ROCKRIDGE GEOTECHNICAL

Prepared for **THE KPA GROUP**

**GEOTECHNICAL INVESTIGATION**
**PROPOSED ADDITION**
**SUNNYVALE VA FACILITY**
**1080 INNOVATION WAY**
**Sunnyvale, California**

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May 27, 2014
Project No. 13-591

ROCKRIDGE GEOTECHNICAL

May 27, 2014
Project No. 13-591

Mr. Hratch Kouyoumdjian, S.E.
The KPA Group
One Kaiser Plaza, Suite 445
Oakland, CA 94612

Subject: Geotechnical Investigation
Proposed Addition
Sunnyvale VA Facility
1080 Innovation Way
Sunnyvale, California

Dear Mr. Kouyoumdjian:

We are pleased to present our geotechnical investigation report for the proposed addition to the existing building at 1080 Innovation Way in Sunnyvale, California. Our services were provided in accordance with our contract with The KPA Group, dated April 28, 2014.

We understand the Department of Veterans Affairs (VA) has acquired 4.4 acres in the former Onizuka Air Force Station (AFS), including three existing buildings (Buildings 1002, 1018, and 1034). Buildings 1018 and 1034 are slated for future demolition. Building 1002, which was the former headquarters building for the Onizuka AFS, is a two-story, 51,000-square-foot, steel-framed structure that was constructed in 1962 with a substantial addition in 1964. Plans are to renovate and seismically retrofit Building 1002, construct a new lobby adjacent to the northeast corner of the building, and construct a two-story addition adjacent to the north side of the building. We previously performed a geotechnical investigation at the site for the proposed renovation and lobby addition, the results of which were presented in a report dated October 29, 2013. The addition will include a lobby with two new elevators and a courtyard and will have a finished floor elevation approximately 2-1/2 feet above the current pavement grade.

Based on the results of our geotechnical investigation, we conclude there are no major geotechnical or geological issues that would preclude construction of the proposed addition. We conclude the proposed addition may be supported on conventional spread footings bottomed in the engineered fill (new or existing) and/or stiff/dense native soil.

The recommendations contained in our report are based on a limited subsurface exploration. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to

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observe site grading and drilled pier installation during which time we may make changes in our recommendations, if deemed necessary.

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.

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

Enclosure

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**SUNNYVALE VA FACILITY**
**1080 INNOVATION WAY**
**Sunnyvale, California**

## 1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical for the proposed addition to the existing building at 1080 Innovation Way in Sunnyvale, California. The building is located on east side of Innovation Way, south of its intersection with North Mathilda Avenue, as shown on the attached Site Location Map (Figure 1).

We understand the Department of Veterans Affairs (VA) has acquired 4.4 acres in the former Onizuka Air Force Station (AFS), including three existing buildings (Buildings 1002, 1018, and 1034). Buildings 1018 and 1034 are slated for future demolition. Building 1002, which was the former headquarters building for the Onizuka AFS, is a two-story, 51,000-square-foot, steel-framed structure that was constructed in 1962 with a substantial addition in 1964. Plans are to renovate and seismically retrofit Building 1002, construct a new lobby adjacent to the northeast corner of the building, and construct a two-story addition adjacent to the north side of the building. We previously performed a geotechnical investigation at the site for the proposed renovation and lobby addition, the results of which were presented in a report dated October 29, 2013. The two-story addition will include a lobby with two new elevators and a courtyard and will have a finished floor elevation approximately 2-1/2 feet above the current pavement grade.

## 2.0 SCOPE OF SERVICES

Our scope of work consisted of evaluating subsurface conditions at the site by performing two cone penetration tests (CPTs), and performing engineering analyses to develop conclusions and recommendations regarding:

- site seismicity and seismic hazards, including the potential for liquefaction and lateral spreading, and total and differential settlement resulting from liquefaction and/or cyclic densification
- the most appropriate foundation type(s) for the proposed addition
- design criteria for the recommended foundation type(s), including vertical and lateral capacities for each of the foundation type(s)
- estimates of foundation settlement
- subgrade preparation for slabs-on-grade and exterior concrete flatwork
- 2013 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

## 3.0 FIELD INVESTIGATION

Subsurface conditions at the site were originally investigated on September 27, 2013 by performing three CPTs, designated as CPT-1 through CPT-3. A supplemental investigation was performed on May 6, 2014 by advancing two additional CPTs, designated as CPT-4 and CPT-5. The approximate CPT locations are shown on Figure 2. Prior to performing the CPTs, we contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained Precision Locating, LLC, a private utility locator, to check that the CPT locations were clear of existing utilities.

The CPTs were each advanced to a depth of 45 feet below the existing ground surface (bgs) by John Sarmiento & Associates of Orinda, California 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 tests were conducted. Accumulated data were processed by computer to provide engineering information such as the soil behavior types and approximate strength characteristics of the soil encountered. The logs for the CPTs performed at the site are presented on Figures A-1 through A-5 in Appendix A.

## 4.0 SUBSURFACE CONDITIONS

The Regional Geologic Map (Figure 3) indicates the site is underlain by Holocene alluvium (Qha). The results of our field investigation indicate the CPT-1 and CPT-2 locations are blanketed by about three feet of dense granular soil, which is likely engineered fill that was placed during the original construction of Building 1002 and its addition in 1962 and 1964, respectively. At the CPT-3, CPT-4, and CPT-5 locations, the dense granular fill appears to only be 1-1/2 to 3 feet thick and it is underlain by very stiff to hard native clay that generally extends to near the groundwater table at a depth of 9-1/2 to 12 feet. The soil below a depth of about to 9-1/2 to 12 feet bgs consists of heterogeneous alluvial sediments that consist predominantly of stiff to very stiff clays and silts with some isolated zones of medium stiff clay interbedded with discontinuous thin sand layers to the maximum depth explored of 45 feet bgs. The granular layers were encountered at varying depths in each CPT and are less than about three feet thick. The medium stiff clays and silts were primarily encountered between depths of 13 and 29 feet bgs and are thickest at the CPT-2 location.

Groundwater was measured in the CPT holes at depths ranging between 11.0 and 12.6 feet bgs prior to grouting; however, the groundwater level may not have been stabilized at the time the measurements were taken. Groundwater was reportedly measured as shallow as 8 feet below grade in a boring drilled at the site in November 1958. The depth to groundwater is expected to vary several feet seasonally. We recommend that a design groundwater depth of 8 feet bgs be used for design.

## 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[1], lateral spreading[2] and cyclic densification.[3] 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, Hayward, and Calaveras Faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean characteristic Moment magnitude[4] [Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

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

2 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.

3 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

4 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 1**
**Regional Faults and Seismicity**

| Fault Segment | Approximate Distance from Site (km) | Direction from Site | Mean Characteristic Moment Magnitude |
|---|---|---|---|
| Monte Vista-Shannon | 11 | Southwest | 6.50 |
| Total Hayward | 15 | Northeast | 7.00 |
| Total Hayward-Rodgers Creek | 15 | Northeast | 7.33 |
| N. San Andreas - Peninsula | 15 | Southwest | 7.23 |
| N. San Andreas (1906 event) | 15 | Southwest | 8.05 |
| Total Calaveras | 20 | East | 7.03 |
| N. San Andreas - Santa Cruz | 26 | South | 7.12 |
| San Gregorio Connected | 35 | West | 7.50 |
| Zayante-Vergeles | 36 | South | 7.00 |
| Mount Diablo Thrust | 40 | Northeast | 6.70 |
| Greenville Connected | 43 | East | 7.00 |

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 43 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 about 11 kilometers from the Monte Vista-Shannon Fault and about 15 kilometers from both the San Andreas and Hayward faults. 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.

The site has been mapped within a zone of liquefaction potential on the map titled *State of California, Seismic Hazard Zones, Milpitas Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated October 19, 2004. Special Publication 117 by CGS (2008) recommends subsurface investigations in mapped liquefaction potential areas be performed using rotary-wash borings and/or cone penetration tests.

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 an assumed high groundwater depth of 7 feet bgs. In accordance with the 2013 California Building Code (CBC), we used a peak ground acceleration of 0.5 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 1.

Our liquefaction analyses indicate there are a few thin layers of potentially liquefiable soil below the groundwater at the CPT locations, as well as zones of clay and silt which may be susceptible to "cyclic softening". We performed analyses to estimate ground surface settlement associated with liquefaction (referred to as post-liquefaction reconsolidation) following a moment magnitude 8.05 earthquake. On the basis of our analyses, we conclude the potential liquefaction-induced settlement at the site during a major earthquake will be less than about 0.6 inches. A summary of the liquefaction analyses is presented in Appendix B.

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 very 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 CPTs indicate the soil above the groundwater at the site consists of cohesive soil and dense granular soil, which are not susceptible to cyclic densification. Accordingly, we conclude the potential for ground surface settlement resulting from cyclic densification is very 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 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our engineering analyses using the data from the CPTs, we conclude there are no major geotechnical or geological issues that would preclude construction of the proposed improvements. Our conclusions and recommendations regarding site preparation and grading, foundation and slab support, and seismic design are presented in the following sections.

### 6.1 Foundation Support and Settlement

We conclude the proposed addition may be supported on conventional spread footings bearing on the existing dense granular fill and/or stiff/dense native soil. Footings for the addition may be designed using an allowable bearing pressure of 4,000 pounds per square foot (psf) for dead-plus-live loads with a one-third increase for total loads. New continuous footings should be at least 18 inches wide and isolated footings should be at least 24 inches square. Interior footings should be bottomed at least 18 inches below the bottom of the capillary break. Exterior footings should be bottomed at least 24 inches below the finished outside grade or 18 inches below the capillary break, whichever is lower. We estimate total settlement of the addition will be about ¾ inch and differential settlement will be less than ½ inch over a horizontal distance of 25 feet. The majority of the settlement will occur during construction.

For evaluation of existing footings and design of new footings, we recommend a modulus of vertical subgrade reaction ($k_{v1}$) of 300 pounds per cubic inc (pci) be used. This modulus value should be scaled to account for footing width (B) using the following equation:

$$k_s = \frac{k_{v1}}{B} [(m+0.5)/1.5m]$$

Where: B = Width of loaded area

$k_{v1}$ = Modulus of vertical subgrade reaction for one-foot-square plate

mB = Length of loaded area

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 equivalent fluid weight of 300 pounds per cubic foot (pcf). Passive resistance for the upper foot of soil should be ignored unless it is confined by a pavement or slab. Frictional resistance should be computed using a base friction coefficient of 0.35. The passive pressure and frictional resistance values include a factor of safety of at least 1.5.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If suitable bearing material is not encountered at the minimum footing depth, the footing excavation should be deepened until the excavation suitable bearing material is reached. The deepened portion of the footing should be filled with controlled low-strength material (CLSM) with a minimum 28-day unconfined compressive strength of 100 psi. The bottoms and sides of the footing excavations should be wetted just prior to concrete placement. We should check footing excavations prior to placement of reinforcing steel.

### 6.2 Subgrade Preparation, Grading, and Fill Placement

Site clearing should include removal of all existing foundations, slabs, pavements, and underground utilities in the areas to received new improvements. Demolished asphalt and concrete should be taken to a recycling facility. Where utilities that are removed extend off site, they should be capped or plugged with grout at least five feet from the perimeter of the addition. It may be feasible to abandon utilities in-place by filling them with grout, provided they will not impact future utilities or the building foundations. The utility lines, if encountered, should be addressed on a case-by-case basis.

The soil subgrade in areas to receive fill or improvements (including exterior and interior slabs-on-grade) should be scarified to a depth of at least eight inches, moisture-conditioned to above above optimum moisture content, and compacted to at least 90 percent relative compaction[5].

[5] 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.

The upper eight inches of the subgrade beneath slabs or pavements that will receive vehicular traffic should be compacted to at least 95 percent relative compaction. Excavated on-site soil can be reused as fill provided it contains no debris, organic material, or rocks or lumps larger than three inches. If imported fill (select fill) is required, it should also be free of rock or lumps larger than three inches or other deleterious material, and should have a liquid limit less than 40 and a plasticity index (PI) less than 12. All fill materials should be approved by the Geotechnical Engineer at least three days before use at the site.

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. Fill placed for the building pad should extend at least five feet outside the building footprint. If the fill thickness will be greater than five feet, the fill should be compacted to at least 95 percent relative compaction. Imported sand or gravel fill containing less than 10 percent fines (particles passing the No. 200 sieve) should be compacted to at least 95 percent relative compaction.

#### 6.2.1 Utility Trenches

Excavations for utility trenches can be readily made with a backhoe. Based on our investigation, it appears the upper two to four feet of soil is granular and may not stand vertically in trenches. Where cuts deeper than four feet are required, the excavation sides should be sloped or shored in accordance with the CAL-OSHA requirements provided in Title 8 (Construction Safety Orders) of the California Code of Regulations.

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 as according to the recommendations previously presented. 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.

Where utility trenches backfilled with sand or gravel enter the building pad, an impermeable plug consisting of lean concrete, at least three feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

### 6.3 Concrete Slabs-on-Grade

The slabs-on-grade for the addition should be supported on at least 12 inches of granular fill compacted to at least 90 percent relative compaction. If highly expansive soil is exposed at subgrade level for the slab-on-grade floor, it should be excavated to a depth of at least 12 inches below pad subgrade and replaced with on-site or imported select fill meeting the requirements presented above in Section 6.2.

To minimize water vapor transmission through the building floor slab, we recommend installing a capillary moisture break and a water vapor retarder beneath the floor. 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 dry at the time concrete is placed. 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 2.

**TABLE 2**
**Gradation Requirements for Capillary Moisture Break**

| Sieve Size | Percentage Passing Sieve |
|---|---|
| *Gravel or Crushed Rock* | |
| 1 inch | 90 – 100 |
| 3/4 inch | 30 – 100 |
| 1/2 inch | 5 – 25 |
| 3/8 inch | 0 – 6 |
| *Sand* | |
| 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 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.

### 6.4 Exterior Concrete Flatwork

Exterior concrete flatwork that will not receive vehicular traffic (i.e. sidewalk) should be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction. Prior to placement of the aggregate base, the upper eight inches of the subgrade soil should be scarified, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction.

### 6.5 Seismic Design

For design in accordance with the 2013 SFBC, we recommend Site Class D be used. The latitude and longitude of the site are 37.4049 and -122.0248, respectively. Hence, in accordance with the 2013 SFBC, we recommend the following:

- $S_S$ = 1.500g, $S_1$ = 0.600g
- $S_{MS}$ = 1.500g, $S_{M1}$ = 0.900g
- $S_{DS}$ = 1.000g, $S_{D1}$ = 0.600g
- $PGA_M$ = 0.500g
- Seismic Design Category D for Risk Categories I, II, and III.

## 7.0 ADDITIONAL GEOTECHNICAL SERVICES

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 installation of new foundations and fill placement and compaction. These observations will allow us to compare actual with anticipated subsurface conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

## 8.0 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 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.

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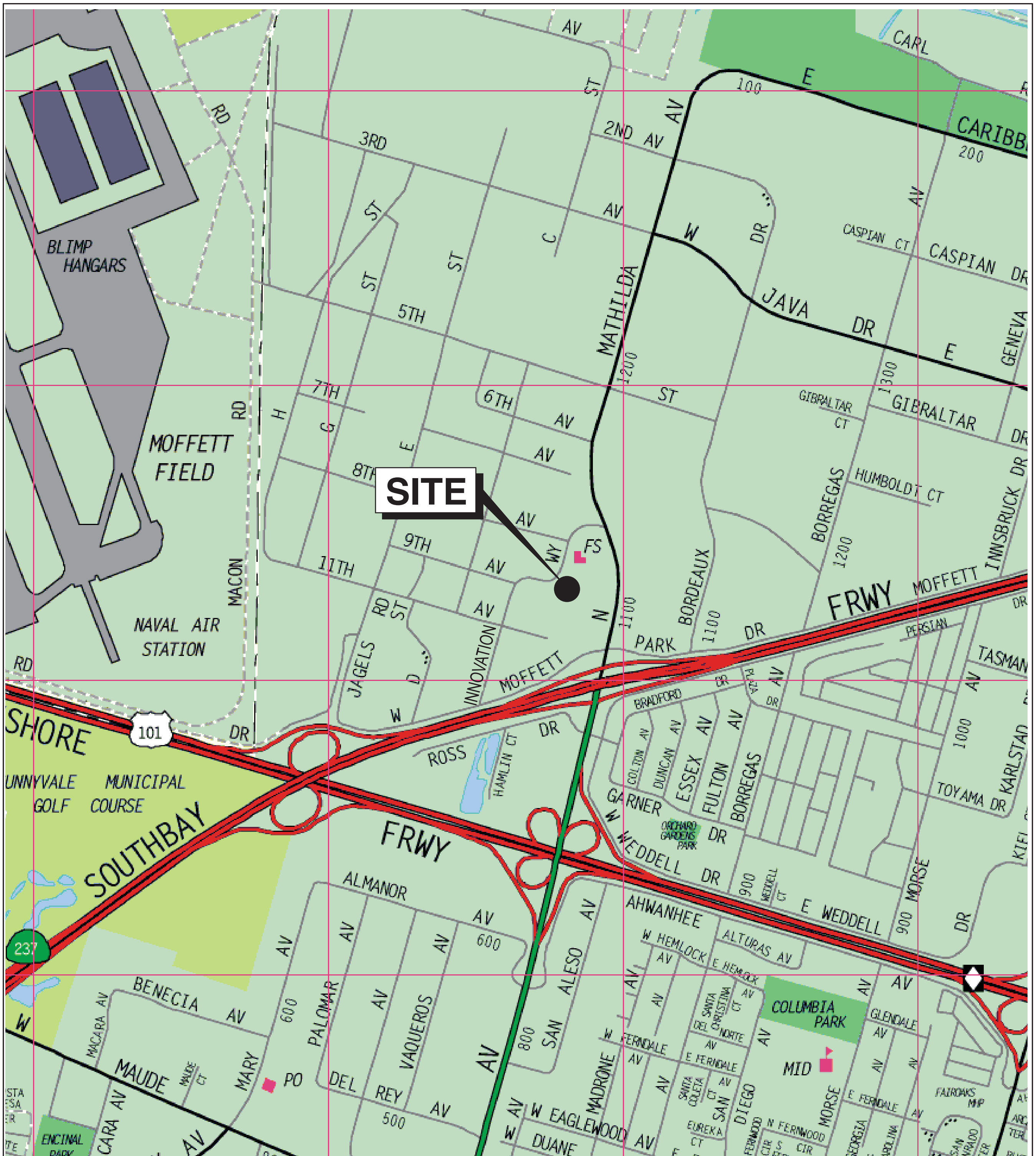
# FIGURES




Base map: The Thomas Guide
Santa Clara County

0 1/4 1/2 Mile

Approximate scale

**SUNNYVALE VA FACILITY**
**1080 INNOVATION WAY**
Sunnyvale, California

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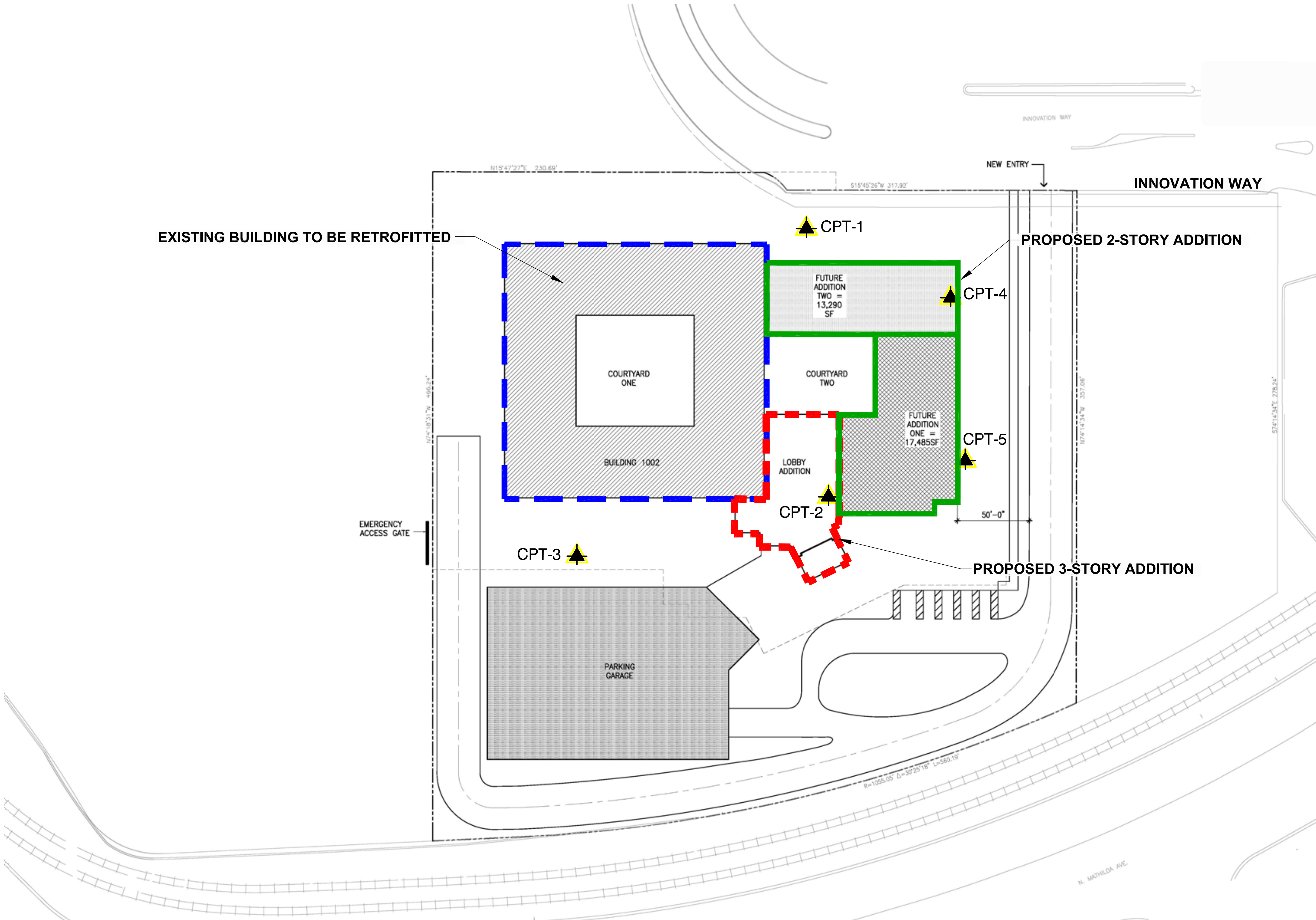
**SITE LOCATION MAP**

| Date 10/17/13 | Project No. 13-591 | Figure 1 |
|---|---|---|




**EXPLANATION**

CPT-1 ▲ Approximate location of cone penetration test performed by Rockridge Geotechnical; 9-25-13

Existing building to be retrofitted

Proposed 3-story addition

Proposed 2-story addition

0 — 80 Feet
Approximate scale

**SUNNYVALE VA FACILITY**
**1080 INNOVATION WAY**
Sunnyvale, California

**SITE PLAN**

| Date 05/26/14 | Project No. 13-591 | Figure 2 |
|---|---|---|

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Reference: Base map from a drawing titled "Site Plan - Future - (N.I.C)", Drawing Number CU104 by The KPA Group, dated 04-22-2014.


Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2013.

**EXPLANATION**

| | |
|---|---|
| af | Artificial Fill |
| Qhym | Mud deposits (late Holocene) |
| Qha | Alluvium (Holocene) |
| | Geologic contact: dashed where approximate and dotted where concealed, queried where uncertin |

0 4000 8000 Feet

Approximate scale

**SUNNYVALE VA FACILITY**
**1080 INNOVATION WAY**
Sunnyvale, California

ROCKRIDGE GEOTECHNICAL

## REGIONAL GEOLOGIC MAP

Date 10/09/13 | Project No. 13-591 | Figure 3


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

**EXPLANATION**

- Strike slip
- Thrust (Reverse)
- Normal

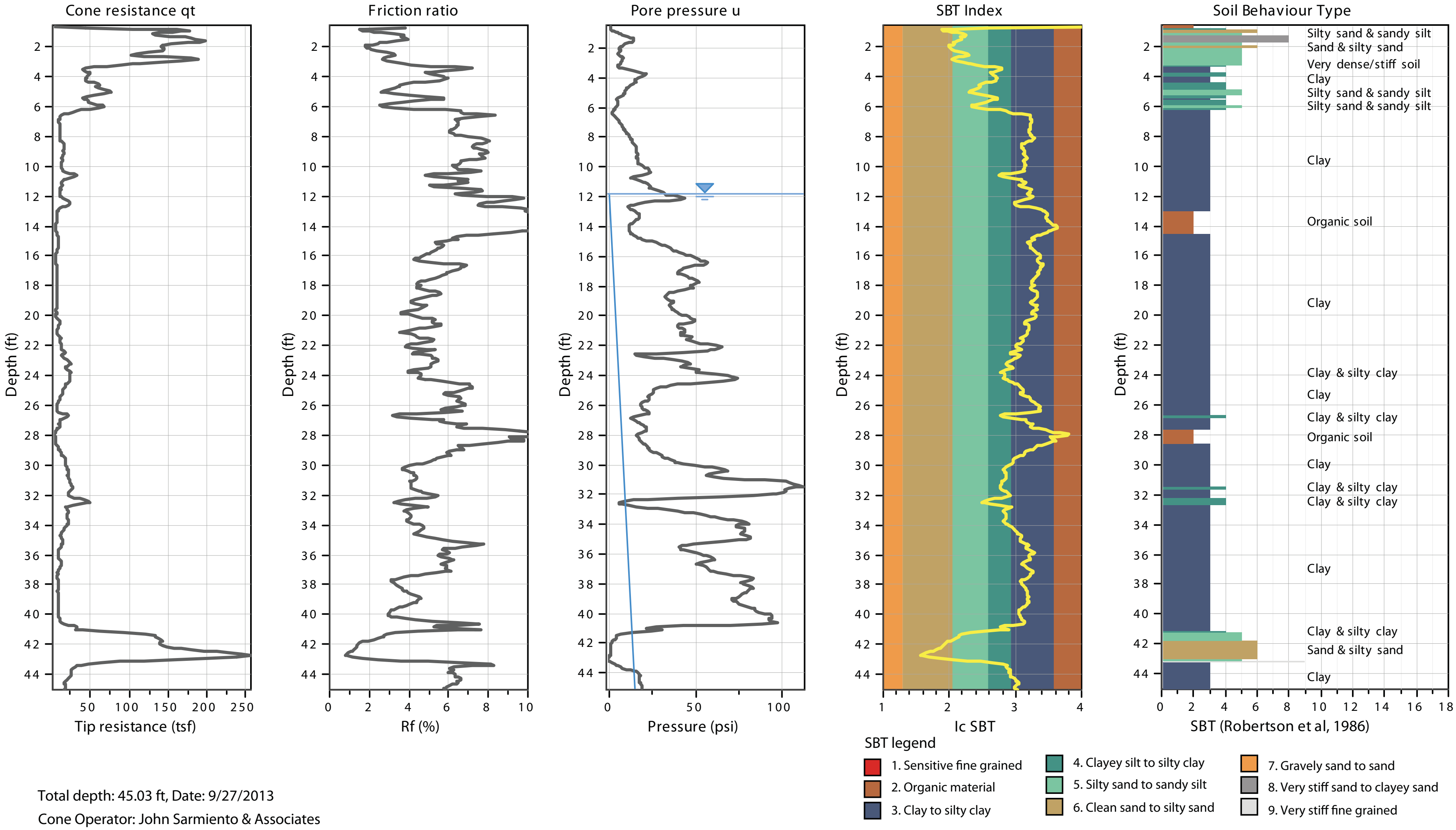
N

0 5 10 Miles

Approximate scale

| | |
|---|---|
| **SUNNYVALE VA FACILITY**<br>**1080 INNOVATION WAY**<br>Sunnyvale, California | **REGIONAL FAULT MAP** |
| ROCKRIDGE GEOTECHNICAL | Date 10/09/13 \| Project No. 13-591 \| Figure 4 |

# APPENDIX A
## Cone Penetration Test Results

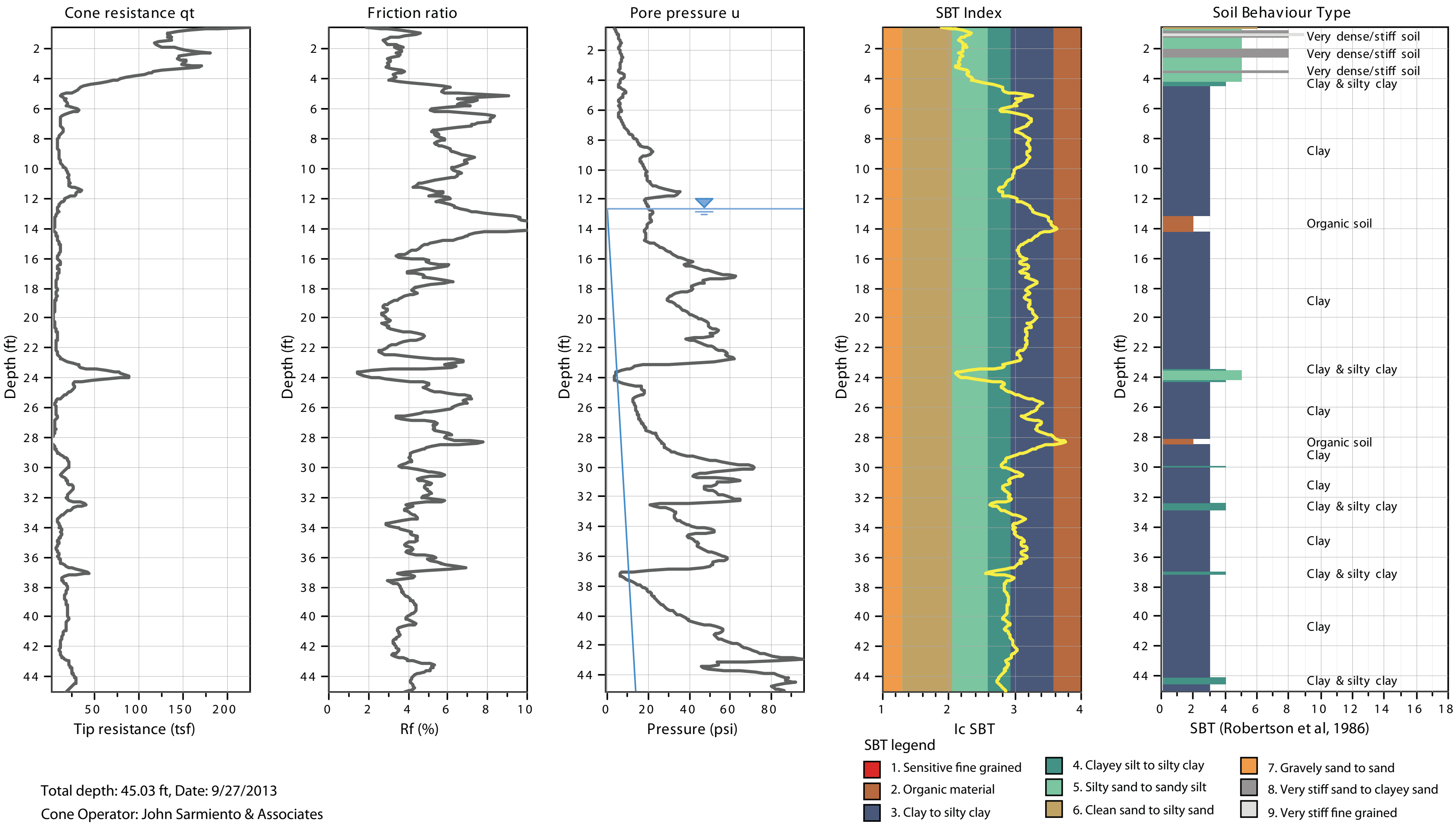



Total depth: 45.03 ft, Date: 9/27/2013

Cone Operator: John Sarmiento & Associates

Groundwater measured at 11.8 feet bgs.

| | |
|---|---|
| **SUNNYVALE VA FACILITY**<br>**1080 INNOVATION WAY**<br>Sunnyvale, California | **CONE PENETRATION TEST RESULTS**<br>**CPT-1** |
| ROCKRIDGE GEOTECHNICAL | Date 05/19/13 \| Project No. 13-591 \| Figure A-1 |

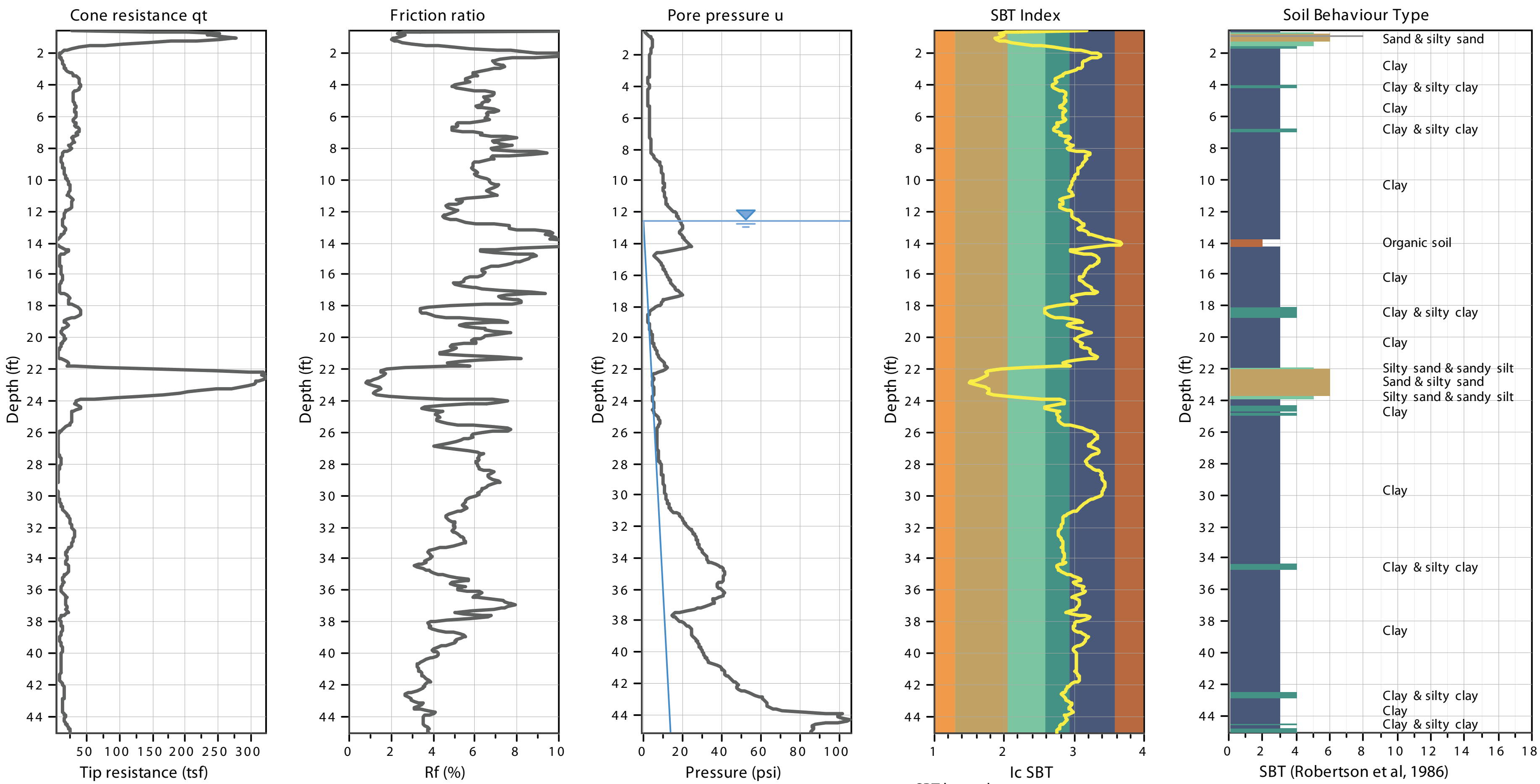



Total depth: 45.03 ft, Date: 9/27/2013

Cone Operator: John Sarmiento & Associates

Groundwater measured at 12.6 feet bgs.

| SUNNYVALE VA FACILITY<br>1080 INNOVATION WAY<br>Sunnyvale, California | CONE PENETRATION TEST RESULTS<br>CPT-2 | | |
|---|---|---|---|
| ROCKRIDGE GEOTECHNICAL | Date 05/19/13 | Project No. 13-591 | Figure A-2 |




SBT legend

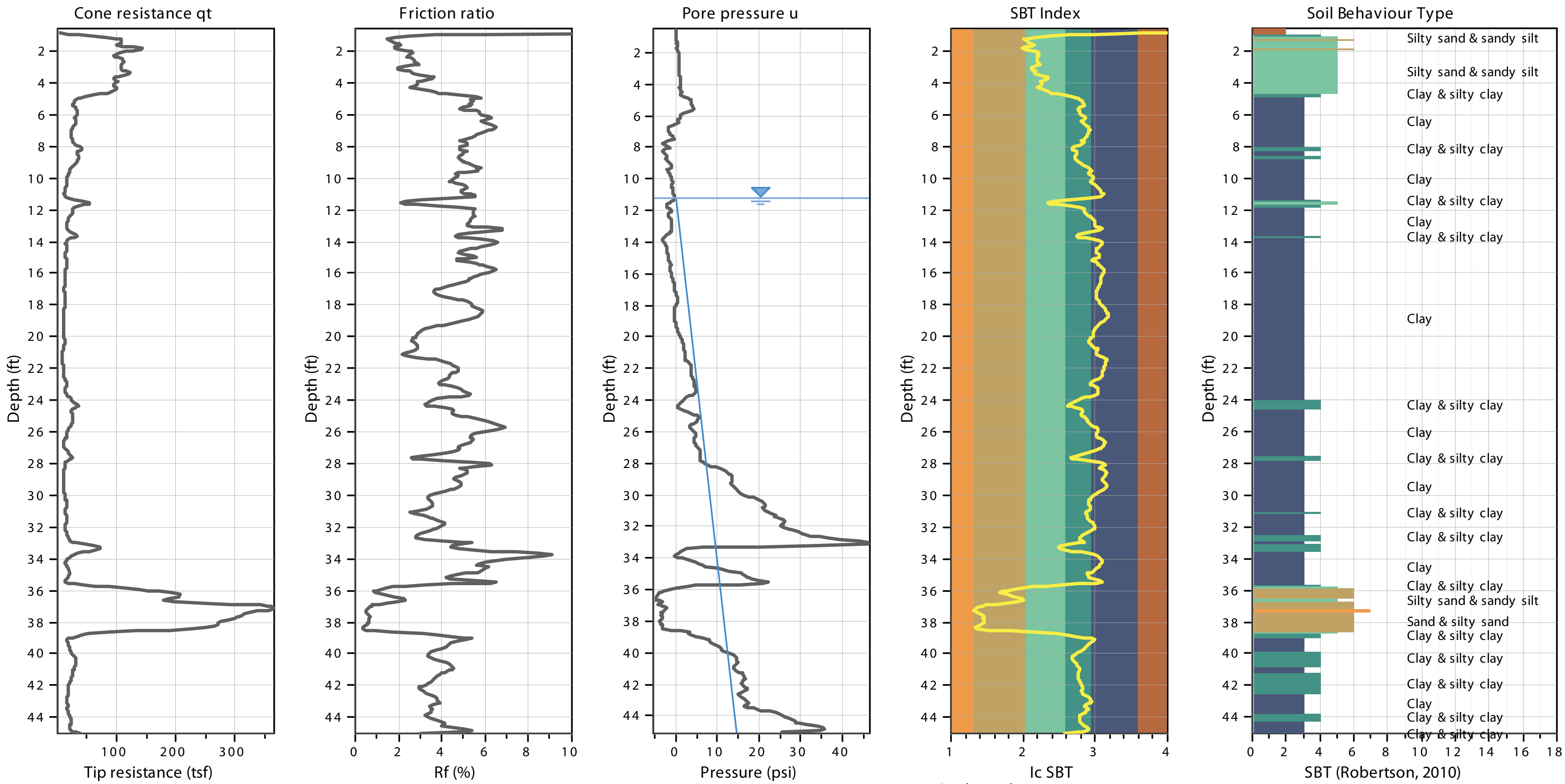
- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

Total depth: 45.07 ft, Date: 9/27/2013

Cone Operator: John Sarmiento & Associates

Groundwater measured at 12.6 feet bgs.

| SUNNYVALE VA FACILITY<br>1080 INNOVATION WAY<br>Sunnyvale, California | CONE PENETRATION TEST RESULTS<br>CPT-3 | | |
|---|---|---|---|
| ROCKRIDGE GEOTECHNICAL | Date 05/19/13 | Project No. 13-591 | Figure A-3 |

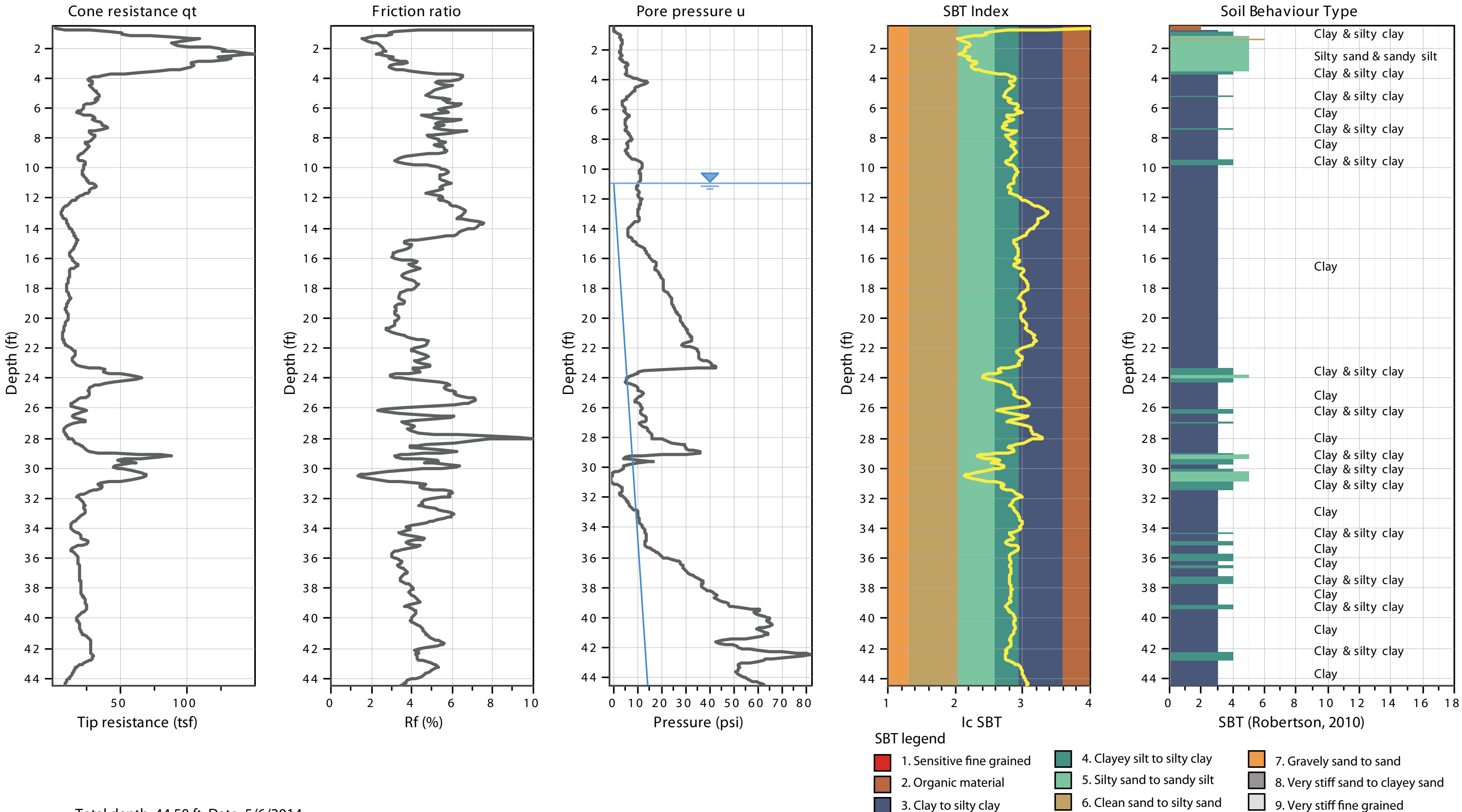



SBT legend

| | | |
|---|---|---|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

Total depth: 45.02 ft, Date: 5/6/2014
Cone Operator: John Sarmiento & Associates
Groundwater measured at 11.2 feet bgs.

| | |
|---|---|
| **SUNNYVALE VA FACILITY**<br>**1080 INNOVATION WAY**<br>Sunnyvale, California | **CONE PENETRATION TEST RESULTS**<br>**CPT-4** |
| ROCKRIDGE GEOTECHNICAL | Date 05/19/14 \| Project No. 13-591 \| Figure A-4 |




Total depth: 44.50 ft, Date: 5/6/2014

Cone Operator: John Sarmiento & Associates

Groundwater measured at 11.0 feet bgs.

**SUNNYVALE VA FACILITY**
**1080 INNOVATION WAY**
Sunnyvale, California

ROCKRIDGE GEOTECHNICAL

# CONE PENETRATION TEST RESULTS
## CPT-5

| Date 05/19/14 | Project No. 13-591 | Figure A-5 |
|---|---|---|

**APPENDIX B**
**Summary of Liquefaction Analyses**

**Rockridge Geotechnical, Inc.**
270 Grand Avenue
Oakland, CA 94610

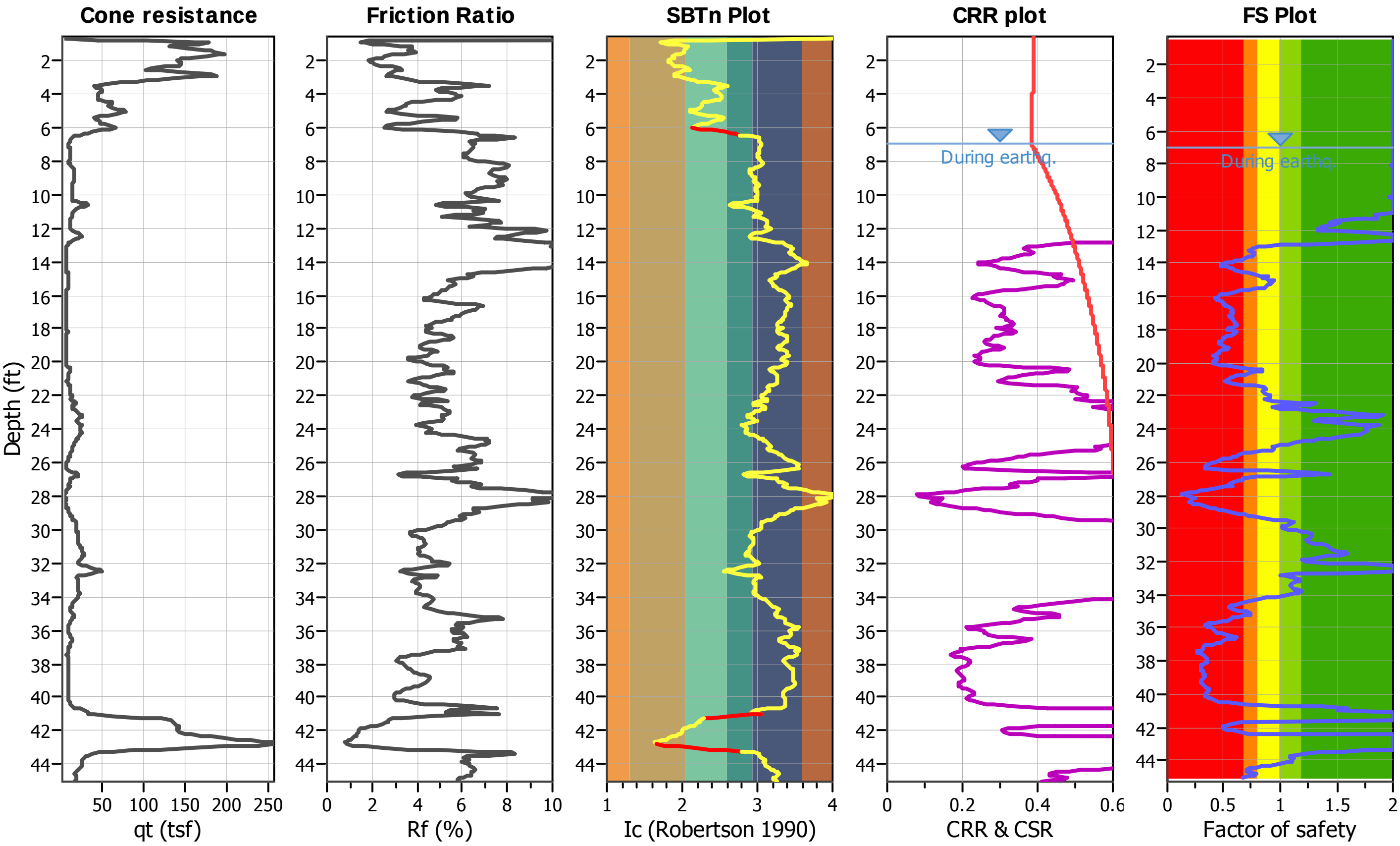
## LIQUEFACTION ANALYSIS REPORT

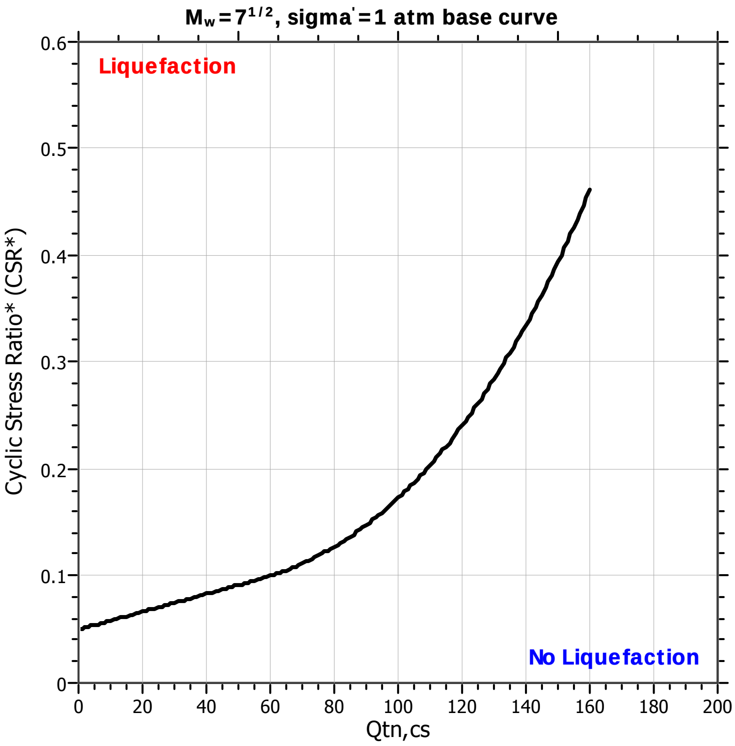
**Project title : 13-591 Sunnyvale VA** **Location : Sunnyvale, CA**

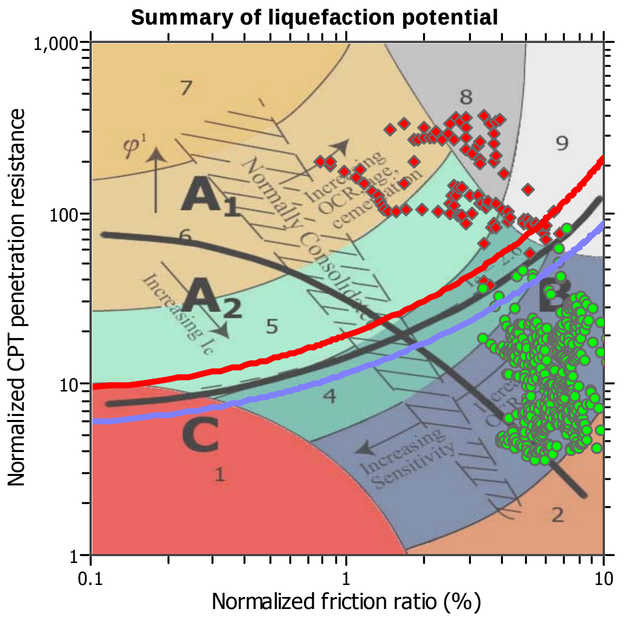
**CPT file : SUNVA-1**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 11.80 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 7.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 8.05 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.50 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

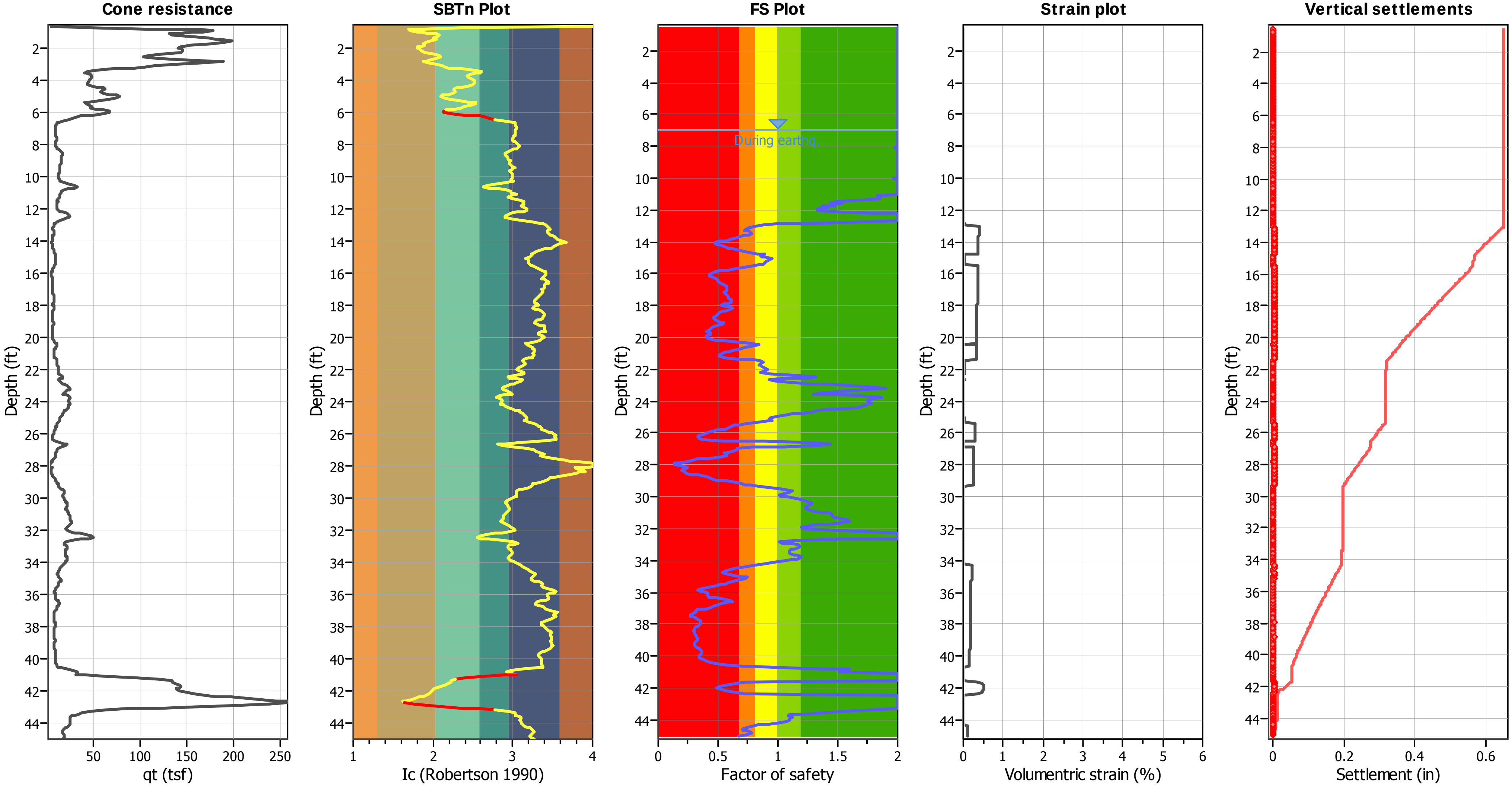



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction 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 |
| Volumentric strain: | Post-liquefaction volumentric strain |

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Oakland, CA 94610

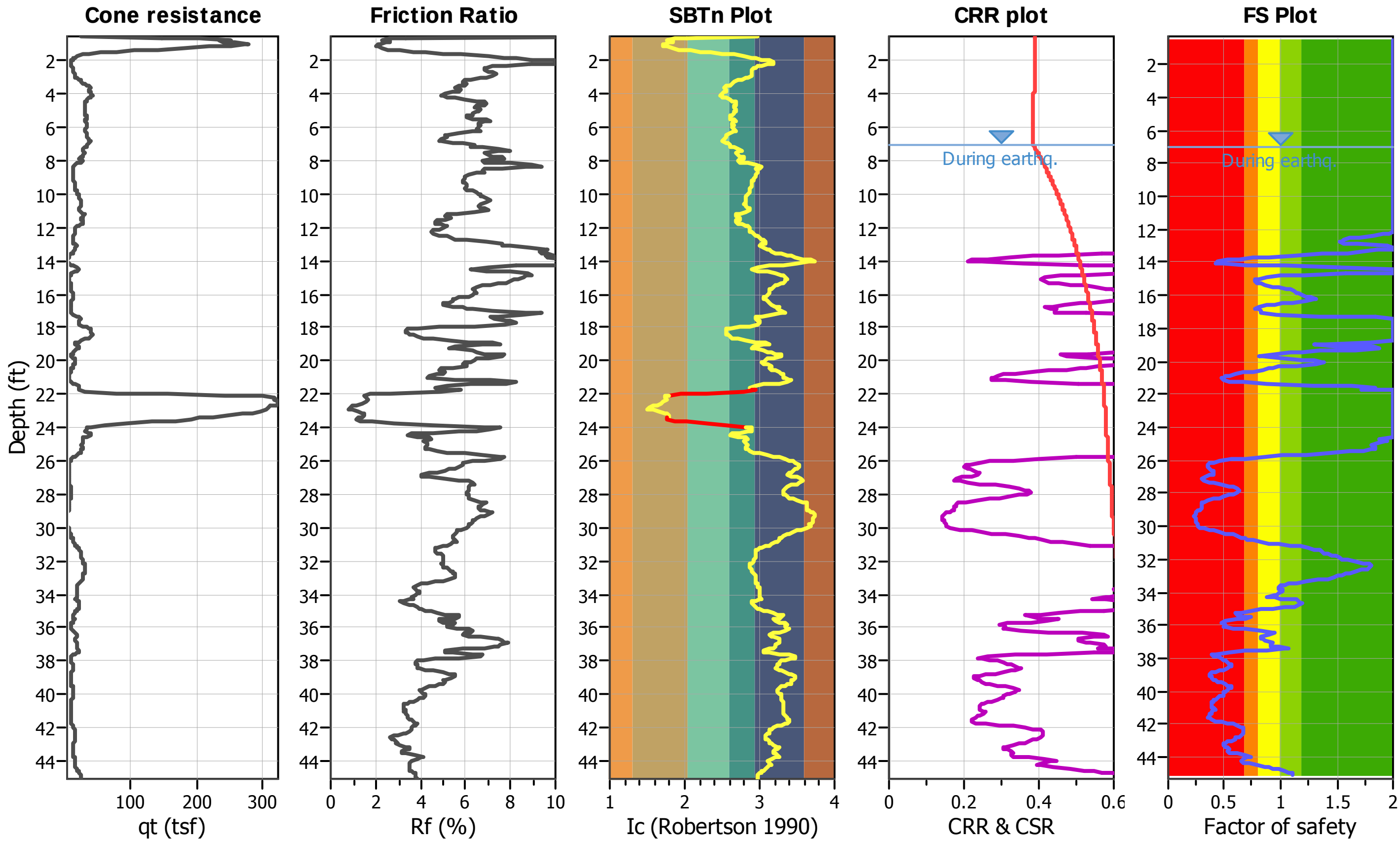
## LIQUEFACTION ANALYSIS REPORT

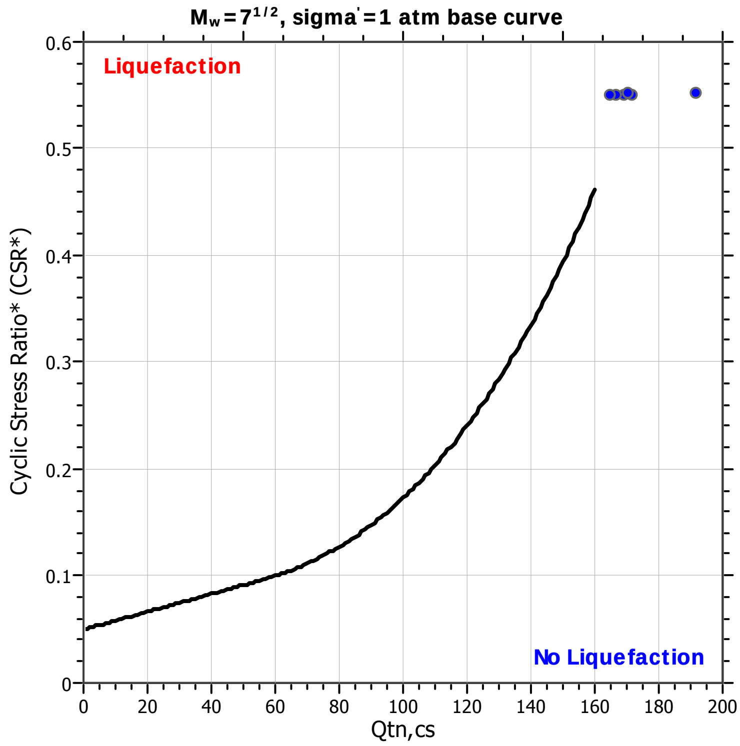
**Project title : 13-591 Sunnyvale VA** **Location : Sunnyvale, CA**

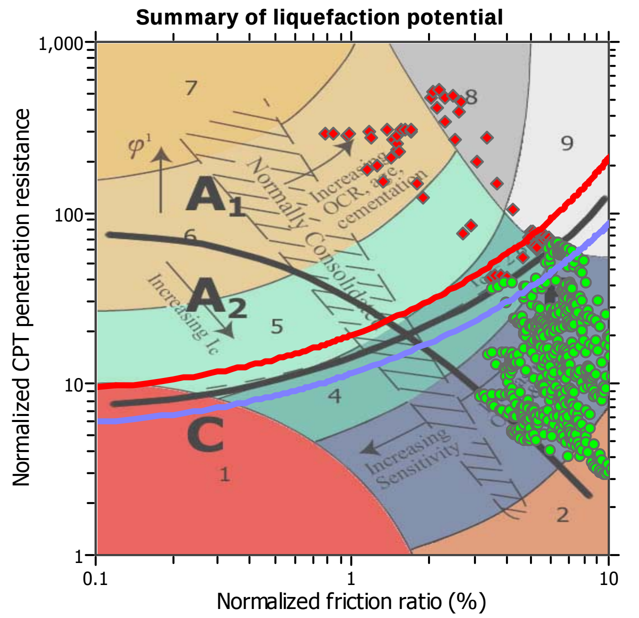
**CPT file : SUNVA-3**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 12.60 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 7.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 8.05 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.50 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

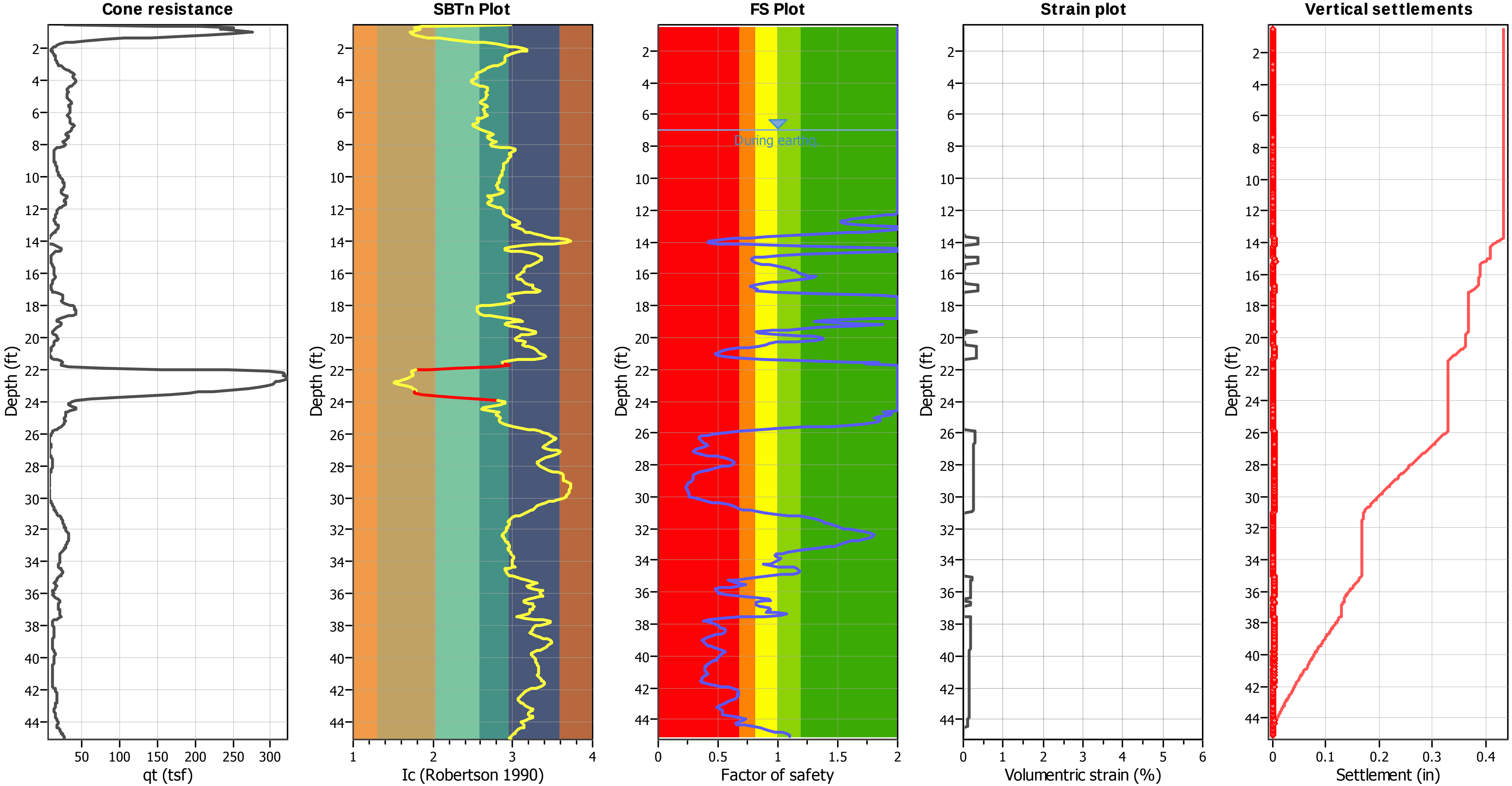



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction 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
Volumentric strain: Post-liquefaction volumentric strain

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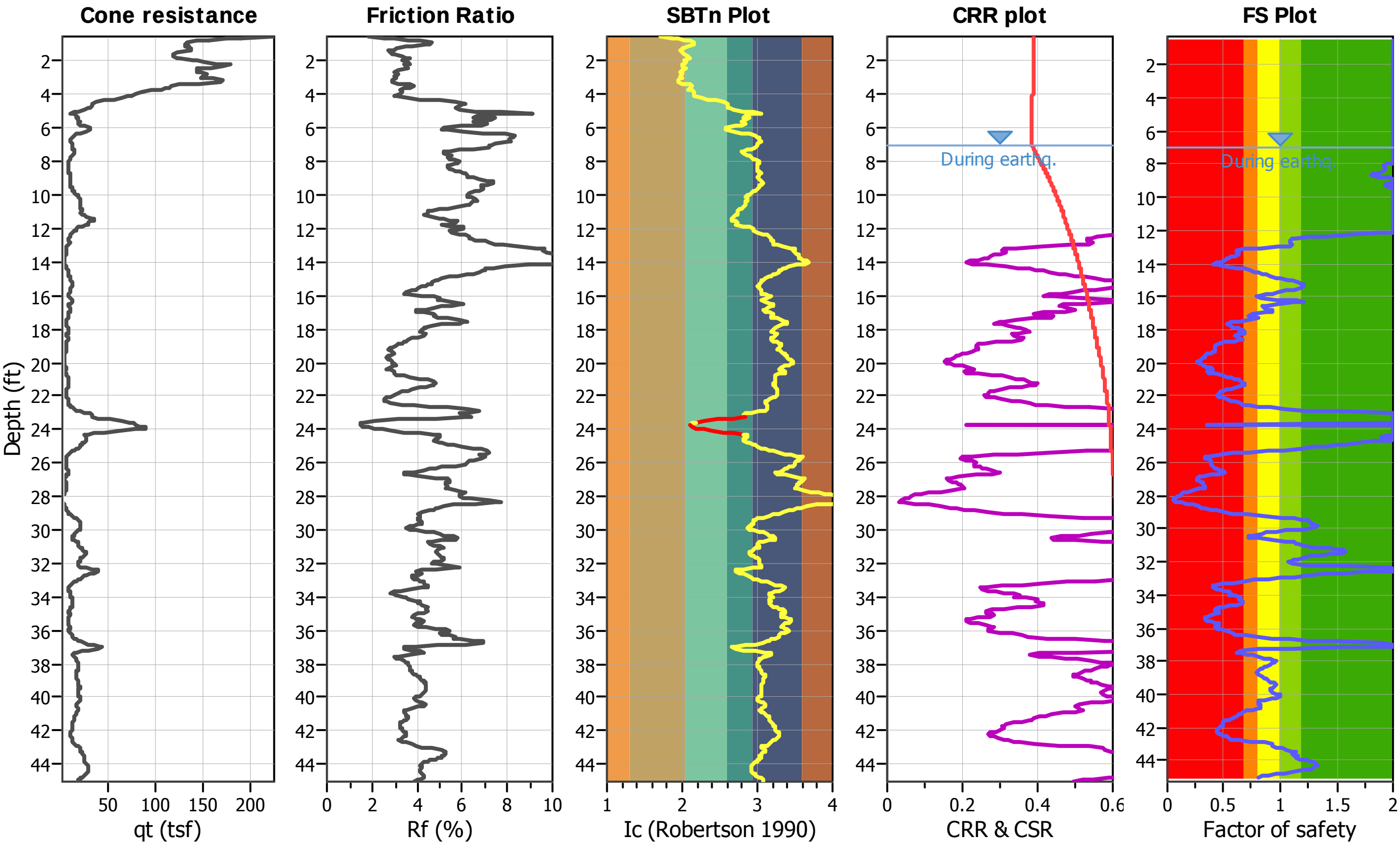
## LIQUEFACTION ANALYSIS REPORT

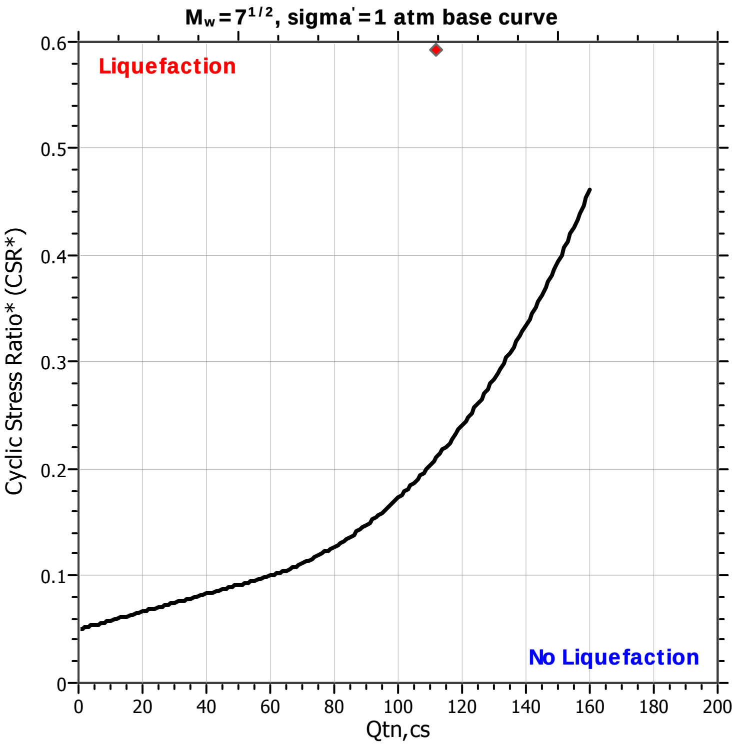
**Project title : 13-591 Sunnyvale VA** **Location : Sunnyvale, CA**

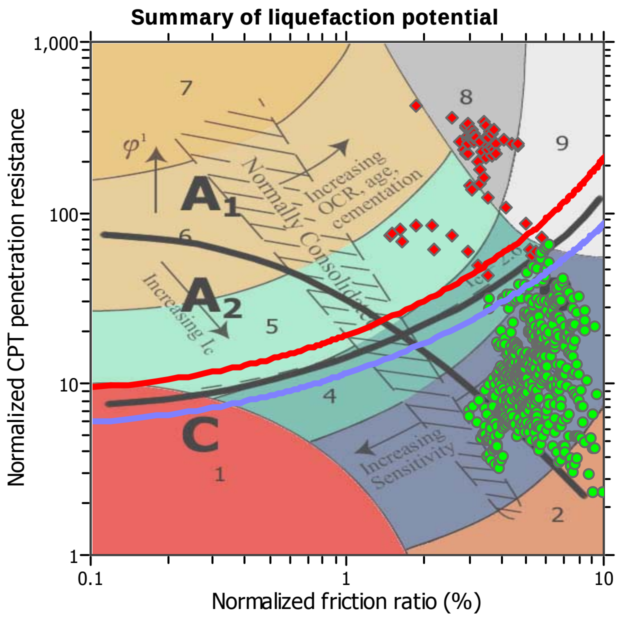
**CPT file : SUNVA-2**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 12.60 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 7.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 8.05 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.50 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

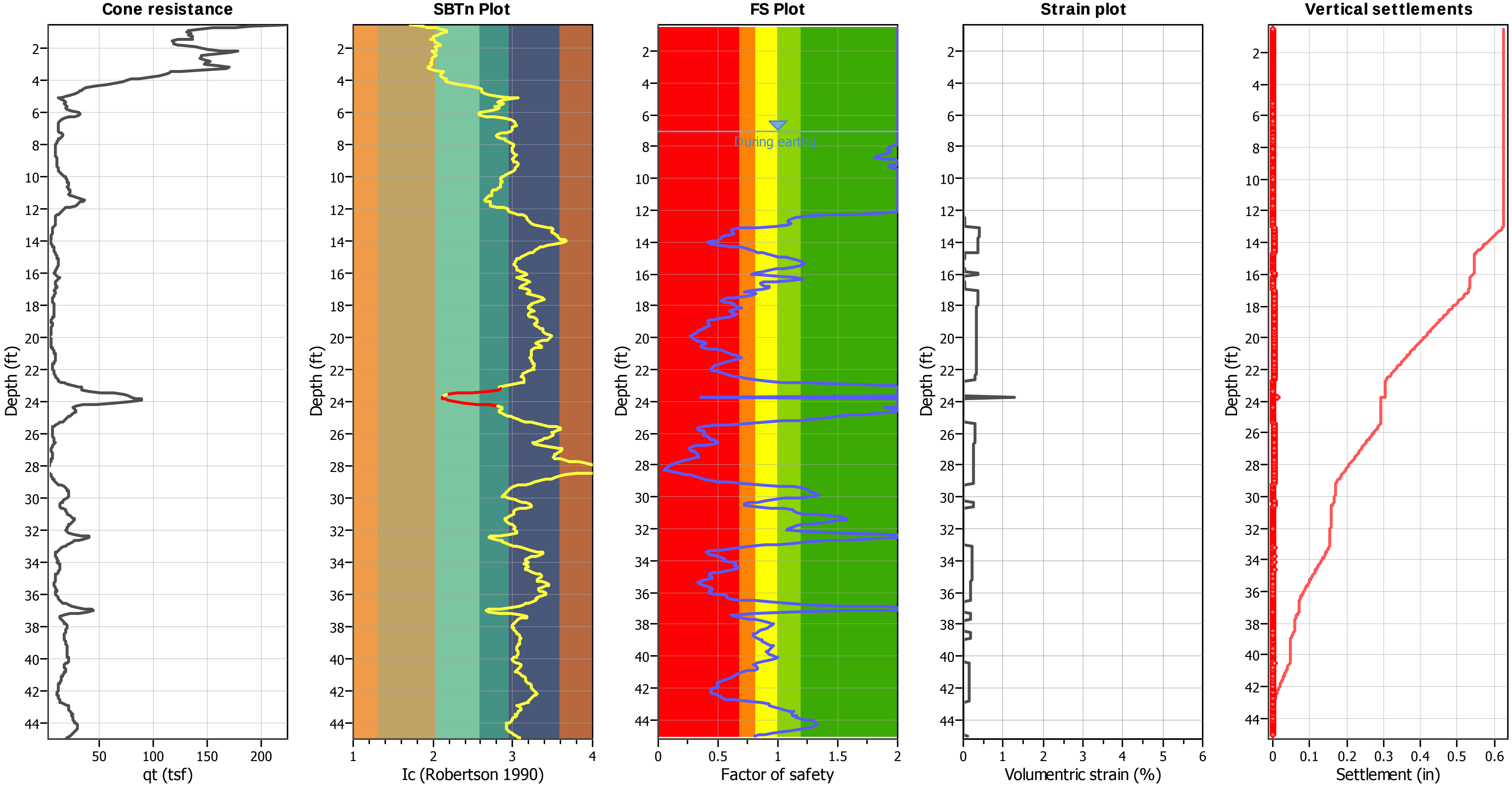



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction 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 |
| Volumentric strain: | Post-liquefaction volumentric strain |

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Oakland, CA 94610

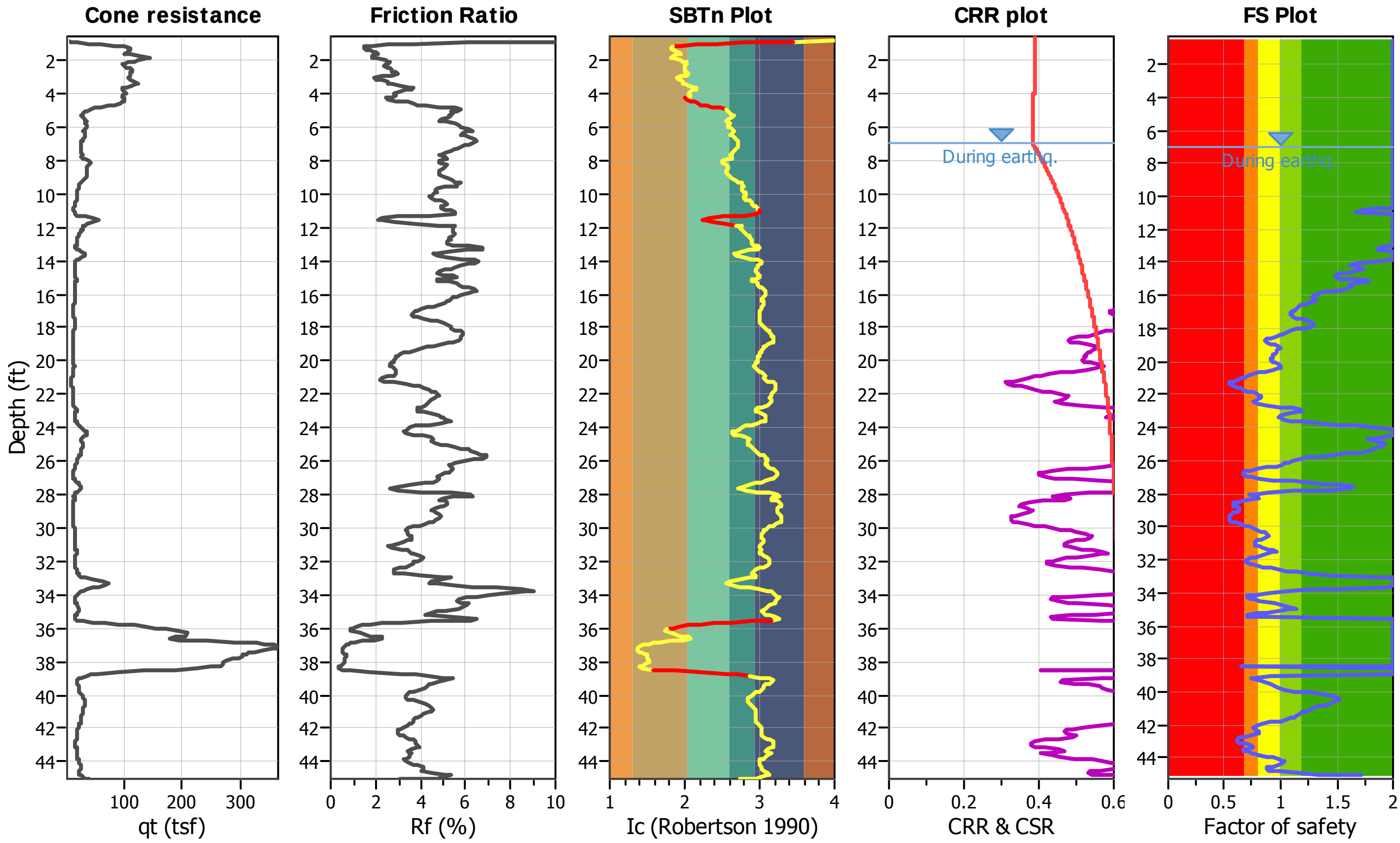
## LIQUEFACTION ANALYSIS REPORT

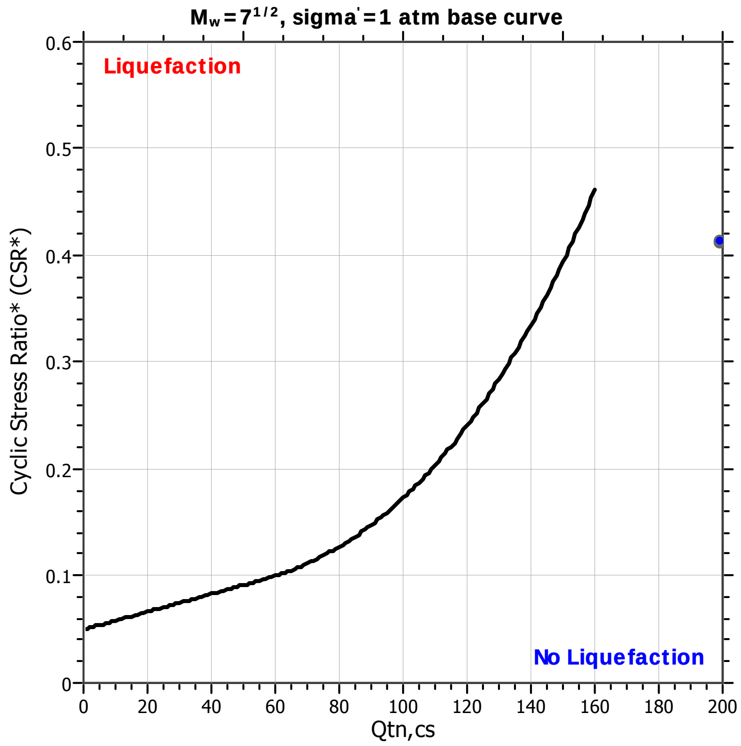
**Project title : 13-591 Sunnyvale VA** **Location : Sunnyvale, CA**

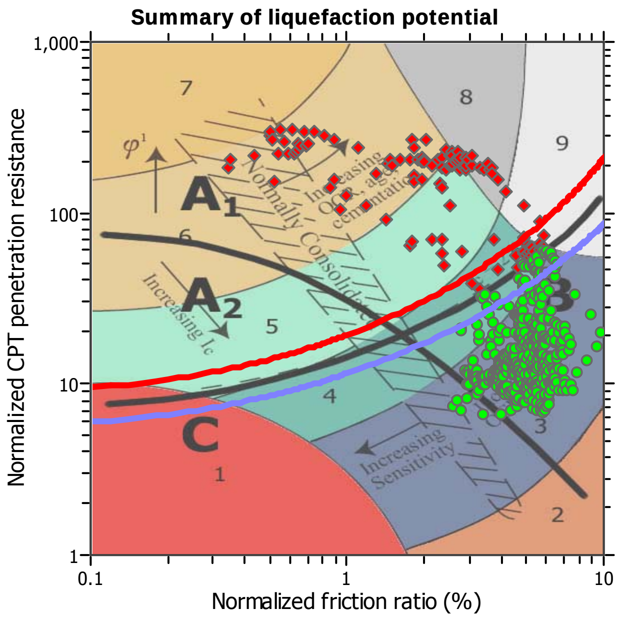
**CPT file : SUNVA-4**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 11.20 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 7.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 8.05 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.50 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

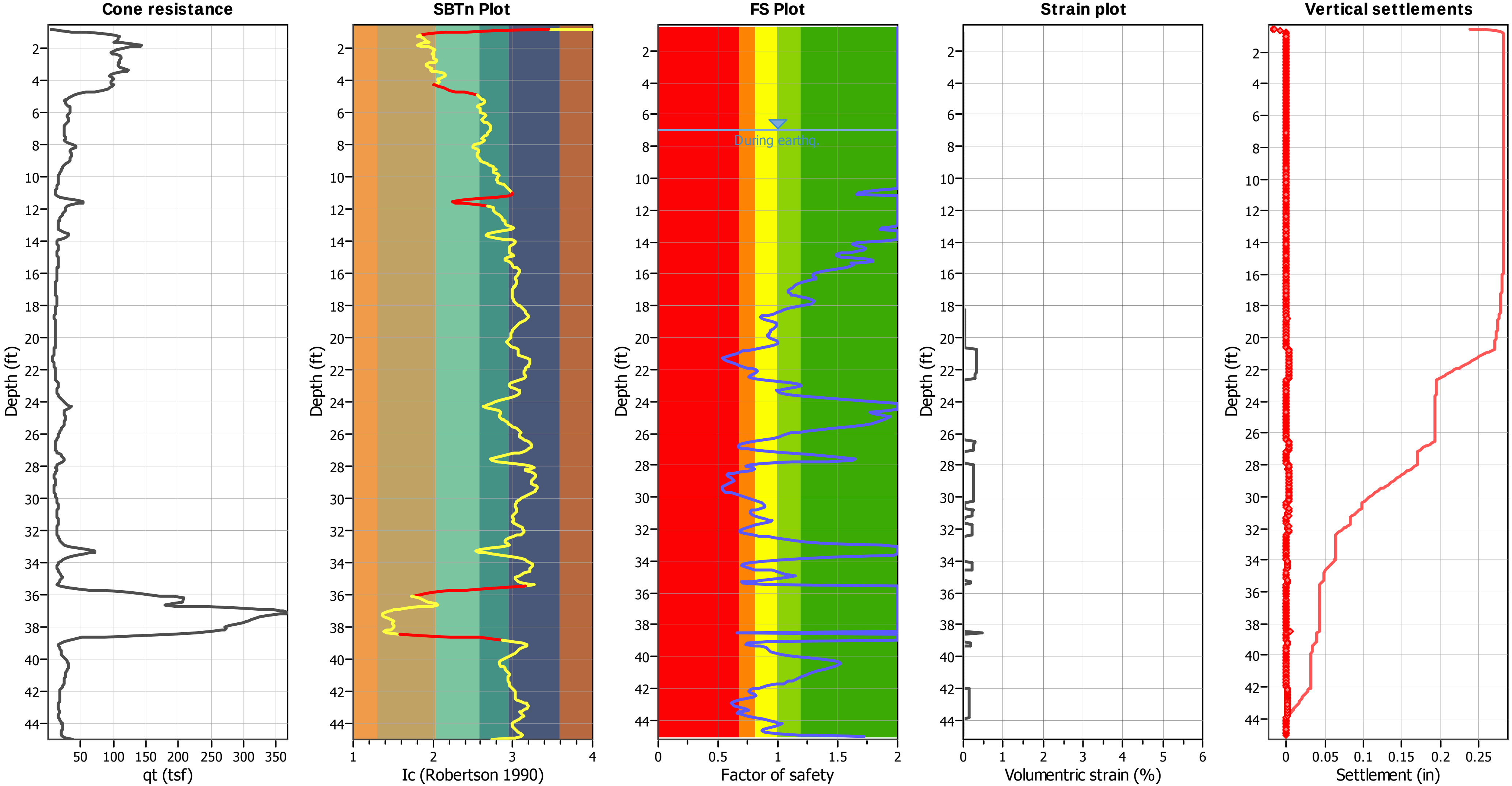



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## Estimation of post-earthquake settlements




**Abbreviations**

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| Volumentric strain: | Post-liquefaction volumentric strain |

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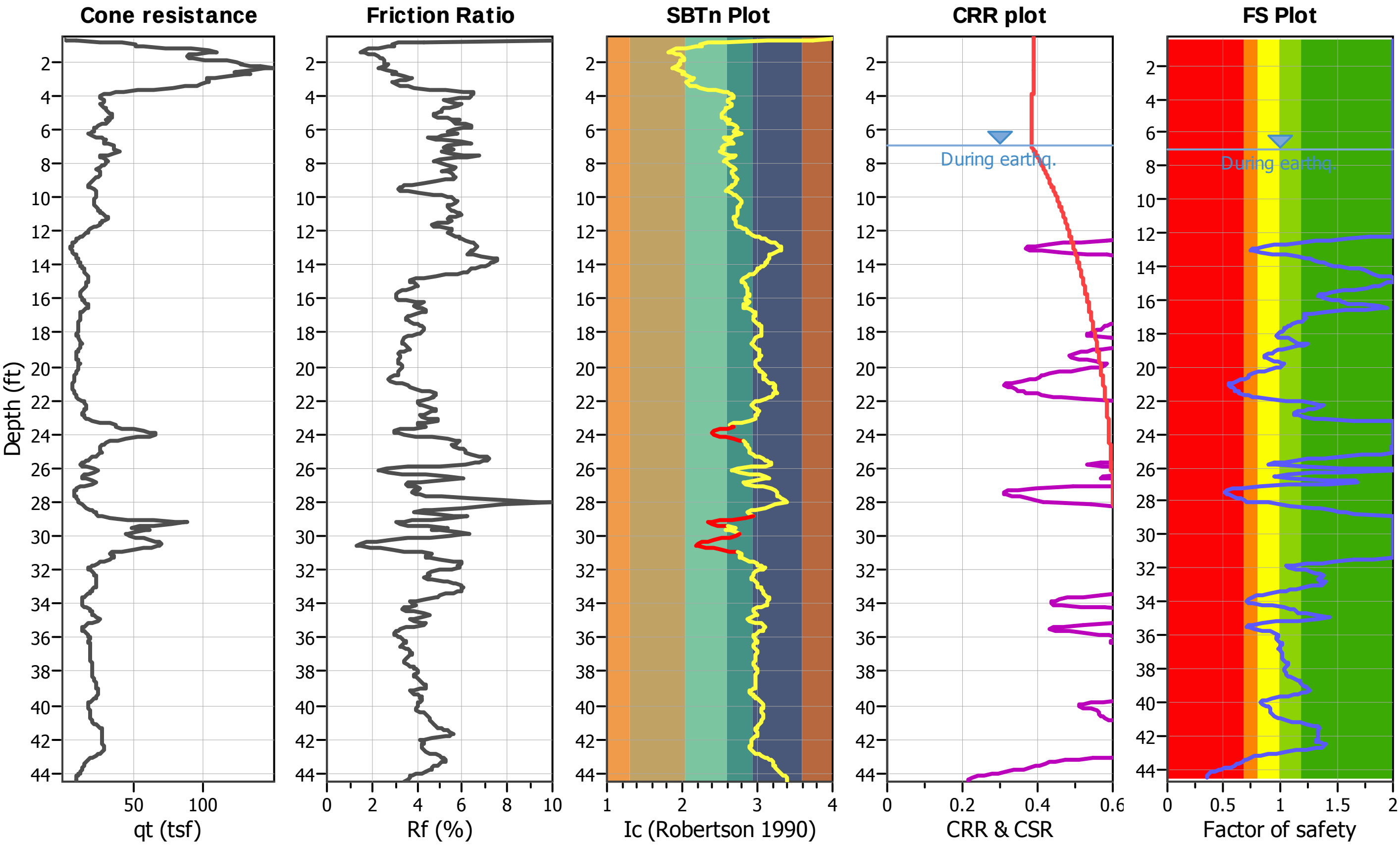
# LIQUEFACTION ANALYSIS REPORT

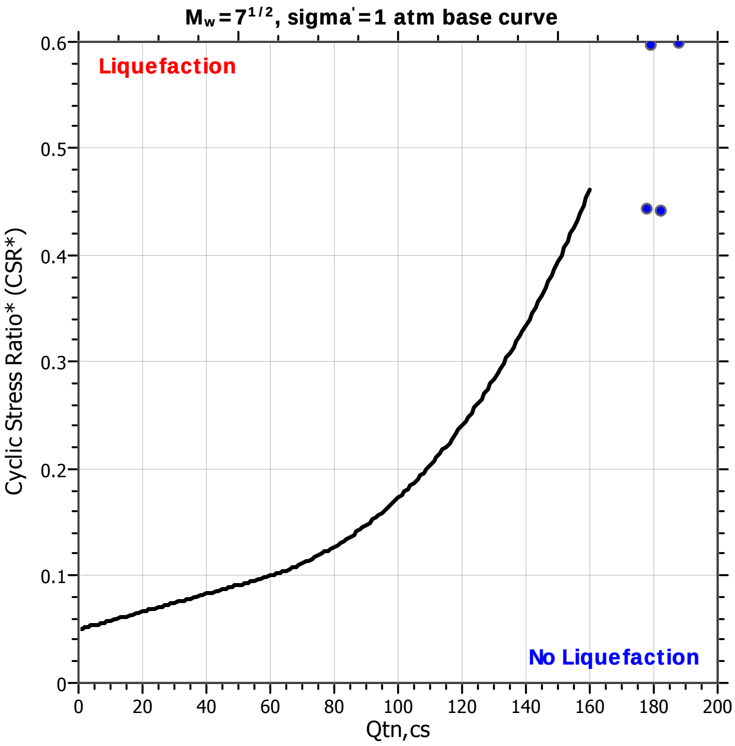
**Project title : 13-591 Sunnyvale VA** **Location : Sunnyvale, CA**

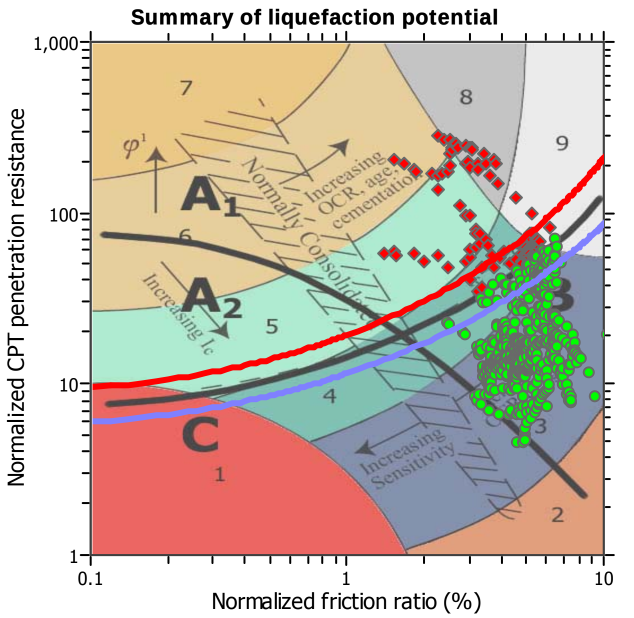
**CPT file : SUNVA-5**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 11.00 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 7.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 8.05 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.50 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

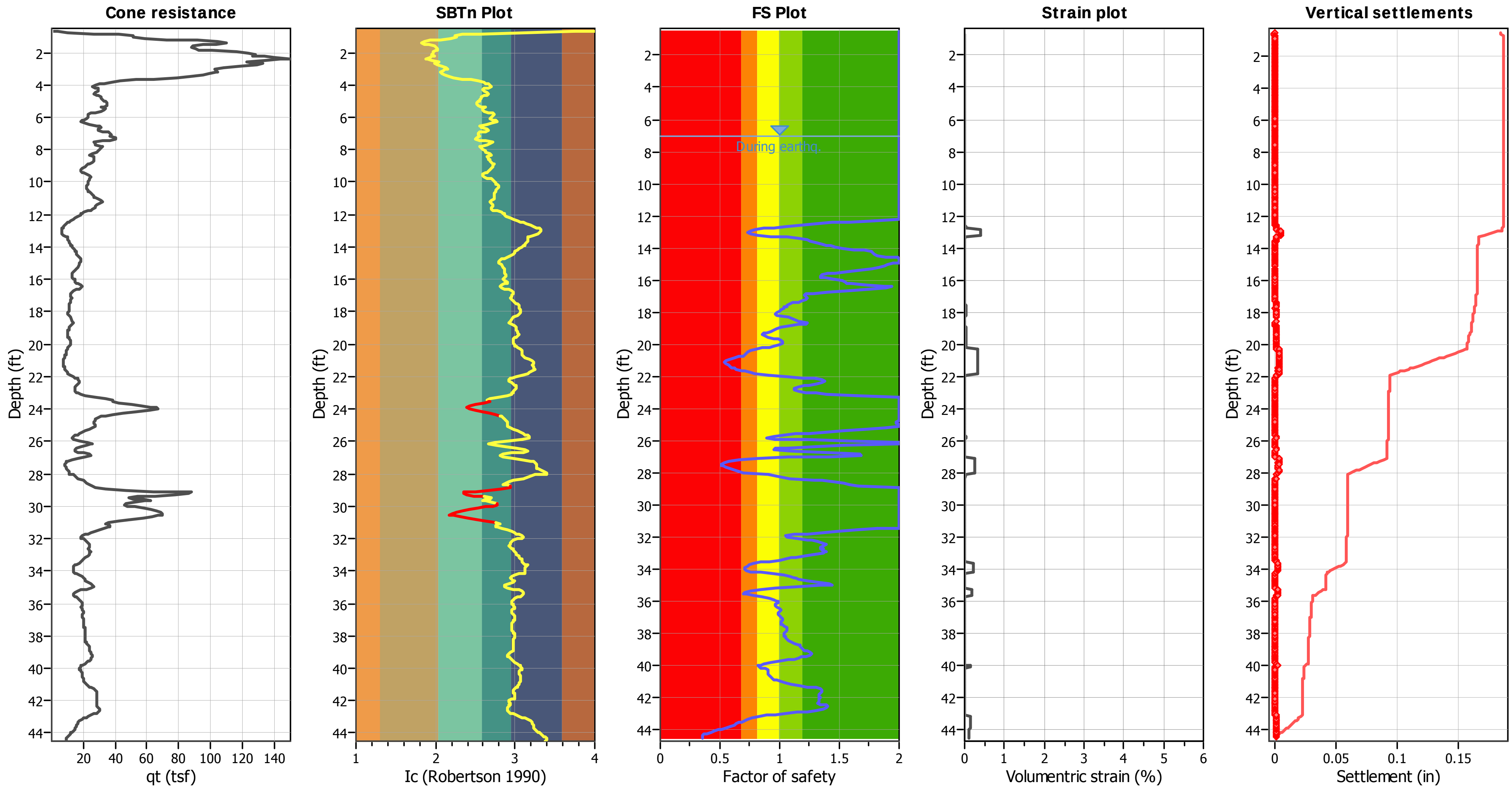



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction 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 |
| Volumentric strain: | Post-liquefaction volumentric strain |