COUNTY OF SAN MATEO - PLANNING AND BUILDING DEPARTMENT

ATTACHMENT J

WAYNE
TING &
ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

Project No. 5264
5 May 2019

Ms. Fuli Li
1855 Sunshine Valley Road
Moss Beach, CA 94038

Subject: **UPDATE GEOTECHNICAL INVESTIGATION**
Proposed Single Family Structure
1855 Sunshine Valley Road
Moss Beach, California

References:
1. Geotechnical Study
By Sigma Prime Geosciences, Inc, August 2018
2. Guidelines for Evaluating and Mitigating Seismic Hazards in California
Special Publication 117A, Division of Mines and Geology, 2008
3. Seismic Hazard Zone Report for the Montara Mountain 7.5-Minute Quadrangle, San Mateo County, California, 2019
4. Assessment of the Liquefaction Susceptibility of Fine-Grain Soils
By Jonathan D. Bray and Rodolfo B. Sancio, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, September 2006, pp.1165-1177
5. Estimating Liquefaction-Induced Lateral Displacements Using the Standard Penetration Test or Cone Penetration Test,
G. Zhang, P.K Robertson, M.ASCE, and R.W.I. Brachman

Dear Ms. Li:

In accordance with your authorization, **Wayne Ting & Associates, Inc. (WTAI)** has completed a geotechnical investigation for the proposed single family structure at the subject site. The purpose of this study was to investigate the subsurface conditions and obtain geotechnical data for use in the design and construction of the proposed single family structure. The scope of this investigation included the following:

a. Review of Reference 1.
b. A site and area reconnaissance by the Project Engineer.
c. An excavation, logging and sampling of one exploratory boring.
d. Laboratory testing of the selected soil samples.
e. An engineering analysis of the data and information obtained.
f. Preparation and writing of this report which presents our findings, conclusions, and recommendations.

Our findings indicate that the proposed improvements are feasible from a geotechnical engineering standpoint provided the recommendations in this report are carefully followed.

FIELD INVESTIGATION

WTAI conducted the field investigation on April 23, 2019. The field investigation consisted of a site reconnaissance and an excavation of one exploratory boring. The boring was excavated using a truck mounted drill-rig with 6-inch augers. The approximate location of the boring is shown on the Site Plan, Figure 1.

Soils encountered during the excavation operation were continuously logged in the field. Relatively undisturbed samples were obtained by dynamically driving 18 inches using a 3.0-inch outside diameter Modified California Sampler with a 140-pound hammer free falling 30 inches. Blow counts were recorded for every 6-inch penetration interval, and reported corresponding to the last 12 inches of penetration. These samples were then sealed and returned to the laboratory for testing. The classifications, descriptions, natural moisture contents, dry densities, and depths of the obtained samples are shown in the Boring Log, Figure 2 of Appendix A.

LABORATORY TESTING

CLASSIFICATION

The field classifications of the samples were visually verified in the laboratory in accordance with the Unified Soil Classification System. These classifications are presented in the Boring Log, Figure 2.

MOISTURE-DENSITY

The natural moisture contents and/or dry weights were determined for selected soil samples obtained during our field investigation. The data is presented in the aforementioned Boring Log.

ATTERBERG LIMITS

The liquid limit and plasticity index of the soil are taken from Reference 1:

Sample	***Liquid Limit***	***Plasticity Index***
Brown Sandy Clay (CL)	31%	14

The Atterberg Limits tests indicate that representative samples of the soils are of low plasticity. The expansion potentials for these soils are thus low.

SUBSURFACE SOIL CONDITIONS

The following soil descriptions were derived from our site reconnaissance and information obtained from our exploratory boring samples. Detailed descriptions of the materials encountered in the exploratory boring and results of the laboratory testing are presented in the Boring Log, Figure 2.

Boring 1 soil encountered at the site consisted of 5.0 feet of dark brown clayey sand, loose and moist, below the existing ground surface (BGS), followed by dark brown sandy clay, firm to stiff and very moist, to 14.0 feet BGS, followed by gray silty sand, fine to medium, moist to very moist, medium dense to dense, to 32.0 feet BGS, followed by granodiorite bedrock, weathered and fractured, to the maximum refusal depth explored of 37.0 feet BGS.

Groundwater was encountered at 31.0 feet below the existing ground surface in the exploratory boring at the time of our field study. Fluctuations in the groundwater table are anticipated to vary with respect to seasonal rainfall. Historical groundwater level is 3.0 feet according to Reference 3.

SEISMIC CONSIDERATIONS

According to the published maps by the International Conference of Building Officials (I.C.B.O.), in February 1998, the distances from active faults to the subject site are listed in the following table.

Fault Name	Distance (kilometers)	Direction From Site
San Gregorio North	0.3	East
San Andreas	10.4	Northeast

CALIFORNIA BUILDING CODE SITE CHARACTERIZATION

The following design values are base on the geologic information, longitude and latitude of the site, and the USGS computer program. Furthermore, in according with Chapter 16 of the 2016 California Building Code (CBC), the site seismic design values are provided as follow:

CBC Category/Coefficient 2010 ASCE 7-10 (with March 2013 errata)	Design Value
Short-Period MCE at 0.2s, Ss	2.330
1.0s Period MCE, S1	0.988
Soil Profile Type, Site Class	Sd
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.5
S_{MS} = Fa x S_s Spectral Response Accelerations	2.330
S_{M1} = Fv x S_1 Spectral Response Accelerations	1.482

$S_{DS} = 2/3 \times S_{MS}$	Design Spectral Response Accelerations	1.553
$S_{D1} = 2/3 \times S_{M1}$	Design Spectral Response Accelerations	0.988

** Latitude: 37.5285607 Longitude: -122.5081457

It is noted that final values should be determined by the project structural engineer according to risk categories of the proposed improvements.

QUANTITATIVE LIQUEFACTION ANALYSIS USING SPT

Liquefaction is a phenomenon in which saturated (submerged), cohesionless soils are subjected to a temporary loss of strength due to the buildup of pore water pressures, especially as a result of cyclic loadings induced by earthquakes or ground shaking. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical deformations, if not confined. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine sands.

Our liquefaction analysis followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for SPT analysis update simplified procedures presented by Seed and Idriss (1971). These methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, and earthquake magnitude. The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65\,(a_{max}/g)(s_{vo}/s_{vo})r_d$$

Where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g, is the acceleration of gravity, s_{vo} and s_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. We evaluated the liquefaction potential of the soil strata encountered assuming the ground water depth of 3.0 feet according to Reference 3. In addition, a peak ground acceleration, PGAm, of 0.912g from USGS (PSHA, 10% exceedance in 50 years) and magnitudes of 7.86 were obtained from Reference 3 for analysis.

The factor of safety (FS) against liquefaction can be expressed as the ratio of the CRR to CSR. FS = CRR/CSR. If the FS for a soil layer is less than 1.0, the soil layer is considered liquefiable during a moderate to a large seismic event. If CRR/0.65 $(a_{max}/g)(s_{vo}/s_{vo})r_d$ is 1.0 or larger, the soil layer can be considered to be non-liquefiable.

We analyze the site liquefaction potential utilizing a computer program call GeoSuite by GeoAdvanced; this program is based on the most recent publications of NCEER Workshop and procedure outline in SP117A Implementation.

Based on our analysis using Idriss & Boulanger (2008) and the factor of safety 1.0, the settlement results of the liquefaction analysis are presented in following Table 1 and in Appendix A.

TABLE 1

SPT Boring No.	*Ground Water Depth*	*Dry Settlement (inches)*	*Saturated Settlement (inches)*	*Total Settlement (inches)*	*Differential Settlement (inches)*
1	3.0 feet	0	1.46	1.46	0.73

Total Settlement: Saturated settlement plus dry settlement

Estimates of volumetric change for dry settlement were made by Yi (2010). Estimates of volumetric change for saturated settlement were made by Idriss & Boulanger (2008). As discussed in the Southern California Earthquake Center report (SCEC, 1999), differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement.

LATERAL SPREADING

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvium material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soils displace laterally toward the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

The current creek is located close to the proposed site improvements with a free face of about approximately 4-5 feet deep. Our GeoSuite program analyzed the displacement of 9.9 inches. Therefore, it to WTAI's opinion that the probability of lateral spreading affecting the site during a seismic event is high.

LIQUEFACTION MITIGATION

Liquefaction mitigation measures generally falls into many categories. The preferred approach is to do grading operation by removing the loose top sand layers at least 12 inches below the bottom of creek and backfilling using onsite soils. The backfill soils should be compacted to 90 percent. As the top layers are mitigated by grading operation, then the total and differential liquefaction induced settlement will be reduced to approximately 0.7 and 0.35-inch. Mat slab foundation can be used to handle this settlement.

After reviewing the Reference 2 through and 5, WTAI opinion is that the lateral spreading only occurs when there is liquefiable soil present. As the potential liquefiable soils are improved by the grading operation then the lateral spreading for the site will be low.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

1. Based on the results of our investigation, WTAI concludes that the subject site is geotechnically suitable for the proposed structure provided the recommendations presented in this report are incorporated into the project plans and specifications. WTAI should review the foundation plans and specifications so that comments can be made regarding the interpretation and implementation of our geotechnical recommendations in the design and specifications.

2. It is recommended that WTAI be retained for observation during foundation construction phases to help determine that the design requirements are fulfilled. Our firm should be notified at least 2 working days prior to grading and foundation operations on the property.

3. The recommendations given in this report are applicable only for the design of the previously described structure and only at the location indicated on the site plan. They should not be used for any other purpose.

4. The following recommendations are based on the information provided, results of the subsurface investigation and laboratory testing, as well as our experience with similar soil conditions. The possibility of subsurface conditions at the site vary from those encountered in the boring always exists. If there are any unusual conditions differing significantly from those described herein, this firm should be notified immediately to review the effects on the performance of the designed foundations.

SITE PREPARATION AND GRADING

5. Prior to grading, the proposed structure areas should be cleared of all obstructions and deleterious materials.

6. After clearing, these areas should be stripped of all organic topsoil. It is estimated that stripping depths of 4 to 6 inches may be necessary. However, final stripping depths should be determined by WTAI in the field. The predominantly organic material from the stripping should be removed from the site.

7. After the stripping, the upper 6 feet of sandy soil measured from the proposed pad grade should be overexcavated to 5 feet beyond the proposed structure. The top 12 inches of exposed native ground should be scarified and then recompacted to 2 percent above optimum moisture content, a minimum degree of relative compaction of 90 percent. Relative compaction is based on the maximum dry density as determined by ASTM D1557 Latest Version Laboratory Test Procedure.

8. After the recompaction of native soil, the site may be filled to the desired finished grade using the overexcavated on site native soils and compacted to a minimum relative compaction of 90 percent, at 2% above optimum moisture content. Compaction of each layer shall be continuous over the entire fill area and continued until the required density is obtained.

FOUNDATION

9. Due to the liquefaction induced settlement, the proposed structure should be supported on a mat slab foundation. The design concepts of the mat slabs are to reinforce them in such a way that they are rigid enough to move as a single unit in the event of differential soil movement. If properly reinforced and constructed, differential movement should not impart damaging stress to the structure itself. Although these systems are able to resist some movement, the possibility exists that if there are some differential movements, some tilting may occur.

10. Modulus of subgrade reaction of 50 k.c.f. should be used for the design. The bottoms of the perimeters of slabs should be 8.0 inches below the recommended bottom of crushed rock in item 14a.

11. The slabs should be designed based on the allowable bearing capacity of 1,500 p.s.f. due to dead loads plus design live loads, and 1,800 p.s.f. due to all loads which include wind or seismic forces.

12. The available resistance to lateral loads when utilizing structural slabs is limited to a sliding resistance along the base of the slabs. Sliding resistance between the bottom of the slabs and the underlying soil should be based on a friction value of 0.30.

13. Settlements under the anticipated loads are estimated that the total settlement will be approximately 1.5-inches, and post-construction differential settlements across the structure should not exceed approximately 1.0-inch during the life of the structures following construction.

CONCRETE SLAB-ON-GRADE

14. To reduce the cracking potential of the concrete slabs under the proposed residential structure areas the following recommendations are made:

a. 4-inches thick of 3/4-inch clean crushed rock acting as a cushion and capillary break between the subsoil and the slab.

b. In areas where moisture transmission through slabs is undesirable, a better impermeable membrane of such as, Buthuteme, Paraseal or equal should be placed according to the instruction of the manufacture and the specification of foundation plans.

c. Design waterproofing for the concrete is not within the purview of WTAI. Waterproofing should be designed by a professional waterproofing designer.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

15. Our client should recognize that every effort made to evaluate the subsurface conditions at this site is based on the samples recovered from the test boring and the results of laboratory tests on these samples. The owner or his representative should be reminded that unanticipated subsurface conditions are commonly encountered and cannot be fully determined by taking subsurface samples, and frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended, to accommodate these required extra costs.

16. The conclusions and recommendations contained in this report will not be considered valid after a period of two years unless the changes are reviewed, and the conclusions of this report are modified or verified in writing. This report is prepared for the exclusive use of this project. Our professional services, findings, and recommendations were prepared in accordance with generally accepted engineering principles and practices. No other warranty, expressed or implied, is made.

17. This report is issued with the understanding that it is the responsibility of the owner or his representative, to ensure the information and recommendations contained in this report are brought to the attention of the Architect, Engineer and Contractor. In all cases, the contractor shall retain responsibility for the quality of the work and for repairing defects regardless of when they are found. It is also the responsibility of the contractor for conforming to the project plans and specifications.

Should you have any questions relating to the contents of this report, please contact our office at your convenience.

Very truly yours,

WAYNE TING & ASSOCIATES, INC.

Wayne Ting

REGISTERED PROFESSIONAL ENGINEER
WAYNE L. TING
No. C 46276
Exp. 12/31/20
CIVIL
STATE OF CALIFORNIA

Wayne L Ting, C.E.
Principal Engineer

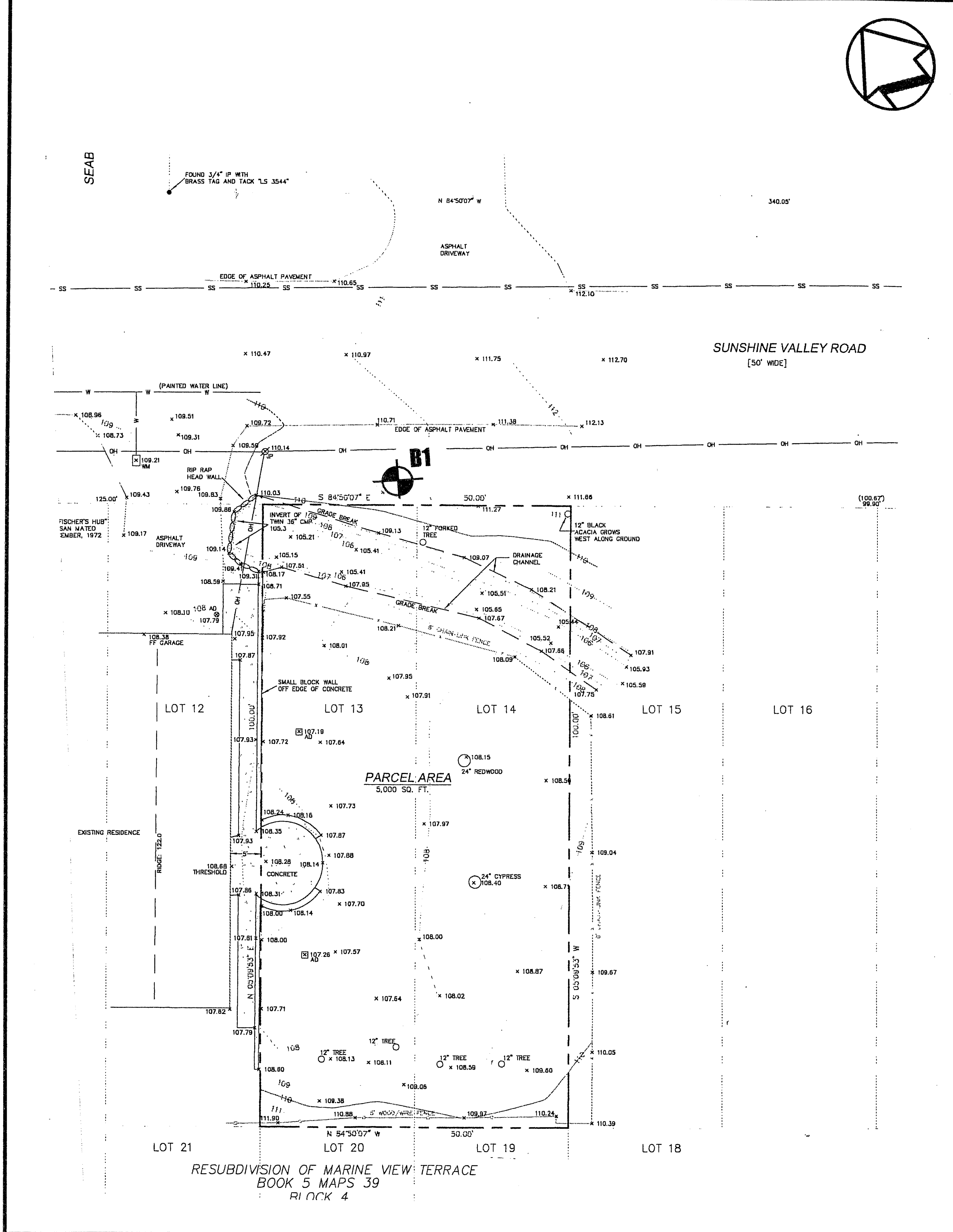
Copy: 1 to Ms. Li

APPENDIX A

Site Plan, Figure 1

CPT Boring Log, Figures 2 and 3

CPT Analysis, Figures 4 and 5



WAYNE TING & ASSOCIATES, INC.	*Site Plan*	*Figure No. 1*
GEOTECHNICAL CONSULTANTS	Scale : N/A	*Page No. 10*

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
0	Asphaltic concrete pavement							
1	Dark brown clayey sand, fine to medium, loose and very moist		SC					
3		1-1		11	98.0	24.0		
5	Dark brown sandy clay, firm and very moist		CL					
6		1-2		9	85.1	32.9		
8	stiff	1-3		16	108.5	20.3	2.5	
12	Gray sandy clay, firm and moist							
13		1-4		7	108.0	18.5		
14	Gray silty sand, fine to medium, moist and medium dense		SP					
18	very moist	1-5		25	100.9	24.1		
23	very dense and moist	1-6		>50	111.2	14.3		
25								

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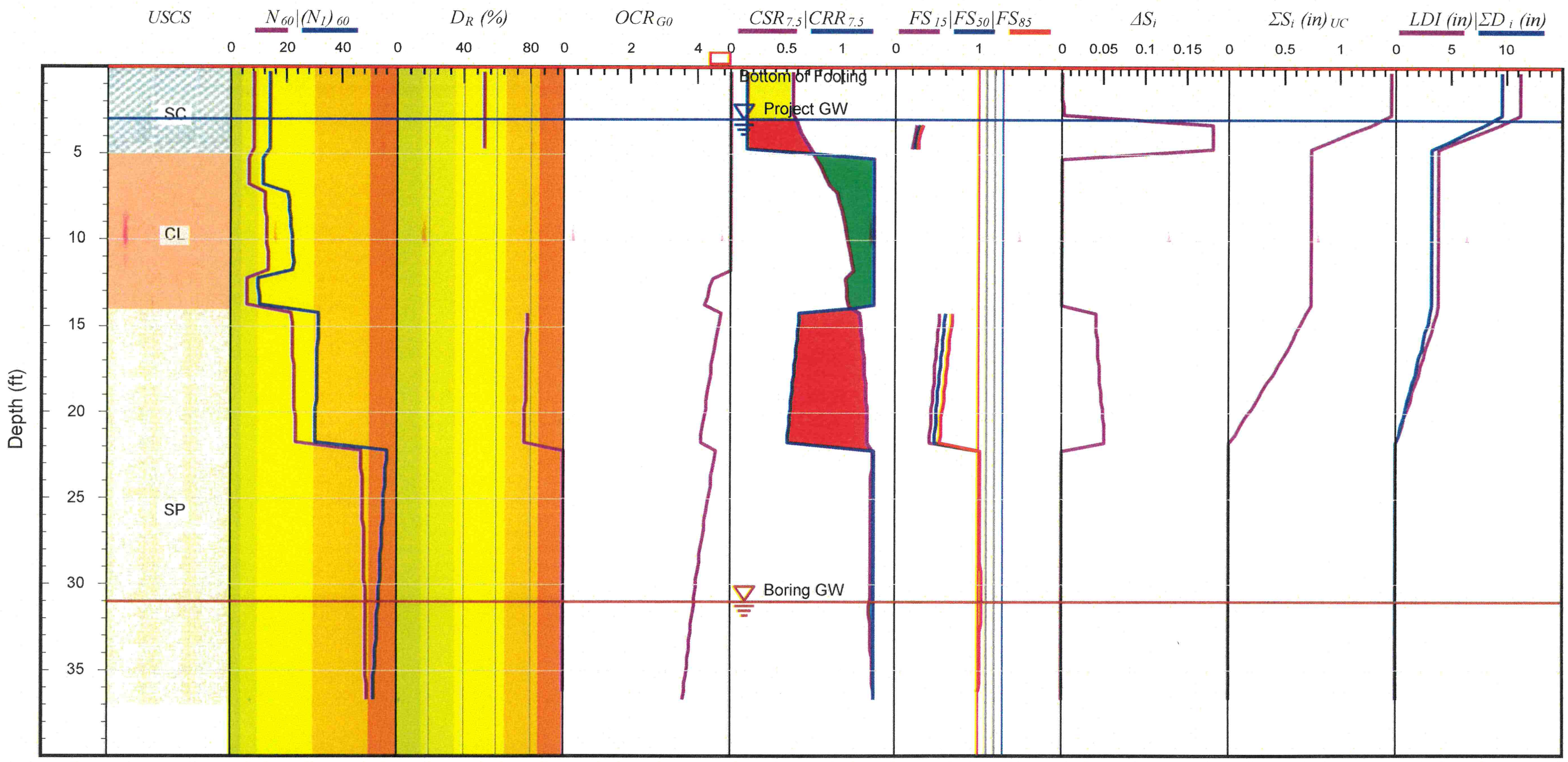
BORING LOG NO. 1

Date Drilled: 4 September 2001 *By:* R.W.

Figure No. 2

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
25–27	Gray silty sand, dense and very moist		SP					
27	gray sand, very moist and dense							
28		1-7		45	111.8	17.9		
30–31	(water at 31.0 feet)							
32	Gray sandstone, weathered, medium and very moist							
33		1-8		>50	115.1	15.7		
34		1-9		>50		16.9		
37		1-10		>50		14.3		
37	Boring terminated at 37.0 feet. Groundwater encountered at 28.0 feet							
38–50								



SC	SP
CL	

Earthquake & Groundwater Information:
Magnitude = 7.86
Max. Acceleration = 0.912 g
Project GW = 3 ft
Maximum Settlement = 1.46 in
Settl. at Bottom of Footing = 1.46 in

Liquefaction: Boulanger & Idriss (2010-16)
Settl.: [dry] UCLA (2008-14); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand; Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

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Liquefaction Potential - SPT Data

Project:	Sunshine Valley Road				
Location:	Moss Beach				
Job Number:	5264	Boring No.:	1	Enclosure:	0

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Prepared at 5/5/2019 4:06:11 PM

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