APPENDIX C

Geotechnical Investigation

5150 El Camino Real
Los Altos, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Vahe Tashjian
Dutchints Development
289 S. San Antonio Road, #204
Los Altos, CA 94022

PREPARED BY
ENGEO Incorporated

March 9, 2018

PROJECT NO.
14723.000.000

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

ENGEO
Expect Excellence

Project No.
14723.000.000

March 9, 2018

Mr. Vahe Tashjian
Dutchints Development
289 S. San Antonio Road, #204
Los Altos, CA 94022

Subject: 5150 El Camino Real
Los Altos, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Tashjian:

We prepared this preliminary geotechnical report for the proposed residential development located at 5150 El Camino Real in Los Altos, California as outlined in our agreement dated January 23, 2018. This report presents our geotechnical observations, as well as our preliminary conclusions and recommendations. We also provide preliminary site grading, drainage, and foundation recommendations for use during land planning.

Based upon our initial assessment, the proposed residential development at 5150 El Camino Real is feasible from a geotechnical standpoint. Design-level exploration(s) should be conducted prior to site development once more detailed land plans have been prepared. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical report for design planning of the proposed residential development at 5150 El Camino Real in Los Altos, California. We prepared this report as outlined in our agreement dated January 23, 2018. Dutchints Development authorized us to conduct the following scope of services:

- Review of published geologic maps and available data.
- Conducting five Cone Penetration Tests (CPTs) ranging up to 45 feet deep.
- Prepare this preliminary geotechnical exploration report

This report was prepared for the exclusive use of our client and their consultants for evaluation of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the preliminary conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

Figure 1 displays a Site Vicinity Map. This site is located on the south edge of El Camino Real, adjacent the intersection of South Rengstorff Avenue and El Camino Real in Los Altos, California. The site is bordered by single- and multi-family residential structures along the west and south edges and by a single-story commercial structure to the east. Figure 2 shows site boundaries, and our exploratory locations. The site is approximately 3.8 acres

The site is currently occupied by a two- and three-story u-shaped office structure with asphalt paved perimeter parking areas.

1.3 PROJECT DESCRIPTION

Based on our discussion with Dutchints Development, we understand the following site improvements are proposed:

- 144 to 300, multi-family housing units.
- Paved streets, parking and drive lanes.
- Utilities and other infrastructure improvements.
- Concrete flatwork.
- Water quality facilities.

Civil grading plans were not available for our review; however, based on the proposed development and site conditions, we anticipate minor cuts and fills. We anticipate building loads will be typical of the proposed structure type.

2.0 FINDINGS

2.1 FIELD EXPLORATION

Our field exploration included advancing 5 Cone Penetration Tests (CPT) at various locations on the site. We performed our field exploration on February 15, 2018.

The location and elevations of our explorations are approximate. We estimated the locations of features shown on Figure 2; they should be considered accurate only to the degree implied by the method used.

2.1.1 Cone Penetration Tests

We retained a CPT rig to push the cone penetrometer to a maximum depth of approximately 45 feet in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). CPT logs are presented in Appendix A.

We retrieved near surface samples in three of the CPTs to perform plasticity index testing.

2.2 SITE BACKGROUND

Review of historic aerial photography indicates that the site was used for agricultural purposes until approximately the late-1940s or the early-1950s. Beginning in the mid-1950s, the site was incrementally developed through the mid- to late-1960s with various structures and paved parking. This previous development was razed in the mid-1980s to make way for the current development, which appeared around the same time.

2.3 GEOLOGY AND SEISMICITY

2.3.1 Geology

The study area is located within the Coast Ranges geomorphic province of California. The Coast Ranges are dominated by a series of northwest-trending mountain ranges that have been folded and faulted in a tectonic regime that involves both translational and compressional deformation. Regional geologic maps locate the site in the broad, north-south trending, alluvial filled Santa Clara Valley. Regional geologic mapping prepared by Graymer (2000) indicates the site is underlain by Pleistocene-age alluvial fan and fluvial deposits (Qpaf) as shown on Figure 3.

2.3.2 Seismicity

The San Francisco Bay Area contains numerous active earthquake faults. Nearby active faults are listed in Table 2.3.2-1. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Bryant and Hart, 2007). Figure 5 shows the approximate locations of these faults and significant historic earthquakes recorded within the San Francisco Bay Region.

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site, therefore, is not anticipated.

The active faults mapped within 20 miles of the site are listed in Table 2.3.2-1 by proximity to the site with their estimated maximum moment magnitude.

TABLE 2.3.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site
Latitude: 37.39604 **Longitude:** -122.10293

FAULT NAME	DISTANCE FROM SITE (MILES)	MAXIMUM MOMENT MAGNITUDE
Monte Vista-Shannon	3.3	6.5
North San Andreas	5.9	8.1
Hayward-Rodgers Creek	13.2	7.3
Calaveras	16.5	7.0
San Gregorio	17.6	7.5

The Working Group on California Earthquake Probabilities (WGCEP, 2008) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF generated an overall probability of 63 percent for the Bay Area as a whole, and a probability of 31 percent for the Hayward fault, 21 percent for the Northern San Andreas fault, and 7 percent for the Calaveras fault.

2.4 SURFACE CONDITIONS

The site is currently occupied by a two- and three-story u-shaped office structure with asphalt paved perimeter parking areas and minor landscaping.

2.5 SUBSURFACE CONDITIONS

We retrieved three grab samples of near-surface soil at 1-CPT3, 1-CPT4, and 1-CPT5 from approximately 2 feet below ground surface (bgs). The material was generally a brown to olive-brown, sandy lean clay.

The CPT data processing provides interpretations of the subsurface conditions based on empirical correlations with the cone tip and sleeve friction resistance. In general, the CPT data interpretations indicated that roughly the upper 20 feet of the site is underlain by clay and silty clay with relatively thin intermittent layers of sand and silty sand. The site is further underlain with interbedded layers of medium dense to dense sand, and stiff clay to the bottom of our explorations at approximately 45 feet bgs. Interpretations of CPT-05 indicated a soil profile of predominantly sand and gravel to the bottom of the exploration where refusal was met at approximately 35 feet bgs. Additional exploration should be performed to further characterize the variations in the subsurface prior to final project design.

The plasticity indices of the clay layers ranged from 18 to 36, indicating a moderate to high expansion potential.

Given the previous agricultural use of the site, the near surface soil was likely exposed to seasonal historic tilling, discing, and otherwise disruption over the years of cultivation. While this near-surface soil is likely native to the site, due to the previous agricultural use and absence of grading records, it should be considered non-engineered fill from an engineering standpoint.

The Site Plan (Figure 2) and CPT data (Appendix A) provide subsurface interpretations at each exploration location. The CPT data graphically depict the subsurface conditions encountered at the time of the exploration.

2.6 GROUNDWATER CONDITIONS

Groundwater was not observed nor measured due to the method of exploration used. Groundwater mapping in the Seismic Hazard Zone Report for the Mountain View 7.5-Minute Quadrangle, Santa Clara, Alameda, and San Mateo Counties, CA (CGS, 2006) indicates groundwater may be encountered at approximately 20 to 25 feet bgs at the site.

The groundwater levels at the site may fluctuate with time due to seasonal conditions, rainfall, and irrigation practices.

2.7 LABORATORY TESTING

We tested select samples recovered during drilling activities to determine various soil characteristics as presented on the following table.

TABLE 2.7-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
Moisture Content	ASTM D-2216	Appendix B
Plasticity Index	ASTM D-4318	Appendix B

3.0 PRELIMINARY CONCLUSIONS

From a geotechnical engineering standpoint, the site is suitable for the proposed development, provided the preliminary geotechnical recommendations in this report and future design-level geotechnical exploration studies are properly incorporated into the design plans and specifications.

A design-level geotechnical exploration should be performed as part of the design process. The exploration may include borings, additional cone penetration tests, test pits, and additional laboratory soil testing to provide data for preparation of specific recommendations regarding grading, foundation design, and drainage for the proposed development. The exploration will also allow for more detailed evaluations of the geotechnical issues, discussed below, and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

The primary geotechnical concerns that could affect development on the site are expansive soil, non-engineered fill, and liquefaction hazards. We summarize our conclusions below.

3.1 EXPANSIVE SOIL

We tested samples of the existing near-surface soil for plasticity index (PI) to estimate expansive potential. As discussed in Section 2.4, the existing near-surface soil samples tested yielded PIs that ranged from 18 to 36, which indicate moderate to high expansion potential.

Expansive soil can change in volume with changes in moisture. They can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Successful construction on expansive soil requires special attention during grading. It is imperative to keep exposed soils moist by occasionally sprinkling. If the soil drys, it is extremely difficult to remoisturize the soil without excavation, moisture conditioning, and recompaction.

Conventional grading operations, incorporating our fill placement specifications tailored to the expansive characteristics of the soil, and use of a mat foundation (either post-tensioned or conventionally reinforced) are generally cost-effective measures to address the expansive potential of the foundation soils.

3.2 NON-ENGINEERED FILL

Disturbed native and non-engineered fills can undergo excessive settlement, especially under new fill or building loads. As previously mentioned, the site had likely been seasonally tilled during agricultural usage and with no grading records, should be treated as non-engineered soil. Non-engineered soil is prone to settlement under new structural loads or may exhibit volume loss when compacted during grading operations. To mitigate the effects of the disturbed near-surface materials, we recommend complete removal and recompaction. Section 4.4 provides recommendations for fill subgrade preparation to address this material.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, liquefaction, and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lateral spreading, landslides, tsunamis, flooding or seiches is considered low at the site.

3.3.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, ground rupture is unlikely at the subject property.

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the current California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage, but with some nonstructural damage, and (3) resist major earthquakes without collapse, but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.3.3 Liquefaction

Liquefaction is the loss of strength to soil layers due to cyclic loading or seismic shaking. Generally, loose coarse-grained material will undergo liquefaction under a seismic event. Based on observations of soil behavior under seismic shaking and laboratory testing, some fine-grained material, such as silt and clay, can also undergo liquefaction or cyclic softening. In order for a soil to be potentially liquefiable, it must be saturated. For this site, we considered the design groundwater depth to be at 22½ feet bgs.

We performed a detailed liquefaction potential analysis of the CPT soundings to estimate liquefaction potential using the computer software CLiq Version 2.1.6.11 developed by GeoLogismiki. We used a Peak Ground Acceleration (PGA) value of 0.6g as outlined in the latest building code and moment magnitude of 8.1. We performed our analysis of liquefaction potential using the Robertson (2009) method due to the fact that our site soil matches well with the criteria developed by the author. The criteria being that sand-like soil is evaluated based on density, intermediate soil is evaluated based on density and amount of fines, and clay-like soil is evaluated based on undrained shear strength.

The results of these calculations are presented in Appendix C, with our estimation of post-earthquake settlements. The analysis sheets in Appendix C summarize the CPT tip resistance, computed factor of safety, volumetric strain, and resulting settlement as a function of depth for each CPT. The plots directly show which soil layers liquefy and which do not. They also relate to soil behavior type zones that may contribute to site settlement, as well as the relative contribution of each zone, and the distribution of settlements with depth.

The analysis indicates that layers of medium dense sand and clay will settle up to approximately 2 inches due to cyclic softening and liquefaction. Based on the high end of the calculated total liquefaction settlements, the site improvements should be designed to withstand a differential settlement of 1 inch over a 30-foot distance and perform as intended. To mitigate the differential settlement for structures, we recommend post-tensioned or traditional reinforced mat foundations.

For design purposes, we recommend obtaining subsurface geotechnical data below the proposed foundation once the building layout and type are known. Sampling of the potentially liquefiable layers would provide additional information to further refine the liquefaction analysis and potentially reduce the total settlement.

3.3.4 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area region, but based on the site location, the offset is expected to be minor.

3.4 2016 CBC SEISMIC DESIGN PARAMETERS

We provide the 2016 CBC seismic design parameters in the table below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.4-1: 2016 CBC Seismic Design Parameters, Latitude: 37.39604 **Longitude:** -122.10293

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.55
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.68
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.55
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.02
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.03
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.68
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.60
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.60
Long period transition-period, T_L	12 sec

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

The following preliminary recommendations are for initial land planning and preliminary estimating purposes. Final recommendations regarding site grading and foundation construction will be provided after additional design-level geotechnical exploration has been undertaken.

4.1 GENERAL SITE CLEARING

Site development will commence with the removal of existing improvements and their foundations, and buried structures, including abandoned utilities and their backfill. All debris or soft compressible soil should be removed from any location to be graded, from areas to receive fill or structures, and from those areas to serve as borrow. Because the site was previously used for agriculture, we typically expect that the upper 2 to 3 feet of soil will need to be reworked to produce appropriately moisture conditioned and compacted material. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

Existing vegetation should be removed from areas to receive fill or structures, or those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. The actual depths of tree root removal should be determined by the Geotechnical Engineer's representative in the field. Subject to approval by the Landscape Architect, strippings and organically contaminated soils can be used in landscape areas. Otherwise, such soil should be removed from the study areas. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping is permitted.

4.2 SELECTION OF MATERIALS

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), we anticipate the site soil is suitable for use as engineered fill provided they are broken down to 6 inches or less in size. Other materials and debris, including trees with their root balls, should be removed from the study areas.

Imported fill material should meet the above requirements and have a plasticity index similar to onsite soil material. We should be given the opportunity to sample and test proposed imported fill material at least 5 days prior to delivery to the site.

4.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible.

Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather;
2. Mixing with drier materials;
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated and approved by the Geotechnical Engineer prior to implementation.

4.4 FILL COMPACTION

4.4.1 Grading in Structural Areas

The contractor should perform the following compaction control requirements for subgrade preparation and fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 12 inches.
2. Moisture condition soil to at least 4 percentage points over the optimum moisture content; **and**
3. Compact the soil to between 87 and 92 percent relative compaction. Compact the upper 6 inches of finish pavement subgrade to at least 90 percent relative compaction prior to aggregate base placement.

The contractor should compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to a minimum moisture content of optimum prior to compaction.

4.4.2 Landscape Fill

The contractor should process, place and compact fill in accordance with the recommendations in Section 5.0 except compact to at least 85 percent relative compaction (ASTM D1557).

4.5 SITE DRAINAGE

4.5.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.3 specifies minimum slopes of 5 percent away from foundations. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following:

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.

2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.

3. Do not allow water to pond near foundations, pavements, or exterior flatwork.

4.6 STORMWATER INFILTRATION AND SELECT PROJECT RISK LEVEL FACTORS

Due to the density of the site soil and high fines content (percentage passing the No. 200 sieve), the near-surface site soil is expected to have a low permeability value for stormwater infiltration in grassy swales or permeable pavers, unless subdrains are installed. Therefore, Best Management Practices should assume that limited stormwater infiltration will occur at the site.

4.7 STORMWATER BIORETENTION AREAS

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements subject to lateral loads (such as from impact or traffic patterns), additional design considerations may be recommended. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend we be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

The contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

4.8 LANDSCAPING CONSIDERATION

As the near-surface soil is moderately to highly expansive, we recommend greatly restricting the amount of surface water infiltration near structures, pavements, flatwork, and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements.

- Using low precipitation sprinkler heads.

- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system.

- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements.

- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements.

- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that these items be incorporated into the landscaping plans.

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

We developed preliminary foundation recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. The following preliminary recommended foundation options address the effects of the native expansive soil and differential soil movement:

1. Post-tensioned mat foundation.
2. Structural mat foundation.

For design purposes, we recommend obtaining subsurface geotechnical data below the proposed foundation once the building layout and type are known to develop design-level foundation recommendations.

5.1 STRUCTURAL MAT FOUNDATIONS

The proposed residential structures may be supported on structural mat foundation systems. We anticipate that structural mats constructed on swelling soil will move differentially. Structural mats may need to be stiffened to reduce differential movements due to swelling/shrinkage to a value compatible with the type of superstructure that will be constructed on them. The structural engineer should be consulted on this matter. We recommend that it be designed for an edge cantilever length of 8 feet with a random, interior unsupported span of 25 feet. Additionally, foundations should be designed for 1 inch of differential movement over a distance of 30 feet for the seismic case.

The perimeter should be thickened by 2 inches, and the minimum soil backfill height against the slab at the perimeter should be 6 inches. For preliminary planning purposes, structural mat foundations should be designed for a uniform bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live load. This value may be increased to 1,500 psf under individual columns or walls to accommodate stress concentrations at those locations. These values can be increased by one-third for seismic loading.

The thickness of the structural mat will be driven by the structural design. The structural mat should be underlain by a water vapor transmission reduction system as in Section 5.1.

5.2 POST-TENSIONED MAT FOUNDATIONS

The proposed residential structures may also be supported on post-tensioned (PT) mat foundations bearing on prepared native soil or compacted fill.

For preliminary planning purposes, PT mats should be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live loads, with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for all loads including wind or seismic. In addition to the parameters below, foundations should be designed for 1 inch of differential movement over a distance of 30 feet for the seismic case.

5.3 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with mats, water vapor from beneath the mat will migrate through the foundation and into the building. This water vapor can be reduced but not eliminated. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. Where water vapor migrating through the mat would be undesirable, we recommend the following measures to reduce water vapor transmission upward through the mat foundations.

1. Install a vapor retarder membrane directly beneath the mat. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E 1745-11 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.”

2. Concrete should have a concrete water-cement ratio of no more than 0.5.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Consider and implement adequate moist cure procedures for mat foundations.

5. Protect foundation subgrade soils from seepage by providing impermeable plugs within utility trenches.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

5.4 SUBGRADE TREATMENT FOR MAT FOUNDATIONS

The subgrade material under structural mats should be uniform. The upper 12 inches of pad subgrade should be moisture conditioned to a moisture content of at least 4 percentage points above optimum. The subgrade should be thoroughly soaked prior to placing the concrete. The subgrade should not be allowed to dry prior to concrete placement.

6.0 PRELIMINARY PAVEMENT DESIGN

6.1 FLEXIBLE PAVEMENTS

Based on the site soil, a Resistance (R-Value) of 5 is appropriate for design. The design sections may be reduced based on R-Value testing of samples collected from actual pavement subgrade. Using the traffic indices provided by the civil engineer, we developed the following recommended pavement sections using Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in Table 6.1-1 below.

TABLE 6.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION BASED ON R-VALUE 5	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3.0	10.0
6	3.5	13.0
7	4.0	16.0
9	5.5	20.5
11	7	25.0

Notes: AC is asphalt concrete
AB is Class 2 aggregate base material with a minimum R-value of 78

Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, Civil Engineer, and appropriate public agency.

6.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 4 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

6.3 SUBGRADE AND AGGREGATE BASE COMPACTION

The contractor should compact finish subgrade and aggregate base in accordance with the design-level geotechnical report. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

6.4 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the proposed residential development at 5150 El Camino Real in Los Altos, California. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, notify the proper regulatory officials immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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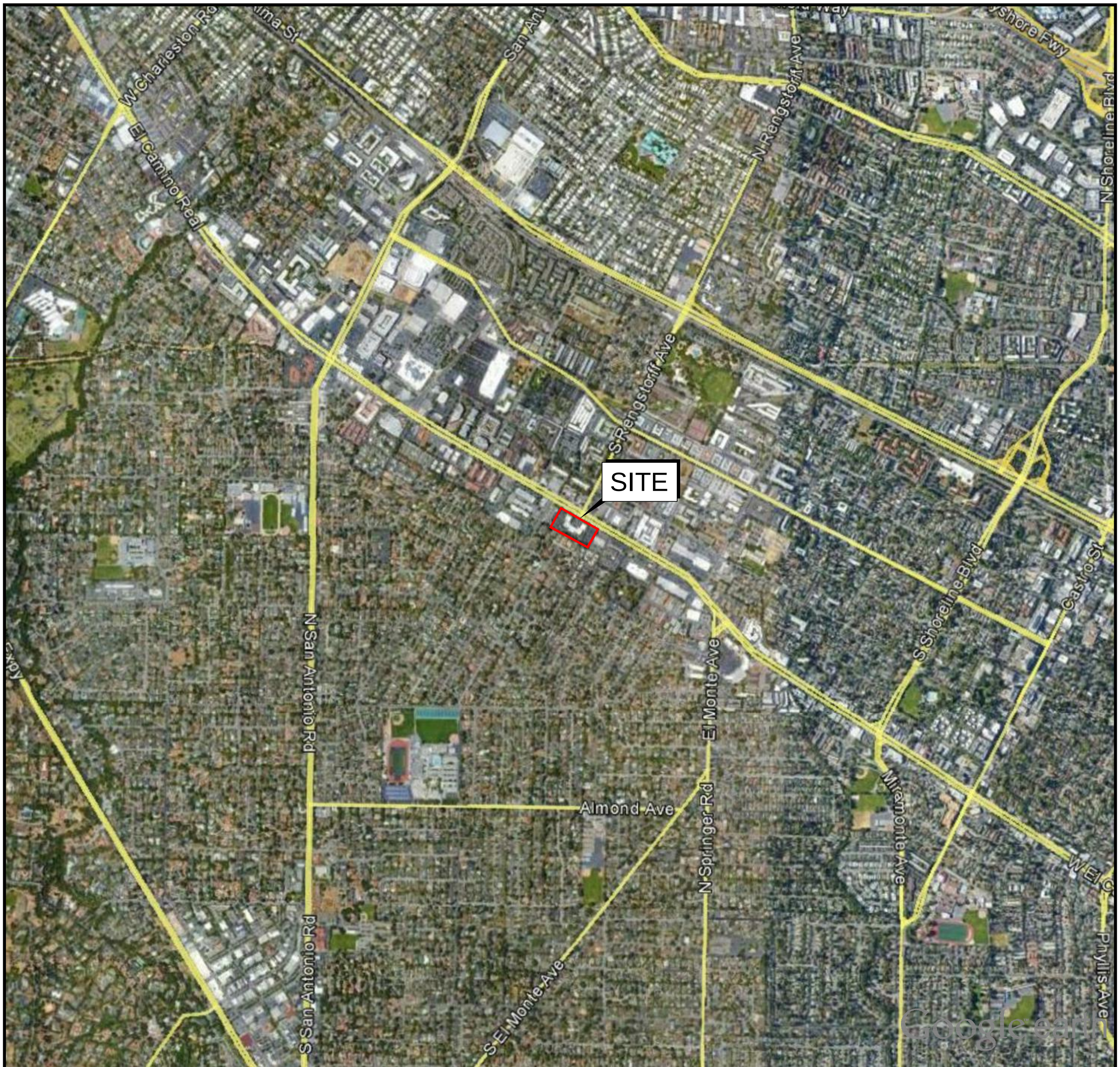
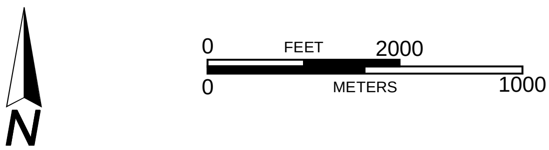
FIGURES





BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP 5150 WEST EL CAMINO REAL LOS ALTOS, CALIFORNIA	PROJECT NO.: 14723.000.000		FIGURE NO. 1
		SCALE: AS SHOWN		
		DRAWN BY: GLJ	CHECKED BY: TTB	

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14723\000\000-PGEX\14723000000-PGEX-1-VICMAP-0218.dwg Plot Date: 2-20-18 gjaffe

ORIGINAL FIGURE PRINTED IN COLOR

N

0 FEET 100
0 METERS 50

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT5 CONE PENETRATION TEST (ENGEO, 2018)

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	SITE PLAN 5150 WEST EL CAMINO REAL LOS ALTOS, CALIFORNIA	PROJECT NO.: 14723.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: TTB	FIGURE NO. 2

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14723\000\000-PGEX\14723000000-PGEX-2-SITEPLAN-0218.dwg Plot Date: 2-20-18 gjaffe

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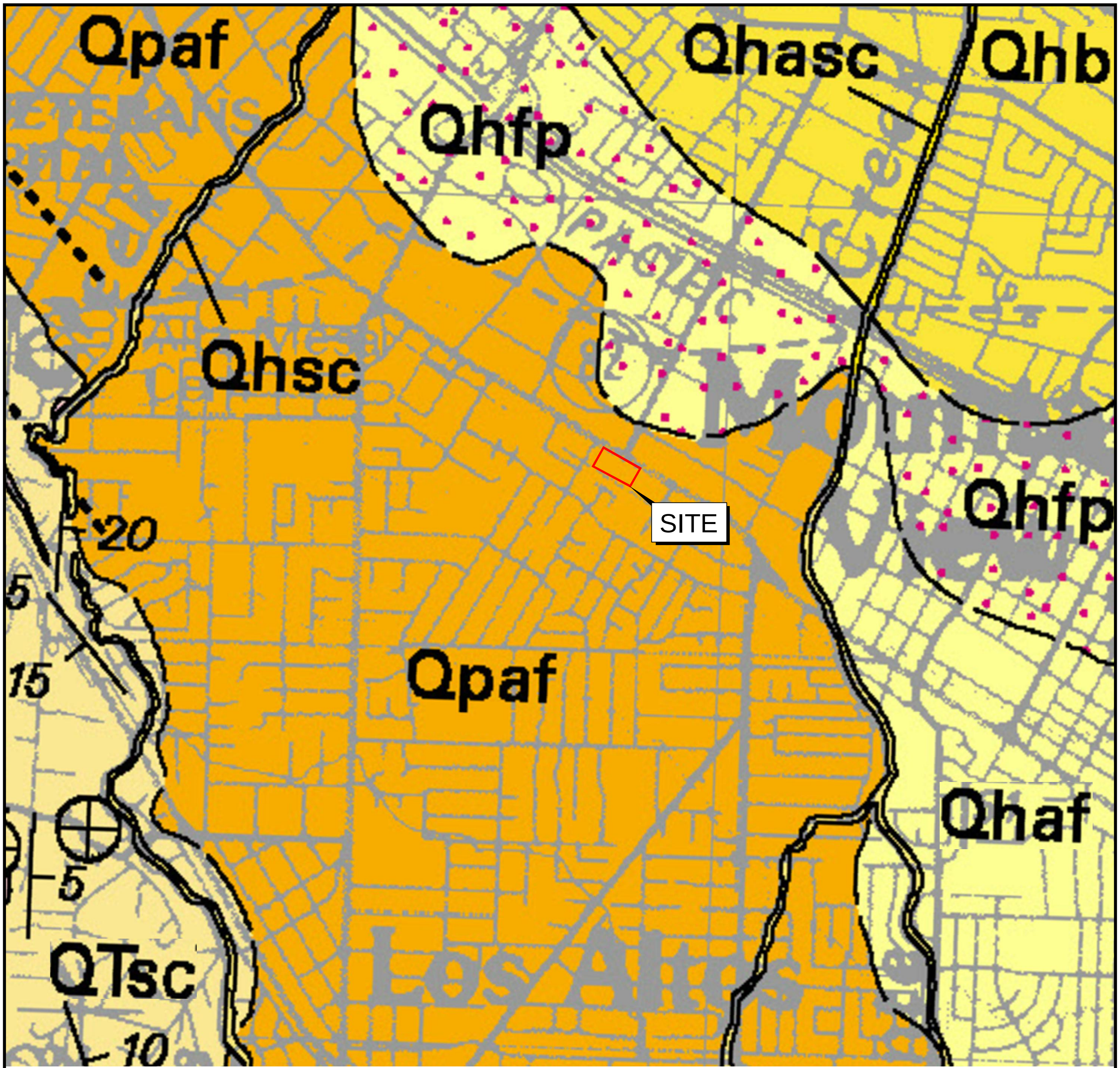

EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

STRIKE AND DIP OF STRATA

INCLINED   HORIZONTAL

N

0 FEET 2000

0 METERS 1000

Qhasc	ARTIFICIAL STREAM CHANNELS
Qhsc	STREAM CHANNEL DEPOSITS
Qhb	BASIN DEPOSITS
Qhfp	FLOOD-PLAIN DEPOSITS
Qhaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (HOLOCENE)
Qpaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (PLEISTOCENE)
QTsc	SANTA CLARA FORMATION

BASE MAP SOURCE: GRAYMER, 2000

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
5150 WEST EL CAMINO REAL
LOS ALTOS, CALIFORNIA

PROJECT NO.: 14723.000.000	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: GLJ	CHECKED BY: TTB

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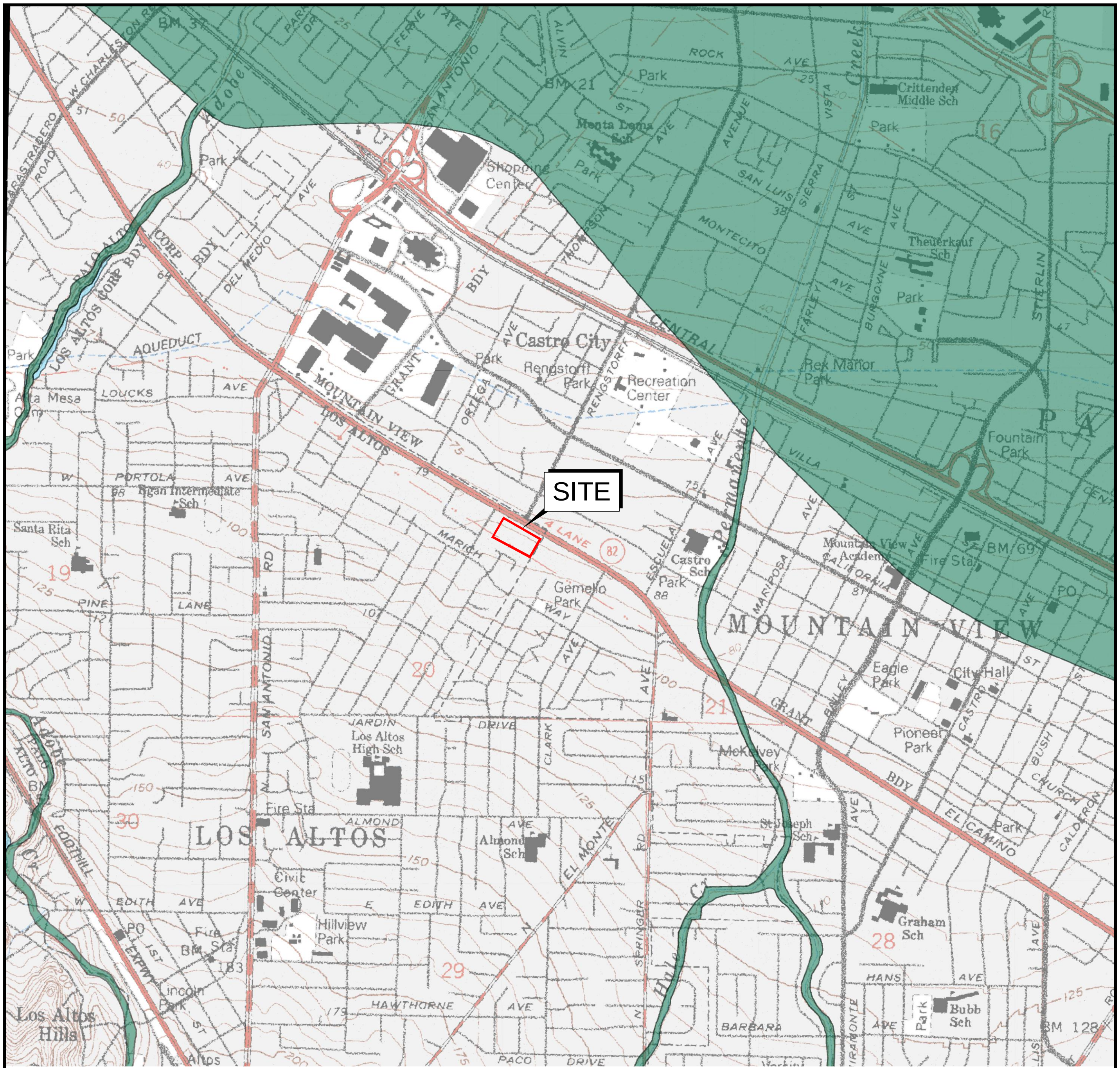
ORIGINAL FIGURE PRINTED IN COLOR



N

0 FEET 2000
0 METERS 1000

EXPLANATION

LIQUEFACTION

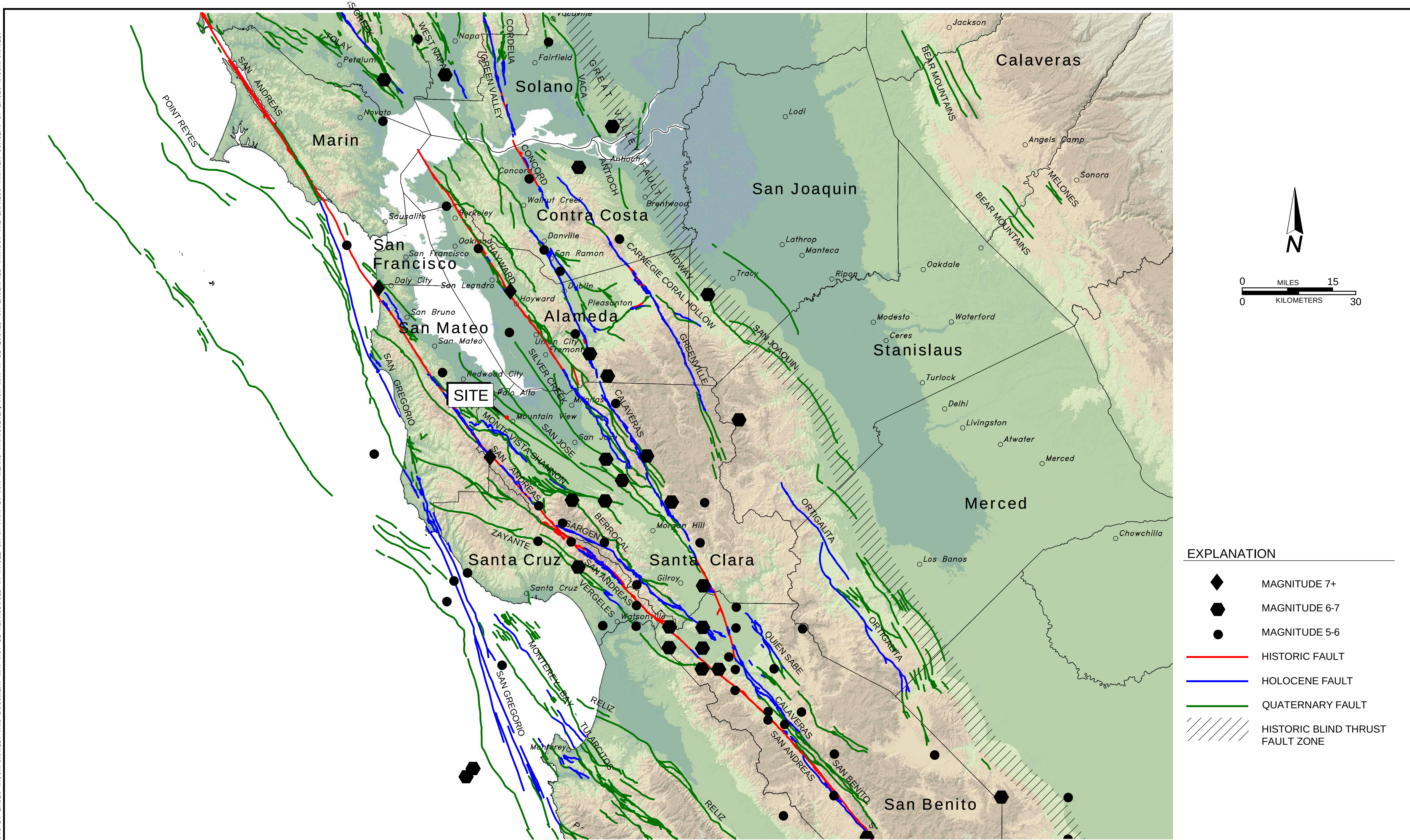
AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2006

ENGEO Expect Excellence	SEISMIC HAZARD ZONE MAP 5150 WEST EL CAMINO REAL LOS ALTOS, CALIFORNIA	PROJECT NO.: 14723.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: TTB	FIGURE NO. 4

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14723\000\000-PGEX\14723000000-PGEX-4-SHZM-0218.dwg Plot Date: 2-20-18 gjaffe

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY
5150 WEST EL CAMINO REAL
LOS ALTOS, CALIFORNIA

PROJECT NO.: 14723.000.000	FIGURE NO.
SCALE: AS SHOWN	5
DRAWN BY: GLJ	CHECKED BY: TTB

ORIGINAL FIGURE PRINTED IN COLOR

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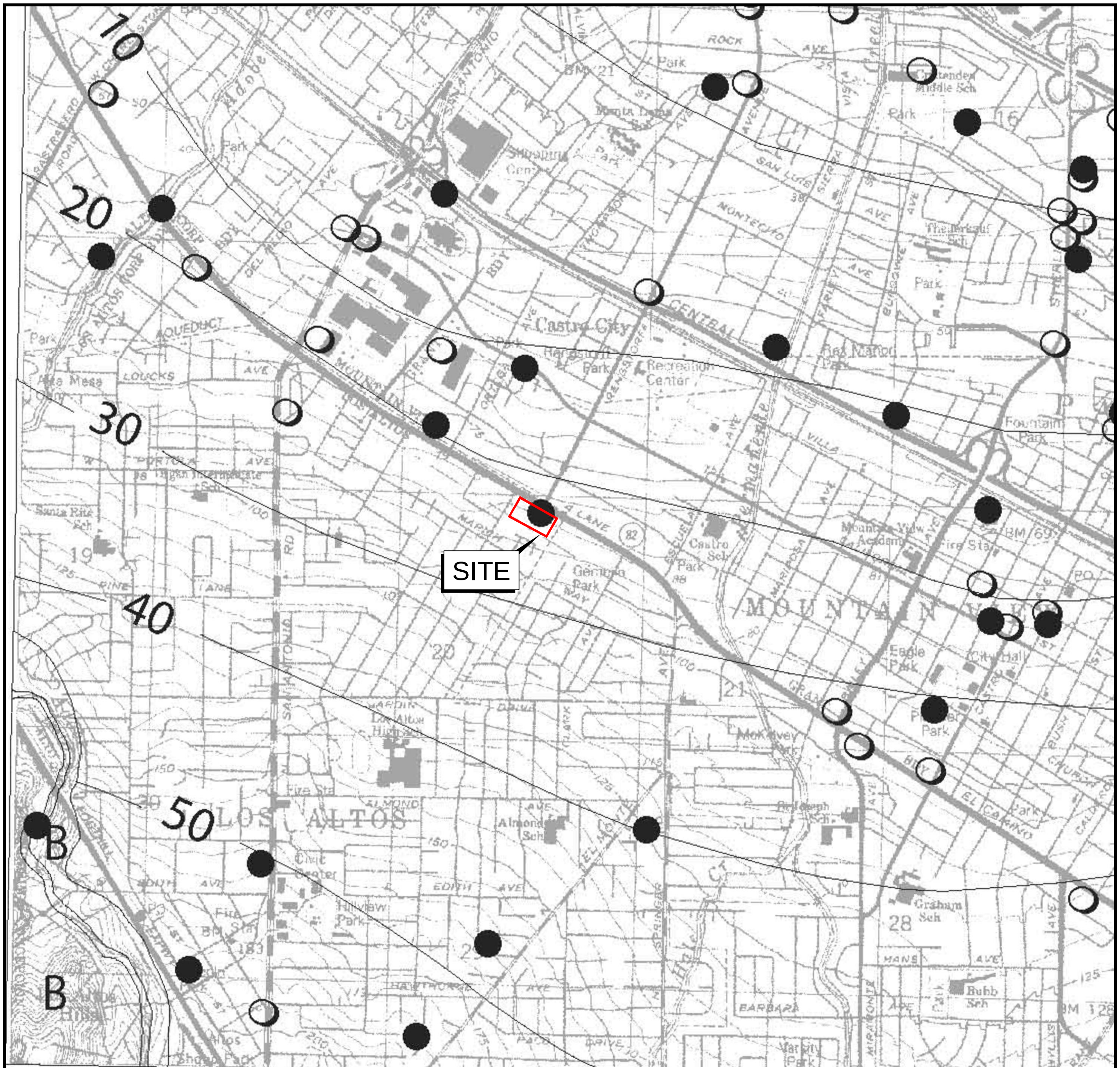


N

0 FEET 2000

0 METERS 1000

EXPLANATION

50 — Depth to ground water, in feet

● Geotechnical borings used in liquefaction evaluation

○ Ground-water level data provided by the Santa Clara Valley Water District

B Bedrock (Santa Clara Formation)

BASE MAP SOURCE: KNUDSEN, 2000

ENGEO
Expect Excellence

DEPTH TO GROUNDWATER
5150 WEST EL CAMINO REAL
LOS ALTOS, CALIFORNIA

PROJECT NO.: 14723.000.000	
SCALE: AS SHOWN	
DRAWN BY: GLJ	CHECKED BY: TTB

FIGURE NO.
6

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14723\000\000-PGEX\14723000000-PGEX-5-GRNDWATER-0218.dwg Plot Date: 2-20-18 gjaffe

ORIGINAL FIGURE PRINTED IN COLOR

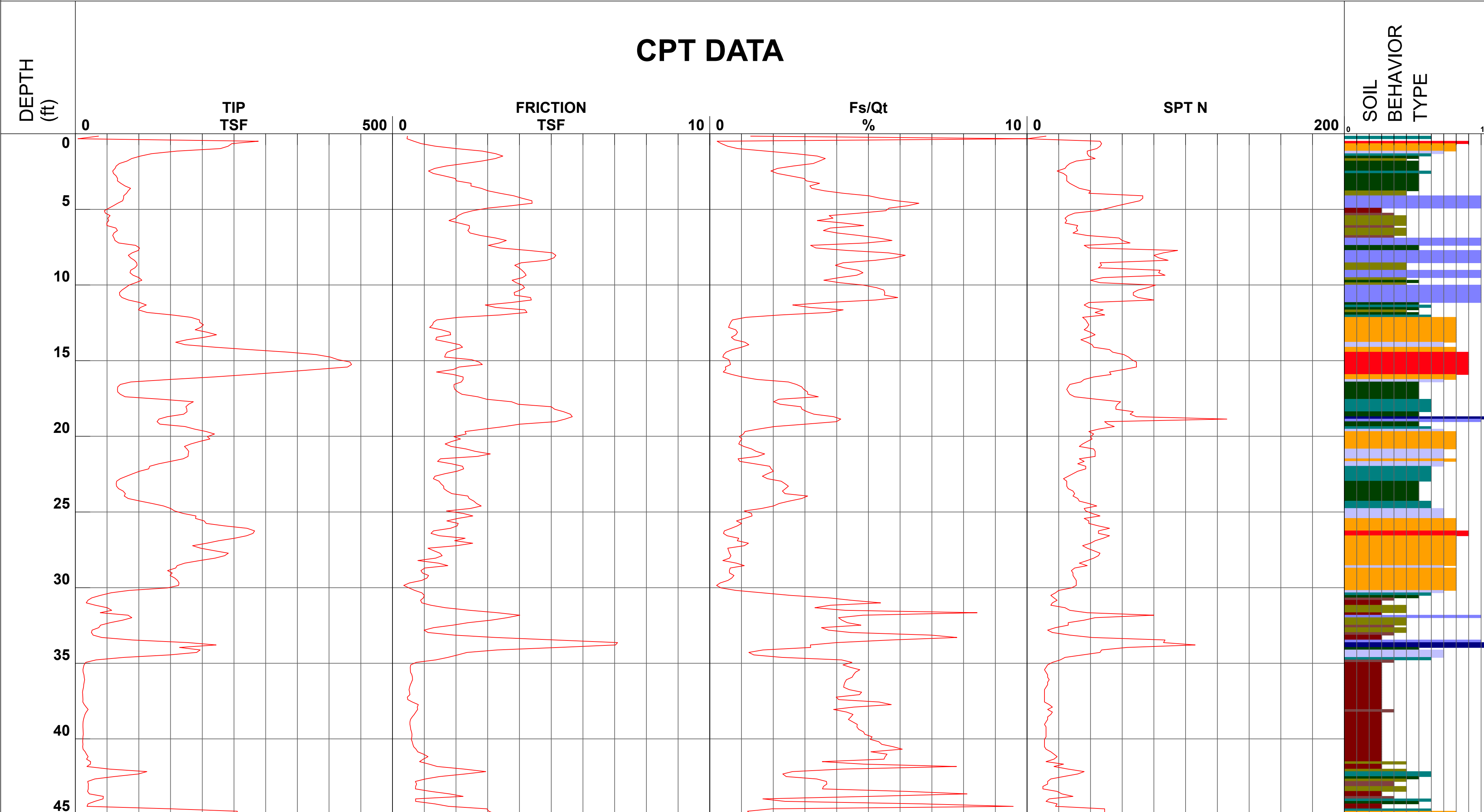
APPENDIX A

CONE PENETRATION TESTS

Engeo Inc

Project	5150 El Camino Real	Operator	BH-RB-JO	Filename	SDF(060).cpt
Job Number	14723.000.000	Cone Number	DDG1418	GPS	
Hole Number	1-CPT1	Date and Time	2/15/2018 12:16:14 PM	Maximum Depth	45.44 ft
EST GW Depth During Test		34.00 ft			

Net Area Ratio .8



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

Cone Size 10cm squared

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

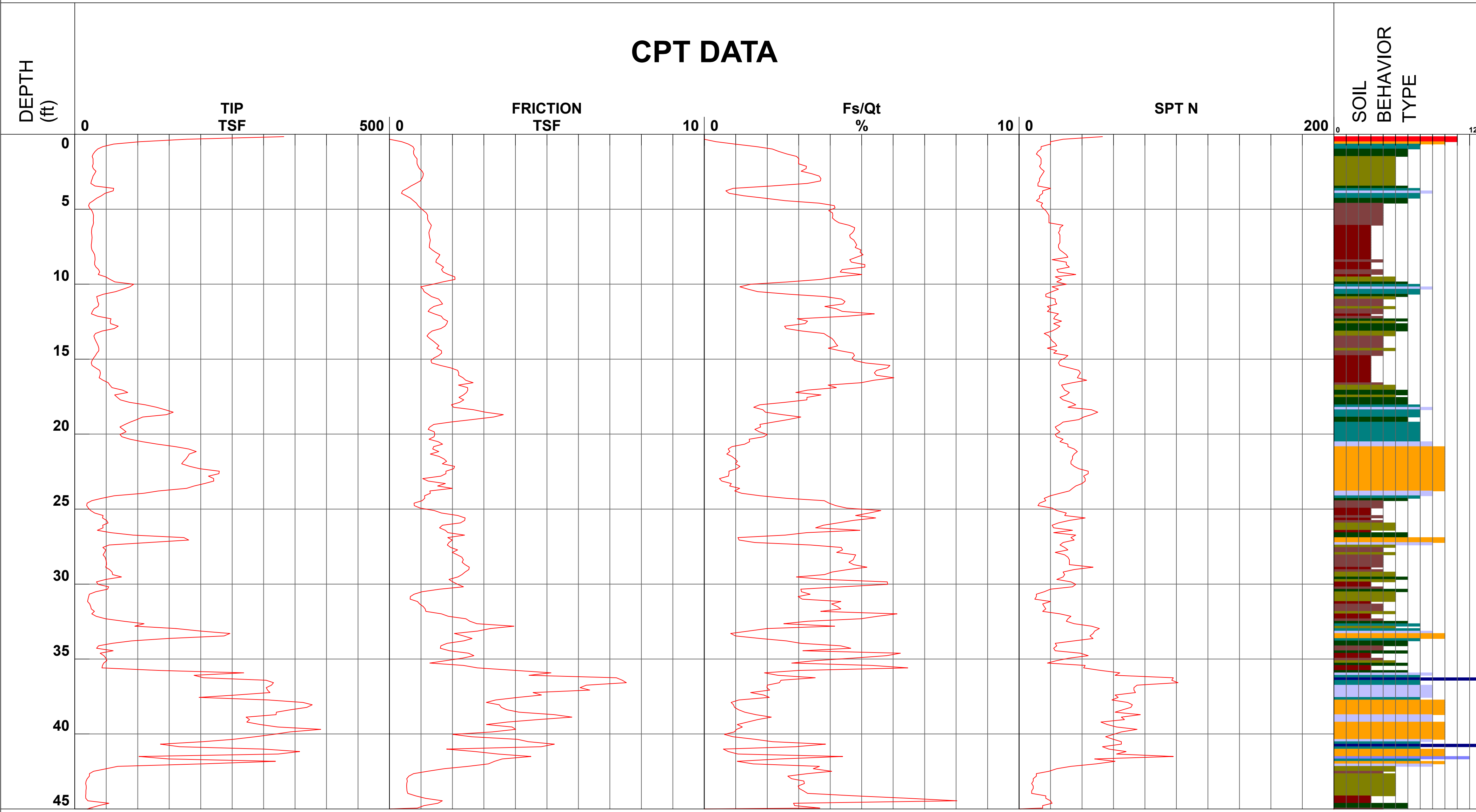
Engeo Inc

Middle Earth Geo Testing Inc.

Project	5150 El Camino Real	Operator	BH-RB-JO	Filename	SDF(063).cpt
Job Number	14723.000.000	Cone Number	DDG1418	GPS	
Hole Number	1-CPT2	Date and Time	2/15/2018 2:17:59 PM	Maximum Depth	45.44 ft
EST GW Depth During Test		32.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

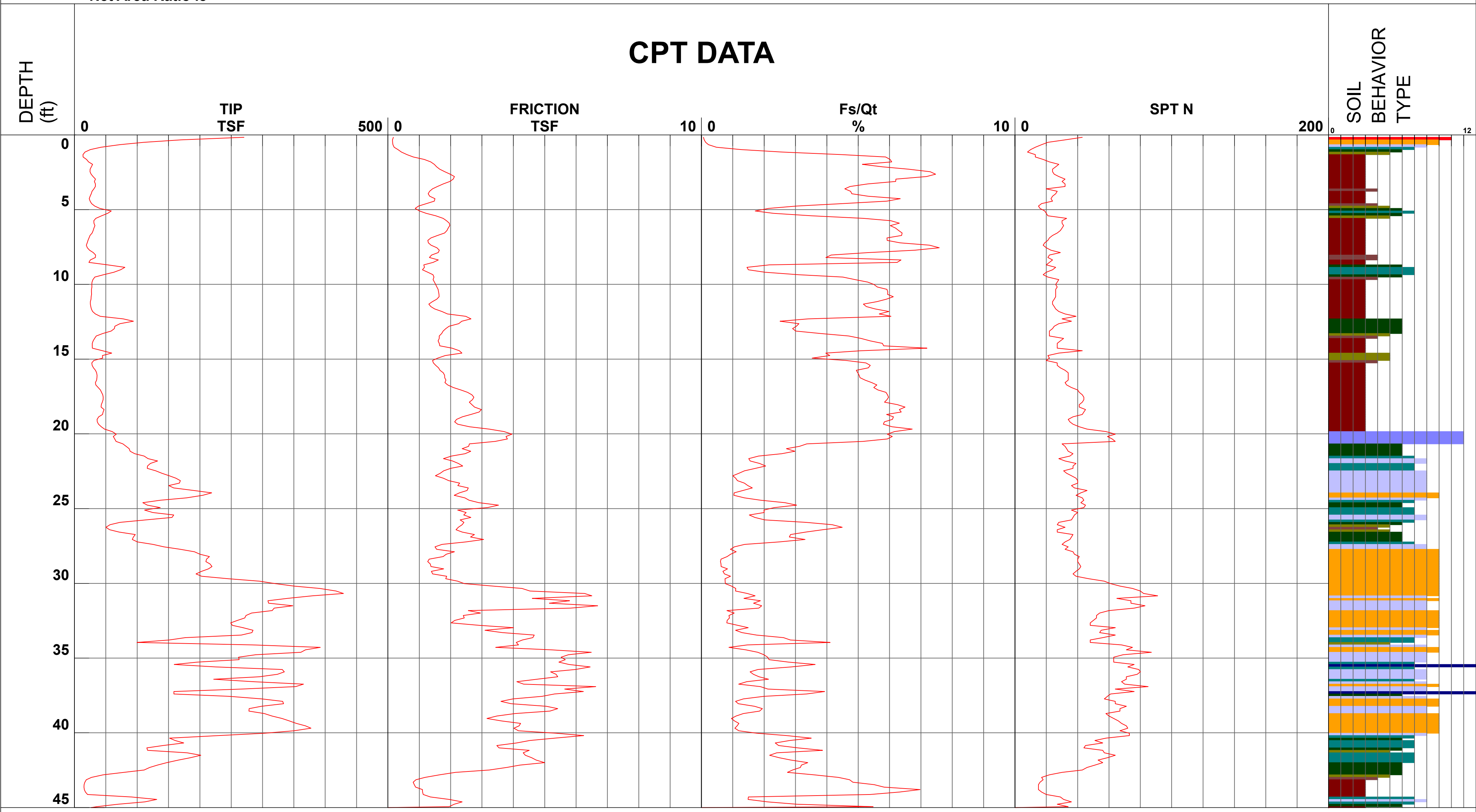
S*Soil behavior type and SPT based on data from UBC-1983

Engeo Inc

Project	5150 El Camino Real	Operator	BH-RB-JO	Filename	SDF(062).cpt
Job Number	14723.000.000	Cone Number	DDG1418	GPS	
Hole Number	1-CPT3	Date and Time	2/15/2018 1:37:02 PM	Maximum Depth	45.44 ft
EST GW Depth During Test		34.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

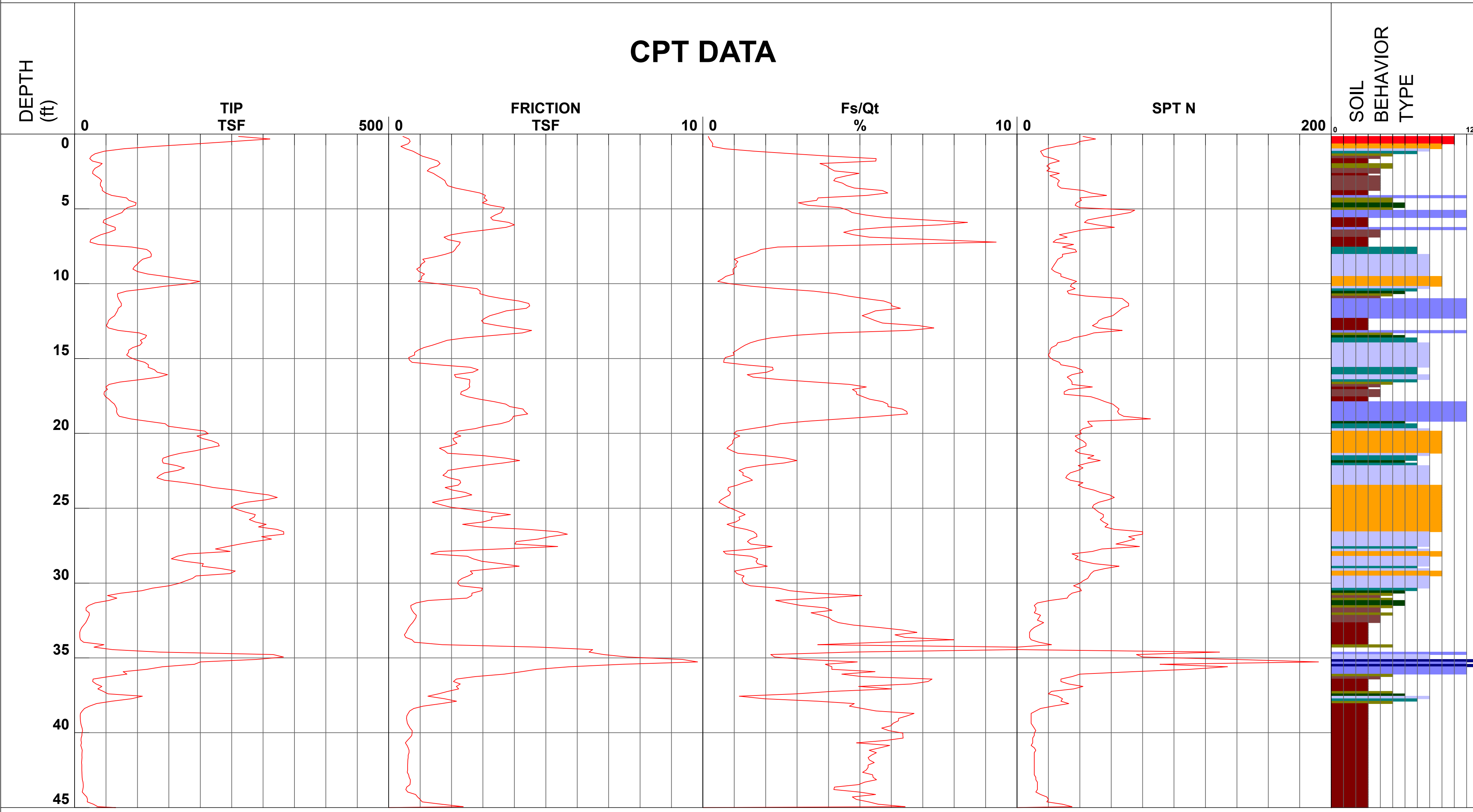
S*Soil behavior type and SPT based on data from UBC-1983

Engeo Inc

Project	5150 El Camino Real	Operator	BH-RB-JO	Filename	SDF(064).cpt
Job Number	14723.000.000	Cone Number	DDG1418	GPS	
Hole Number	1-CPT4	Date and Time	2/15/2018 3:04:40 PM	Maximum Depth	45.44 ft
EST GW Depth During Test		31.00 ft			

Net Area Ratio .8

CPT DATA



- 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 (*)

Cone Size 10cm squared

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

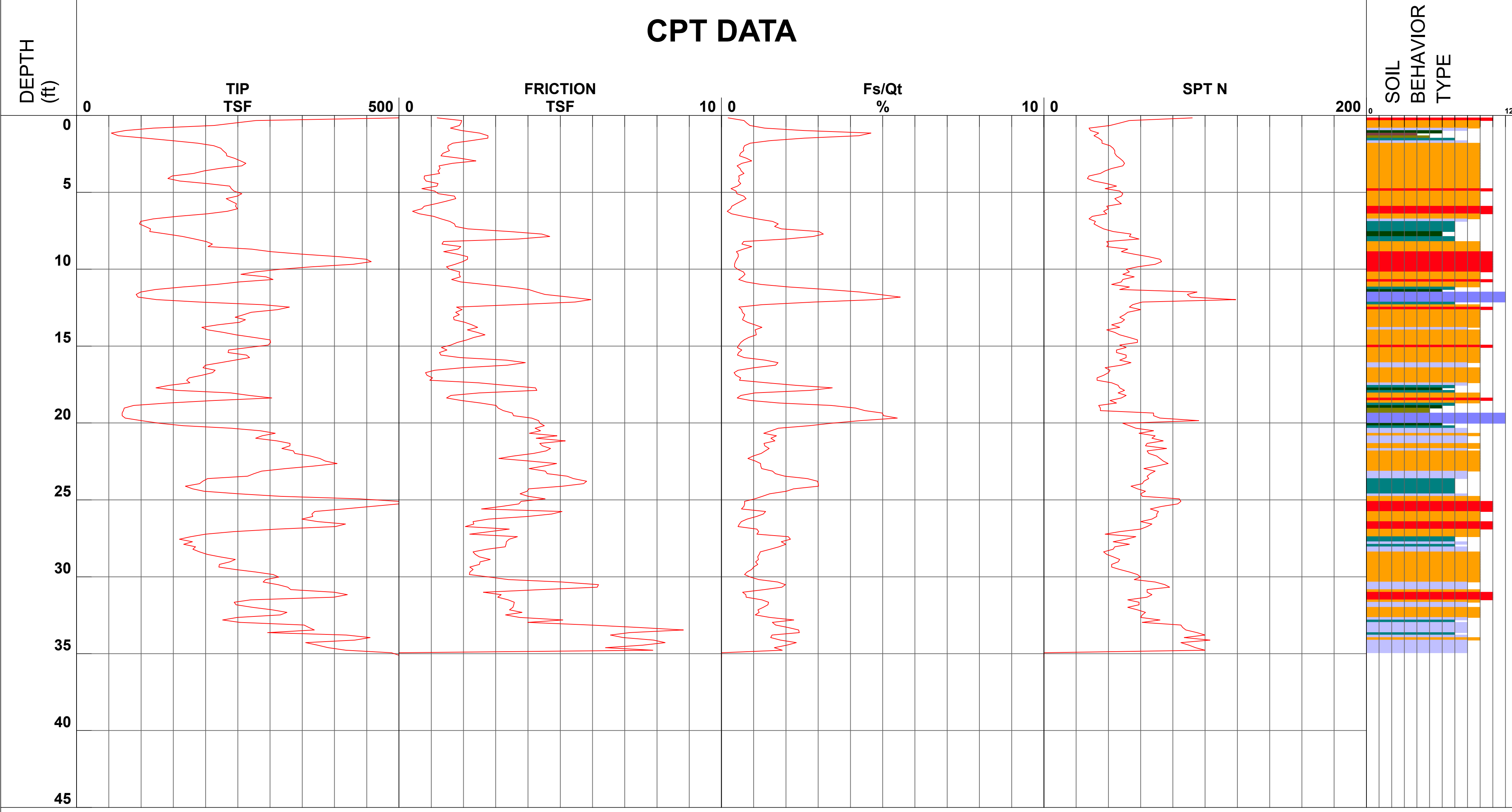
Engeo Inc

Middle Earth Geo Testing Inc.

Project	5150 El Camino Real	Operator	BH-RB-JO	Filename	SDF(061).cpt
Job Number	14723.000.000	Cone Number	DDG1418	GPS	
Hole Number	1-CPT5	Date and Time	2/15/2018 1:01:18 PM	Maximum Depth	35.27 ft
EST GW Depth During Test		34.00 ft			

Net Area Ratio .8

CPT DATA



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

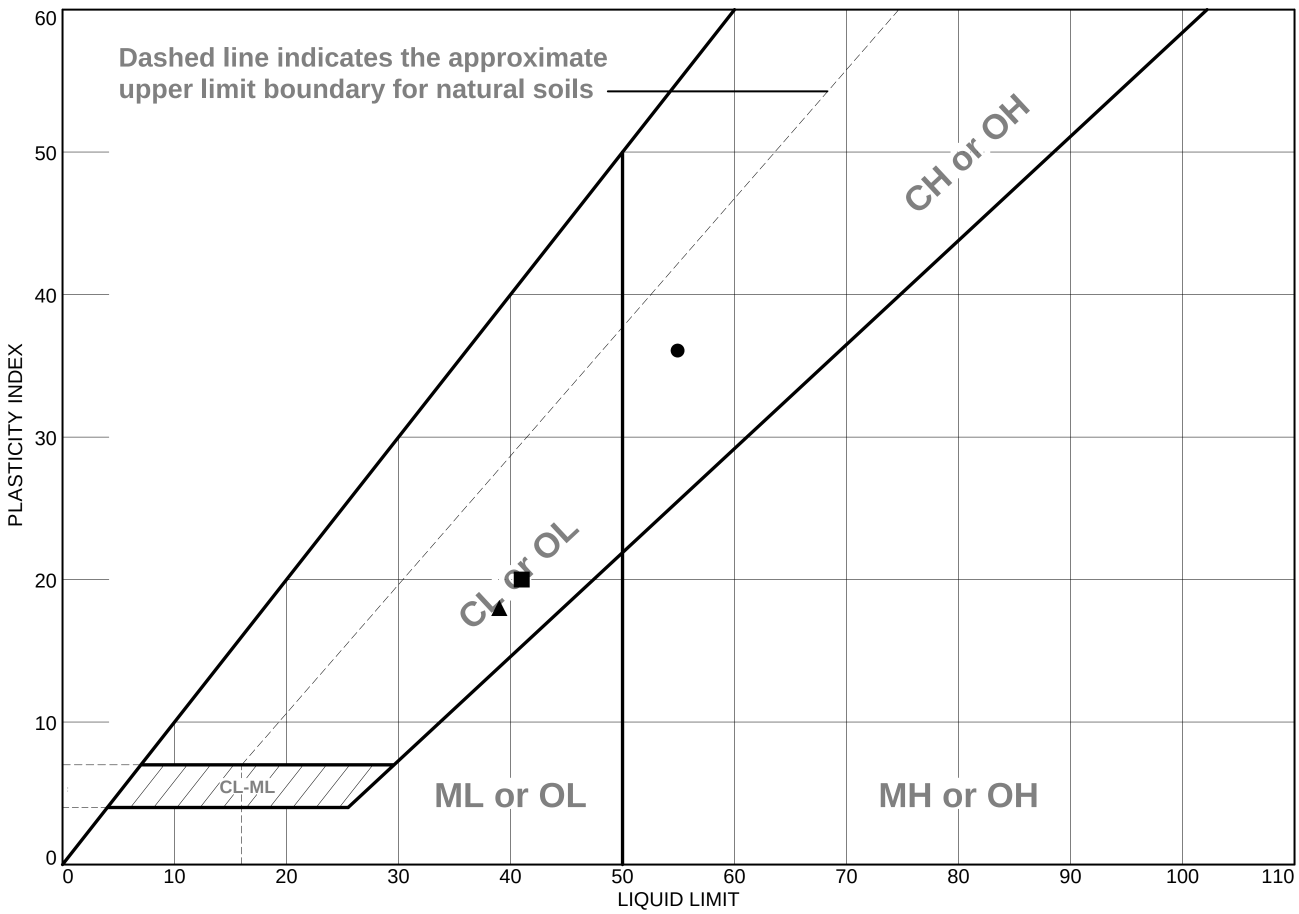
Cone Size 10cm squared

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

APPENDIX B

LABORATORY TEST DATA

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	55	19	36			
■	See exploration logs	41	21	20			
▲	See exploration logs	39	21	18			

Project No. 14723.000.000 **Client:** Dutchints Development

Project: 5150 El Camino Real

●**Sample Number:** 1-CPT3 @ 2
■**Sample Number:** 1-CPT4 @ 2
▲**Sample Number:** 1-CPT5 @ 2

Remarks:
●ASTM D4138, Wet method
■ASTM D4138, Wet method
▲ASTM D4138, Wet method

ENGEO INCORPORATED

Tested By: M. Bromfield **Checked By:** G. Criste

MOISTURE CONTENT DETERMINATION

ASTM D2216

BORING/SAMPLE ID	1-CPT3	1-CPT4	1-CPT5					
DEPTH (ft)	**2.0**	**2.0**	**2.0**					
Method A or B	B	B	B					
%MOISTURE	**18.2**	**13.1**	**6.4**					

PROJECT NAME: **5150 El Camino Real**
PROJECT NUMBER: **14723.000.000**
CLIENT: **Dutchints Development**
PHASE NUMBER: **001**

DATE: **02/21/18**

ENGEO
Expect Excellence

Tested by: M. Quasem   Reviewed by: M. Bromfield

APPENDIX C

LIQUEFACTION ANALYSIS

ENGEO Inc.
2010 Crow Canyon Place #250
San Ramon, CA 94583

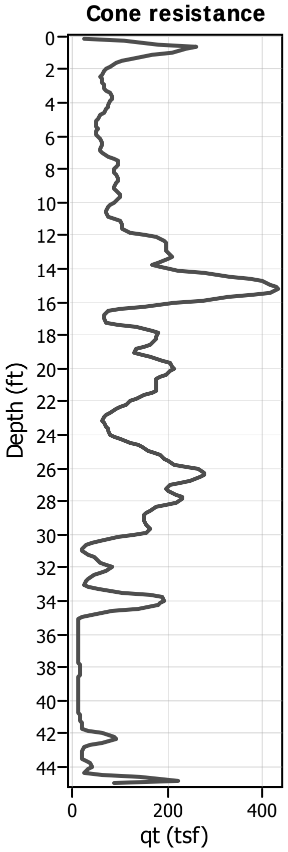
LIQUEFACTION ANALYSIS REPORT

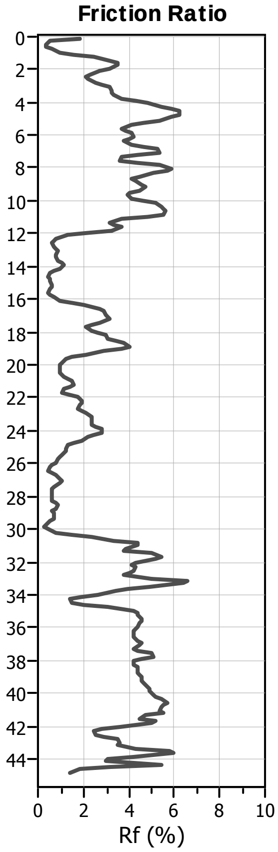
Project title : 14723.000.000 - 5150 El Camino Real **Location : Los Altos, CA**

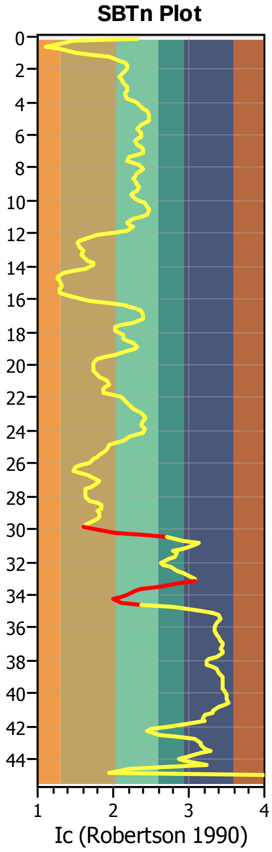
CPT file : 1-CPT1

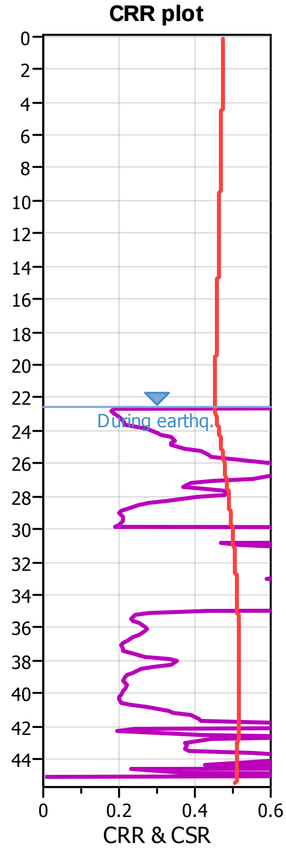
Input parameters and analysis data

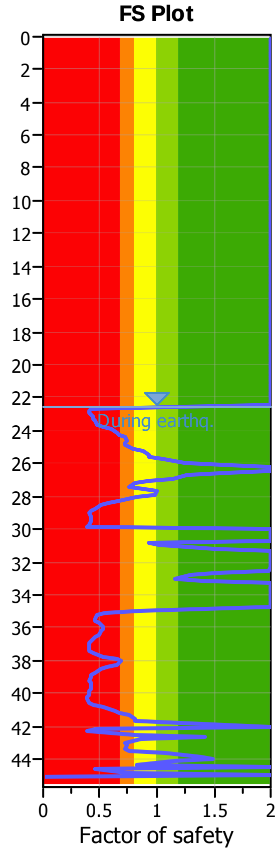
Analysis method:	Robertson (2009)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	22.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	No		

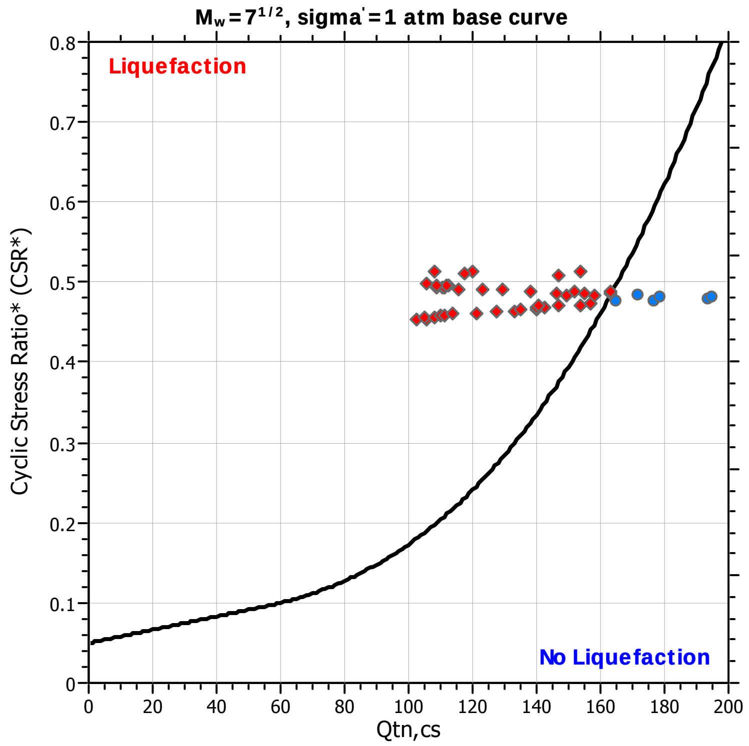


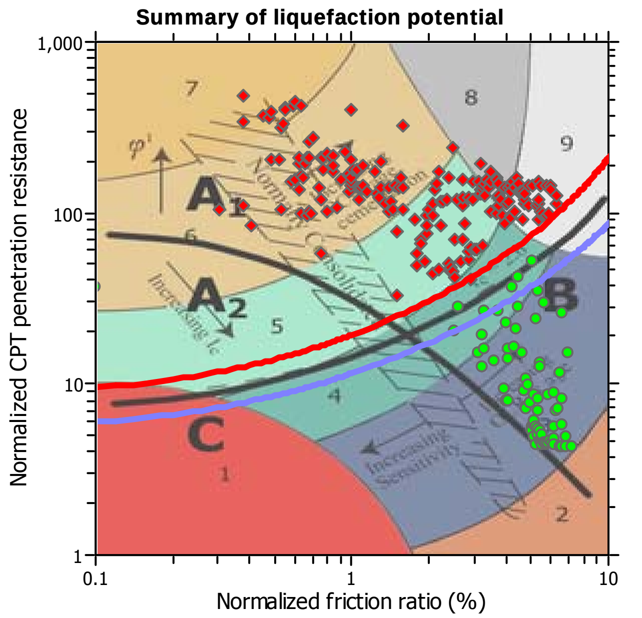






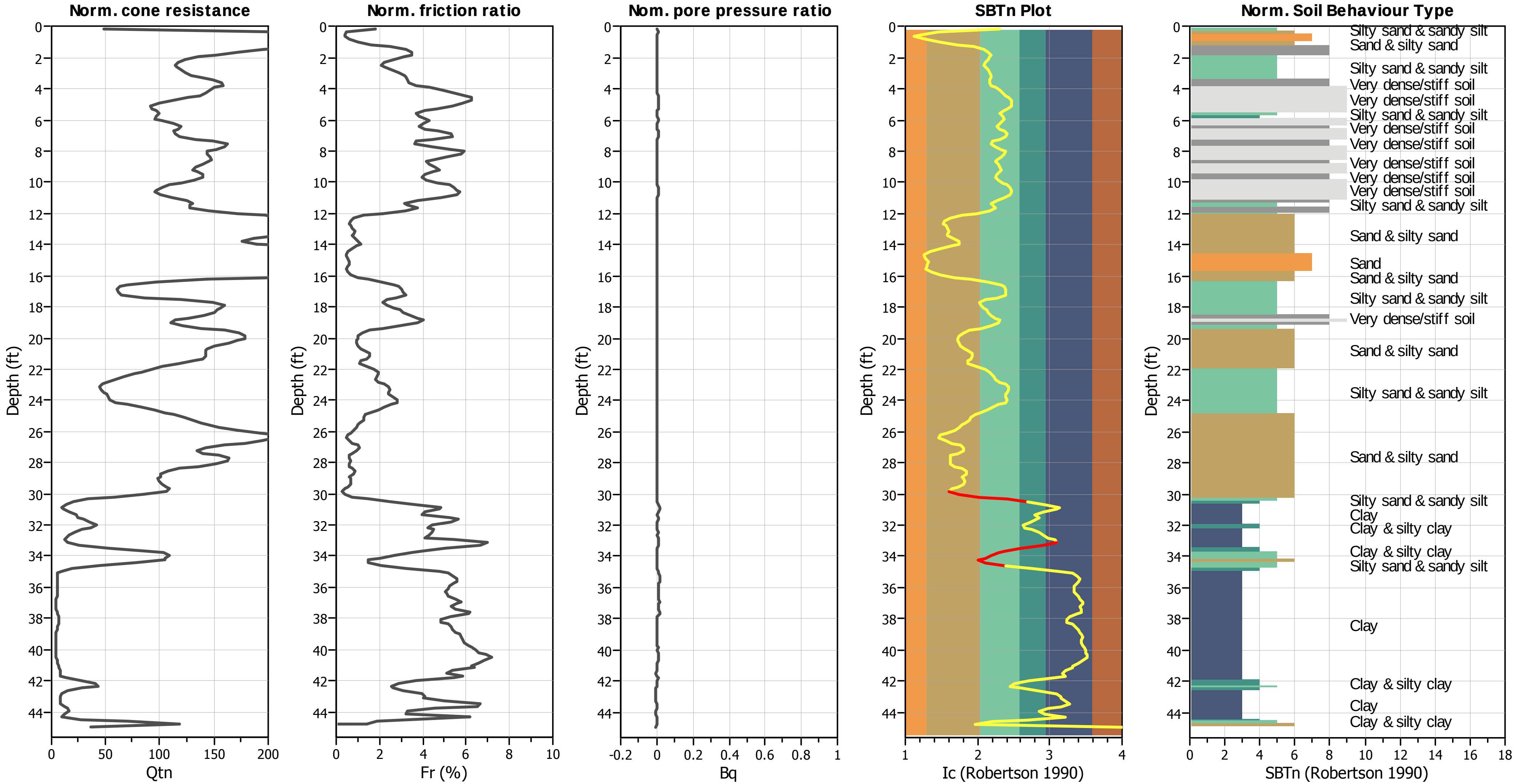






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



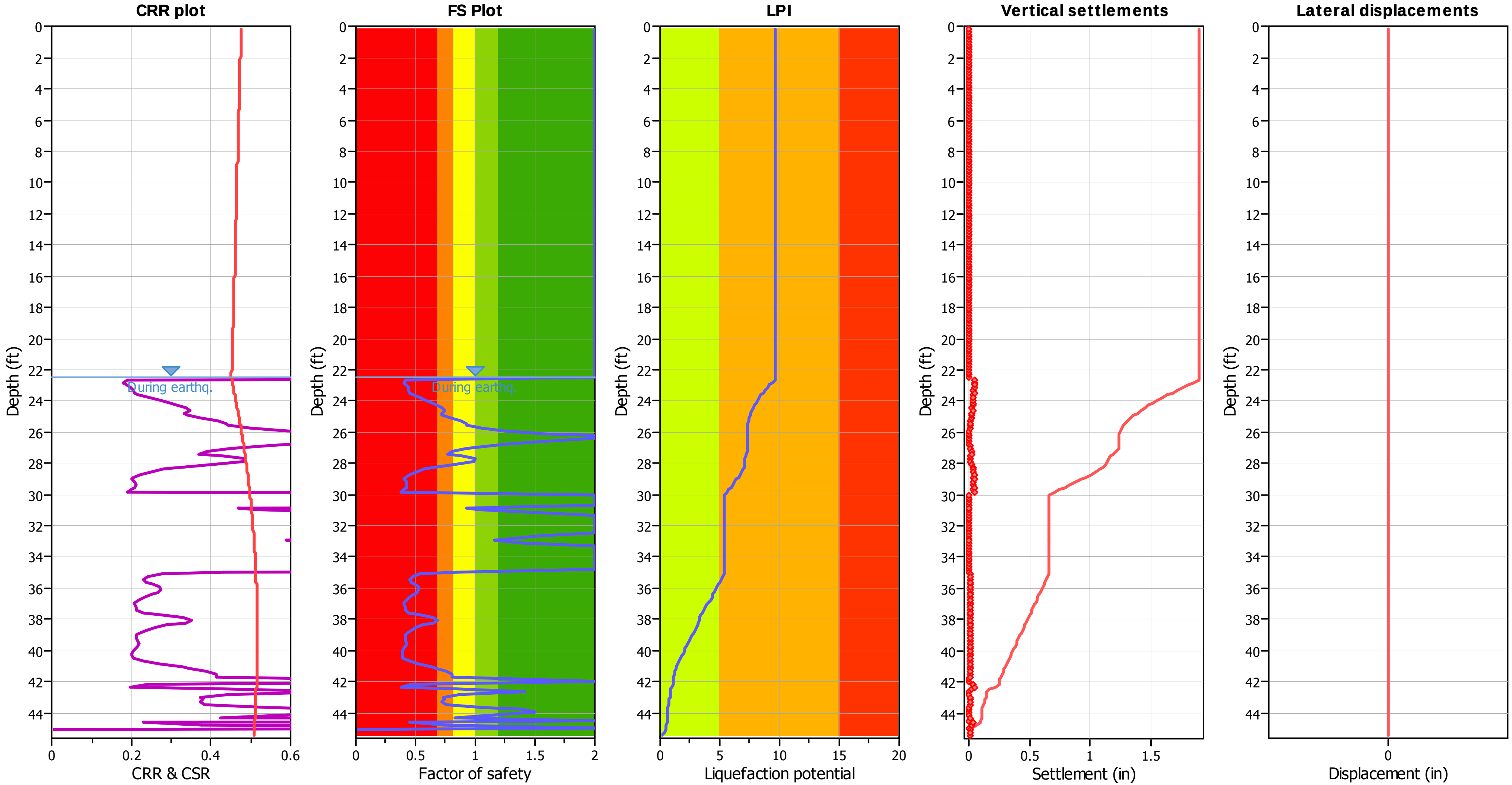
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Inc.
2010 Crow Canyon Place #250
San Ramon, CA 94583

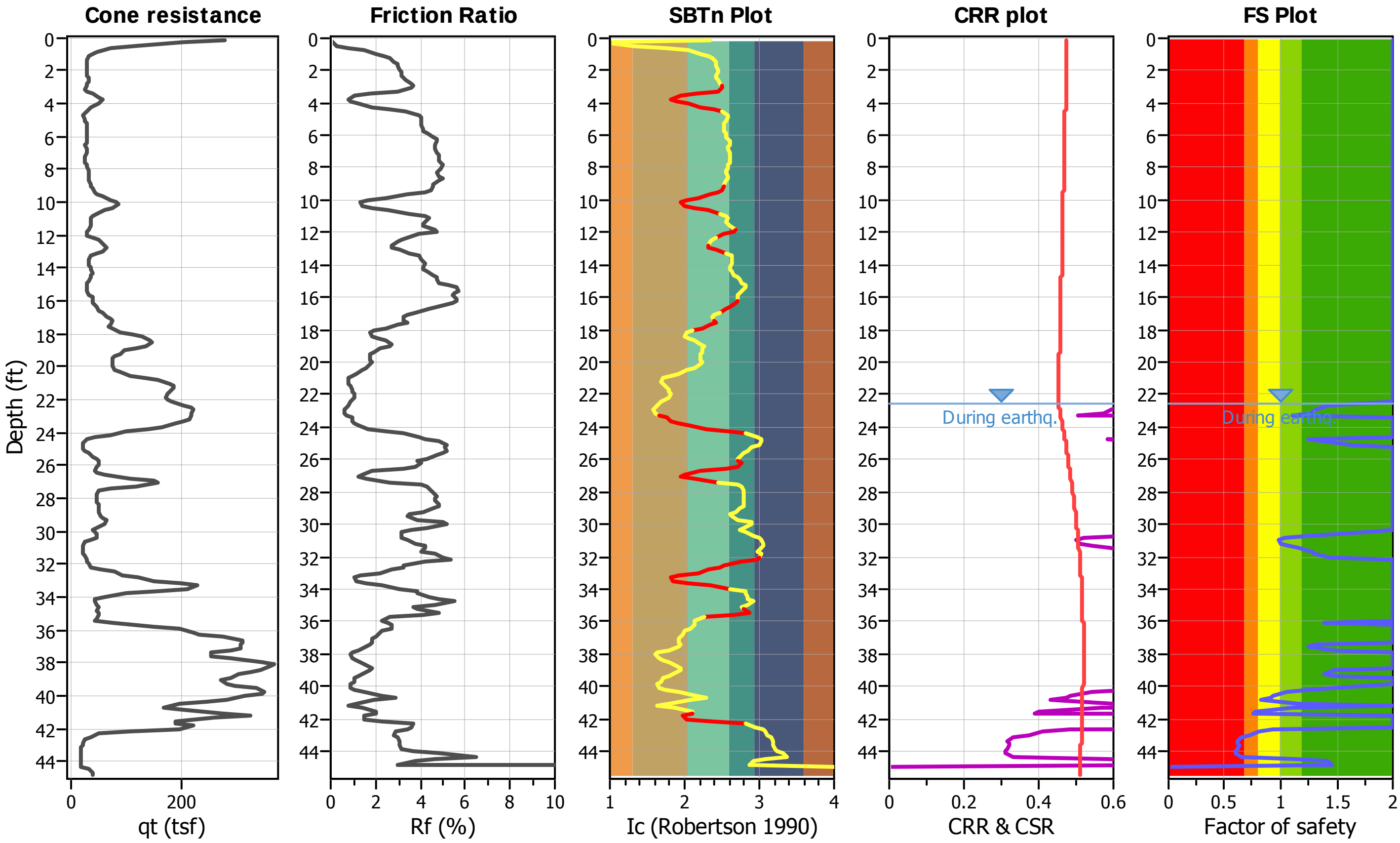
LIQUEFACTION ANALYSIS REPORT

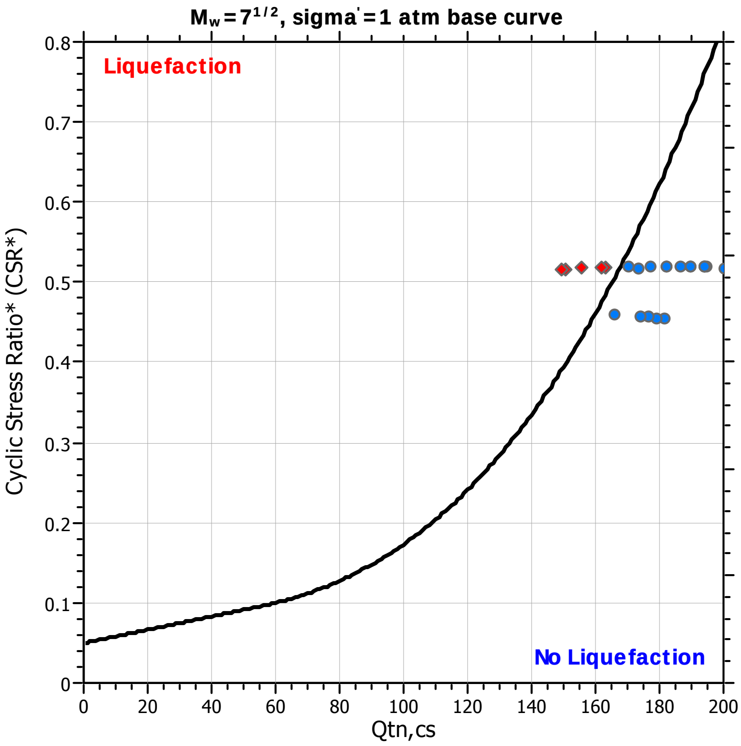
Project title : 14723.000.000 - 5150 El Camino Real **Location : Los Altos, CA**

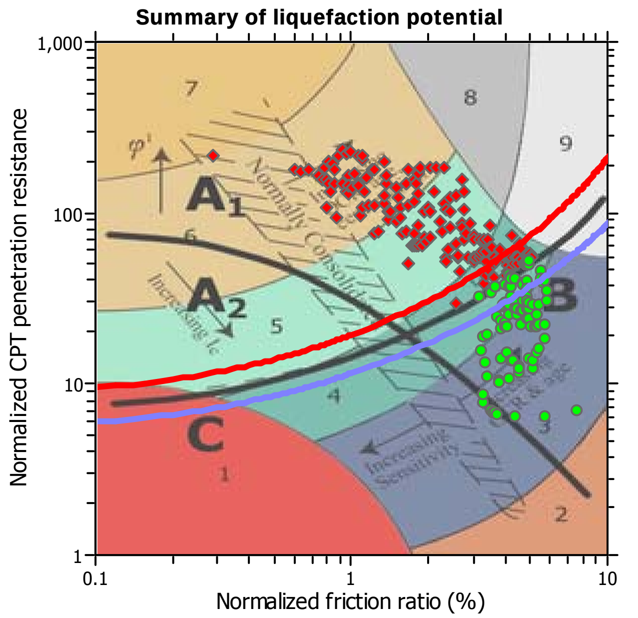
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	22.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	No		

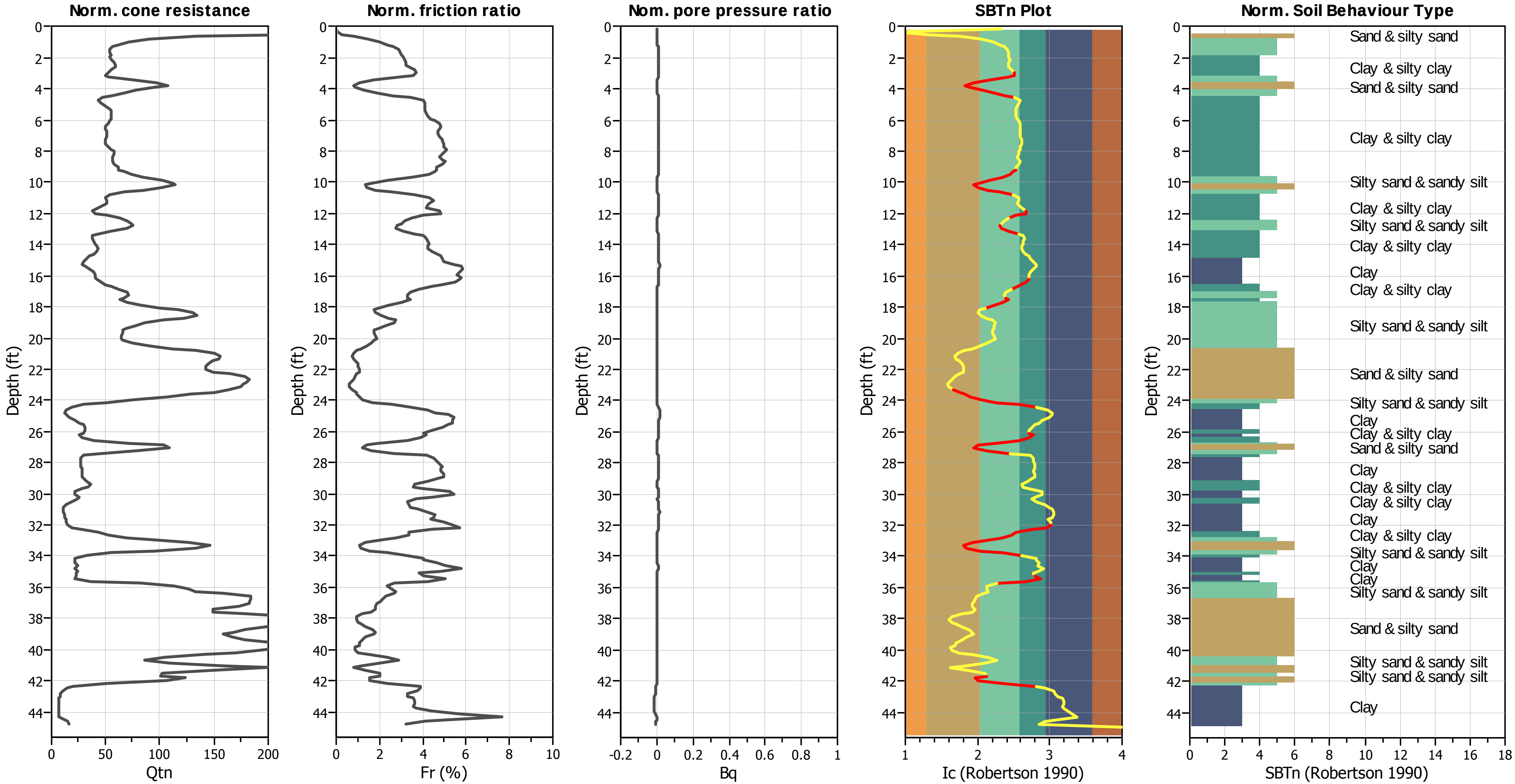






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



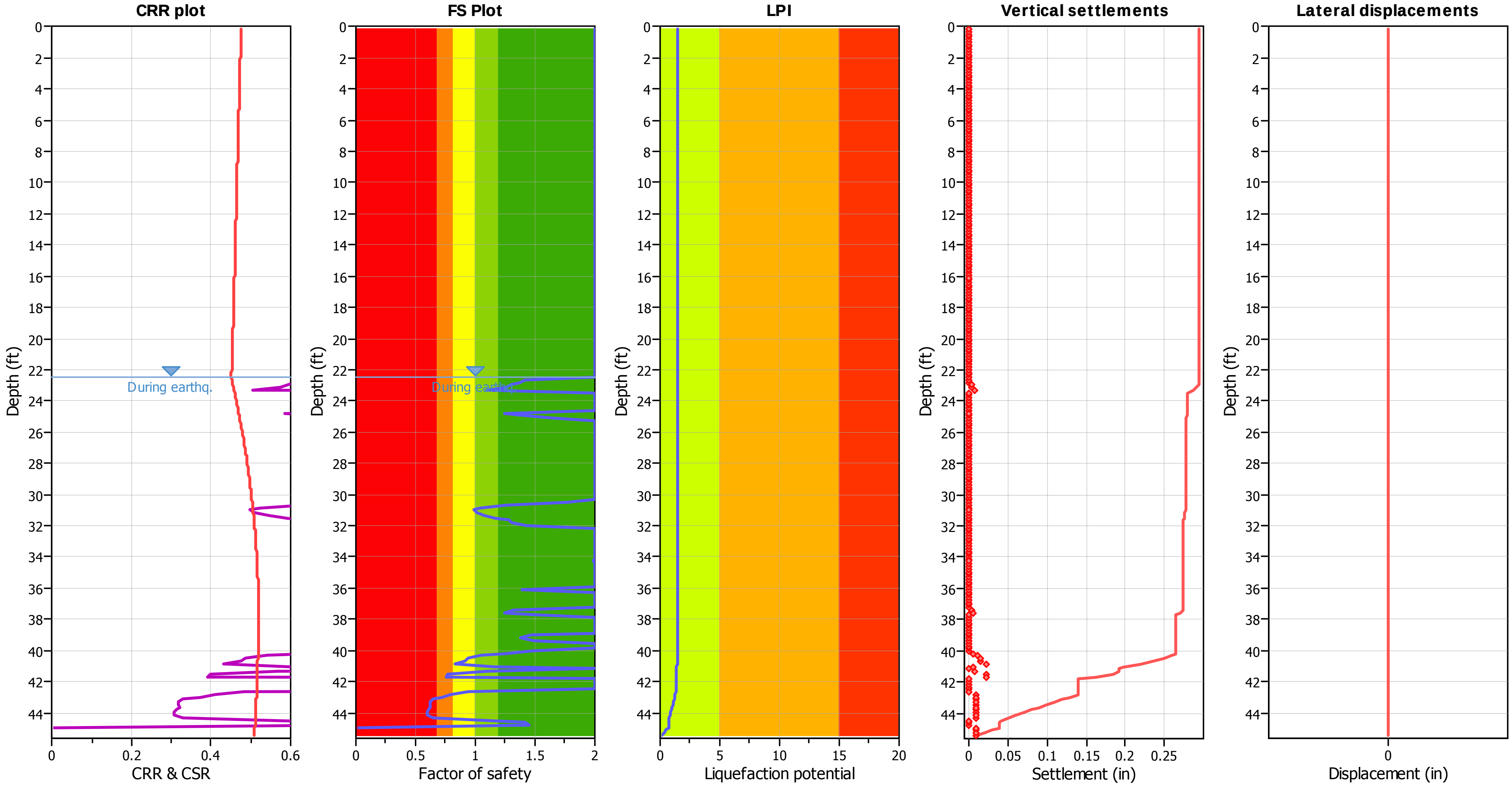
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Inc.
2010 Crow Canyon Place #250
San Ramon, CA 94583

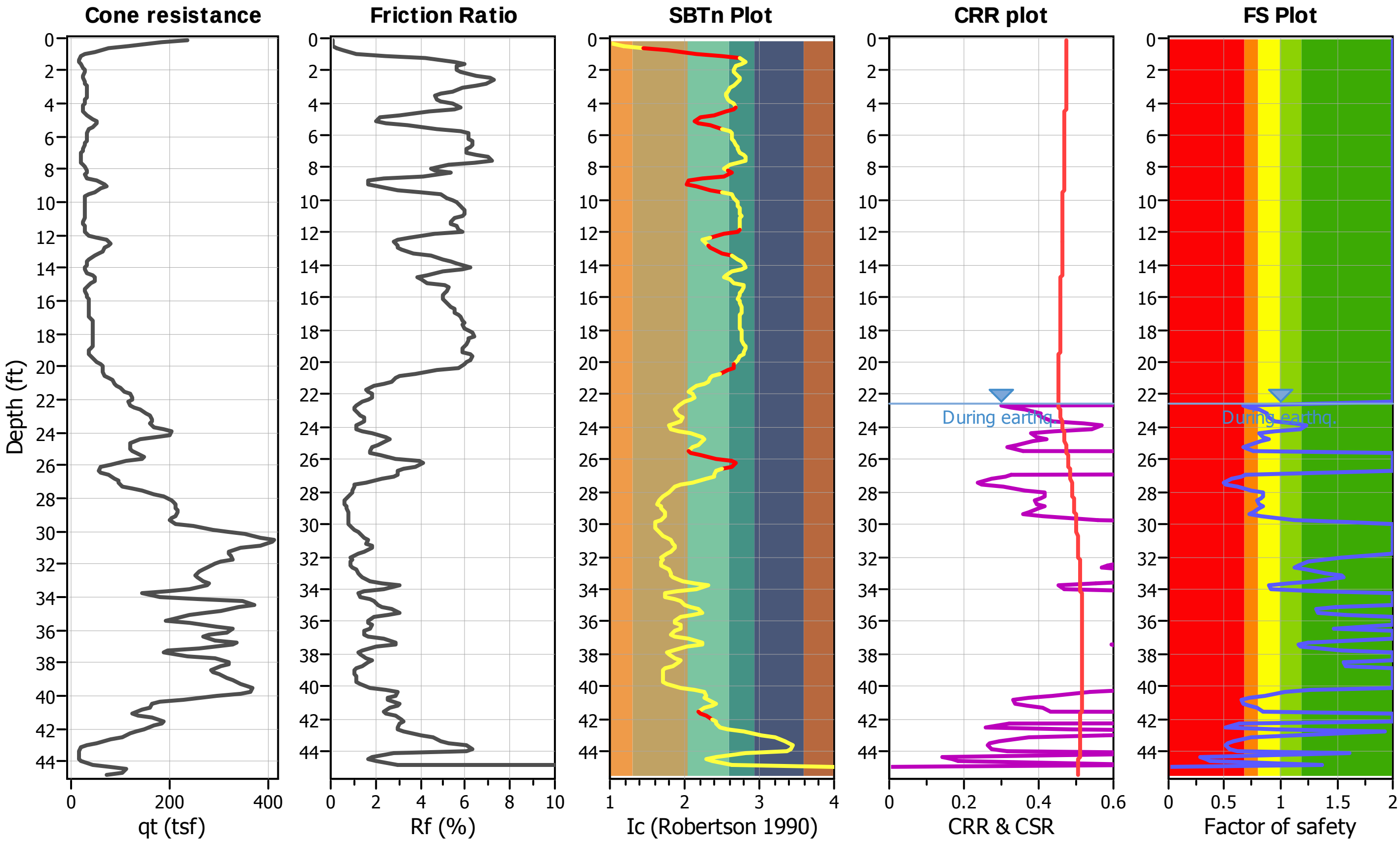
LIQUEFACTION ANALYSIS REPORT

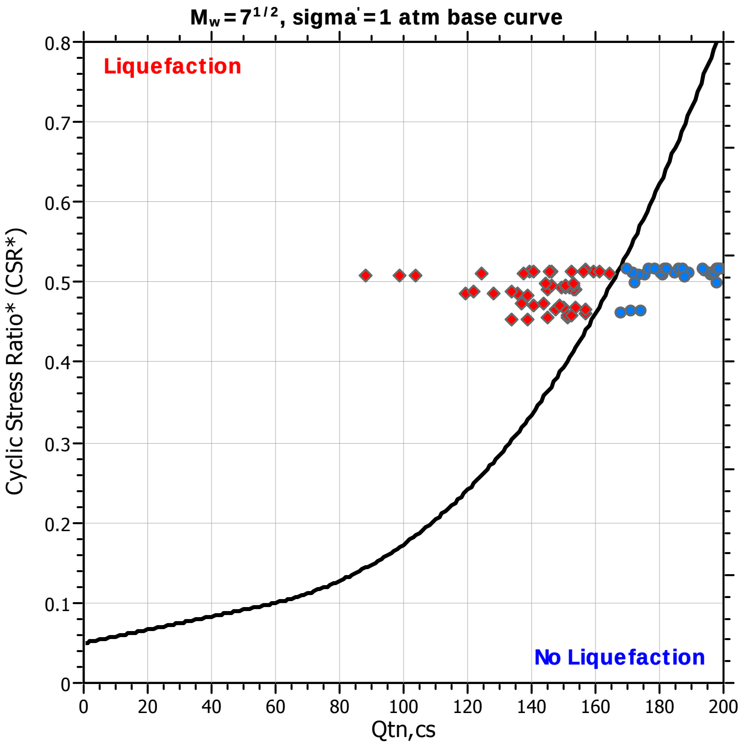
Project title : 14723.000.000 - 5150 El Camino Real **Location : Los Altos, CA**

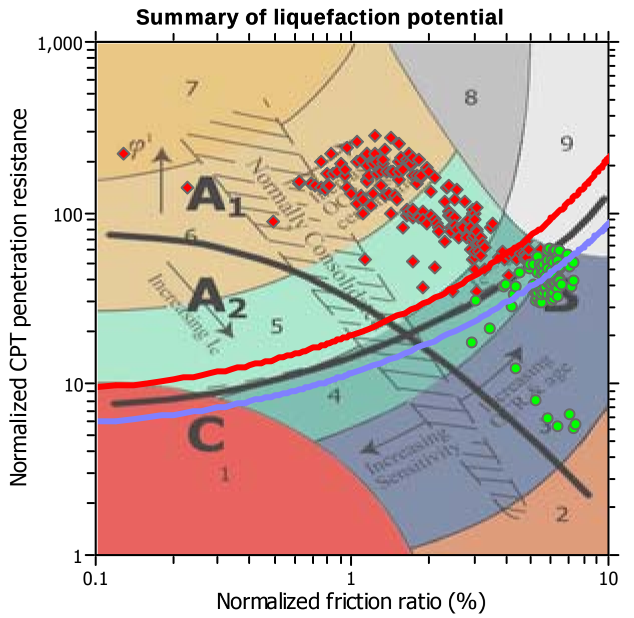
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	22.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	No		

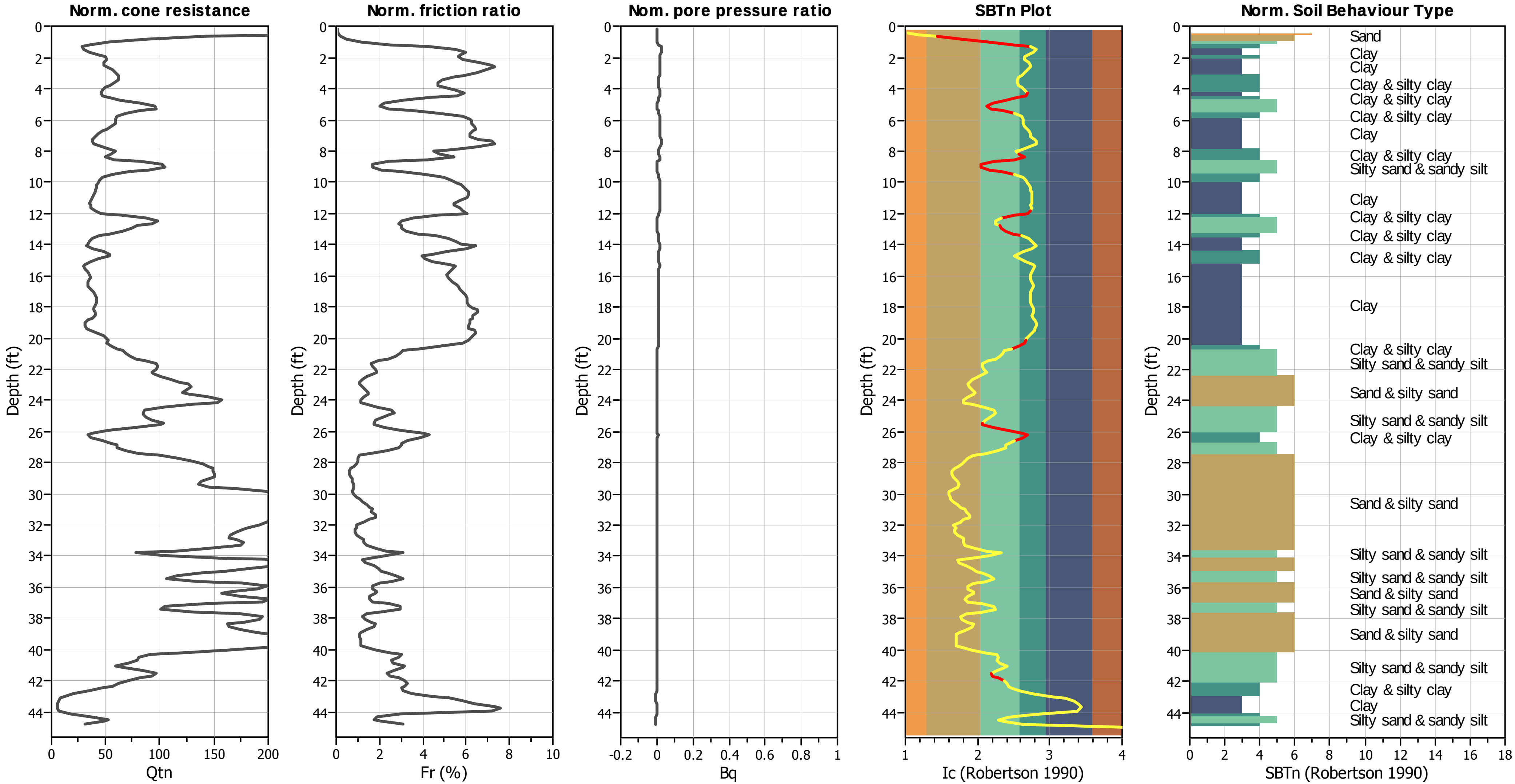






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



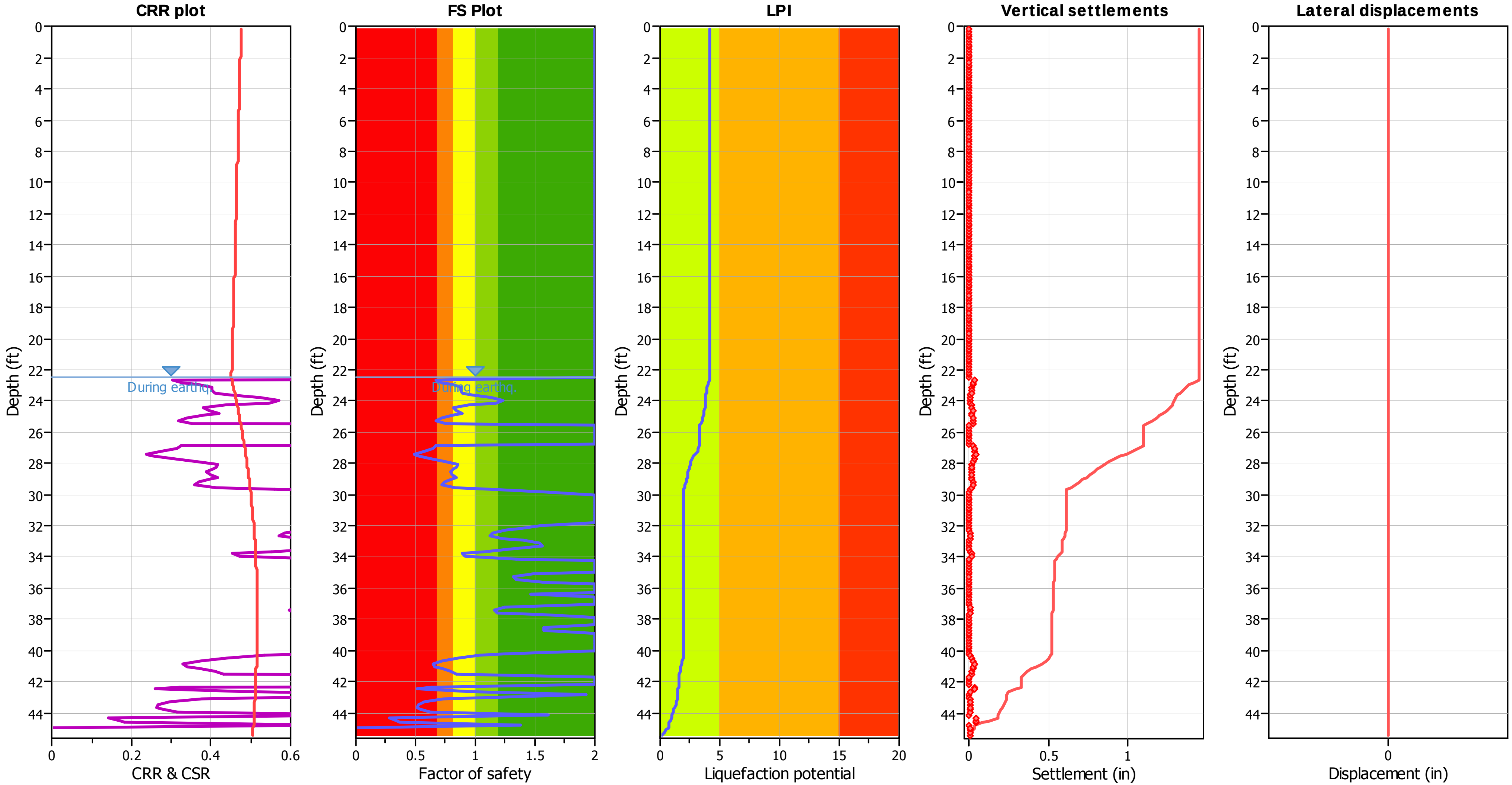
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

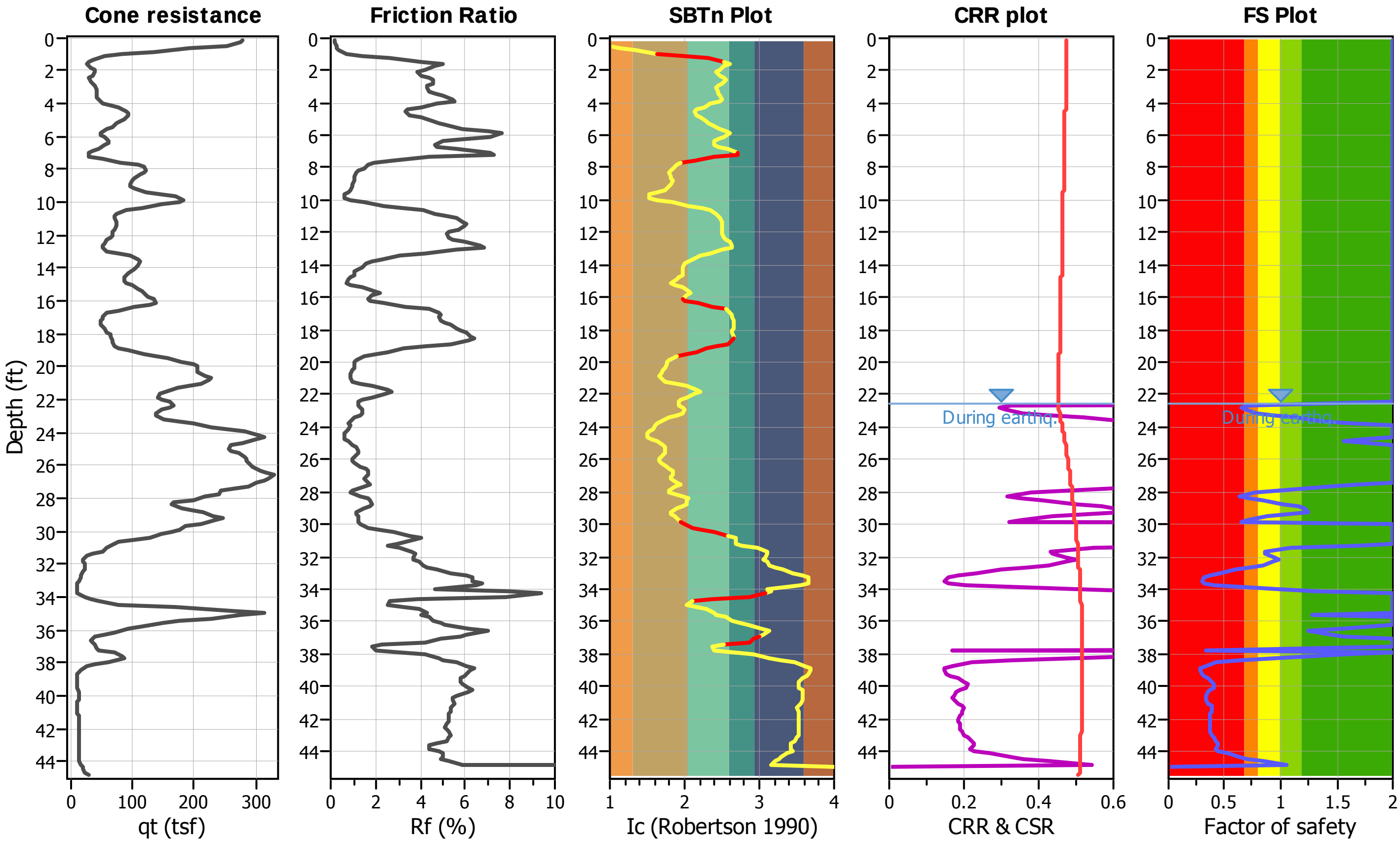
LIQUEFACTION ANALYSIS REPORT

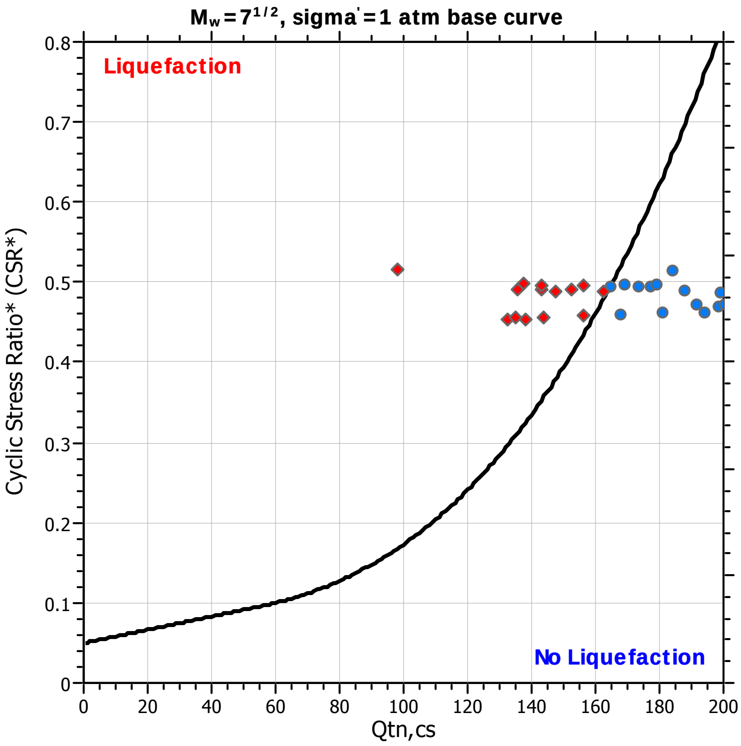
Project title : 14723.000.000 - 5150 El Camino Real **Location : Los Altos, CA**

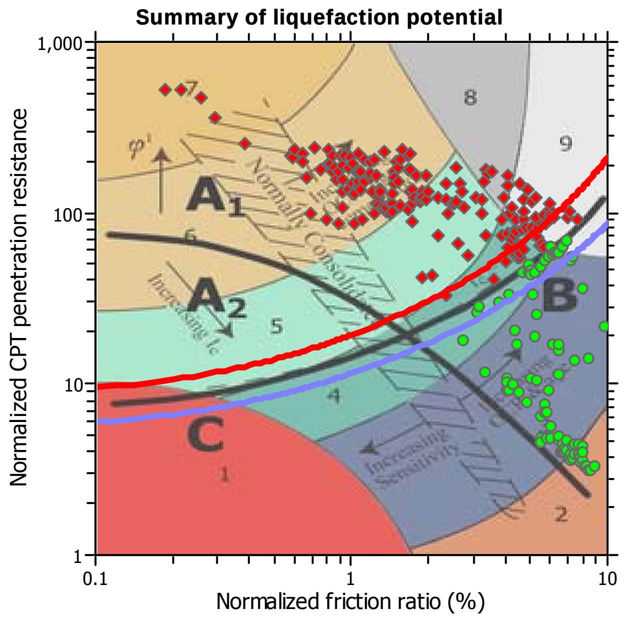
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	22.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	No		

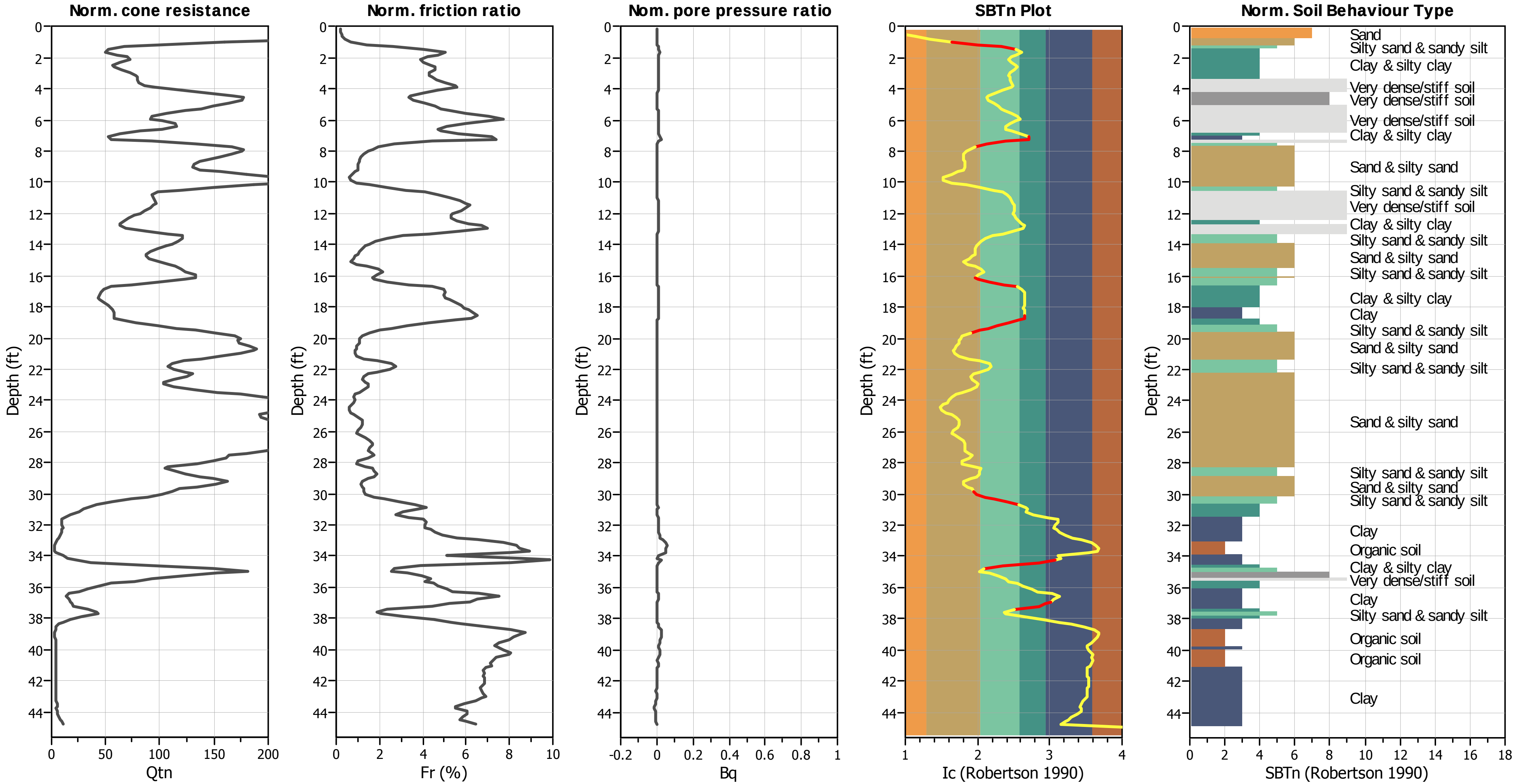






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



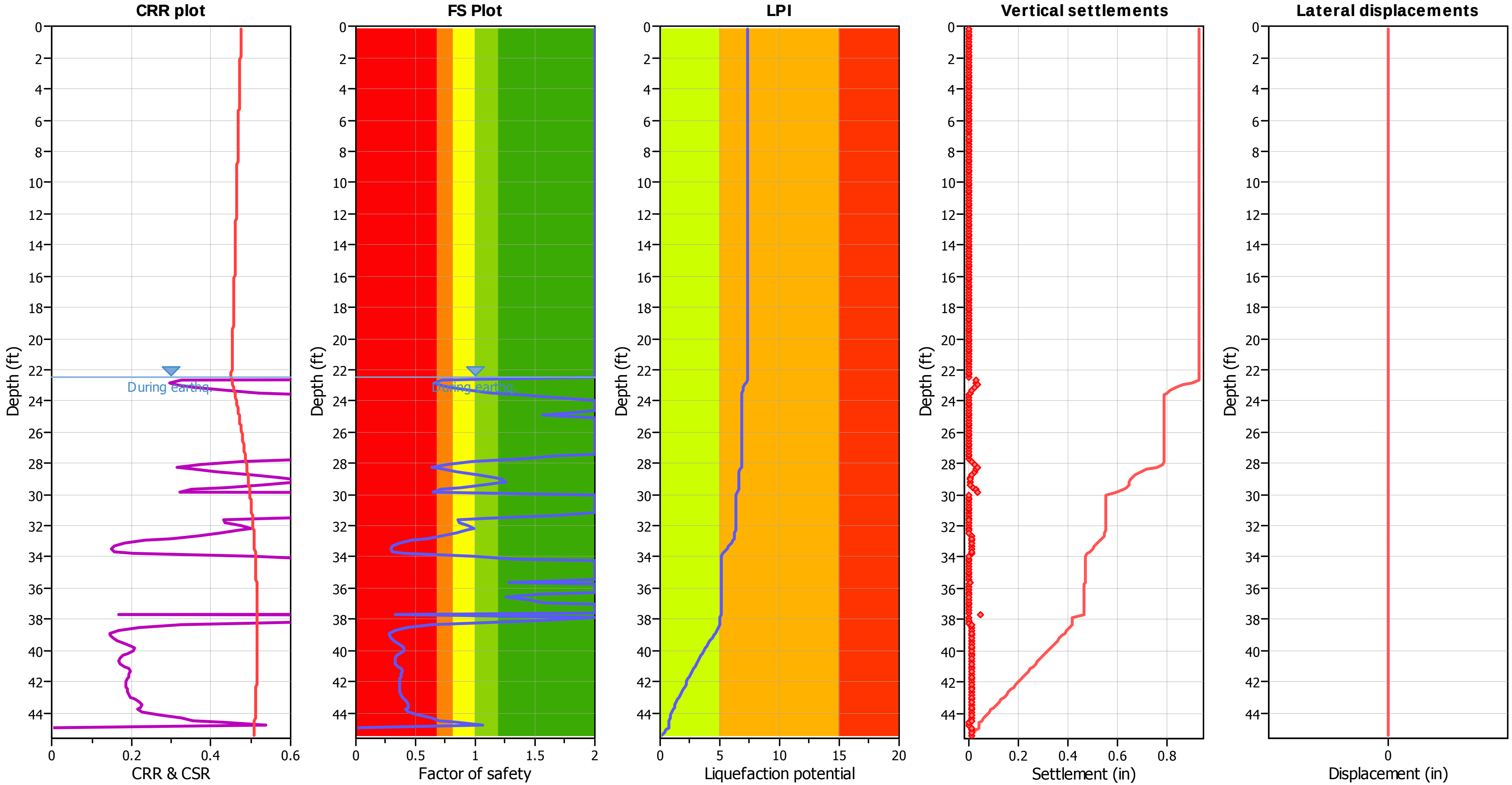
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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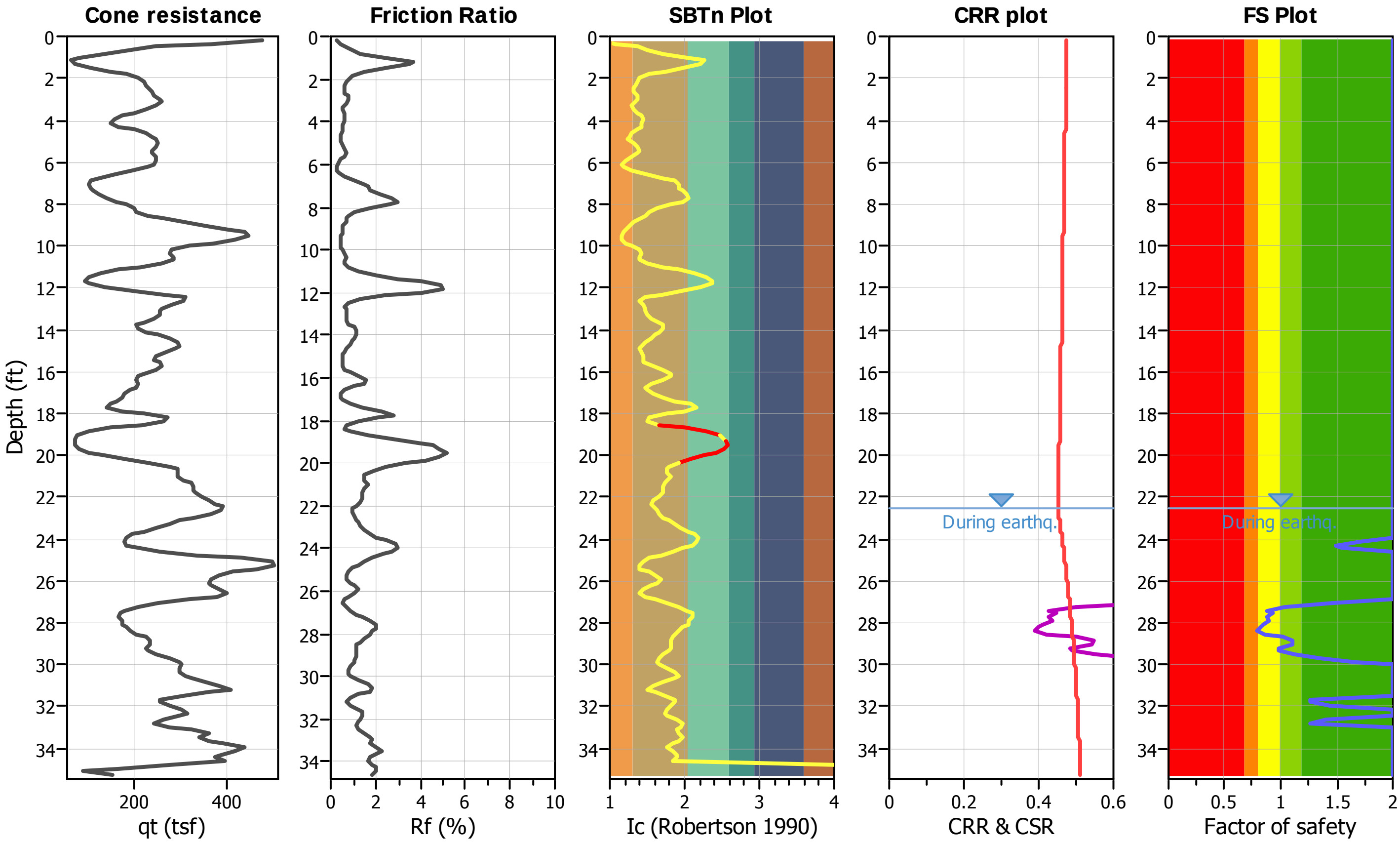
LIQUEFACTION ANALYSIS REPORT

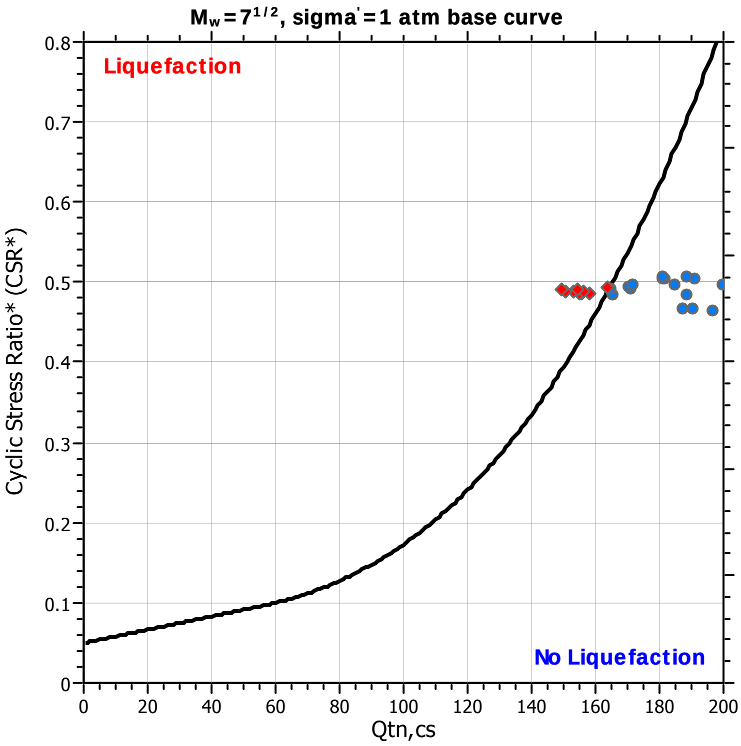
Project title : 14723.000.000 - 5150 El Camino Real **Location : Los Altos, CA**

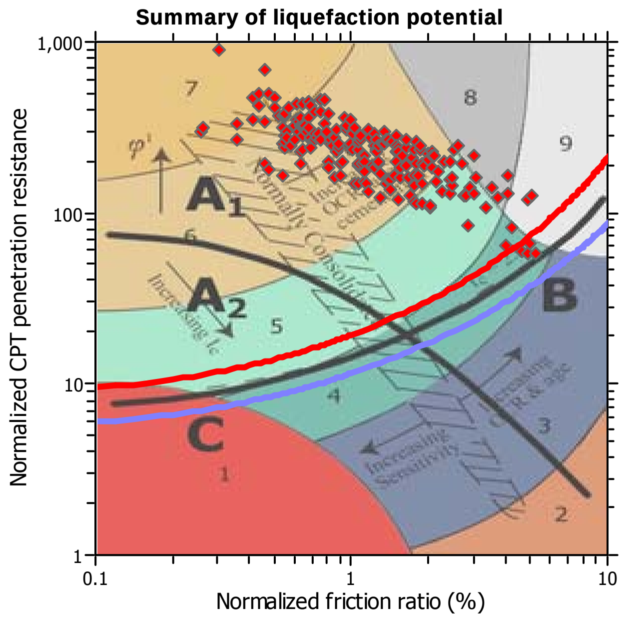
CPT file : 1-CPT5

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	22.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	No		

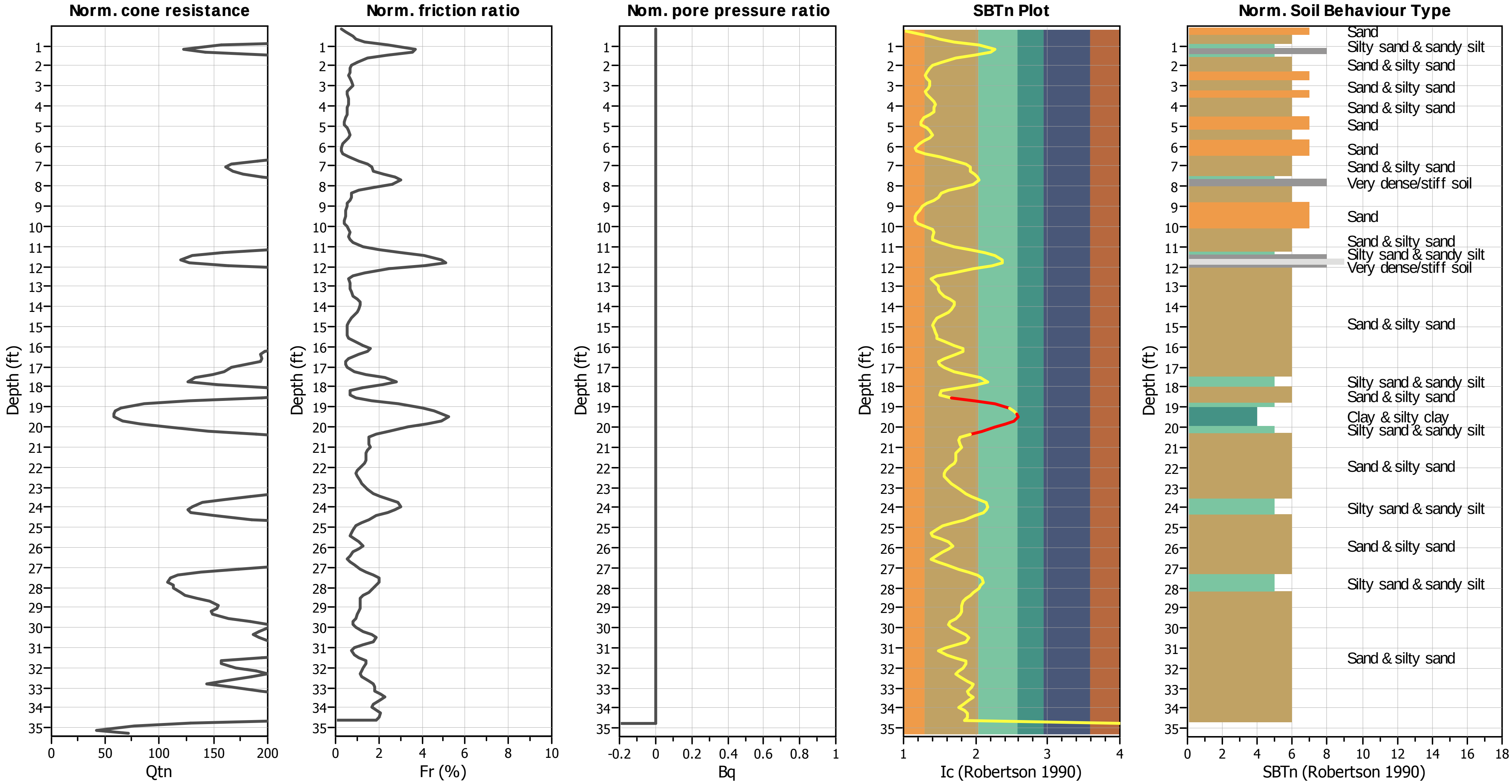






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



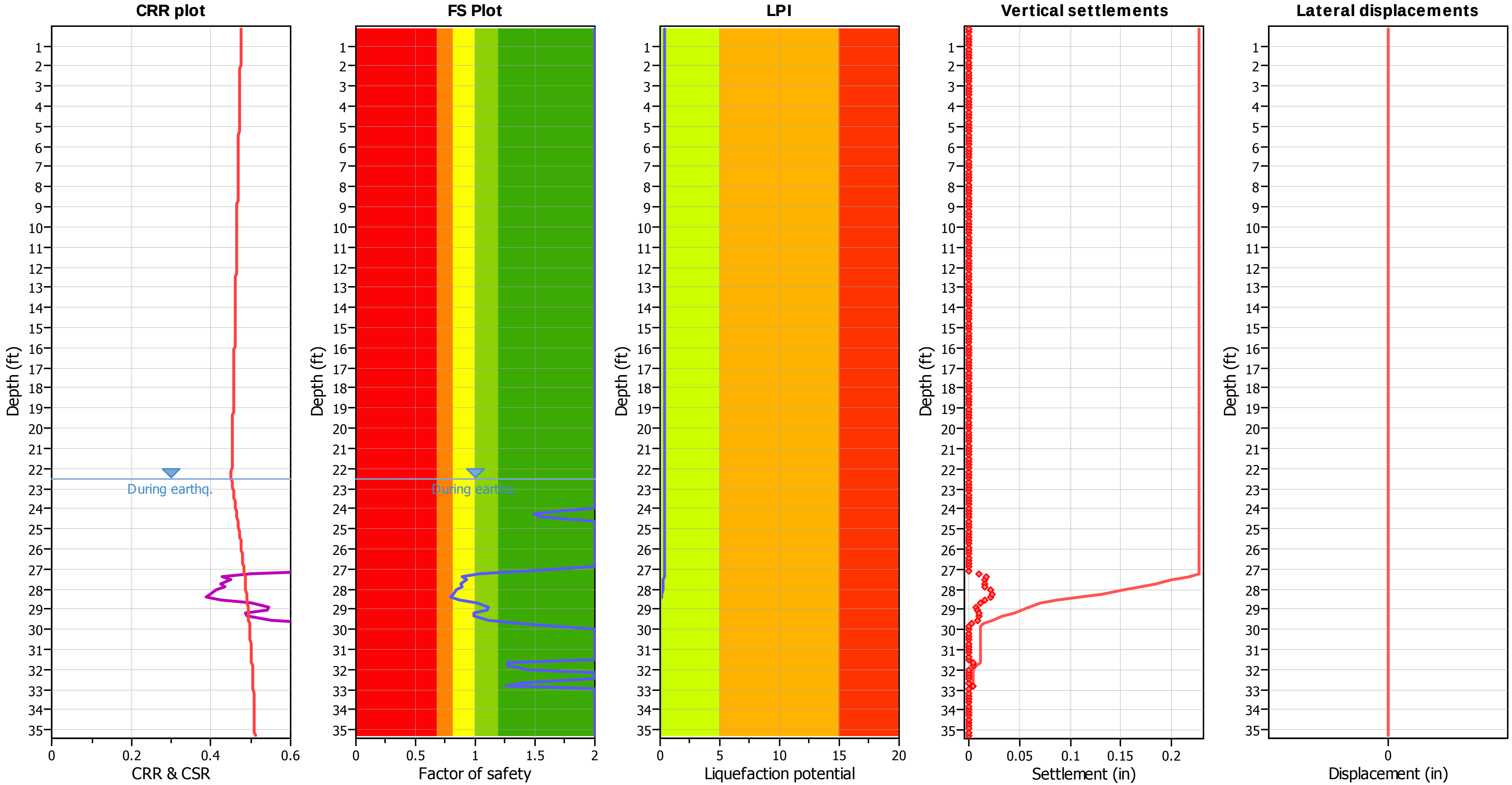
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	22.50 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

SAN RAMON

SAN FRANCISCO

SAN JOSE

OAKLAND

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RENO

ROCKLIN

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WELLINGTON

AUCKLAND

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