMES C. PORTER, DIRECTOR OF PUBLIC WORKS
R. C. E. # 48056 / EXPIRES 12-31-2019

REGISTERED PROFESSIONAL ENGINEER · JAMES C. PORTER · C 48056 · EXPIRES 12/31/19 · CIVIL · STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER · GARY PARIKH · No. G.E. 666 · Exp. 12/31/21 · GEOTECHNICAL · STATE OF CALIFORNIA

PARIKH
Practicing in the Geosciences

BKF
ENGINEERS/SURVEYORS/PLANNERS
255 SHORELINE DRIVE, SUITE 200
REDWOOD CITY, CA 94065
(650) 482-6300

COUNTY OF SAN MATEO PUBLIC WORKS

REVISION	DATE	DESIGNED BY: DW	MIDCOAST MULTI-MODAL TRAIL	SCALE: AS NOTED
		CHECKED BY: DW	**BRIDGE**	DATE: 12/2/19
		DRAWN BY: KO	**LOG OF TEST BORINGS 1 OF 2**	FILE NO.:

JAMES C. PORTER, DIRECTOR OF PUBLIC WORKS
SAN MATEO COUNTY

555 COUNTY CENTER, 5th FLOOR
REDWOOD CITY, CALIFORNIA 94063

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3 4

SHEET OF

DRAWING NAME: S:\ACAD\2017-108-GEO\LOTB\1 OF 2.dwg
PLOT TIME: 12-02-19

NOTES:

Standard Penetration Test Sampler: I.D. = 1.4"; O.D. = 2"
Modified California Sampler: I.D. = 2.5"; O.D. = 3"
Hammer Assembly: A 140 lb hammer with a 30" drop (Automatic Hammer)

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

See Caltrans 2018 Standard plans A10F, A10G, and A10H for Soil and Rock Legends.

All dimensions are in feet unless otherwise shown.

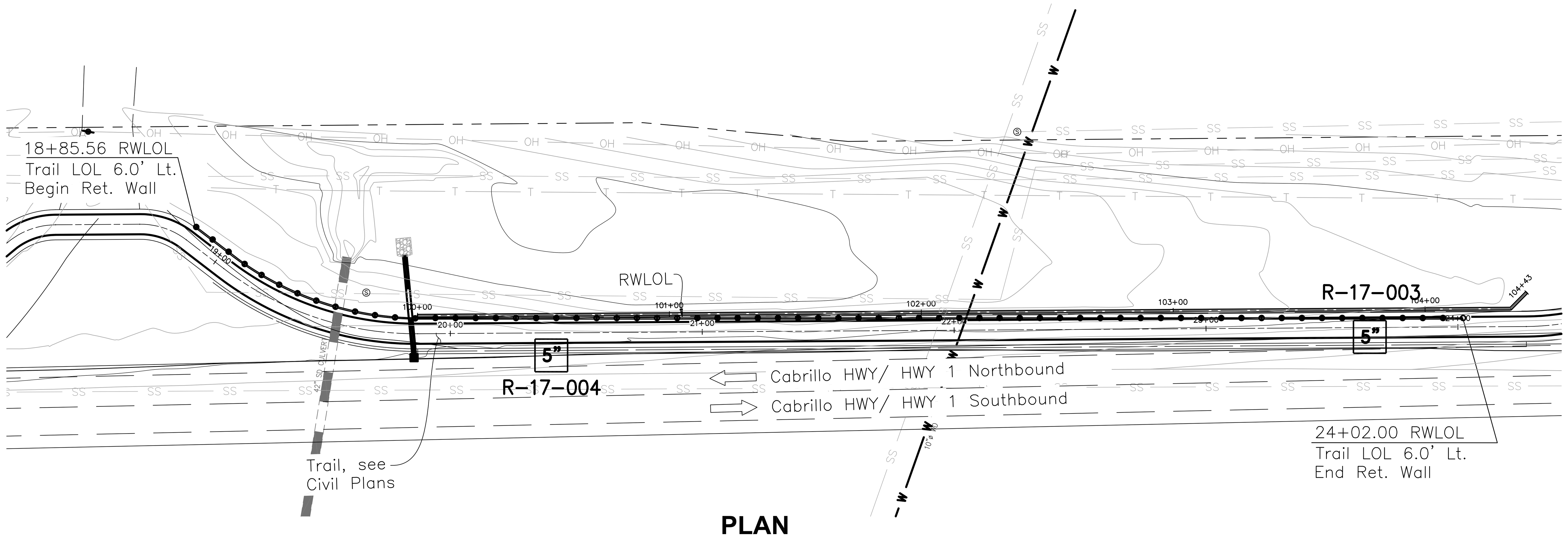
Basemap is prepared by Cornerstone 2017.

BENCHMARK STATEMENT:

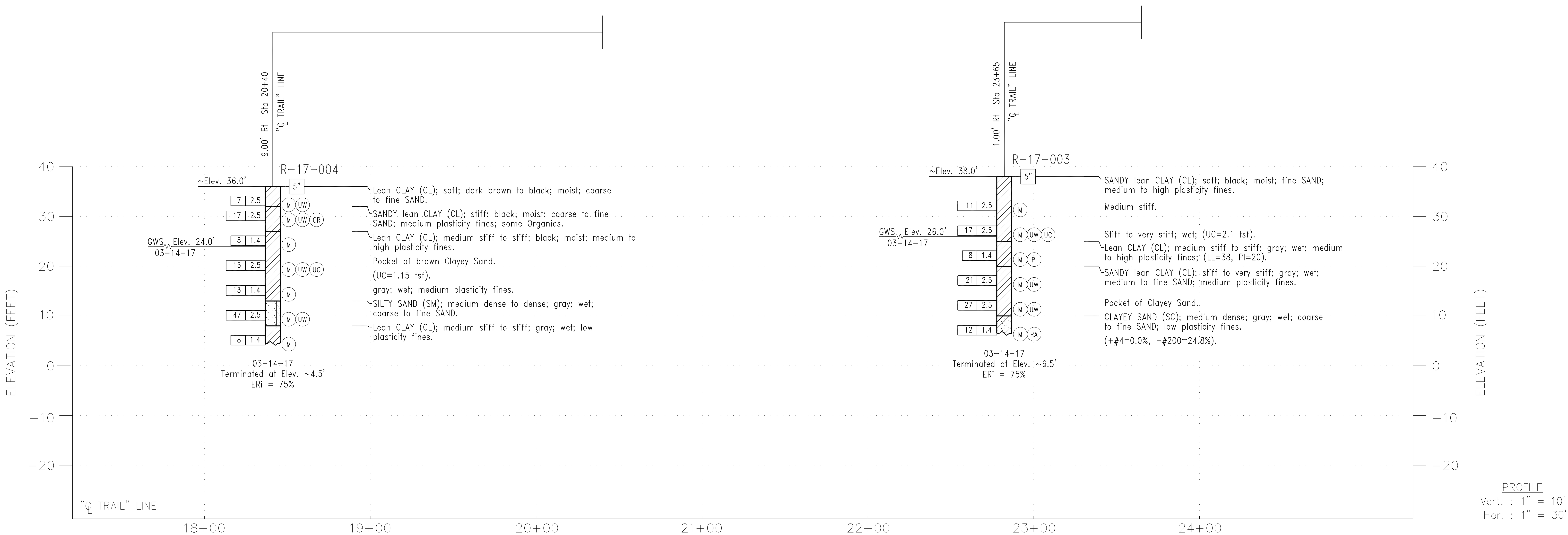
THE ELEVATIONS SHOWN ON THIS SURVEY WERE BASED ON NAVD-88.

THE SITE BENCHMARK IS MAG NAIL SET IN THE ASPHALT NEAR THE EDGE OF PAVEMENT ON THE EA NORTH OF MIRADA ROAD.

ELEVATION = 69.42.



PLAN
1" = 30'



PROFILE
Vert. : 1" = 10'
Hor. : 1" = 30'

APPROVED:

DATE:

JAMES C. PORTER, DIRECTOR OF PUBLIC WORKS
R. C. E. # 48056 / EXPIRES 12-31-2019

REGISTERED PROFESSIONAL ENGINEER JAMES C. PORTER C 48056 EXPIRES 12/31/19 CIVIL STATE OF CALIFORNIA

100% SUBMITTAL - FOR REVIEW ONLY

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/21 GEOTECHNICAL STATE OF CALIFORNIA

PARIKH
Practicing in the Geosciences

BKF
ENGINEERS/SURVEYORS/PLANNERS
255 SHORELINE DRIVE, SUITE 200
REDWOOD CITY, CA 94065
(650) 482-6300

COUNTY OF SAN MATEO PUBLIC WORKS

REVISION	DATE

DESIGNED BY: DW
CHECKED BY: DW
DRAWN BY: KO

MIDCOAST MULTI-MODAL TRAIL
RETAINING WALL
LOG OF TEST BORINGS 2 OF 2

SCALE: AS NOTED
DATE: 12/2/19
FILE NO.:

JAMES C. PORTER, DIRECTOR OF PUBLIC WORKS
SAN MATEO COUNTY

555 COUNTY CENTER, 5th FLOOR
REDWOOD CITY, CALIFORNIA 94063

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3 4

SHEET OF

DRAWING NAME: S:\ACAD\2017-108-GEO\LOTB\2 OF 2.dwg
PLOT TIME: 12-02-19

APPENDIX B

LABORATORY TESTS

Classification Tests
The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System. The results are presented in "Log of Test Borings", Appendix A.

Moisture-Density
The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM D 2216. This information was used to classify and correlate the soils. The results are presented in the summary table on Plate B-2.

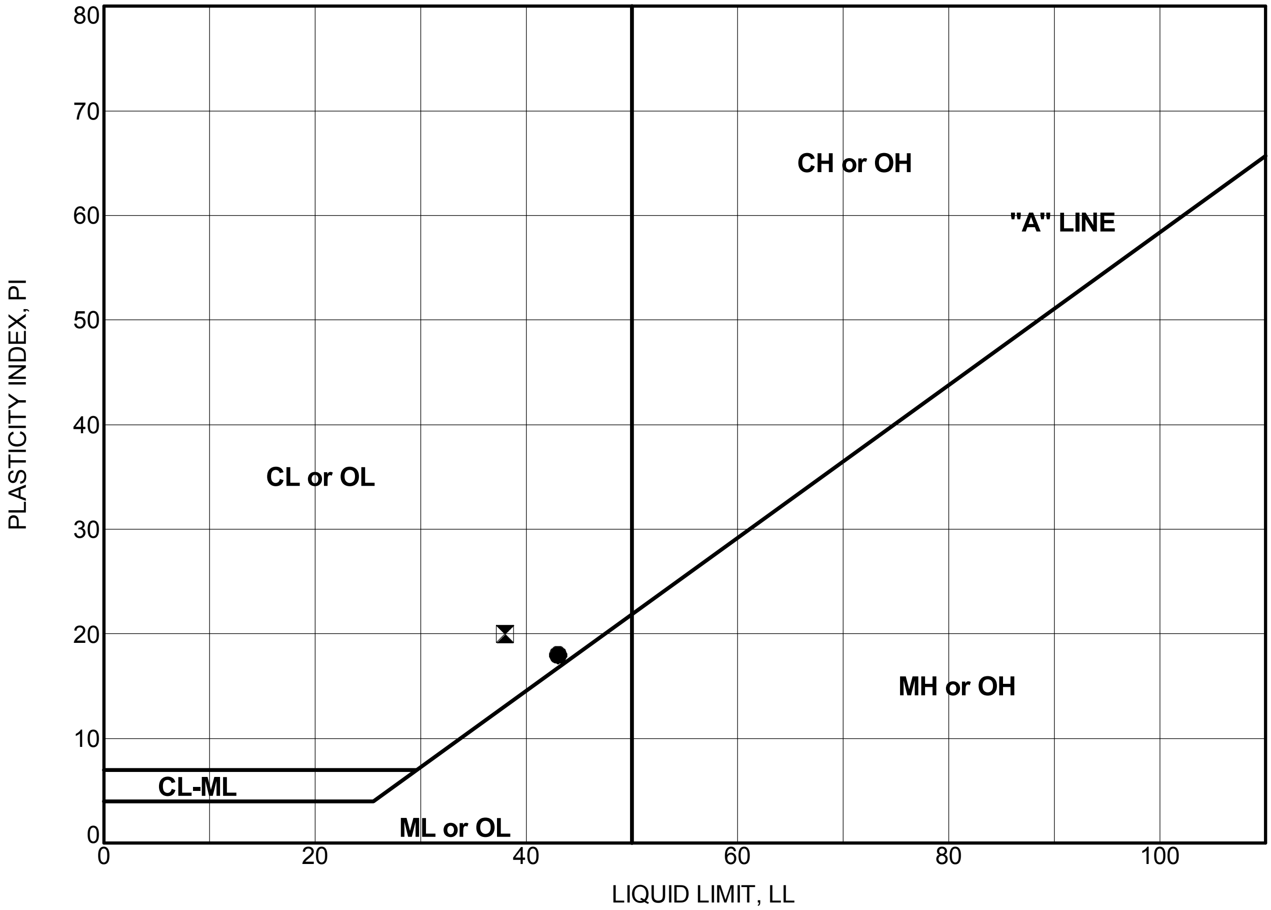
Atterberg Limits
The Atterberg Limits (ASTM D 4318) were determined on selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the effective strength characteristics and expansion potential. The tests results are presented on Plate B-3, Plasticity Chart.

Grain Size Classification
Grain size classification tests (ASTM D 422) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plates B-4A and B-4B, Grain Size Distribution Curves.

Unconfined Compression Tests
Strength tests were performed on selected samples. Unconfined compression tests were performed in general accordance with ASTM D 2166. The results are presented on Plates B-5A through B-5D.

Corrosion Tests
Corrosion tests were performed on selected samples to determine the corrosion potential of the soils according to California Test Methods 643, 417 and 422. The tests were performed by Sunland Analytical. The test results are presented on Plates B-6A, B-6B, and B-6C.

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Shear Strength (tsf)
R-17-001	1	3.0	CL	18.4	107.7						
R-17-001	2	6.0	CL	15.9	116.9						
R-17-001	3	11.0	SC	13.2	120.7				0.0	30.8	
R-17-001	4	16.0	CL	20.7	106.6						UC = 0.65
R-17-001	5	21.0	SM	16.3	118.7				3.5	24.1	
R-17-001	6	26.0	CL	36.4	87.6						
R-17-001	7	31.0	CL	16.8	-						
R-17-001	8	36.0	SM	12.9	121.0						
R-17-001	9	41.0	SM	13.9	-						
R-17-001	10	46.0	SC	16.6	-				4.5	23.9	
R-17-001	11	51.0	SM	16.3	-						
R-17-002	1	3.0	CL	12.6	106.1	43	25	18			
R-17-002	2	6.0	CL	13.8	123.3						
R-17-002	3	11.0	SC	16.2	-						
R-17-002	4	16.0	SM	17.4	-				3.4	19.7	
R-17-002	5	21.0	SC	20.8	103.5						
R-17-002	6	26.0	SM	19.3	-				2.6	14.7	
R-17-002	7	31.0	CL	17.2	-						
R-17-002	8	36.0	CL	15.1	115.2						UC = 0.3
R-17-002	9	41.0	CL	20.5	-						
R-17-002	10	46.0	CL	15.8	113.8						
R-17-002	11	51.0	SC	18.4	-						
R-17-003	1	6.0	CL	20.4	-						
R-17-003	2	11.0	CL	20.4	103.2						UC = 1.05
R-17-003	3	16.0	CL	21.2	-	38	18	20			
R-17-003	4	21.0	CL	17.8	112.6						
R-17-003	5	26.0	CL	15.6	112.2						
R-17-003	6	31.0	SC	19.3	-				0.0	24.8	
R-17-004	1	3.0	CL	16.0	105.2						
R-17-004	2	6.0	CL	19.5	103.4						
R-17-004	3	11.0	CL	29.0	-						
R-17-004	4	16.0	CL	26.5	96.8						UC = 0.60
R-17-004	5	21.0	CL	19.3	-						
R-17-004	6	26.0	SM	18.4	108.1						
R-17-004	7	31.0	CL	28.6	-						



PLASTICITY CHART

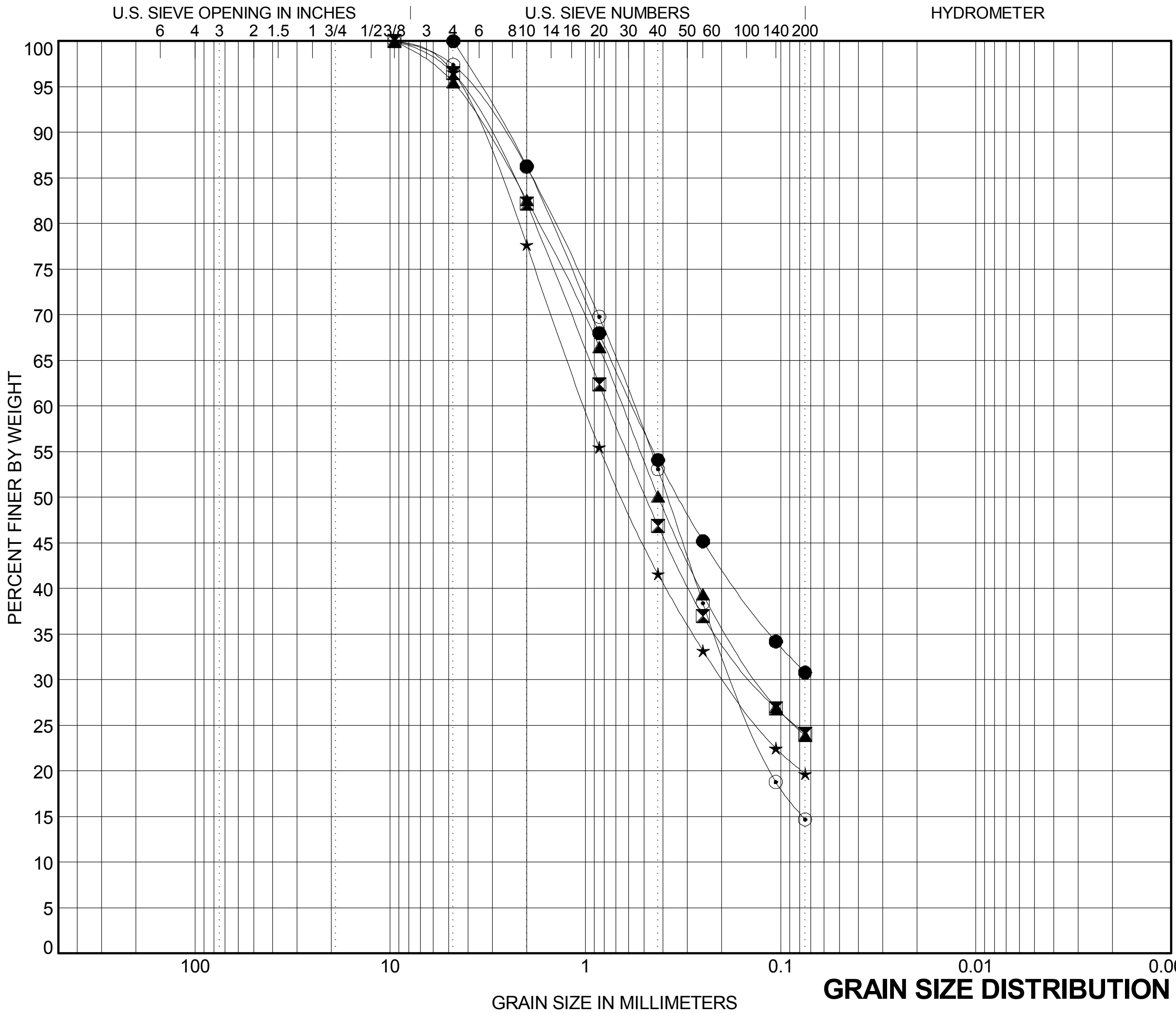
Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
R-17-002		3.0	●		43	25	18	SANDY lean CLAY
R-17-003		16.0	⧗		38	18	20	Lean CLAY

PARIKH
Practicing in the Geosciences

MID-COAST MULTI-MODAL TRAIL PHASE 2
SAN MATEO COUNTY, CALIFORNIA

JOB NO: 2017-108-GEO

PLATE NO: B-3



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

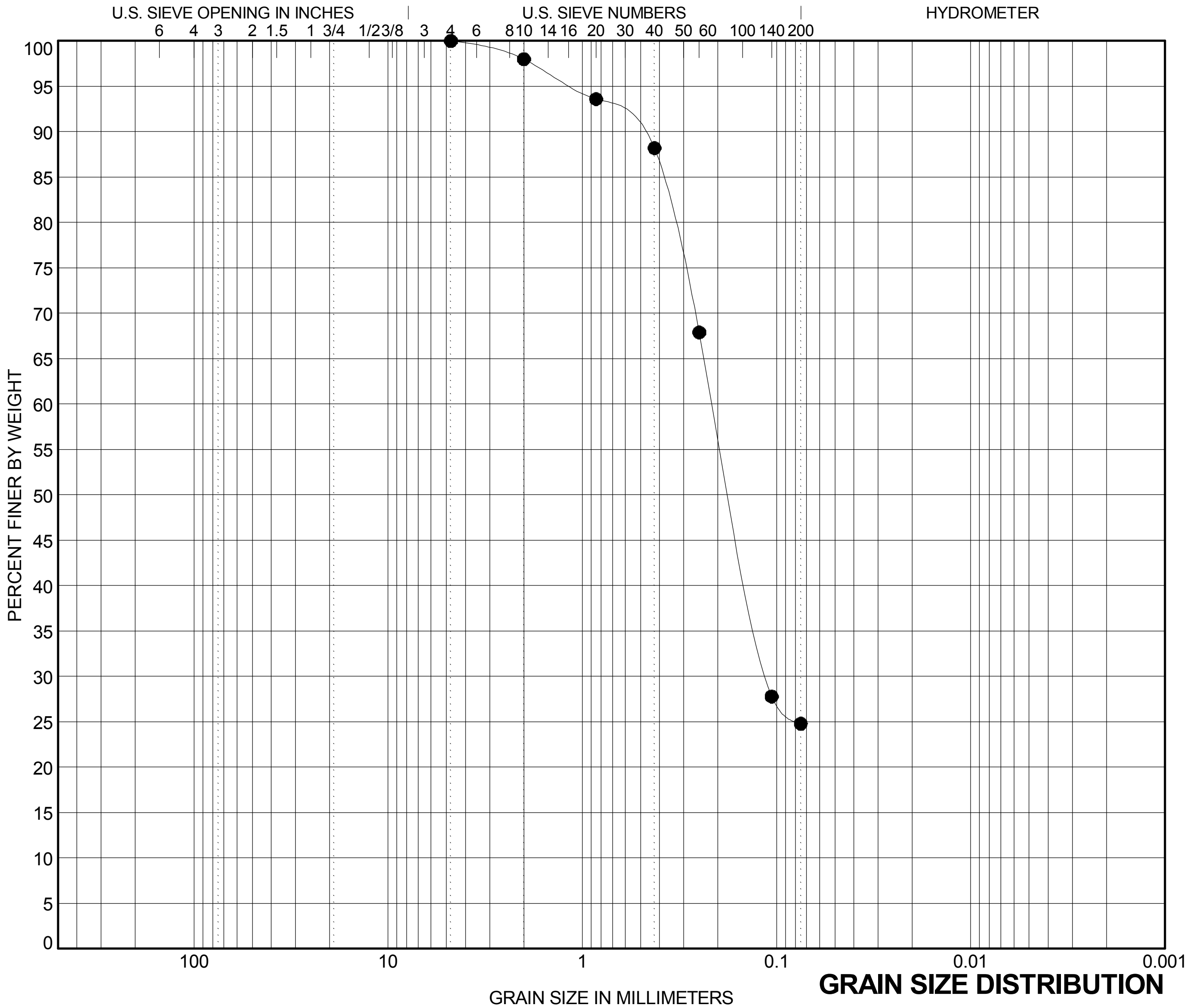
	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	R-17-001	3	11.0	CLAYEY SAND					
⧗	R-17-001	5	21.0	SILTY SAND					
▲	R-17-001	10	46.0	CLAYEY SAND					
★	R-17-002	4	16.0	SILTY SAND					
⊙	R-17-002	6	26.0	SILTY SAND					

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	R-17-001	3	11.0	4.75	0.57			0.0	69.2	30.8	
⧗	R-17-001	5	21.0	9.5	0.763	0.138		3.5	72.4	24.1	
▲	R-17-001	10	46.0	9.5	0.646	0.13		4.5	71.6	23.9	
★	R-17-002	4	16.0	9.5	1.011	0.193		3.4	76.9	19.7	
⊙	R-17-002	6	26.0	9.5	0.566	0.173		2.6	82.7	14.7	

PARIKH
Practicing in the Geosciences

MID-COAST MULTI-MODAL TRAIL PHASE 2
SAN MATEO COUNTY, CALIFORNIA

JOB NO: 2017-108-GEO | **PLATE NO: B-4A**



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**R-17-003**	**6**	**31.0**	**CLAYEY SAND**					

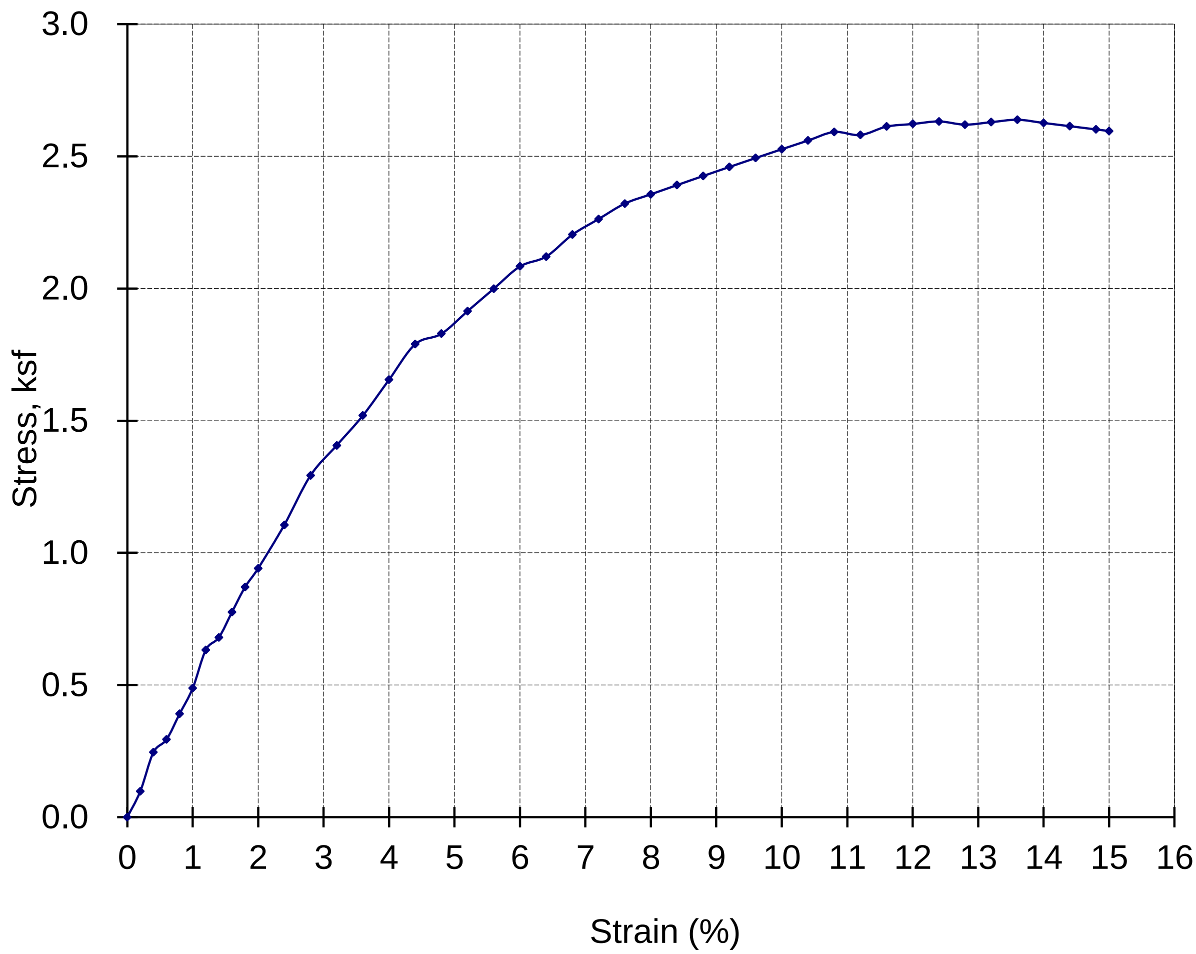
	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**R-17-003**	**6**	**31.0**	**4.75**	**0.211**	**0.111**		**0.0**	**75.2**	**24.8**	

PARIKH
Practicing in the Geosciences

MID-COAST MULTI-MODAL TRAIL PHASE 2
SAN MATEO COUNTY, CALIFORNIA

JOB NO: 2017-108-GEO	PLATE NO: B-4B

UNCONFINED COMPRESSION TEST



Boring No.: R-17-001

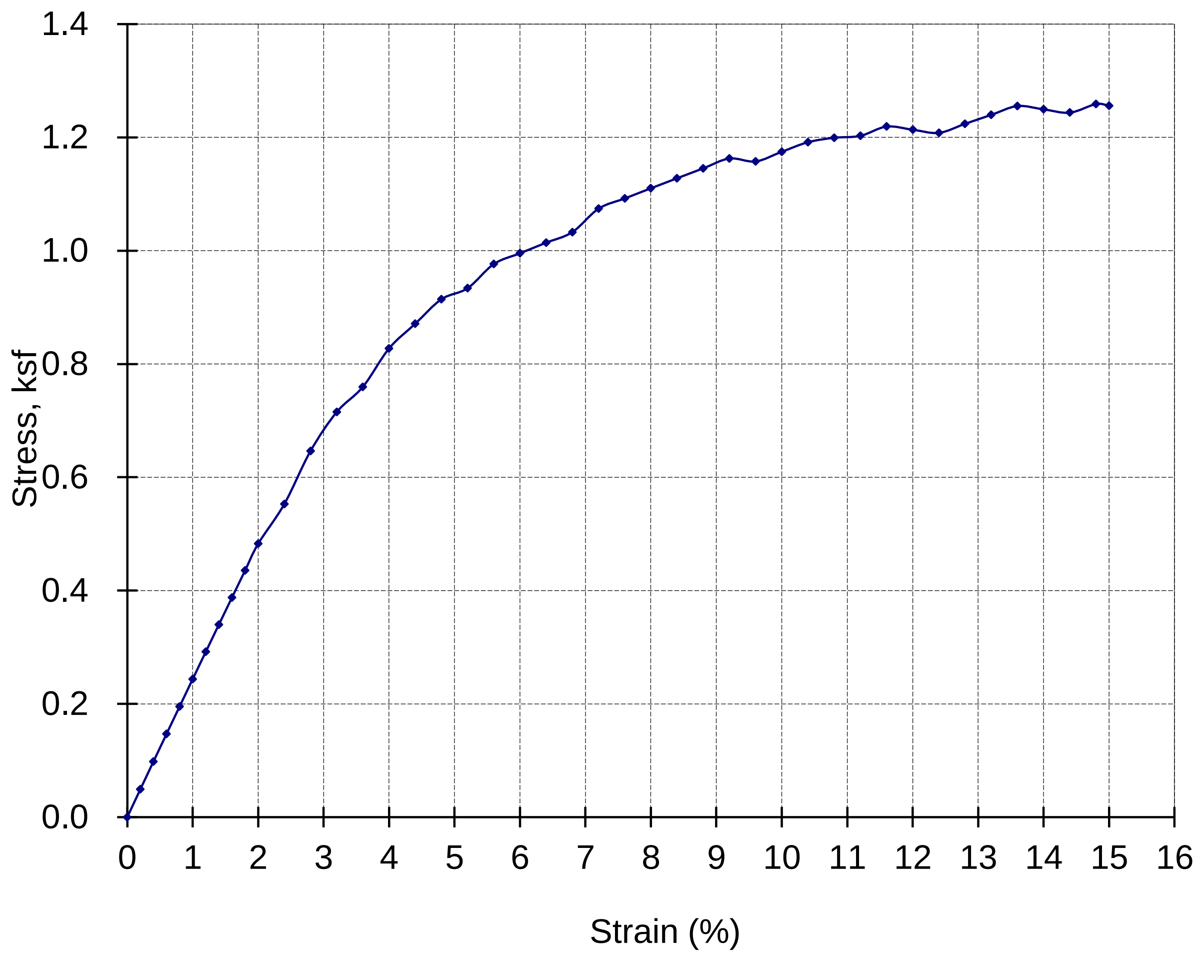
Sample No. : 4 **Maximum Strength (ksf):** 2.60

Depth (feet): 16 **Strain @ Failure (%):** 15.00

Material Description:
Lean Clay, stiff

PARIKH CONSULTANTS, INC. **GEOTECHNICAL CONSULTANTS** **MATERIALS TESTING**	**MID-COAST MULTI-MODAL TRAIL PHASE 2** **SAN MATEO COUNTY, CALIFORNIA**
	JOB NO.: 2017-108-GEO **PLATE NO.:** B-5A

UNCONFINED COMPRESSION TEST



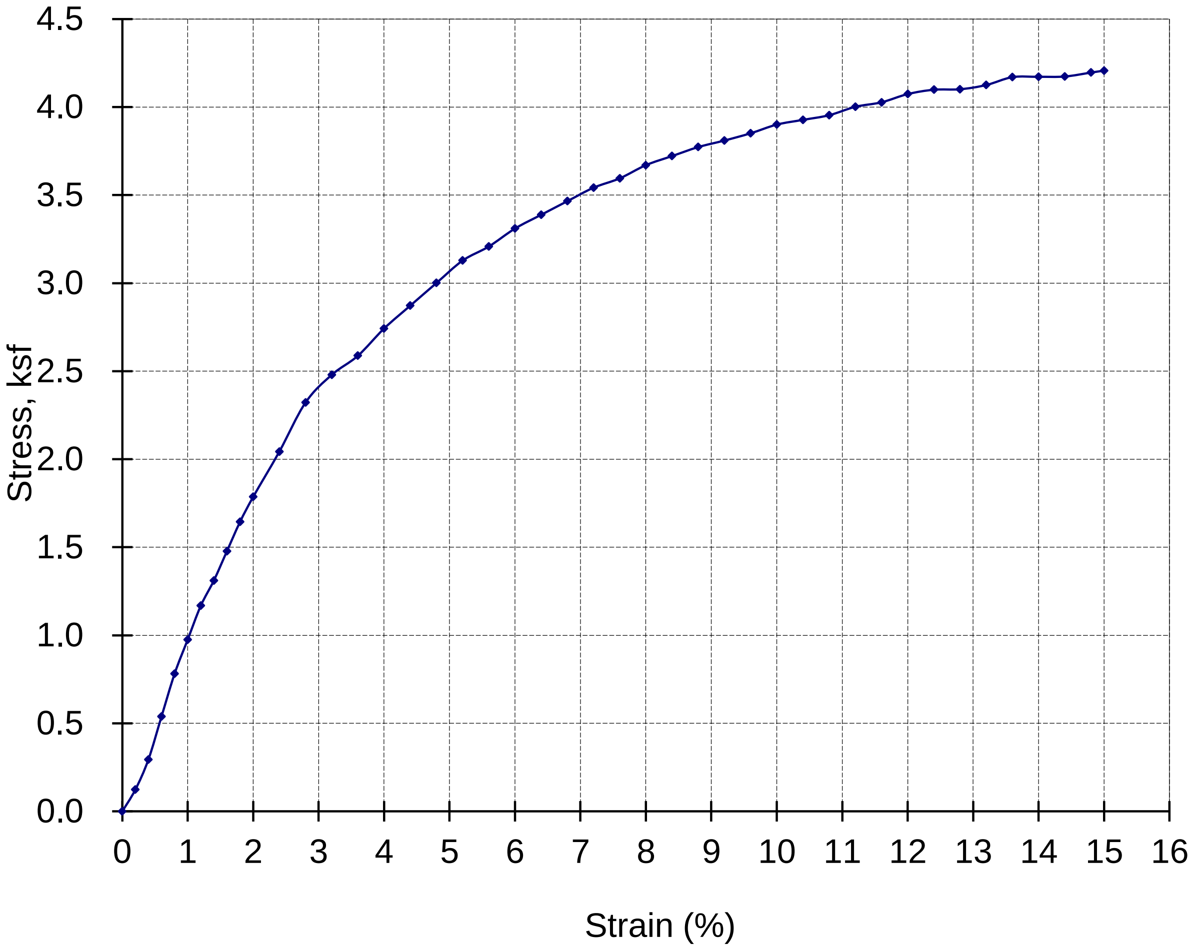
Boring No.: R-17-002

Sample No. : 8 **Maximum Strength (ksf):** 1.30

Depth (feet): 36 **Strain @ Failure (%):** 15.00

Material Description:
Lean Clay, medium stiff

UNCONFINED COMPRESSION TEST



Boring No.: R-17-003

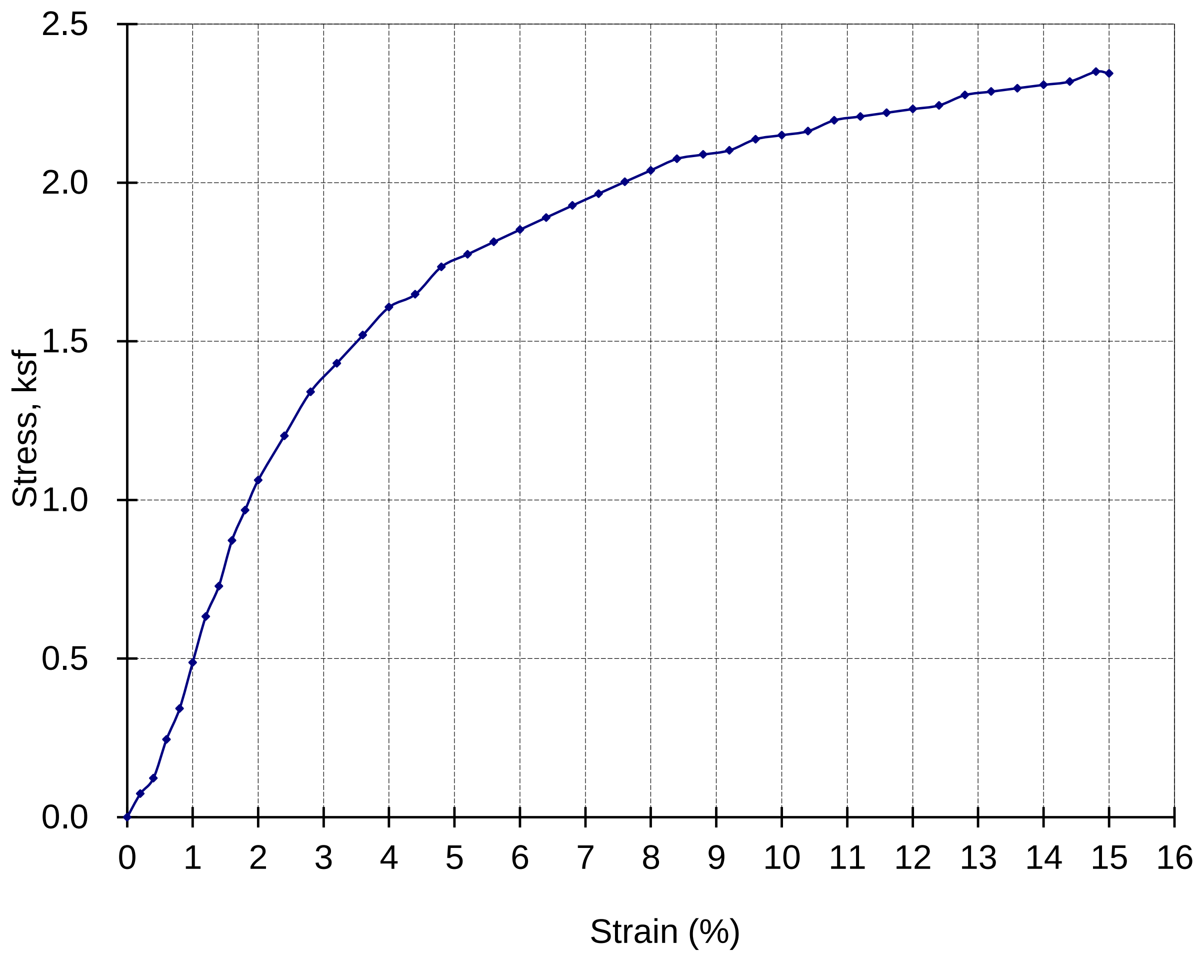
Sample No. : 2 **Maximum Strength (ksf):** 4.20

Depth (feet): 11 **Strain @ Failure (%):** 15.00

Material Description:
Lean Clay, stiff

PARIKH CONSULTANTS, INC. **GEOTECHNICAL CONSULTANTS** **MATERIALS TESTING**	**MID-COAST MULTI-MODAL TRAIL PHASE 2** **SAN MATEO COUNTY, CALIFORNIA**	
	JOB NO.: 2017-108-GEO	**PLATE NO.:** B-5C

UNCONFINED COMPRESSION TEST



Boring No.: R-17-004

Sample No. : 4 **Maximum Strength (ksf):** 2.30

Depth (feet): 16 **Strain @ Failure (%):** 15.00

Material Description:
Lean Clay, stiff

PARIKH CONSULTANTS, INC. **GEOTECHNICAL CONSULTANTS** **MATERIALS TESTING**	**MID-COAST MULTI-MODAL TRAIL PHASE 2** **SAN MATEO COUNTY, CALIFORNIA**	
	JOB NO.: 2017-108-GEO	**PLATE NO.:** B-5D

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 04/05/2017
Date Submitted 03/29/2017

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 2017-108-GEO Site ID : R-17-001 @ 3 FT
Thank you for your business.

* For future reference to this analysis please use SUN # 73890-154099.

EVALUATION FOR SOIL CORROSION

Soil pH	6.27	
Minimum Resistivity	3.75 ohm-cm (x1000)	
Chloride	9.5 ppm	00.00095 %
Sulfate	6.0 ppm	00.00060 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

PLATE NO. B-6A

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 04/05/2017
Date Submitted 03/29/2017

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 2017-108-GEO Site ID : R-17-002 @ 11 FT
Thank you for your business.

* For future reference to this analysis please use SUN # 73890-154100.

EVALUATION FOR SOIL CORROSION

Soil pH	6.45	
Minimum Resistivity	3.22 ohm-cm (x1000)	
Chloride	30.0 ppm	00.00300 %
Sulfate	14.3 ppm	00.00143 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 04/05/2017
Date Submitted 03/29/2017

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 2017-108-GEO Site ID : R-17-004 @ 6 FT
Thank you for your business.

* For future reference to this analysis please use SUN # 73890-154101.

EVALUATION FOR SOIL CORROSION

Soil pH	6.86	
Minimum Resistivity	2.20 ohm-cm (x1000)	
Chloride	31.8 ppm	00.00318 %
Sulfate	44.4 ppm	00.00444 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

APPENDIX C

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME **Mid-coast Multi-Modal Trail Phase 2**
PROJECT NO. **2017-108-GEO**
BORING NO. **R-17-001**

BOREHOLE DIA (in) 5
GW DEPTH (ft)= 13.5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

HAMMER ENERGY = 75%

FAULT INFO

a_{max} (g)= 0.6
FAULT M_w = 8

MSF = 0.94

					CYCLIC STRESS RATIO (CSR				*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*											*S.=($CRR_{7.5}$/CSR)*MSF*Kσ*K*			
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	Kσ	Kα	F.S.	Volumetric Strain (%)
1	2	2	7	MC	250	250	1.00		4.6	1.3	0.75	1.0	1.0	4.3	1.66	7.1				1.00	1		
2	5	2	8	MC	625	625	0.99		5.2	1.3	0.75	1.0	1.0	4.9	1.45	7.1				1.00	1		
3	10	1	12	MC	1250	1250	0.98	0.38	7.8	1.3	0.80	1.0	1.0	7.8	1.21	9.4	31%	15.7	0.17	1.00	1		1.9
4	15	2	10	MC	1875	1781	0.97		6.5	1.3	0.85	1.0	1.0	6.9	1.05	7.3				1.00	1		
5	20	1	11	MC	2500	2094	0.96	0.45	7.2	1.3	0.95	1.0	1.0	8.5	0.98	8.3	24%	13.4	0.14	0.99	1	0.30	2
6	25	2	9	MC	3125	2406	0.94		5.9	1.3	0.95	1.0	1.0	6.9	0.92	6.4				0.96	1		
7	30	2	10	SPT	3750	2719	0.92		10.0	1.3	1.00	1.2	1.0	15.0	0.86	12.9				0.92	1		
8	35	1	46	MC	4375	3031	0.89	0.50	29.9	1.3	1.00	1.0	1.0	37.4	0.81	30.3	15%	34.2		0.85	1		
9	40	1	45	SPT	5000	3344	0.85	0.50	45.0	1.3	1.00	1.2	1.0	67.5	0.77	51.7	15%	56.7		0.81	1		
10	45	1	11	SPT	5625	3656	0.80	0.48	11.0	1.3	1.00	1.2	1.0	16.5	0.73	12.0	24%	17.4	0.19	0.84	1	0.31	1.7
11	50	1	26	SPT	6250	3969	0.75	0.46	26.0	1.3	1.00	1.2	1.0	39.0	0.69	26.9	15%	30.7		0.77	1		

Total Liquefaction Settlement (in.)= 3.4

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = 2.2/(1.2 + \sigma_v'/P_a)$ with a maximum value of 1.7 per Kayen et al. (1992) as cited in Youd et al. (2001).
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = \alpha + \beta\ (N_1)_{60}$

where α and β = coefficients determined from the following relationships

for FC $\leq$ 5% $\alpha = 0$, $\beta = 1.0$
for 5% < FC < 35% $\alpha = \exp(1.76-(190/FC^2))$, $\beta = (0.99+(FC^{1.5}/1000))$
for FC $\geq$ 35% $\alpha = 5.0$, $\beta = 1.2$

Reference: Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME	**Mid-coast Multi-Modal Trail Phase 2**	*SOIL GROUPS*	*FAULT INFO*
PROJECT NO.	**2017-108-GEO**	1. GRAVELS, SANDS AND NONPLASTIC SILTS	
BORING NO.	**R-17-002**	2. CLAYS AND PLASTIC SILTS	a_{max} *(g)=* 0.6
			FAULT M_w *=* 8
BOREHOLE DIA (in)	5	*HAMMER ENERGY =* 75%	
GW DEPTH (ft)=	12.5		*MSF =* 0.94

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*											*S.=($CRR_{7.5}$/CSR)*MSF*Kσ*K*			
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	Kσ	Kα	F.S.	Volumetric Strain (%)
1	2	2	9	MC	250	250	1.00		5.9	1.3	0.75	1.0	1.0	5.5	1.66	9.1				1.00	1		
2	5	2	13	MC	625	625	0.99		8.5	1.3	0.75	1.0	1.0	7.9	1.45	11.5				1.00	1		
3	10	1	6	SPT	1250	1250	0.98	0.38	6.0	1.3	0.80	1.2	1.0	7.2	1.21	8.7	30%	14.7	0.16	1.00	1		
4	15	1	8	SPT	1875	1719	0.97	0.41	8.0	1.3	0.85	1.2	1.0	10.2	1.07	10.9	20%	15.3	0.16	1.00	1	0.37	1.9
5	20	1	9	MC	2500	2031	0.96	0.46	5.9	1.3	0.95	1.0	1.0	6.9	0.99	6.9	30%	12.7	0.14	1.00	1	0.28	2.1
6	25	1	15	SPT	3125	2344	0.94	0.49	15.0	1.3	0.95	1.2	1.0	21.4	0.93	19.8	15%	23.2	0.26	0.95	1	0.47	1.3
7	30	2	27	SPT	3750	2656	0.92		27.0	1.3	1.00	1.2	1.0	40.5	0.87	35.2				0.89	1		
8	35	2	41	MC	4375	2969	0.89		26.7	1.3	1.00	1.0	1.0	33.3	0.82	27.3				0.85	1		
9	40	2	18	SPT	5000	3281	0.85		18.0	1.3	1.00	1.2	1.0	27.0	0.77	20.9				0.84	1		
10	45	2	65	MC	5625	3594	0.80		42.3	1.3	1.00	1.0	1.0	52.8	0.73	38.8				0.79	1		
11	50	1	29	SPT	6250	3906	0.75	0.47	29.0	1.3	1.00	1.2	1.0	43.5	0.70	30.4	30%	39.7		0.77	1		

Total Liquefaction Settlement (in.)= 3.2

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = 2.2/(1.2 + \sigma_v'/P_a)$ with a maximum value of 1.7 per Kayen et al. (1992) as cited in Youd et al. (2001).
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = \alpha + \beta\ (N_1)_{60}$

   where α and β = coefficients determined from the following relationships

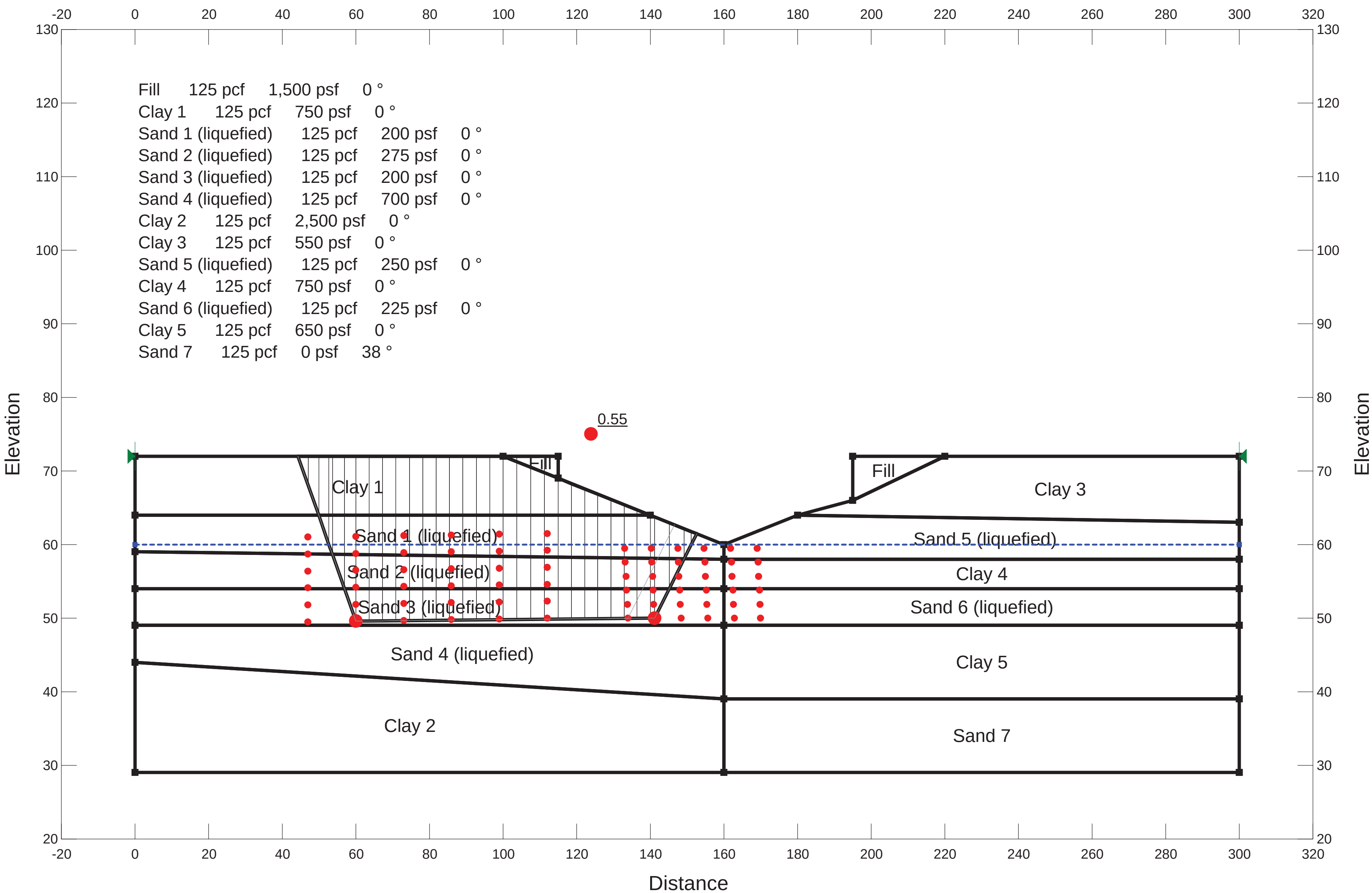
   for FC $\leq$ 5% $\alpha = 0$, $\beta = 1.0$

   for 5% < FC < 35% $\alpha = \exp(1.76-(190/FC^2))$, $\beta = (0.99+(FC^{1.5}/1000))$

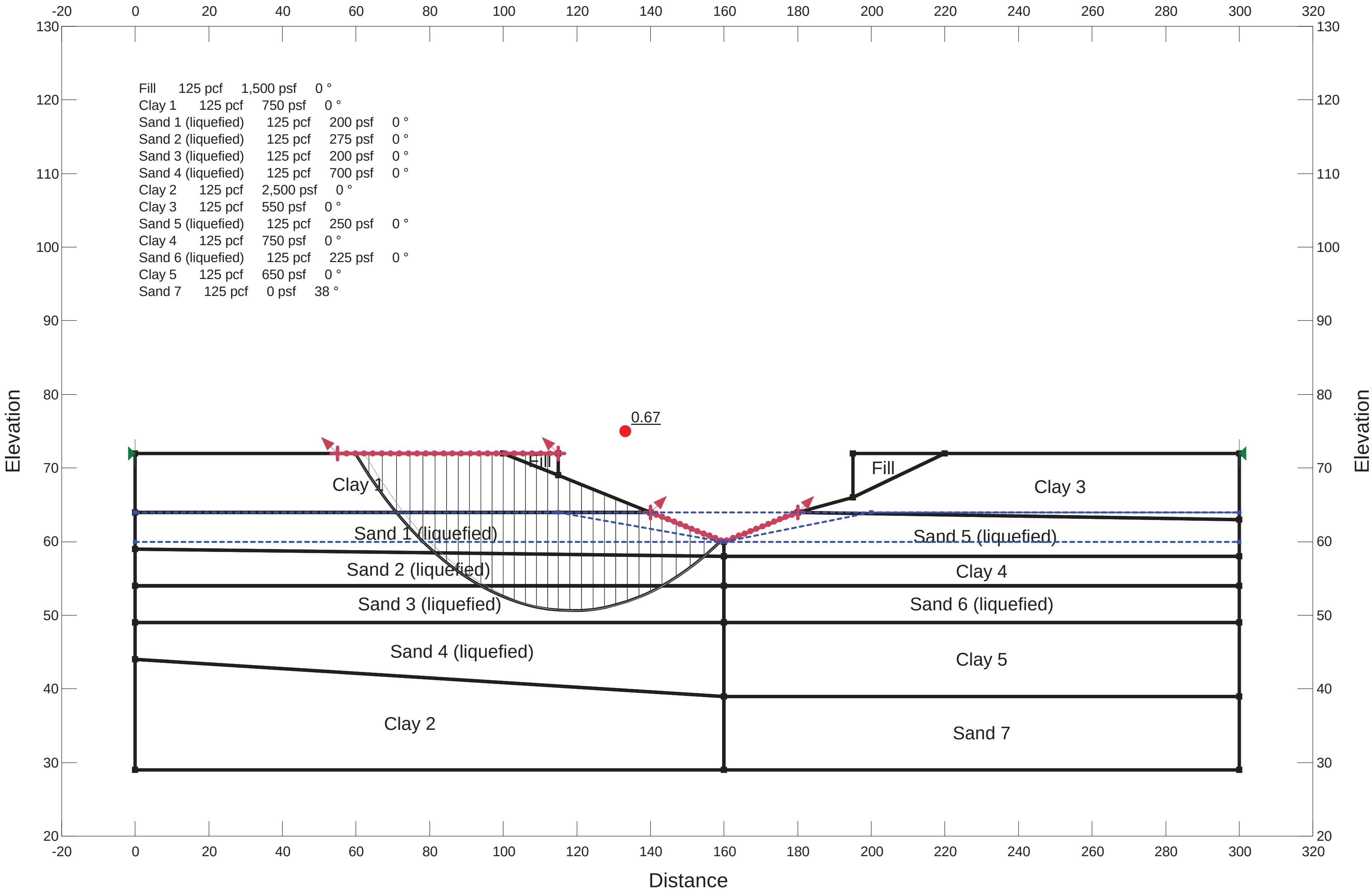
   for FC $\geq$ 35% $\alpha = 5.0$, $\beta = 1.2$

Reference: Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

Pseudo-Static Analysis with Seismic Coefficient = 0.2g (1/3 PGA) and Liquefied Sands



Pseudo-Static Analysis with Seismic Coefficient = 0.2g (1/3 PGA) and Liquefied Sands



SETTLEMENT ANALYSIS

PROJECT NAME **Mid-coast Multi-Modal Trail Phase 2**
PROJECT NO. **2017-108-GEO**
BORING NO. **R-17-001**

Embankment H (ft)=	5	*Contact Pressure (psf)=*	625	*Contact Area, B (ft)=*	20	*Cr/Cc=*	20.0%	***GROUPS*** 1. GRAVELS AND SANDS	
Unit Weight (pcf)=	125	*GW Level (ft)=*	13.5	*Contact Area, L (ft)=*	50	*Ei*	75%	2. CLAYS AND SILTS	
				Plain Strain? (Y/N)=	n				

Soil Type	Depth From	Depth To	BLOW COUNT	SAMPLER TYPE	AVG SPT-N	γ_T (pcf)	γ' (pcf)	ω	σ_v' (psf)	$\Delta\sigma_v'$ (psf)	Su (psf)	Pp (psf)	$Cr/1+e_0$	$Cc/1+e_0$	Settlements (in) OC	NC	SAND	Sum
2	0	4	7	MC	6	127.5	127.5	18.0%	255	546.3	683	2730	0.0240	0.1202	0.574			0.574
2	4	9	8	MC	7	135.5	135.5	16.0%	849	417.4	780	3120	0.0230	0.1151	0.240			0.240
1	9	14	12	MC	10	136.6	74.2	13.0%	1373	322.6							0.121	
2	14	18	10	MC	8	128.7	66.3	21.0%	1691	263.0	975	3900	0.0255	0.1277	0.077			0.077
1	18	23	11	MC	9	138.1	75.7	16.0%	2013	218.9							0.066	
2	23	28	9	MC	7	119.5	57.1	36.0%	2345	181.9	878	3510	0.0330	0.1648	0.064			0.064
2	28	33	10	SPT	13	125.0	62.6	17.0%	2644	153.7	1500	6000	0.0235	0.1177	0.035			0.035
1	33	38	46	MC	37	136.6	74.2	13.0%	2986	131.7							0.014	
1	38	43	45	SPT	56	125.0	62.6	14.0%	3328	114.2							0.008	
1	43	48	11	SPT	14	125.0	62.6	17.0%	3641	99.9							0.016	
1	48	52	28	SPT	35	125.0	62.6	16.0%	3923	89.3							0.007	
1																		
														Estimated Settlement (in)=	*0.99*	*0.00*	*0.23*	***1.22***

SETTLEMENT ANALYSIS

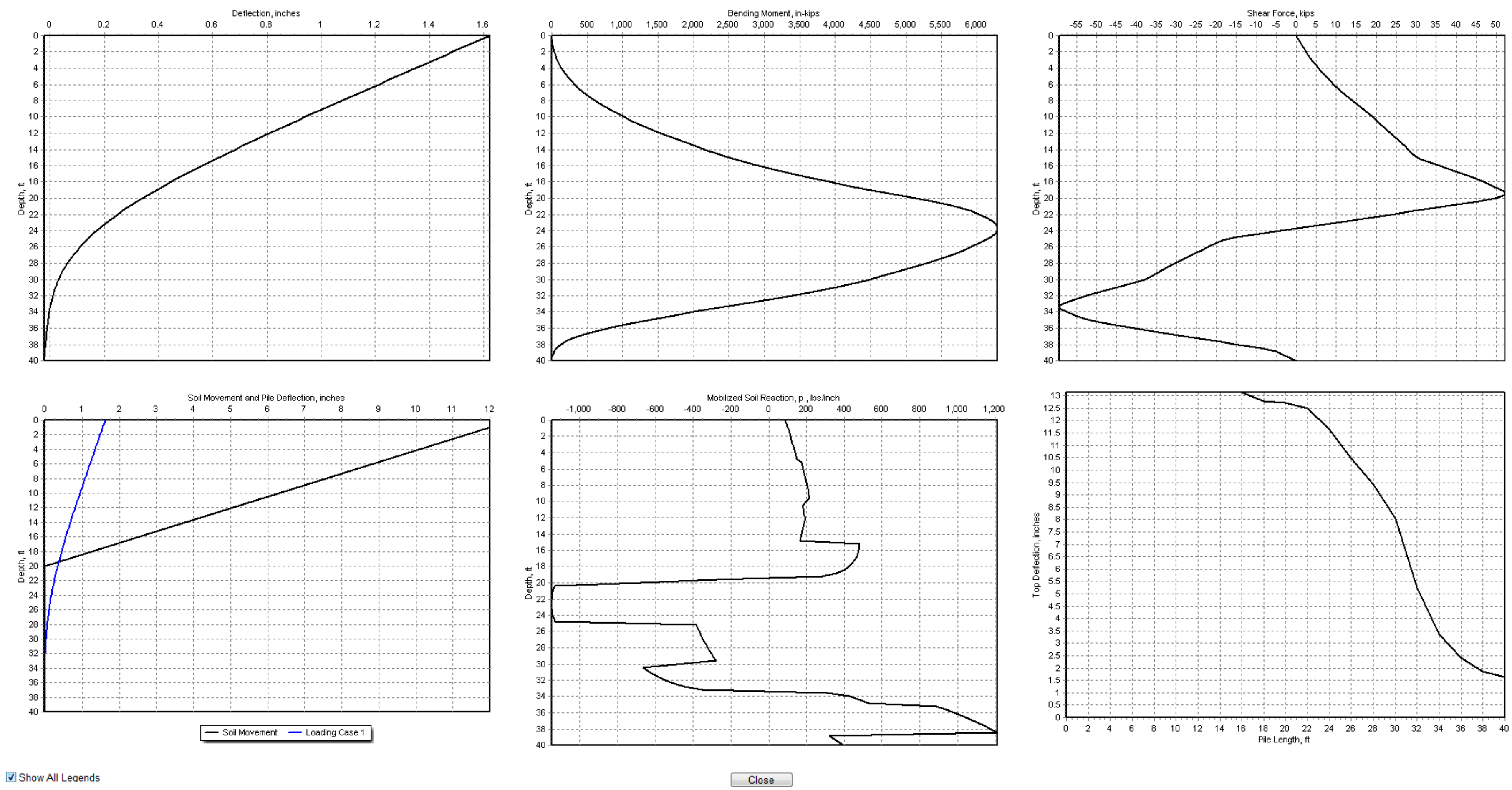
PROJECT NAME	**Mid-coast Multi-Modal Trail Phase 2**
PROJECT NO.	**2017-108-GEO**
BORING NO.	**R-17-002**

Embankment H (ft)=	5	*Contact Pressure (psf)=*	625	*Contact Area, B (ft)=*	20	*Cr/Cc=*	20.0%		
Unit Weight (pcf)=	125	*GW Level (ft)=*	12.5	*Contact Area, L (ft)=*	50	*Ei*	75%		
				Plain Strain? (Y/N)=	n				

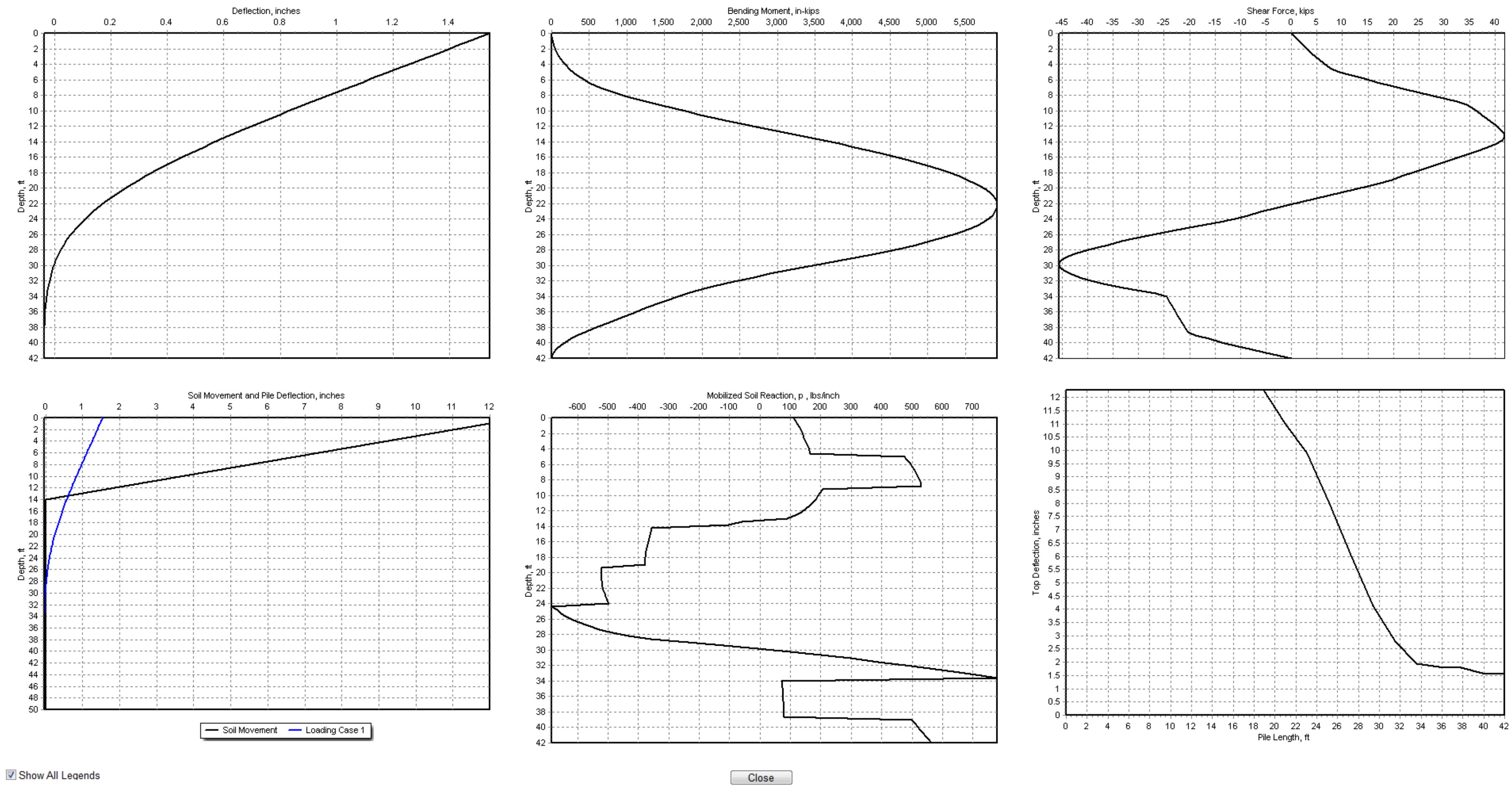
GROUPS
1. GRAVELS AND SANDS
2. CLAYS AND SILTS

Soil Type	Depth From	To	BLOW COUNT	SAMPLER TYPE	AVG SPT-N	γ_T (pcf)	γ' (pcf)	ω	σ_v' (psf)	$\Delta\sigma_v'$ (psf)	Su (psf)	Pp (psf)	$Cr/1+e_0$	$Cc/1+e_0$	Settlements (in) OC	NC	SAND	Sum
2	0	4	9	MC	7	119.5	119.5	12.6%	239	546.3	878	3510	0.0213	0.1066	0.529			0.529
2	4	8	13	MC	11	140.3	140.3	13.8%	759	429.3	1268	5070	0.0219	0.1096	0.205			0.205
1	8	13	6	SPT	8	125.0	62.6	16.2%	1196	338.7							0.155	
1	13	18	8	spt	10	125.0	62.6	17.4%	1509	268.8							0.095	
1	18	23	9	MC	7	125.0	62.6	20.8%	1822	218.9							0.076	
1	23	28	15	SPT	19	125.0	62.6	19.3%	2135	181.9							0.038	
2	28	33	27	SPT	34	125.0	62.6	17.2%	2448	153.7	4050	16200	0.0236	0.1182	0.038			0.038
2	33	38	41	MC	33	132.6	70.2	15.1%	2780	131.7	3998	15990	0.0226	0.1129	0.027			0.027
2	38	43	18	SPT	23	125.0	62.6	20.5%	3112	114.2	2700	10800	0.0253	0.1264	0.024			0.024
2	43	47	65	MC	53	131.8	69.4	15.8%	3407	101.2	6338	25350	0.0229	0.1146	0.014			0.014
1	47	52	29	SPT	36	125.0	62.6	18.4%	3702	90.4							0.009	
1																		
													Estimated Settlement (in)=		*0.84*	*0.00*	*0.37*	***1.21***

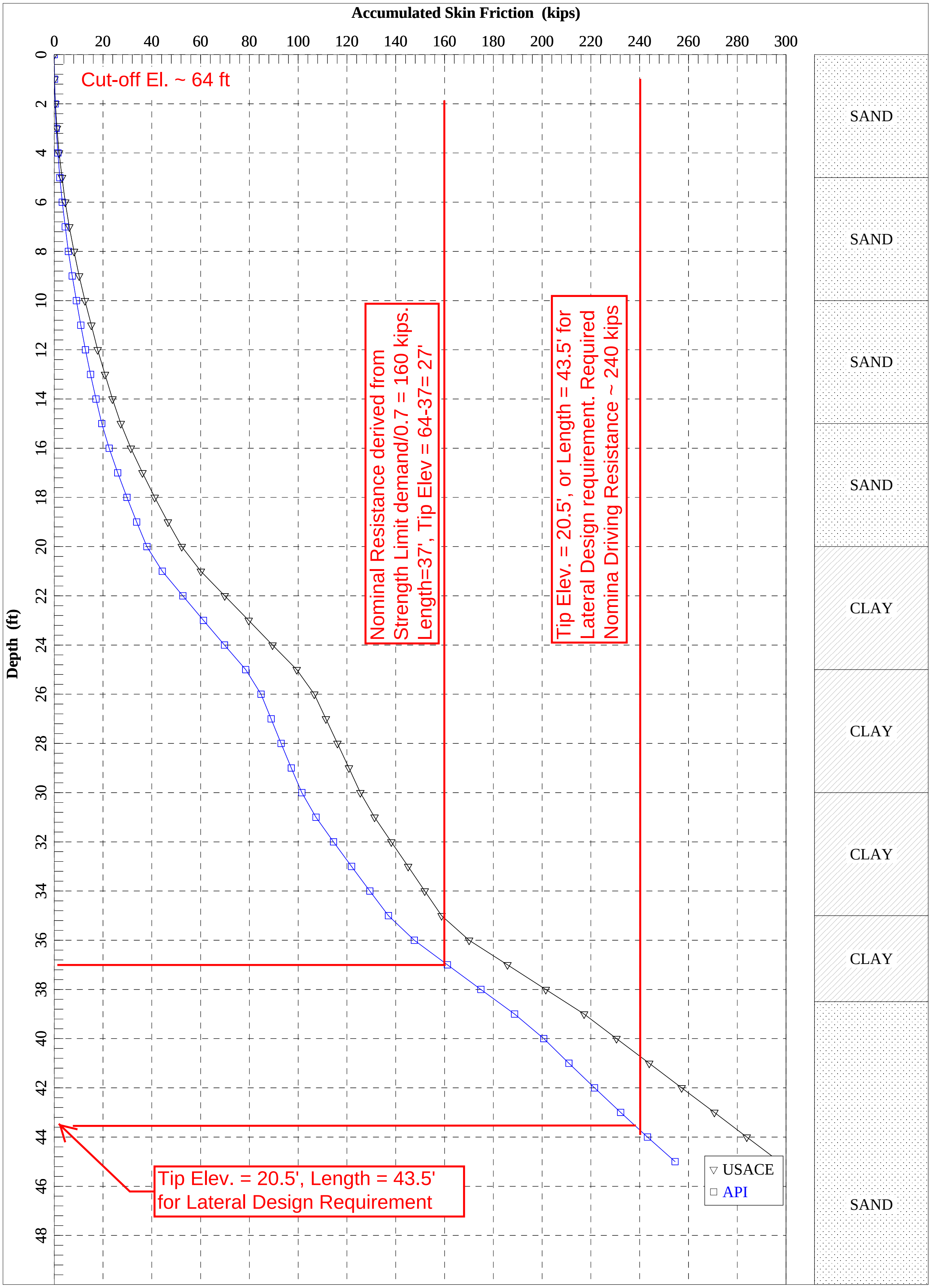
LPILE ANALYSIS OUTPUT
Abutment 1, CISS = 30 x 5/8 in, Cut-off El. = 64 ft, R-17-002
With Lateral Spreading Soil Movement at Pile Top



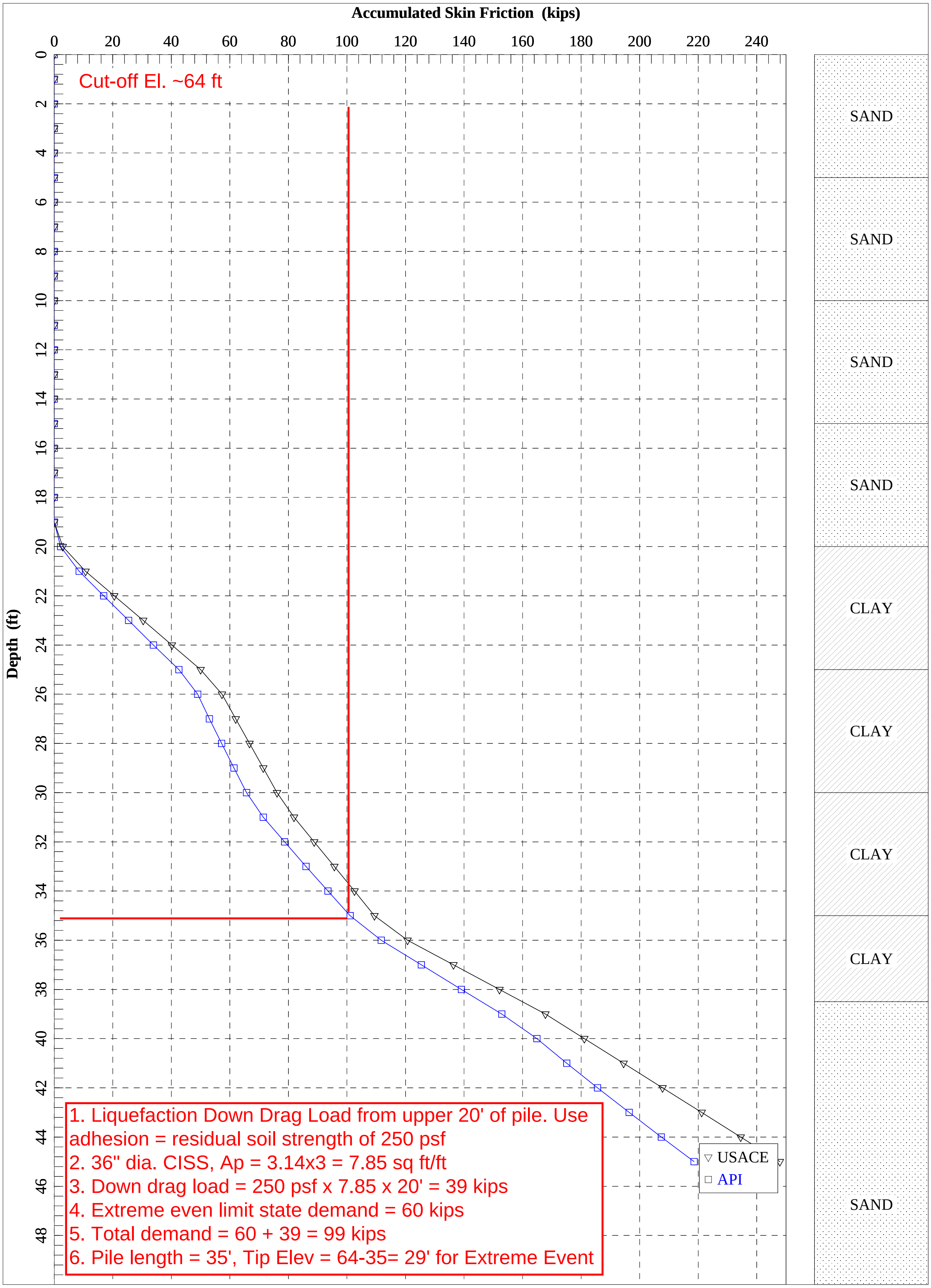
LPILE ANALYSIS OUTPUT
Abutment 2, CISS = 30 x 5/8 in, Cut-off El. = 63 ft, R-17-001
With Lateral Spreading Soil Movement at Pile Top



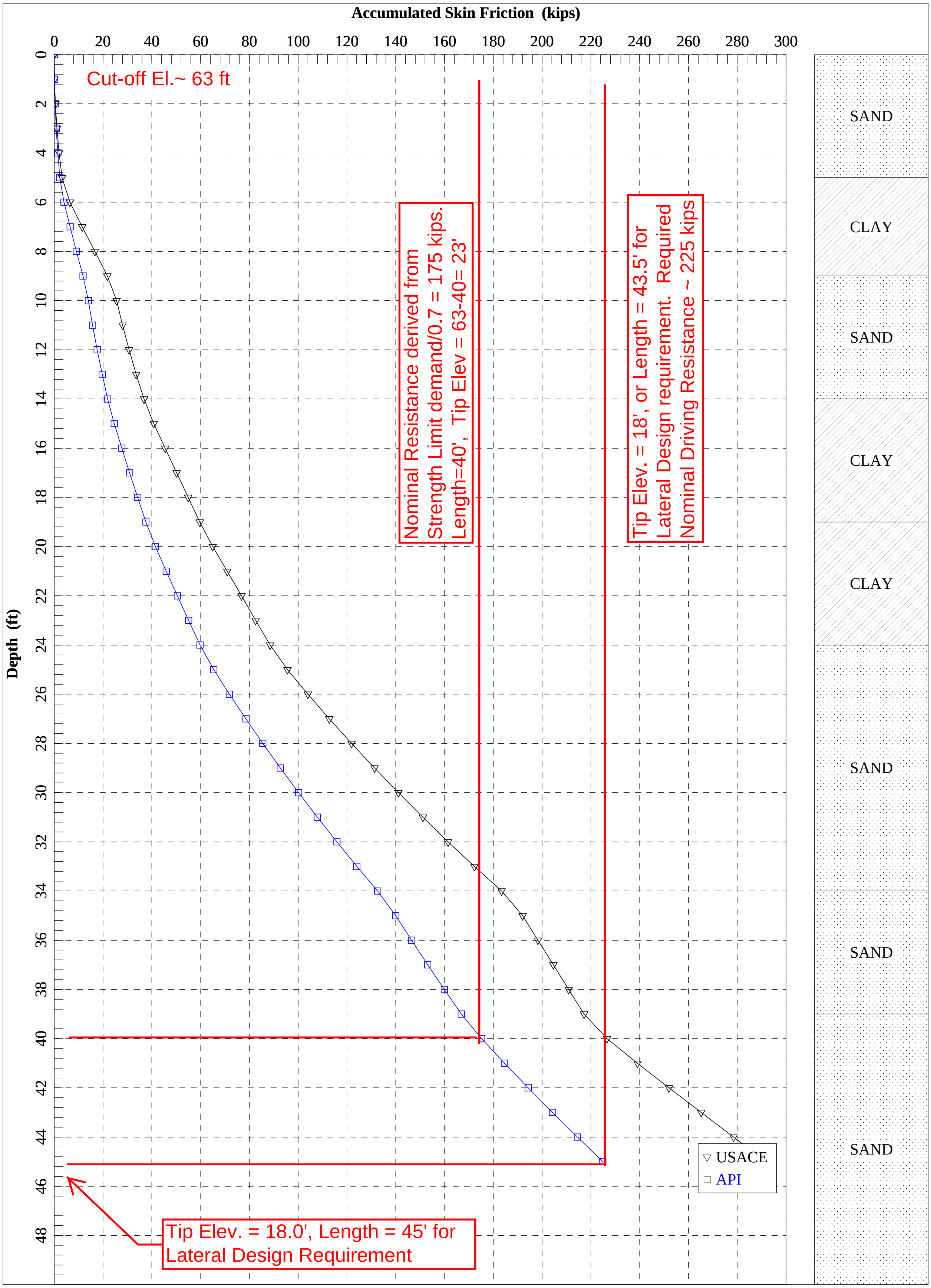
Abutment 1
CISS = 30 x 5/8 inch, Strength Limit, R-17-002



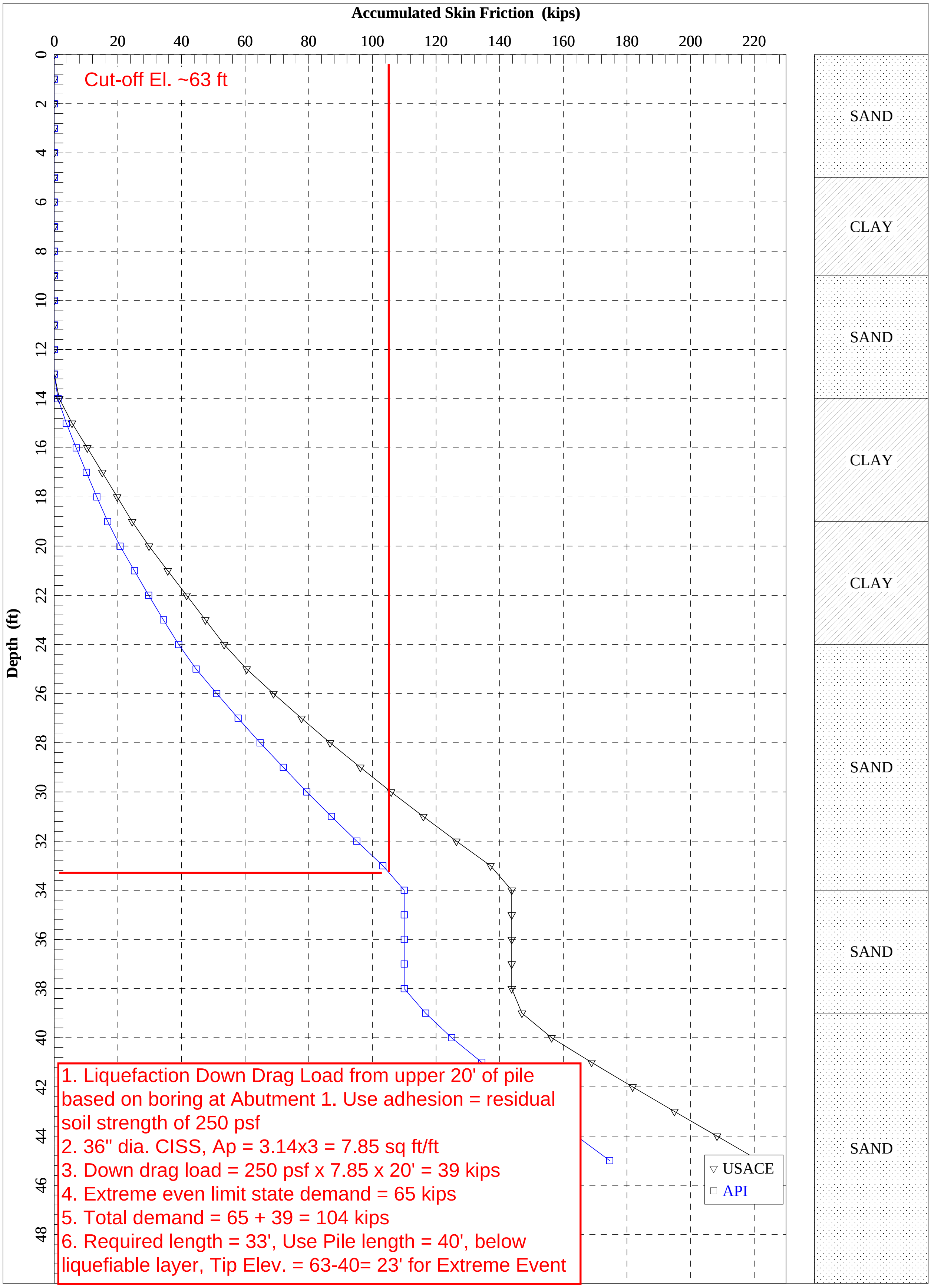
Abutment 1
CISS = 30 x 5/8 inch, Extreme Event Limit,
R-17-002



Abutment 2
CISS = 30 x 5/8 inch, Strength Limit, R-17-001



Abutment 2
CISS = 30 x 5/8 inch, Extreme Event Limit,
R-17-001



APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 08/2011)

General Project Information (OSFP Liaison to complete)			**Review Phase** (OSFP Liaison to complete)	**Reviewer Information** (Reviewer Liaison to complete)	
Dist:	**4**	☐	PSR/PDS (Review No.)	Reviewer Name:	**Kanax Kanagalingam; Mahmood Momenzadeh**
Proj ID (Phase):	**0417000246**	☐	APS/PSR (Review No.)	Functional Unit:	**Geotech Design-West**
Project Name:	**Midcoast Multi-modal Trail-Phase 2-Ped O/C and Retaining wall**	☐	APS/PR (Review No.)	Cost Center:	**59-3660**
OSFP Liaison:	**Emil Vergara**	☐	Type Selection	Phone Number:	**510-622 5772; 510-286 5732**
Phone:	**916-227-8360**	☑	65% PS&E Unchecked Details	e-mail:	Thangalingam.kanagalingam@dot.ca.gov Mahmood.momenzadeh@dot.ca.gov
E-mail:		☐	PS&E (Review No. 1)	Date of Review:	**08/23/2018**
		☐	Construction	Structure Name*:	
		☑	Other: FOUNDATION REPORT	Br No*:	
				(*Use if necessary to when comment sheets are by individual structure)	

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	E-mail	Response Date
Y. David Wang	**Parikh Consultants**			

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	Page 1, Section 2.0	Include text to describe the datum used for the project	Concur.	
2	FR	Page 5, Section 10.0	Based on the updated 2018 corrosion guidelines, Caltrans considers a site is corrosive when soil contains a sulfate concentration of 1,500 ppm or grater. Please update the text.	Concur.	
3	FR	Page 7, Section 11.2	Include classification of the site per the Caltrans Seismic Design Criteria (SDC), “Classification of Soils.”	Concur. It is “Marginal Soil” per SDC 1.7, which is categorized as S2 per SDC 2.0.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer’s use)

Submittal Data (Reviewer to complete)
Project ID: Reviewer: Str Name*:
Date of Review: Functional Unit: Br No*. *=if applicable

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
4	FR	Pages 9-11; Section 11.3	State that the structure enhancement approach using CISS piles with 5/8-inch wall thickness was selected as the preferred option for limiting the effect of lateral spreading; Also mention that the ground improvement option was not selected for this project.	Concur.	
5	FR	Page 13, Section 12.3	Report states, "Since the pile cut-off elevations are within the potentially liquefiable soils, down drag forces caused by liquefied soils are insignificant and do not control design". Please clarify this statement.	We updated the design to account for down drag load. The load is calculated using residual soil strength (Sr = 250 psf) as the adhesion through the liquefaction zone. The additional load is considered in the Extreme Event limit state case of the LRFD design.	
6	FR	Page 13, Section 12.3	Based on the report, the post-liquefaction settlement is estimated to be about 3 to 4 inches. Therefore, this post-liquefaction settlement will induce down drag forces on the piles. However, down drag was not considered in the pile capacity calculations.	Concur. Down drag load is now considered in the Extreme Event limit state. See response for Item No. 5.	
7	FR	Page 13, Section 12.3	Driving Resistance was included in Table 12.4, that is different from nominal resistance. Explain the approach used to calculate the driving resistance.	The Driving Resistance is estimated from normal soil condition without liquefaction and is intended for use in the field for pile installation. The nominal resistance is estimated for design and conservatively assumed liquefaction condition.	
8	FR	Page 13, Section 12.3	APILE analysis results included in Appendix C show that the skin friction within each liquefiable soil layer is zero. Please describe the assumptions involved in the calculation of nominal axial capacity.	The APILE analyses assumed liquefaction state and no capacity contribution in the liquefied zone. Per current comment, we have added down drag load due to liquefaction for the Extreme Even limit state.	
9	FR	Tables 12.5 and 12.6	Appendix B of the report includes several moisture-density test results for sandy and clayey soil samples collected from different depths. However, same effective unit weights were used for both sandy and clayey soils.	Noted. In our experience, the soil density of 120-125 pcf has been commonly used. The lab tests confirmed that these are reasonable values and do not affect design from practical standpoint for project like this.	
10	FR	Page 16, Section 12.5	For abutment and wing walls with no hydrostatic pore pressure buildup behind the wall, the recommended incremental active seismic pressure is 30 pcf equivalent fluid pressure (in a regular triangular shape). For walls with hydrostatic pressure, the	The site PGA is 0.6 g. We followed AASHTO LRFD specs using 50% PGA as the design Kh (= 0.3 g). Assuming structure backfill of 34-35 deg and conservatively no cohesion, the total Kae is 0.5 (AASHTO LRFD Appendix A11). With a little bit of cohesion (say 50-100 psf) for seismic case per AASHTO, the total Kae can	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved
(for Reviewer's use)

Submittal Data (Reviewer to complete)

Project ID: Reviewer: Str Name*:

Date of Review: Functional Unit: Br No*. *=if applicable

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
			recommended incremental active seismic pressure is 90 pcf equivalent fluid pressure. Could you describe the approach used for the calculation of these incremental active seismic pressures and associated pressure distributions (regular triangular shape)?	be in the range of 0.4 to 0.5. The static Ka is ~0.28, so the delta Kae is ~0.12 to 0.22. For drained case, the incremental earth pressure is about 125 x 0.22 (max) = 27 pcf (say, 30 pcf for design). For undrained case, (125-62.4) pcf x ~0.45 = 28 pcf. With water pressure, the total lateral seismic pressure is ~90 pcf. The normal triangular distribution is per AASHTO LRFD specs Appendix A11.3.1 for routine wall design.	
11	FR	Page 17, Section 13.0	Incremental active seismic pressure for the soldier pile retaining wall is provided as 30 pcf equivalent fluid pressure (in a regular triangular shape). See comment 6.	We understand that the comment refers to Item No. 10. The same response above applies.	
12	FR	Page 19, Section 15.0	Include a reference for the limitations provided for the potential materials that can be used as fill.	Concur. Caltrans standard specifications (Sect. 19-6.02) state that using min. sand equivalent value of 10 within 2.5 ft of finished grade.	
13	90% Design Plans	General	Include notes for horizontal and vertical datum used for the plans.	Concur	
14	FR & 90% Design Plans	S2: Sheet 34 of 40 and FR: Table 12.4	The recommendations presented for “Specified Tip Elevation” and “Driving Resistance” in the pile data tables shown on the Plan Sheet 34 of 40 (S2) and FR Table 12.4 did not match. Please correct.	Concur.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer’s use)

OSFP Rev Form 12/2010

Office of Special Funded Projects
Comment & Response Form

(Revised 08/2011)

General Project Information (OSFP Liaison to complete)		**Review Phase** (OSFP Liaison to complete)	**Reviewer Information** (Reviewer Liaison to complete)	
Dist:	**4**	☐ PSR/PDS (Review No.)	Reviewer Name:	**Betty Lee**
Proj ID (Phase):	**0417000246**	☐ APS/PSR (Review No.)	Functional Unit:	**Geotech Design--West**
Project Name:	**Midcoast Multi-modal Trail-Phase 2-Ped O/C and Retaining wall**	☐ APS/PR (Review No.)	Cost Center:	**59-3660**
OSFP Liaison:	**Emil Vergara**	☐ Type Selection	Phone Number:	**510-286-4825**
Phone:	**916-227-8360**	☑ 65% PS&E Unchecked Details	e-mail:	**Betty_lee@dot.ca.gov**
E-mail:		☐ PS&E (Review No. 1)	Date of Review:	**3/28/2018**
		☐ Construction	Structure Name*:	
		☑ Other: FOUNDATION REPORT	Br No*:	
			(*Use if necessary to when comment sheets are by individual structure)	

Consultant Information (to be filled in by Consultant)				
Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	E-mail	Response Date
Peter Wei	**Parikh Consultants**	**408-452-9000**	**pwei@parikhnet.com**	**4/5/2018**

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	12	Axial capacity of proposed piles was analyzed with APILE using working stress method. Per Caltrans policy, we are required to use LRFD method.	The axial pile capacity demands were determined according to the Caltrans LRFD method. Geotechnical resistance factors of 0.7 and 1.0 have been applied to the factored loads at the strength/construction limit state and extreme event limit state, respectively, to obtain the required nominal axial pile capacity. APILE was used to help analyze axial pile capacity with the built-in “Revised API Recommended Practice 2A Method (1987-2007).” The API method uses an adhesion factor α for cohesive soils and a lateral earth coefficient *k* for cohesionless soils in combination with effective overburden pressures and shear strengths or friction angles of soils to	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Submittal Data (Reviewer to complete)

Project ID: Reviewer: Str Name*:

Date of Review: Functional Unit: Br No*. *=if applicable

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
				estimate the skin friction on piles. There is no connection between the API method and "working stress method" or "LRFD method." The reference of American Petroleum Institute (API, 2007) "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design" was incorrectly cited on Page 12, Section 12.3, of the 1/25/18 foundation report, and will be removed from the next report submittal.	
2					
3					
4					
5					

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(for Reviewer's use)

OSFP Rev Form 12/2010