GEOTECHNICAL EVALUATION
CITY CENTER SAN RAMON
2678 BISHOP DRIVE
SAN RAMON, CALIFORNIA

PREPARED FOR:
Sunset Development Company
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San Ramon, California 94583

PREPARED BY:
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January 18, 2016
Project No. 402676001

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Mr. Rob Elliott
Sunset Development Company
2600 Camino Ramon, Suite 201
San Ramon, California 94583

Subject: Geotechnical Evaluation
City Center San Ramon
2678 Bishop Drive
San Ramon, California

Dear Mr. Elliott:

In accordance with your authorization, we have performed a geotechnical evaluation for the City Center San Ramon project to be located at 2678 Bishop Drive in San Ramon, California. This report presents our geotechnical findings, conclusions and recommendations for design and construction of the proposed improvements.

As an integral part of our role as the geotechnical engineer-of-record, we request the opportunity to review the construction plans and to provide follow-up construction observation and testing services.

We appreciate the opportunity to be of service on this project.

Sincerely,
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1. INTRODUCTION

In accordance with your request, Ninyo & Moore has performed a geotechnical evaluation for the City Center San Ramon project at 2678 Bishop Drive in San Ramon, California (Figure 1). The purpose of our study was to evaluate the subsurface conditions at the site to provide recommendations for the design and construction of the project improvements.

2. SCOPE OF SERVICES

Ninyo & Moore's scope of services included review of pertinent geologic and geotechnical background data, performance of a site reconnaissance, subsurface evaluation, geotechnical laboratory testing, and preparation of recommendations for the project. Specifically, we performed the following tasks:

- Reviewed readily available background materials, including topographic maps, geologic maps, fault location maps, and seismic hazard maps.
- Conducted a site reconnaissance to mark the subsurface exploration locations for utility clearance and to observe site conditions including topographic features, drainage characteristics, and surficial geologic conditions.
- Coordinated with Underground Service Alert (USA) to locate underground utilities in the vicinity of the subsurface exploration locations.
- Obtained a boring permit from Contra Costa County.
- Performed a subsurface evaluation consisting of six (6) cone penetration test (CPT) soundings and six (6) small-diameter borings. Cone tip resistance, sleeve friction, and pore pressure were electronically measured and recorded at vertical intervals of approximately 2 inches to evaluate the soil behavior type index of the materials encountered. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings, and collected bulk and relatively undisturbed samples for geotechnical laboratory testing. The borings were backfilled with Portland cement grout in accordance with the boring permit.
- Performed geotechnical laboratory testing on soil samples collected during the subsurface study to evaluate in-situ moisture and density, percentage of soil particles finer than the No. 200 sieve, particle size distribution, Atterberg limits, consolidation characteristics, soil corrosivity, expansion index, unconfined compressive strength, and direct shear strength.

- Compilation and engineering analysis of the field and laboratory data from our subsurface evaluation and the findings from our geologic review.
- Preparation of this geotechnical evaluation report presenting our findings, conclusions and recommendations for design and construction of the City Center San Ramon development.

3. SITE AND PROJECT DESCRIPTION

The proposed City Center San Ramon will be constructed on a parcel located at 2678 Bishop Drive in San Ramon, California (Figure 1). The parcel is bounded to the north by Bishop Drive, to the east by Camino Ramon, to the south by Bollinger Canyon Road, and to the west by Sunset Drive. The project site covers approximately 14½ acres and is relatively flat with elevations ranging between approximately 437 and 447 feet above Mean Sea Level (MSL). The site is currently developed with four, two-story commercial office buildings around a central landscaped area with asphalt paved parking around the perimeter of the parcel.

The proposed project will demolish the existing buildings and construct a two to three-story building at the existing grade with two levels for retail space and three levels of parking for a total footprint area of approximately 227,000 square feet (Figure 2). The anticipated column loads will range between approximately 130 kips and 530 kips for dead load, and between approximately 160 kips and 670 kips for dead plus reduced live loads.

4. FIELD EXPLORATION AND LABORATORY TESTING

Our geotechnical field exploration of the site included a site reconnaissance performed on October 27, 2015 and a subsurface exploration conducted on November 10 and 11, 2015. The subsurface exploration for the proposed City Center San Ramon consisted of drilling, logging, and sampling six (6) exploratory borings using a truck-mounted drill rig and six (6) CPT soundings. Boring B-1 was advanced to a depth of approximately 65 feet using mud rotary drilling. Borings B-2 through B-6 were advanced to depths ranging between approximately 25 and 35 feet using hollow stem flight augers. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings, and collected bulk and relatively undisturbed

samples for geotechnical laboratory testing. The locations of the borings are presented on Figure 2. Descriptions of the subsurface materials encountered are presented in the following sections. Detailed logs of the borings are presented in Appendix A.

The CPT soundings were advanced using a truck-mounted rig with a 20-ton reaction capacity to depths ranging between approximately 25 and 75 feet below the existing ground surface. Cone tip resistance, sleeve friction, and pore pressure were electronically measured and recorded at vertical intervals of approximately 2 inches to evaluate the soil behavior type index of the materials encountered. The locations of the soundings are presented on Figure 2. The logs of the soundings are presented in Appendix B.

The laboratory testing for this study included tests to evaluate in-situ moisture and density, percentage of soil particles finer than the No. 200 sieve, particle size distribution, Atterberg limits, consolidation characteristics, soil corrosivity, expansion index, unconfined compressive strength, and direct shear strength. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The results of the other laboratory tests performed are presented in Appendix C.

5. GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional and site geology, subsurface soils, and groundwater conditions pertinent to the study area are provided in the following sections.

5.1. Regional Geologic Setting

The project alignment is located on the eastern side of the San Francisco Bay in the Coast Ranges Province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys stretching approximately 600 miles from the Oregon border to the Santa Ynez River. They are formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill

structural valleys and line continental margins. The San Francisco Bay Area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.

5.2. Site Geology

Published geologic maps (Dibblee, 2005; Helley & Graymer, 1997; Wagner, Bortungo, and McJunkin, 1991) indicate that the study area is underlain by Holocene alluvium (Figure 4). Helley & Graymer (1997) indicate that the site is underlain by Holocene alluvial fan and fluvial deposits. The alluvial fan deposits are described as brown or tan, medium dense to dense, gravelly sand or sandy gravel that generally grades upward to sandy or silty clay. The fluvial deposits, which are found near distal fan edges, are described as typically brown, medium dense sand that fines upward to sandy or silty clay. Dibblee (2005) indicates that the hills west and east of the site are generally composed of interbedded pebble conglomerate, sandstone, and claystone of the Pliocene Orinda Formation which is also known as the Green Valley Formation. The borings and soundings for this study generally encountered alluvium.

5.3. Subsurface Conditions

Materials encountered during our subsurface evaluation generally consisted of asphalt pavement and alluvium. Generalized descriptions of the units encountered are provided in the subsequent sections. More detailed descriptions are presented on the boring logs in Appendix A.

5.3.1. Asphalt Concrete Pavement

Soundings CPT-1 through CPT-6 were advanced through asphalt pavements. The pavement sections encountered in our borings consisted of approximately 6 inches of

asphalt concrete. Variations in the thickness of the asphalt concrete may be encountered due to past maintenance, utility work, or other factors.

5.3.2. Alluvium

Alluvium was encountered in the borings and soundings below the pavement, where present, to the depths explored. The alluvium encountered generally consisted of light brown to dark brown, dry to wet, stiff to hard, clay, silty clay, sandy clay, and clayey silt; and medium dense to dense, sand, silt, and gravel with trace to some clay.

5.4. Groundwater

Groundwater was encountered during our subsurface exploration. The depth to groundwater measured in Borings B-1, B-2, B-3, and B-5 after drilling ranged between approximately 16 feet (Boring B-2) and 17½ feet (Boring B-3). However, groundwater may rise to a higher level than observed during our subsurface exploration due to the relatively low seepage rate in clay and the limited time for observation. Fluctuations in the groundwater level across the site and over time may occur due to seasonal precipitation, variations in topography or subsurface hydrogeologic conditions, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration. Piezometers can be installed to further evaluate the depth to groundwater in the study area and fluctuation in groundwater levels.

6. GEOLOGIC HAZARDS AND GEOTECHNICAL CONSIDERATIONS

This study evaluated a number of geologic hazards and geotechnical considerations relevant to the proposed construction, including seismic hazards, static settlement, soil corrosivity, expansive soils, and excavation characteristics. These issues are discussed in the following subsections.

6.1. Seismic Hazards

The study alignment is within a seismically active region. The seismic hazards considered in this study include the potential for ground surface rupture and ground shaking due to seismic activity, seismically induced liquefaction, and dynamic settlement. These potential hazards are discussed in the following subsections.

6.1.1. Strong Ground Motion

Based on historic activity, the potential for future strong ground motion in the project area is considered significant. Design recommendations for structures to address seismic shaking are presented in Section 8.2. The 2013 California Building Code (CBC) specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. A site-specific ground motion hazard analysis was performed in accordance with Section 21.5 of ASCE 7-10 to evaluate the MCE_G peak ground acceleration. The results of our site-specific ground motion hazard analysis indicate that the MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) is 0.976g. The assumptions and models utilized for this analysis are listed on Figure 5.

6.1.2. Faulting and Ground Surface Rupture

There are numerous recognized faults in northern California. Selected characteristics, as evaluated by the Working Group on California Earthquake Predictions (WGCEP, 2008), for recognized and postulated faults (Caltrans, 2012a) near the site are presented in Table 1. The fault characteristics in the table are presented in order of decreasing peak ground acceleration (PGA) based on a deterministic seismic hazard analysis utilizing the Chiou & Youngs (2008) and Campbell & Bozorgnia (2008) attenuation relationships.

Table 1 – Parameters for Nearby Faults

Fault Name (Section)	ID	Type	Max Moment Magnitude	Distance to Site (kilometers)
Calaveras (North)	130	Strike Slip	6.9	0.58
Pleasanton	135	Strike Slip	6.6	0.57
Hayward (South)	137	Strike Slip	7.3	14.8
Mount Diablo Thrust	129	Reverse	6.6	9.43
Hayward (North)	123	Strike Slip	7.3	16.0
Clayton	125	Strike Slip	6.9	14.7
Great Valley 06 (Midland)	116	Reverse	6.8	18.2
Greenville (North)	131	Strike Slip	6.9	15.3
San Andreas (Peninsula)	134	Strike Slip	8.0	44.4
Concord	122	Strike Slip	6.6	15.4

The site is not located within an Alquist-Priolo Fault-Rupture Hazard Zone established by the state geologist (CDMG, 1982) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the California Geological Survey (CGS), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,000 years (CGS, 2007). The closest known fault considered to be active is the Calaveras fault to the west. The fault rupture hazard zone associated with the Calaveras fault is 2,300 feet from the site (CDMG, 1982).

Based on our review of the referenced geologic maps, there are no known faults at the project site and the site is not located within a Fault-Rupture Hazard Zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

6.1.3. Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soil of low plasticity (liquefaction) or in wet, sensitive, cohesive soil (strain softening). Liquefaction is generally not a concern at depths more than 50 feet below ground surface. Liquefaction and strain softening can

result in a loss of foundation bearing capacity or lateral spreading of sloping or unconfined ground. Liquefaction can also generate sand boils leading to subsidence at the ground surface.

Regional studies of liquefaction susceptibility (Witter et al., 2006) indicate that the liquefaction susceptibility at the site is moderate.

We encountered deposits of medium to very dense granular soil in our borings and soundings. To further quantify the potential for liquefaction, we performed an analysis following the method documented by Youd et al. (2001) to evaluate liquefaction susceptibility using CPT data collected during our subsurface exploration. The analysis assumed a groundwater depth of 16 feet and considered a PGA of 0.976g corresponding to a Magnitude 6.9 earthquake on the Calaveras fault. The results of our analysis (presented in Appendix D) indicate that a few relatively thin layers of sandy and silty soil below the assumed groundwater level will liquefy under the considered ground motion based on a factor of safety against liquefaction of less than one. The liquefiable layers are not more than 10 inches thick with a total combined thickness of less than 2 feet. In addition, the liquefiable layers are generally more than 20 feet deep and were not encountered at a consistent elevation. For these reasons, we do not consider the impact of liquefaction and related hazards including loss of foundation bearing capacity, lateral spreading, and sand-boil-induced ground subsidence, to be significant concerns for this site.

Estimates of undrained and remolded shear strength based on CPT tip resistance and sleeve friction, respectively, indicate that the cohesive soils during our subsurface exploration are not particularly sensitive. As such we do not regard seismically induced strain-softening behavior as a design consideration.

6.1.4. Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil leading to surficial settlements. Dynamic settlement is not limited to the near surface environment and may occur in both dry and saturated sand and silt. Cohesive soil is not typically susceptible to dynamic settlement.

During our subsurface exploration, we encountered medium to very dense granular soil. We evaluated the potential for dynamic settlement based on the procedure described by Zhang et al (2002) for saturated soil and by Robertson and Shao (2010) for dry soil. Our analysis utilized the CPT data collected during our subsurface exploration, assumed a groundwater depth of 16 feet, and considered a PGA of 0.976g corresponding to a Magnitude 6.9 earthquake on the Calaveras fault. The results of our analysis (presented in Appendix D) indicate that the dynamic settlement will be relatively minor with a total dynamic settlement of approximately 1½ inch due to the assumed ground motion. We estimate that the differential dynamic settlement will be about ¾ inch over a lateral distance of approximately 30 feet. Recommendations for shallow foundations are provided. We estimate that the total dynamic settlement for footings bearing at 3 feet below the existing grade or at 2 feet on select engineered fill will be about 1 inch with a differential dynamic settlement of about ½ inch over a lateral distance of approximately 30 feet.

6.2. Static Settlement

Static settlement will be a design consideration due to the range and magnitude of the anticipated column loads. Recommendations for footing subgrade preparation and allowable bearing capacity are provided with consideration for static settlement due to the relatively large proposed column loads. Recommendations are also provided for optional ground improvement measures to reduce the estimated static settlements and increase the allowable bearing capacity for footings.

6.3. Corrosive/Deleterious Soil

An evaluation of the corrosivity of the on-site materials was conducted to assess the impact to concrete and metals at the site. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, electrical resistivity, chloride content, and soluble sulfate content was performed on samples of the alluvium material. The results of the corrosivity tests are presented in Appendix C. California Department of Transportation (Caltrans) defines a corrosive environment as an area within 1,000 feet of brackish water or where the soil contains more than 500 parts per million (ppm) of chlorides, sulfates of 0.2 (2,000 ppm) percent or more, or has a pH of 5.5 or less (Caltrans, 2012b). The criteria used to evaluate the deleterious nature of soil on concrete are listed in Table 2. Based on these criteria and our test results, the soil on site does not meet the definition of a corrosive environment and the sulfate exposure to concrete is negligible. Ferrous metal will undergo corrosion on site, but special mitigation measures are not needed. Recommended corrosion mitigation measures are presented in Section 8.4.

Table 2 – Criteria for Deleterious Soil on Concrete

Sulfate Content Percent by Weight	Sulfate Exposure
0.0 to 0.1	Negligible
0.1 to 0.2	Moderate
0.2 to 2.0	Severe
> 2.0	Very Severe
Reference: American Concrete Institute (ACI) Committee 318 Table 4.3.1 (ACI, 2012)	

6.4. Expansive Soils

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork, and can cause premature deterioration of pavements. Laboratory testing was performed on selected samples of the near-surface soil to evaluate the expansion index. The tests were performed in general

accordance with the American Society of Testing and Materials (ASTM) Standard D 4829. The test results, presented in Appendix C, indicate that the sample tested has a moderate expansion characteristic. Recommendations are provided for a select fill layer under slabs-on-grade to reduce the potential for differential slab movement due to the shrink/swell behavior of the moderately expansive soil on site.

6.5. Excavation Characteristics

We anticipate that construction of the proposed improvements will involve excavations of up to a few feet in depth for foundation excavations and utility trenches. Our subsurface exploration encountered alluvium consisting of dry to moist, stiff to hard clay and clayey silt, and medium dense silty sand and sandy silt over the depth of the anticipated excavations. We anticipate that heavy earthmoving equipment in good working condition should be able to make the proposed excavations.

Near-vertical cuts in these deposits up to 5 feet in depth should remain stable for a limited period of time. However, sloughing of the sidewall may occur, particularly if the sidewall is disturbed during construction operations, if the excavation is exposed to water, or if cohesionless soil is encountered. Excavation subgrade that is exposed to water may become unstable. Recommendations for excavation stabilization are presented.

7. CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that construction of the proposed project is feasible from a geotechnical perspective. Geotechnical considerations for the project include the following:

- Our subsurface evaluation indicated that the project site is underlain by Holocene alluvium. The alluvium encountered generally consisted of dry to wet, stiff to hard, clay, silty clay, sandy clay, and clayey silt; and medium dense to dense sand, silt, and gravel with trace to some clay.

- Groundwater was encountered during our subsurface evaluation. The depth to groundwater measured in the boring after drilling ranged between approximately 16 feet and 17½ feet below the existing ground surface. Fluctuations in the groundwater levels may occur as discussed in Section 5.4.

- Expansion Index testing indicates that the near-surface soil on site has moderate expansion characteristic. Recommendations for a select fill layer under slabs-on-grade are presented. Alternative recommendations are also presented for lime treatment to reduce the expansion characteristic of the on-site soil to create the layer of low expansion fill in lieu of importing select fill.

- The project alignment is not located within a State of California Fault Rupture Hazard Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps, no surface traces of known faults are present on site; therefore the potential for surface fault rupture is considered low.

- The site could experience relatively strong ground shaking during a significant earthquake on a nearby fault. Recommended seismic design criteria are provided.

- Our analysis indicates that although a few thin layers of soil will liquefy under the considered ground motion, liquefaction is not a design consideration for the site due to the depth and relative thickness of the discrete layers.

- Our analysis indicates that the dynamic settlement due to the considered ground motion will be relatively minor with a total dynamic settlement of approximately 1½ inch and a differential dynamic settlement of approximately ¾ inch over a lateral distance of about 30 feet. Recommendations for shallow foundations are provided. We estimate that the total dynamic settlement for footings bearing at 3 feet below the existing grade or at 2 feet on select engineered fill will be about 1 inch with a differential dynamic settlement of about ½ inch over a lateral distance of approximately 30 feet.

- Static settlement will be a design consideration due to the range and magnitude of the anticipated column loads. Recommendations are provided for optional ground improvement measures to reduce the estimated settlement due to sustained column loads and permit an increase in the allowable bearing capacity for footings.

- Based on the results of our limited soil corrosivity tests during this study and Caltrans corrosion guidelines (2012b), the site does not meet the definition of a corrosive environment.

8. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

8.1. Earthwork

The earthwork should be conducted in accordance with the relevant grading ordinances having jurisdiction, and the following recommendations. The geotechnical consultant should observe earthwork operations. Evaluations performed by the geotechnical consultant during the course of operations may result in new recommendations, which could supersede the recommendations in this section.

8.1.1. Site Preparation

Prior to performing earthwork operations, the site should be cleared of vegetation, surface soils containing roots or other organic matter, surface obstructions (e.g., pavements, aggregate base, etc.), rubble and debris, and other deleterious materials. Active utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout. Trees or other obstructions that extend below finish grade, if any, should be removed and the resulting holes filled with compacted soils. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dumpsite. Soils containing roots or other organic matter may be stockpiled for later use as landscaping fill, as authorized by the owner's representative.

8.1.2. Observation and Removals

Prior to placement of fill or reinforcing steel, the client should request an evaluation of the exposed subgrade by the geotechnical consultant. Materials that are considered

unsuitable should be excavated under the observation of the geotechnical consultant in accordance with the recommendations presented in this section, or the field recommendations of the geotechnical consultant.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic or compressible natural soils; and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade, as evaluated in the field by the geotechnical engineer, is exposed.

8.1.3. Ground Improvement

Ground improvement may be performed to reduce the estimated potential settlement due to sustained column loads and permit an increase in the allowable bearing capacity for footings. The ground improvement may consist of overexcavation and replacement to create a pad of select fill below the footings. The overexcavation should extend to 3 feet below the footing bearing elevation and 3 feet laterally beyond the footing edges. The overexcavation should be backfilled with select fill consisting of aggregate base after moisture conditioning the excavation subgrade to near the optimum moisture and compacting the subgrade to 90 percent or more of the reference density as evaluated by the American Society for Testing and Materials (ASTM) standard D1557. The select fill should be placed and compacted in lifts to 95 percent of ASTM D1557 after moisture conditioning the loose fill to near the optimum moisture content.

Stone columns or aggregate piers may be considered as an alternative method for ground improvement. Recommendations pertinent to ground improvement by stone columns or aggregate piers are provided in the following subsection.

8.1.3.1. Stone Columns and Aggregate Piers

Stone columns (or aggregate piers), consisting of crushed rock installed in a hole created by an auger, vibratory probe, or driven/pushed mandrel and compacted in

lifts by a vibratory probe or rammer/tamper; may be considered as an alternative ground improvement method to reduce the estimated potential settlement due to sustained column loads and permit an increase in the allowable bearing capacity for footings.

The ground improvement program should be designed and constructed by a specialty contractor with experience utilizing the selected ground improvement technique on several previous projects with similar ground conditions. The ground improvement program should be designed to reduce the future building settlement under sustained loads to 1 inch or less and to achieve an allowable bearing capacity of 8,000 pounds per square foot for footings. A pre-production test section should be constructed to demonstrate that the selected ground improvement technique and installation parameters can achieve the design criteria. Static load testing should be performed to evaluate the modulus of the constructed test columns/piers under loading conditions consistent with the proposed building.

The ground improvement contractor should submit qualifications with resumes of key personnel and descriptions of representative projects completed; a ground improvement design with shop drawings that describe the spacing, location, depth, and nominal diameter of the columns/piers; a work procedures plan outlining proposed means and methods for ground improvement; and a quality control plan that describes the measures and procedures to be implemented by the contractor to document that the ground improvement elements achieve the criteria recommended in this section and are constructed in conformance with the work plan and shop drawings.

The quality control program should include a gradation analysis of the aggregate backfill material; monitoring, recording and daily reporting of key parameters; and load testing of the constructed columns/piers. The key parameters for monitoring and reporting should include, as appropriate, start and finish time for column/pier

installation; treatment depth; vibrator amperage draw; and total quantity of backfill added per column or pier. The ground improvement and load testing operations should be observed by the geotechnical engineer.

8.1.4. Materials Requirements

Materials used during earthwork, grading, and paving operations should comply with the requirements listed in Table 3. Materials should be evaluated by the geotechnical consultant for suitability prior to use. The contractor should notify the geotechnical consultant 72 hours prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. The contractor should be responsible for the uniformity of import material brought to the site.

Table 3 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2]
Select (Low Expansion) Fill under slabs	Import	Close-graded with 35 percent or more passing No. 4 sieve and either: Expansion Index of 50 or less, Plasticity Index of 12 or less, or less than 10 percent, by dry weight, passing No. 200 sieve
General Fill for uses not otherwise specified	Import	As per Select (Low Expansion) Fill
	On-site borrow	No additional requirements[1]
Aggregate Base under pavements or flatwork below footings as select fill	Import	Class II; CSS[4] Section 26-1.02
Capillary Break Gravel part of vapor retarding system below select slabs-on-grade	Import	Open-graded, clean, compactable crushed rock or angular gravel; nominal size 3/4" or less
Controlled Low Strength Material (CLSM)	Import	CSS[4] Section 19-3.02F
Vapor Retarding Membrane	Import	15 mil, Class A plastic membrane as per ASTM E 1745
Pipe/Conduit Bedding Material	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
Trench Backfill above bedding material	Import or on-site borrow	As per General Fill plus 100 percent passing 3-inch sieve or 2-inch sieve in top 12 inches.

[1] In general, fill should not consist of pea-gravel and should be free of rocks or lumps more than 3-inches in median dimension, trash, debris, roots, vegetation or other deleterious material.

[2] In general, import fill should be tested or documented to be non-corrosive[3] and free from hazardous materials in concentrations above levels of concern.

[3] Non-corrosive as defined by the Corrosion Guidelines version 2.0 (Caltrans, 2012b).

[4] CSS is California Standard Specifications (Caltrans, 2010)

8.1.5. Excavation Stabilization and Temporary Slopes

Excavations, including footing and trench excavations, shall be stabilized in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations, Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA). Stabilization shall consist of shoring sidewalls or laying slopes back. Table 4 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, cantilever or internally-

braced shoring systems may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 4 may be used to design or select an internally-braced shoring system or trench shield conforming to the OSHA guidelines. The recommendations listed in this table are based upon the limited subsurface data provided by our exploratory borings and reflect the influence of the environmental conditions that existed at the time of our exploration. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse. Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation.

Table 4 – OSHA Material Classifications and Allowable Slopes

Formation	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4] (psf)
Cohesive Alluvium (above groundwater)	Type B	1:1 h:l v (45°)	45·D + 72
Granular Alluvium (above groundwater)	Type C	1 h: 1½v (34°)	80·D + 72

1 Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.

2 In layered soil, layers shall not be sloped steeper than the layer below.

3 Temporary excavations less than 5 feet deep may be made with vertical side slopes and remain unshored if judged to be stable by a competent person (29 CFR, Part 1926.650).

4 'D' is depth for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures. Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The geotechnical engineer should be consulted for additional recommendations if the proposed excavations cross below a plane extending down and away from the foundation bearing surfaces of the adjacent structure at an angle of 2:1 (horizontal to vertical).

The excavation bottoms may become unstable and subject to pumping under heavy equipment loads if the excavation subgrade is exposed to water. The contractor should be prepared to stabilize the bottom of the excavations. In general, unstable bottom conditions may be mitigated by scarifying the subgrade and aerating the soil to achieve a moisture content near the optimum, dewatering to depress groundwater levels below the bottom of the excavation, overexcavating to a suitable depth and replacing the wet material with suitable fill, compacting a layer of crushed rock fill into the subgrade, or using geogrid to stabilize additional fill. Specific recommendations for excavation stabilization will be influenced by the nature of the excavation and the conditions encountered during construction.

8.1.6. Construction Dewatering

The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Sump pits, trenches, or similar measures should be used to depress the water level below the bottom of the excavation. Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

8.1.7. Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with the recommendations in Section 8.1.5. Utility trenches should be backfilled with materials that conform to the recommendations in Section 8.1.3. Bedding materials should conform to the specifications of the pipe or conduit manufacturer. Trench backfill should be compacted in accordance with Section 8.1.10 of this report. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting is not recommended.

8.1.8. Lime Treatment

In lieu of importing select material with low expansion characteristics, on-site soil may be mixed with quicklime to create zone of material with low expansion characteristics below slabs-on-grade as described in Section 8.3.2. On-site materials containing roots or other organic matter are not suitable for lime treatment and should be stripped from the area at which the lime treatment is to be performed. A specialty contractor should be retained to spread and mix the lime. The lime or selected stabilizing agent may be spread in dry form or as slurry. Dry lime should be spread and mixed in a fashion that reduces potential for dusting. Casting or tailgating of dry lime is not recommended. Quicklime should be mixed into the soil at a rate of 3 percent by dry weight soil. The spreading and mixing procedure should produce a consistent distribution of stabilizing agent in the treated soil. Mixing and pulverizing should continue until the treated soil does not contain untreated soil clods larger than 1 inch and the quantity of untreated soil clods retained on the No. 4 sieve is less than 40 percent of the dry soil mass. Water should be added as-needed during the mixing process to achieve a moisture content above the optimum for the lime-soil mixture. The lime-soil mixture should be re-mixed following a 16-hour mellowing period after the initial mixing. The lime-soil mixture should be compacted within 3 days after initial mixing.

8.1.9. Subgrade Preparation

Subgrade should be prepared as per the recommendations in Table 5. Prepared subgrade should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of pavements, footings and slabs. Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted.

Table 5 – Subgrade Preparation Recommendations

Subgrade Location	Preparation Recommendations
Utility Trenches	○ Check for unsuitable materials as per Section 8.1.2. ○ Do not scarify. Remove or compact loose/soft material as per Section 8.1.10.
Below Footings	○ Check for unsuitable materials as per Section 8.1.2. ○ Scarify and moisture condition as-needed, then compact as per Section 8.1.10. ○ Cover prepared subgrade with 3-inches of CLSM as per Section 8.1.4 to maintain subgrade condition.
Below Pavements, Slabs-on-grade, and Fill	○ Check for unsuitable materials as per Section 8.1.2. ○ Scarify top 8” then moisture condition and compact as per Section 8.1.10. ○ Keep in moist condition by sprinkling water.

8.1.10. Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 6. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness.

Table 6 – Recommended Compaction Requirements

Fill Type	Location	Compacted Density[1]	Moisture Content[2]
Subgrade	Below pavements and footings	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Select Fill	Under slabs	90 percent	+ 2 percent
Lime Treated Subgrade	In locations not already specified	95 percent	+ 2 percent
Bedding and Pipe Zone	Material below conduit invert to 12 inches above conduit	90 percent	+ 2 percent
Trench Backfill	Top foot below pavements	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Aggregate Base	Under pavements, slabs & footings	95 percent	+ 2 percent
General Fill	Top foot below pavements	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent

[1] Expressed as percent relative compaction or ratio of field density to reference density as evaluated by ASTM D 1557 on a dry density basis for fill, subgrade, and aggregate base, and on a wet density basis for lime treated subgrade.

[2] Target moisture content at compaction relative to the optimum moisture content as evaluated by ASTM D 1557.

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of pavement sections. Fill that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted as per the requirements above.

8.1.11. Rainy Weather Considerations

We recommend that construction be performed during the period between approximately April 15 and October 15, to avoid the rainy season. In the event that grading is performed through the rainy season, the plans for the project should be supplemented to include a stormwater management plan prepared in accordance with the requirements of the relevant agency having jurisdiction. The plan should include details of measures to protect the subject property and adjoining off-site properties from damage by erosion, flooding or the deposition of mud, debris, or construction-related

pollutants, which may originate from the site or result from the grading operation. The protective measures should be installed by the commencement of grading, or prior to the start of the rainy season. The protective measures should be maintained in good working order unless the drainage system is installed by that date and approval has been granted by the building official to remove the temporary devices.

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains. Lime treatment as described in Section 8.1.8 may be utilized to stabilize exposed subgrade and reduce the potential for softening on exposure to water. The geotechnical engineer should be consulted for recommendations to stabilize the site as-needed.

8.2. Seismic Design Criteria

A site specific ground motion analysis was performed in accordance Section 1616A.1.3 of the 2013 California Building Code (CBC). The analysis was based on the procedure in Chapter 21 of the American Society of Civil Engineers (ASCE) 7-10 Standard. The design response spectrum based on the site specific ground motion analysis is presented on Figure 5 and the corresponding seismic design criteria are summarized in Table 7. The spectral ordinates and seismic coefficients based on the mapped values of the risk-targeted spectral response acceleration consistent with Section 11.4 of ASCE 7-10 are presented in the table for comparison.

Table 7 – 2013 California Building Code Seismic Design Criteria

Seismic Design Parameters Evaluated for 37.76299º North Latitude, 121.95932º West Longitude	**Site Specific**	**Section 11.4 ASCE 7-10**
Soil Profile Type	D	D
Site Coefficient F_a	--	1.0
Site Coefficient F_v	--	1.5
Mapped MCE Spectral Response Acceleration at 0.2-second period, S_S	--	2.329
Mapped MCE Spectral Response Acceleration at 1-second period, S_1	--	0.886
Site-Adjusted MCE Spectral Response Acceleration at 0.2-second period, S_{MS}	2.226	2.329
Site-Adjusted MCE Spectral Response Acceleration at 1-second period, S_{M1}	1.532	1.329
Design Spectral Response Acceleration at 0.2-second period, S_{DS}	1.484	1.553
Design Spectral Response Acceleration at 1-second period, S_{D1}	1.021	0.886

8.3. Foundations

The following foundation design parameters and recommendations are provided based on our findings and geotechnical analysis. The foundation design parameters are not intended to preclude differential movement of foundations. Minor cracking (considered tolerable) of foundations may occur. Foundations should be designed in accordance with structural considerations and our geotechnical recommendations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of the structures.

8.3.1. Spread Footings

The parameters listed in Table 8 may be used to design footings to support the proposed building provided that the footing subgrade is prepared in accordance with the recommendations in Section 8.1.9. Alternatively, ground improvement consisting of overexcavation and replacement with select fill or installation of stone columns (or aggregate piers) may be performed reduce the potential static settlement and increase the allowable bearing capacity for the footings. Recommendations for ground improvement are provided in Section 8.1.3. The geotechnical engineer should observe the footing excavations to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

Table 8 – Recommended Bearing Design Parameters for Footings

Support Condition	Footing Width	Bearing Depth[1]	Allowable Bearing Capacity[2]	Static Settlement
Footing on stiff subgrade, moisture conditioned and compacted to 95%	2½ feet or more	3 feet	4,500 psf	1½ inches total ¾ inches differential over 30 feet
Footing on select fill, moisture conditioned and compacted in lifts to 95%	2½ feet or more	2 feet	5,000 psf	1 inch total ½ inch differential over 30 feet
Footing on subgrade reinforced by stone columns (or aggregate piers)	2½ feet or more	2 feet	8,000 psf	1 inch total ½ inch differential over 30 feet

[1] Below the adjacent grade.
[2] Net allowable bearing capacity in pounds per square foot. Listed value includes a Factor of Safety of 3 or more. Allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic loads.

Structures supported on footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 8 for sustained loads plus an additional 1 inch of dynamic settlement with a differential of ½ inch over a lateral span of 30 feet. Footing settlement due to static loads may be further evaluated using a modulus of subgrade reaction. Recommended values for the modulus of subgrade reaction are provided in Table 9. The designer may interpolate between the values in the table for intermediate footing widths.

Table 9 – Footing Modulus of Subgrade Reaction

Support Condition	Footing Width				
	2½ feet	5 feet	8 feet	11 feet	15 feet
Footing on stiff subgrade, moisture conditioned and compacted to 95%	109 pci	45 pci	28 pci	23 pci	19 pci
Footing on select fill, moisture conditioned and compacted in lifts to 95%	192 pci	69 pci	45 pci	33 pci	28 pci
Footing on subgrade reinforced by stone columns (or aggregate piers)	300 pci	110 pci	70 pci	50 pci	45 pci
Notes: Modulus of Subgrade Reaction in pounds per cubic inch (pci).					

The spread footings should be reinforced with deformed steel bars as detailed by the project structural engineer. Where footings are located adjacent to utility trenches or other excavations, the footing bearing surfaces should bear below an imaginary plane extending upward from the bottom edge of the adjacent trench/excavation at a 2:1 (horizontal to vertical) angle. Footings should be deepened or excavation depths reduced as-needed.

A lateral bearing pressure of 300 psf per foot of depth up to 3,000 psf may be used to evaluate the resistance of footings to lateral loads. The recommended lateral bearing pressure is for level ground conditions where the ground adjacent to the foundation is approximately level for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. A friction coefficient of 0.35 may be assumed for evaluating frictional resistance to lateral loads.

8.3.2. Slabs-on-Grade

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. The slab should be reinforced with deformed steel bars

(nominal bar diameter of ⅜ inch or more). Masonry briquettes or plastic chairs should be used to aid in the correct placement of slab reinforcement in the upper half of the slab. Recommendations for concrete and concrete cover over reinforcing steel are provided in Section 8.4. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. Recommendations for vapor retarding systems are provided in Section 8.5. Joints consistent with ACI guidelines (ACI, 2012) may be constructed at periodic intervals to influence crack locations and reduce the potential for random cracking of the slab. Slabs subject to vehicular loading should be 6-inches thick or more and should be underlain by 6 inches of Class 2 aggregate base that conforms with Section 26-1.02 of the California Standard Specifications (Caltrans, 2010) and is compacted to 95 percent of the reference density as evaluated by ASTM D1557. A 6-inch thick slab constructed of concrete having a 5,000 pounds per square inch compressive strength on 6 inches of aggregate base should be suitable for a 20-year traffic index of 10.5 with single and tandem axle loads of up to 18 kips and 32 kips, respectively. The designer should evaluate the suitability of this traffic index and the assumed loads.

To reduce the potential differential movement of slabs-on-grade due to the shrink/swell behavior of the moderately expansive site soil, the slabs should be underlain by 12 inches of select fill with low expansion characteristics conforming to criteria in Section 8.1.4. Alternatively, the on-site soil may be mixed with quicklime as recommended in Section 8.1.8 to reduce the expansion characteristic of the on-site soil. The aggregate base layer under slabs-on-grade subject to vehicular loading may be considered as part of the select fill. Where the slab-on-grade is underlain by a moisture vapor retarding system, the crushed rock and sand layer, if used, may also be considered as part of the select fill.

To reduce the potential for seasonal variation in soil moisture below slabs-on-grade where a perimeter footing or grade beam is not present, the edge of the slab should be thickened or turned-down so that the bottom of the slab perimeter is 2 feet or more

below the adjacent grade. Alternatively, where the slab is underlain by a moisture vapor retarding system, the plastic membrane can be turned down at the slab perimeter to terminate 2 feet below the adjacent grade.

8.4. Concrete

Laboratory testing indicated that the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested. However, due to the variability in the on-site soil and the potential future use of reclaimed water at the site, we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, the concrete should have a water-to-cement ratio of not more than 0.45 and a 3-inch thick or thicker concrete cover should be maintained over reinforcing steel where concrete is cast-in-place against soil. Concrete cover over reinforcing steel for other exposure conditions should conform to ACI guidelines (ACI, 2012).

8.5. Moisture Vapor Retarder

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a polyethylene plastic membrane 15-mil-thick. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. An optional 2-inch thick blotter sand layer may be placed over the plastic membrane. The blotter sand layer should be omitted where the slab is exposed to vehicular traffic. The blotter sand should be maintained in a moist but not saturated condition prior to concrete placement. If the blotter sand layer is omitted; to reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance

with the Manual of Concrete Practice (ACI, 2012). The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork.

A subdrain should be constructed around the foundation perimeter at locations where the exterior grade is at a higher elevation than the moisture vapor retarding system (including the capillary break layer). The subdrain should consist of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the foundation. The pipe should be located below the bottom elevation of the moisture vapor retarding system but above a plane extending down and away from the bottom edge of the foundation at a 2:1 (horizontal to vertical) gradient.

8.6. Surface Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures or retaining walls and is not permitted to pond. Positive drainage consisting of a gradient of 2 percent or more should be established for a distance of 5 feet or more adjacent to structures or retaining walls to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. Pad and roof drainage (from adjacent structures) should be collected and diverted to suitable discharge areas away from structures by non-erodible devices (e.g., gutters, downspouts, concrete swales, etc.). Graded swales, v-ditches, or curb and gutter should be provided at the site perimeter to restrict flow of surface water onto and off of the site. Drainage structures should be periodically cleaned out and repaired, as-needed, to maintain appropriate site drainage patterns.

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should be limited to that which is needed to sustain the plants. Trees should be

restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project.

Pavement surfaces should be crowned and/or sloped to divert water to edge gutters so that it is not permitted to pond or accumulate on the pavement. Edge gutters should be sloped to provide positive drainage to drop inlets or other drainage devices. Cracks that form in the pavement surface should be periodically sealed to reduce moisture intrusion into the aggregate base section.

8.7. Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that the geotechnical consultant review project plans. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified to meet the project requirements.

8.8. Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions disclosed by discrete exploratory borings. The geotechnical consultant in the field during construction should check the interpolated subsurface conditions. During construction, the geotechnical consultant should:

- Observe installation and testing of stone columns or aggregate piers if utilized.
- Observe lime treatment of site soil if performed.

- Observe preparation and compaction of subgrade.
- Check and test imported materials prior to their use as fill.
- Observe placement and compaction of fill.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.
- Observe placement of reinforcing steel in footings and slabs.
- Observe condition of water vapor retarding system prior to concrete placement.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

9. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

10. REFERENCES

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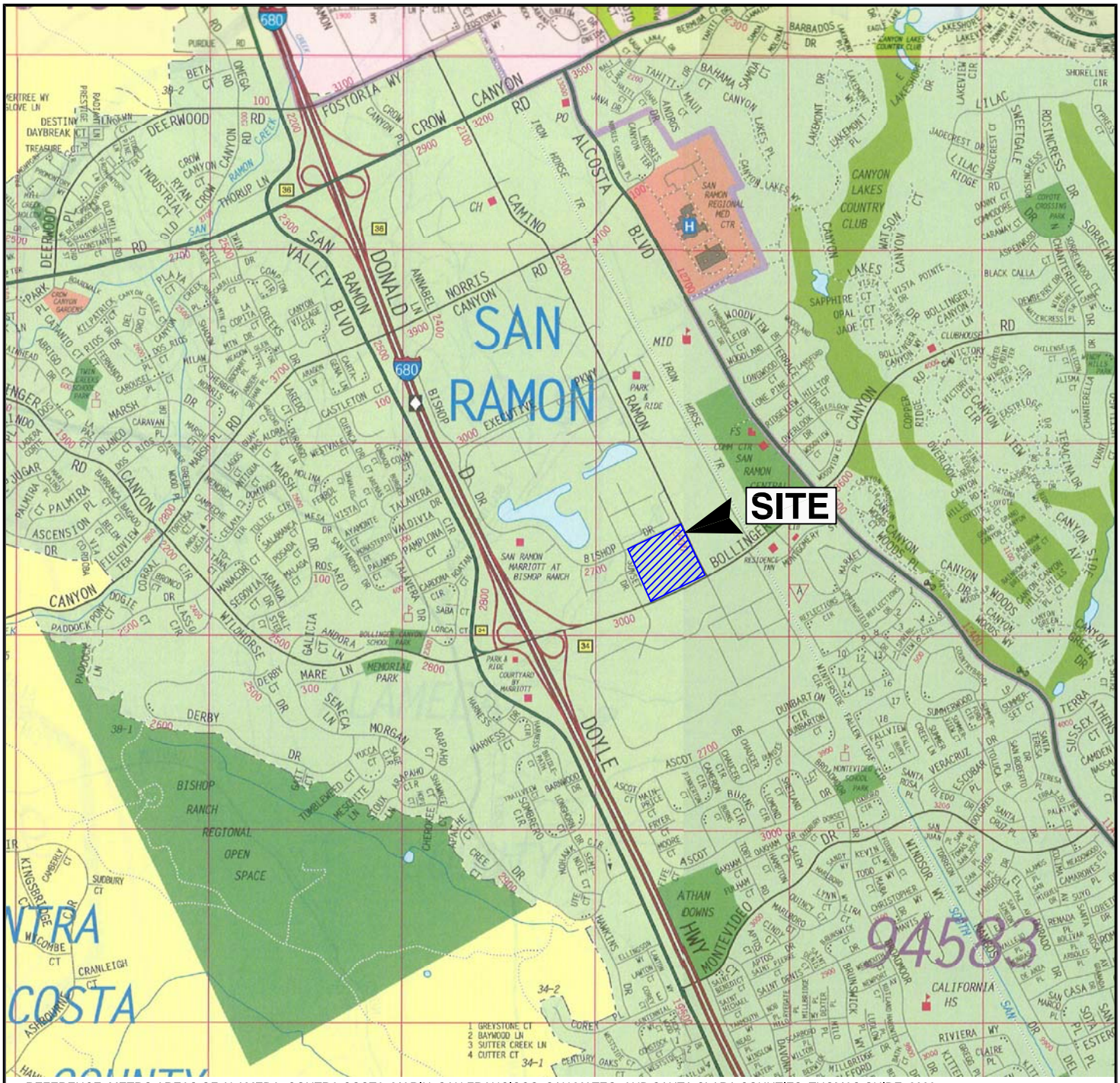


REFERENCE: METRO AREAS OF ALAMEDA, CONTRA COSTA, MARIN, SAN FRANCISCO, SAN MATEO, AND SANTA CLARA COUNTIES, THOMAS GUIDE, 2008.

N

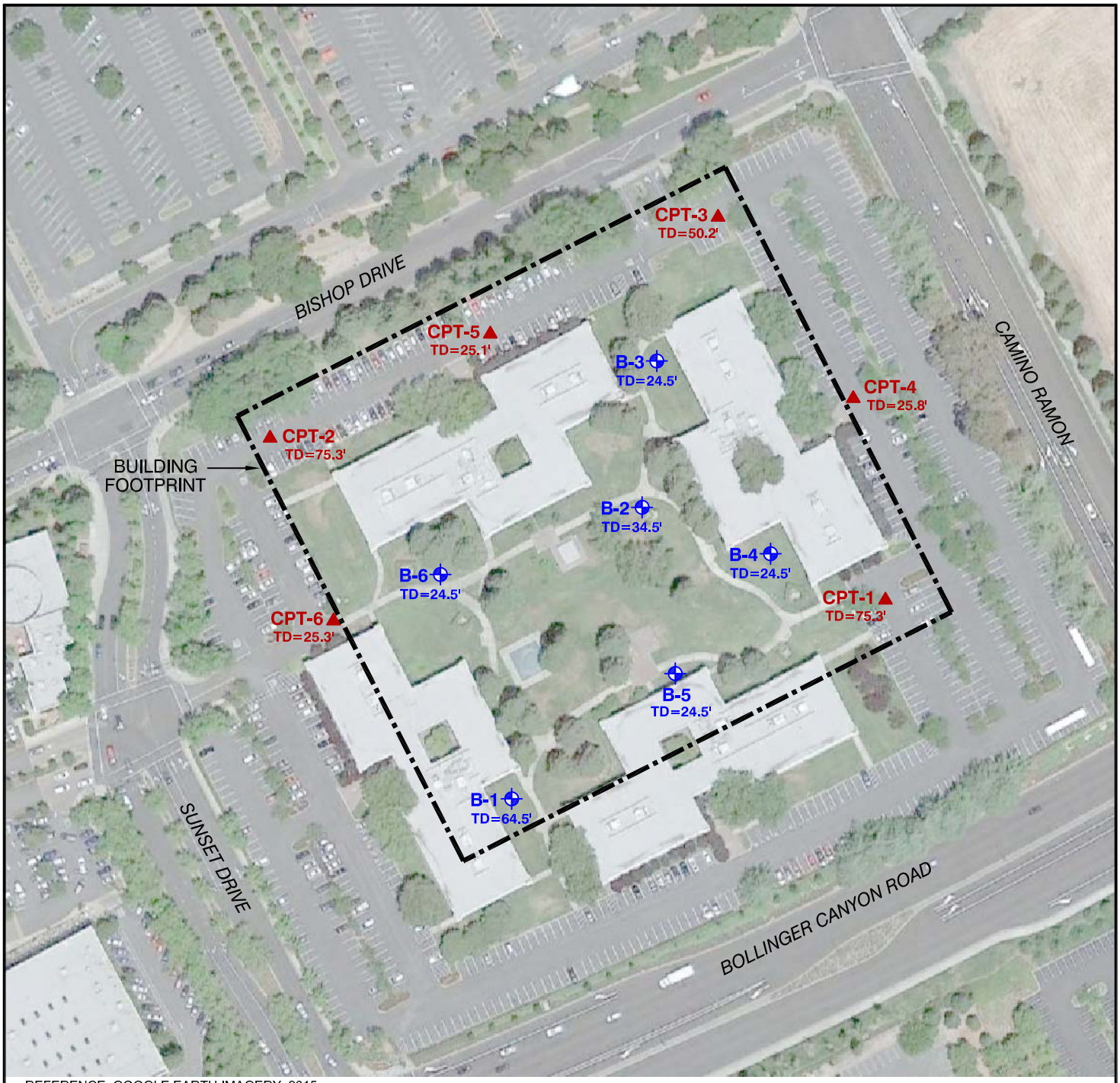
SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore	SITE LOCATION	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	1
402676001 / 1/16		

402676001-SL.dwg, Jan 15, 2016, 1:30pm, SN



REFERENCE: GOOGLE EARTH IMAGERY, 2015.

N

SCALE IN FEET

0 150 300

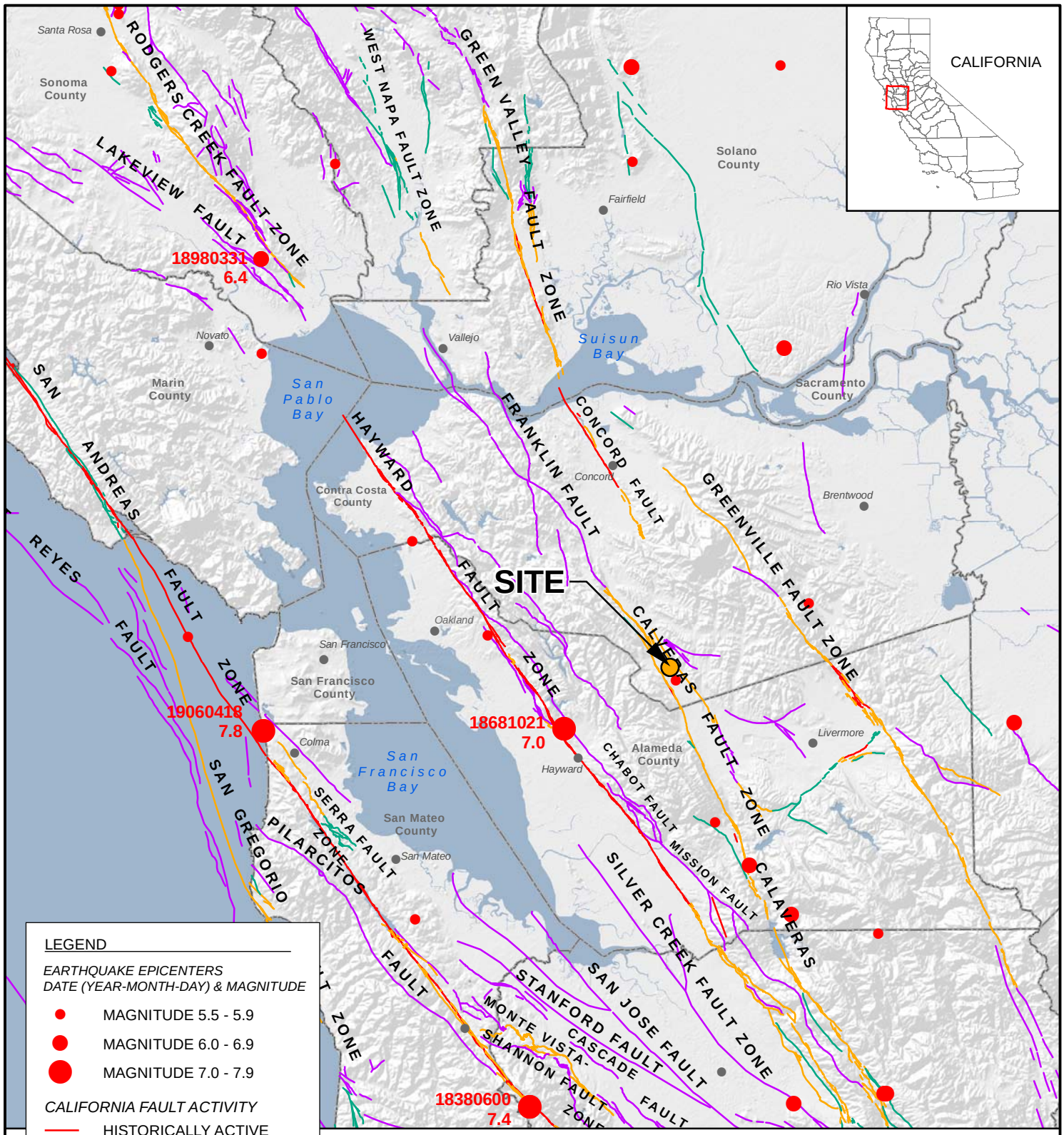
NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

B-6 TD=24.5'	BORING TD=TOTAL DEPTH IN FEET
CPT-6 TD=25.3'	CONE PENETRATION TEST TD=TOTAL DEPTH IN FEET

Ninyo & Moore	BORING LOCATIONS	FIGURE
PROJECT NO. 402676001 / DATE 1/16	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	2

402676001-BL.dwg, Jan 15, 2016, 1:30pm, SN



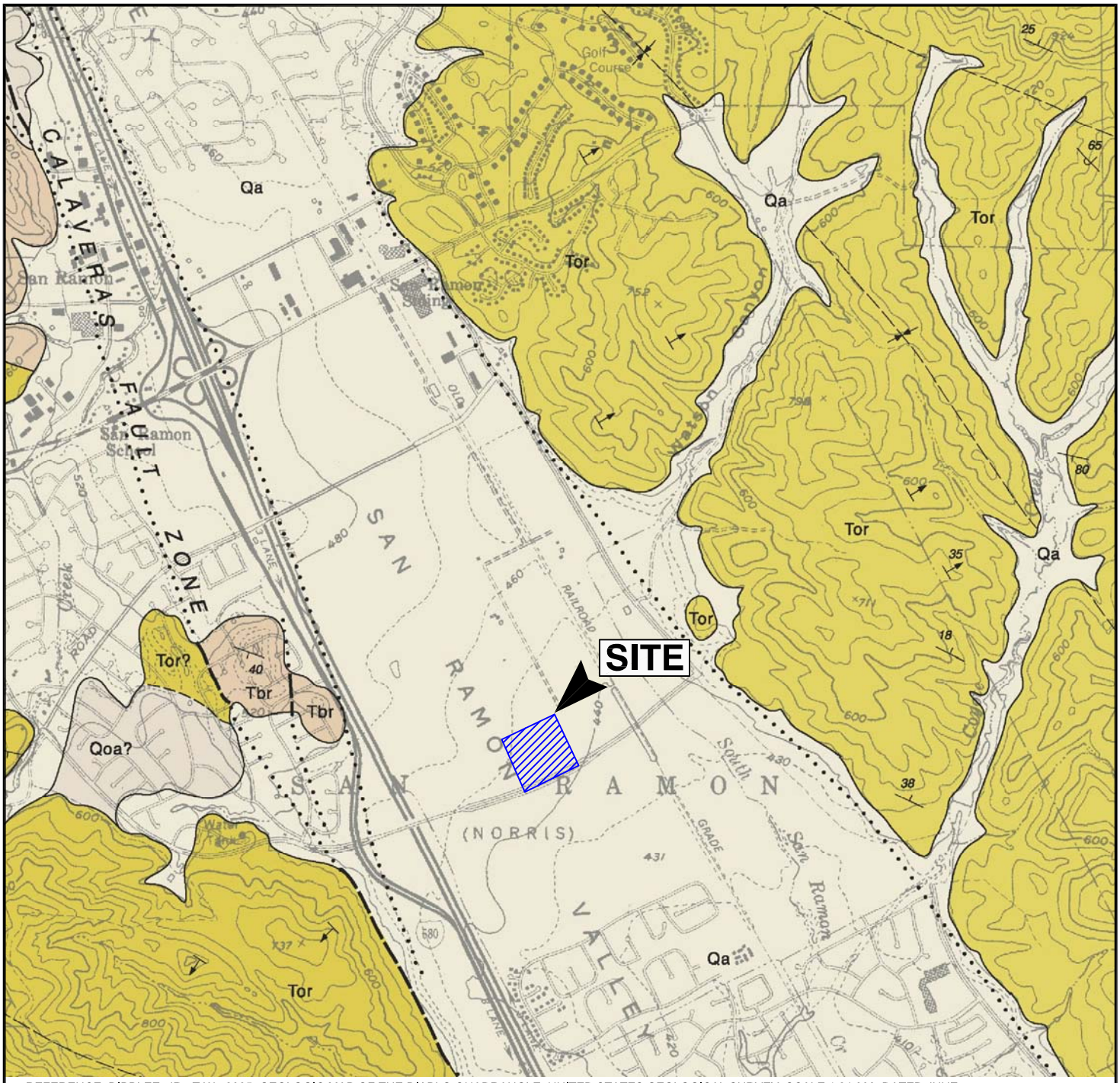
SOURCES: FAULTS - USGS & CGS, 2006, QUATERNARY FOLD AND FAULT DATABASE FOR THE UNITED STATES.
EARTHQUAKES - CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS-49.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore

PROJECT NO.	DATE	FAULT LOCATIONS AND EARTHQUAKE EPICENTERS	FIGURE
402676001	1/16	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	3

fault_loc_EPIC_OAK_central_93.mxd SN



REFERENCE: DIBBLEE, JR., T.W., 2005, GEOLOGIC MAP OF THE DIABLO QUADRANGLE, UNITED STATES GEOLOGICAL SURVEY, SCALE 1:24,000, DATED JUNE.

N

SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

Qa	SURFICIAL SEDIMENTS
Qoa	OLDER SURFICIAL SEDIMENTS
Tor	ORINDA FORMATION
Thr	BRIONES FORMATION AND ASSOCIATED ROCKS

GEOLOGIC CONTACT

35 STRIKE AND DIP

SYNCLINE

FAULT

Ninyo & Moore

PROJECT NO.	DATE
402676001	1/16

REGIONAL GEOLOGY

CITY CENTER SAN RAMON
2678 BISHOP DRIVE
SAN RAMON, CALIFORNIA

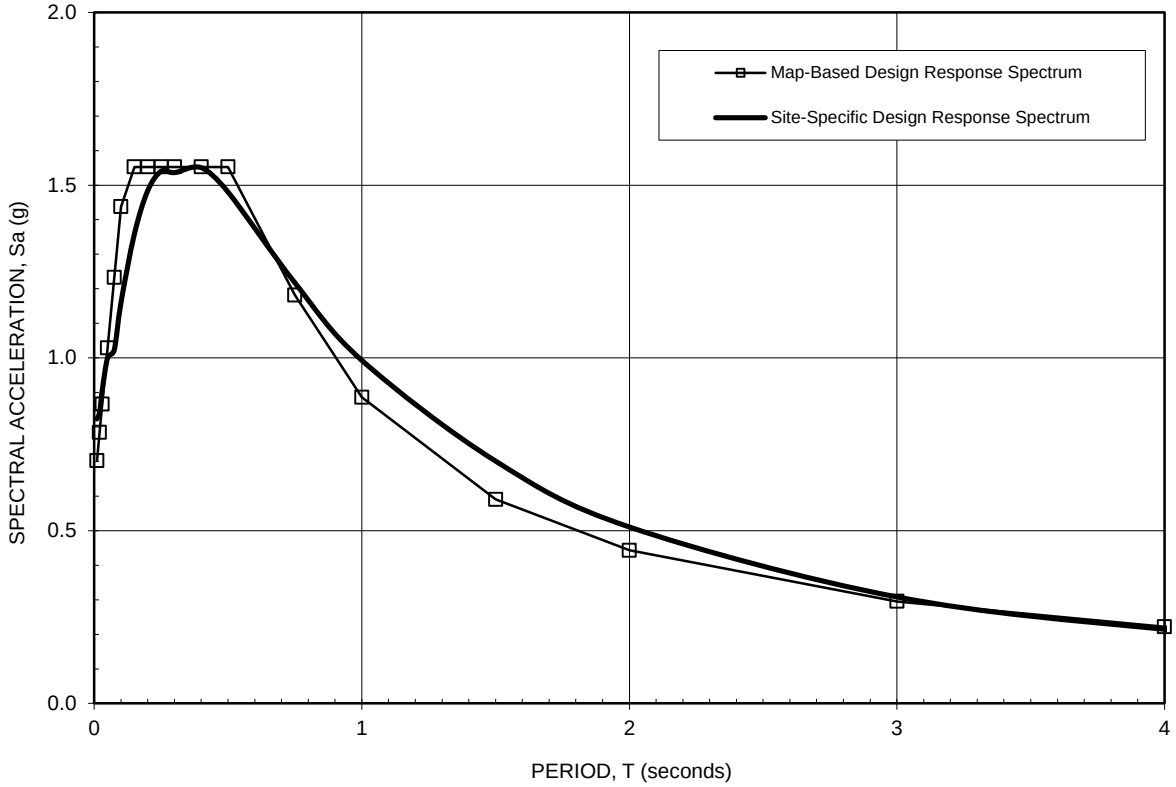
FIGURE

4

402676001-RG.dwg, Jan 15, 2016, 1:31pm, SN

PERIOD (seconds)	MAP-BASED DESIGN RESPONSE SPECTRUM Sa, (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa, (g)
0.010	0.703	0.823
0.050	1.029	1.000
0.100	1.437	1.160
0.150	1.553	1.358
0.200	1.553	1.484
0.250	1.553	1.540
0.300	1.553	1.536
0.400	1.553	1.551
0.500	1.553	1.480
0.750	1.181	1.217
1.000	0.886	0.993

S_{DS} = 1.484	S_{D1} = 1.021	S_{MS} = 2.226	S_{M1} = 1.5319	PGA_M = 0.976



NOTES:

1 Probabilistic Ground Motion is the Maximum Considered Earthquake (MCE) ground motion having 2% probability of exceedance In 50 years using Chiou & Youngs (2008), Campbell & Bozorgnia (2008), and Boore & Atkinson (2008) attenuation relationships.

2 Deterministic ARS is 84th percentile of the median values from attenuation relationships by Chiou & Youngs (2008), Campbell & Bozorgnia (2008), and Boore & Atkinson (2008) considering a Mw 6.9 event on the Calaveras fault located 0.6 kilometers from the site. It conforms with the lower bound limit per ASCE 7-10 Section 21.2.2 as modified by 2009 NEHRP Recommended Seismic Provisions.

3 Site-Specific MCE_R is the lesser of spectral ordinates of deterministic and probabilistic ARS at each period, adjusted for risk, per ASCE 7-10 Section 21.2.3. Site-Specific Design Response Spectrum conforms with lower bound limit per ASCE 7-10 Section 21.3.

4 Map-Based Design Response Spectrum is computed from mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-10 Section 11.4. It is presented for comparison.

Ninyo & Moore	ACCELERATION RESPONSE SPECTRA	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	5
402676001 / 1/16		

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following methods.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin brass liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

Field Testing

The following test was performed in the field to evaluate soil properties.

Static Cone Penetrometer

A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 1.5 square centimeters was manually pushed 6 inches into the soil. The penetrometer was instrumented to measure the Cone Penetration Index (Q_c) computed as the peak force on the cone divided by the cone base area. The Cone Penetration Index is reported in kilograms per square centimeter (ksc) on the boring logs at the depth of the test as a measure of the relative density or consistency of the soil encountered.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0	■							Bulk sample.
		■						Modified split-barrel drive sampler.
		▲						2-inch inner diameter split-barrel drive sampler.
		☒						No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.
		▌						Sample retained by others.
5		◤						Standard Penetration Test (SPT).
		╱						No recovery with a SPT.
		▭	XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
		╱						No recovery with Shelby tube sampler.
		▤						Continuous Push Sample.
10				♀				Seepage.
				▽				Groundwater encountered during drilling.
				▼				Groundwater measured after drilling.
						▦	SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
						▨	CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

Explanation of Boring Log Symbols

PROJECT NO.	DATE	FIGURE

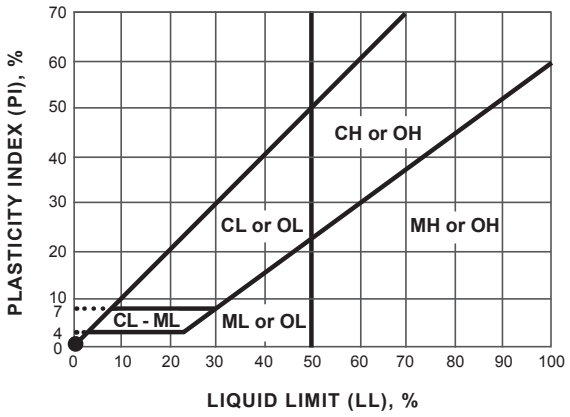
SOIL CLASSIFICATION CHART PER ASTM D 2488

PRIMARY DIVISIONS			SECONDARY DIVISIONS	
			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

PLASTICITY CHART



APPARENT DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

CONSISTENCY - FINE-GRAINED SOIL

CONSIS-TENCY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

PROJECT NO.	DATE	FIGURE

DATE DRILLED 11/11/2015 BORING NO. B-1

GROUND ELEVATION 442' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger, 4" Mud Rotary, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	SOD: Approximately 3 inches thick.
								ALLUVIUM: Brown to dark brown, moist, very stiff, sandy CLAY. @1': Qc=47 ksc. @2': Qc=65 ksc. @3': Qc>70 ksc.
			14					Very stiff.
			12					Stiff.
10								
			8					
				▽				Wet.
			18	23.7	100.1			Very stiff. Begin mud rotary.
20								
			7	24.9				Gray, stiff.
			10					Very Stiff.
30								
			9	27.9				Stiff.
			15	21.6				Very stiff.
40								

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BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-1

DATE DRILLED 11/11/2015 BORING NO. B-1

GROUND ELEVATION 442' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger, 4" Mud Rotary, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40							CL	ALLUVIUM: (continued) Olive gray, wet, very stiff, sandy CLAY.
		■	13					Yellowish brown.
		■	9	21.4				Brown, stiff.
50							SP	Brown, wet, medium dense, poorly-graded SAND.
		▱	14					
		■	38	8.5			GW	Grayish brown, wet, very dense, well-graded GRAVEL; some sand.
60							CL	Light brown, wet, very stiff, lean CLAY.
		■	13	35.4				
								Total depth = 64.5 feet.
								Groundwater was encountered at a depth of approximately 17.1 feet in borehole during drilling. Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.
70								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
								Backfilled with cement grout on 11/11/2015.
80								

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BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-2

DATE DRILLED 11/10/2015 BORING NO. B-2

GROUND ELEVATION 442' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						CL	SOD: Approximately 3 inches thick.
	■							ALLUVIUM: Light brown to brown, dry, stiff, CLAY; little sand. @1': Qc=30 ksc. @2.5': Qc=33 ksc.
	■	■	11	18.8	103.2			
							CL-ML	Light brown, dry, very stiff, silty CLAY.
		■	18					
10								
		■	11				CL	Light brown, dry to moist, stiff, lean CLAY; little sand.
				▼				Wet.
		■	11					Brown.
20								
		■	10	21.2	103.5			Olive brown; some sand.
		◤	13					Very stiff. Gray; trace sand.
30								
		◤	15	16.8			SP-SM	Brown, wet, medium dense, poorly graded SAND; few silt.
								Total depth = 34.5 feet.
								Groundwater was measured at a depth of approximately 16 feet in borehole about 5 minutes after drilling. Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.
40								

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BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-3

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. Backfilled with cement grout on 11/10/2015.
50								
60								
70								
80								

DATE DRILLED 11/10/2015 BORING NO. B-2

GROUND ELEVATION 442' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

Ninyo & Moore

BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-4

DATE DRILLED 11/10/2015 BORING NO. B-3

GROUND ELEVATION 443' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow Stem Auger, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						CL	SOD: Approximately 3 inches thick.
	■							ALLUVIUM: Dark brown, dry, very stiff, sandy CLAY. @1': Qc=45 ksc. @2': Qc>70 ksc; hard. @4': Qc>70 ksc.
		■	15				CL-ML	Light brown, moist, very stiff, silty CLAY; some sand.
10								
		■	18	15.2	112.6		CL	Brown, moist, very stiff, sandy CLAY; few gravel.
				▼				Wet.
		■	14	24.0	101.2			Very stiff.
20								
		■	56				SP	Light brown, wet, dense, poorly-graded SAND; fine to coarse sand; some gravel.
								Total depth = 24.5 feet.
								Groundwater was measured at a depth of approximately 17.5 feet in borehole about 5 minutes after drilling. Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.
30								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
								Backfilled with cement grout on 11/10/2015.
40								

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BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-5

DATE DRILLED 11/10/2015 BORING NO. B-4

GROUND ELEVATION 444' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow Stem Auger, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						CL	SOD: Approximately 3 inches thick.
	■							ALLUVIUM: Dark brown, moist, stiff, lean CLAY. @1': Qc=25 ksc. @2': Qc=65 ksc; hard. @3': Qc>70 ksc.
	■	■	19					Light brown; very stiff; some sand.
							CL-ML	Light brown, moist, very stiff, silty CLAY; some sand.
		■	13					
10								
		■	15	21.8	100.5		CL	Light brown, moist, very stiff, lean CLAY; some sand.
		◤	7					Stiff; little sand.
20								
		■	43				SC	Light brown, moist, dense, clayey SAND.
								Total depth = 24.5 feet.
								Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
30								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
								Backfilled with cement grout on 11/10/2015.
40								

Ninyo & Moore

BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-6

DATE DRILLED 11/10/2015 BORING NO. B-5

GROUND ELEVATION 442' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow Stem Auger, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						CL	SOD: Approximately 3 inches thick. ALLUVIUM: Dark brown, dry, very stiff, lean CLAY; some sand. @1': Qc=30 ksc. @2': Qc=62 ksc; hard. @3': Qc=70 ksc.
	■	■	18					Very stiff.
		■	13	25.0	94.4		CL	Light brown; moist; very stiff; little sand.
10								
		■	24					Some sand.
				▼				Groundwater measured at approximately 16.2 feet 5 about minutes after drilling.
		■	13	26.7	95.5			Little sand.
20								
		■	43	14.2	116.6		SM	Light brown, wet, dense, silty SAND. Olive gray, coarse sand.
								Total depth = 24.5 feet.
30								
40								

Groundwater was measured at a depth of approximately 16.2 feet about 5 minutes after drilling. Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

Backfilled with cement grout on 11/10/2015.

Ninyo & Moore

BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-7

DATE DRILLED 11/11/2015 BORING NO. B-6

GROUND ELEVATION 443' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow Stem Auger, CME 850 (Geo Ex)

DRIVE WEIGHT 140 lbs. Auto Hammer DROP 30"

SAMPLED BY JAC LOGGED BY JAC REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						CL	SOD: Approximately 3 inches thick.
	■							ALLUVIUM: Dark brown, moist, very stiff, sandy CLAY. @1': Qc=30 ksc. @2.5': Qc=40 ksc. @3.5': Qc=65 ksc; hard.
	■	■	33	10.5	108.1			Light brown.
	■	■	21	19.5	94.6		CL-ML	Light brown, dry, very stiff, silty CLAY; some sand.
10								
	■	■	12	26.0	96.8		CH	Brown, moist, stiff, fat CLAY; few sand.
		◤	13	31.9				Very stiff.
20								
				▽			SP-SC	Brown, wet, loose, SAND poorly-graded with clay.
	■		7				CL	Gray, wet, stiff, CLAY.
								Total depth = 24.5 feet.
								Groundwater was encountered at a depth of approximately 22 feet during drilling. Groundwater may rise to a higher level due to the relatively slow seepage rate in clay and several other factors as discussed in the report.
30								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
								Backfilled with cement grout on 11/11/2015.
40								

Ninyo & Moore

BORING LOG

CITY CENTER SAN RAMON
2678 BISHOP DRIVE, SAN RAMON, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402676001	1/16	A-8

APPENDIX B

CONE PENETRATION TESTING

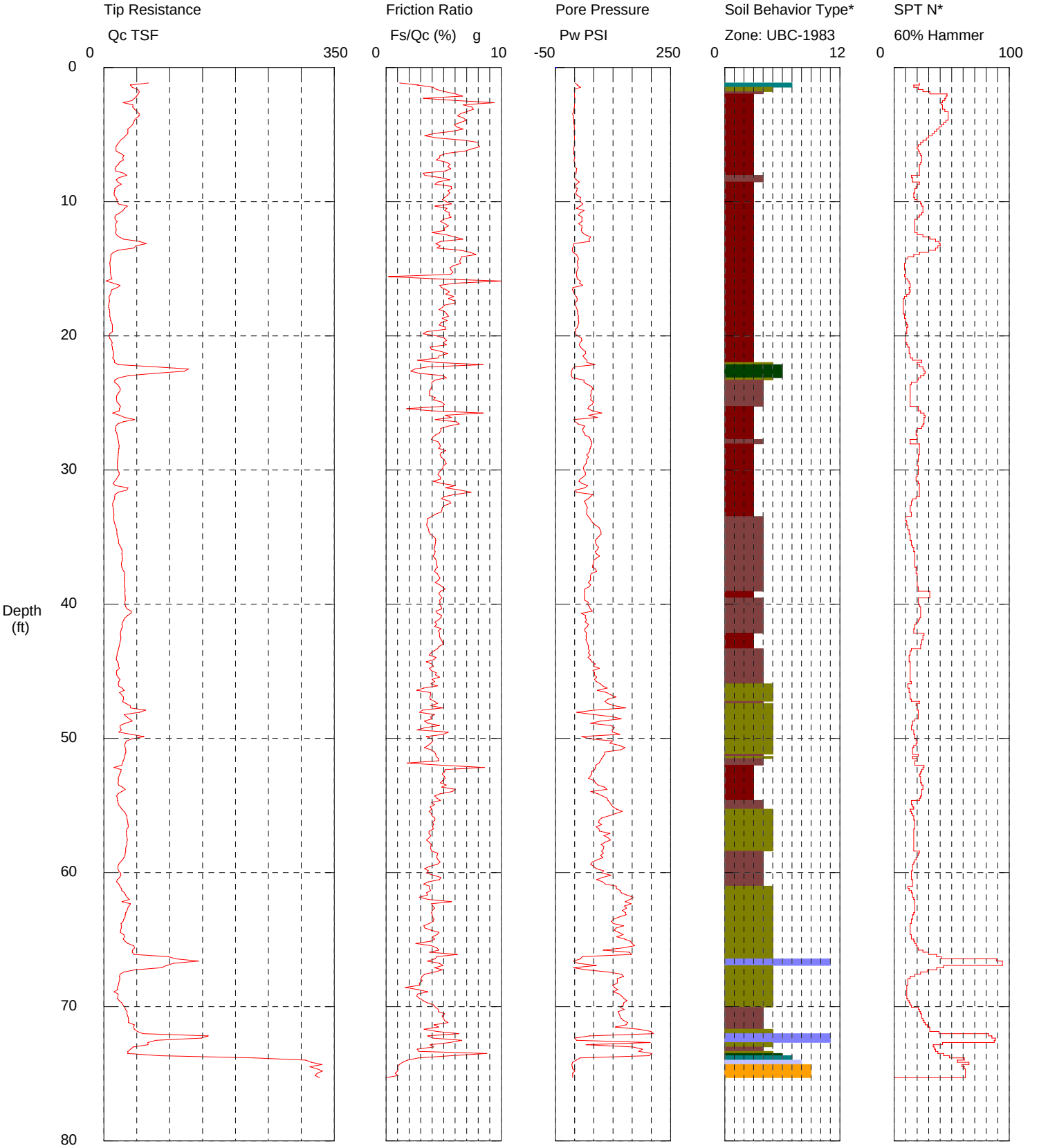
Field Procedure for Cone Penetration Testing

A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 10 square centimeters was hydraulically pushed through the soil using the reaction mass of a 20-ton rig at a constant rate of about 20 millimeter per second in accordance with ASTM D 5778. The penetrometer was instrumented to measure, by electronic methods, the force on the conical point required to penetrate the soil, the force on a friction sleeve behind the cone tip as the penetrometer was advanced, and the pore pressure (Pw) on a transducer behind the cone tip. Penetration data was collected and recorded electronically at intervals of about 2-inches. Cone resistance (Q_c) was calculated by dividing the measured force of penetration by the cone base area. Friction sleeve resistance (F_s) was calculated by dividing the measured force on the friction sleeve by the surface area of the sleeve. The friction ratio (Fs/Qc) was calculated as the ratio of the tip resistance to the sleeve friction. A graph of the computed values of cone resistance (tip) and friction ratio are presented on the logs in the following pages. The tip resistance and friction ratio were used to classify the soil type encountered using the method by Robertson and Campanella (1989). Equivalent SPT blowcounts at a 60 percent energy ratio (N_{60}-values) were calculated from the tip resistance and friction ratio using the method by Jeffries and Davies (1993). A graph of the equivalent N_{60} values (SPT N_{eq}) and the encountered soil types are also presented on the logs in the following pages.

Ninyo & Moore

Operator: Rocco
Sounding: 1
Cone Used: DDG1316

CPT Date/Time: 11/10/2015 7:54:29 AM
Location: San Ramon
Job Number: NMI-545



Maximum Depth = 75.30 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

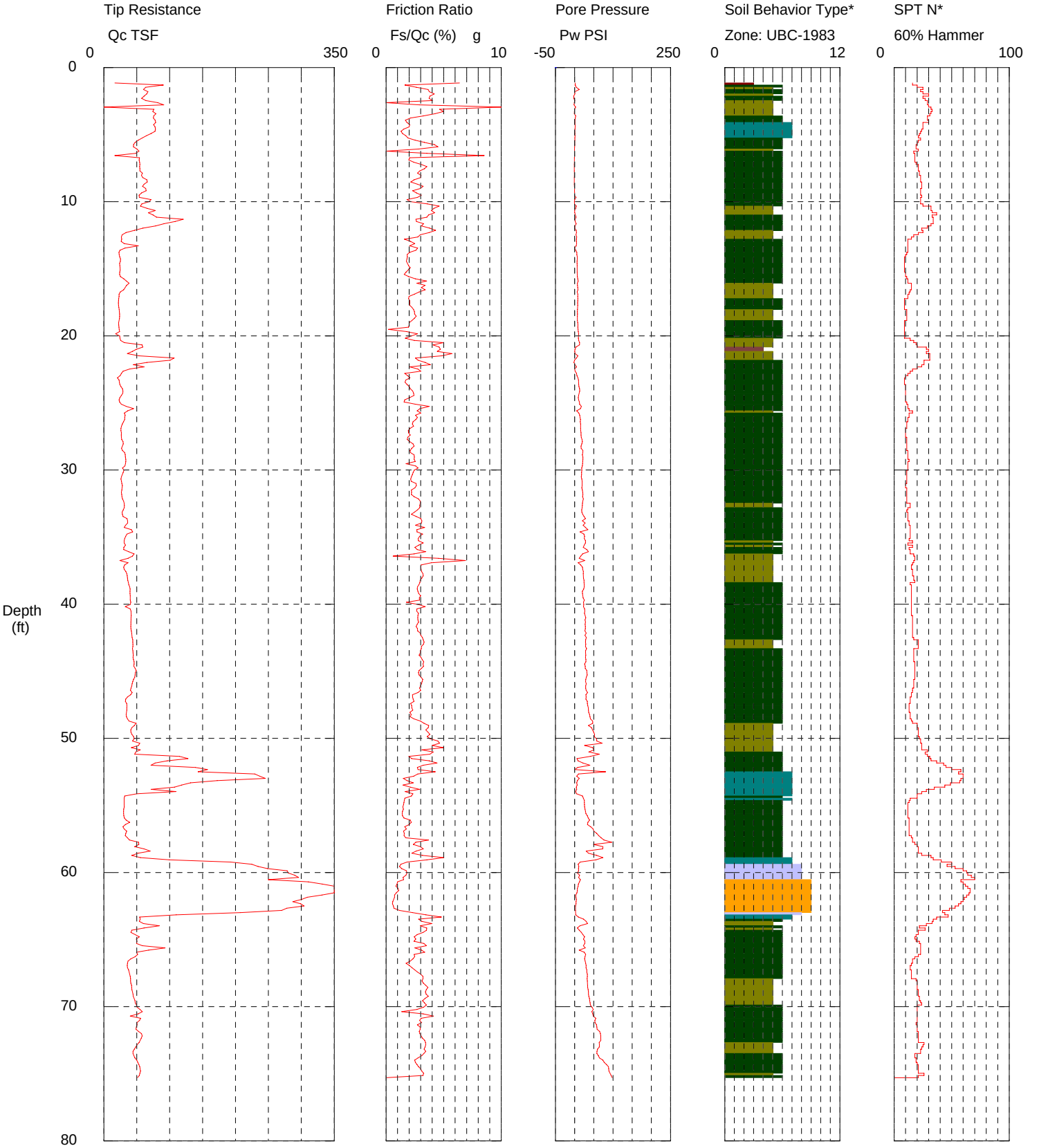
Filter On

*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Operator: Rocco
Sounding: 2
Cone Used: DDG1316

CPT Date/Time: 11/10/2015 10:28:35 AM
Location: San Ramon
Job Number: NMI-545



Maximum Depth = 75.30 feet

Depth Increment = 0.164 feet

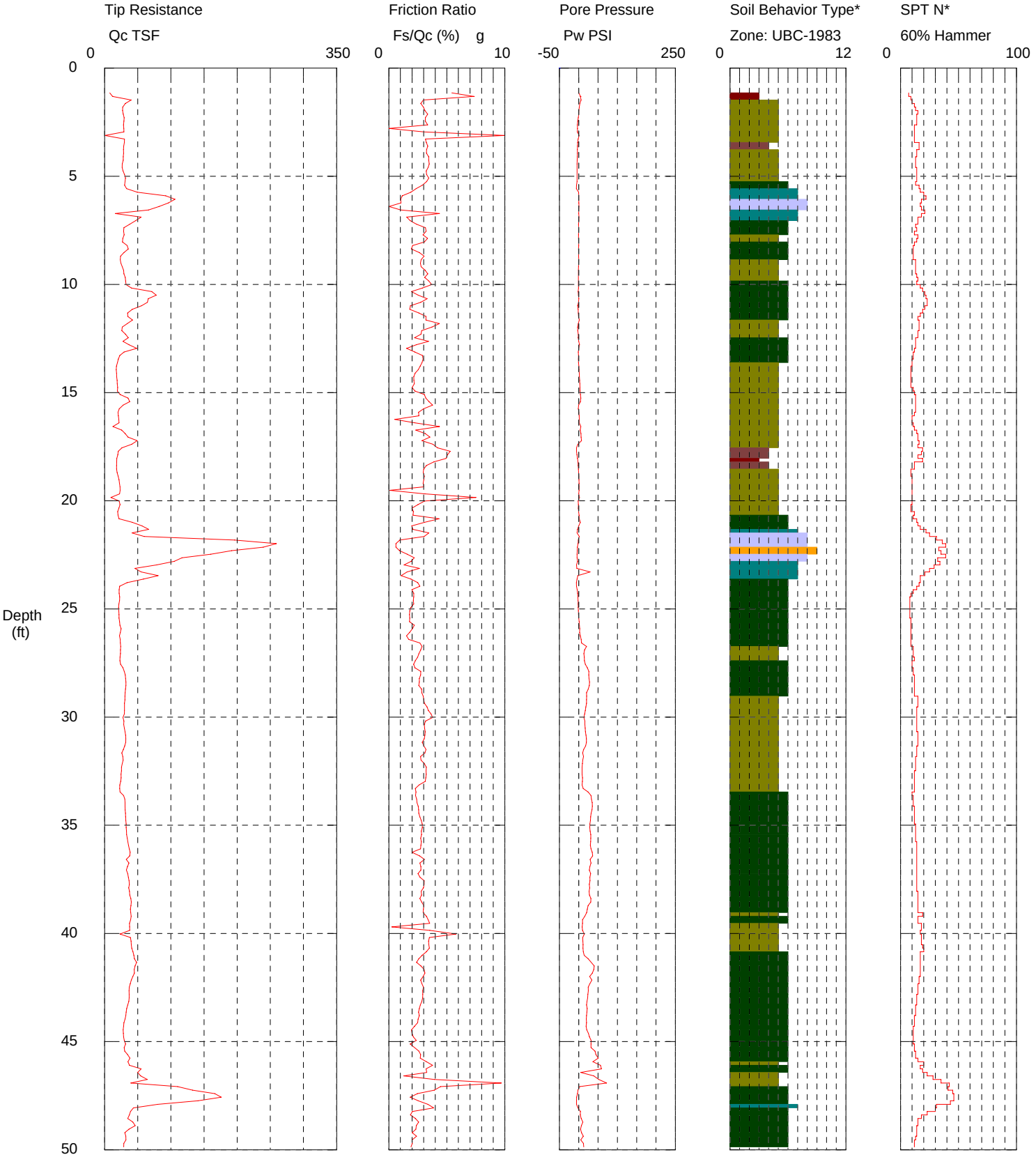
1 sensitive fine grained
2 organic material
3 clay
Filter On
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Operator: Rocco
Sounding: 3
Cone Used: DDG1316

CPT Date/Time: 11/10/2015 11:17:41 AM
Location: San Ramon
Job Number: NMI-545



Maximum Depth = 50.20 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

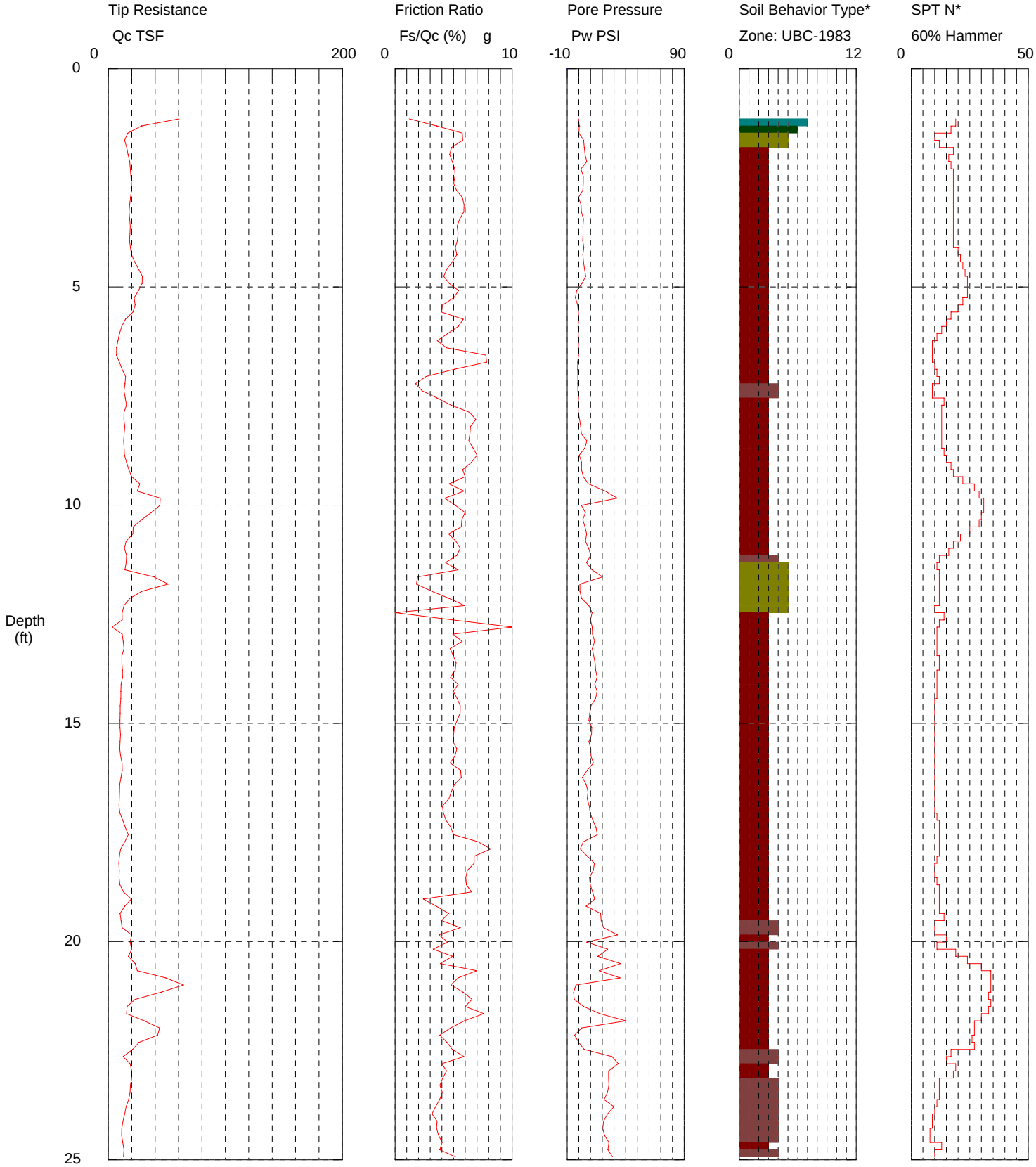
Filter On

*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Operator: Rocco
Sounding: 4
Cone Used: DDG1316

CPT Date/Time: 11/10/2015 9:58:54 AM
Location: San Ramon
Job Number: NMI-545



Maximum Depth = 25.75 feet

Depth Increment = 0.164 feet

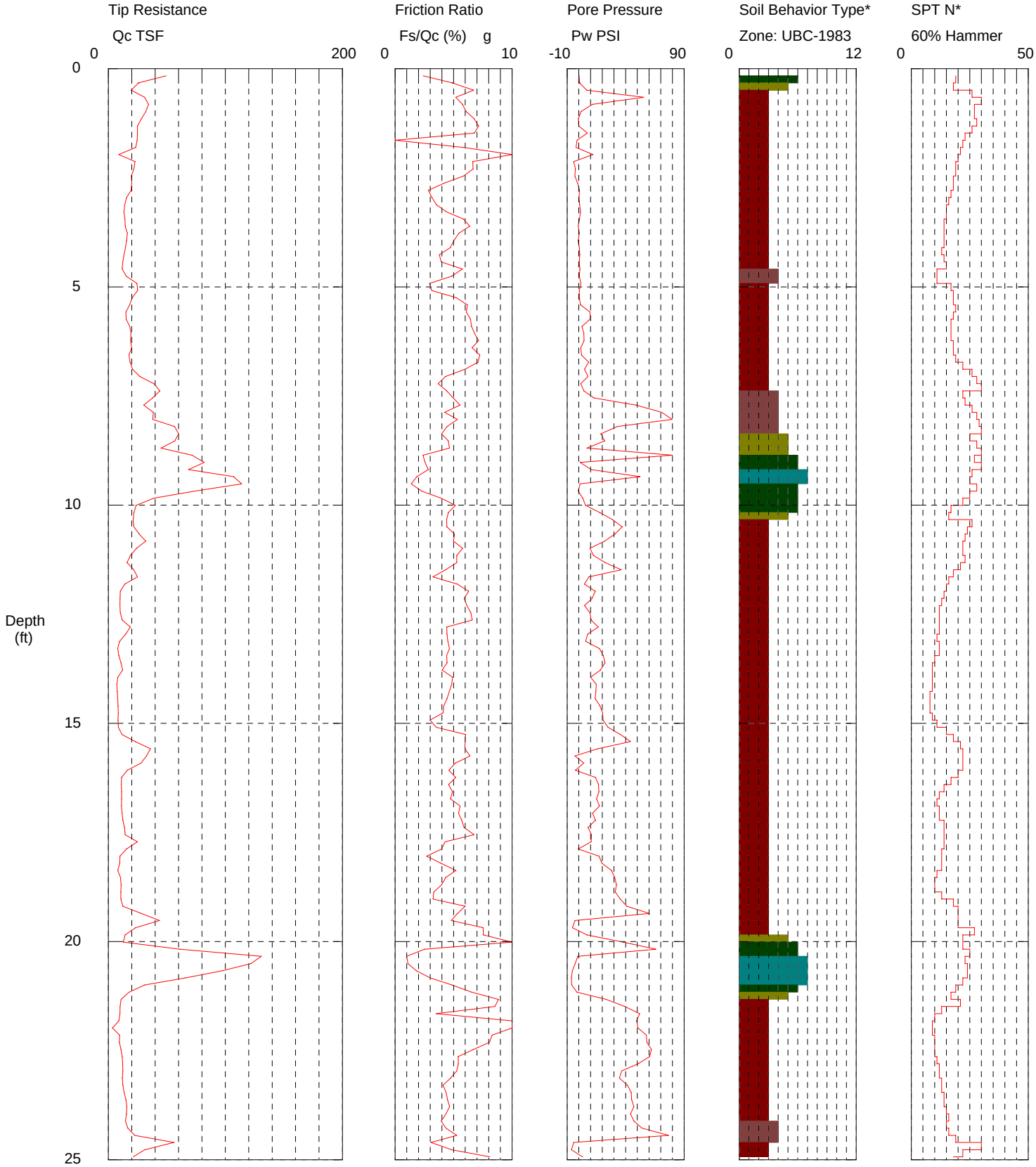
1 sensitive fine grained
2 organic material
3 clay
Filter On
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Operator: Rocco
Sounding: 5
Cone Used: DDG1316

CPT Date/Time: 11/10/2015 12:54:57 PM
Location: San Ramon
Job Number: NMI-545



Maximum Depth = 25.10 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

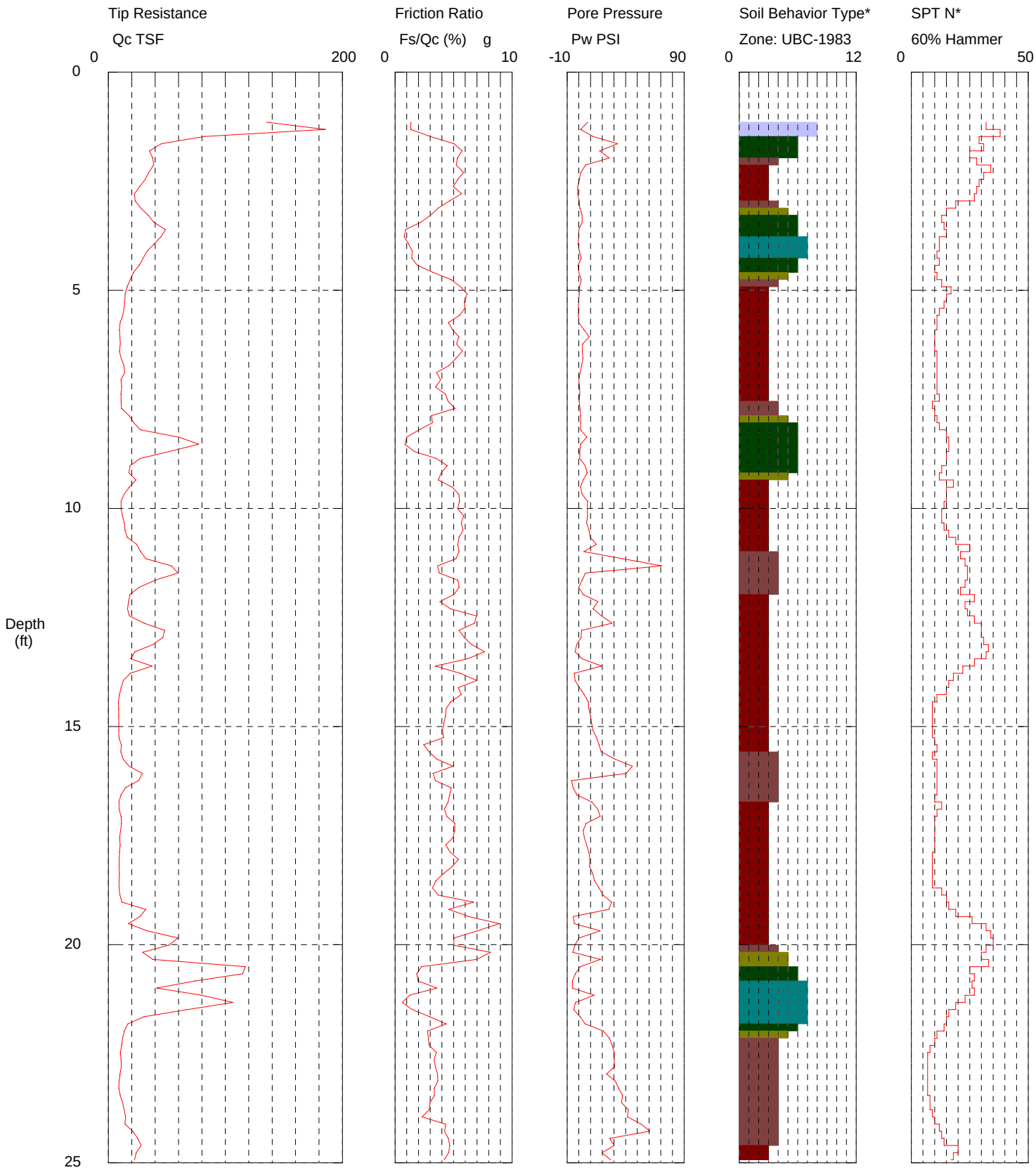
Filter On

*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Operator: Rocco
Sounding: 6
Cone Used: DDG1316

CPT Date/Time: 11/10/2015 12:03:54 PM
Location: San Ramon
Job Number: NMI-545



Maximum Depth = 25.26 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Filter On

*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

Moisture Content

The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216. The test results are presented on the logs of the exploratory borings in Appendix A.

In-Place Density Tests

The dry density of relatively undisturbed samples obtained from the exploratory borings was evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures C-1 through C-4. The test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System (USCS).

200 Wash

Evaluations of the percentage of particles finer than the No. 200 sieve in selected soil samples were performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure C-5.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure C-6.

Consolidation Tests

Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the

amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures C-7 and C-8.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures C-9 and C-10.

Expansion Index Tests

The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the test are presented on Figure C-11.

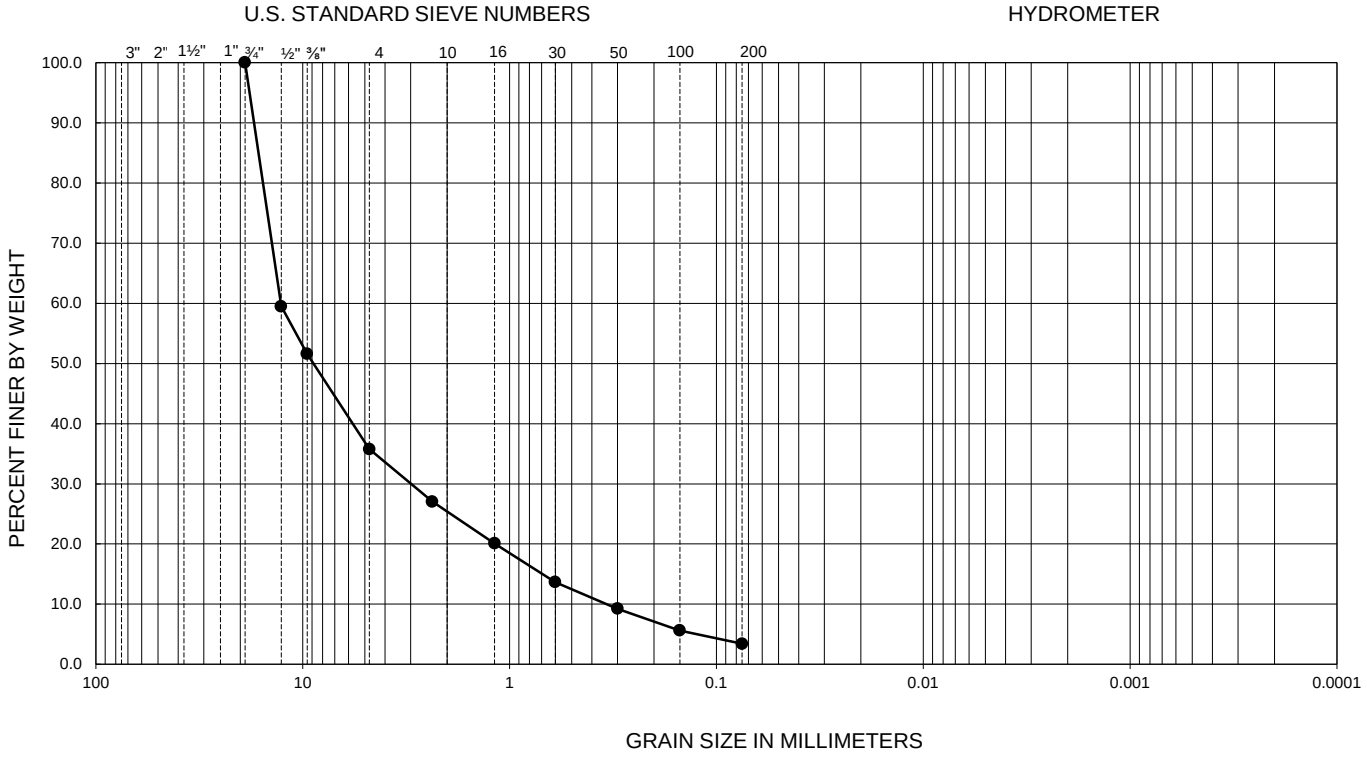
Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on representative samples in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-12.

Unconfined Compression Tests

Unconfined compression tests were performed on relatively undisturbed samples in general accordance with ASTM D 2166. The test results are shown on Figure C-13.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

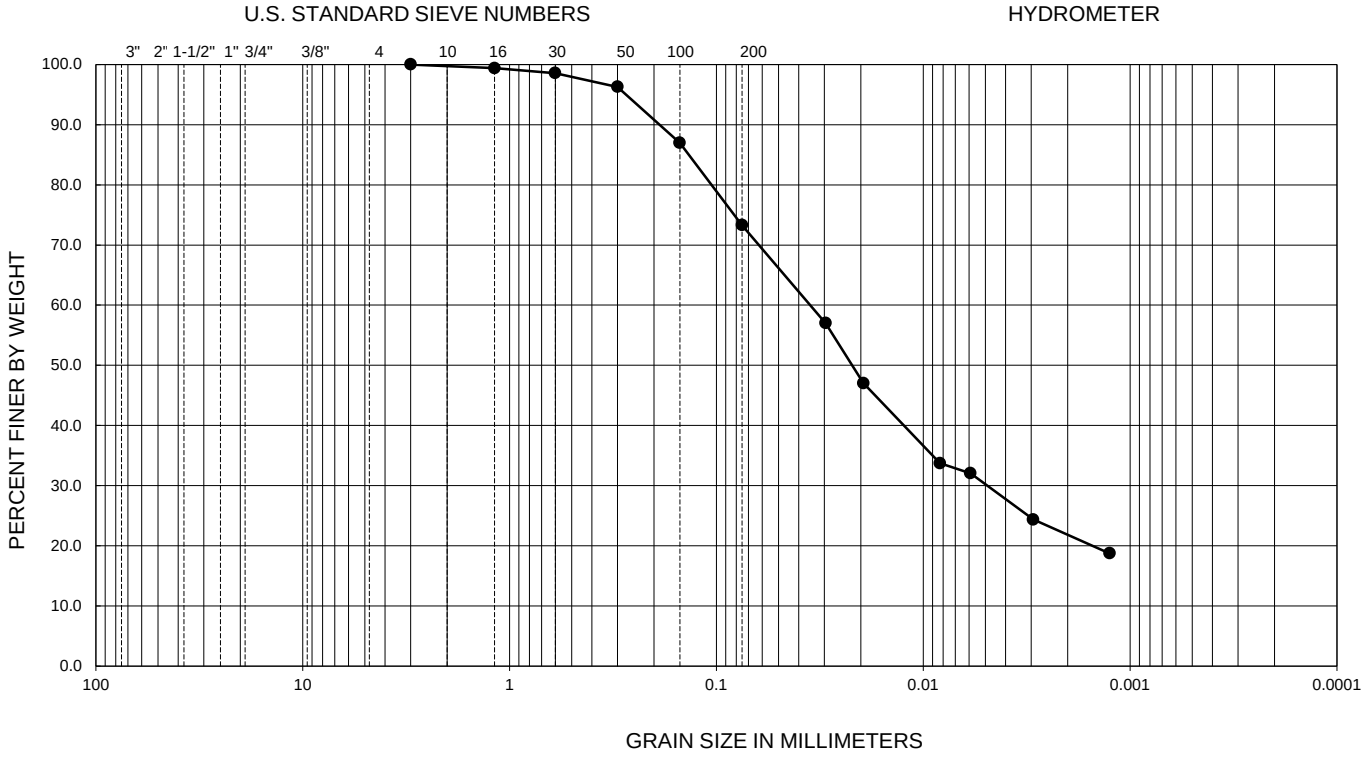


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-1	58.0-59.5	--	--	--	0.35	3.17	12.7	36.1	2.25	3	GW

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-1
402676001 / 1/16		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



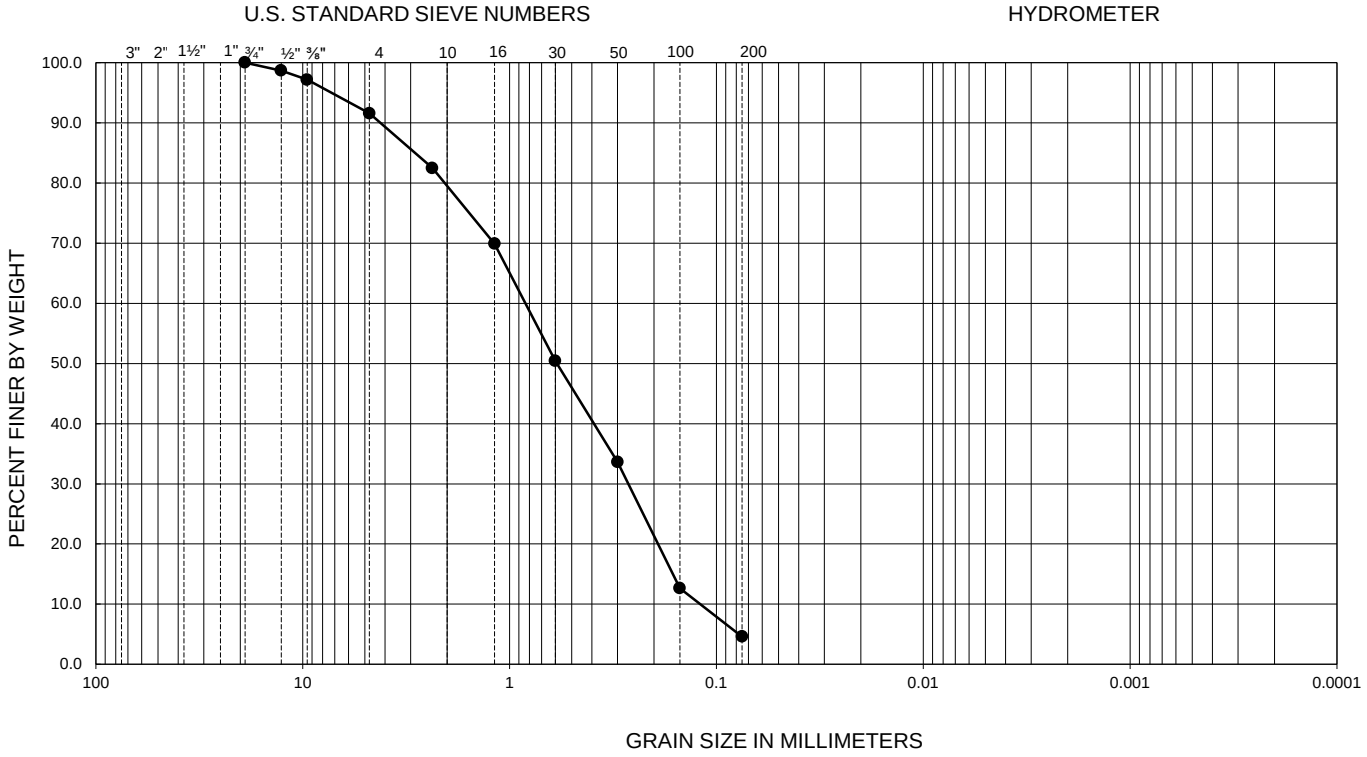
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-2	6.0-6.5	39	20	19	- -	0.01	0.04	- -	- -	73	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-2
402676001 / 1/16		

402676001 Hydrometer B2@6-6.5 Ft 12-1-15

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



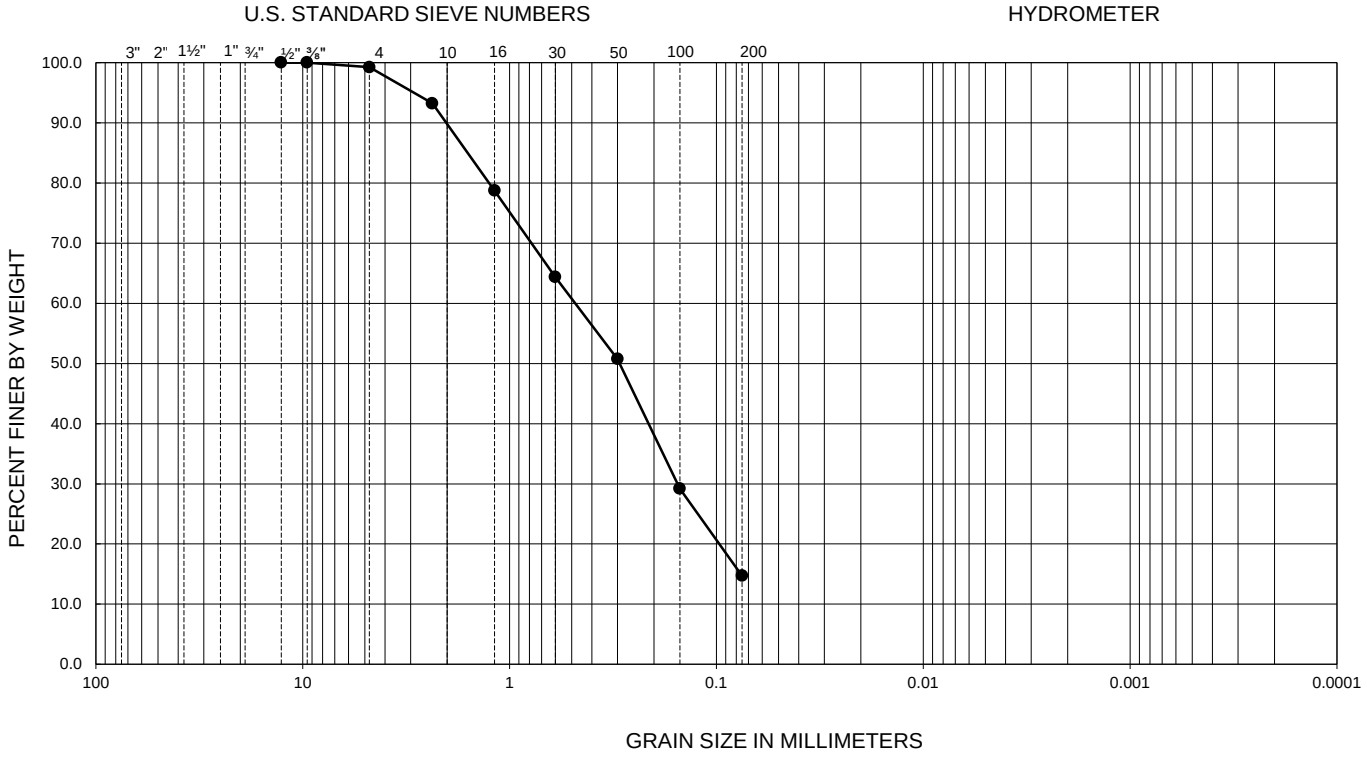
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-2	33.0-34.5	--	--	--	0.13	0.27	0.89	7.1	0.7	5	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-3
402676001 / 1/16		

402676001 SIEVE B2 @ 33'-34.5' 11-20-15

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-5	23.0-24.5	--	--	--	--	0.16	0.50	--	--	15	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-4
402676001 / 1/16		

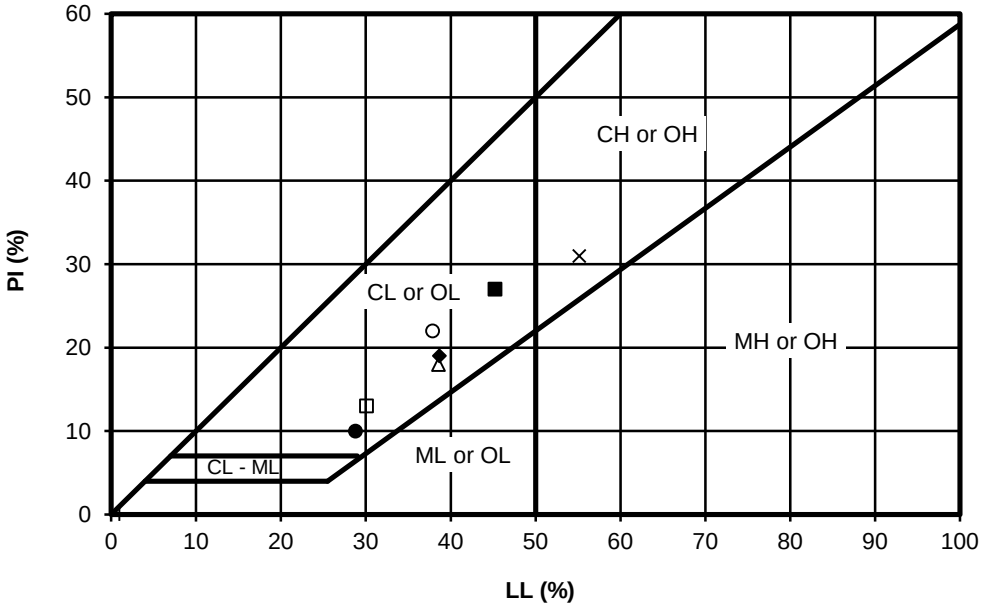
402676001 SIEVE B5 @ 24'-24.5' 11-20-15

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	18.5-19.0	Brown to dark brown sandy CLAY	97	52	CL
B-2	24.0-24.5	Olive brown lean CLAY; some sand	96	56	CL
B-4	14.0-14.5	Light brown lean CLAY; some sand	99	61	CL
B-5	9.0-9.5	Light brown lean CLAY; little sand	100	81	CL
B-6	19.0-19.5	Brown fat CLAY; few sand	100	94	CH

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

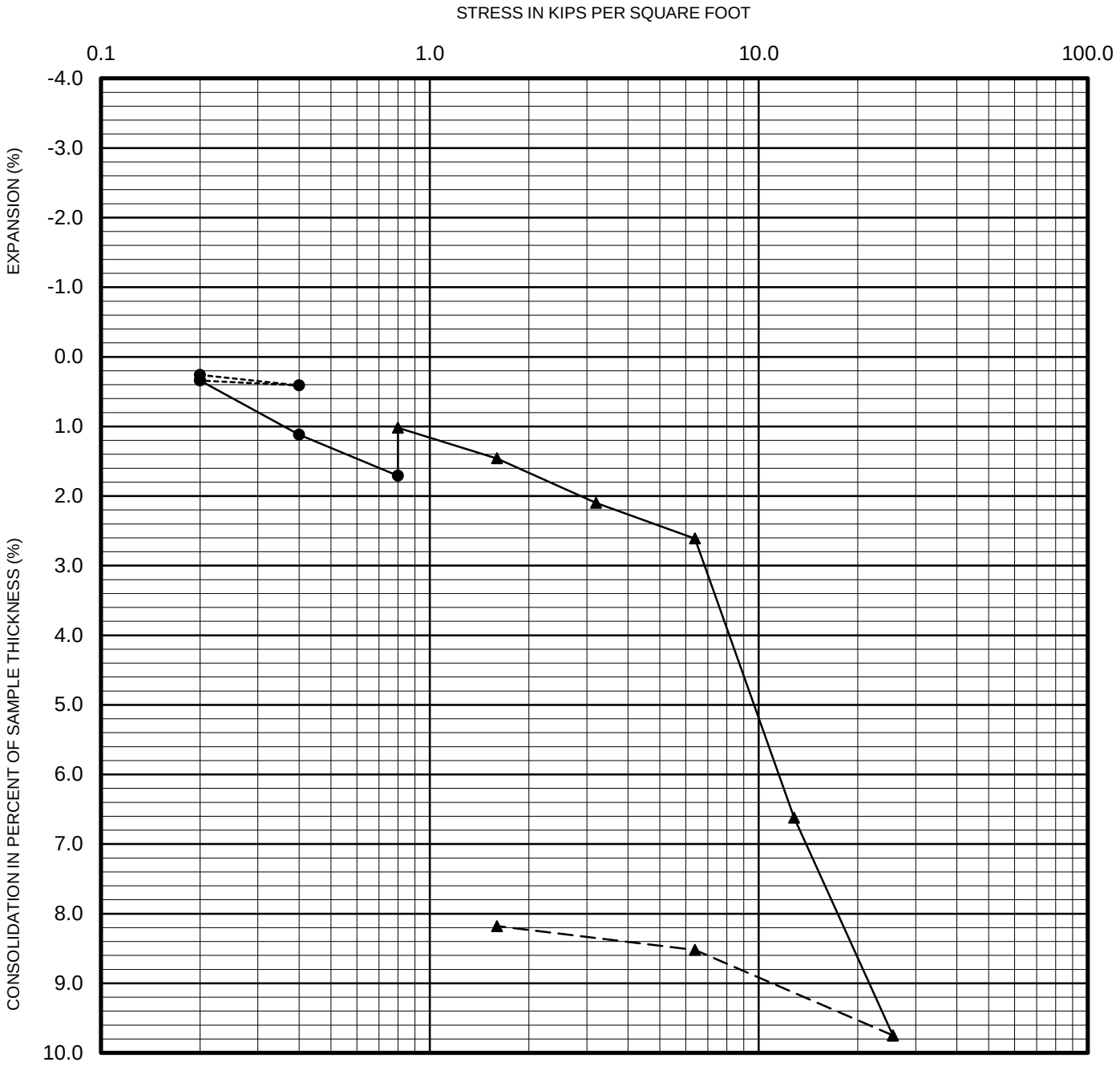
Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT	DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-5
402676001	1/16		

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL (%)	PLASTIC LIMIT, PL (%)	PLASTICITY INDEX, PI (%)	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	18.5-19.0	29	19	10	CL	CL
■	B-1	23.0-24.5	45	18	27	CL	CL
◆	B-2	6.0-6.5	39	20	19	CL	CL
○	B-2	24.0-24.5	38	16	22	CL	CL
□	B-4	14.0-14.5	30	17	13	CL	CL
Δ	B-5	9.0-9.5	39	21	18	CL	CL
X	B-6	19.0-19.5	55	24	31	CH	CH



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

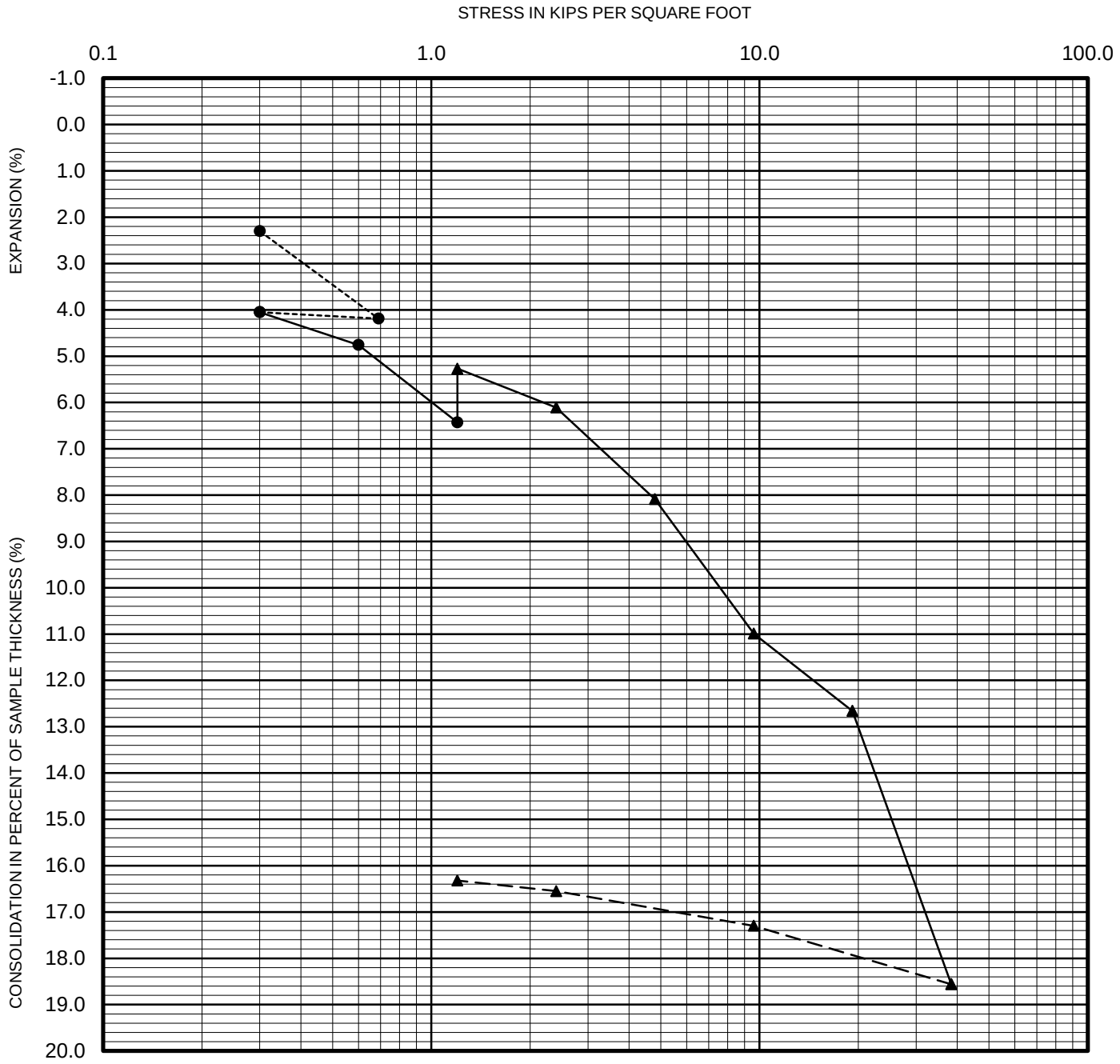
Ninyo & Moore	ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-6
402676001 / 1/16		



- - -●- - -	Seating Cycle	Sample Location	B-1
—●—	Loading Prior to Inundation	Depth (ft.)	6.0-6.5
—▲—	Loading After Inundation	Soil Type	CL
– –▲– –	Rebound Cycle		

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

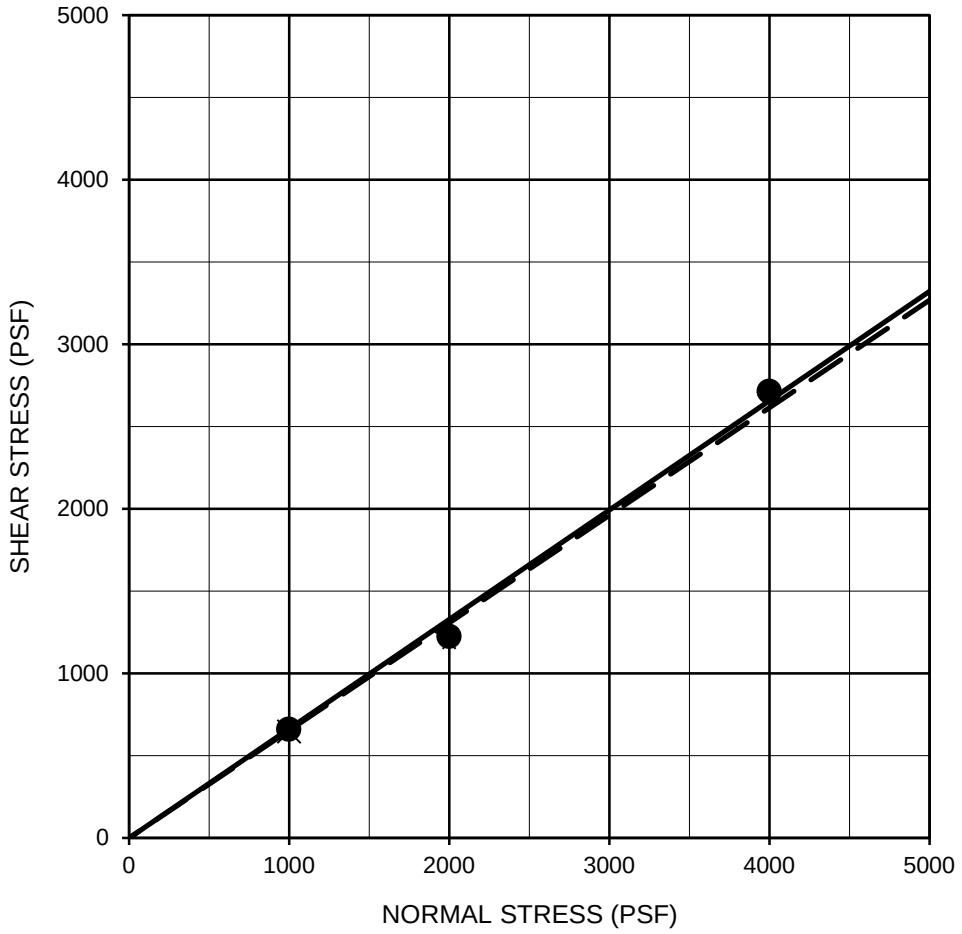
Ninyo & Moore	CONSOLIDATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-7
402676001 / 1/16		



Seating Cycle		Sample Location	B-4
Loading Prior to Inundation		Depth (ft.)	9.0-9.5
Loading After Inundation		Soil Type	CL-ML
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

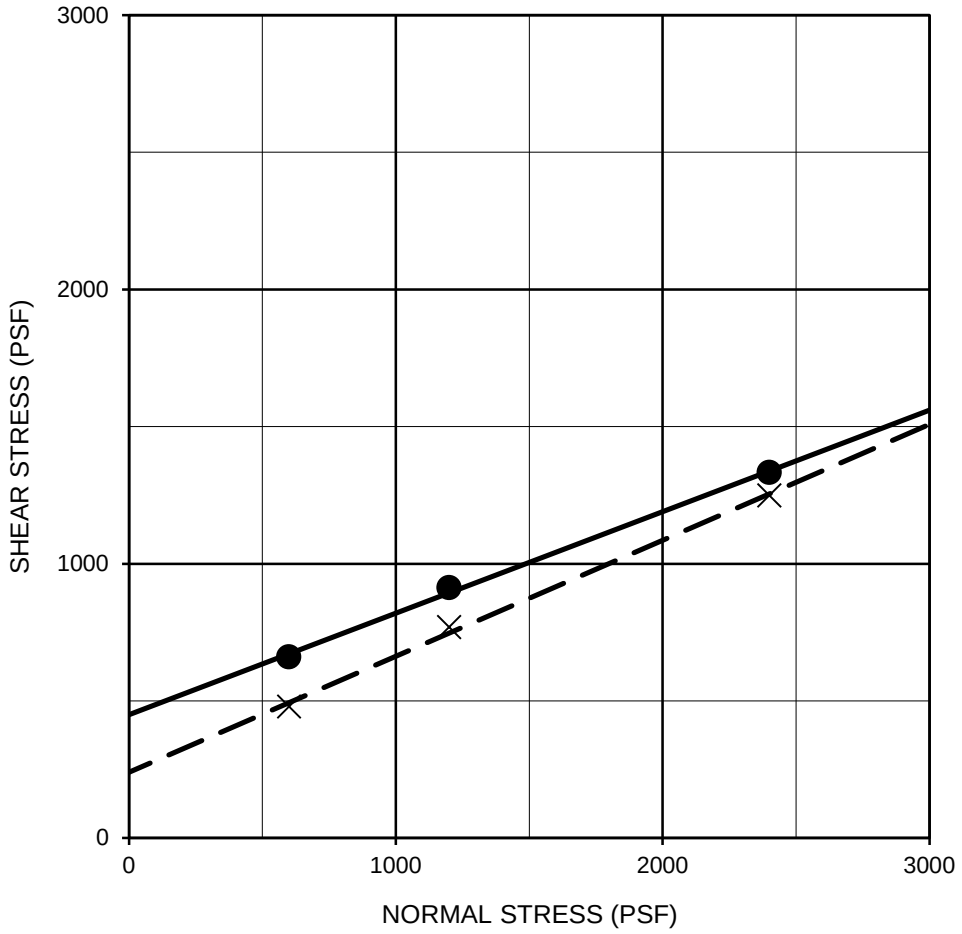
Ninyo & Moore		CONSOLIDATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-8
402676001	1/16		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty CLAY; some sand	—●—	B-3	9.0-9.5	Peak	0	34	CL
Silty CLAY; some sand	- - x - -	B-3	9.0-9.5	Ultimate	0	33	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore	DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-9
402676001 / 1/16		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Lean CLAY; some sand	—●—	B-4	6.0-6.5	Peak	450	20	CL
Lean CLAY; some sand	- - x - -	B-4	6.0-6.5	Ultimate	240	23	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore	DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-10
402676001 / 1/16		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-6	0.0-5.0	11.5	103.0	26.4	0.079	79	Medium

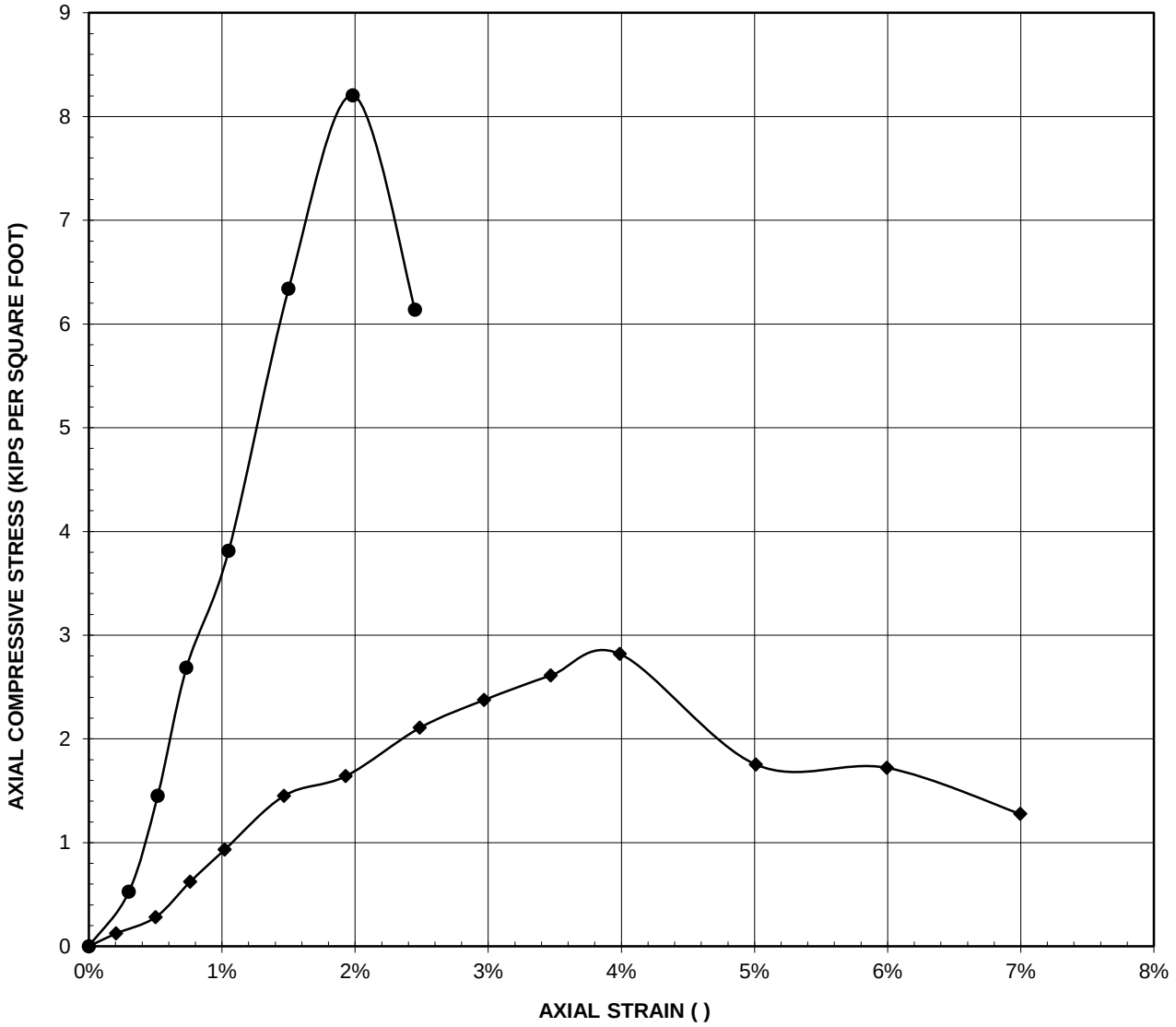
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4829

Ninyo & Moore		EXPANSION INDEX TEST RESULTS	FIGURE
PROJECT NO.	DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-11
402676001	1/16		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	0.0-5.0	7.1	6,499	90	0.009	70

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-12
402676001	1/16		



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Brown lean CLAY; some sand	CL	B-2	6.0-6.5	18.8	103.2	1.00	1.41
●	Light brown sandy CLAY	CL	B-6	6.0-6.5	10.5	108.1	1.22	4.10

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166

Ninyo & Moore	UNCONFINED COMPRESSION RESULTS	FIGURE
PROJECT NO. / DATE	CITY CENTER SAN RAMON 2678 BISHOP DRIVE SAN RAMON, CALIFORNIA	C-13
402676001 / 1/16		

402676001 UC B-2 @ 6-6.5 Ft 11-24-15

APPENDIX D

LIQUEFACTION AND DYNAMIC SETTLEMENT CALCULATIONS

LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	CITY CENTER SAN RAMON		
Project Number:	402676001		
Calculation By:	JAC	Date:	12/4/2015
Checked By:	PCC	Date:	12/15/2015

CPT Sounding: CPT-1
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	16
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.976
Design Earthquake Moment Magnitude, Mw	6.9
Magnitude Scaling Factor, MSF	1.24
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.08

Estimated dry soil dynamic settlement (in)	0.00
Estimated saturated soil dynamic settlement (in)	0.03
Total estimated dynamic settlement (in)	0.03

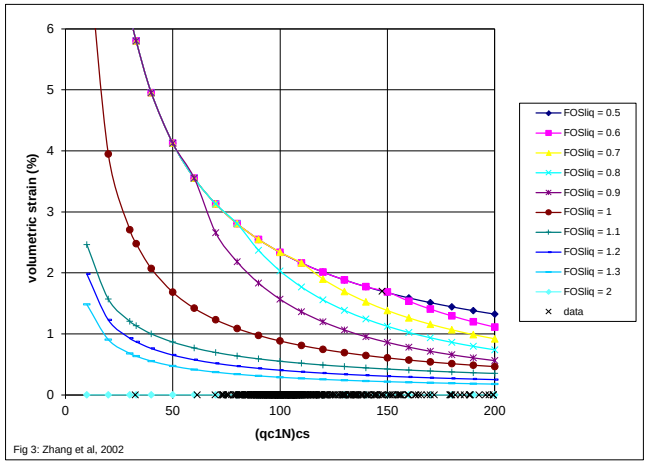
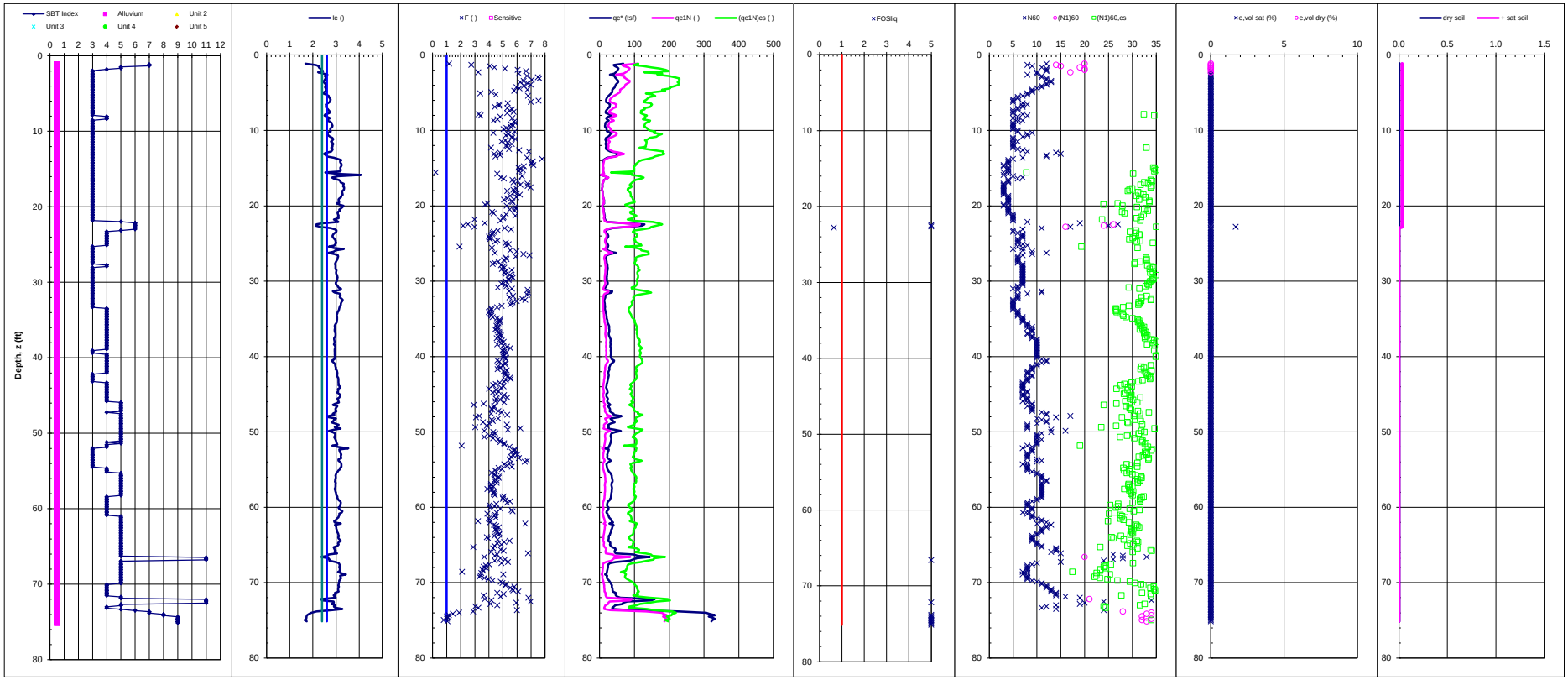


Fig 3: Zhang et al, 2002



LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	CITY CENTER SAN RAMON		
Project Number:	402676001		
Calculation By:	JAC	Date:	12/4/2015
Checked By:	PCC	Date:	12/9/2015

CPT Sounding: CPT-2
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	16
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.976
Design Earthquake Moment Magnitude, Mw	6.9
Magnitude Scaling Factor, MSF	1.24
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.08

Estimated dry soil dynamic settlement (in)	0.53
Estimated saturated soil dynamic settlement (in)	0.51
Total estimated dynamic settlement (in)	1.04

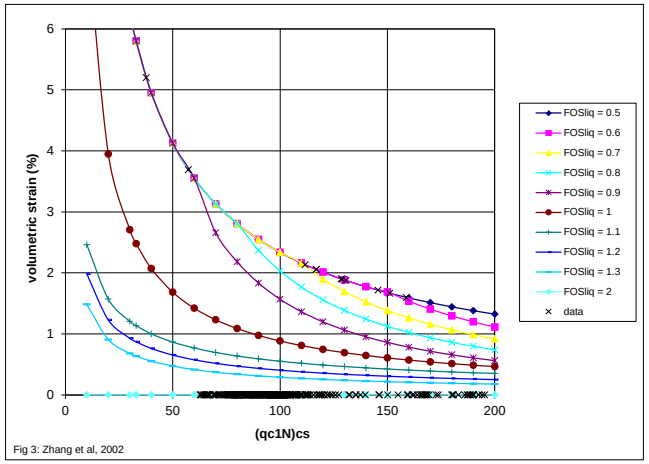
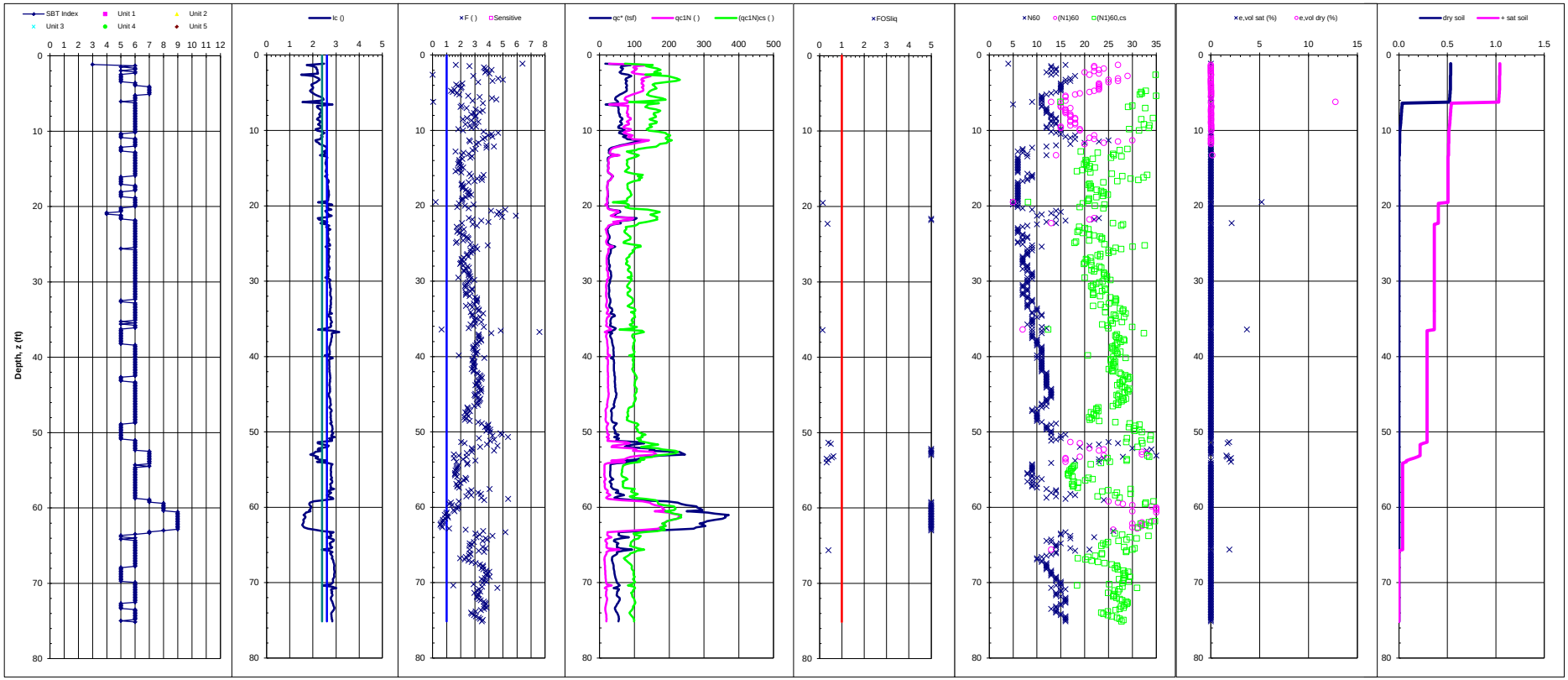


Fig 3: Zhang et al, 2002



LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	CITY CENTER SAN RAMON		
Project Number:	402676001		
Calculation By:	JAC	Date:	12/4/2015
Checked By:	PCC	Date:	12/9/2015

CPT Sounding: CPT-3
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	16
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.976
Design Earthquake Moment Magnitude, Mw	6.9
Magnitude Scaling Factor, MSF	1.24
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.08

Estimated dry soil dynamic settlement (in)	0.84
Estimated saturated soil dynamic settlement (in)	0.65
Total estimated dynamic settlement (in)	1.49

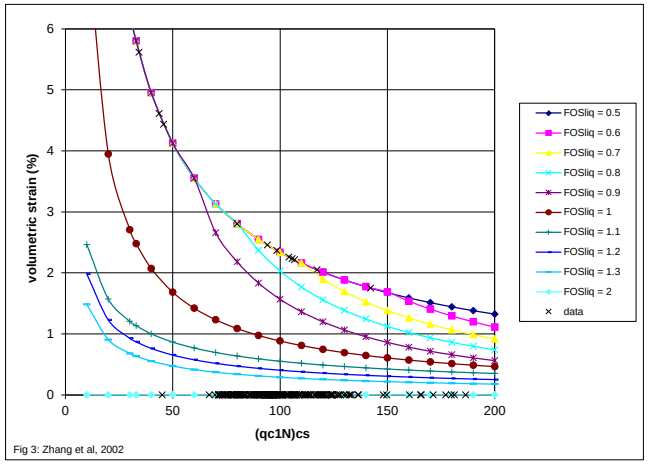
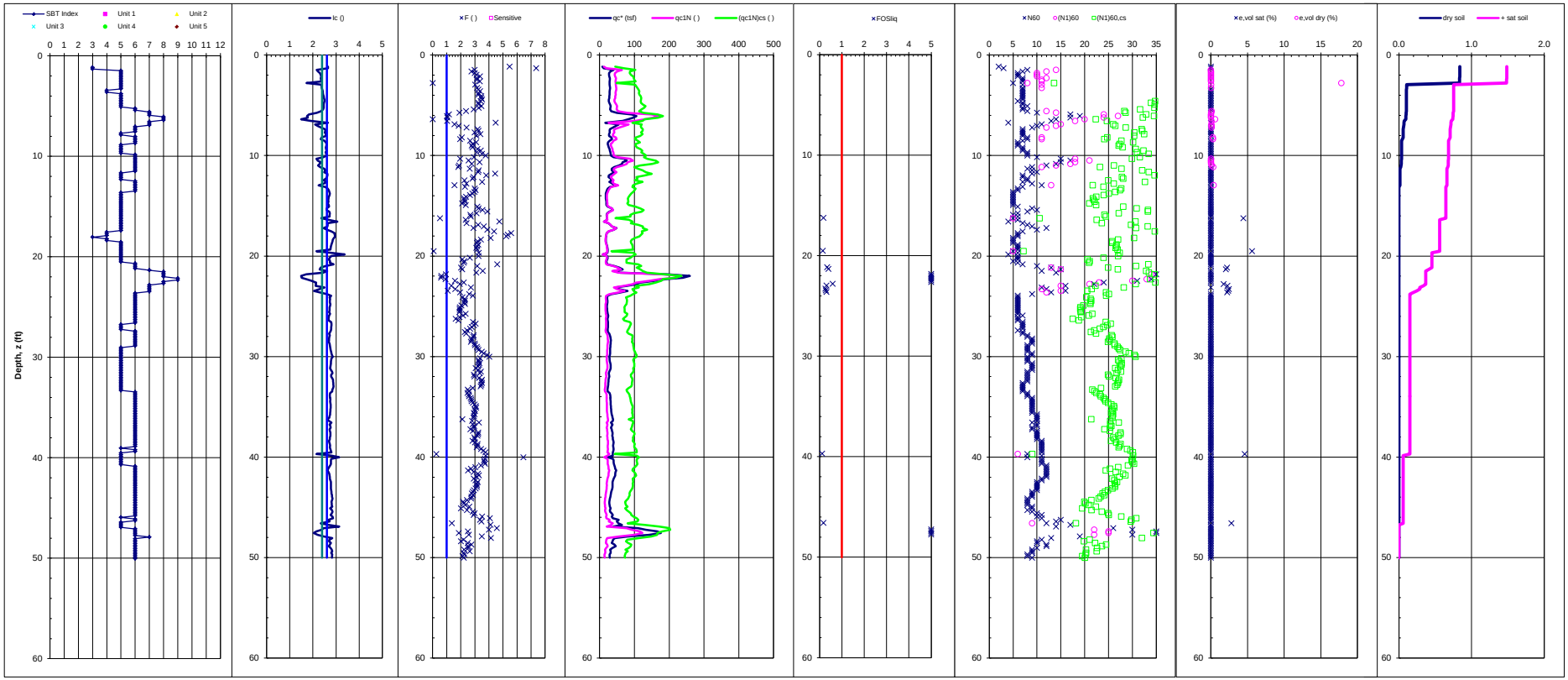


Fig 3: Zhang et al, 2002



LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	CITY CENTER SAN RAMON		
Project Number:	402676001		
Calculation By:	JAC	Date:	12/4/2015
Checked By:	PCC	Date:	12/9/2015

CPT Sounding: CPT-4
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	16
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.976
Design Earthquake Moment Magnitude, Mw	6.9
Magnitude Scaling Factor, MSF	1.24
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.08

Estimated dry soil dynamic settlement (in)	0.02
Estimated saturated soil dynamic settlement (in)	0.00
Total estimated dynamic settlement (in)	0.02

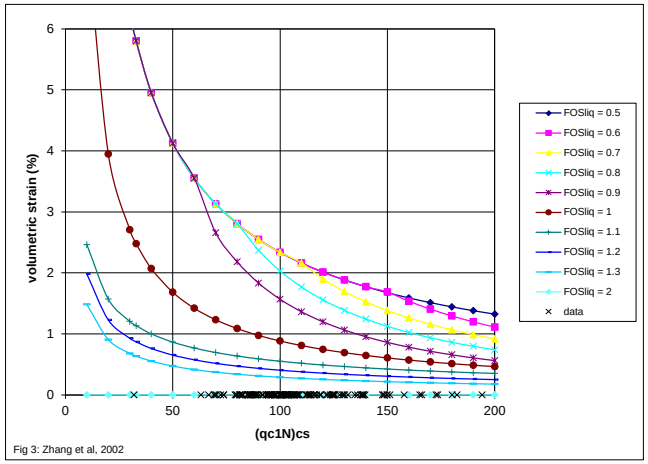
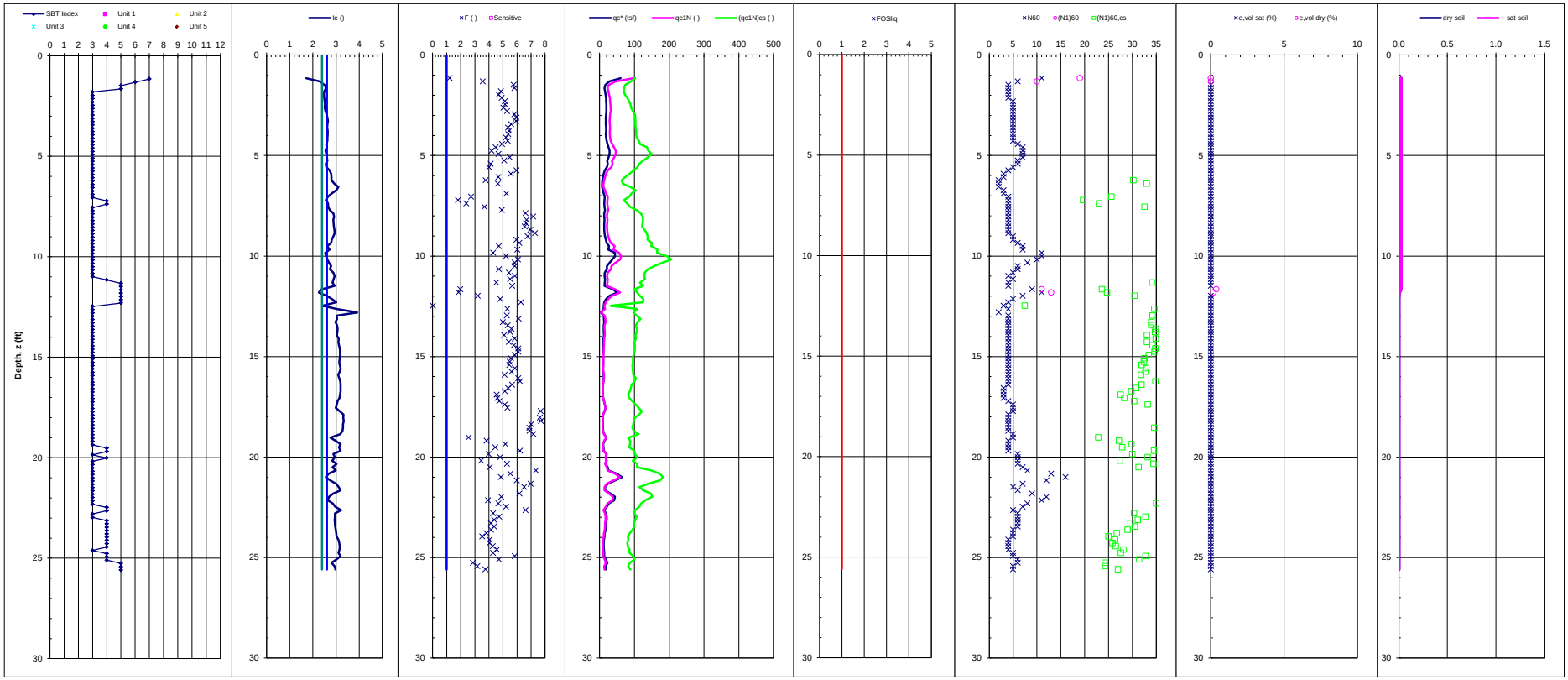


Fig 3: Zhang et al, 2002



LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	CITY CENTER SAN RAMON		
Project Number:	402676001		
Calculation By:	JAC	Date:	12/4/2015
Checked By:	PCC	Date:	12/9/2015

CPT Sounding: CPT-5
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	16
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.976
Design Earthquake Moment Magnitude, Mw	6.9
Magnitude Scaling Factor, MSF	1.24
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.08

Estimated dry soil dynamic settlement (in)	0.37
Estimated saturated soil dynamic settlement (in)	0.15
Total estimated dynamic settlement (in)	0.52

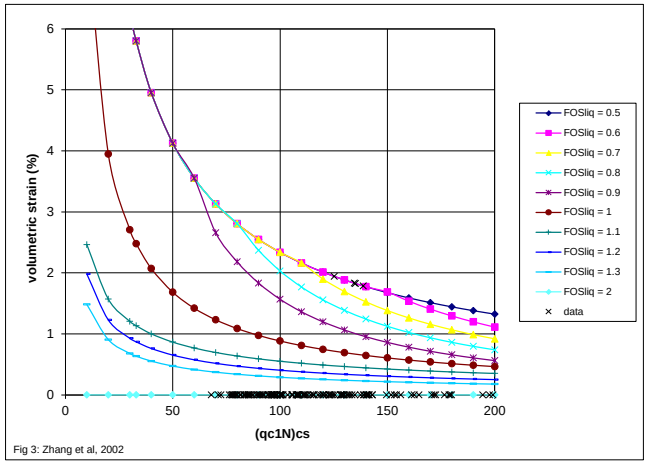
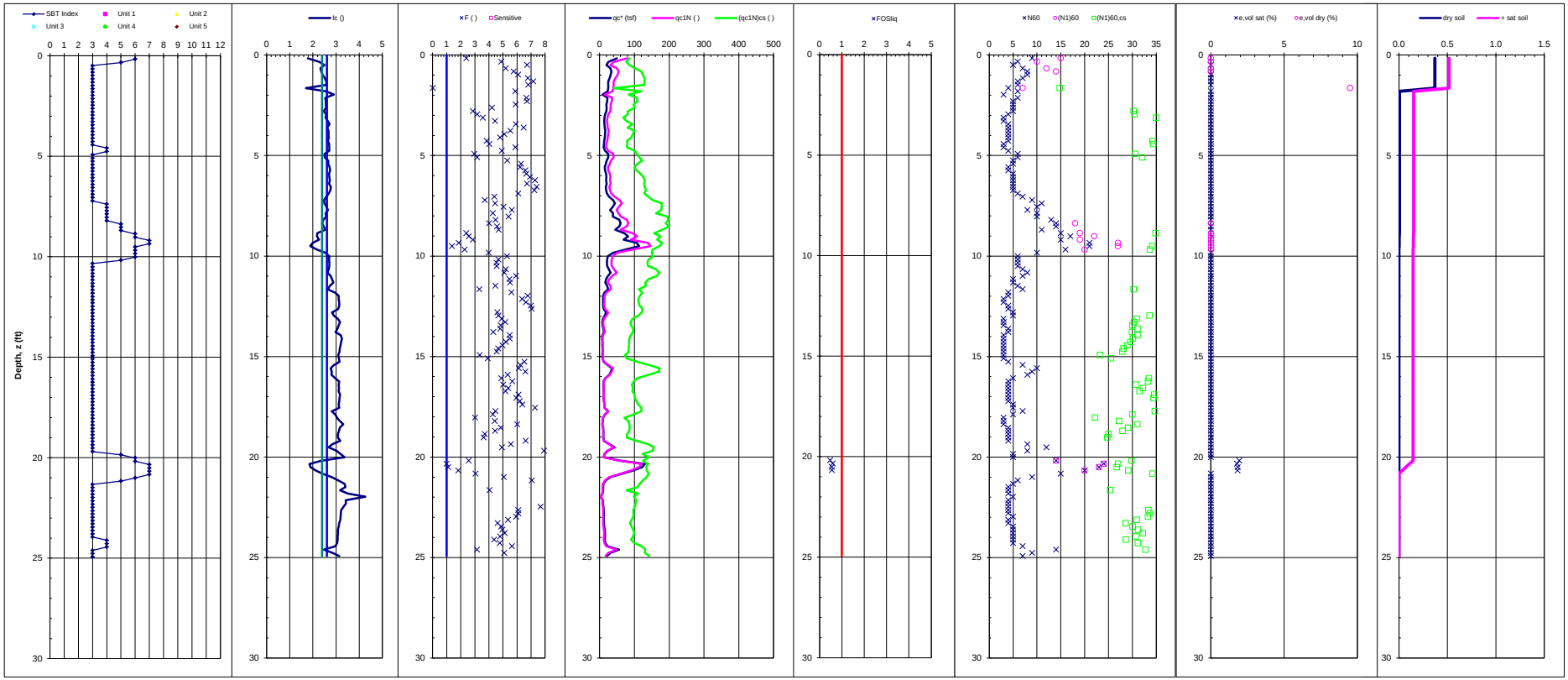


Fig 3: Zhang et al, 2002



LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	CITY CENTER SAN RAMON		
Project Number:	402676001		
Calculation By:	JAC	Date:	12/4/2015
Checked By:	PCC	Date:	12/9/2015

CPT Sounding: CPT-6
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	16
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.976
Design Earthquake Moment Magnitude, Mw	6.9
Magnitude Scaling Factor, MSF	1.24
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.08

Estimated dry soil dynamic settlement (in)	0.08
Estimated saturated soil dynamic settlement (in)	0.20
Total estimated dynamic settlement (in)	0.28

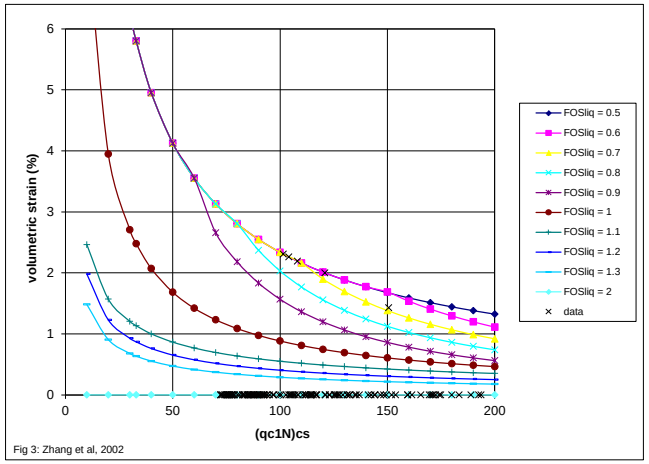
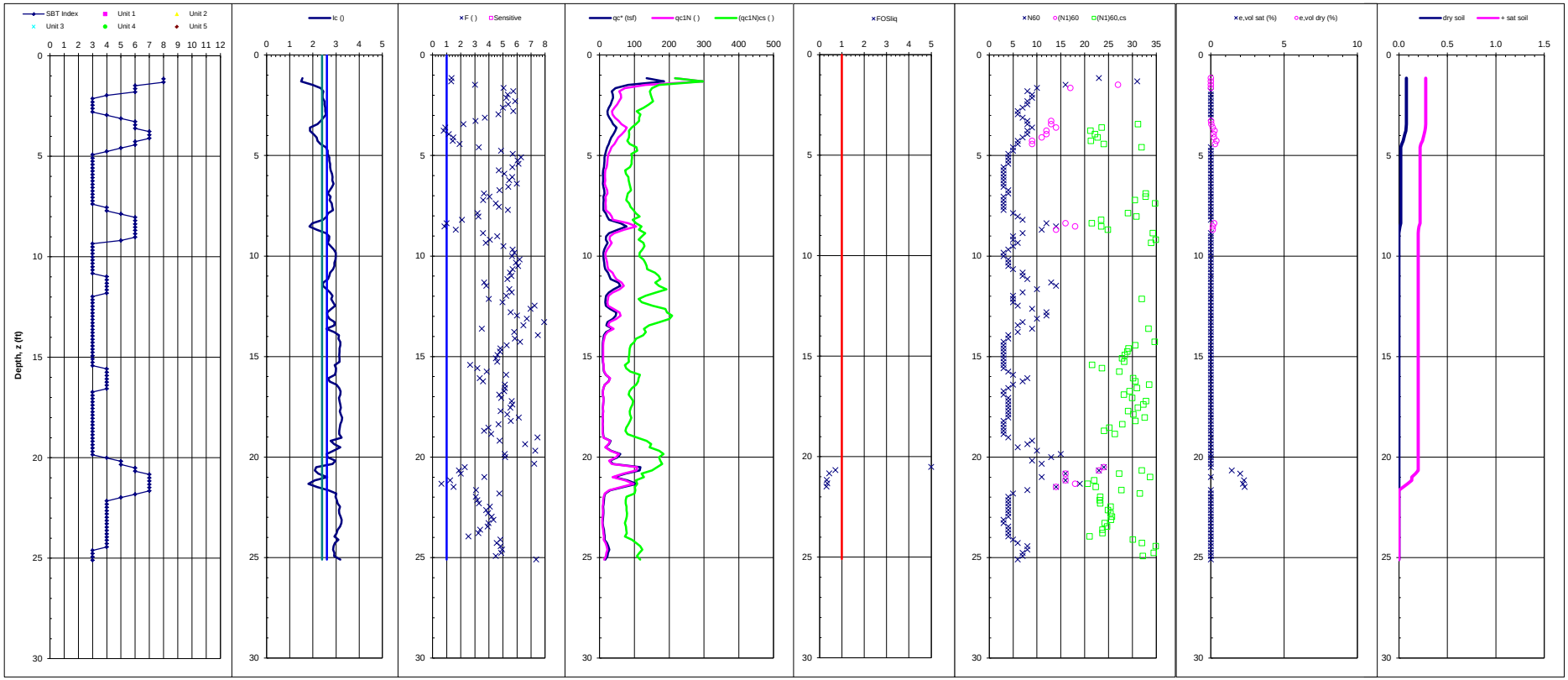



Fig 3: Zhang et al, 2002