

## **APPENDIX I**

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### **GEOTECHNICAL INVESTIGATION AND PEER REVIEW**

Prepared for **Summerhill Apartment Communities**

**FINAL GEOTECHNICAL INVESTIGATION  
PROPOSED RESIDENTIAL DEVELOPMENT  
1008, 1016, AND 1028 CAROLAN AVENUE /  
935 ROLLINS ROAD  
BURLINGAME, CALIFORNIA**

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February 28, 2014  
Project No. 13-560

February 28, 2014  
Project No. 13-560

Ms. Elaine Breeze  
Vice President of Development  
SummerHill Apartment Communities  
777 S. California Avenue  
Palo Alto, California 94304

Subject: Revised Final Geotechnical Investigation  
Proposed Residential Development  
1008, 1016, and 1028 Carolan Avenue/935 Rollins Road  
Burlingame, California

Dear Ms. Breeze,

We are pleased to present the results of our revised final geotechnical investigation for the proposed residential development to be constructed at the properties located at 1008, 1016, and 1028 Carolan Avenue and 935 Rollins Road in Burlingame, California. Our services were provided in accordance with our proposal dated November 1, 2013.

The site is located between Carolan Avenue and Rollins Road, west of their intersections with Toyon Drive. The site consists of four contiguous parcels occupying a total area of about 5.4 acres. The properties are currently occupied by commercial buildings with adjacent asphalt-paved driveways and parking lots. The existing site grades vary from about Elevation<sup>1</sup> 8-1/2 feet at the northeast edge of the site (along Rollins Road), to about Elevation 9-1/2 feet near the southwest edge of the site (along Carolan Avenue), to about Elevation 11-1/2 feet near the center of the site.

Based on conceptual drawings prepared by Seidel Architects, dated January 20, 2014, we understand the proposed site development includes constructing an apartment building that will occupy the northwestern 75 percent of the site. The building will consist of four stories of wood-framed residential units above a two-level concrete podium. The concrete podium will house a parking garage, the lower level of which will have a

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<sup>1</sup> Existing site grades are based on topographic information shown on the drawing titled "Carolan Burlingame, Existing Conditions," Sheet C1.0, prepared by BKF Engineers, dated October 8, 2013 (Elevation Datum: NGVD29)

Ms. Elaine Breeze  
Summerhill Apartment Communities  
February 28, 2014  
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finished slab at Elevation 4.4 feet. An approximately 40-foot-wide strip along the southwest and southeast edges of the apartment building (along Carolan Avenue and the entry driveway) will be constructed at-grade and immediately adjacent to the below-grade garage level. Structural loads for the podium structure are not currently available. The proposed development also includes constructing 22 townhomes along the southeastern 25 percent of the site. The townhomes will be two-story, wood-framed structures constructed at-grade. A new entry driveway, arrival court, and EVA road/pedestrian paseo will run from Carolan Avenue to Rollins Road between the podium structure and townhomes.

Based on the results of our geotechnical investigation and engineering analyses, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development include: 1) the potential for differential settlement under static foundation loads due to compression of the surficial fill and thin weak layers within the underlying native deposits, 2) the potential for up to 2 inches of liquefaction-induced settlement in the northeast portion of the site, and 3) shallow groundwater relative to the proposed below-grade parking level. We conclude the below-grade portion of the proposed apartment building can be supported on a stiffened mat foundation, however, the upper 18 inches of native soil beneath the mat should be lime- and/or cement-treated to stabilize the underlying saturated sensitive soil to provide a firm working surface. The at-grade portions of the apartment building may be supported on spread footings founded on compacted aggregate piers (CAPs) that extend below the zone of weak marsh deposits. We conclude the townhome structures should be supported on post-tensioned (PT) slab foundations, provided they can be designed to accommodate the estimated differential settlements presented in our report. In our opinion, a cantilevered soldier pile and lagging shoring system would be the most suitable and economical temporary shoring system for the project site.

Our report contains specific recommendations regarding earthwork and grading, foundation design, and other geotechnical issues. The recommendations contained in our report are based on limited subsurface exploration and laboratory testing programs. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. Therefore, we should be engaged to observe foundation installation, ground improvement, and fill placement, during which time we may make changes in our recommendations, if deemed necessary.

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We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,  
ROCKRIDGE GEOTECHNICAL, INC.


Logan D. Medeiros, P.E., G.E.  
Senior Project Engineer


Craig S. Shields, P.E., G.E.  
Principal Geotechnical Engineer

Enclosure

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## **APPENDIX C**

Summary of Liquefaction Analyses
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## **APPENDIX D**

Planting Suitability Test Results by Soil & Plant Laboratory, Inc.
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**REVISED FINAL GEOTECHNICAL INVESTIGATION  
PROPOSED RESIDENTIAL DEVELOPMENT  
1008, 1016, AND 1028 CAROLAN AVENUE / 935 ROLLINS ROAD  
Burlingame, California**

## **1.0 INTRODUCTION**

This report presents the results of the final geotechnical investigation performed by Rockridge Geotechnical for the proposed residential development to be constructed at the properties located at 1008, 1016, and 1028 Carolan Avenue and 935 Rollins Road in Burlingame, California. The site is located between Carolan Avenue and Rollins Road, west of their intersections with Toyon Drive, as shown on the Site Location Map, Figure 1.

The site consists of four contiguous parcels occupying a total area of about 5.4 acres. The properties are currently occupied by commercial buildings with adjacent asphalt-paved driveways and parking lots, as shown on the Site Plan, Figure 2. The existing site grades vary from about Elevation<sup>1</sup> 8-1/2 feet at the northeast edge of the site (along Rollins Road), to about Elevation 9-1/2 feet near the southwest edge of the site (along Carolan Avenue), to about Elevation 11-1/2 feet near the center of the site.

Based on conceptual drawings prepared by Seidel Architects, dated January 20, 2014, we understand the proposed site development includes constructing an apartment building that will occupy the northwestern 75 percent of the site. The building will consist of four stories of wood-framed residential units above a two-level concrete podium. The concrete podium will house a parking garage, the lower level of which will have a finished slab at Elevation 4.4 feet, which corresponds to a depth of about 4 to 7 feet below existing site grades. An approximately 40-foot-wide strip along the southwest and southeast edges of the apartment building (along Carolan Avenue and the entry driveway) will be constructed at-grade and immediately adjacent to the

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below-grade garage level. Structural loads for the podium structure are not currently available. Based on our experience on similar projects, we anticipate interior column loads will be on the order of 475 kips, walls loads will be on the order of 10 kips per foot, and the average building weight will be on the order of 650 pounds per square foot (psf). The proposed development also includes constructing 22 townhomes along the southeastern 25 percent of the site. The townhomes will be two-story, wood-framed structures constructed at-grade. A new entry driveway, arrival court, and EVA road/pedestrian paseo will run from Carolan Avenue to Rollins Road between the podium structure and townhomes. We understand the EVA road/pedestrian paseo will likely consist of permeable grass pavers (Grasspave2) and be design to accommodate periodic fire truck access.

## **2.0 SCOPE OF SERVICES**

We previously performed a preliminary geotechnical investigation at the site, the results of which were presented in our report dated August 20, 2013. Our preliminary investigation consisted of evaluating subsurface conditions at the site by advancing four cone penetration tests (CPTs). Our subsequent final investigation was performed in accordance with our Professional Services Agreement with SummerHill Apartment Communities, dated July 2, 2013, Contract Amendment ASA-1, dated November 4, 2013, and Contract Amendment ASA-2, dated November 25, 2013. Our scope of work consisted of further evaluating subsurface conditions at the site by advancing four additional CPTs, drilling four exploratory borings, performing laboratory testing on select soil samples, and performing engineering analyses to develop final conclusions and recommendations regarding:

- the most appropriate foundation type(s) for the proposed structures
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of static and seismically-induced foundation settlement

- surcharging to reduce potential consolidation settlements due to weak deposits beneath the townhomes
- subgrade preparation for pavements and exterior concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- flexible and rigid pavement design
- soil corrosivity
- 2013 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations
- suitability of near-surface soil for future landscaping (evaluated by Soil & Plant Laboratory, Inc.).

### **3.0 FIELD INVESTIGATION**

Subsurface conditions at the site were investigated by drilling four borings, advancing eight CPTs, and performing laboratory testing on select soil samples. Prior to our field investigations, we obtained drilling permits from the San Mateo County Environmental Health Services Division (SMCEHSD). We also contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained Precision Locating, LLC, a private utility locator, to check that the boring and CPT locations were clear of existing utilities. Upon completion, the test borings and CPTs were backfilled with cement grout. Details of the field investigation and laboratory testing are described below.

### **3.1 Cone Penetration Tests**

CPT-1 through CPT-4 were advanced on July 3, 2013 by John Sarmiento & Associates (JSA) of Orinda, California. JSA advanced CPT-5 through CPT-8 on November 19, 2013. CPT-1 and CPT-2 were each advanced to a depth of about 80 feet below the existing ground surface (bgs). CPT-5 through CPT-7 were each advanced to a depth of about 50 feet bgs. CPT-3, CPT-4, and CPT-8 were advanced to practical refusal in very dense / stiff soil at depths of about 24, 27, and 42 feet, respectively. The approximate locations of the CPTs are shown on the Site Plan, Figure 2.

The CPTs were performed by hydraulically pushing a 1.4-inch-diameter cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the soil behavior types, approximate strength characteristics, and the liquefaction potential of the soil encountered. The CPT logs showing tip resistance, friction ratio, and pore pressure, as well as correlated soil behavior type, are presented in Appendix A on Figures A-6 through A-13.

### **3.2 Test Borings**

Four test borings, designated B-1 through B-4, were drilled by Pitcher Drilling Company of East Palo Alto, California at the approximate locations shown on Figure 2. The borings were drilled on November 19, 20 and 23, 2013 using a truck-mounted drill rig equipped with rotary-wash drilling equipment. Borings B-1 and B-2 were drilled to a depth of about 51-1/2 feet, boring B-3 was drilled to a depth of about 48 feet, and boring B-4 was drilled to practical refusal in very dense, cemented sand at a depth of about 25-1/2 feet. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory

testing. The logs of the borings are presented on Figures A-1a through A-4 in Appendix A. The soil encountered in the borings was classified in accordance with the classification charts shown on Figure A-5.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter brass tubes
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.
- Shelby Tubes (ST) - thin-walled steel tubes with a 3.0-inch outside diameter and 2.875-inch inside diameter.

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of granular soils. The Shelby tubes were used to obtain relatively undisturbed samples of very soft to medium stiff fine-grained soils. The S&H and SPT samplers were driven with a 140-pound, automatic hammer falling 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.8 and 1.44, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was used without liners. The blow counts used for this conversion were: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The converted SPT N-values are presented on the boring logs. The Shelby tubes were slowly advanced using the weight of the drill rods and hydraulic pressure, as needed.

Upon completion of drilling, the soil cuttings and drilling fluid from the borings were placed in 55-gallon drums and temporarily stored on site. Laboratory analytical testing was performed on representative samples of the drum contents. The test results indicated the material was non-hazardous and the drums were subsequently disposed of at a landfill.

### **3.3 Laboratory Testing**

We re-examined each soil sample obtained from our borings to confirm the field classifications and select representative samples for geotechnical laboratory testing. Soil samples were tested to measure moisture content, dry density, Atterberg limits (plasticity index), percent passing the No. 200 sieve, consolidation characteristics, undrained shear strength, resistance value (R-value) and corrosivity. The Atterberg limits test is an indirect measurement of the expansion potential of soil. The results of the geotechnical laboratory tests are presented on the boring logs and in Appendix B.

At the request of the project landscape architect, we also obtained three samples of near-surface soil for planting suitability testing by Soil & Plant Laboratory (SPL), Inc. in San Jose. The samples were collected between depths of about 1/2 and 2 feet bgs at borings B-1, B-2, and B-4. SPL's analytical results and recommendations regarding planting suitability at these three locations are presented in Appendix D.

## **4.0 REGIONAL GEOLOGY AND SUBSURFACE CONDITIONS**

### **4.1 Regional Geology**

The site is located in the Coast Ranges geomorphic province of California which is characterized by a series of northwest-trending folded and faulted mountain chains and intervening valleys. The oldest rocks exposed near the site belong to the Franciscan Complex which underlies the ridgelines to the west. The Franciscan Complex is composed of altered sea floor sediments deposited during Cretaceous to Jurassic Periods of geologic time (roughly 65 to 205 million years before present).

The subject site is located on reclaimed lands artificially filled along the western margin of the San Francisco Bay. Sedimentary deposits of the Merced and Colma formations overlap the Franciscan Complex rocks along the base of the hills. The Merced formation consists of Plio-Pleistocene aged (11,000 to 5 million years before present) near-shore sediments deposited along the margins of the bay. The late Pleistocene-aged (125,000 to 11,000 years before present) Colma formation overlaps the Merced formation in many locations west of the site. The Merced and Colma formations, as well as younger Holocene-aged (less than 11,000 years before present) basin sediments have been uplifted and dissected by creeks and streams. The creeks and streams reworked sediments and deposited them further downslope as alluvial fans. Alluvial fans generated from Sanchez Creek were deposited near the site and overlap the young bay sediments.

As shown on the Regional Geologic Map (Brabb, 1998), Figure 4, the subject site is located at the junction between several of the units discussed above. However, these materials are covered by artificial fill that was previously placed around the bay margin to reclaim land. Artificial fill underlies the site and overlaps the younger basin deposits to the east. To the south and west, the fill covers sediments from older fan deposits and Colma formation. Available geologic literature describes the various geologic units in the site vicinity as follows:

***Artificial Fill (af)*** – Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations. Fills placed prior to 1965 were generally made by loosely dumping and are therefore, not well-compacted.

***Holocene Basin Deposits (Qhb)*** – Very fine silty clay to clay deposits occupying flat-floored basins at the distal edge of alluvial fans adjacent to the bay mud. Also contains unconsolidated, locally organic, plastic silt and silty clay deposited in very flat valley floors.

***Pleistocene Alluvial Fan and Fluvial Deposits (Qpaf)*** – Dense gravelly and clayey sand or clayey gravel that fines upward to sandy clay. These deposits display variable sorting and are located along most stream channels in San Mateo County. Qpaf deposits can generally be related to modern stream courses. They are overlain by Holocene deposits on lower parts of the alluvial plain, and incised by channels that are partly filled by Holocene alluvium on higher parts of the alluvial plane.

***Pleistocene Colma Formation (Qc)*** – Friable to loose, fine- to medium-grained arkosic sand with subordinate amounts of gravel, silt and clay.

## **4.2 Subsurface Conditions**

The results of our borings and CPTs indicate the site is generally blanketed by approximately 2 to 5 feet of heterogeneous fill and native material that consists of medium stiff to very stiff fine-grained soil with varying sand and gravel content interbedded with loose to medium dense sands and gravels with varying amounts of fines.

Beneath the southwest edge of the site, CPT-3, CPT-4, and boring B-4 encountered weak, compressible organic clay deposits beneath the fill between depths of about 4 and 8 feet bgs. The weak deposits are near the groundwater table and are likely former marsh deposits. Based on the results of our CPTs and laboratory consolidation tests, we conclude this material is nearly normally consolidated. Normally consolidated fine-grained deposits have an in-situ stress state close to that of their maximum past pressure and are highly compressible under new loads, compared to over-consolidated deposits.

In general, the fill and/or weak deposits are underlain by stiff to very stiff over-consolidated clay with varying sand and gravel content and medium dense to very dense clayey sand with gravel. However, some lightly to moderately overconsolidated, 2- to 3-foot-thick clay layers were encountered between about 20 and 30 feet bgs beneath the northeast portion of the site. In CPT-1, CPT-2, CPT-5, and boring B-1, we encountered a 6- to 12-foot thick layer of very dense clayey sand with gravel material between depths of about 40 and 50 feet bgs, which was underlain by stiff to very stiff clay to the maximum depth explored of about 80 feet bgs.

At the southwest edge of the site, CPT-3, CPT-4, and boring B-4 encountered practical refusal in very dense cemented clayey sand at depths of about 22 to 24 feet bgs, which based on the mapped regional geology, likely corresponds to older Pleistocene alluvial fan deposits or Colma formation. The very dense material was also encountered at depths of about 40 feet bgs in CPT-8 and about 45 feet bgs in boring B-3.



Groundwater was measured in the CPT holes immediately after completion of the soundings and in the borings during drilling. Table 1 presents a summary of the groundwater depths and the corresponding groundwater elevations measured during our field investigations. The majority of the readings may not reflect stabilized groundwater levels, with the exception of boring B-2, which was allowed to stabilize for about 16 hours, during which time the water level rose from 10 feet bgs to 6.5 feet bgs. It is also worth noting that the 2013-2014 winter has been particularly dry thus far.

**TABLE 1**  
**Summary of Groundwater Level Measurements**

<b>Boring / CPT</b>	<b>Date</b>	<b>Measured GW Depth (feet)</b>	<b>Measured GW Elevation (feet)</b>	<b>Notes</b>
B-1	11/20/13	7.0	1.3	Measured during drilling; not necessarily stabilized
B-2	11/19-11/20/13	6.5	4.0	Stabilized for 16 hours; initially measured at 10 feet bgs.
B-3	11/23/13	12.0	-2.5	Measured during drilling; not necessarily stabilized
B-4	11/19/13	8.0	1.1	Measured during drilling; not necessarily stabilized
CPT-1	7/3/13	9.4	-0.2	Measured down hole following sounding; not necessarily stabilized
CPT-2	7/3/13	9.6	0.4	Measured down hole following sounding; not necessarily stabilized
CPT-3	7/3/13	6.0	3.0	Measured down hole following sounding; not necessarily stabilized
CPT-4	7/3/13	5.6	3.3	Measured down hole following sounding; not necessarily stabilized
CPT-5	11/19/13	10.0	0.1	Measured down hole following sounding; not necessarily stabilized
CPT-6	11/19/13	9.7	1.3	Measured down hole following sounding; not necessarily stabilized
CPT-7	11/19/13	12.8	-1.8	Measured down hole following sounding; not necessarily stabilized
CPT-8	11/19/13	15.3	-4.9	Measured down hole following sounding; not necessarily stabilized

The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. To estimate the highest potential groundwater level at the site, we reviewed information on the State of California Water Resources Control Board GeoTracker website (<http://geotracker.swrcb.ca.gov>). The two closest sites with historic groundwater data on the GeoTracker website are at 1095 Carolan Drive, which is approximately 1,200 feet west of the site, and at 1147 Rollins Road, which is approximately 1,000 feet northwest of the site. The data from these two sites indicates the groundwater table slopes down gently to the northeast. The largest groundwater level fluctuations measured at the 1147 Rollins Road site between September 1991 and February 2006 were on the order of about six feet. The largest groundwater level fluctuations measured at the 1095 Carolan Avenue site between December 1998 and February 2010 were on the order of about four feet.

Based on the above data, we estimate the highest groundwater levels at the project site would be about 3 feet higher than the readings obtained in our July 3, 2013 CPTs. We conclude a design groundwater of Elevation 3 feet should be assumed beneath the northeast end of the development and a design groundwater of Elevation 6 feet should be used beneath the southwest end of the development. The design groundwater surface may be estimated by interpolating between these two points.

## 5.0 SEISMIC CONSIDERATIONS

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture, liquefaction<sup>2</sup>, lateral spreading<sup>3</sup> and cyclic densification.<sup>4</sup> The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

### 5.1 Regional Seismicity and Faulting

The site is located in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent shearing along the San Andreas fault system. Movements along this plate boundary in the Northern California region occur along right-lateral strike-slip faults of the San Andreas Fault system.

The major active faults in the area are the San Andreas, San Gregorio, and Hayward Faults. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and mean characteristic Moment magnitude<sup>5</sup> [Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 2.

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<sup>2</sup> Liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences temporary reduction in strength during cyclic loading such as that produced by earthquakes.

<sup>3</sup> Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

<sup>4</sup> Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

<sup>5</sup> Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

**TABLE 2**  
**Regional Faults and Seismicity**

<b>Fault Segment</b>	<b>Approximate Distance from Site (km)</b>	<b>Direction from Site</b>	<b>Mean Characteristic Moment Magnitude</b>
N. San Andreas - Peninsula	3.7	West	7.23
N. San Andreas (1906 event)	3.7	West	8.05
San Gregorio Connected	14	West	7.50
Monte Vista-Shannon	19	Southeast	6.50
Total Hayward	26	Northeast	7.00
Total Hayward-Rodgers Creek	26	Northeast	7.33
N. San Andreas - North Coast	29	Northwest	7.51
Total Calaveras	39	East	7.03
Mount Diablo Thrust	43	Northeast	6.70
Green Valley Connected	48	Northeast	6.80

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude,  $M_w$ , for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an  $M_w$  of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an  $M_w$  of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect

the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an  $M_w$  of 6.9. This earthquake occurred in the Santa Cruz Mountains about 74 kilometers southwest of the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated  $M_w$  for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an  $M_w$  of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ( $M_w = 6.2$ ).

The U.S. Geological Survey's 2007 Working Group on California Earthquake Probabilities has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region during the next 30 years is 63 percent. The highest probabilities are assigned to the Hayward/Rodgers Creek Fault and the northern segment of the San Andreas Fault. These probabilities are 31 and 21 percent, respectively (USGS, 2008).

## **5.2 Geologic Hazards**

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, lateral spreading, and cyclic densification. We used the results of the CPTs to evaluate the potential of these phenomena occurring at the project site. The results of our analyses and evaluation are presented in the following sections.

### **5.2.1 Ground Shaking**

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) subsurface conditions.

The site is less than four kilometers from the San Andreas Fault. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

### **5.2.2 Liquefaction and Associated Hazards**

Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction and lateral spreading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our CPTs. Our liquefaction analyses were performed using the methodology proposed by P.K. Robertson (2009). We also used the relationship proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; a relationship that is an extension of the work by Ishihara and Yoshimine (1992).

Our analyses were performed using the approximate in-situ groundwater depths measured in our CPTs and “during earthquake” groundwater depths that correspond to the approximate design groundwater surface discussed in Section 4.2. In accordance with the 2013 California Building Code (CBC), we used a peak ground acceleration of 0.82 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean ( $MCE_G$ ) peak ground acceleration adjusted for site effects ( $PGA_M$ ). We also used a moment magnitude 8.05 earthquake, which is consistent with the mean characteristic moment magnitude for the San Andreas Fault, as presented in Table 2.

Our liquefaction analyses indicate there are several thin layers of potentially liquefiable soil below depths of about 15 feet in the northeast half of the site (CPT-1, CPT-2, CPT-5, CPT-6, and CPT-7). The potentially liquefiable layers are typically one to four feet thick and are

characterized with fine-grained soil behavior types. We estimate liquefaction-induced ground surface settlement (referred to as post-liquefaction reconsolidation) after a major event on a nearby fault will be about 1-1/2 to 2 inches at the locations of CPT-1, CPT-2, CPT-5, and CPT-6. At the location of CPT-7, we estimate liquefaction-induced settlement will be about 1 to 1-1/2 inches. At the location of CPT-8, we estimate liquefaction-induced settlement will be less than one inch. At the southwest edge of the site, at the locations of CPT-3 and CPT-4, we estimate liquefaction-induced ground surface settlement will be less than 1/2 inch.

Our analyses identified potentially liquefiable layers between depths of 5 and 8 feet bgs in CPT-3 and depths of 3 and 8 feet in CPT-4. These layers correspond to those characterized with a soil behavior type of “organic soil” (potential marsh deposit) and have a relatively high friction ratio. In addition, Atterberg limits testing on the weak organic clay in boring B-4, which was located in close proximity to CPT-4, indicates this material has a plasticity index (PI) of about 19, which is considered too plastic to “liquefy” in the traditional sense [Idriss & Boulanger, 2008]. However, we conclude these materials may be subject to cyclic softening, which may reduce the bearing capacity beneath the at-grade structures supported on shallow foundations. Within the footprint of the proposed apartment building (where it includes a below-grade level), we anticipate the sensitive fine-grained materials in the upper 8 feet will be removed and/or stabilized during excavation for the below-grade foundation.

Considering the relatively flat site grades and the absence of a free face in the site topography, as well as the depth and relative thickness of the potentially liquefiable layers, we conclude the risk of lateral spreading is low.

### **5.2.3 Cyclic Densification**

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The results of our borings and CPTs indicate the soil above the groundwater at the site generally consists of cohesive soil, interbedded with relatively

dense granular soil, which are not susceptible to cyclic densification. One exception is at boring B-2, in which we encountered loose to medium dense silty gravel with sand fill in the upper four feet. A laboratory particle size analysis indicates this material has about 24 percent fines. The existing ground surface elevation at boring B-2 is close to the high point of the site (Elevation 10.5 feet) and we anticipate about 1 to 2 feet of this material will be removed during site grading. In addition, as presented in Section 7.0 of this report, we recommend scarifying and re-compacting the upper 18 inches of building pad and foundation subgrade within the townhome portion of the development. Due to the relatively high fines content and the fact that the majority of the material will either be removed or re-worked, we conclude the potential for ground surface settlement resulting from cyclic densification is low.

#### **5.2.4 Fault Rupture**

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.



## **6.0 DISCUSSIONS AND CONCLUSIONS**

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development include: 1) the potential for differential settlement under static foundation loads due to compression of the surficial fill and thin weak layers within the underlying native deposits, 2) the potential for up to 2 inches of liquefaction-induced settlement in the northeast portion of the site, and 3) shallow groundwater relative to the proposed below-grade parking level. These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

### **6.1 Foundations and Settlement**

#### **6.1.1 Podium Building**

We understand the proposed apartment building will include a concrete podium, the lower level of which will have a top-of-slab at about Elevation 4.4 feet, which corresponds to a depth of about 4 to 7 feet below existing site grades. In our engineering analyses we assumed a foundation subgrade of about Elevation 2.4 feet and an average building weight of about 650 psf. The native soils encountered in our borings and CPTs below the proposed garage subgrade elevation are predominately stiff /dense and overconsolidated. However, the weak marsh deposit encountered in CPT-3, CPT-4, and B-4 extends to about Elevation 1 to 2 feet. This material will not provide adequate support for shallow foundations under static or seismic conditions and will be very sensitive to disturbance during construction. Therefore, the foundation should either be bottomed below the zone of weak clay and/or marsh deposits or the weak material should be removed and replaced with crushed rock up to the design foundation subgrade elevation. Alternatively, the upper 18 inches of mat subgrade can be stabilized in place by mixing lime or cement into the soil. The in-place treatment alternative will perhaps be more cost-effective because it involves less soil being off-hauled and less rock being imported.

Beneath the northeast portion of the site, several 2- to 3-foot-thick layers of moderately compressible clay and organic materials were encountered between about 20 and 30 feet bgs that are normally consolidated to lightly overconsolidated. Stresses associated with new foundation loads may exceed the maximum past pressure (or pre-consolidation pressure) of these deposits, resulting in a new cycle of virgin consolidation. Virgin consolidation results in much higher settlements than re-compression and takes much longer to occur. If the proposed building is supported on a shallow foundation system, settlement will occur due to compression of the underlying clay under static foundation loads. If the proposed apartment building is supported on spread footings, we estimate differential settlement may be more than 2-1/2 inches between columns, including the differential settlement associated with liquefaction, as discussed in Section 5.2. Differential settlements of this magnitude are generally unacceptable for buildings of this type. Differential settlements can be reduced by supporting the below-grade portions of the building on a stiffened mat foundation. Considering the assumed foundation subgrade of Elevation 2.4 feet is lower than the recommended design groundwater level, the foundation system and garage slab should be designed to resist hydrostatic uplift forces and be underlain by a waterproofing. A mat foundation system generally simplifies construction dewatering (discussed below) and the detailing of waterproofing. In addition, a stiffened mat foundation will provide better building performance, by reducing the potential for damaging differential settlements.

Our settlement analyses indicate total settlement of the mat foundation under static load conditions, assuming an average contact pressure of 650 psf, will be on the order of about 2-1/4 inches in the northeast portion of the building and about one to 1-1/2 inches in the southwest portion of the building. We anticipate most of the settlement will occur during construction. The amount of differential settlement between columns will be a function of the mat stiffness and hence its ability to spread the loads between columns, however, we expect the mat can be designed to limit differential settlements to 3/4 inch in 30 feet.

Where the proposed apartment building will be at-grade and immediately adjacent to the below-grade level at the southwest and southeast edges of the building (along Carolan Avenue and the

entry driveway), foundations will need to gain support below the depth of the adjacent garage level and below the weak marsh deposits discussed above. In our opinion it is not feasible to remove and replaced the weak deposits with engineered fill due to the depth of the deposits and the presence of relatively shallow groundwater. In addition, it is likely not economical to support this portion of the building on a deep foundation system. A more economical alternative is to support the building on a shallow foundation system constructed over compacted aggregate piers (CAPs) that extend through the zone of weak deposits.

### **6.1.2 Compacted Aggregate Piers (CAPs)**

Compacted aggregate piers are typically constructed by drilling a 30-inch-diameter shaft and replacing the excavated soil with compacted aggregate. The aggregate consists of clean, open-graded crushed rock below the water table and Class 2 aggregate base above the water table. The aggregate is compacted in approximately 12-inch-thick lifts using a modified hydraulic hammer mounted on an excavator. CAPs develop vertical support through a combination of frictional resistance along the shaft of the pier and improvement of the surrounding soil matrix, allowing use of significantly larger bearing capacities than feasible in unimproved soil. CAPs can also be designed to resist transient uplift loads by installing steel rods in the center of the pier; the rods are attached to a flat steel plate at the base of the CAP. In the context of this project, the primary purpose of CAPs is to extend foundation loads below the weak deposits in the upper 8 to 10 feet, thus reducing the potential for damaging differential foundation settlement between the at-grade and below-grade portions of the building.

CAPs are typically constructed through a design-build contract with a licensed foundation installer. In our experience, a CAP-supported shallow foundation in these conditions can be designed to limit differential settlement to 3/4 inch over a horizontal distance of 30 feet.

### 6.1.3 Townhomes

We understand the townhomes will be located along the southeast edge of the site, adjacent to the neighboring single-family residential lots, and will be constructed at grade. The results of borings and CPTs performed at the site indicate the site is blanketed by 2 to 5 feet of heterogeneous undocumented fill. The fill varies in composition and quality across the site; in some areas the fill was found to be poorly to moderately compacted. If the at-grade townhomes are constructed over the fill in its current condition, the buildings may experience erratic performance due to the inherent variability of these materials. To create a more homogeneous subgrade that will provide uniform support for the proposed buildings, we conclude the upper 18 inches of the undocumented fill (below foundation subgrade) should be scarified, moisture-conditioned, blended, and recompacted.

Furthermore, beneath the southwest edge of the site, CPT-3, CPT-4, and boring B-4 encountered weak, compressible organic clay deposits beneath the fill between depths of about 4 and 8 feet bgs. The weak clay layer is generally about 2 to 3 feet thick, where present, and is nearly normally consolidated, indicating it is highly compressible under new foundation and/or fill loads. This layer may be improved to mitigate excessive long-term consolidation settlement beneath the townhome buildings by surcharging with a temporary soil fill during construction. The purpose of surcharging is to increase the maximum past pressure of the clay so that when the temporary surcharge fill is removed and the new foundation loads are applied, the material is still over-consolidated. Based on the results of our borings and CPTs, we conclude the approximate area requiring surcharging is limited to the townhome area from Carolan Avenue to a point approximately 250 feet northeast.

We conclude the townhome structures should be supported on post-tensioned (PT) slab foundations, provided they can be designed to accommodate the following estimated differential settlements. We estimate PT-slabs will experience less than one inch of total settlement under

static load conditions, most of which will occur during construction. We estimate differential settlement of about 1/2 inch over a horizontal distance of 30 feet may occur under static load conditions. In addition, as presented in Section 5.2, we estimate liquefaction-induced settlement following a major earthquake will range from about 1/2 inch on the southwest end of the site to 2 inches on the northeast edge of the site. We anticipate differential settlement due to liquefaction beneath the townhome structures will be less than 1/2 inch over a horizontal distance of 30 feet.

## **6.2 Groundwater**

As discussed in Section 4.2, we recommend using a design groundwater surface that slopes from Elevation 3 feet at the northeast end of the development to Elevation 6 feet at the southwest end of the development. As discussed in Section 1.0 we anticipate the lower level of the concrete podium will have a finished slab at Elevation 4.4 feet. Therefore, the mat foundation will need to be designed to resist hydrostatic uplift pressures and be underlain by a waterproofing.

Assuming the proposed apartment building will be supported on a 2-foot-thick mat foundation, the foundation subgrade will likely be about Elevation 2.4 feet over much of the excavation. In addition, the excavation will likely extend deeper beneath elevator pits and sumps and, as discussed in Section 6.1, overexcavation may be required in isolated areas to remove any remaining weak, compressible fill and/or marsh deposits that may be present. Therefore, depending on the time of year that excavation is performed, the foundation subgrade may be just above to more than 6 feet below the groundwater. Excavation dewatering will be necessary to construct the below-grade portion of the building. Due to the low permeability of much of the soil and the variable and discontinuous nature of the granular layers present beneath the site, an active dewatering system, such as a series of dewatering wells installed outside the perimeter of the excavation, may have limited effectiveness in drawing down the water level in the center of the excavation. Therefore, the dewatering effort may also require internal passive systems, such as trench drains and sump pumps. A combination of active and passive approaches will likely be required to adequately manage water in the excavation during construction. The construction dewatering system must be capable of maintaining the groundwater level below the foundation

subgrade until sufficient building weight is available to resist the hydrostatic uplift pressure, at which time the groundwater may be allowed to rise to its normal elevation.

The design and proper implementation of the excavation dewatering system should be the responsibility of the contractor. The system should be capable of drawing the water level down about three feet below the bottom of excavation during construction.

### **6.3 Excavation Support**

Construction of a partially below-grade parking level will likely require an excavation extending roughly 5 to 10 feet below the existing ground surface. Portions of the excavation may have sufficient setback from the property lines to be slope cut. Due to the presence of shallow groundwater and isolated granular soil layers, the material exposed in the slope cuts will most likely be considered Cal-OSHA Type C material, which requires limiting temporary slope cuts to a maximum inclination of 1.5:1 (horizontal:vertical). Considering there may be insufficient space to slope the sides of the excavation in some areas, shoring may be required. There are several key considerations in selecting a suitable shoring system. Those we consider of primary concern are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures
- proper construction of the shoring system to reduce potential for ground movement
- the presence of shallow groundwater
- cost.

Several methods of shoring are available; however, in our experience conventional soldier pile and lagging shoring is most suitable and economical in these soil conditions. A soldier pile and lagging system usually consists of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds. For buildings that include one below-grade level, a soldier pile and lagging system can provide shoring without tiebacks, and therefore will not encroach beyond the

property lines. If the excavation extends deeper than about 12 feet, a soldier pile and lagging system may require tiebacks, which may extend beneath the neighboring properties.

## **7.0 RECOMMENDATIONS**

### **7.1 Site Preparation and Grading**

Site clearing should include removal of all existing pavements, former foundation elements, and underground utilities. Any vegetation and organic topsoil (if present) should be stripped in areas to receive improvements (i.e., building, pavement, or flatwork). Tree roots with a diameter greater than 1/2 inch within three feet of subgrade should be removed. Excessively dry soil at tree removal locations, as determined in the field by the geotechnical engineer, should also be excavated and replaced. Demolished asphalt concrete should be taken to an asphalt recycling facility. Aggregate base beneath existing pavements may be re-used as select fill if carefully segregated.

During excavation for the below-grade parking levels, we anticipate portions of the excavation will extend below groundwater. After the existing fill is removed, the native material exposed at the bottom of the excavation will be easily disturbed by construction equipment and foot traffic. The excavation subgrade will be sensitive to disturbance, especially under construction equipment wheel loads. We recommend only tracked equipment be used when the excavation approaches two feet of the mat subgrade. The final foot of excavation should be performed with a track-mounted excavator with a smooth bar welded across the teeth. Even with lightweight tracked equipment, the exposed subgrade will likely be sensitive, especially if the excavation is not adequately dewatered.

We recommend four feet of soil be stockpiled over the southwestern 250 feet of the townhome area in order to surcharge the underlying weak clay deposits. Soil from the excavation for the podium structure may be used for the surcharge. Provided the surcharge is left in place for at least 2 months, wick drains will not be required to expedite primary consolidation. After the surcharge soil is removed, the entire townhome area should be cut to approximate finished

subgrade elevation, scarified to a depth of 18 inches (in multiple lifts, if necessary, depending on equipment used), moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction.<sup>6</sup> Note that the soil may be above optimum moisture content and therefore, moisture-conditioning may require aerating to lower the moisture content.

All fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 3.

**TABLE 3**  
**Summary of Compaction Requirements**

<b>Location</b>	<b>Required Relative Compaction (percent)</b>	<b>Moisture Requirement</b>
Townhome and at-grade apartment building pad subgrade – low-plasticity	90+	Above optimum
Mat subgrade – lime- or cement-treated	90+	Above optimum
General fill – lime- or cement-treated clay	90+	Above optimum
General fill – expansive clay	88 – 93	3+% above optimum
Utility trench backfill – expansive clay	88 – 93	3+% above optimum
Utility trench backfill – low-plasticity	90+	Above optimum
Utility trench - clean sand or gravel	95+	Near optimum
Pavement subgrade – low-plasticity	95+	Above optimum
Pavement - aggregate base	95+	Near optimum
Exterior slabs – low-plasticity	90+	Above optimum
Exterior slabs – select fill	90+	Above optimum

<sup>6</sup> Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.



### **7.1.1 Soil Subgrade Stabilization**

In some areas, soft, wet soil may be exposed during grading, causing the subgrade to deflect and rut under the weight of grading equipment. If heavy wheeled equipment is used close to the water table, these materials may become disturbed and soften. In these areas, some form of subgrade stabilization may be required. Several options for stabilizing subgrade are presented below. Aeration will likely not be effective where the podium subgrade extends below or near the groundwater table. Overexcavation and replacement with engineered fill is generally not cost effective below the groundwater table over a large area. Therefore, we recommend chemical treatment for stabilizing the mat subgrade beneath the podium structure.

#### Aeration

Aeration consists of mixing and turning the soil to naturally lower the moisture content to an acceptable level. Aeration typically requires several days to a week of warm, dry weather to effectively dry the material. Material to be dried by aeration should be scarified to a depth of at least 12 inches; the scarified material should be turned at least twice a day to promote uniform drying. Once the moisture content of the aerated soil has been reduced to acceptable levels, the soil should be compacted in accordance with our previous recommendations. Aeration is typically the least costly subgrade stabilization alternative; however, it generally requires the most time to complete and may not be effective if the soft material extends to great depths.

#### Overexcavation

Another method of achieving suitable subgrade in areas where soft, wet soil is exposed is to overexcavate the soft subgrade soil and replace it with drier, granular material. If the soft material extends to great depths, the upper 18 to 24 inches of soft material may be overexcavated and a geotextile tensile fabric (Mirafi 500X or equivalent) placed beneath the granular backfill to help span over the weaker material. The fabric should be pulled tight and placed at the base of the overexcavation, extending at least two feet laterally beyond the limits of the overexcavation in all directions. The fabric should be overlapped by at least two feet at all seams. Granular

material such as Class 2 aggregate base should then be placed and compacted over the geotextile tensile fabric.

Where very soft subgrade conditions are encountered, a bi-directional geogrid, such as Tensar TriAx TX-140 or equivalent, may be required in lieu of tensile fabric. Where geogrids are used the depth of overexcavation will likely be on the order of 12 to 18 inches. The geogrids should be overlapped by at least two feet and tied with hog rings or nylon ties at a spacing not to exceed 10 feet. The geogrids should be covered with a well-graded granular fill such as Class 2 aggregate base; open-graded rock should not be used. All backfill placed over the geogrid should be compacted in accordance with our previous recommendations.

#### Chemical Treatment

Lime and/or cement have been successfully used to dry and stabilize fine-grained soils with varying degrees of success. Lime- and/or cement-treatment will generally decrease soil density, change its plasticity properties, and increase its strength. The degree to which lime will react with soil depends on such variables as type of soil, mineralogy, quantity of lime, and length of time the lime-soil mixture is cured. Cement is generally used when a significant amount of granular material or low-plasticity silt is present in the soil. The quantity of lime and/or cement added generally ranges from 3 to 7 percent by weight and should be determined by laboratory testing. The specialty contractor performing the chemical treatment should select the most appropriate additive and quantity for the soil conditions encountered. Beneath the mat foundation of the podium structure, we recommend using at least 5 percent lime and/or cement by weight to at least 18 inches below mat subgrade to stabilize the weak marsh deposits. An average dry density of 105 pounds per cubic foot (psf) should be assumed in determining the quantity of lime and/or cement.

If chemical treatment is used to stabilize soft subgrade, a treatment depth of about 18 inches below the final soil subgrade will likely be required. The soil being treated should be scarified and thoroughly broken up to full depth and width. The treated soil should not contain rocks or

soil clods larger than three inches in greatest dimension. Treated soil should be compacted to at least 90 percent RC, and at least 95 percent RC in the upper six inches of pavement subgrade.

### **7.1.2 Select Fill**

Select fill should consist of imported soil that is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction beneath concrete flatwork and sidewalks. Beneath vehicular pavements, the select fill should be compacted to at least 95 percent relative compaction. Samples of proposed select fill material should be submitted to the geotechnical engineer at least three business days prior to use at the site.

The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not provided, a minimum of two weeks will be required to perform any necessary analytical testing.

### **7.1.3 Exterior Flatwork Subgrade Preparation**

We recommend a minimum of four inches of imported (select) material be placed beneath proposed exterior concrete flatwork, including patio slabs and sidewalks; the select fill should extend at least one foot beyond the slab edges. Prior to placement of the select fill, the exposed subgrade should be scarified to a depth of 12 inches, moisture-conditioned, and re-compacted. The subgrade and select fill beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the requirements provided above in Table 3.

#### **7.1.4 Utility Trench Backfill**

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted in accordance with the recommendations presented in Table 3. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

#### **7.1.5 Drainage and Landscaping**

Positive surface drainage should be provided around the building to direct surface water away from foundations and below-grade walls. To reduce the potential for water ponding adjacent to the building, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations and below-grade walls.

### **7.2 Foundations**

Provided the estimated total and differential settlements presented in Section 6.1 are acceptable, the below-grade portion of the proposed podium structure may be supported on a stiffened mat foundation with the upper 18 inches of subgrade chemically treated with lime and/or cement. The at-grade portions of the apartment building may be supported on spread footings founded on compacted aggregate piers (CAPs) extending below the zone of weak deposits (approximately

Elevation 1 to 2 feet). The townhome structures may be supported on P-T slab foundations. The mat foundation system should be waterproofed and designed to resist hydrostatic uplift pressures. Specific recommendations for the design and construction of each foundation type are presented in the following sections.

### **7.2.1 Mat Foundation**

The mat foundation should be supported on firm and unyielding, lime- or cement-treated native soil, as described in Section 7.1. The top of the mat foundation may be used as the garage slab or a thin layer of concrete (topping slab) may be placed above the mat to provide a smooth wearing surface.

For design of a mat foundation, we recommend using a coefficient of vertical subgrade reaction of 30 kips per cubic foot (kcf) under DL+LL conditions. This value has been reduced to account for the size of the mat/equivalent footings (therefore, this is not  $k_{v1}$  for 1-foot-square plate). To check the behavior of the mat under seismic loads (including seismic), a coefficient of vertical subgrade reaction of 60 kcf should be used. Once a structural engineer evaluates the initial distribution of bearing stress on the bottom of the mat and corresponding deflections, we can review the distribution and revise the coefficients of subgrade reaction, if appropriate. We recommend the mat be designed for allowable bearing pressures of 3,000 psf for dead-plus-live loads and 4,000 psf for total loads (including seismic and wind loads); we anticipate the average bearing pressure will be significantly lower. Localized higher bearing pressures may be acceptable; however, this should be reviewed on a case-by-case basis. The mat should also be designed to resist the hydrostatic uplift pressures associated with the recommended design groundwater surface presented in Section 6.2.

Lateral forces can be resisted by a combination of friction along the base of the mat and passive resistance against the vertical faces of the mat foundation perpendicular to the direction of earthquake shaking. To calculate the passive resistance against the vertical faces of the mat under transient loading, we recommend using a uniform (rectangular) distribution of 1,500 psf.

The allowable friction factor will depend on the type of waterproofing used at the base of the mat. For bentonite-based water proofing membranes, such as Paraseal or Voltex, a friction factor of 0.12 should be used (assumes a bentonite friction angle of 10 degrees). If Preprufe is used, a base friction factor of 0.20 should be used. Friction factors for other types of waterproofing membranes can be provided upon request. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

The mat subgrade will be sensitive to disturbance due to its proximity to the groundwater table. The final two feet of excavation and fine grading of the mat subgrade should be performed with tracked equipment, in order to minimize heavy concentrated loads that may disturb the wet soil. Rubber-tired equipment and dump trucks should not be operated on the final mat subgrade. To provide a firm mat subgrade and to stabilize any weak clay deposits that may remain in place following excavation, we recommend the final 18 inches of soil subgrade be lime- and/or cement-treated in accordance with the recommendations presented in Section 7.1.1. The final prepared subgrade should be free of standing water, debris, and disturbed materials and be approved by the geotechnical engineer prior to placing a mud slab, waterproofing, or reinforcing steel. The mat subgrade should be kept moist following excavation and maintained in a moist condition until concrete is placed.

### **7.2.2 Footings Supported on Compacted Aggregate Piers**

Where conventional spread footings are used to support the at-grade portions of the apartment building, they should be underlain by CAPs that extend below the zone of weak marsh deposits and the adjacent below-grade parking level. The lengths and spacing of the CAPs should be determined by the design-build contractor that installs the CAPs; however, for planning purposes, it may be assumed that CAPs will be about 10 feet long. At a minimum, the CAPs (not including the bulb at the bottom) should extend at least two feet below the bottom of the weak deposits. We should review the design prior to construction and observe installation of production CAPs. The design capacity and load-settlement behavior of the CAPs should be

verified by at least one load test in compression and one test in tension, if uplift elements are used. We should provide load test parameters, oversee the testing program, and confirm that acceptable factors of safety and load-settlement characteristics exist for the design.

CAP-supported footings should be bottomed at least 18 inches below the lowest adjacent soil subgrade. Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Footings to be constructed near the adjacent basement wall or underground utility trenches should be bottomed below an imaginary line extending up at an inclination of 1.5:1 (horizontal:vertical) from the bottom of the basement wall or utility trench. Alternatively, the CAPs may be cement-treated within this zone of influence, which would eliminate the need for deepening the footings.

We estimate the allowable bearing pressure for CAP-supported footings would be about 5,000 psf for dead-plus-live loads and 6,600 psf for total loads. The allowable bearing pressure and estimated footing settlements should be confirmed by the design-build contractor. Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute passive resistance, we recommend using an allowable uniform pressure of 1,300 psf (rectangular distribution). The upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.50. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms of the footing excavations should be tamped with a jumping jack compactor to remove CAP disturbance caused by the excavation. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. We should check footing excavations prior to placement of reinforcing steel.

### 7.2.3 Post-Tensioned (P-T) Slab Foundations

We recommend P-T slabs be at least 10 inches thick. The edges of the foundation should be thickened such that the foundation edge is bottomed at least nine inches below the adjacent exterior grade or three inches below the bottom of the capillary moisture break, whichever is lower. Where a P-T slab is constructed near a bioswale or other stormwater treatment area, the edge of the slab should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal:vertical) from the base of the bioswale/treatment area. The maximum bearing pressure beneath the P-T slab should not exceed 2,500 psf under dead-plus-live-load conditions and 3,300 psf under total load conditions. For design of P-T slabs, we recommend using the parameters presented below in Table 4.

**TABLE 4**  
**P-T Slab Design Parameters**

Parameter	Value
Thornwaite Moisture Index	-10
Edge moisture variation distance	
edge lift	5.0 feet
center lift	9.0 feet
Percentage fines	50%
Percentage of clay	30%
Liquid limit	30%
Plasticity Index	12%
Suction Variance at Ground	1.5 pF
Differential Soil Movement	
edge lift	1.0 inches
center lift	-0.75 inches



Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the foundation and friction along the bottom of the mat or P-T slab. Passive resistance may be computed using an equivalent fluid weight of 300 pounds per cubic foot (pcf). The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30 where the slab is in contact with native soil and 0.20 where the slab is underlain by a vapor retarder. These values include a factor of safety of at least 1.5 and may be used in combination without reduction.

To reduce water vapor transmission through the P-T slabs, we recommend a vapor retarder be placed between the bottom of the P-T slab and the underlying subgrade soil. The vapor retarder should be at least 15 mils thick and meet the requirements for Class A vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

If required by the structural engineer, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. If a layer of sand is placed between the mat and the vapor retarder, we recommend the concrete have a w/c ratio less than 0.50. If the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45 and water not be added in the field. If necessary, workability may be increased by adding plasticizers. In addition, the slab should be properly cured.

Before floor coverings are placed over P-T slab foundations, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

The subgrade for the P-T slabs should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation may eventually heave, which may result in cracking and distress. We should check the foundation subgrade prior to placement of reinforcing steel.

### **7.3 Apartment Building Concrete Slab-On-Grade**

Where the apartment building is at-grade and founded on CAP-supported footings, the ground floor slab may consist of a concrete slab-on-grade. The soil subgrade beneath the floor slabs, should be scarified, moisture-conditioned, and compacted in accordance with the recommendations presented in Section 7.1.

In locations where water vapor transmission through the slab-on-grade is undesirable, we recommend installing a capillary moisture break and a water vapor retarder beneath the slab. A vapor retarder and capillary moisture break are often not required beneath parking garage floor slabs because there is sufficient air circulation to allow evaporation of moisture that is transmitted through the slab; however, we recommend the vapor retarder and capillary break be installed below the slab-on-grade in electrical/mechanical rooms and any areas in or adjacent to the parking garage that will be used for storage, office, retail, and/or will receive a floor covering or coating.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. If required by the structural engineer, the vapor

retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. However, excess water trapped in the sand could eventually be transmitted as vapor through the slab. Therefore, if rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 5.

**TABLE 5**  
**Gradation Requirements for Capillary Moisture Break**

Sieve Size	Percentage Passing Sieve
<i>Gravel or Crushed Rock</i>	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
<i>Sand</i>	
No. 4	100
No. 200	0 – 5

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If the concrete is poured directly over the vapor retarder (no sand layer), we recommend the w/c ratio of the concrete not exceed 0.45 and water not be added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before floor coverings, if

any, are placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

#### **7.4 Cantilevered Soldier Pile and Lagging Shoring**

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. A structural engineer knowledgeable in this type of construction should design the shoring. We should review the geotechnical aspects of the proposed shoring system to ensure that it meets our requirements. During construction, we should observe the installation of the shoring system and check the condition of the soil encountered during excavation.

As discussed in Section 6.3, we conclude cantilevered soldier-pile-and-lagging is a viable shoring system for this project. We recommend the cantilevered soldier pile-and-lagging shoring system be designed to resist an active equivalent fluid weight of 40 pcf where the retained ground surface is level. In locations where minimizing lateral deflections is critical, such as near adjacent buildings or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 60 pcf (level ground surface) plus any foundation surcharge loads. Where construction equipment will be working behind the walls within a horizontal distance equal to the wall height, the design should include a surcharge pressure of 250 psf. The above pressures should be assumed to act over the entire width of the lagging installed above the excavation. The active pressure need only be assumed to act over one pile width below the bottom of the excavation and may be reduced to 21 pcf below the "during-construction" groundwater level. The above pressures assume that during construction, the groundwater level is drawn down below the bottom of excavation.

Passive resistance at the toe of the soldier pile should be computed using an equivalent fluid weight of 150 pcf, but the passive pressure should be limited to 1,600 psf with depth. The recommended passive pressure assumes the groundwater will be at about the bottom of

excavation. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. The shoring designer should check that the specified minimum concrete strength is sufficient to spread the anticipated loads to three soldier pile widths. If the soldier piles are vibrated into place, rather than being placed into drilled holes with concrete, then the passive pressure should only be applied to three beam flange widths. These passive pressure values include a factor of safety of at least 1.5.

Based on the results of our borings and CPTs, we expect that some of the soil to be retained by the shoring has insufficient cohesion to stand vertically, especially if the groundwater is not adequately pumped down below the bottom of the excavation. Where exposed, the clay materials may be capable of temporarily maintaining four-foot vertical cuts. However, where granular materials are exposed, much smaller incremental cuts may be necessary during excavation and lagging placement. If voids are created behind lagging boards due to localized caving or overcutting, they should be filled with cement slurry or hand-packed soil prior to proceeding with excavation.

The contractor should establish survey points on the shoring and on the ground surface at critical locations behind the shoring prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring during construction.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch. A monitoring program should be established to evaluate the effects of the construction on the adjacent properties.

## 7.5 Permanent Below-Grade Walls

Permanent below-grade walls should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, and traffic loads (if vehicular traffic is expected within 10 feet of the wall). We recommend the permanent below-grade walls be designed for the more critical of the following criteria where they are above the design water level:

- At-rest equivalent fluid weight of 60 pounds per cubic foot (pcf), or
- Active equivalent fluid weight of 40 pcf, plus a seismic equivalent fluid weight of 12 pcf.

Portions of the below-grade walls that are below the design groundwater level should be designed for an at-rest pressure of 94 pcf or an active-plus-seismic equivalent fluid weight of 83 plus 12 pcf. The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. Where the below-grade wall is subject to traffic loading within 10 feet of the wall, an additional uniform lateral pressure of 100 psf applied to the upper 10 feet of the wall.

The lateral earth pressures recommended are applicable to walls that are backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the shoring or the back of the wall. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the wall or just above the design groundwater level (whichever is higher). The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC or equivalent). A proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil, designed to work in conjunction with the drainage panel may be used in lieu of the perforated pipe surrounded by gravel described above. The collector pipe should outlet into the storm drain system outside the garage, if possible. We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is

appropriate for its intended use. To protect against moisture migration into the below-grade parking level, we recommend that the below-grade walls be water-proofed and water stops be installed at all construction joints.

## **7.6 Flexible and Rigid Pavement Design**

Design recommendations for non-permeable asphalt concrete and Portland cement concrete pavements are presented in the following sections.

### **7.6.1 Flexible (Asphalt Concrete) Pavement Design**

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We performed a resistance value (R-value) test on a sample of the silty gravel with sand encountered in boring B-2 in the upper two feet, which resulted in an R-value of 54. However, based on our experience and the inherent variability of the surficial fills present on this site, we assumed a lower design R-value of 25. Recommended pavement sections for traffic indices ranging from 4.5 to 7.0 are presented in Table 6.

**TABLE 6**  
**AC Pavement Sections**

<b>Traffic Index</b>	<b>Asphaltic Concrete (inches)</b>	<b>Class 2 Aggregate Base R = 78 (inches)</b>
4.5	2.5	7.0
5.0	3.0	7.0
5.5	3.0	8.0
6.0	3.5	9.0
6.5	4.0	9.0
7.0	4.0	11.0

For regular garbage truck traffic (three garbage trucks making two site visits per week), we recommend a traffic index of 5.0 be used for flexible pavement design. The upper six inches of

the subgrade should be moisture-conditioned and compacted in accordance with requirements presented in Table 3 in Section 7.1. The subgrade should be proof-rolled to confirm it is non-yielding prior to placement of the AB. If soft, pumping subgrade is encountered, subgrade stabilization measures, as outlined in Section 7.1.1 will be necessary. The aggregate base should be moisture-conditioned to near optimum and compacted to at least 95 percent relative compaction. The AB should also be proof-rolled to confirm it is non-yielding prior to paving.

If pavements are adjacent to irrigated landscaped areas (including infiltration basins), curbs adjacent to those areas should extend through the aggregate base and at least three inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the pavement section. If drip irrigation is used in the landscaping adjacent to the pavement, however, the deepened curb is not required.

#### **7.6.2 Rigid (Portland Cement Concrete) Pavement**

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle load of 32,000 pounds and the garbage truck traffic discussed above. The recommended rigid pavement section for these axle loads is five inches of Portland cement concrete over six inches of Class 2 aggregate base. Where fire truck traffic is expected, the pavement section should consist of 6-1/2 inches of Portland cement concrete over six inches of Class 2 aggregate base.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt concrete pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For areas that will receive weekly garbage truck traffic, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch spacing in both directions. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those described above for asphalt concrete pavement.



## **7.7 Pavers**

Recommendations for non-permeable and permeable pavers, as well as grass pavers are presented in the following sections.

### **7.7.1 Non-Permeable Concrete Pavers**

Non-permeable concrete pavers for pedestrian traffic should be underlain by at least 6 inches of select fill compacted to at least 90 percent RC. The soil subgrade beneath pavers should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction.

Where non-permeable concrete pavers will be subject to vehicular traffic, we recommend they consist of fully dentated, interlocking shapes and be at least 80 millimeters (3.15 inches) thick. The pavers should be placed on a 1- to 2-inch-thick sand leveling course underlain by Class 2 AB. The AB thickness beneath non-permeable pavers subject to vehicular traffic should be consistent with that recommended in Table 6 for asphalt pavement for the appropriate traffic index. The subgrade and AB should be compacted in accordance with the recommendations for PCC pavement in Section 7.6.

### **7.7.2 Permeable Concrete Pavers**

We recommend permeable interlocking concrete pavements (ICP) be designed in accordance with the guidelines presented by the Interlocking Concrete Pavement Institute (ICPI 2005). These guidelines include specific recommendations for permeable aggregate subbase, base, and bedding courses to be placed beneath ICP pavements. We recommend permeable pavements for both vehicular and pedestrian traffic be designed for no exfiltration of water into the subgrade soil. This requires installing a subdrain system at the base of the pervious aggregate materials, which are underlain by an impermeable liner.

The soil subgrade beneath ICP pavements should be prepared and compacted in accordance with the recommendations presented in Section 7.1. In addition, the subgrade should be a firm and

non-yielding surface. The subgrade should be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to placing the impermeable membrane and aggregate base materials. The soil subgrade at the bottom of the permeable section should slope down toward the drain pipe trench at a gradient of at least two percent. The perforated pipe should slope down to a suitable outlet at a minimum gradient of one percent. The pipe should be placed with the perforations down on a minimum of two inches of permeable subbase.

ICPI's guidelines call for 1-1/2 to 2 inches of bedding material consisting of ASTM No. 8 aggregate directly below the pavers. This material is also recommended for fill material between the pavers. As shown in Table 7 below, this material consists of fine gravel with 10 to 30 percent sand.

**TABLE 7**  
**Gradation Requirements for ASTM No. 8 Aggregate**

Sieve Size	Percentage Passing Sieve
1/2 inch	100
3/8 inch	85 – 100
No. 4	10 – 30
No. 8	0 – 10
No. 16	0 – 5

The ASTM No. 8 bedding should be underlain by a permeable base course of ASTM No. 57 crushed aggregate. As shown in Table 8, ASTM No. 57 aggregate consists of open-graded gravel with a gradation between that of the 3/4-inch drain rock and the ASTM No. 8 aggregate.

**TABLE 8**  
**Gradation Requirements for ASTM No. 57 Aggregate**

Sieve Size	Percentage Passing Sieve
1-1/2 inch	100
1 inch	95 – 100
1/2 inch	25 – 60
No. 4	0 – 10
No. 8	0 – 5

The ASTM No. 57 permeable base course should be underlain by a permeable subbase course of ASTM No. 2 crushed aggregate. The gradation requirements for ASTM No. 2 crushed aggregate subbase are presented in Table 9.

**TABLE 9**  
**Gradation Requirements for ASTM No. 2 Aggregate**

Sieve Size	Percentage Passing Sieve
3 inch	100
2-1/2 inch	90-100
2 inch	35-70
1-1/2 inch	0-15
3/4 inch	0 -5

The No. 2 aggregate subbase course should be placed in lifts not exceeding 6 inches in loose thickness and compacted using a smooth-drum roller that weighs a minimum of 10 tons, operated in static (non-vibratory) mode. The subsequent course of No. 57 aggregate may be

placed in one lift and should be compacted with a smooth-drum roller in vibratory mode with sufficient passes to create an unyielding surface. Placement and compaction of the permeable aggregate base and subbase should be performed under the observation of our field engineer. Following compaction of the No. 57 aggregate, the No. 8 bedding, not exceeding 2 inches in loose thickness, should be placed and screeded to a level, undisturbed surface immediately prior to paver installation.

The required thicknesses of the permeable aggregate base and subbase courses depends on the infiltration and water storage design requirements, as well as the traffic loading demand. Our recommendations for the minimum permeable ICP pavement sections subject to vehicular traffic (including fire and garbage trucks) are presented in Table 10. Also included in Table 10 is a recommended section for permeable ICPs subject to pedestrian traffic only.

**TABLE 10**  
**Recommended Pavement Sections for**  
**Permeable Interlocking Concrete Pavers**

<b>Pavement Type</b>	<b>ASTM No. 8 Bedding Aggregate (inches)</b>	<b>ASTM No. 57 Stone Base (inches)</b>	<b>ASTM No. 2 Stone Subbase (inches)</b>
Pedestrian	1.5-2.0	4.0	6.0
Vehicular	1.5-2.0	4.0	10.0

The above recommended ICP pavement sections are based on the ICPI technical guidelines (ICPI 2005). From a geotechnical standpoint, it is acceptable to design the pedestrian ICP section to exclude the No. 2 subbase course, in which case the No. 57 base course should be increased to 10 inches. If this approach is used, the perforated pipe should include a filter fabric sleeve to prevent the finer aggregate from entering the perforations.

### 7.7.3 Grass Pavers

If permeable grass pavers, such as Grasspave 2, are used for portions of the Emergency Vehicle Access (EVA) road/pedestrian paseo, they should be designed in accordance with the recommendations presented in this section of the report, as well as the manufacturer's specifications.

We recommend that permeable grass pavers that are not designed as a stormwater treatment system be supported directly on a layer of compacted sandy gravel that is at least 12 inches thick and compacted to at least 95 percent relative compaction. The sandy gravel should meet the gradation requirements called for in the manufacturer's specification. A subdrain, as described above for permeable concrete pavers (ICP), should be placed below the sandy gravel layer. The native soil subgrade should be sloped and compacted as described above for ICPs. A layer of Tensar TriAx TX160 geogrid (or equivalent) should be placed on the compacted subgrade prior to placement of the sandy gravel layer.

## 7.8 Seismic Design

We understand the proposed building will be designed using the seismic provisions in the 2013 California Building Code (CBC). The latitude and longitude of the site are  $37.58678^{\circ}$  and  $-122.35751^{\circ}$ , respectively. Although the 2013 CBC calls for a Site Class F designation for sites underlain by liquefiable soil, we conclude a Site Class D is appropriate considering the potentially liquefiable layers are generally thin and discontinuous. Therefore, the soil profile will not incur significant nonlinear behavior during strong ground shaking. For seismic design we recommend Site Class D be used. For seismic design in accordance with the 2013 CBC, which references the 2010 ASCE 7 Standard, we recommend the following:

- Site Class D
- $S_S = 2.103g$ ,  $S_1 = 0.997g$
- $S_{MS} = 2.103g$ ,  $S_{M1} = 1.495g$
- $S_{DS} = 1.402g$ ,  $S_{D1} = 0.997g$
- Seismic Design Category E (for Risk Categories I, II and III).

## **7.9 Soil Corrosivity**

Corrosivity analyses were performed by Sunland Analytical on samples of the native clayey sand with gravel from boring B-1 at 7-1/2 feet bgs and gravelly clay from boring B-4 in the upper 2 feet. The corrosivity test results are presented in more detail in Appendix B of this report. The results of the corrosivity analyses indicate both samples are “corrosive” with respect to resistivity. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection. The results indicate that sulfate ion concentrations are sufficiently low to not pose a threat to buried concrete. In addition, the chloride ion concentrations are insufficient to adversely impact steel reinforcement in concrete structures below ground.

## **8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION**

Prior to construction, Rockridge Geotechnical should review the project plans and specifications to verify that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, placement and compaction of fill, installation of foundations, and shoring installation. These observations will allow us to compare actual with anticipated soil conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

## **9.0 LIMITATIONS**

This geotechnical study has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations

presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

## REFERENCES

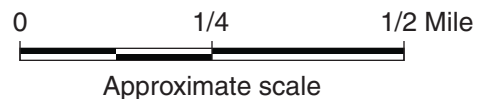
- American Concrete Institute (2001), "Guide for Design and Construction of Concrete Parking Lots", report ACI 330R-01.
- Brabb, E.E., Graymer, R.W., Jones, D.L. (1998), Geology of the Onshore Part of San Mateo, California: Derived from Digital Database Open-File 98-137.
- California Division of Mines and Geology (1996), Probabilistic seismic hazard assessment for the State of California, DMG Open-File Report 96-08.
- California Geological Survey (2008), Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117.
- Cao, T., Bryant, W. A., Rowshandel, B., Branum D. and Wills, C. J. (2003). "The Revised 2002 California Probabilistic Seismic Hazard Maps"
- Idriss, I.M., Boulanger, R.W. (2008), "Soil Liquefaction During Earthquakes," Earthquake Engineering Research Institute (EERI), MNO-12.
- Ishihara, K. (1985), "Stability of Natural Deposits During Earthquakes."
- Lew, M., Sitar, N. (2010), "Seismic Earth Pressures on Deep Building Basements," SEAOC 2010 Convention Proceedings.
- Robertson, P.K. (2009), "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009.
- Robertson, P.K. (2009), "Interpretation of Cone Penetration Tests - A Unified Approach", Canadian Geotechnical Journal, Vol. 46, No. 11, pp 1337-1355.
- Toppozada, T.R. and Borchardt G. (1998). "Re-evaluation of the 1936 "Hayward Fault" and the 1838 San Andreas Fault Earthquakes." Bulletin of Seismological Society of America, 88(1), 140-159.
- U.S. Geological Survey, (2008), The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): prepared by the 2007 Working Group on California Earthquake Probabilities, U.S. Geological Survey Open File Report 2007-1437.
- Zhang, G., Robertson, P.K., Brachman, R., (2002), "Estimating Liquefaction Induced Ground Settlements from the CPT", Canadian Geotechnical Journal, 39: pp 1168-1180.



## FIGURES



Base map: The Thomas Guide  
San Mateo County



**1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD**  
Burlingame, California



## SITE LOCATION MAP

Date 07/04/13




Project No. 13-560

Figure 1

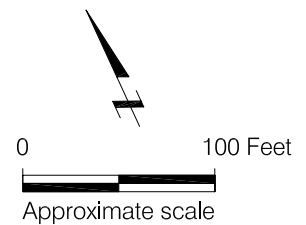




#### EXPLANATION

- B-1  Approximate location of boring by Rockridge Geotechnical, Inc., November, 2013
- CPT-5  Approximate location of Cone Penetration Test by Rockridge Geotechnical, Inc., November, 2013
- CPT-1  Approximate location of Cone Penetration Test by Rockridge Geotechnical, Inc., July, 2013

 Site boundary



Base map: Google Earth Pro, 2013.

**1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD**  
Burlingame, California



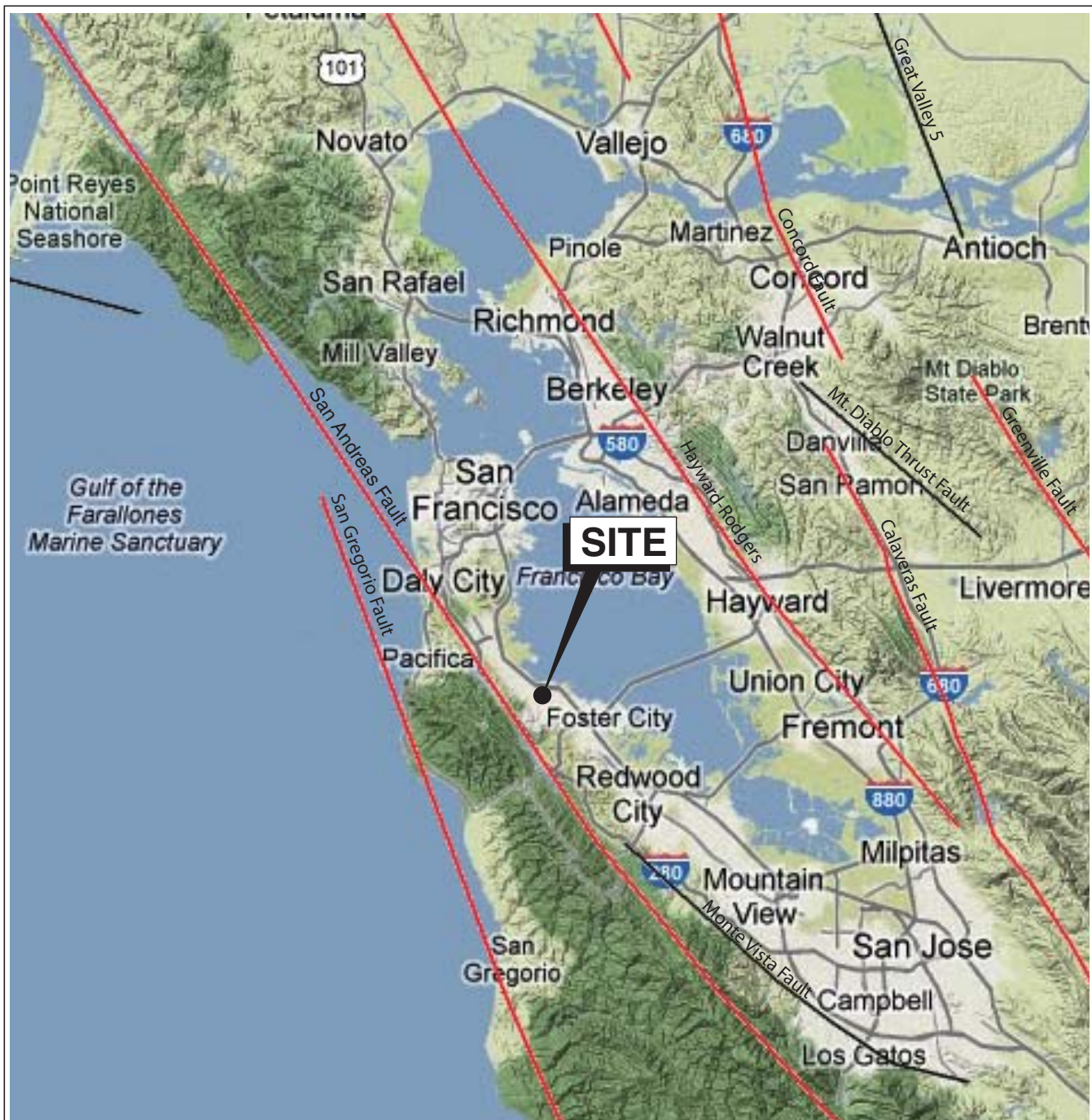
#### SITE PLAN

Date 11/29/13

Project No. 13-560

Figure 2

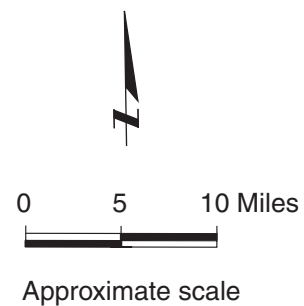




Base Map: U.S. Geological Survey, National Seismic Hazards Maps - Fault Sources, 2008.

#### EXPLANATION

- Strike slip
- Thrust (Reverse)
- Normal

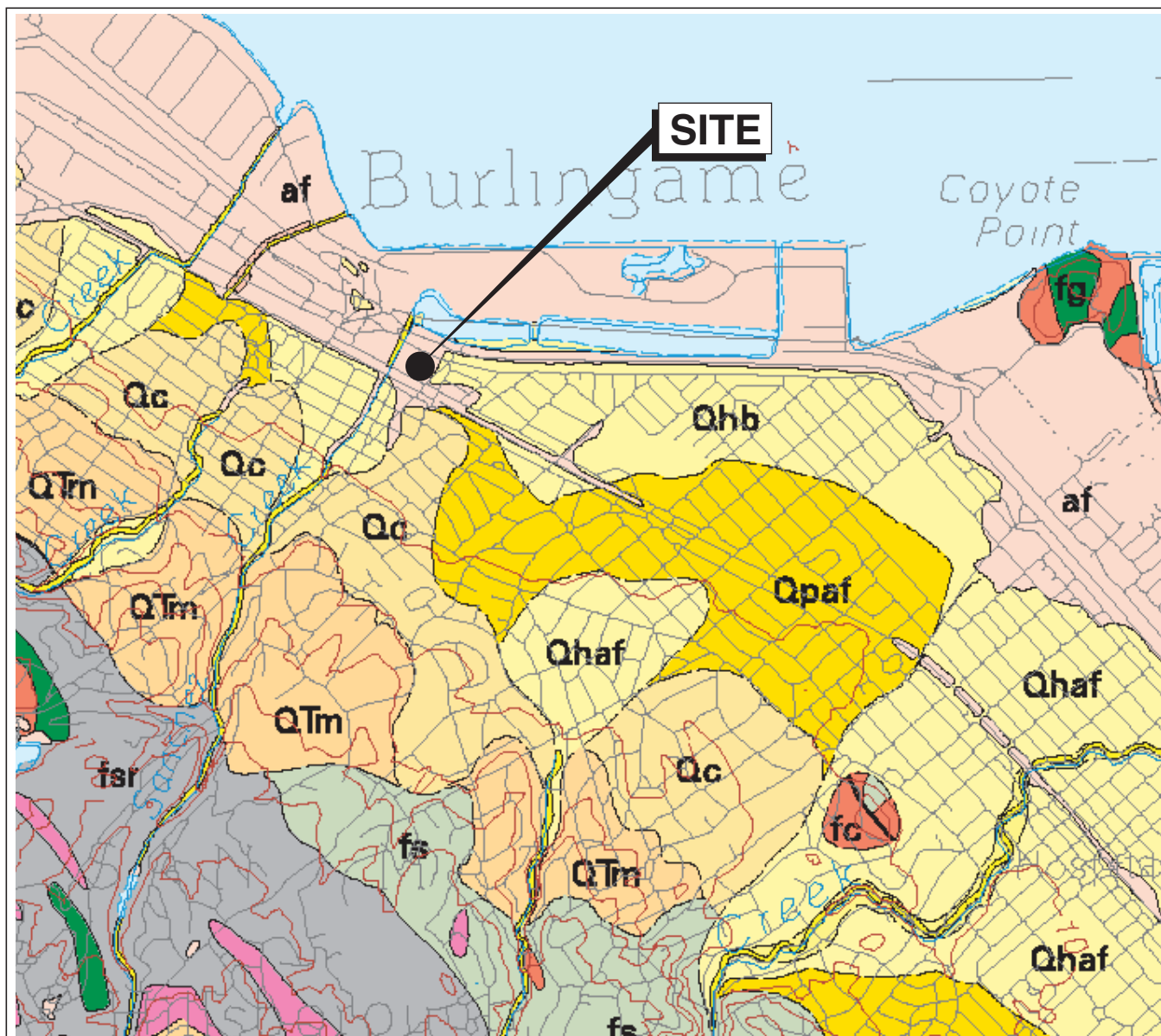


1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

**ROCKRIDGE**  
GEOTECHNICAL

#### REGIONAL FAULT MAP

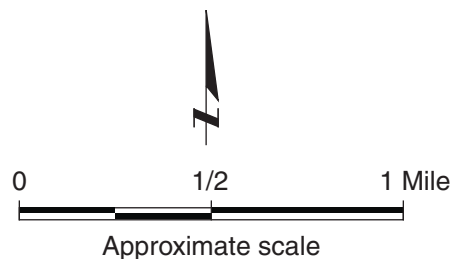
Date 07/04/13	Project No. 13-560	Figure 3
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Base map: Geology of the Onshore Part of San Mateo County, California: Derived from the Digital Database open-file 98-137.

- af - Artificial fill (Historic)
- Qpaf - Alluvial Fan and Fluvial Deposits (Pleistocene)
- Qhaf - Alluvial Fan and Fluvial Deposits (Holocene)
- Qhb - Basin deposits (Holocene)
- QTm - Merced Formation (lower and upper Pleistocene)
- Qc - Colma Formation (Pleistocene)
- fs - Sandstone
- fsr - Sheared rock (melange)
- fc - Chert

- Contact-Depositional or intrusive contact, dashed where approximately located, dotted where concealed
- Fault-Dashed where approximately located, small dashes where inferred, dotted where concealed, queried where location is uncertain



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## REGIONAL GEOLOGIC MAP

Date 07/08/13

Project No. 13-560

Figure 4

## **APPENDIX A**

### **Boring Logs and Cone Penetration Test Results**

PROJECT: 1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

# Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: T. Williams

Date started: 11/20/13

Date finished: 11/20/13

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches'

Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

## LABORATORY TEST DATA

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	Blows/ 6"	SPT N-value								
						Approximate Ground Surface Elevation: 8.3 feet <sup>2</sup>						
1	GRAB					2.5 inches Asphalt Concrete (AC)						
2						4 inches Aggregate Base (AB)						
3					CL	CLAY with SILT and SAND (CL) olive-brown, moist Planting Suitability Test; see Appendix D						
4	GRAB											
5						CLAYEY SAND with GRAVEL (SC) yellow-brown, moist				24		
6												
7						▽ (11/20/13; 10:45 AM)						
8	S&H		7 13 16	23		medium dense, wet Corrosivity Test; see Appendix B				18	19.9	110
9						dense						
10					SC							
11	SPT		7 16 17	48								
12						very dense, increased clay content						
13	SPT		7 19 21	58								
14												
15						medium dense						
16	SPT		5 6 7	19								
17						CLAY with SAND (CL) yellow-brown, moist, logged from drill cuttings						
18					CL							
19												
20						CLAY with SAND (CL) yellow-brown with light brown mottling, stiff, wet Consolidation Test; see Appendix B					25.5	100
21	S&H		0 6 8	11								
22												
23												
24												
25					CL	light olive with yellow-brown mottling, very stiff, increase sand content						
26	S&H		5 9 11	16								
27												
28												
29												
30												

ROCKRIDGE 13-560.GPJ TR.GDT 1/13/14



Project No.:

13-560

Figure:

A-1a








PROJECT:

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

## Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value <sup>1</sup>			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		10 17 25			CLAY with SAND (CL) (continued) light olive, hard, with trace fine gravel						
32												
33												
34												
35												
36	S&H		8 8 30	30	CL	brown, very stiff to hard						
37												
38												
39												
40												
41	SPT		8 18 19	53	SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, very dense, wet						
42						interbedded gravel layers						
43												
44												
45												
46	SPT		3 4 4	12	CL	CLAY (CL) yellow-brown mottled with olive, stiff, wet						
47												
48												
49												
50												
51	SPT		21 40 26	95	SP-SC	SAND with CLAY and GRAVEL (SP-SC) yellow-brown, very dense, wet						
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51.5 feet below ground surface.  
Boring backfilled with cement grout.  
Groundwater encountered at a depth of 7 feet during drilling.

<sup>1</sup> SPT and S&H blow counts for the last two increments were converted to SPT N-Values using factors of 1.44 and 0.8, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was used without liners.

<sup>2</sup> Ground surface elevation based on Topographic Survey by BKF Engineers, dated 10/08/13 (NGVD 29 Datum).



Project No.:

13-560

Figure:

A-1b



PROJECT: 1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

# Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: T. Williams

Date started: 11/19/13

Date finished: 11/20/13

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches'

Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

## LABORATORY TEST DATA

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	Blows/ 6"	SPT N-value								
						Approximate Ground Surface Elevation: 10.5 feet <sup>2</sup>						
1	GRAB					2.5 inches Asphalt Concrete (AC)						
2						3-4 inches Aggregate Base (AB)						
3	S&H		5	14	GM	SILTY GRAVEL with SAND (GM) olive-gray, loose to medium dense, moist R-Value Test; see Appendix B Planting Suitability Test; see Appendix D				24		
4			8									
5	S&H		4	18	CL	SANDY CLAY with GRAVEL (CL) yellow-brown, very stiff, moist, sandstone fragments LL = 32, PI = 14; see Appendix B					20.3	104
6			8									
7	S&H		6	16	SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, medium dense, wet (11/20/13; 7:00 AM)						
8			10									
9	S&H		7	19		increased gravel content, strong hydrocarbon odor				17		
10			11									
11	S&H		5	17	SC	CLAYEY SAND (SC) gray, medium dense, wet, strong hydrocarbon odor (11/19/13; 3:00 PM) LL = 27, PI = 10; see Appendix B				49	18.9	100
12			9									
13	SPT		10	16		SANDY CLAY (CL) yellow-brown, very stiff, wet, interbedded thin clayey sand lenses decreased sand content						
14			6									
15			5									
16	S&H		1	12		stiff						
17			6									
18			9									
19												
20												
21	SPT		12	16	SC	very stiff, with thin silty sand lenses						
22			8									
23			3									
24												
25												
26	S&H		6	17		with trace gravel						
27			9									
28			12									
29												
30												

ROCKRIDGE 13-560.GPJ TR.GDT 1/13/14



Project No.:

13-560

Figure:

A-2a

PROJECT:

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

## Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value <sup>1</sup>			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		5	14		SANDY CLAY (CL) (continued) stiff	PP		1,900			
32			7									
33			10									
34												
35												
36	S&H		19	40		yellow-brown mottled with light gray, hard						
37			18									
38			32									
39						CLAYEY SAND (SC) yellow-brown, medium dense, wet						
40												
41	S&H		4	15								
42			8		SC							
43			10									
44												
45												
46	S&H		7	25								
47			12			CLAYEY GRAVEL with SAND (GP) olive-gray, medium dense, wet						
48			19		GP							
49												
50						SANDY CLAY (CL) yellow-brown mottled with gray, hard, wet						
51	SPT		21	37	CL							
52			10									
53			16									
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51.5 feet below ground surface.  
Boring backfilled with cement grout.  
Groundwater stabilized at depth of about 6.5 feet after 16 hours.

<sup>1</sup> SPT and S&H blow counts for the last two increments were converted to SPT N-Values using factors of 1.44 and 0.8, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was used without liners.

<sup>2</sup> Ground surface elevation based on Topographic Survey by BKF Engineers, dated 10/08/13 (NGVD 29 Datum).



Project No.:

13-560

Figure:

A-2b

PROJECT:

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

## Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: A. Rikli

Date started: 11/23/13

Date finished: 11/23/13

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches'

Hammer type: Automatic Hammer

Sampler: Sprague &amp; Henwood (S&amp;H), Standard Penetration Test (SPT), Shelby Tube (ST)

## LABORATORY TEST DATA

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	Blows/ 6"	SPT N-value								
						Approximate Ground Surface Elevation: 9.5 feet <sup>2</sup>						
1						3 inches Asphalt Concrete (AC)						
2						4 inches Aggregate Base (AB)						
3						CLAY with SAND (CH)						
4	ST		25-35 psi		CH	olive, stiff, moist	PP		2,000			
5												
6	S&H		5	67		SANDY CLAY (CL)					15.0	118
7			38			yellow-brown, hard, moist						
8	SPT		9	40	CL					55		
9			13									
10			15			CLAYEY SAND (SC)						
11	S&H		3	27	SC	yellow-brown, medium dense, wet				28	18.4	111
12			13									
13	S&H		3	11		CLAY with SAND (CL)						
14			4			yellow-brown, stiff, wet, trace gravel	TxJU	800	1,900		24.3	102
15			10			(11/23/13; after 5 minutes)						
16	S&H		6	23	CL	very stiff	PP		1,500			
17			12									
18			17									
19												
20						GRAVELLY CLAY with SAND (CL)						
21	S&H		13	49		yellow-brown mottled with olive, hard, wet						
22			30									
23			31									
24												
25												
26	S&H		8	17	CL	very stiff						
27			10									
28			11									
29						increased gravel content						
30												

ROCKRIDGE 13-560.GPJ TR.GDT 1/13/14



Project No.:

13-560

Figure:

A-3a

PROJECT:

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

## Log of Boring B-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value <sup>1</sup>			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		5 7 10	24	CL	GRAVELLY CLAY with SAND (CL) (continued)						
32												
33												
34												
35												
36	S&H		2 6 7	10		CLAY (CL) yellow-brown, stiff, wet, trace sand and gravel Consolidation Test; see Appendix B	PP		700		26.5	95
37												
38												
39					CL							
40												
41	S&H		3 3 8	9		increased gravel content	PP		400		30.8	94
42												
43												
44												
45												
46	S&H		19 33 50/4"	66/ 10"		CLAYEY SAND (SC) yellow-brown mottled with gray, very dense, wet						
47	SPT		40 44 50/3"	135/ 9"	SC							
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 48 feet below ground surface.  
Boring backfilled with cement grout.  
Groundwater encountered at a depth of 12 feet during drilling.

<sup>1</sup> SPT and S&H blow counts for the last two increments were converted to SPT N-Values using factors of 1.44 and 0.8, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was used without liners.

<sup>2</sup> Ground surface elevation based on Topographic Survey by BKF Engineers, dated 10/08/13 (NGVD 29 Datum).

**ROCKRIDGE**  
GEOTECHNICAL

Project No.:

13-560

Figure:

A-3b

ROCKRIDGE 13-560.GPJ TR.GDT 1/13/14

PROJECT: **1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD**  
Burlingame, California

# Log of Boring B-4

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: T. Williams

Date started: 11/19/13

Date finished: 11/19/13

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches'

Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

## LABORATORY TEST DATA

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	Blows/ 6"	SPT N-Value <sup>1</sup>								
						Approximate Ground Surface Elevation: 9.1 feet <sup>2</sup>						
1	GRAB				CL	1.5 inches Asphalt Concrete (AC)						
2						2 inches concrete						
3	S&H		5	11	ML	4 inches Aggregate Base (AB)						
4			6			GRAVELLY CLAY (CL)						
5	S&H		8			red-brown, moist				52	17.5	106
6						Corrosivity Test, see Appendix B						
7	S&H		2	9		SANDY SILT (ML)						
8			4			dark gray, stiff, moist						
9			7			Planting Suitability Test; see Appendix D						
10	ST		25		CL	increase sand content	TxUU	400	880		25.9	100
11						CLAY (CL)						
12						dark gray, medium stiff, moist, organic						
13	S&H		6	19	CL	LL = 30, PI = 19; Consolidation Test; see Appendix B					20.8	101
14			11			CLAY with SAND (CL)	PP		700			
15	S&H		10			yellow-brown, medium stiff to very stiff, wet						
16			16			(11/19/13; 8:45 AM)						
17	S&H		7			SANDY CLAY with GRAVEL (CL)						
18			15			olive-brown mottled with yellow-brown and dark						
19	S&H		26		CL	brown, hard, wet, with sandstone fragments						
20												
21	SPT		9	23		CLAYEY SAND (SC)						
22			9			yellow-brown, medium dense, wet						
23			10			LL = 37, PI = 21; see Appendix B				22	21.9	
24												
25	SPT		7	25		with trace gravel						
26			8									
27			13									
28												
29												
30	SPT		90/6"	108/6"	SC	CLAYEY SAND (SC)						
						yellow-brown, very dense, cemented, wet						

Boring terminated at a depth of 25.5 feet below ground surface.  
Boring backfilled with cement grout.  
Groundwater encountered at a depth of 8 feet during drilling.

<sup>1</sup> SPT and S&H blow counts for the last two increments were converted to SPT N-Values using factors of 1.44 and 0.8, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was used without liners.  
<sup>2</sup> Ground surface elevation based on Topographic Survey by BKF Engineers, dated 10/08/13 (NGVD 29 Datum).

**ROCKRIDGE  
GEOTECHNICAL**

Project No.:

13-560

Figure:

A-4

UNIFIED SOIL CLASSIFICATION SYSTEM			
Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4	76.2 to 4.76
	3" to 3/4"	76.2 to 19.1
	3/4" to No. 4	19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200	4.76 to 0.075
	No. 4 to No. 10	4.76 to 2.00
	No. 10 to No. 40	2.00 to 0.420
	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

C	Core barrel	PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter	S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube	SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube	ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

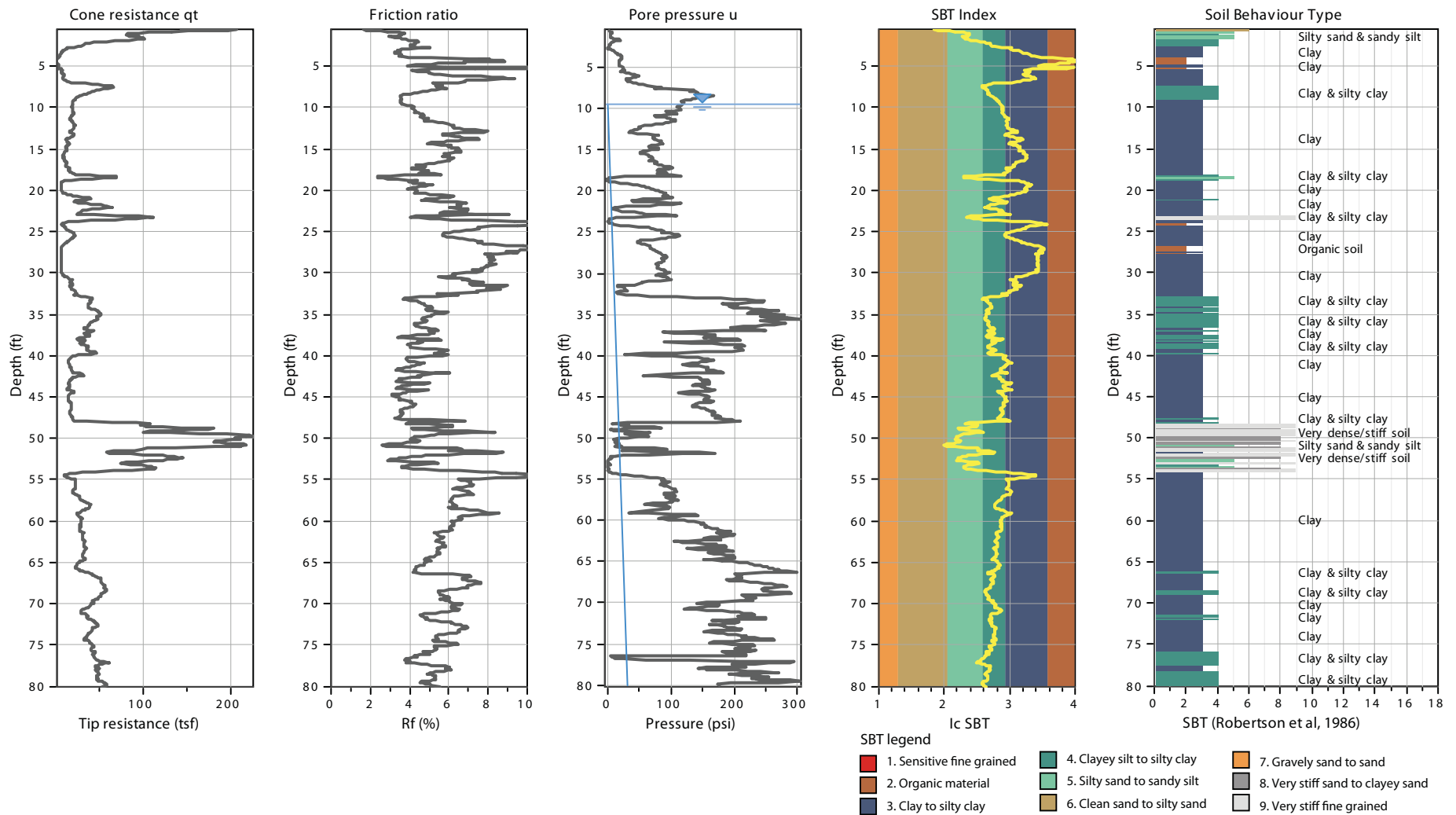
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GEOTECHNICAL

Date 11/29/13

Project No. 13-560

Figure A-5



Total depth: 80.05 ft, Date: 7/3/2013  
 Surface Elevation: 9.20 ft  
 Cone Operator: John Sarmiento & Associates

1008, 1016, & 1028 CAROLAN AVENUE/  
 935 ROLLINS ROAD  
 Burlingame, California

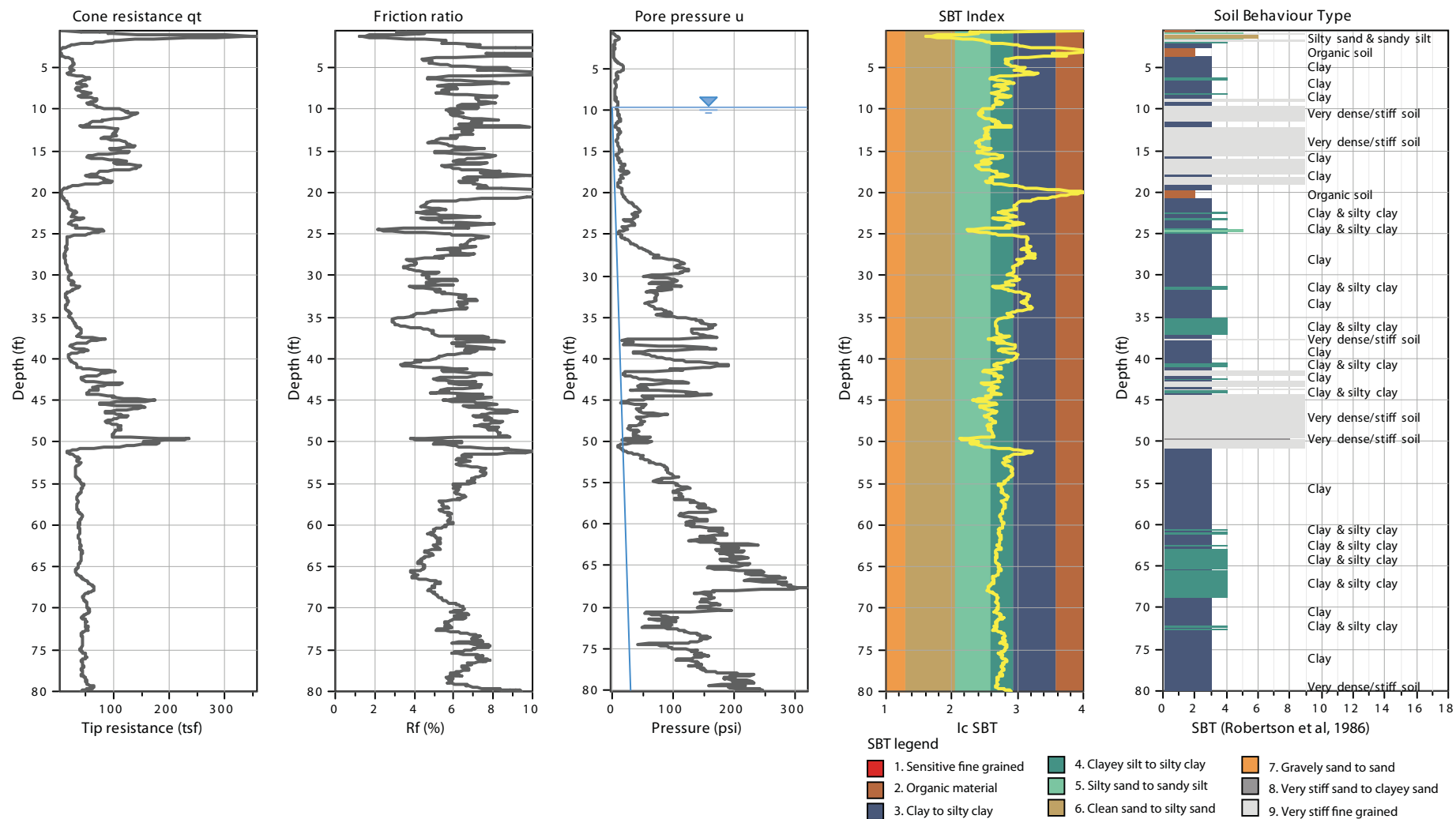
**ROCKRIDGE**  
**GEOTECHNICAL**

## CONE PENETRATION TEST RESULTS CPT-1

Date 12/06/13

Project No. 13-560

Figure A-6



Total depth: 80.04 ft, Date: 7/3/2013

Surface Elevation: 10.00 ft

Cone Operator: John Sarmiento & Associates

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California



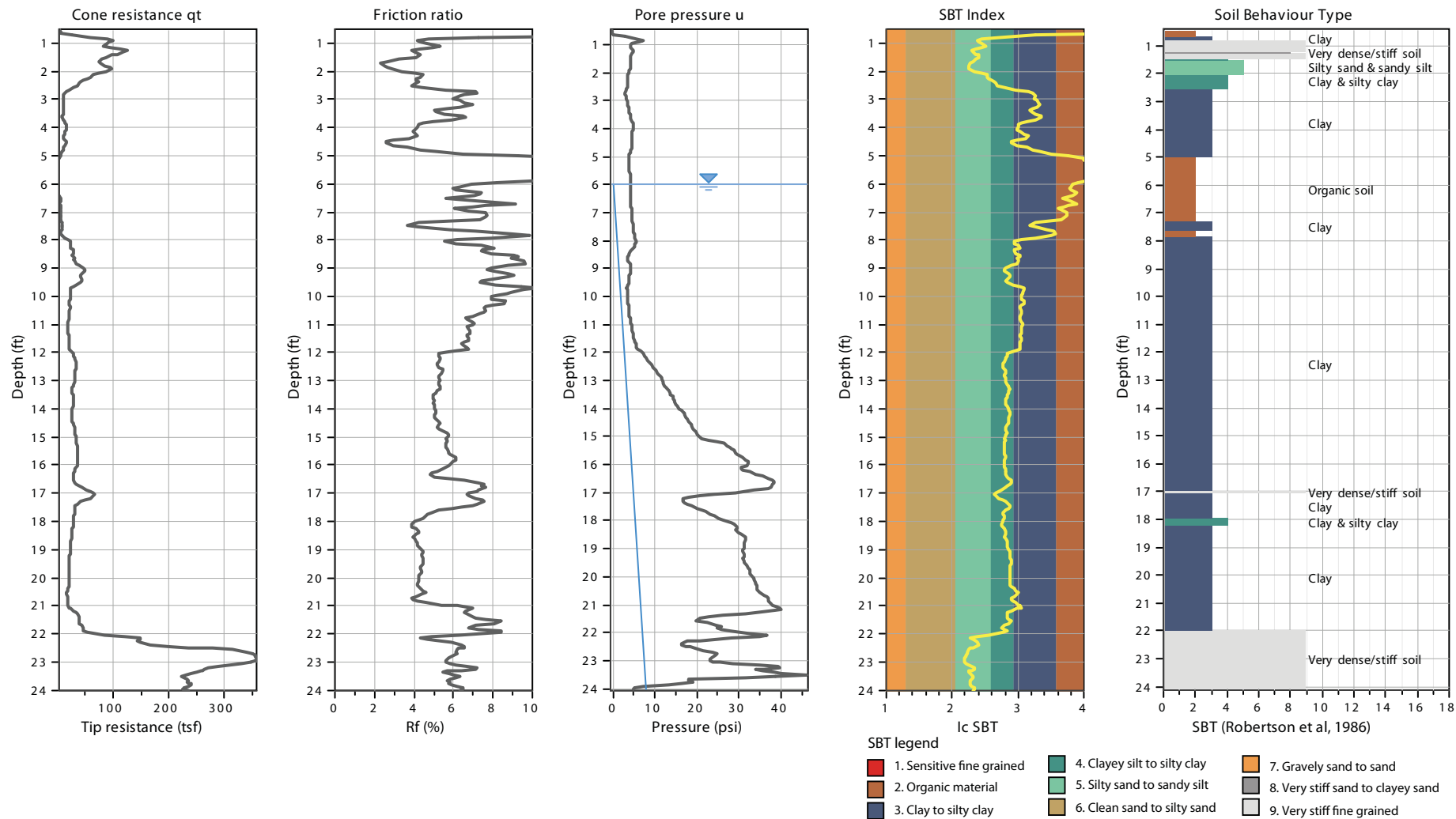
## CONE PENETRATION TEST RESULTS CPT-2

Date 12/06/13

Project No. 13-560

Figure A-7





Total depth: 24.02 ft, Date: 7/3/2013

Surface Elevation: 9.00 ft

Cone Operator: John Sarmiento & Associates

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935 ROLLINS ROAD  
Burlingame, California

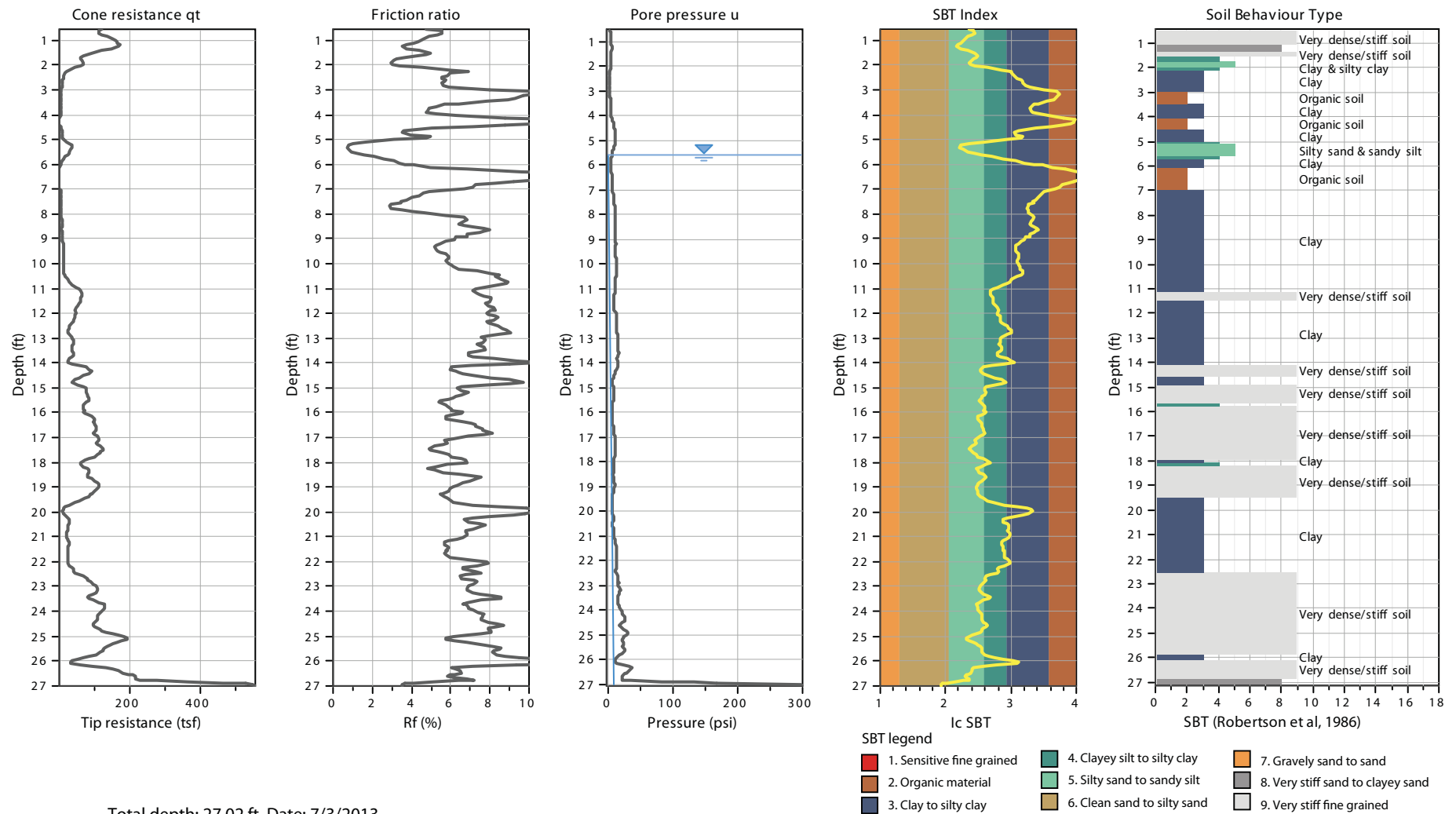
**ROCKRIDGE**  
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## CONE PENETRATION TEST RESULTS CPT-3

Date 12/06/13

Project No. 13-560

Figure A-8



Total depth: 27.02 ft, Date: 7/3/2013

Surface Elevation: 8.90 ft

Cone Operator: John Sarmiento & Associates

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

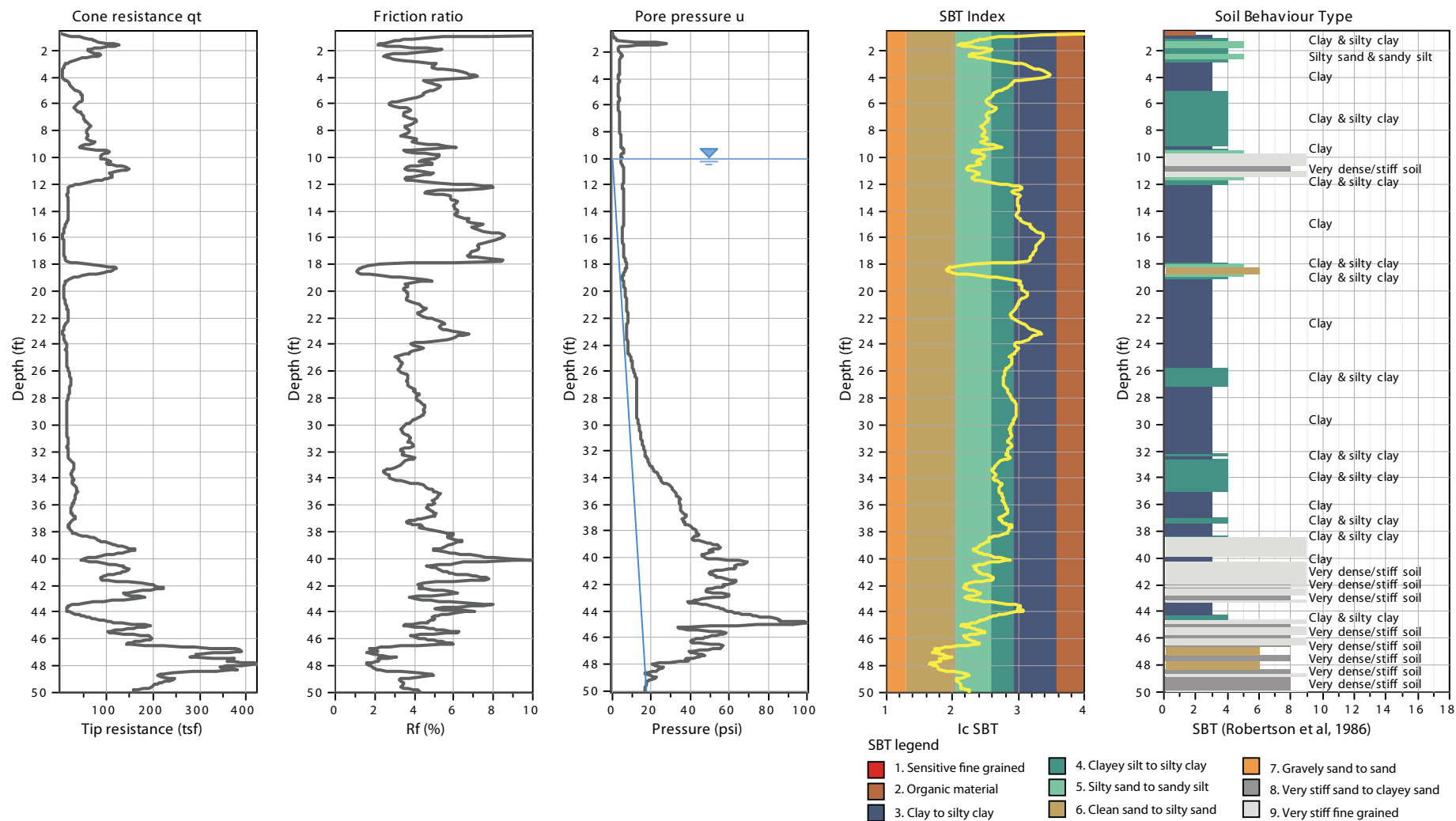


## CONE PENETRATION TEST RESULTS CPT-4

Date 12/06/13

Project No. 13-560

Figure A-9



Total depth: 50.04 ft, Date: 11/19/2013

Surface Elevation: 10.10 ft

Cone Operator: John Sarmiento & Associates

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

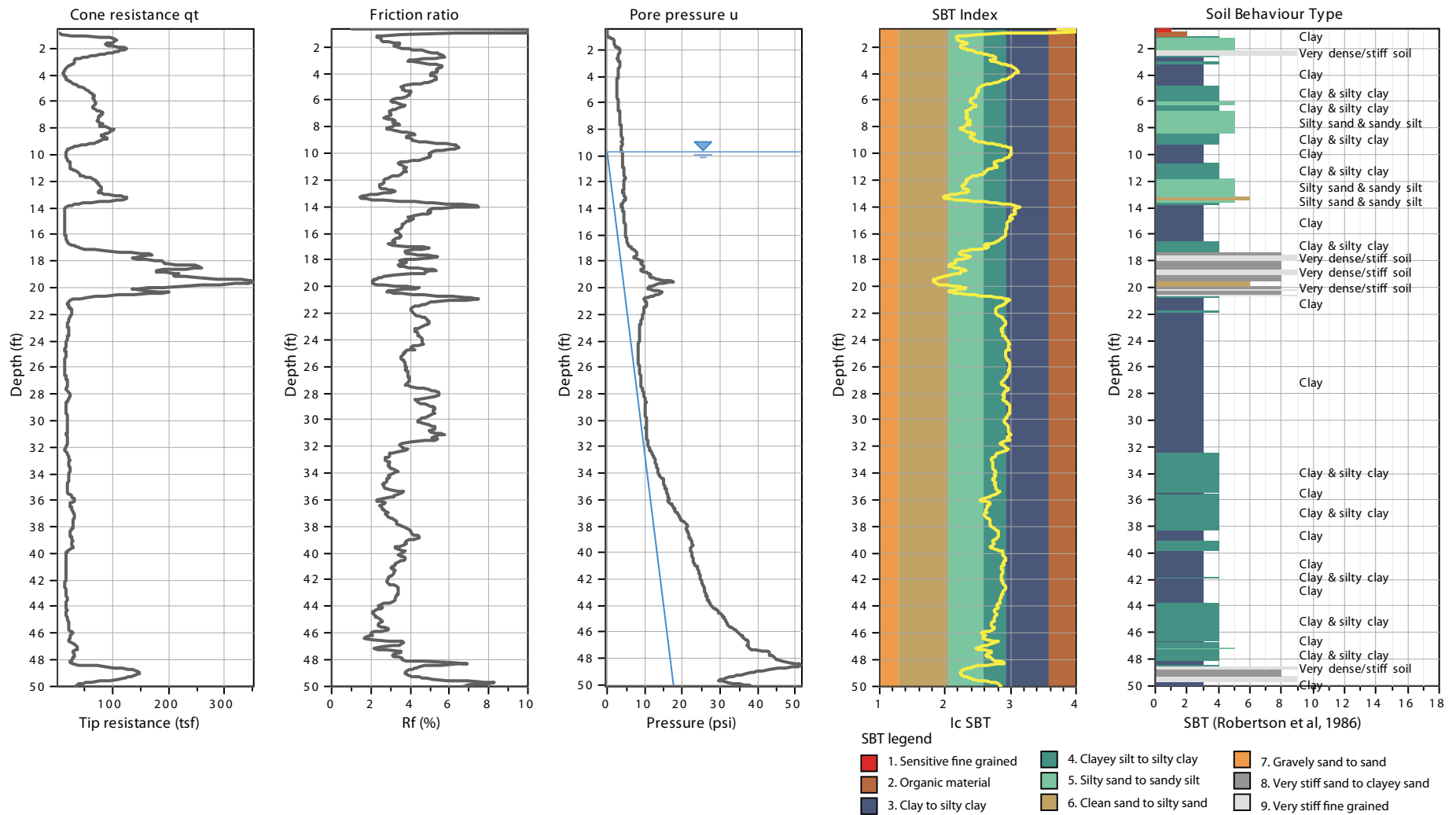
**ROCKRIDGE**  
GEOTECHNICAL

## CONE PENETRATION TEST RESULTS CPT-5

Date 12/06/13

Project No. 13-560

Figure A-10



Total depth: 50.02 ft, Date: 11/19/2013

Surface Elevation: 11.00 ft

Cone Operator: John Sarmiento & Associates

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

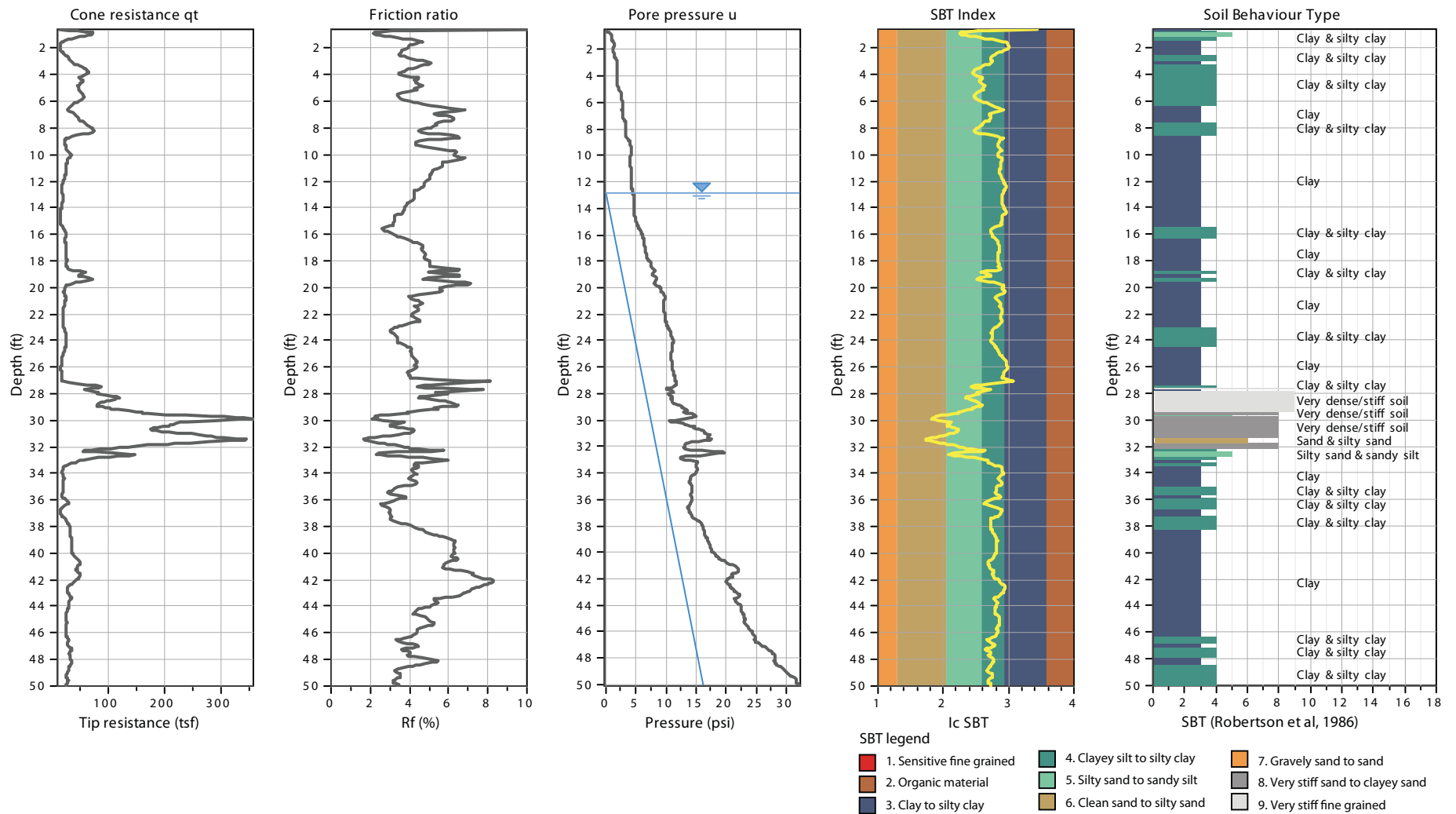
**ROCKRIDGE**  
GEOTECHNICAL

## CONE PENETRATION TEST RESULTS CPT-6

Date 12/06/13

Project No. 13-560

Figure A-11



Total depth: 50.01 ft, Date: 11/19/2013

Surface Elevation: 11.00 ft

Cone Operator: John Sarmiento & Associates

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

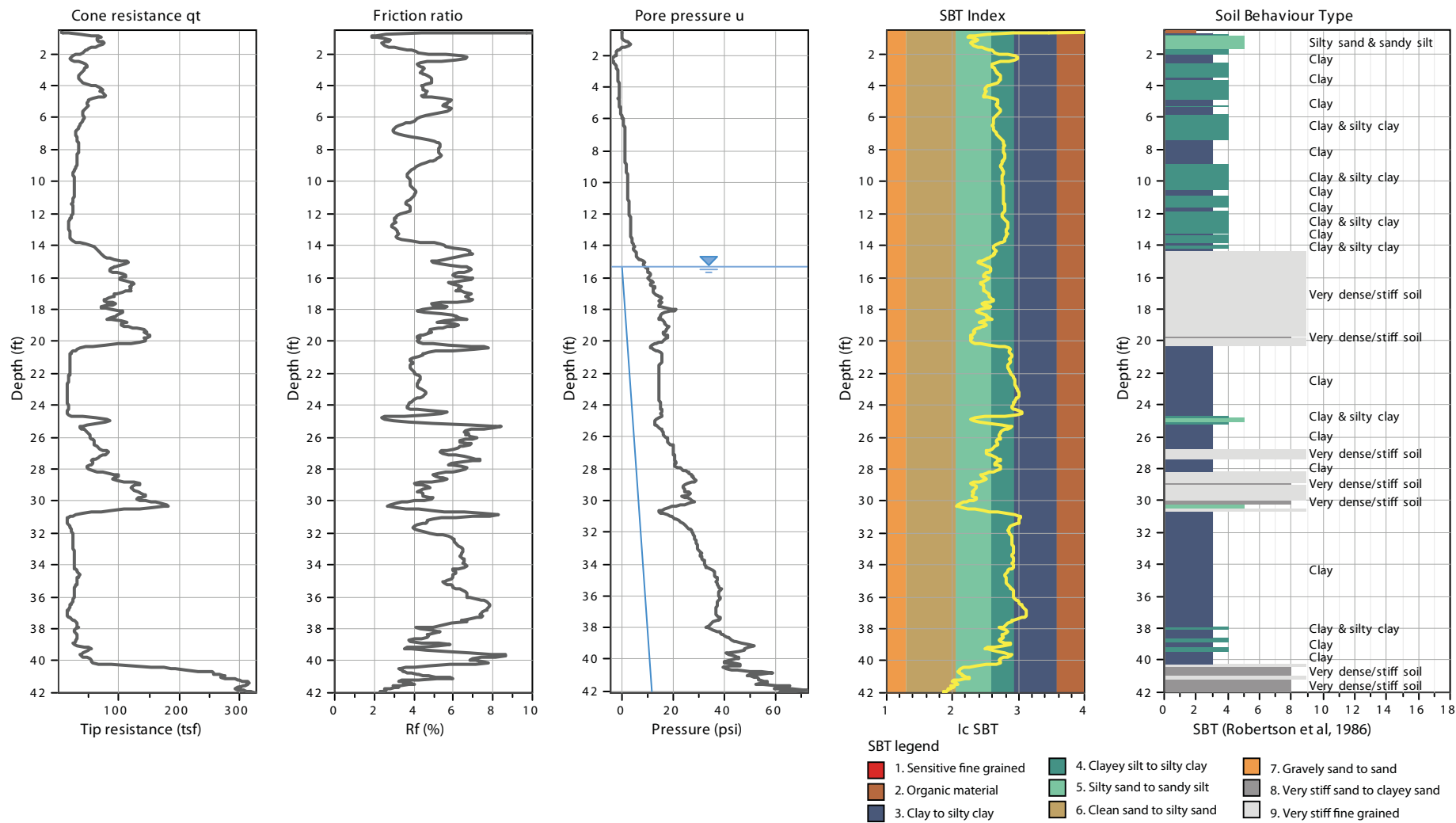
**ROCKRIDGE**  
GEOTECHNICAL

## CONE PENETRATION TEST RESULTS CPT-7

Date 12/06/13

Project No. 13-560

Figure A-12



Total depth: 42.02 ft, Date: 11/19/2013

Surface Elevation: 10.40 ft

Cone Operator: John Sarmiento & Associates

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California



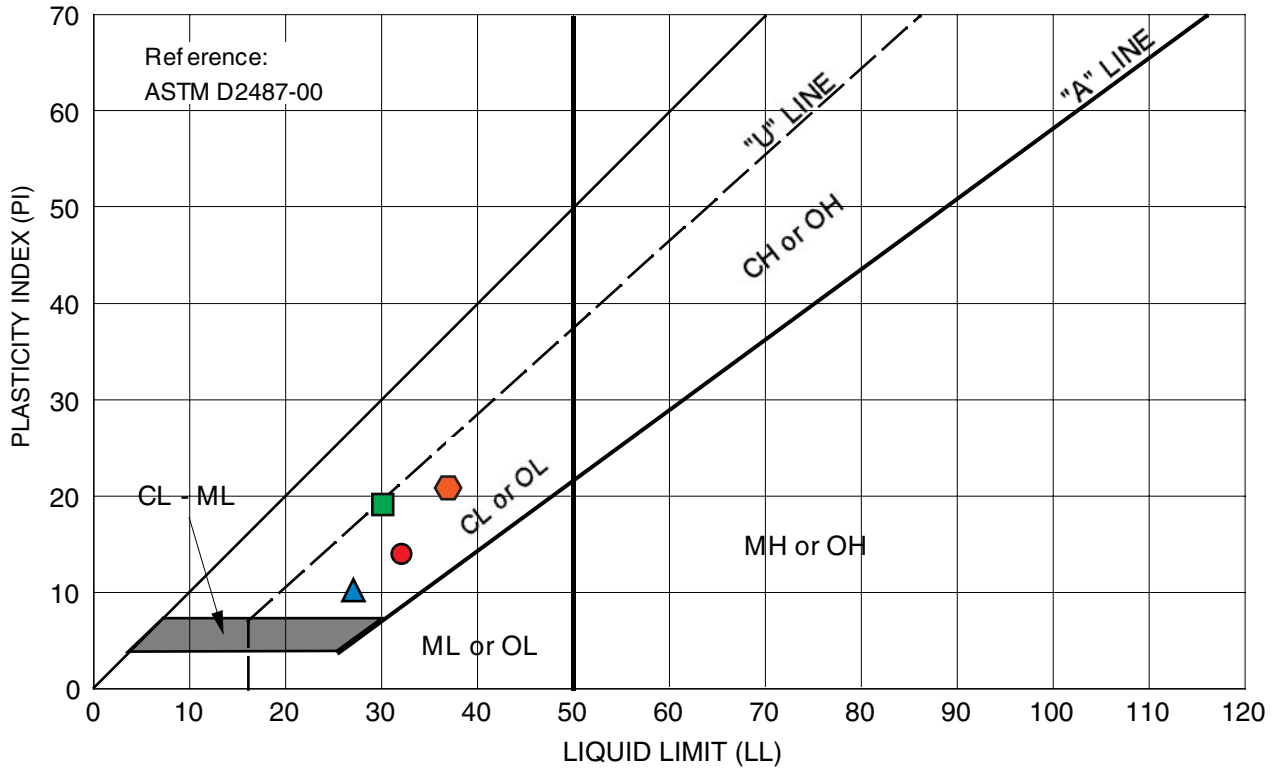
## CONE PENETRATION TEST RESULTS CPT-8

Date 12/06/13

Project No. 13-560

Figure A-13

**APPENDIX B**  
**Laboratory Test Results**



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-2 at 4.5 feet	SANDY CLAY with GRAVEL (CL), yellow-brown	20.3	32	14	--
▲	B-2 at 10.5 feet	CLAYEY SAND (SC), gray	18.9	27	10	49
■	B-4 at 6.0 feet	CLAY (CL), dark gray	20.8	30	19	--
⬡	B-4 at 15.0 feet	CLAYEY SAND (SC), yellow-brown	21.9	37	21	22

1008, 1016, & 1028 CAROLAN AVENUE/  
935 ROLLINS ROAD  
Burlingame, California

**ROCKRIDGE**  
GEOTECHNICAL

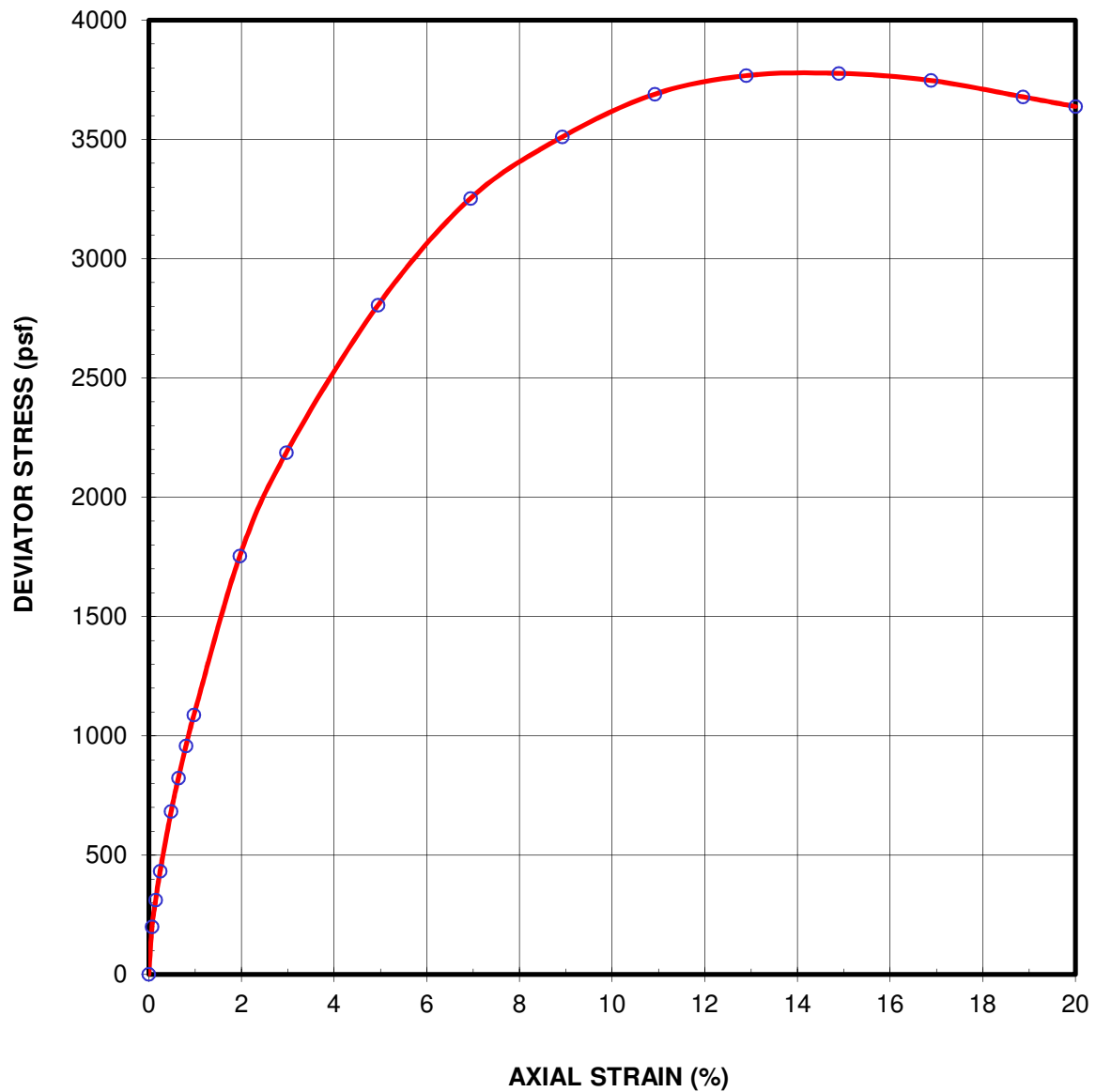
## PLASTICITY CHART


Date 12/20/13

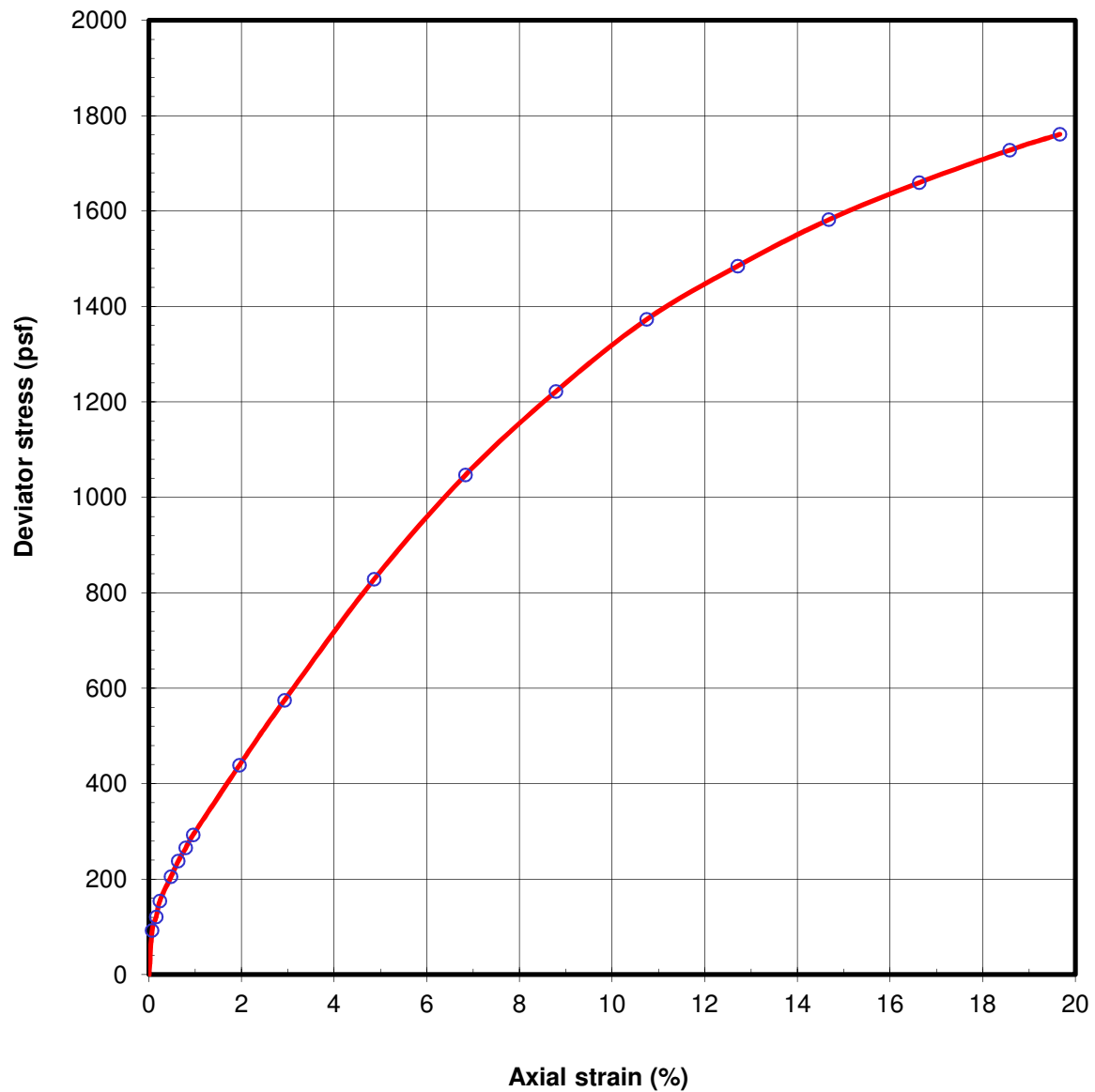
Project No. 13-560


Figure B-1

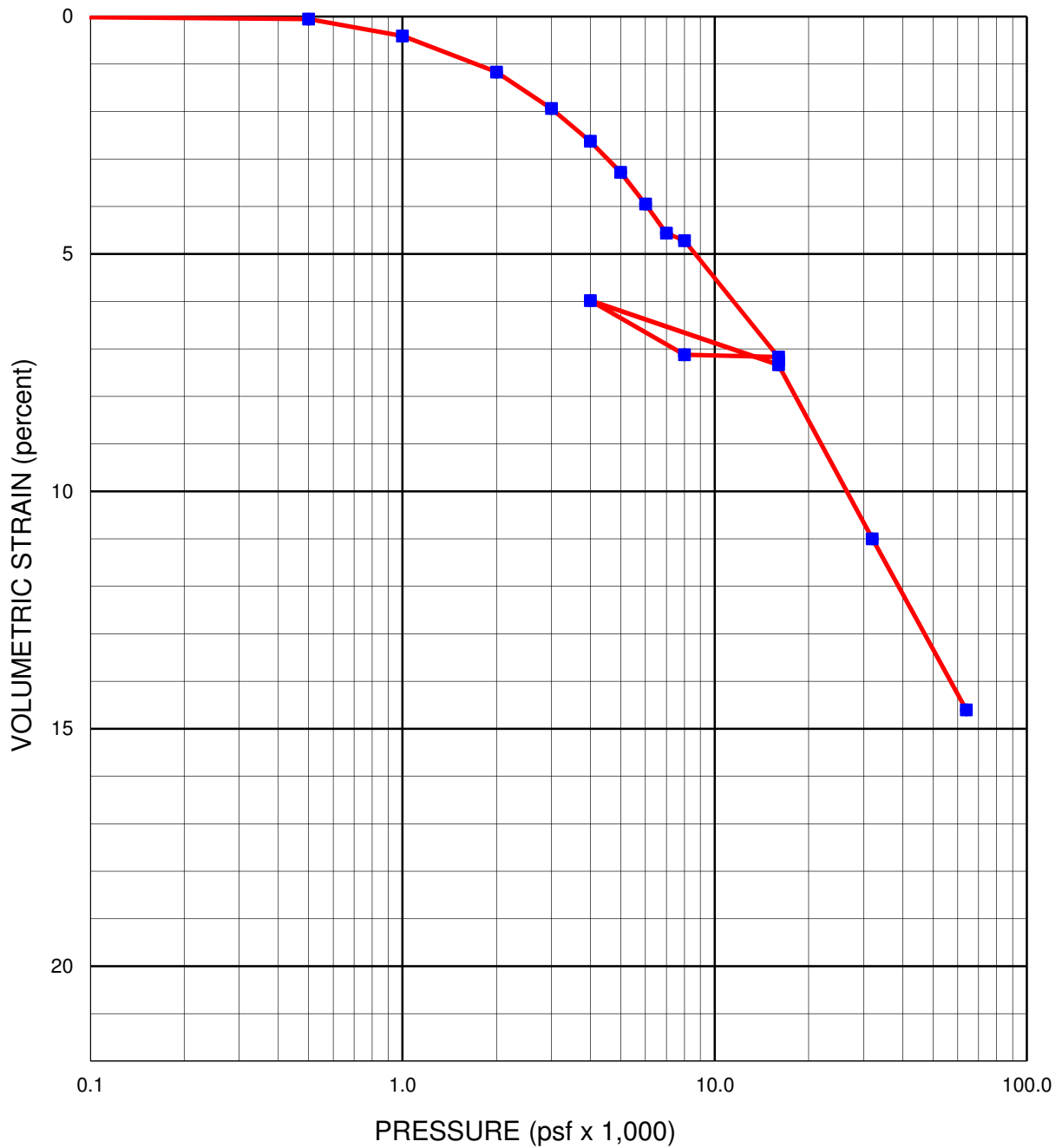





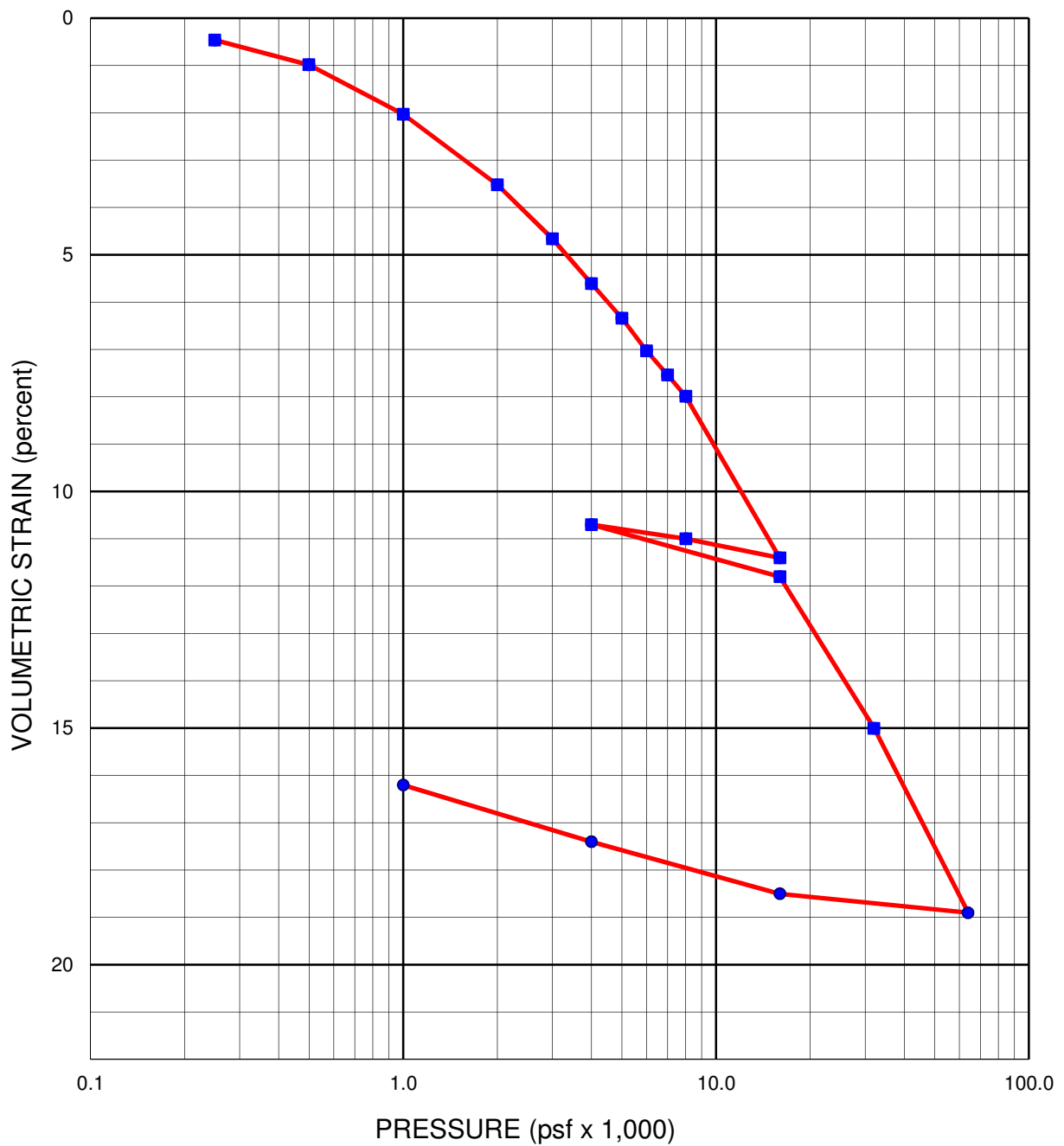
Sampler Type: Sprague & Henwood		Shear Strength: 1900 psf	
Diameter (in): 2.41	Height (in): 5.63	Strain at Failure: 15 %	
Moisture Content: 24.3 %		Confining Pressure: 800 psf	
Dry Density: 102 pcf		Strain Rate: 1%/min	
Description: CLAY with SAND (CL), yellow-brown		Source: B-3 at 13.0 feet	
<b>1008, 1016, &amp; 1028 CAROLAN AVENUE /</b> <b>935 ROLLINS ROAD</b> Burlingame, California		<b>UNCONSOLIDATED-UNDRAINED</b> <b>TRIAXIAL COMPRESSION TEST</b>	
 <b>ROCKRIDGE</b> <b>GEOTECHNICAL</b>		Date: 01/03/14	Project No. 13-560
		Figure B-2	




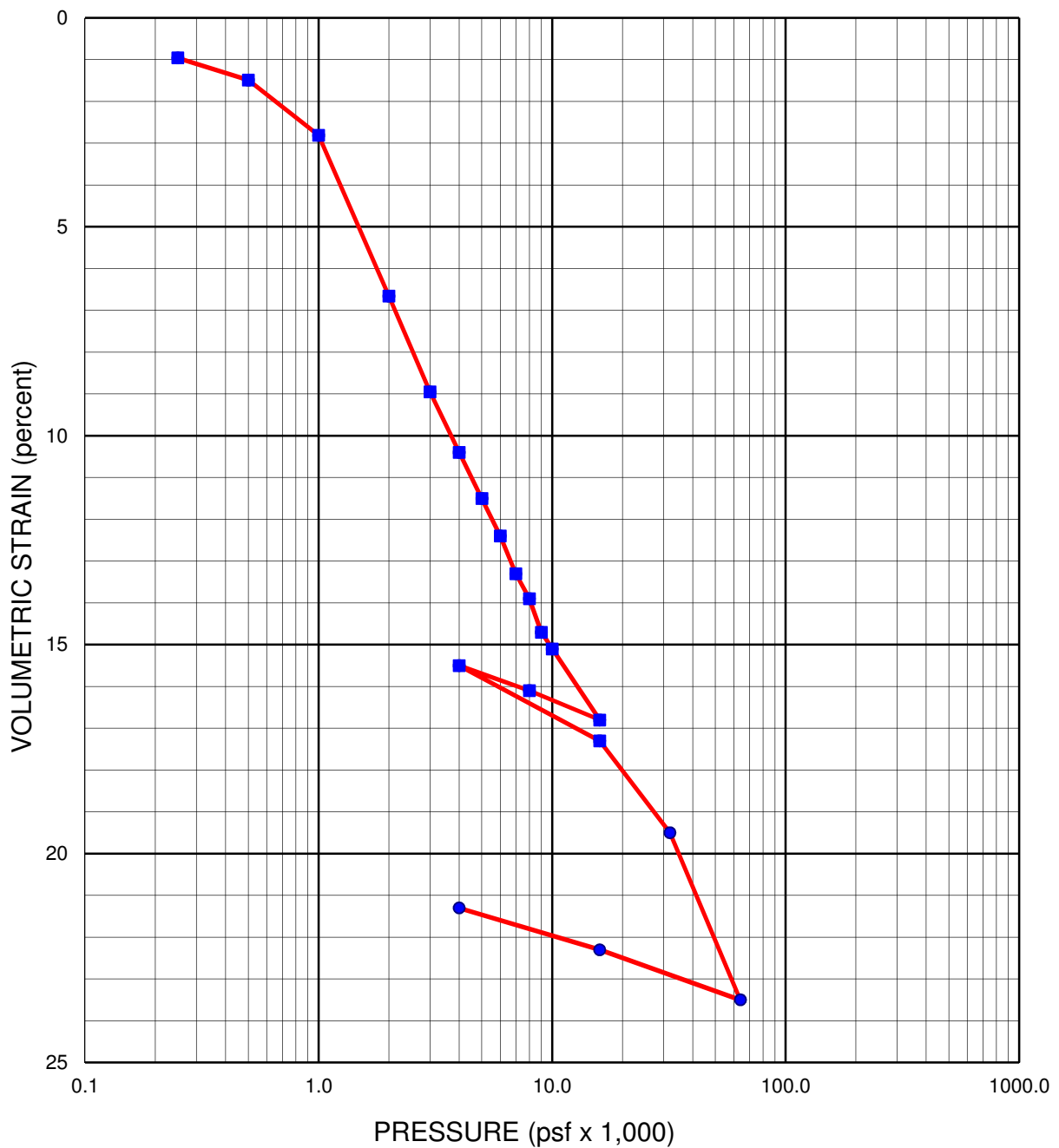
Sampler Type: Sprague & Henwood		Shear Strength: 880 psf	
Diameter (in): 2.41	Height (in): 5.35	Strain at Failure: 20 %	
Moisture Content: 25.9 %		Confining Pressure: 400 psf	
Dry Density: 100 pcf		Strain Rate: 1%/min	
Description: CLAY (CL), dark gray		Source: B-4 at 5.0 feet	
<b>1008, 1016, &amp; 1028 CAROLAN AVENUE /</b> <b>935 ROLLINS ROAD</b> Burlingame, California		<b>UNCONSOLIDATED-UNDRAINED</b> <b>TRIAxIAL COMPRESSION TEST</b>	
 <b>ROCKRIDGE</b> <b>GEOTECHNICAL</b>		Date: 01/03/14	Project No. 13-560
		Figure B-3	




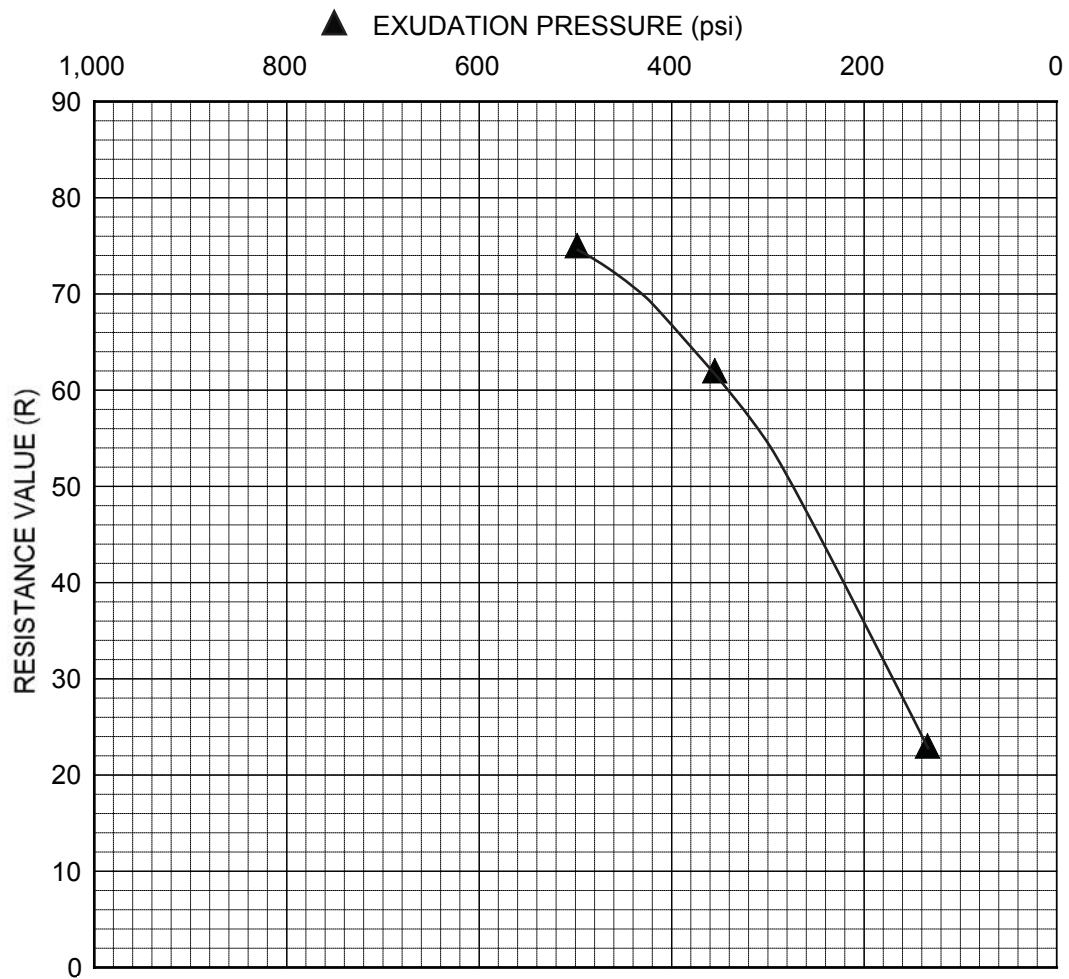
Sampler Type: Sprague & Henwood				Condition	Before Test		After Test	
Diameter (in)	2.00	Height (in)	0.75	Water Content	w <sub>o</sub>	25.5 %	w <sub>f</sub>	20.0 %
Overburden Pressure, p <sub>o</sub>	1,800	psf		Void Ratio	e <sub>o</sub>	0.71	e <sub>f</sub>	0.55
Preconsol. Pressure, p <sub>c</sub>	5,500	psf		Saturation	S <sub>o</sub>	98.0 %	S <sub>f</sub>	100 %
Compression Ratio, C <sub>ec</sub>	0.13			Dry Density	γ <sub>d</sub>	100 pcf	γ <sub>d</sub>	111 pcf
Recompression Ratio, C <sub>er</sub>		0.01	LL	PL	PI	G <sub>s</sub>	2.70	(assumed)
Description: CLAY with SAND (CL), yellow-brown				Source: B-1 at 21.0 feet				
1008, 1016, & 1028 CAROLAN AVENUE / 935 ROLLINS ROAD Burlingame, California				CONSOLIDATION TEST REPORT				
 ROCKRIDGE GEOTECHNICAL				Date	01/03/14	Project No.	13-560	Figure B-4



Sampler Type: Sprague & Henwood				Condition	Before Test		After Test	
Diameter (in)	2.00	Height (in)	0.75	Water Content	$w_o$	26.5 %	$w_f$	18.5 %
Overburden Pressure, $p_o$	2,900	psf		Void Ratio	$e_o$	0.80	$e_f$	0.51
Preconsol. Pressure, $p_c$	3,400	psf		Saturation	$S_o$	91 %	$S_f$	100 %
Compression Ratio, $C_{ec}$	0.14			Dry Density	$\gamma_d$	95 pcf	$\gamma_d$	114 pcf
Recompression Ratio, $C_{er}$	0.01	LL	--	PL	--	PI	--	$G_s$ 2.75 (assumed)
Description: CLAY (CL), yellow-brown				Source: B-3 at 36.0 feet				
1008, 1016, & 1028 CAROLAN AVENUE / 935 ROLLINS ROAD Burlingame, California				CONSOLIDATION TEST REPORT				
 ROCKRIDGE GEOTECHNICAL				Date	01/03/14	Project No.	13-560	Figure B-5




Sampler Type: Sprague & Henwood			Condition		Before Test			After Test		
Diameter (in)	2.49	Height (in)	1.00	Water Content		w <sub>o</sub>	20.8 %	w <sub>f</sub>	13.8 %	
Overburden Pressure, p <sub>o</sub>		770	psf	Void Ratio		e <sub>o</sub>	0.70	e <sub>f</sub>	0.38	
Preconsol. Pressure, p <sub>c</sub>		800	psf	Saturation		S <sub>o</sub>	82 %	S <sub>f</sub>	100 %	
Compression Ratio, C <sub>ec</sub>			0.20	Dry Density		γ <sub>d</sub>	101 pcf	γ <sub>d</sub>	125 pcf	
Recompression Ratio, C <sub>er</sub>			0.02	LL 30	PL 11	PI 19	G <sub>s</sub> 2.75	(assumed)		
Description: CLAY ( CL), dark gray					Source: B-4 at 6.25 feet					
1008, 1016, & 1028 CAROLAN AVENUE / 935 ROLLINS ROAD Burlingame, California					CONSOLIDATION TEST REPORT					
					Date	01/03/14	Project No.	13-560	Figure B-6	



#### Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	250	119.9	10.0	0.00	108	2.53	134	23	23
2	350	133.7	9.0	0.00	47	2.49	355	62	62
3	350	139.6	8.5	0.00	31	2.49	498	75	75

Test Results	Material Description		
R-Value at 300 psi exudation pressure = 54	SILTY GRAVEL with SAND (GM), olive-gray		
	Sample Source: B-2 at 0 to 2 feet		
1008, 1016, & 1028 CAROLAN AVENUE/ 935 ROLLINS ROAD Burlingame, California	R-VALUE TEST REPORT		
 ROCKRIDGE GEOTECHNICAL			
	Date 12/20/13	Project No. 13-560	Figure B-7



## Sunland Analytical

11353 Pyrites Way, Suite 4  
Rancho Cordova, CA 95670  
(916) 852-8557

Reprint of Report  
Dated \_\_\_\_\_

Date Reported 12/18/2013  
Date Submitted 12/13/2013

To: Logan Medeiros  
Rockridge Geotechnical, Inc.  
270 Grand Ave  
Oakland, CA 94610

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

The reported analysis was requested for the following location:  
Location : 13-560-CAROLAN+ROLLN Site ID : B1-3A @ 7.5 FT.  
Thank you for your business.

\* For future reference to this analysis please use SUN # 66098-136829.

---

### EVALUATION FOR SOIL CORROSION

Soil pH	7.27	
Moisture	14.0	%
Minimum Resistivity	1.02	ohm-cm (x1000)
Chloride	216.2 ppm	00.02162 %
Sulfate	94.1 ppm	00.00941 %
Redox Potential	(+) 191	mv
Sulfate Reducing Bacteria Presence - NEGATIVE		

### METHODS

pH and Min. Resistivity CA DOT Test #643 Mod. (Sm. Cell)  
Sulfate CA DOT Test #417, Chloride CA DOT Test #422  
Redox Potential ASTM D1498m, Sulfate Reducing Bacteria AWWA C105-72



## Sunland Analytical

11353 Pyrites Way, Suite 4  
Rancho Cordova, CA 95670  
(916) 852-8557

**Reprint of Report**

**Dated** \_\_\_\_\_

**How Data Added**

Date Reported 12/20/2013  
Date Submitted 12/13/2013

To: Logan Medeiros  
Rockridge Geotechnical, Inc.  
270 Grand Ave  
Oakland, CA 94610

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

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Location : 13-560-CAROLAN+ROLLN Site ID : B1-3A @ 7.5 FT.  
Thank you for your business.

\* For future reference to this analysis please use SUN # 66098-136829.

### Extractable Sulfide Analysis

TYPE OF TEST	RESULTS	UNITS
Sulfide	ND	mg/kg

### DETECTION LIMITS

Sulfide 0.05

Method 9031m, ND = Below Detection Limits





## Sunland Analytical

11353 Pyrites Way, Suite 4  
Rancho Cordova, CA 95670  
(916) 852-8557

Date Reported 12/18/2013  
Date Submitted 12/13/2013

To: Logan Medeiros  
Rockridge Geotechnical, Inc.  
270 Grand Ave  
Oakland, CA 94610

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

The reported analysis was requested for the following location:  
Location : 13-560-CAROLAN+ROLLN Site ID : B4-1 @ 0-2 FT.

Thank you for your business.

\* For future reference to this analysis please use SUN # 66098-136830.

---

### EVALUATION FOR SOIL CORROSION

Soil pH	7.51		
Moisture	9.0	%	
Minimum Resistivity	1.88	ohm-cm (x1000)	
Chloride	17.0 ppm	00.00170	%
Sulfate	83.1 ppm	00.00831	%
Redox Potential	(+) 187	mv	

Sulfate Reducing Bacteria Presence - TRACE

### METHODS

pH and Min. Resistivity CA DOT Test #643 Mod. (Sm. Cell)  
Sulfate CA DOT Test #417, Chloride CA DOT Test #422  
Redox Potential ASTM D1498m, Sulfate Reducing Bacteria AWWA C105-72



## Sunland Analytical

11353 Pyrites Way, Suite 4  
Rancho Cordova, CA 95670  
(916) 852-8557

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To: Logan Medeiros  
Rockridge Geotechnical, Inc.  
270 Grand Ave  
Oakland, CA 94610

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

The reported analysis was requested for the following:  
Location : 13-560-CAROLAN+ROLLN Site ID : B4-1 @ 0-2 FT.

Thank you for your business. to this analysis please use SUN # 66098-136830.

### Extractable Sulfide Analysis .

TYPE OF TEST	RESULTS	UNITS
-----	-----	-----
Sulfide	0.27	mg/kg

### DETECTION LIMITS

Sulfide 0.05

Method 9031m, ND = Below Detection Limits

**APPENDIX C**  
**Summary of Liquefaction Analyses**

## LIQUEFACTION ANALYSIS REPORT

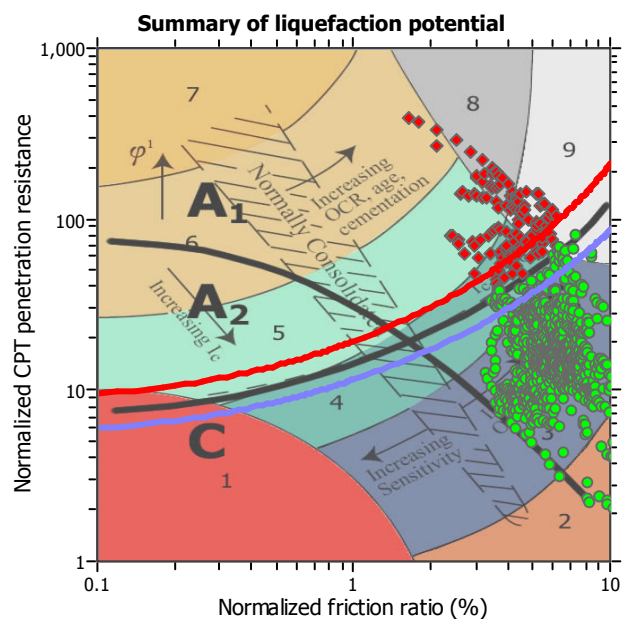
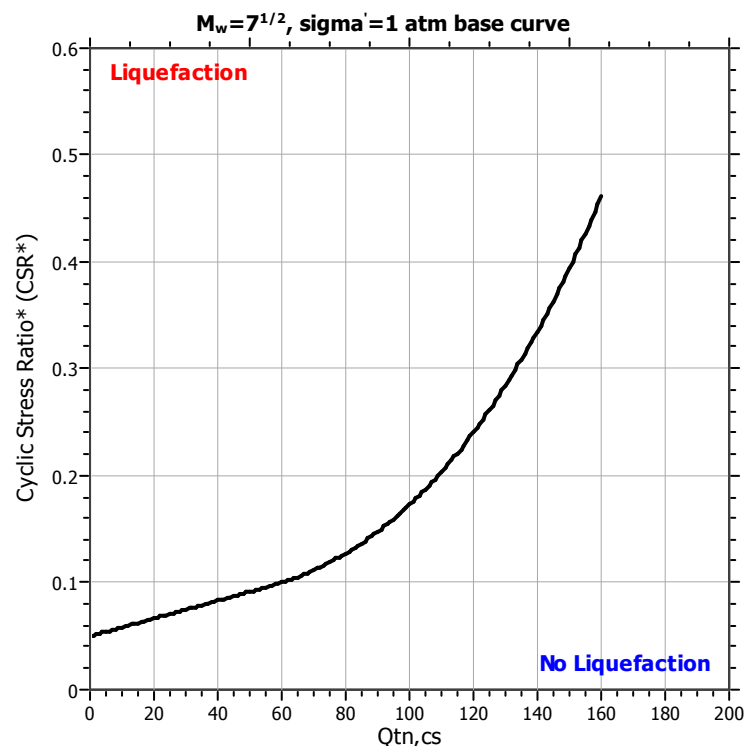
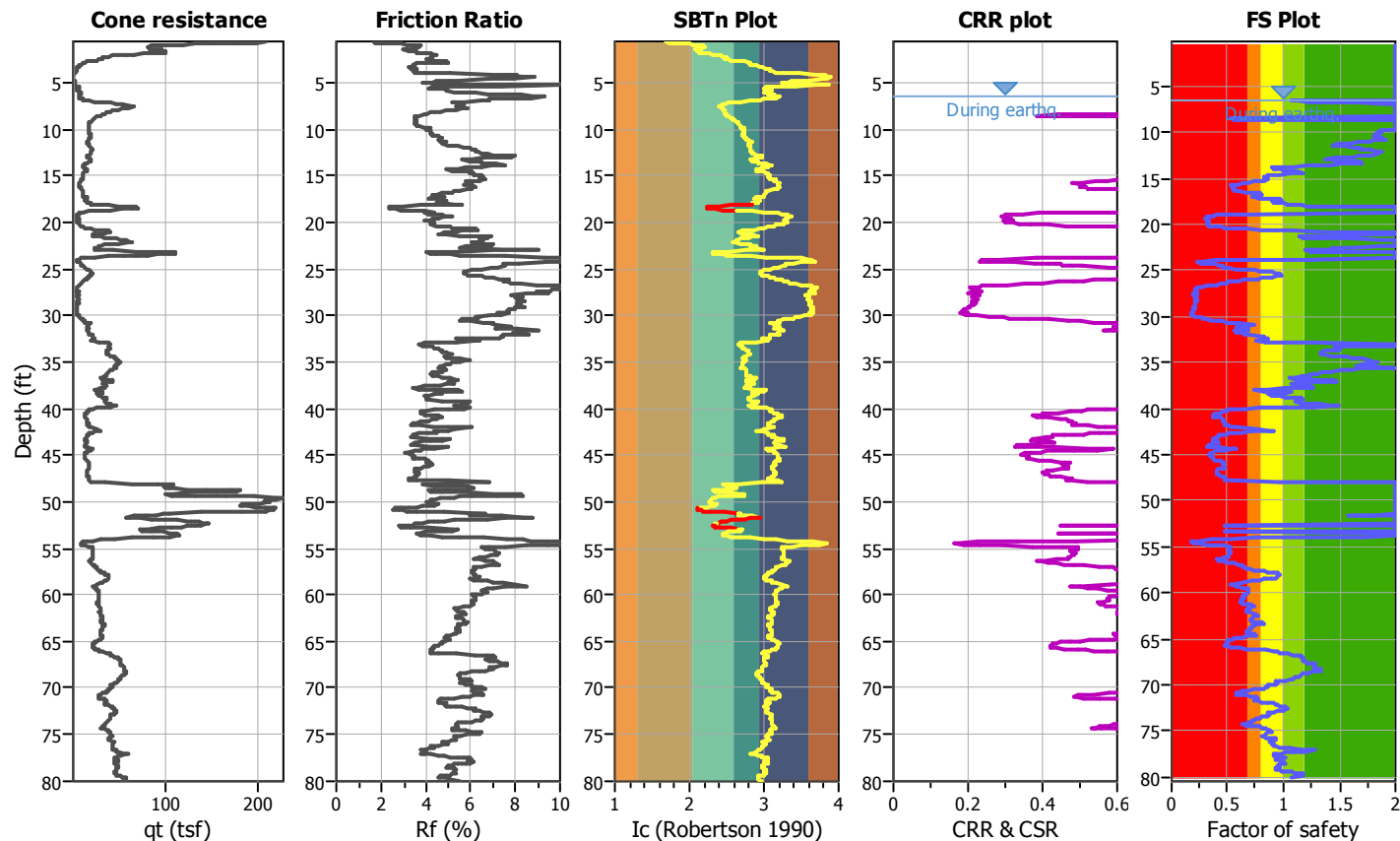
**Project title : 13-560 - Carolan Avenue and Rollins Road**

**Location : Burlingame, CA**

**CPT file : CPT-1**

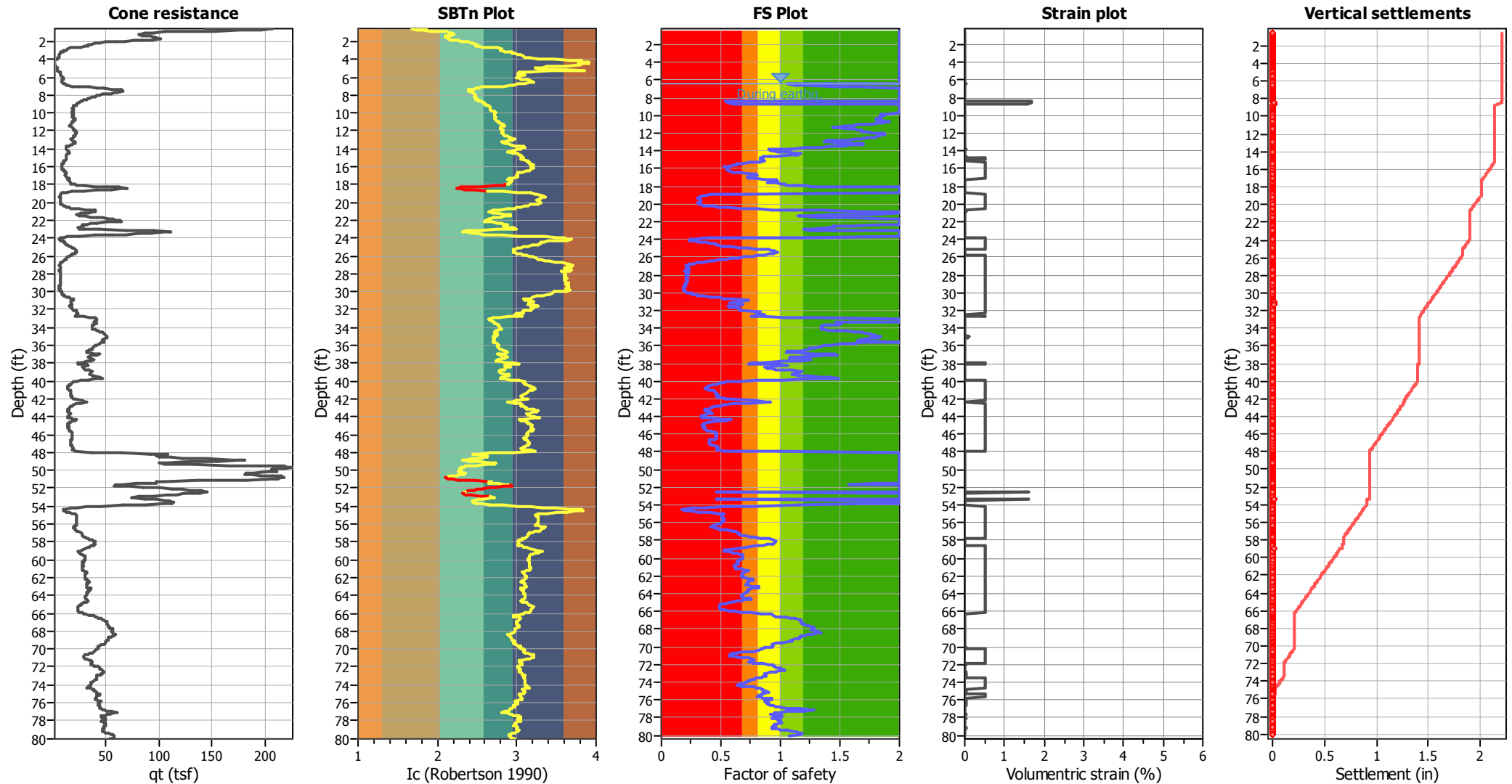
### Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	9.40 ft	Use fill:	No	Clay like behavior	
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	6.50 ft	Fill height:	N/A	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_0$ applied:	Yes	MSF method:	Method based



Zone A<sub>1</sub>: Cyclic liquefaction likely depending on size and duration of cyclic loading  
 Zone A<sub>2</sub>: Cyclic liquefaction and strength loss likely depending on loading and ground geometry  
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening  
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## Estimation of post-earthquake settlements



### Abbreviations

$q_c$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain

## LIQUEFACTION ANALYSIS REPORT

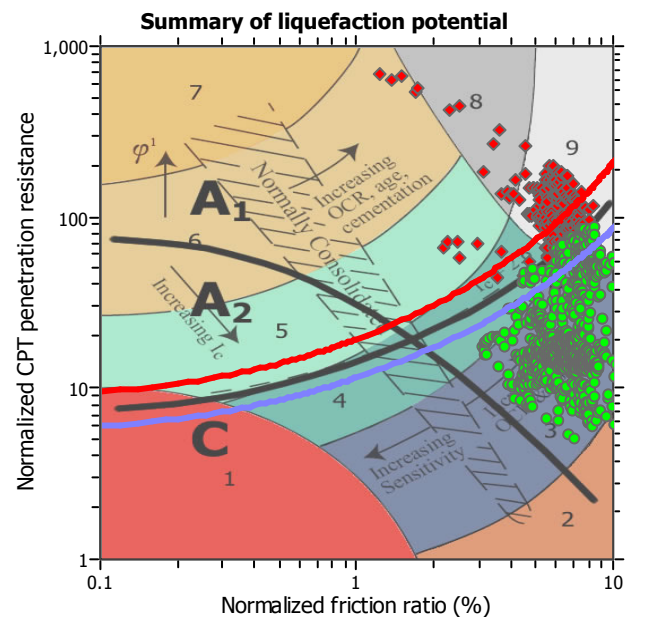
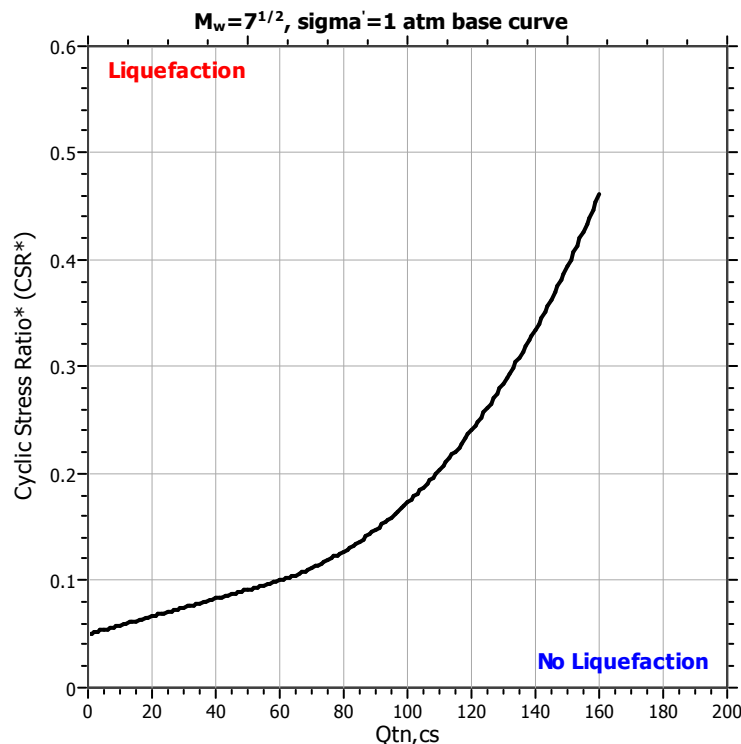
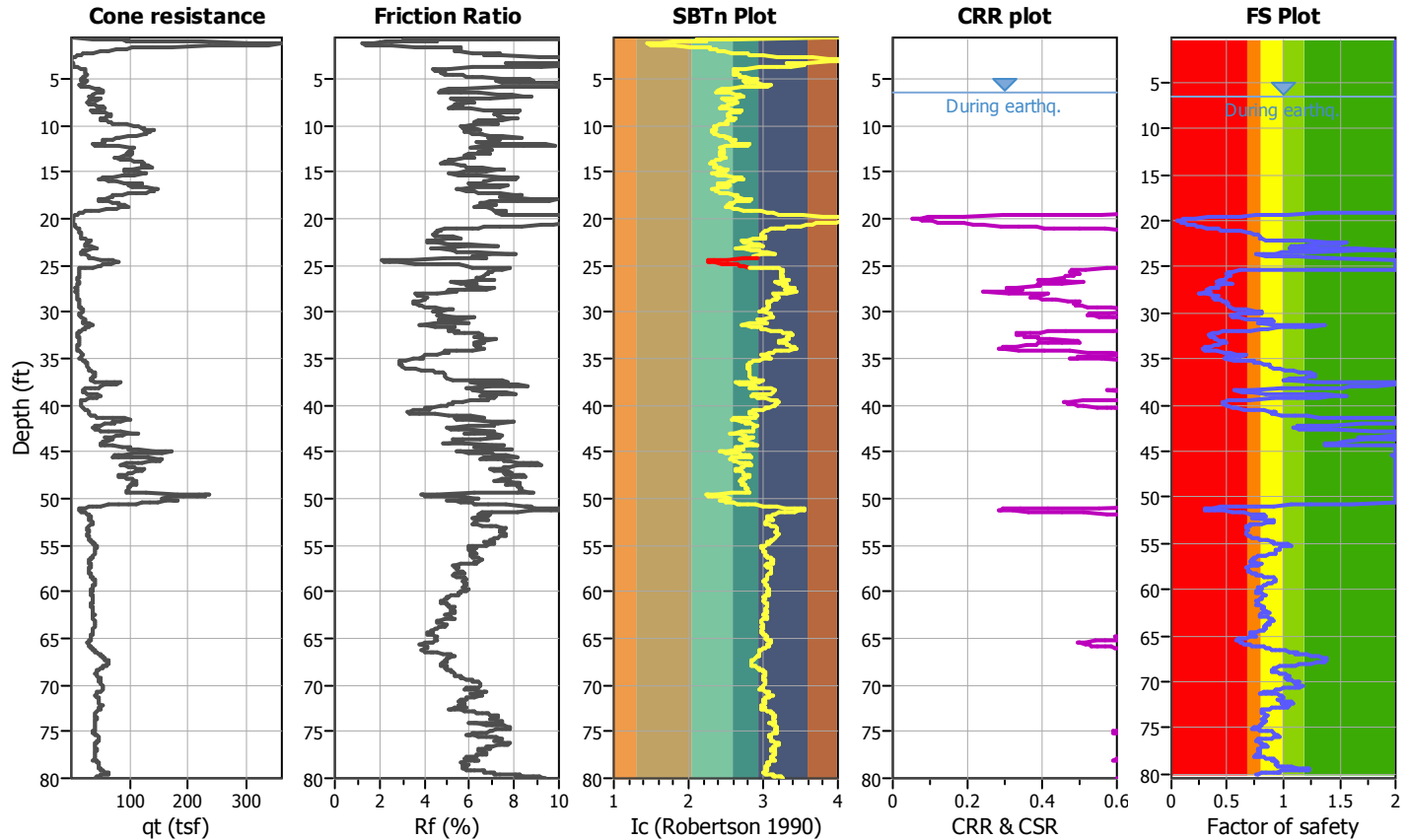
**Project title : 13-560 - Carolan Avenue and Rollins Road**

**Location : Burlingame, CA**

**CPT file : CPT-2**

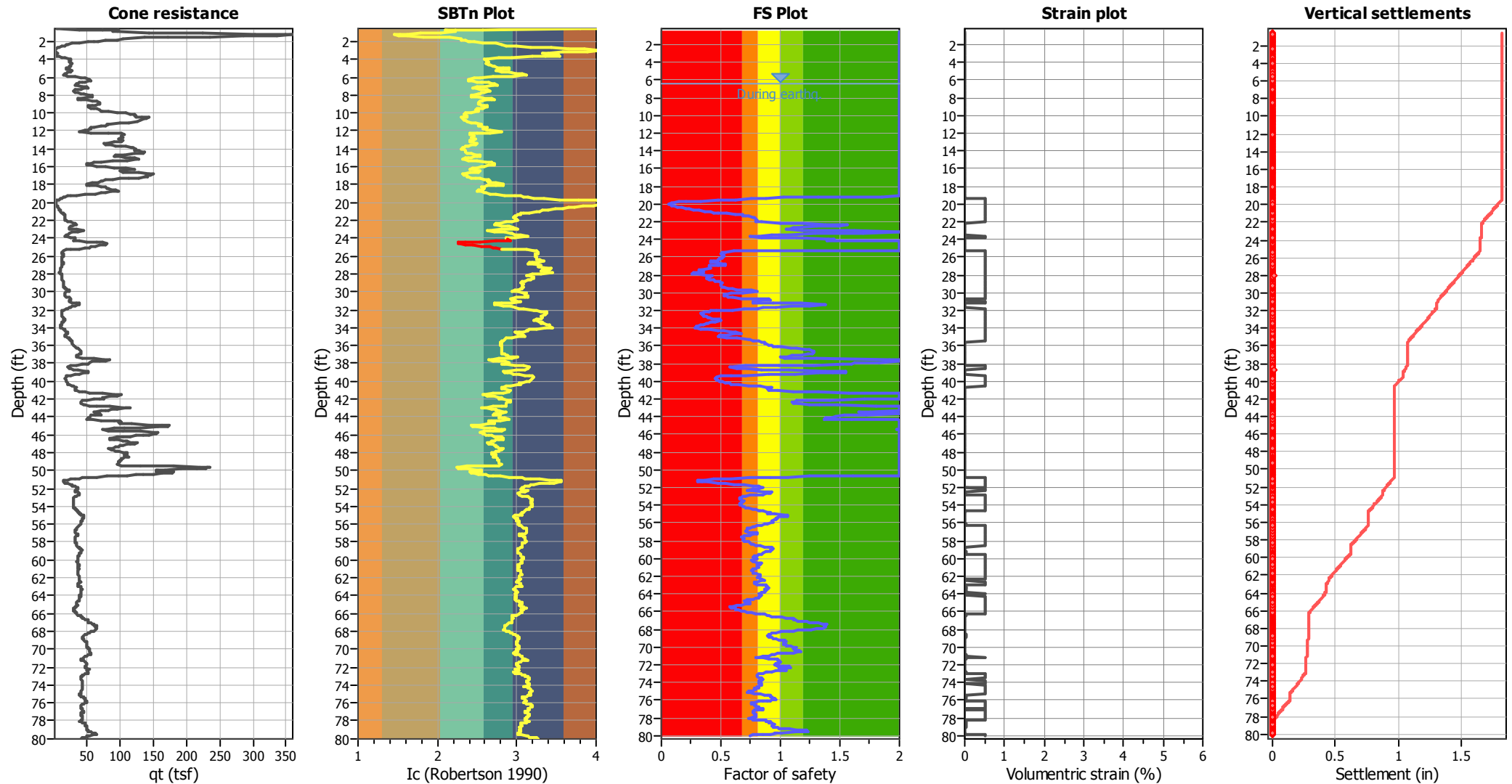
### Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	9.60 ft	Use fill:	No	Clay like behavior	
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	6.50 ft	Fill height:	N/A	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_g$ applied:	Yes	MSF method:	Method based



Zone A<sub>1</sub>: Cyclic liquefaction likely depending on size and duration of cyclic loading  
 Zone A<sub>2</sub>: Cyclic liquefaction and strength loss likely depending on loading and ground geometry  
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening  
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## Estimation of post-earthquake settlements



### Abbreviations

$q_t$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain



## LIQUEFACTION ANALYSIS REPORT

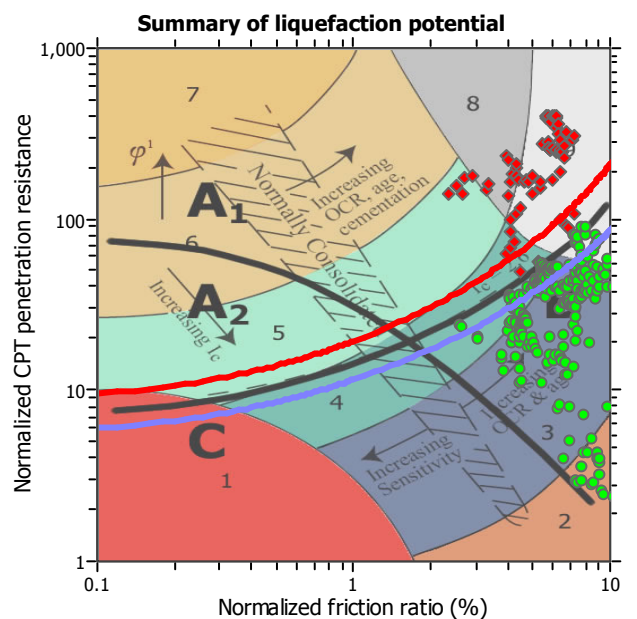
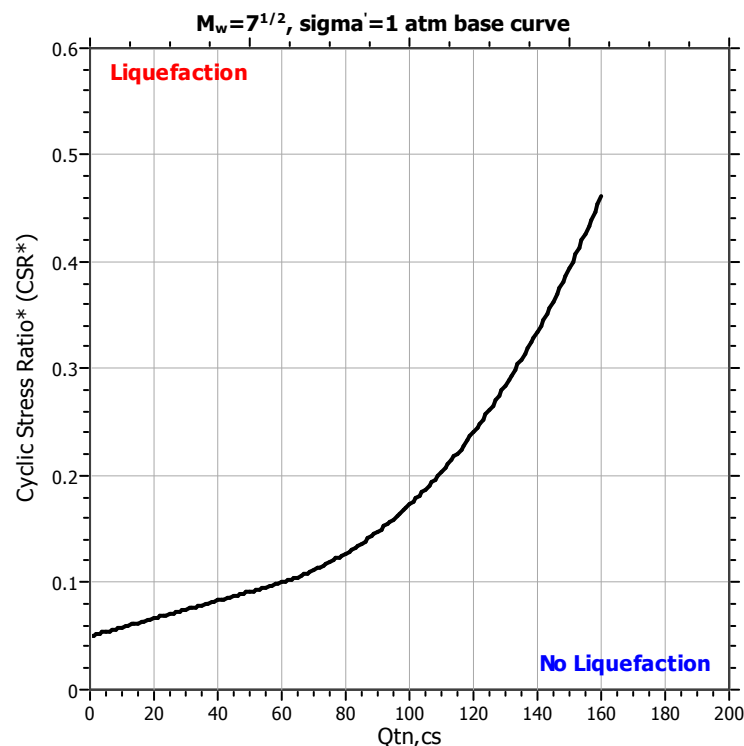
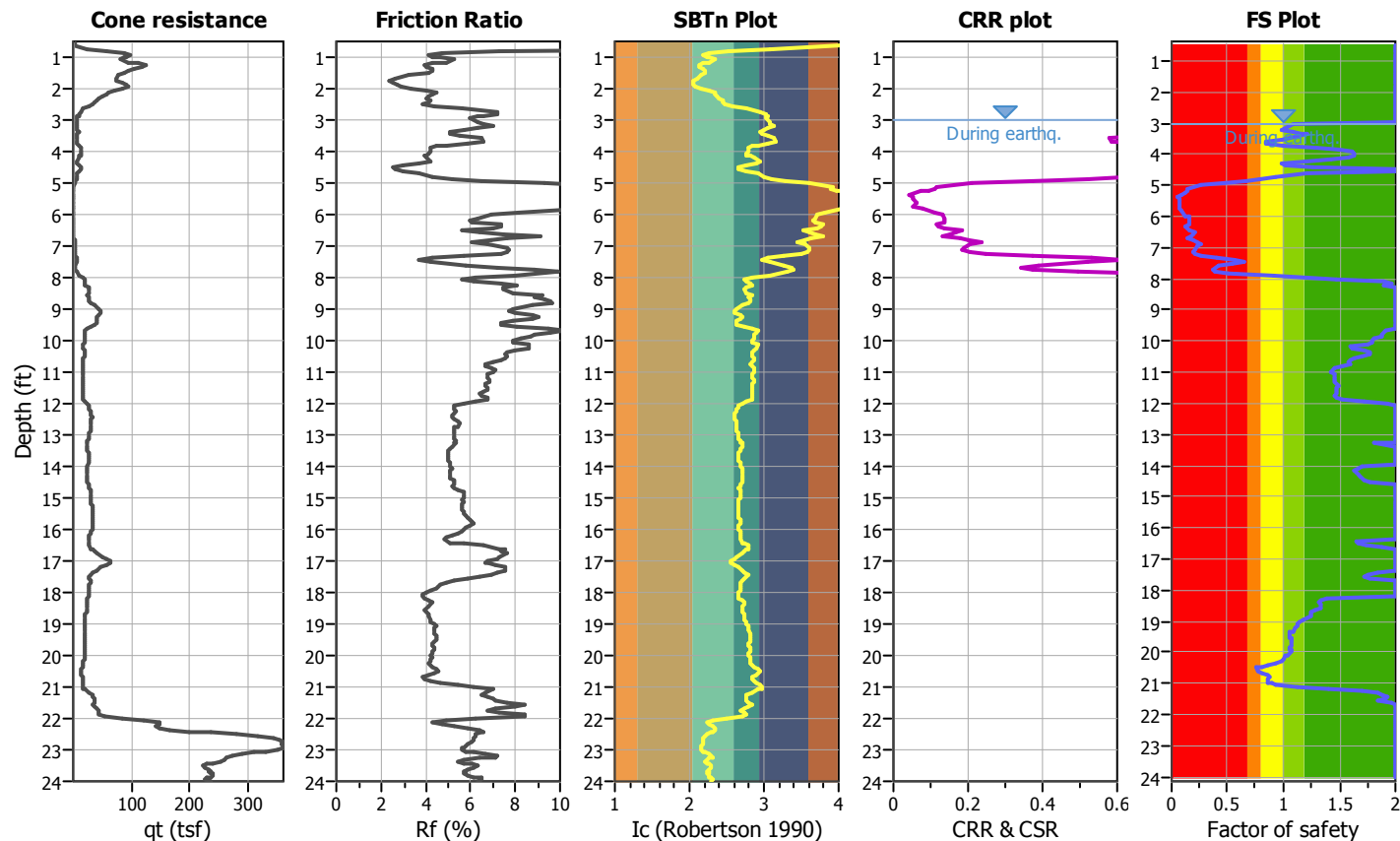
**Project title : 13-560 - Carolan Avenue and Rollins Road**

**Location : Burlingame, CA**

**CPT file : CPT-3**

### Input parameters and analysis data

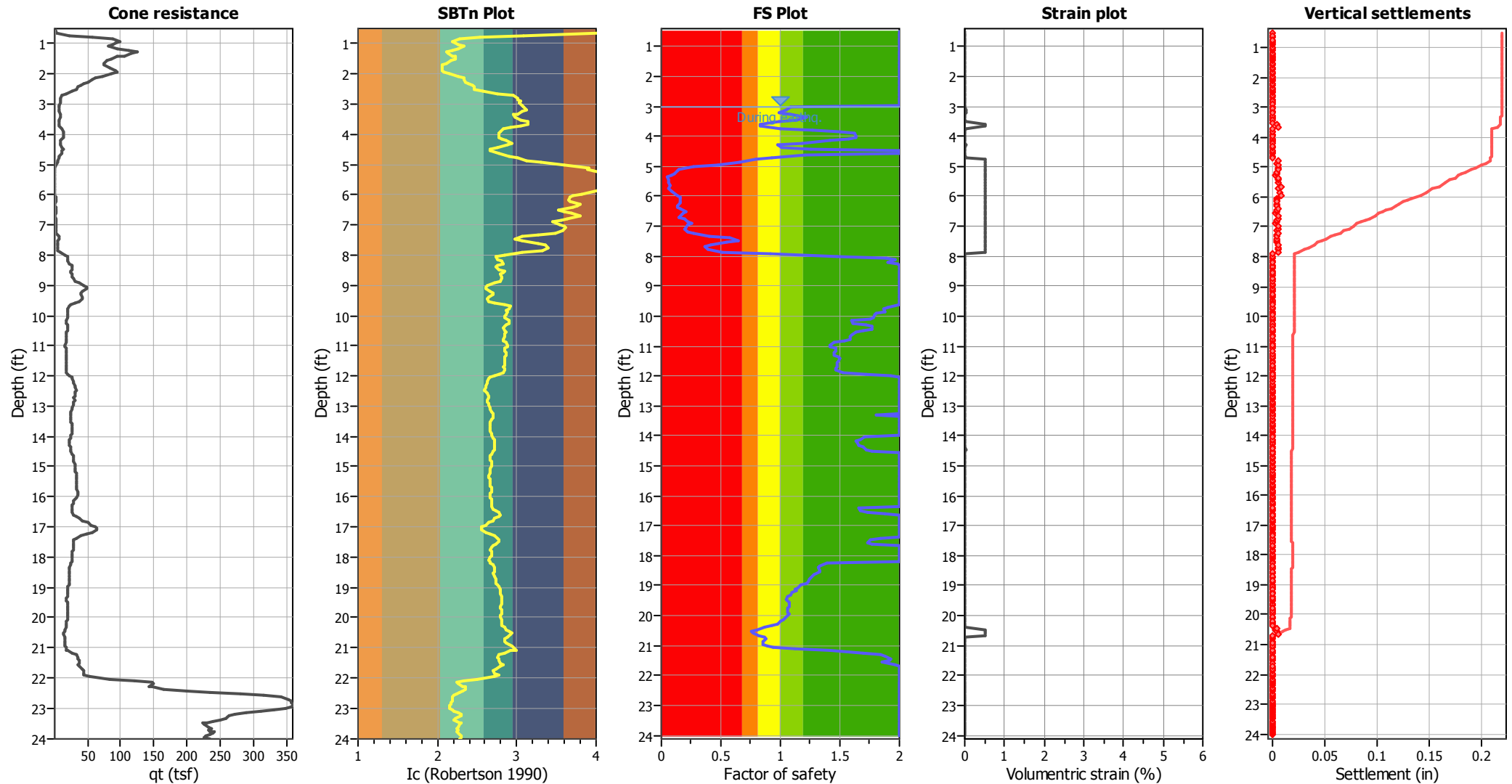
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Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_g$ applied:	Yes	MSF method:	Method based



Zone A<sub>1</sub>: Cyclic liquefaction likely depending on size and duration of cyclic loading  
 Zone A<sub>2</sub>: Cyclic liquefaction and strength loss likely depending on loading and ground geometry  
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening  
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry



## Estimation of post-earthquake settlements



### Abbreviations

$q_c$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain

## LIQUEFACTION ANALYSIS REPORT

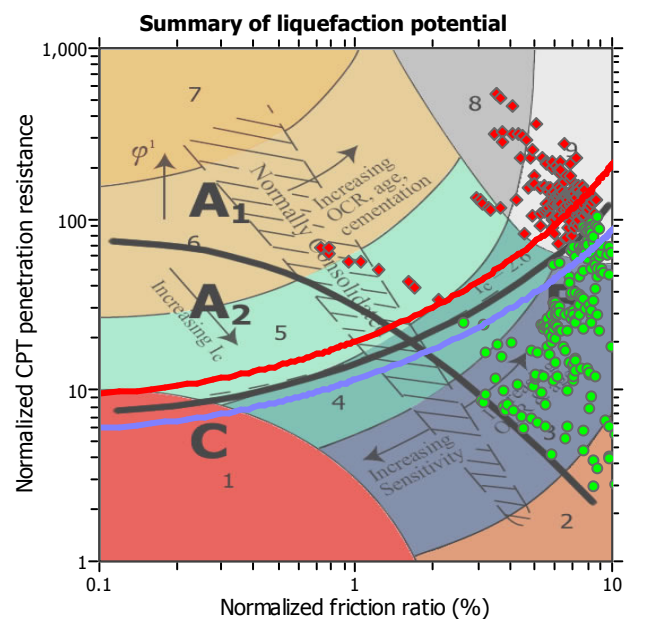
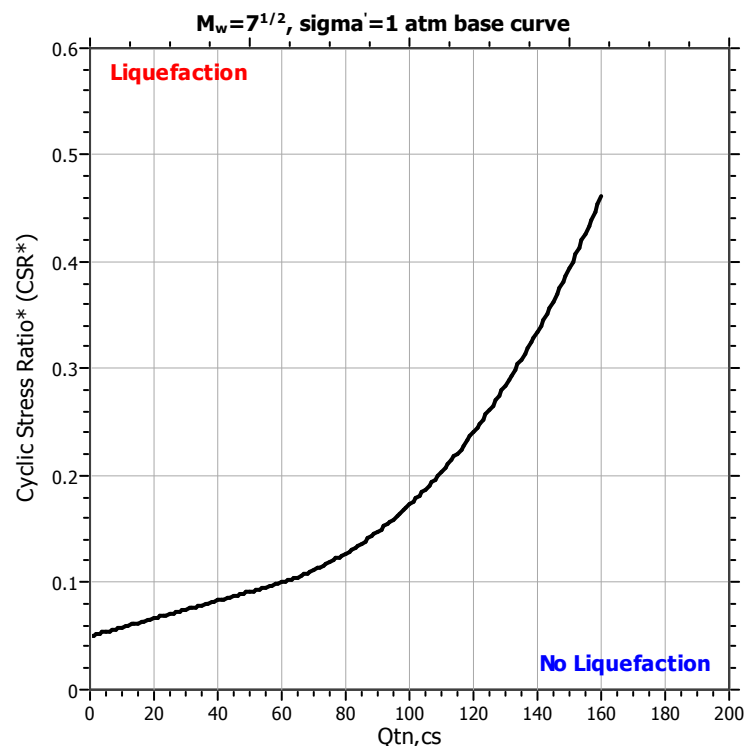
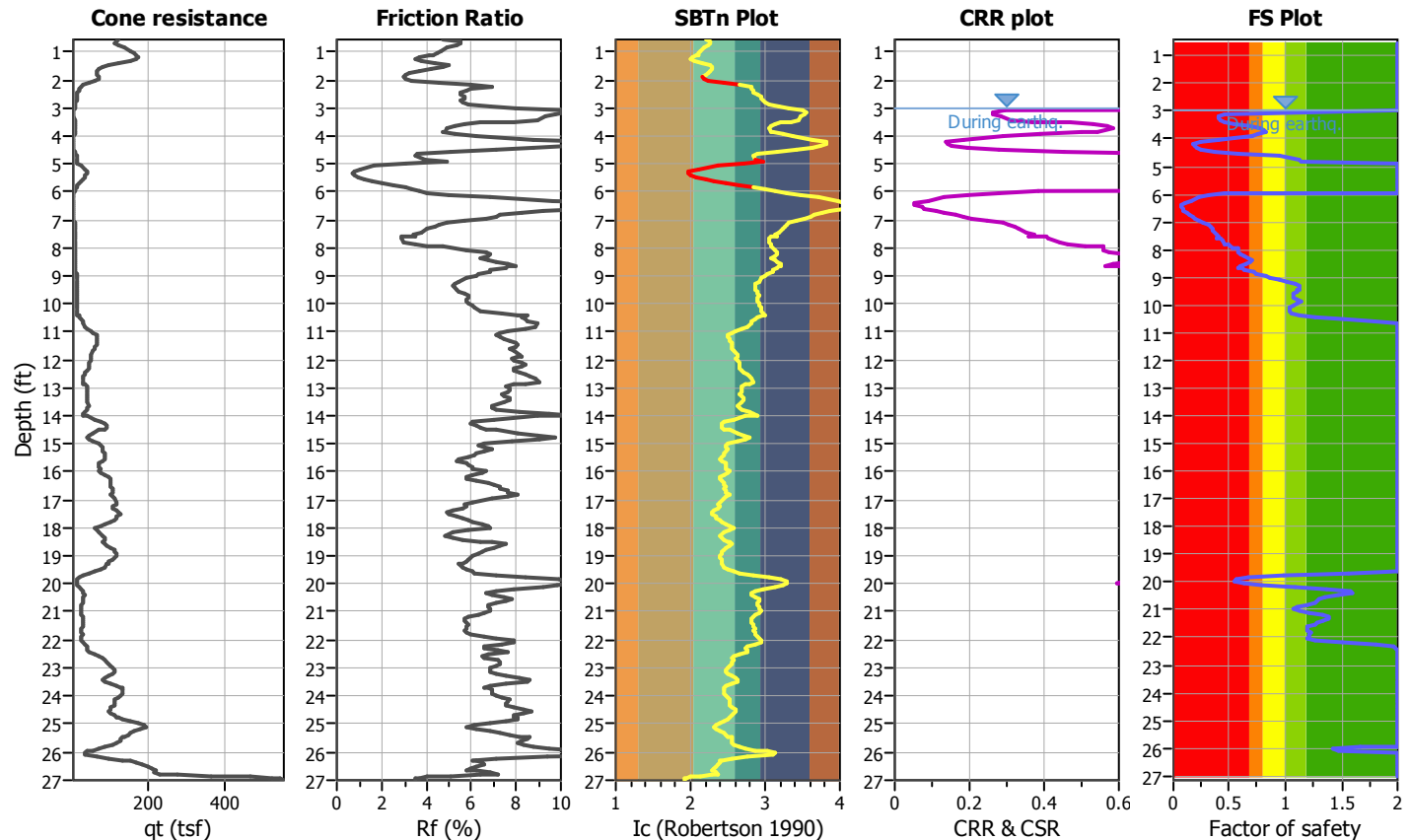
**Project title : 13-560 - Carolan Avenue and Rollins Road**

**Location : Burlingame, CA**

**CPT file : CPT-4**

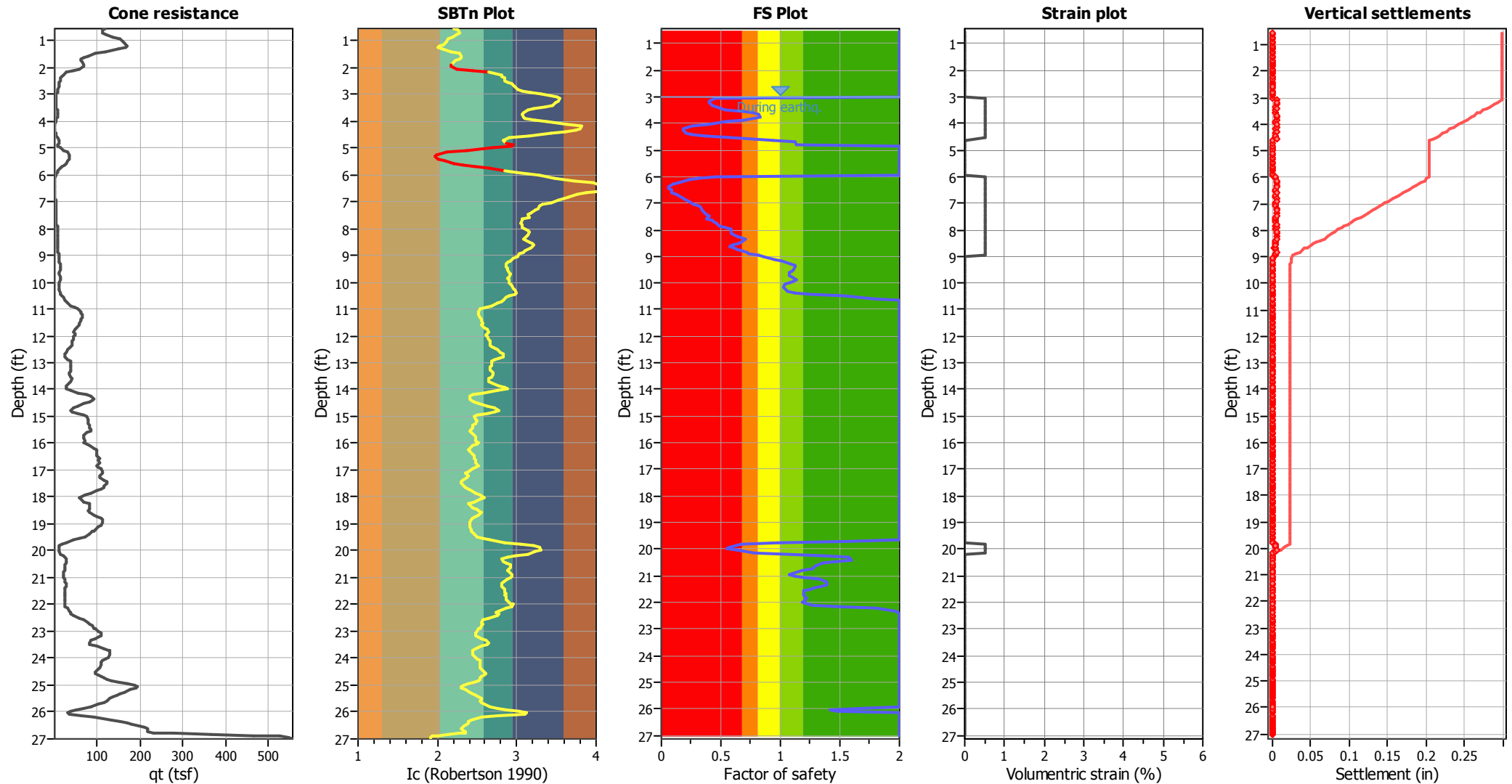
### Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	5.60 ft	Use fill:	No	Clay like behavior	
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_0$ applied:	Yes	MSF method:	Method based



Zone A<sub>1</sub>: Cyclic liquefaction likely depending on size and duration of cyclic loading  
 Zone A<sub>2</sub>: Cyclic liquefaction and strength loss likely depending on loading and ground geometry  
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening  
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## Estimation of post-earthquake settlements



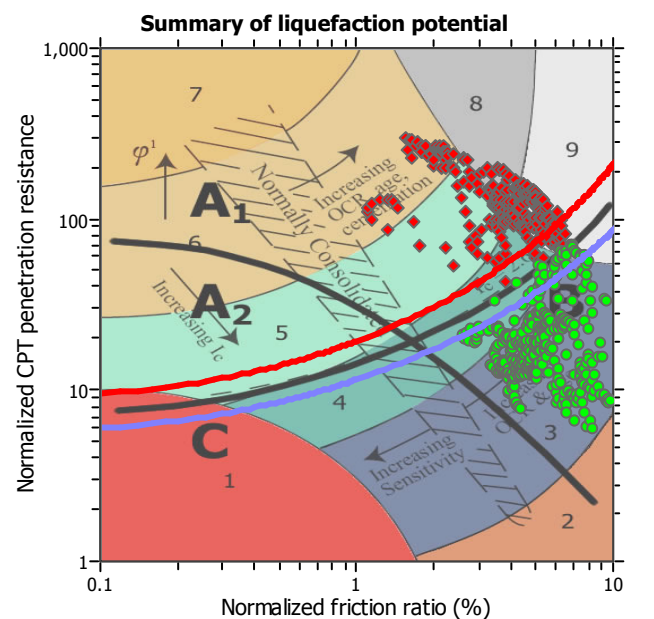
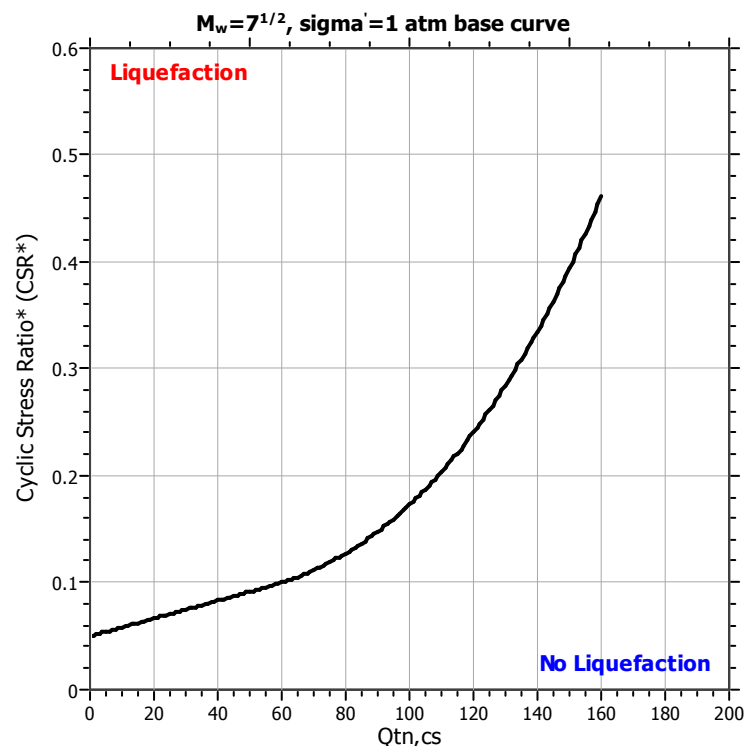
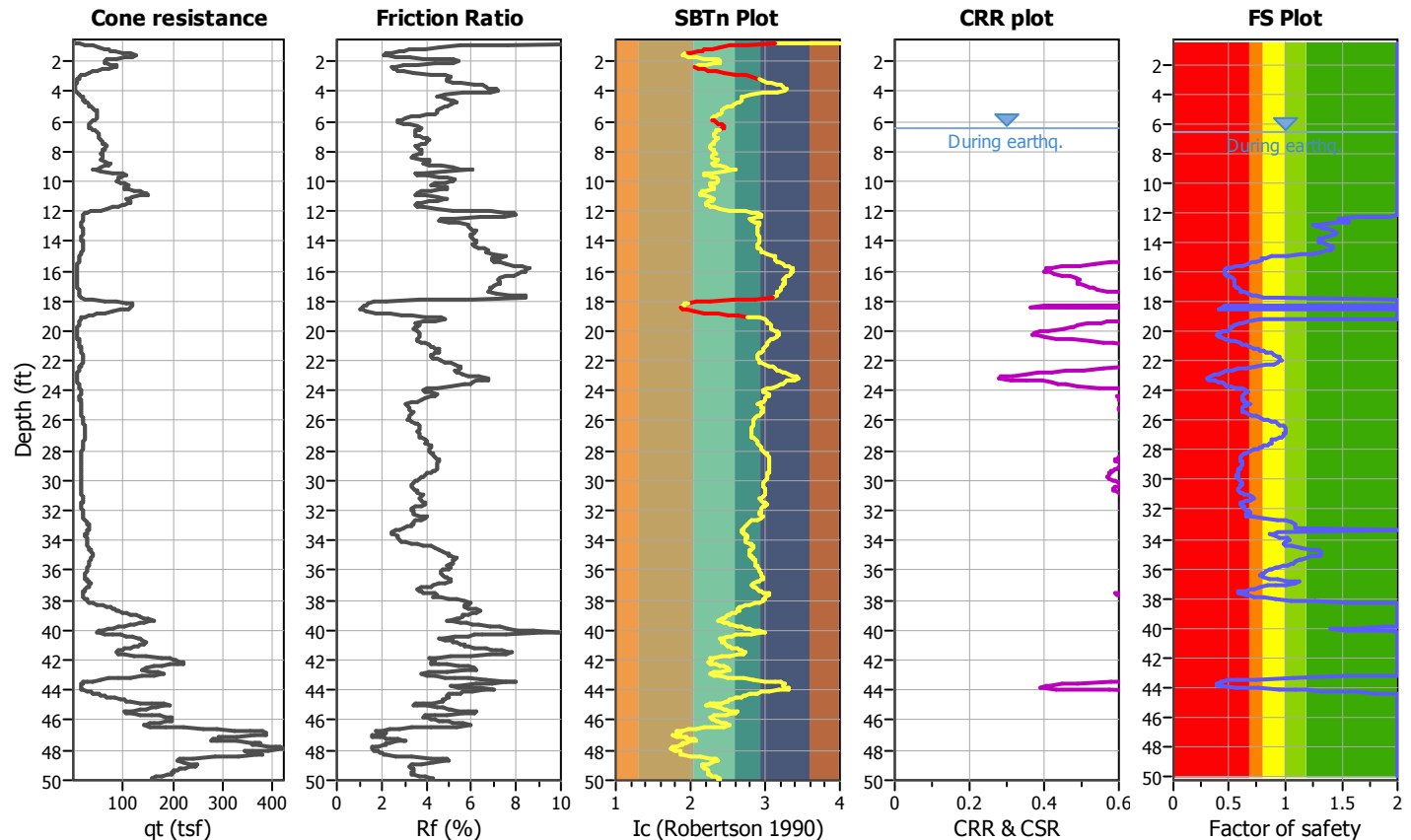
### Abbreviations

$q_c$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain

## LIQUEFACTION ANALYSIS REPORT

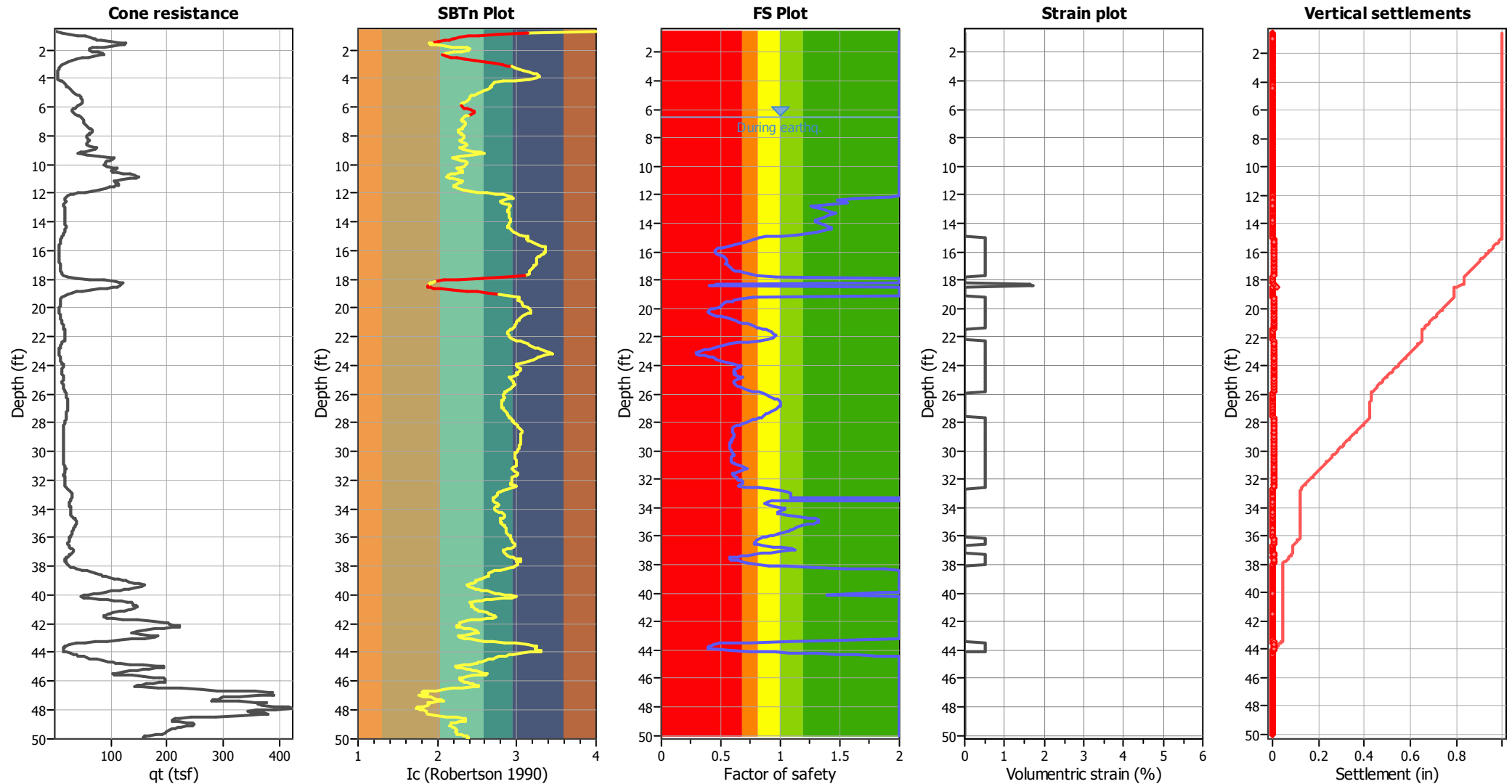
**Project title : 13-560 - Carolan Avenue and Rollins Road**
**Location : Burlingame, CA**
**CPT file : CPT-5**
**Input parameters and analysis data**

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Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_0$ applied:	Yes	MSF method:	Method based



Zone A<sub>1</sub>: Cyclic liquefaction likely depending on size and duration of cyclic loading  
 Zone A<sub>2</sub>: Cyclic liquefaction and strength loss likely depending on loading and ground geometry  
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening  
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## Estimation of post-earthquake settlements



### Abbreviations

$q_t$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain

**LIQUEFACTION ANALYSIS REPORT**

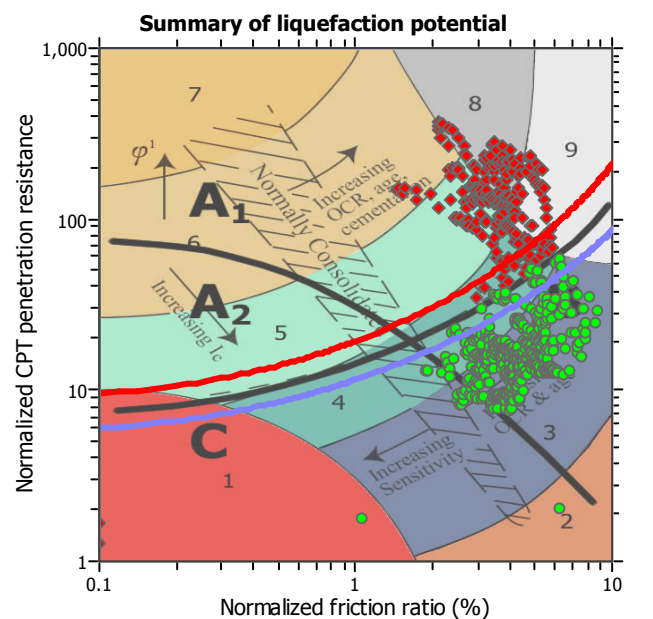
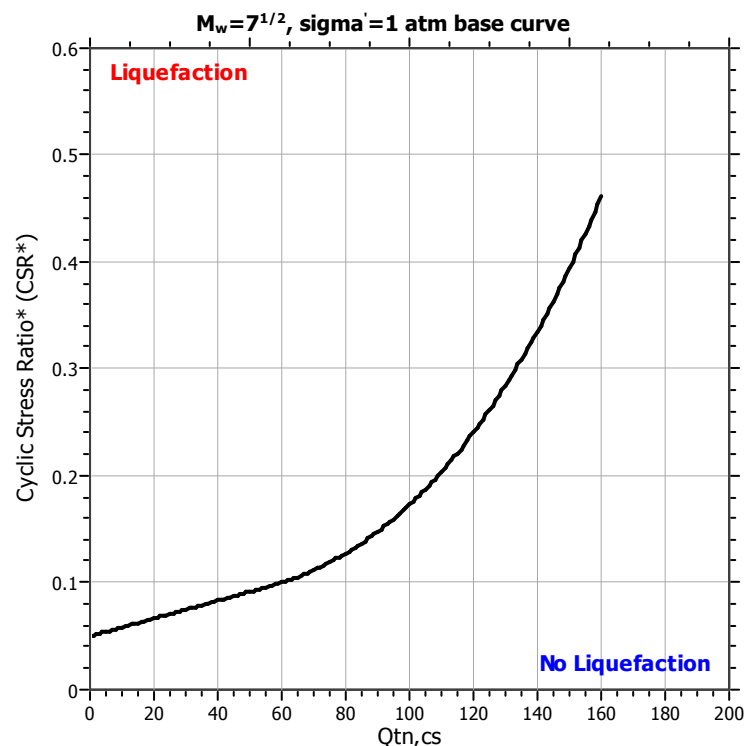
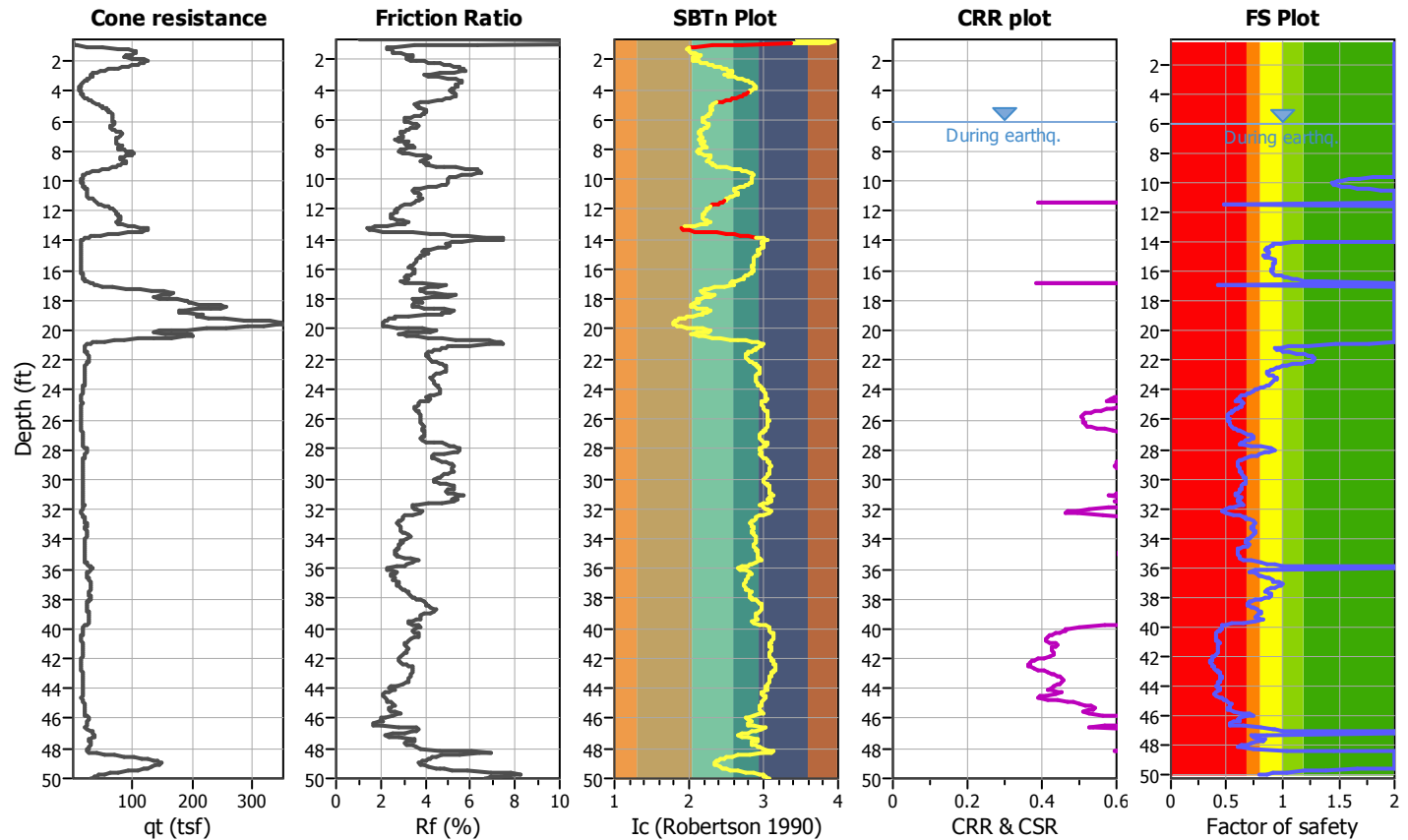
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**Location : Burlingame, CA**

**CPT file : CPT-6**

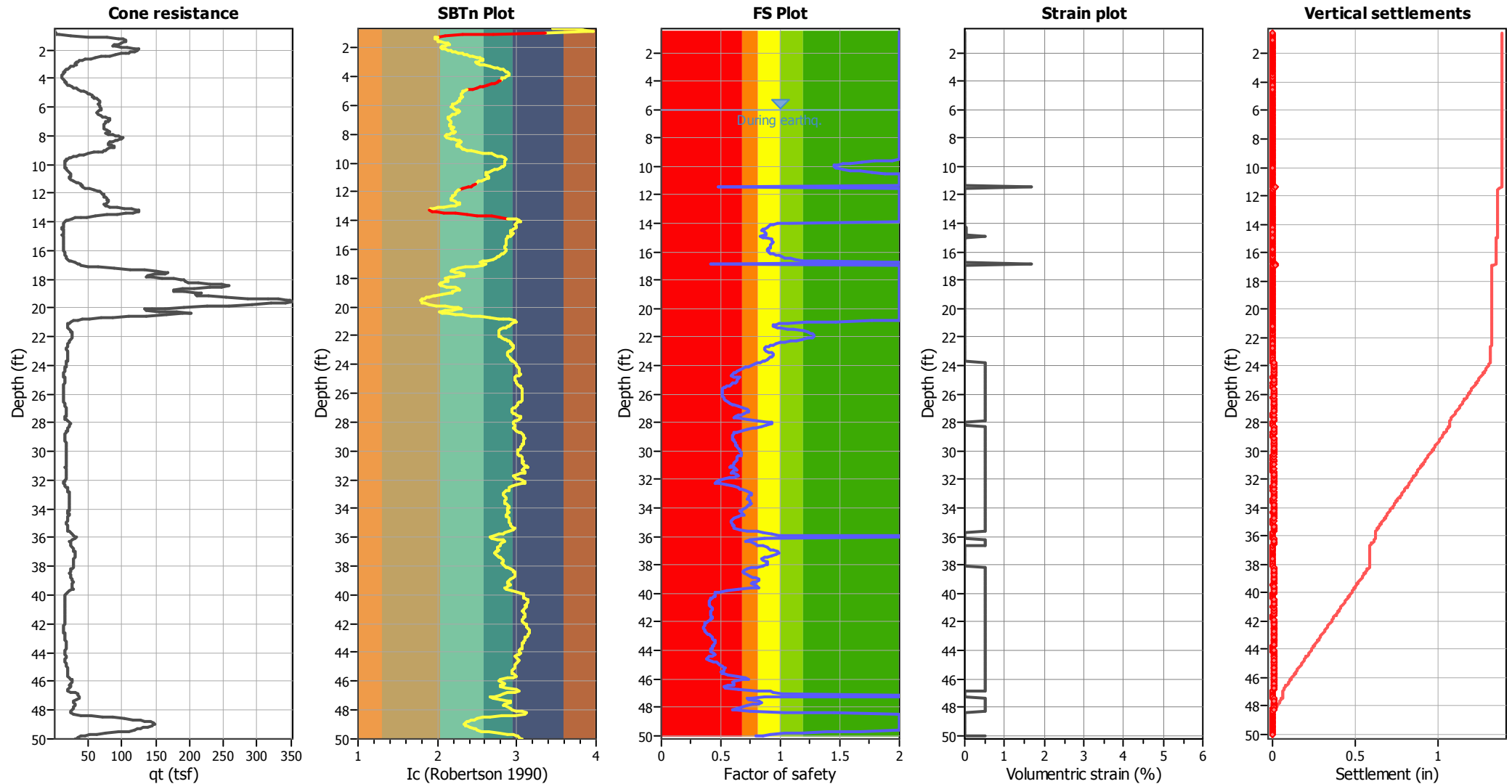
**Input parameters and analysis data**

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Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_0$ applied:	Yes	MSF method:	Method based



Zone A<sub>1</sub>: Cyclic liquefaction likely depending on size and duration of cyclic loading  
 Zone A<sub>2</sub>: Cyclic liquefaction and strength loss likely depending on loading and ground geometry  
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening  
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## Estimation of post-earthquake settlements



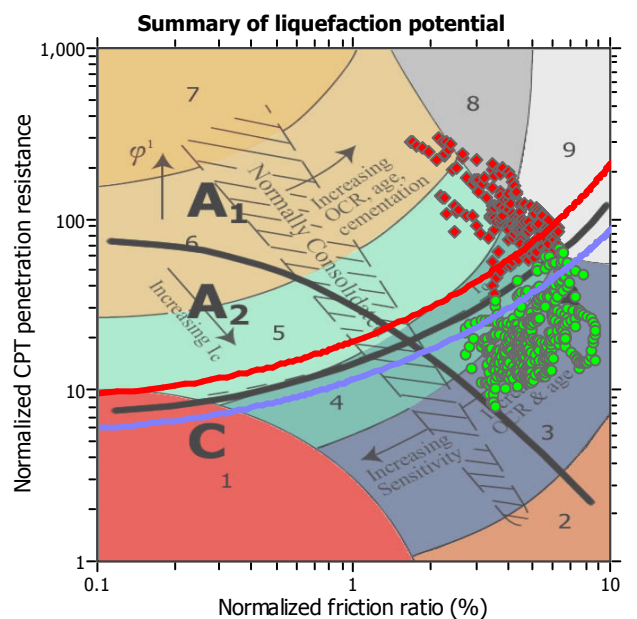
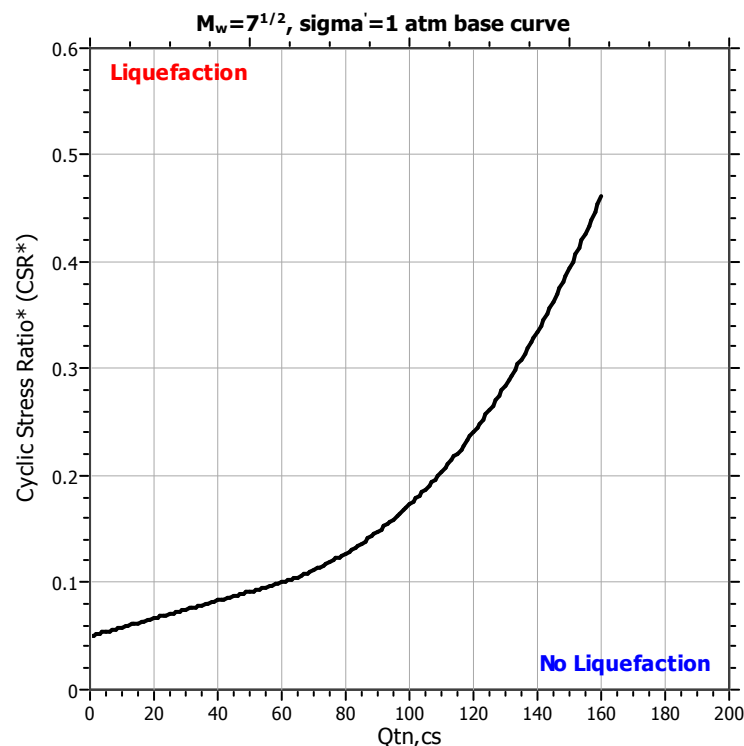
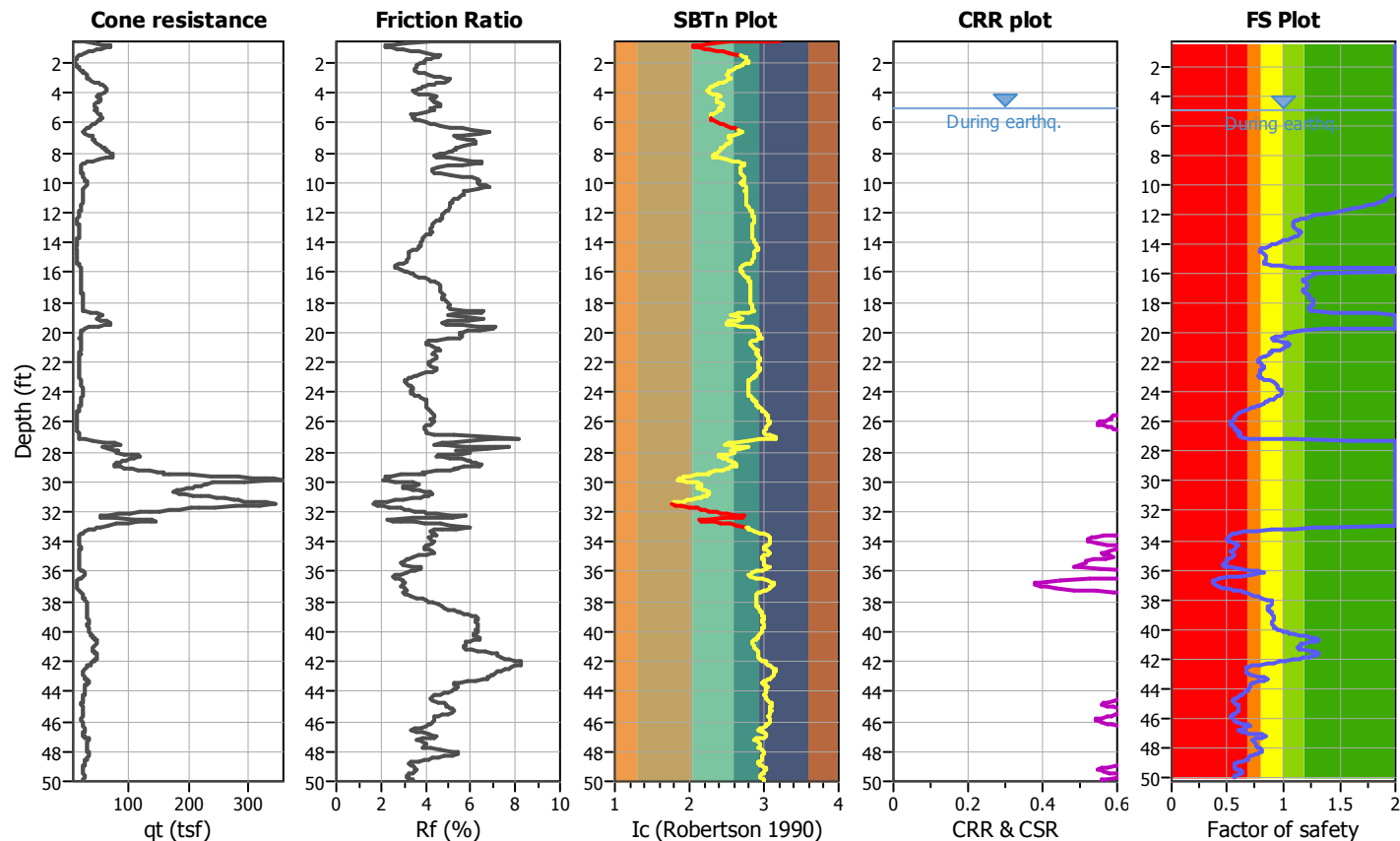
### Abbreviations

$q_c$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain



**LIQUEFACTION ANALYSIS REPORT**
**Project title : 13-560 - Carolan Avenue and Rollins Road**
**Location : Burlingame, CA**
**CPT file : CPT-7**
**Input parameters and analysis data**

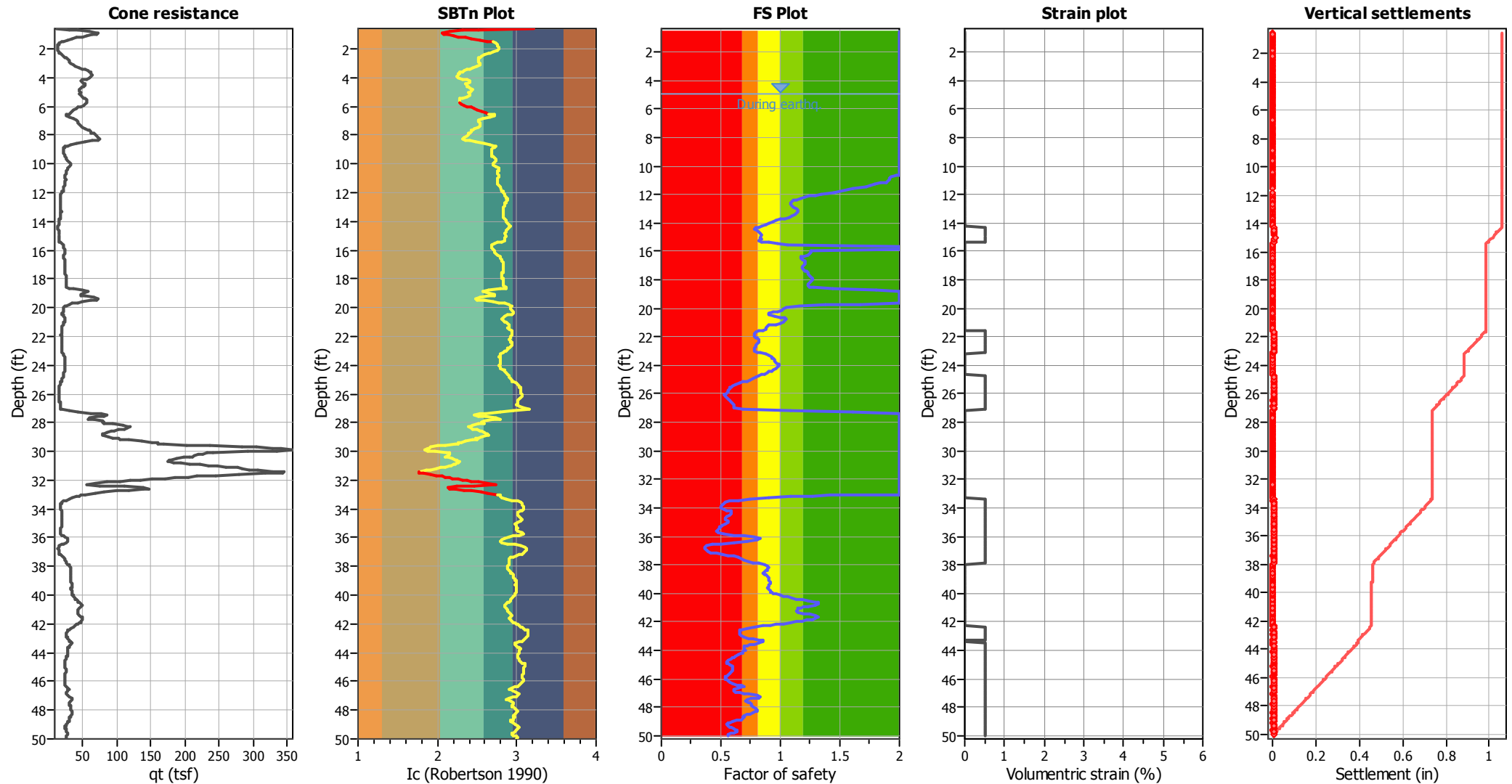
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Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_0$ applied:	Yes	MSF method:	Method based



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## Estimation of post-earthquake settlements



### Abbreviations

$q_c$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain

**LIQUEFACTION ANALYSIS REPORT**

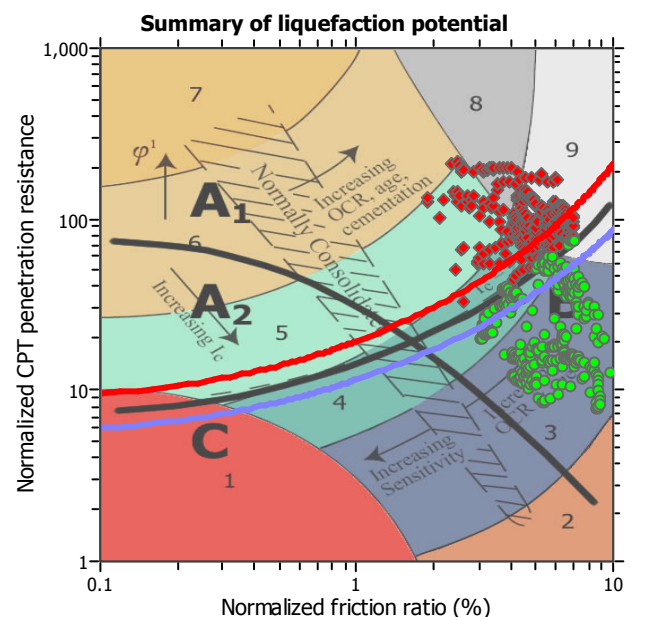
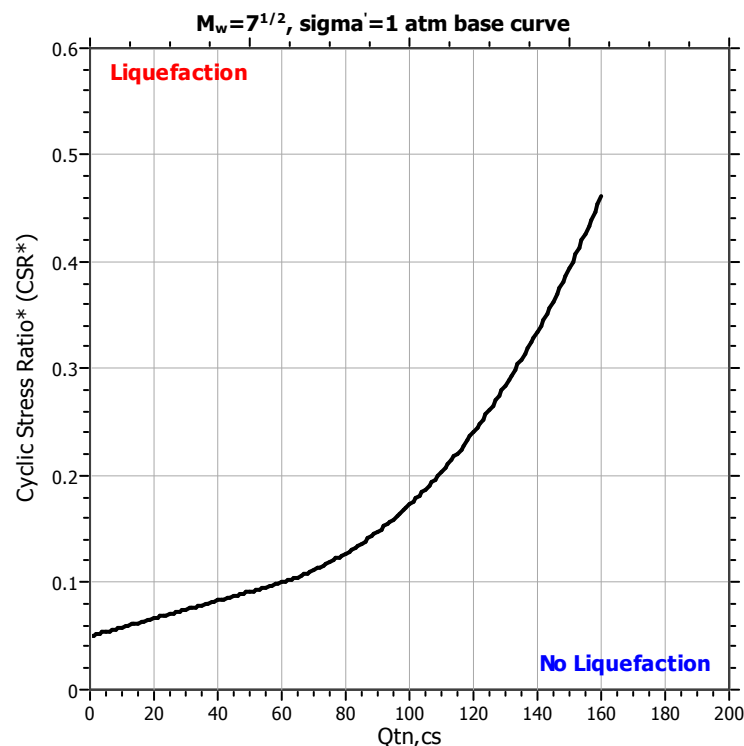
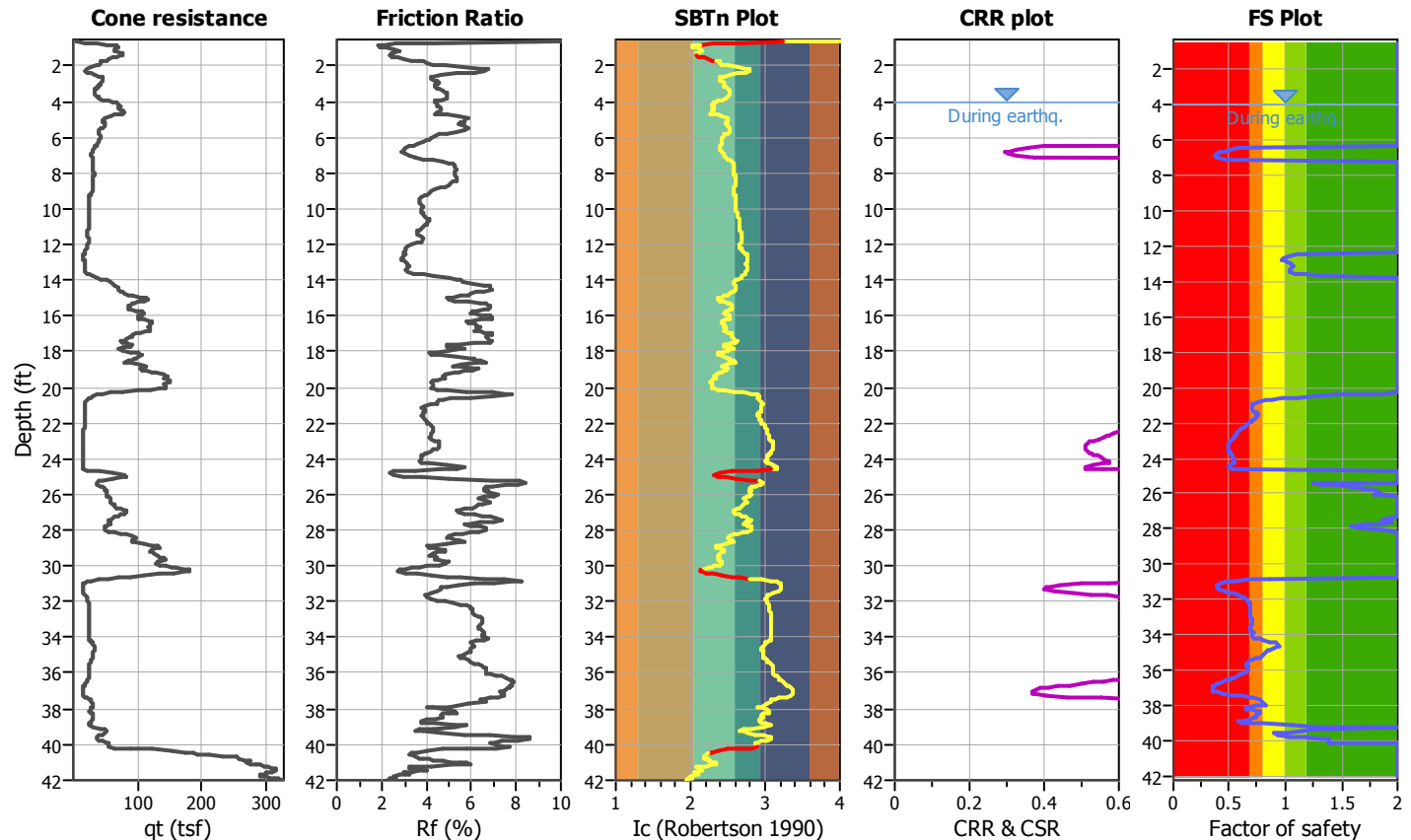
**Project title : 13-560 - Carolan Avenue and Rollins Road**

**Location : Burlingame, CA**

**CPT file : CPT-8**

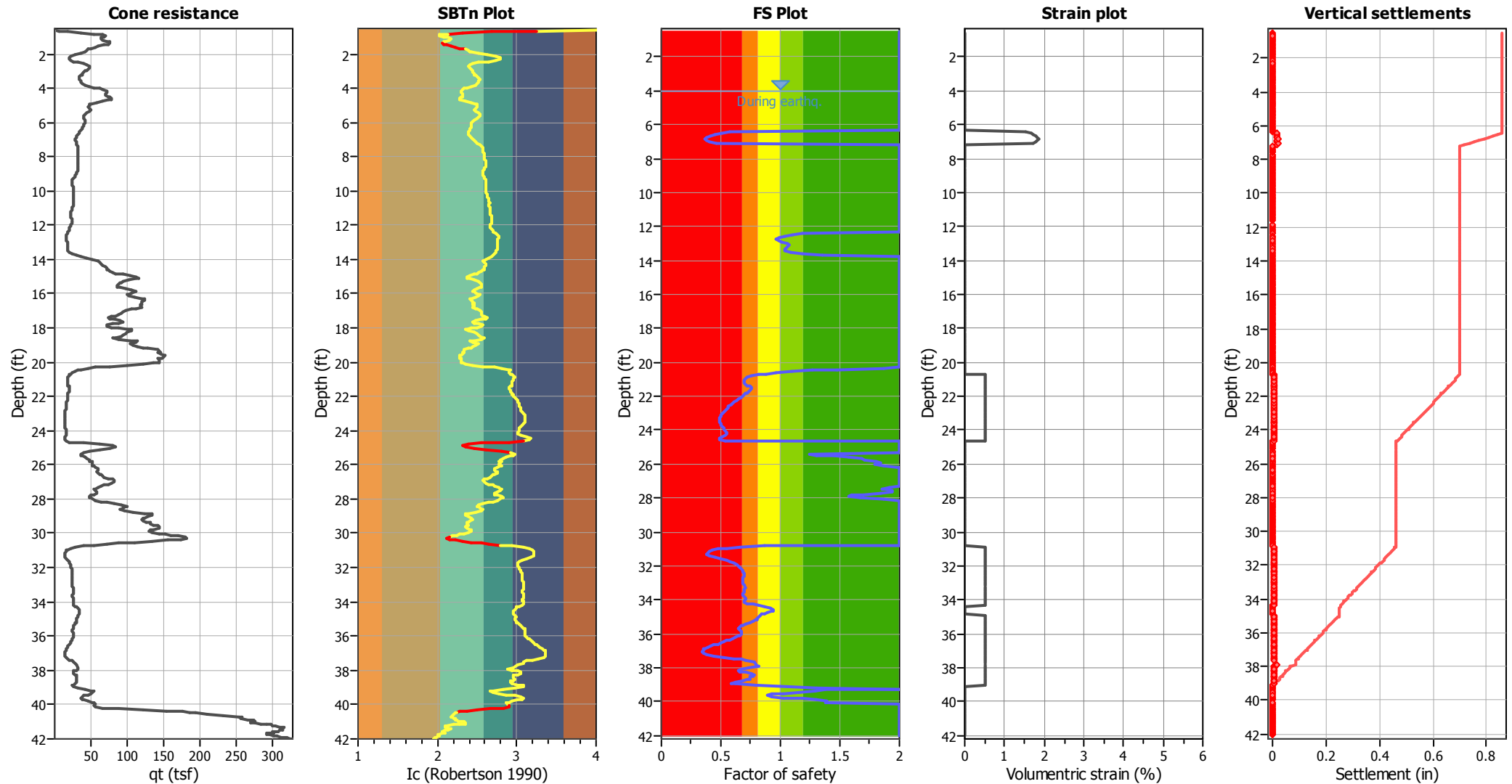
**Input parameters and analysis data**

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Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude $M_w$ :	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	$K_0$ applied:	Yes	MSF method:	Method based



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## Estimation of post-earthquake settlements



### Abbreviations

$q_c$ : Total cone resistance (cone resistance  $q_c$  corrected for pore water effects)  
 $I_c$ : Soil Behaviour Type Index  
 FS: Calculated Factor of Safety against liquefaction  
 Volumetric strain: Post-liquefaction volumetric strain

**APPENDIX D**  
**Planting Suitability Test Results**  
**by Soil & Plant Laboratory, Inc.**



San Jose Office  
December 19, 2013  
Report 13-337-0053

Rockridge Geotechnical, Inc.  
270 Grant Ave.  
Oakland, CA 94610

Attn: Logan Medeiros

RE: Carolan & Rollins, Burlingame, Job #13-560

## Background

Three samples were received on December 3, 2013 identified as soil from an area where new landscaping will be installed. Fertilizer and amendment recommendations were requested. The samples were analyzed for horticultural suitability, fertility and physical characteristics. The results of the analyses are attached.

## Analytical Results and Comments

### B-1 @ 0.5 to 2.0 feet

The reaction of the sample is slightly acidic at a pH of 6.8. This is within the range preferred by most plants. Salinity (ECe), sodium and boron are safely low. The SAR shows sodium is adequately balanced by soluble calcium and magnesium; this balance is important for soil structure quality and how it relates to water infiltration in this soil.

According to the USDA Soil Classification, the less than 2mm fraction of the sample is classified as loam. The organic content is low at 1.1% dry weight. Based on this information the average estimated infiltration rate is 0.32 inch per hour. Infiltration rates may vary due to differences in compaction across the site. The 54.6% silt plus clay present indicates the potential for slow drainage and high water holding capacity. Additional subdrainage may be helpful for large specimens in flat areas.

In terms of fertility, nitrogen, phosphorus and potassium are low. Calcium, magnesium and sulfate are all sufficient for proper plant nutrition.

Boron is safely low for general ornamental plants and may be below optimum levels for plant nutritional purposes. Irrigation water often supplies sufficient boron to meet plant nutritional requirements. However, if boron is low in the irrigation water and/or plants show symptoms of boron deficiency after they are well established, consider an application of a product containing boron at the manufacturer's label rate. Boron deficiency symptoms often include stunted or deformed younger growth and tight internodes. If palms are being installed in this soil, watch for deformation of younger tissue and "hooking" of fronds. Tissue testing can be performed to identify a boron deficiency if it is suspected.

### B-2 @ 0.5 to 2.0 feet

The reaction of the sample is moderately alkaline at a pH of 7.8. This pH is above the range preferred by most plants. Soil sulfur is recommended to help decrease the pH to a more favorable range. Soil sulfur works slowly and most efficiently only to the depth incorporated. Reaction of the soil sulfur to create acidity will be minimal until soil temperatures warm up.

Salinity (ECe), sodium and boron are safely low. The SAR shows sodium is adequately balanced by soluble calcium and magnesium.



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Rockridge Geotechnical, Inc.  
Report 13-337-0053

According to the USDA Soil Classification, the less than 2mm fraction of the sample is classified as sandy loam. The organic content is low at 1.5% dry weight. The 27.6% gravel present classifies it as gravelly. Based on this information the average estimated infiltration rate is moderate at 0.29 inch per hour. Infiltration rates may vary due to differences in compaction across the site.

In terms of fertility, nitrogen is low and potassium is fair. Phosphorus, calcium, magnesium and sulfate are all sufficient for proper plant nutrition.

#### B-4 @ 1.0 to 2.0 feet

The reaction of the sample is slightly alkaline at a pH of 7.5. This pH is within the range preferred by most plants. Salinity (ECe), sodium and boron are safely low. The SAR shows sodium is adequately balanced by soluble calcium and magnesium.

According to the USDA Soil Classification, the less than 2mm fraction of the sample is classified as sandy clay loam. The organic content is low at 1.4% dry weight. The 19.5% gravel present classifies it as gravelly. Based on this information the average estimated infiltration rate is moderate at 0.27 inch per hour. Infiltration rates may vary due to differences in compaction across the site.

In terms of fertility, potassium is low and calcium is fair. Nitrogen, phosphorus, magnesium and sulfate are all sufficient for proper plant nutrition.

#### **Recommendations**

Nitrogen, phosphorus, potassium and calcium fertilizers are recommended. A nitrogen stabilized organic amendment or composted greenwaste product is also recommended in order to help improve soil nutrient and water holding capacity. If a composted greenwaste product is chosen that would also provide additional phosphorus and potassium as well as needed micronutrients.

#### To Prepare for Mass Planting:

Drainage of the root zone should be improved by first loosening the top 10 inches of any undisturbed or compacted soil. The following materials should then be evenly spread and thoroughly blended with the top 6 inches of soil to form a homogenous layer:

##### Amount/1000 Square Feet B1

5 cubic yards	Nitrogen Stabilized Organic Amendment*
8 pounds	Calcium Nitrate (15.5-0-0)
3 pounds	Triple Superphosphate (0-45-0)*
8 pounds	Potassium Sulfate (0-0-50)*

##### Amount/1000 Square Feet B2

5 cubic yards	Nitrogen Stabilized Organic Amendment*
8 pounds	Calcium Nitrate (15.5-0-0)
4 pounds	Potassium Sulfate (0-0-50)*
8 pounds	Soil Sulfur



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Rockridge Geotechnical, Inc.  
Report 13-337-0053

Amount/1000 Square Feet B4

5 cubic yards	Nitrogen Stabilized Organic Amendment*
8 pounds	Calcium Nitrate (15.5-0-0)
8 pounds	Potassium Sulfate (0-0-50)*

To Prepare Backfill For Trees and Shrubs:

- Excavate planting pits at least twice as wide as the diameter of the rootball.
- Soil immediately below the root ball should be left undisturbed to provide support but the sides and the bottom around the side should be cultivated to improve porosity.
- The top of the rootball should be at or slightly above final grade.
- The top 12 inches of backfill around the sides of the rootball of trees and shrubs may consist of the above amended soil or may be prepared as follows:

3 parts	Site Soil
1 part	Nitrogen Stabilized Organic Amendment*

*Uniformly blended with:*

Amount per Cubic Yard B1

1/3 pound	Calcium Nitrate (15.5-0-0)
1/4 pound	Triple Superphosphate (0-45-0)*
1/3 pound	Potassium Sulfate (0-0-50)*

Amount per Cubic Yard B2

1/3 pound	Calcium Nitrate (15.5-0-0)
1/4 pound	Potassium Sulfate (0-0-50)*
1/3 pound	Soil Sulfur

Amount per Cubic Yard B4

1/3 pound	Calcium Nitrate (15.5-0-0)
1/3 pound	Potassium Sulfate (0-0-50)*

\*The rate may change based on the analysis of the chosen organic amendment. This rate is based on 270 lbs. dry weight of organic matter per cubic yard of amendment. If a composted greenwaste amendment is chosen that contains a substantial amount of phosphorus or potassium, the triple superphosphate or potassium sulfate should be reduced or omitted.

- Backfill below 12 inches required for 24 inch box or larger material should not contain the organic amendment, soil sulfur or calcium nitrate but should still contain the triple superphosphate and potassium sulfate at the recommended rate.
- Ideally a weed and turf free zone should be maintained just beyond the diameter of the planting hole. A 2-4 inch deep layer of coarse mulch can be placed around the tree or shrub. Mulch should be kept a minimum 4 inches from the trunk.
- Irrigation of new plantings should take into consideration the differing texture of the rootball substrate and surrounding soil matrix to maintain adequate moisture during this critical period of establishment.

## Maintenance

Maintenance fertilization may rely primarily on a nitrogen only program supplemented with a complete fertilizer in the fall and spring. You may begin applying Ammonium Sulfate (21-0-0) at a rate of 5 pounds per 1000 square feet 45-60 days after planting with refertilization every 45-60 days. Alternatively, slow release Sulfur-coated Urea (43-0-0) may be applied at a 5 pound rate with refertilization scheduled at 3 month intervals. Once the landscape



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Rockridge Geotechnical, Inc.

Report 13-337-0053

has become well established the frequency of fertilization should be decreased depending on color and rate of growth desired. In the winter for a quick greening effect, calcium nitrate (15.5-0-0) may be applied at a 6 pound rate if needed. In the spring and fall substitute a complete fertilizer such as 16-6-8 to help insure continuing adequate phosphorus and potassium.

Alternatively, organic sources of fertilizer such as Alfalfa, Blood, Soybean and Cotton Seed Meal may be applied per the label rate. Alfalfa Meal at a rate of 20 pounds per 1000 square feet would provide slow release nitrogen for 2-3 months or a combination of Blood and Feather Meal at a total of 16 pounds per 1000 square feet would provide nitrogen for 3-4 months. Once the landscape has become well established the frequency of fertilization should be decreased depending on color and rate of growth desired. In the spring and fall substitute a complete fertilizer such as 5-5-5 to help insure continuing adequate phosphorus and potassium. Or, nutrient rich composted greenwaste may be spread in a 1 to 2 inch layer, which generally carries enough nutrition to boost complete nutrition though a source of nitrogen might also be added at a half rate to assure adequate nitrogen availability.

If we can be of any further assistance, please feel free to contact us.

Annmarie Lucchesi

Emailed 5 Pages: [ldmedeiros@rockridgegeo.com](mailto:ldmedeiros@rockridgegeo.com)





Oakland CA 94610

Project : Carolan & Rollins  
Burlingame  
Job # 13-560

Report No : **13-337-0053**  
Purchase Order :  
Date Recd : 12/03/2013  
Date Printed : 12/09/2013  
Page : 1 of 1

## COMPREHENSIVE SOIL ANALYSIS

Sample Description - Sample ID	Half Sat %	pH	ECe dS/m	NO <sub>3</sub> -N ppm	NH <sub>4</sub> -N ppm	PO <sub>4</sub> -P ppm	K ppm	Ca ppm	Mg ppm	Cu ppm	Zn ppm	Mn ppm	Fe ppm	Organic % dry wt.	Lab No.
	TEC	Qual Lime		Sufficiency Factors											
B1	17	6.8	0.6	2	9	7	19	879	257					1.1	25276
	66	None		0.3		0.3	0.2	0.8	1.7						
B2	15	7.8	0.6	1	10	23	64	819	137					1.5	25277
	54	None		0.4		1.3	0.7	0.9	1.1						
B4	21	7.5	1.6	4	54	28	52	1322	605					1.4	25278
	117	None		1.4		1.1	0.3	0.7	2.3						

Saturation Extract Values						SAR	Gravel %		Percent of Sample Passing 2 mm Screen					USDA Soil Classification	Lab No.
Ca meq/L	Mg meq/L	Na meq/L	K meq/L	B ppm	SO <sub>4</sub> meq/L		Coarse 5 - 12	Fine 2 - 5	Sand			Silt .002-.05	Clay 0-.002		
									Very Coarse 1 - 2	Coarse 0.5 - 1	Med. to Very Fine 0.05 - 0.5				
2.3	1.6	1.7	0	0.06	2.2	1.2	0.4	2.5	1.8	2.3	41.2	31.4	23.2	Loam	25276
2.5	1.1	2.6	0.1	0.38	4.9	1.9	19.2	8.4	7.0	9.1	50.2	18.4	15.2	Gravelly Sandy Loam	25277
3.6	1.1	0.9	5.4	0.19	2.1	0.6	10.1	9.4	6.6	6.8	42.9	22.4	21.2	Gravelly Sandy Clay Loam	25278

Sufficiency factor (1.0=sufficient for average crop) below each nutrient value. N factor based on 200 ppm constant feed. SAR = Sodium adsorption ratio. Half Saturation %=approx field moisture capacity. Nitrogen(N), Potassium(K), Calcium(Ca) and Magnesium(Mg) by sodium chloride extraction. Phosphorus(P) by sodium bicarbonate extraction. Copper(Cu), Zinc(Zn), Manganese(Mn) & Iron(Fe) by DTPA extraction. Sat. ext. method for salinity (ECe as dS/m), Boron (B), Sulfate(SO<sub>4</sub>), Sodium(Na). Gravel fraction expressed as percent by weight of oven-dried sample passing a 12mm(1/2 inch) sieve. Particle sizes in millimeters. Organic percentage determined by Walkley-Black or Loss on Ignition.

\* LOW , SUFFICIENT , HIGH

## **PEER REVIEW**

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Prepared By: Cornerstone Earth Group

Date: June 23, 2014  
Project No.: 118-55-1  
Prepared For: Ms. Kristy Weis  
**DAVID J. POWERS & ASSOCIATES**  
1871 The Alameda, Suite 200  
San Jose, California 95126  
Re: Geotechnical Peer Review  
Carolan Avenue and Rollins Road  
Residential Development  
1008 Carolan Avenue  
Burlingame, California

Dear Ms. Weis:

As requested, we have performed a peer review of the Geotechnical Investigation Report prepared by Rockridge Geotechnical for the proposed residential development located at 1008, 1016, and 1028 Carolan Avenue and 935 Rollins Road in Burlingame, California. Based on our review, we conclude that the report is within the current standard of practice and generally addresses the geotechnical issues and concerns at the subject site. This letter presents minor comments pertaining to our review of the referenced geotechnical report.

The following documents were referenced during our review:

- Conceptual plan set including architectural and civil titled, "Carolan Avenue / Rollins Road, Burlingame, California, prepared by Seidel Architects, dated May 12, 2014.
- Final Geotechnical Investigation Report prepared by Rockridge Geotechnical, dated February 28, 2014.

Based on our review, we have the following comments.

1. The site is mapped as being partially underlain by Bay Mud (Helley and Lajoie, 1979), identified as Marsh Deposits by the consultant. Rockridge recommends surcharging for the southern end of the townhomes. Further exploration prior to surcharging should be performed to determine the limits of the marsh deposits. The consultant may want to consider a site specific geologic map indicating areas underlain by the compressible organic clay deposits.
2. We suggest a more detailed description of the surcharge program be provided, including surcharge amount and timing estimates, as well as a description of the monitoring

program (e.g. settlement plates, piezometers, ect.) to verify completion of the surcharge program.

3. The results of the consultant's liquefaction analysis indicate that liquefaction-induced settlement could significantly impact the proposed improvements at the site. Based on the 2013 California Building Code (CBC), which refers to Table 20.3-1 of ASCE 7-10, the site should likely be classified as Site Class F. A site-specific seismic analysis may be required, the exception being for structures having fundamental periods of vibration equal to or less than 0.5 seconds. The consultant should coordinate with the Project Structural Engineer to determine if a site-specific seismic analysis is needed.
4. The consultant identifies the depth of undocumented fill encountered within the footprint of the proposed townhome buildings was up to about 5 feet below the existing grade (Boring B-4). The consultant's recommendations to remove and replace 18 inches of soil below the bottom of footings does not address the full depth of undocumented fill encountered. The performance of undocumented fill under the townhomes may be difficult to predict if the undocumented fill is not removed and replaced as engineered fill in its entirety.

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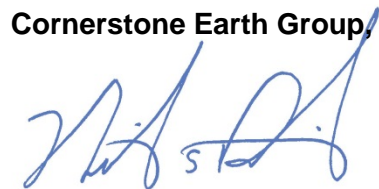
## Closure

Our peer review comments presented in this letter has been prepared for the sole use of David J. Powers & Associates specifically for the referenced Project. Our professional services were performed in accordance with generally accepted geotechnical engineering principles and practices at this time and location.

If you have any questions, please call and we will be glad to discuss them with you.

Sincerely,

**Cornerstone Earth Group, Inc.**



Nicholas S. Devlin, P.E.  
Project Engineer



Danh T. Tran, P.E.  
Senior Principal Engineer

NSD:CBB:DTT

Copies: Addressee (1 by email)

## **RESPONSE TO PEER REVIEW**

---

Prepared By: Rockridge Geotechnical, Inc.

## MEMORANDUM

**TO:** Elaine Breeze, Summerhill Apartment Communities

**FROM:** Logan D. Medeiros, Rockridge Geotechnical, Inc.

**DATE:** November 7, 2014

**PROJECT:** Proposed Residential Development  
1008, 1016, and 1028 Carolan Avenue/935 Rollins Road

**SUBJECT:** Response to Peer Review Comments

---

This memorandum presents our responses to peer review comments presented by Cornerstone Earth Group (CEG) in their letter dated June 23, 2014. CEG reviewed our February 28, 2014 geotechnical report for the subject project and the May 12, 2014 conceptual drawings and presented four comments:

Comment #1: *"The site is mapped as being partially underlain by Bay Mud (Helley and Lajoie, 1979), identified as Marsh Deposits by the consultant. Rockridge recommends surcharging for the southern end of the townhomes. Further exploration prior to surcharging should be performed to determine the limits of the marsh deposits. The consultant may want to consider a site specific geologic map indicating areas underlain by the compressible organic clay deposits."*

Response: Despite the referenced geologic mapping, the results of our site-specific exploratory borings and CPTs do not indicate typical Young Bay Mud is present beneath the site. The isolated thin (2 to 3 feet thick) layer of weak, compressible marsh deposits was encountered in boring B-4 and CPT-4 in the southeast corner of the site, but not in CPT-7, approximately 250 feet northeast along the proposed townhome alignment. Rather than performing additional investigation to further characterize the lateral extents of these deposits, we recommended the entire townhome area from Carolan Avenue to about 250 feet northeast be surcharged during construction. Based on discussions with Summerhill Apartment Communities, we understand the proposed construction schedule can easily accommodate the proposed surcharge program and soil is readily available from the podium building excavation. Therefore, further investigation is not necessarily warranted from a geotechnical standpoint.

Elaine Breeze  
Summerhill Apartment Communities  
November 7, 2014  
Page 2

Comment #2: *“We suggest a more detailed description of the surcharge program be provided, including surcharge amount and timing estimates, as well as a description of the monitoring program (e.g. settlement plates, piezometers, ect.) to verify completion of the surcharge program.”*

Response: As presented in Section 7.1 of our report, we recommend four feet of soil be stockpiled over the southwestern 250 feet of the townhome area in order to surcharge the underlying weak clay deposits. Soil from the excavation for the podium structure may be used for the surcharge. Provided the surcharge is left in place for at least 2 months, wick drains will not be required to expedite primary consolidation. After the surcharge soil is removed, the entire townhome area should be cut to approximate finished subgrade elevation.

Considering the weak layer is relatively thin, the proposed 4-foot-thick surcharge greatly exceeds the estimated weight of the 2-story townhome structures, and the conservative minimum 2-month time frame for the surcharge, we conclude extensive monitoring is not warranted. That being said, monitoring of a settlement plate is a reasonable and industry-accepted approach to confirming when the intended surcharge settlement is substantially complete. Depending on the actual construction schedule, monitoring of one or more settlement plates may be considered.

Comment #3: *“The results of the consultant’s liquefaction analysis indicate that liquefaction-induced settlement could significantly impact the proposed improvements at the site. Based on the 2013 California Building Code (CBC), which refers to Table 20.3-1 of ASCE 7-10, the site should likely be classified as Site Class F. A site-specific seismic analysis may be required, the exception being for structures having fundamental periods of vibration equal to or less than 0.5 seconds. The consultant should coordinate with the Project Structural Engineer to determine if a site-specific seismic analysis is needed.”*

Response: Although the 2013 CBC calls for a Site Class F designation for sites underlain by liquefiable soil, we conclude a Site Class D is appropriate considering the potentially liquefiable layers are generally thin and discontinuous. Therefore, the soil profile will not incur significant nonlinear behavior during strong ground shaking. Furthermore, the majority of the potentially liquefiable soil layers identified using the Robertson (2009) liquefaction methodology are characterized

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with “clay to silty clay” soil behavior type (SBT). These materials are not typically expected to exhibit classic sand-like liquefaction behavior. This is, in part, a result of the very high  $PGA_M$  required for liquefaction analyses by the latest building code—in this case 0.82g. Although these materials may be subject to cyclic softening during a major earthquake, we do not expect them to have a significant impact on the site response, and conclude that site Class D is appropriate, given the proposed building height. The building period has not been provided to us by the structural engineer.

Comment #4: *“The consultant identifies the depth of undocumented fill encountered within the footprint of the proposed townhome buildings was up to about 5 feet below the existing grade (Boring B-4). The consultant’s recommendations to remove and replace 18 inches of soil below the bottom of footings does not address the full depth of undocumented fill encountered. The performance of undocumented fill under the townhomes may be difficult to predict if the undocumented fill is not removed and replaced as engineered fill in its entirety.”*

Response: Considering the current relative density of the fill, as indicated by the recorded CPT tip resistances and SPT blowcounts, the fact that the upper 18 inches of existing fill will be re-worked, the relatively lightweight building type (2-story, wood-framed), and the fact that the recommended foundations consist of stiffened P-T slabs designed to accommodate up to one inch of differential settlement over 30 feet (static plus seismic), we conclude the inherent uncertainties associated with the undocumented fill have been appropriately accounted for.

We trust this memorandum provides the responses needed at this time. Please call, if you have questions.