Appendix E:
Geology and Soils Geotechnical Supporting Information

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DEL HOMBRE LANE
WALNUT CREEK, CALIFORNIA

PRELIMINARY GEOTECHNICAL REPORT

SUBMITTED TO

Ms. Kristen Gates
Hanover R.S. Limited Partnership
5847 San Felipe, Suite 3600
Houston, TX 77057

PREPARED BY
ENGEO Incorporated

April 6, 2018

PROJECT NO.
14813.000.000

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April 6, 2018

Ms. Kristen Gates
Hanover R.S. Limited Partnership
5847 San Felipe, Suite 3600
Houston, TX 77057

Subject: Del Hombre Lane
Walnut Creek, California

PRELIMINARY GEOTECHNICAL REPORT

Dear Ms. Gates:

With your authorization, we performed a preliminary geotechnical report for the Del Hombre Lane property located in Walnut Creek, California. This report presents the results of our geotechnical observations, as well as our preliminary conclusions and recommendations for the project. Additionally, we also provide preliminary site grading, treatment of potential geologic hazards, and foundation recommendations for use during land planning.

Based upon our initial assessment, the proposed residential development is feasible from a geotechnical standpoint. The primary geotechnical concerns for the site development include presence of shallow groundwater, disturbed near-surface soils and non-engineered fills, expansive soil, compressibility of relatively soft alluvial clay at depth, seismic ground motions, and soil corrosivity. This report provides our preliminary conclusions and recommendations for planning. A design-level exploration should be conducted prior to site development once more detailed land plans and structural loads have been prepared.

We are pleased to have been of service on this project and are prepared to consult further with you and your design team as the project progresses. If you have any questions regarding the contents of this report, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

Todd Bradford, PE

Theodore P. Bayham, GE, CEG

REGISTERED PROFESSIONAL ENGINEER
THEODORE P. BAYHAM
No. 2480
GEOTECHNICAL
STATE OF CALIFORNIA

Spencer Waganaar, EIT
tb/tpb/sw/jf

TABLE OF CONTENTS

LETTER OF TRANSMITTAL

1.0 INTRODUCTION 1
- 1.1 PURPOSE AND SCOPE 1
- 1.2 PROJECT LOCATION 1
- 1.3 PROPOSED PROJECT 1
- 1.4 SITE HISTORY 2

2.0 PRELIMINARY FINDINGS 2
- 2.1 GEOLOGY AND SEISMICITY 2
  - 2.1.1 Geology 2
  - 2.1.2 Seismicity 2
- 2.2 FIELD EXPLORATION 3
- 2.3 SURFACE CONDITIONS 3
- 2.4 SUBSURFACE CONDITIONS 4
- 2.5 GROUNDWATER CONDITIONS 4
- 2.6 LABORATORY TESTING 4

3.0 PRELIMINARY CONCLUSIONS 5
- 3.1 SHALLOW GROUNDWATER 5
- 3.2 DISTURBED NEAR-SURFACE SOIL 5
- 3.3 EXPANSIVE SOIL 5
- 3.4 COMPRESSIBLE SOIL 6
- 3.5 SEISMIC HAZARDS 6
  - 3.5.1 Ground Rupture 6
  - 3.5.2 Ground Shaking 6
  - 3.5.3 Ground Lurching 7
  - 3.5.4 Soil Liquefaction/Cyclic Softening 7
  - 3.5.5 Lateral Spreading 8
- 3.6 SOIL CORROSION POTENTIAL 8
- 3.7 2016 CBC SEISMIC DESIGN PARAMETERS 8

4.0 PRELIMINARY RECOMMENDATIONS 8
- 4.1 GENERAL SITE CLEARING 9
- 4.2 DISTURBED NEAR-SURFACE SOIL 9
- 4.3 EXPANSIVE SOIL MITIGATION 9
- 4.4 SELECTION OF MATERIALS 9
- 4.5 FILL COMPACTION 9
- 4.6 PRELIMINARY FOUNDATION RECOMMENDATIONS 10
  - 4.6.1 Conventionally Reinforced Mat Foundations 10
  - 4.6.2 Uplift Forces 11
- 4.7 EXCAVATION AND SHORING 11
  - 4.7.1 Dewatering 11
  - 4.7.2 Pre-Construction Survey And Construction Monitoring 12
- 4.8 SURFACE DRAINAGE 12
- 4.9 PRELIMINARY BASEMENT WALL RECOMMENDATIONS 12
  - 4.9.1 Wall Drainage 13

TABLE OF CONTENTS (Continued)

5.0 FUTURE STUDIES 13

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS 13

SELECTED REFERENCES

FIGURES

APPENDIX A – CPT Data

APPENDIX B – Liquefaction Analysis

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical report is to provide an assessment of the potential geotechnical constraints associated with the use of the site for the proposed residential apartment development. The scope of our services included a site visit, review of published geologic maps, review of existing subsurface data on the site and in the area, performing subsurface exploration, and preparation of this report identifying potential geotechnical hazards.

This preliminary report was prepared for the exclusive use of our client and their consultants for planning purposes only. Design-level exploration and laboratory testing should be performed to facilitate preparation of construction drawings and performance of land development. 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

The property is located on the east side of Del Hombre Lane at the intersection of Del Hombre Lane and Roble Road in Walnut Creek, California (Figure 1). The site is a compilation of five separate APN properties. The approximately 2½-acre property includes five parcels in a mixed residential/commercial area of Walnut Creek.

The address, APNs and acreage associated with the property are listed in Table 1.2-1 below:

TABLE 1.2-1: APNs Associated with the Property

ADDRESS	ACREAGE	APN
3010 Del Hombre Lane	0.49	148-170-001-9
3018 Del Hombre Lane	0.19	148-170-022-5
3050 Del Hombre Lane	0.92	148-170-041-5
3070 Del Hombre Lane	0.52	148-170-042-3
112 Robles Road	0.37	148-170-037-3

The property is currently occupied by two residential houses. These houses appear to be currently occupied. Furthermore, the site is bordered by Del Hombre Lane to the west, residential apartments to the north and south, and residential houses to the east.

1.3 PROPOSED PROJECT

Based on discussions and a preliminary concept plan (8/15/2017), the proposed project will be a residential apartment complex equipped with two courtyard areas and a community pool. Additionally, the structure will have one level of underground basement parking. The superstructure will be five stories of residential living apartments. It is anticipated that the above-grade residential structure will be a concrete-podium-style structure with the basement and first floor or two comprised of concrete with the remaining floors of the superstructure wood framed. Therefore, we anticipated building loads to be moderate and similar to other buildings of this size.

1.4 SITE HISTORY

We reviewed historic aerial photographs and topographic maps to determine if discernable changes in topography or surface modifications pertaining to the property have been recorded.

We reviewed historic aerial photographs using historicaerials.com and Google Earth, with available aerial photographs spanning from 1939 to 2017. Based on these aerial photographs, the subject property appears to have been primarily used for agricultural purposes in the 1940s to 1960s, with a single house appearing to be on the property along with several dirt roads. Development around the site appeared to be in the late 1960s, with train tracks and parking lots being constructed on the west side of the property.

Photographs taken in late 1980s indicate that an additional four houses were located on the property and the surrounding area contained a number of residential and commercial buildings. Since that time, three of the houses appear to have been demolished, leaving three vacant lots on the property. Aside from the noted construction and removal of residential houses, the site appears to have remained relatively unchanged from the aerial photographs over this 78-year period.

Topographic maps support information observed during the aerial photograph review, illustrating a relatively flat site with little to no change in topographical elevation over the 118-year span (1897 to 2017) of published topographic maps reviewed.

2.0 PRELIMINARY FINDINGS

2.1 GEOLOGY AND SEISMICITY

2.1.1 Geology

The site is situated in the Coast Ranges geomorphic province of California. The Coast Ranges have experienced a complex geological history characterized by Late Tertiary folding and faulting that has resulted in a series of northwest-trending mountain ranges and intervening valleys. Bedrock in the Coast Ranges consists of igneous, metamorphic and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-order faults.

According to the published geologic map covering the site by Nilsen (1975), the site is underlain by Holocene alluvial deposits. Mapping by Dibblee (2005) indicates the site is underlain alluvium (Qa), specifically alluvial gravels. Graymer (1994) further classifies the Holocene alluvial deposits at the site as being floodplain deposits (Qhfp), consisting of sandy to silty clay, with lenses of coarser material, and alluvial deposits (Qhaf), consisting of gravelly sand which grades to silty clay near the edges of the deposit. The regional geologic map is included on Figure 4.

2.1.2 Seismicity

The site is not located within a State of California Earthquake Fault Hazard Zone (1982) for active faults, and no known faults cross the site. 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) (Hart, 1997). The nearest known active fault surface trace is the Green Valley Connected Fault, which is mapped approximately 2.2 miles east of the site.

Other active faults near the site are summarized in Table 2.1.2-1 and include the Mount Diablo Thrust fault, Calaveras Fault, and Hayward Fault. Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (greater than Moment Magnitude 7) earthquakes have been recorded and can be expected to occur in the future. Figure 5 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region.

TABLE 2.1.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site
Latitude: 37.9294 **Longitude:** -122.0539

FAULT NAME	DISTANCE FROM SITE (miles)	DIRECTION FROM SITE	MAXIMUM MOMENT MAGNITUDE (Ellsworth)
Green Valley Connected	2.2	East	6.8
Mount Diablo Thrust	3.7	South	6.7
Calaveras	8.1	South	7.0
Hayward-Rodgers	11.2	East	6.8
Greenville Connected	12.4	Southeast	7.0
West Napa	19.2	Northwest	6.7

2.2 FIELD EXPLORATION

We performed our field exploration on March 14 and 15, 2018. We retained the services of a subcontractor with a CPT rig to advance five cone penetration tests to depths of up to 70 feet below ground surface (bgs) 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). Pore pressure dissipation tests were performed in 1-CPT1, 1-CPT3, 1-CPT4, 1-CPT5 to measure the approximate subsurface phreatic surface. Figure 2 shows the approximate CPT locations and CPT logs are presented in Appendix A.

The locations of our explorations are approximate and estimated by visual assessment using aerial photographs on Figure 2. These locations should be considered accurate only to the degree implied by the method used.

2.3 SURFACE CONDITIONS

The relatively flat property is currently occupied by two existing single-story residential houses. We observed a distressed, overgrown, and obsolete concrete driveway in an east-west orientation, various distressed fences, and pole-mounted above-ground utilities. The remaining area are covered with grass and trees.

2.4 SUBSURFACE CONDITIONS

In general, our CPTs generally encountered soft to medium stiff clay and silty clay throughout the stratigraphy in 1-CPT1 (60 feet), 1-CPT2 (40 feet), and 1-CPT4 (50 feet). Lenses of sand were encountered in 1-CPT3 (between 62 and 72 feet bgs) and 1-CPT5 (between 22 and 34 feet bgs).

Consult the Site Plan (Figure 2) and CPT logs (Appendix A) for location and specific subsurface conditions at each location. The logs graphically depict the subsurface conditions encountered at the time of the exploration.

2.5 GROUNDWATER CONDITIONS

We did not measure groundwater directly from the recent CPTs; however, we did perform pore pressure dissipation testing at 1-CPT1, 1-CPT3, 1-CPT4, and 1-CPT5 to determine approximate groundwater levels. Readings from these tests indicate a groundwater level of approximately 15 to 20 feet bgs. Table 2.5-1 summarizes our interpretations of the pore pressure dissipation tests.

TABLE 2.5-1: Pore Pressure Dissipation Test Interpretations

SOURCE	APPROXIMATE DEPTH TO GROUNDWATER (feet)
1-CPT1	18
1-CPT3	20
1-CPT4	17
1-CPT5	15

Shallow groundwater can:

1. Delay grading activities, especially compacting soil below basement elevations.

2. Require construction dewatering.

3. Cause moisture damage to sensitive floor coverings.

4. Transmit moisture and vapor through foundations causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.

5. Require waterproofing and damp-proofing for the proposed below-grade portions of structure.

2.6 LABORATORY TESTING

We did not collect samples in the field during the exploration. Soil sampling and testing, including plasticity index, strength evaluations, moisture content, unit weight, sulfate, and grain size distribution should be incorporated into a future design-level geotechnical exploration.

3.0 PRELIMINARY CONCLUSIONS

From a geotechnical engineering viewpoint, the site is suitable for the proposed development provided the preliminary conclusions and recommendations are incorporated into planning. The primary geotechnical concerns for the site development include presence of shallow groundwater, disturbed near-surface soil and non-engineered fills, expansive soil, compressibility of relatively soft alluvial clay at depth, seismic ground motions, and soil corrosivity. We summarize our preliminary conclusions and recommendations in the following sections of this report.

A design-level geotechnical exploration should be performed to provide recommendations for development.

3.1 SHALLOW GROUNDWATER

Depending on the final design depth of the proposed excavation of the planned structure, groundwater could influence the bottom of the excavation given our observations and anticipated seasonal groundwater fluctuations. As such, temporary construction dewatering, and design of below-grade portions of the structure for hydrostatic conditions and waterproofing are anticipated.

3.2 DISTURBED NEAR-SURFACE SOIL

Disturbed native soils and non-engineered fills can undergo excessive settlement, especially under new fill or building loads. As previously mentioned, we encountered disturbed site soil that had likely been seasonally tilled during agricultural usage and as such, should be treated as non-engineered soil. In addition, previous development likely included substructures, septic and buried utilities. To mitigate the effects of the disturbed near-surface materials and non-engineered fill, we recommend complete removal and recompaction of loose and disturbed soils and non-engineered fill. Section 4.2 provides recommendations for fill subgrade preparation to address the fill.

3.3 EXPANSIVE SOIL

Based on our project experience in the vicinity and review of the CPT data, we anticipate that expansive soil is present at the site. Expansive soil is susceptible to shrink and swell that could affect structures, slabs-on-grade, pavements, etc. The proposed building foundation depth will inherently mitigate much of the expansive soil concern. However, ground surface improvements such as paving, sidewalks, and retaining walls could be adversely affected by expansive soil.

Mitigation measures typically used to reduce adverse effects of expansive soils for projects, such as this, may include one or several of the following: (1) using a rigid mat foundation (such as post-tensioned slab-on-grade foundations) designed to resist potential movement; (2) deepening foundations to below the zone of moisture fluctuation (such as using deep embedded footings, drilled piers, or piles); (3) placement of a layer of low expansive “select” import fill or use of lime treatment of the soil below the building combined with the use of shallow foundations (such as continuous and spread column footings). The most appropriate foundation solution depends on the building type; however, where feasible, Option 1 generally represents the lowest risk as long as the foundation can be made stiff enough to resist the potential swell and shrinkage forces.

Successful performance of structures on expansive soil requires special attention during construction. It is imperative that exposed soil be kept moist prior to placement of concrete for foundation construction. It is extremely difficult to re-moisturize clayey soil without excavation, moisture conditioning, and re-compaction.

In addition, site grading and treatment of expansive soils may include selective moisture conditioning requirements and compaction within selected ranges. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the compaction. We present expansive soil mitigation recommendations in Section 4.3 of this report.

3.4 COMPRESSIBLE SOIL

Soil can undergo long-term settlement when new loads are introduced by structures or equipment onto saturated clayey deposits. Based on a review of the CPT logs, clayey deposits extend throughout the subsurface soil. The low tip resistance throughout the clayey material indicates it has relatively soft to medium stiff consistency, which is potentially compressible when subject to moderate to high foundation loads. As a result, areas extending below the proposed structure could be subject to varying amounts of consolidation settlement. Characterization of these soft soil layers in relation to foundations should be evaluated to provide foundation design criteria as part of the future design-level geotechnical report.

3.5 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. Common secondary seismic hazards include ground shaking, ground lurching, liquefaction, lateral spreading, landslides, tsunamis, or seiches. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of landslides, tsunamis, and seiches is low to negligible at the site.

3.5.1 Ground Rupture

The site is not located within a State of California Earthquake Fault Hazard Zone and no known faults cross the site. Therefore, it is our opinion that ground rupture is unlikely at the subject property.

3.5.2 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 latest 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).

3.5.3 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. The potential for the formation of these cracks is considered greater at contacts between deep alluvium or fill and bedrock. Such an occurrence does not appear likely at this site and any offset or strain would be minor.

3.5.4 Soil Liquefaction/Cyclic Softening

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop and liquefaction of susceptible soil to occur.

Review of the Seismic Hazard Zone Map of the Walnut Creek Quadrangle (California Geological Survey, 1993) indicates that the site is not located within a mapped liquefaction zone. To assess liquefaction potential, we performed liquefaction analyses utilizing data obtained from the five CPT probes advanced as part of the current field exploration. We assigned a design groundwater level of 15 feet below the existing ground surface and an estimated excavation of 15 feet for the underground parking level. A peak ground acceleration (PGA) of 0.68g, and a Moment Magnitude (Mw) of 7.0; these values are based on the 2016 California Building Code and the commonly accepted potential earthquake magnitude of the closest faults. We performed our analyses using the computer software CLiq Version 1.7 developed by GeoLogismiki, using methods developed by Robertson (2009).

Our analysis identified predominately clay and silty clay layers extending throughout most of the CPT locations. Based on our experience in the area and preliminary analysis, the clay shows low susceptibility to cyclic softening. Susceptibility of the clay to cyclic softening is dependent on the plasticity and water content.

Furthermore, interbedded sandy silt layers, approximately 2 to 6 feet thick were encountered between approximately 22 and 34 feet bgs in 1-CPT5, which may be considered susceptible to liquefaction. Preliminary analysis indicates approximately 3½ inches of liquefaction settlement at this location; however, since this material was not observed in any other CPTs, we do not consider it representative of the entire site's liquefaction susceptibility. Furthermore, based on Ishihara (1985), the non-liquefiable layer above is adequate to mitigate against surface expressions of the liquefiable material, thus reducing the overall predicted settlement potential.

Considering 1-CPT5 to be an outlier, the other CPT locations indicate liquefaction settlements can range from 0.2 to 0.5 inch. Further evaluation through soil borings and laboratory testing, during a design-level report, is necessary to determine the magnitude of potential liquefaction settlements at the site.

3.5.5 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Generally, effects of lateral spreading are most significant at the free face or the crest of a slope and diminishes with distance from the slope. Given the relatively level site topography, the potential for lateral spreading at the site is remote.

3.6 SOIL CORROSION POTENTIAL

We did not perform sampling and testing for corrosion potential as part of this preliminary study. Representative samples of the soil should be collected during the design-level geotechnical exploration, to determine the potential for corrosion on buried metal and the potential for sulfate attack on foundation concrete. Based on these future test results, the corrosion potential can be described and the recommended concrete design parameters can be developed in accordance with the guidelines presented in the California Building Code. If subsurface transformers are proposed for the development, we recommend that the subsurface samples be obtained and tested in accordance with recommendations set forth by Pacific Gas and Electric.

3.7 2016 CBC SEISMIC DESIGN PARAMETERS

We characterized the site as Site Class D in accordance with the 2016 CBC. We provide the 2016 CBC seismic design parameters in Table 3.7-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.7-1: 2016 CBC Seismic Design Parameters
Latitude: 37.9294 Longitude: -122.0539

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.80
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.63
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.80
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.94
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.20
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.63
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.68

4.0 PRELIMINARY 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 future site-specific, design-level geotechnical exploration has been undertaken.

4.1 GENERAL SITE CLEARING

Site development should commence with all debris being removed from the site location, so the site can be graded to receive fill or erect structures. The depth of removal of such materials should be determined by our representative in the field at the time of grading. Following clearing, the site should be stripped to remove surface organic materials. Stripping should extend from the ground surface to a depth of at least 2 to 3 inches below the surface. Strippings should be removed from the site or, if considered suitable by the landscape architect and owner, used in landscape fill.

All excavations below design grade should be cleaned to a firm undisturbed native soil surface determined in the field by a representative of our firm. 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.

4.2 DISTURBED NEAR-SURFACE SOIL

As described previously, we anticipate the presence of disturbed near-surface soil. Such soil can undergo excessive settlement, especially under new fill or building loads. In areas outside of the proposed excavation that will receive at-grade improvements, we recommend complete removal of this material down to undisturbed soil as stated in Section 3.2. The bottom of the removed area should then be scarified and moisture conditioned before placing new engineered fill. Fill placement specifications may be found in Section 4.5.

4.3 EXPANSIVE SOIL MITIGATION

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil or constructing the upper 18 inches of the building pad with non-expansive fill, can reduce the risk of structural damage associated with the expansive soil conditions. Generally, these recommendations include compaction control to reduce over-compaction of the soil and moisture conditioning the soil to well above the optimum moisture content.

The soil expansion potential of the site soil should be evaluated further at the time of design-level study and mitigated during grading activities and through appropriate improvement design.

4.4 SELECTION OF MATERIALS

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, high organic content soil (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. Other material and debris, including trees with their root balls, should be removed from the project site.

4.5 FILL COMPACTION

For land planning and cost estimating purposes, the following compaction control requirements should be anticipated for general fill areas and utility trench backfill:

Onsite expansive soils with Plasticity Index over 12:

- Test Procedures: ASTM D-1557.
- Required Moisture Content: Not less than 3 percentage points above optimum moisture content.
- Required Relative Compaction: Relative compaction between 87 and 92 percent.

Low expansive soils and/or import with Plasticity Index less than 12:

- Test Procedures: ASTM D-1557.
- Required Moisture Content: Not less than optimum moisture content.
- Minimum Relative Compaction: Not less than 90 percent for low-expansive fill.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material. In the event that imported fill material is characterized and following the design level geotechnical report, the recommendations may change with respect to the soil type.

4.6 PRELIMINARY FOUNDATION RECOMMENDATIONS

At the time of this report, we have been provided preliminary development plans, but no structural loads for the proposed building. We assume building loads will be similar to other structures of its type. The major considerations in foundation design at this site are near-surface expansive soil, bearing capacity and settlement due to compressible clay. These effects can be addressed or reduced by the choice of a proper foundation system and a foundation designed to be sufficiently stiff to move as a rigid structure with a potential for differential movements.

4.6.1 Conventionally Reinforced Mat Foundations

The structure may be supported on a rigid mat foundation. The design thickness of the mat foundation will be driven by the structural design and design-level geotechnical recommendations.

Based on CPT logs, preliminary average maximum allowable bearing pressure can be estimated to be approximately 2,000 to 3,000 psf. These values may be increased by one-third when considering transient loads, such as wind or seismic. If a spring constant is needed for initial design, a preliminary modulus of subgrade reaction (ks) of 75 pounds per square inch per inch of deflection (psi/in) can be considered for the preliminary mat foundation.

Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soil and by passive earth pressure acting against the side of the foundation. A coefficient of friction of 0.30 can be used between concrete and the subgrade. Passive pressures can be taken as equivalent to the pressure developed by a fluid having a weight of 300 pounds per cubic foot (pcf).

Preliminary settlement calculations estimate total settlement less than 2 inches with differential settlement reaching half these values over a 50-foot span. Specific load-induced settlement and refined bearing capacities will be performed as part of the design-level geotechnical exploration report using soil characteristics derived from laboratory tests.

4.6.2 Uplift Forces

We anticipate that the basement level garage could be below the groundwater level and may need to be designed for hydrostatic uplift loads. Uplift resistance can be provided by the weight of the foundation elements and structural loads. Additional resistance to uplift may be provided by installing hold-down piers or anchors, if necessary. The pier/anchor capacity should be evaluated using an allowable skin friction of 500 psf. This value may be increased by 30 percent for wind and seismic loading. The piers/anchors should be spaced no closer than 3 times the shaft diameter and have a minimum embedment length of 10 feet. If piers are used, a combination of dewatering, casing, and placement of concrete utilizing tremie methods may be required to facilitate construction. Hold-down anchors should be prestressed to 120 percent of the design capacity and then locked off at 75 percent of the design load.

4.7 EXCAVATION AND SHORING

Excavation shoring will be required to protect adjacent improvements and streets during excavation for the below-grade parking. The design of the shoring should be sufficiently rigid to prevent detrimental movement of the temporary shoring and possible damage of the adjacent buildings, adjacent streets, facilities or other improvements.

Given the proposed excavation depth, it will likely be necessary to restrain the shoring by using a single-level or multi-level system of tieback anchors or to provide internal bracing. Prior to tieback design and construction, permission from the adjacent property owner, City of Walnut Creek or other jurisdictions will have to be obtained if tiebacks are to encroach into those adjacent properties. Tieback anchors should be designed to avoid adjacent underground utilities. Permanent or temporary tiebacks may be installed through the selected shoring system with 15 to 20 degree inclinations.

Excavation, dewatering and shoring are temporary works that are typically the responsibility of the contractor to design, install, maintain and monitor. An experienced shoring and dewatering system designer should be retained to select and design these systems.

Assuming one level of below-grade construction, temporary shoring consisting of drilled or driven soldier piles with timber lagging may be assumed. Additionally, groundwater may be expected near the bottom of the excavation based on historical groundwater levels. During design-level study, the soil conditions will be further assessed and soil parameters for use in excavation and shoring design will be developed.

4.7.1 Dewatering

Dewatering will be required for excavations extending below the groundwater table, to allow for construction under dry conditions. However, extensive dewatering could cause adjacent streets and other improvements to experience unacceptable settlement during construction. The amount of dewatering required could be reduced significantly by using a relatively impervious shoring system and dewatering from inside the excavation. Ultimately, the selection and design of the dewatering system should be the responsibility of the contractor.

There are a number of variables that will influence the effectiveness of a dewatering system, including the number, depth, screened interval, and pumping rate of wells. The local sewer agency may prohibit the discharge of groundwater into the system or may charge a fee to do so.

4.7.2 Pre-Construction Survey And Construction Monitoring

Excavation dewatering and construction will take place adjacent to the adjacent building, existing streets, improvements, and underground utilities. We recommend that a pre-construction survey (e.g. crack survey) and monitoring program for the surrounding culverts, buildings, roadways, utilities, etc., which may be affected by construction activities, be performed before and during construction. This survey will form a basis for any damage claims and also assist the contractor in assessing the performance of the shoring or excavation slopes. The pre-construction survey should record the elevation and horizontal position of all existing installations within 50 feet minimum and may consist of photographs, video tapes, topographic survey, etc.

We also recommend that a system of construction monitoring be installed. This may consist of inclinometers and groundwater monitoring wells that are installed within a distance of 5 to 15 feet from the excavation towards the existing buildings. Vibration monitoring should be considered during operations of heavy equipment such as pile driving, demolition, etc. In addition, a settlement survey should initially be performed on a weekly basis during excavation and on a monthly basis, approximately one month after the excavation has been completed, at a minimum.

4.8 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. 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 surface drainage collection system to reduce ponding of water at the ground surface near the foundation, pavements or exterior flatwork.

4.9 PRELIMINARY BASEMENT WALL RECOMMENDATIONS

We understand that preliminary plans have one level of underground parking. We estimate this will equate to an approximate excavation depth of 10 to 15 feet. For planning purposes, the below earth pressures should be considered for preliminary structural designs:

TABLE 4.9-1: Earth Pressures

LOADING CONDITION	EQUIVALENT FLUID PRESSURES (PCF)	
	WITHOUT HYDROSTATIC PRESSURES (PCF)	WITH HYDROSTATIC PRESSURE (PCF)
Cantilever (Active)	45	85
Restrained (At-Rest)	65	95

Because the site is in a seismically active area, the design should be check for a seismic condition, in which the wall pressure is determined by adding the earth pressure due to earthquake shaking to the static lateral earth pressure. The incremental seismic force is approximated by the triangular pressure, with the force acting at one third of the height of the wall. This force is calculated as $11H^2$, in which H is the height of the wall.

4.9.1 Wall Drainage

Wall drainage may be provided using a 4-inch-diameter perforated pipe embedded in Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about one foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper 1 foot of wall backfill should consist of clayey soils. Drainage should be collected by perforated pipes and directed to a sump or, if feasible, drained into the storm drain system.

5.0 FUTURE STUDIES

As previously discussed, a site-specific design-level geotechnical exploration should be performed as part of the design process. The exploration would include additional explorations and laboratory soil testing to provide additional data for preparation of specific recommendations regarding the following items:

- Grading, existing fill removal and fill compaction
- Consolidation settlement
- Foundation design
- Building retaining walls
- Relatively shallow groundwater
- Temporary shoring and excavation
- Site drainage and landscaping irrigation
- Pavement recommendations

The exploration will also allow for more detailed evaluations of the geotechnical issues discussed in this report and afford the opportunity to provide specific recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents preliminary geotechnical recommendations for design of the improvements discussed in Section 1.3 for the subject Del Hombre Lane, Walnut Creek project. 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 preliminary 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 onsite 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.

SELECTED REFERENCES

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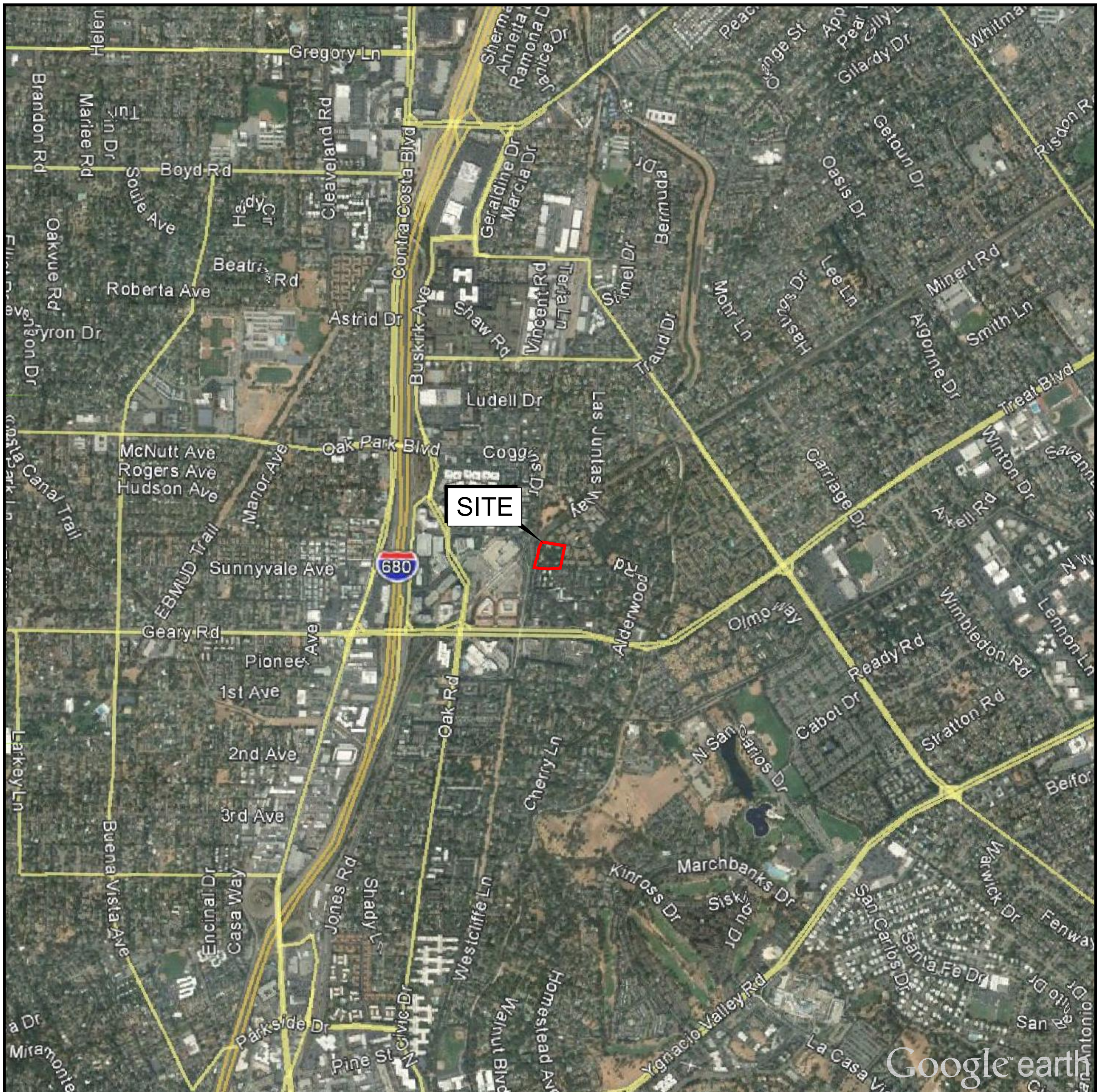
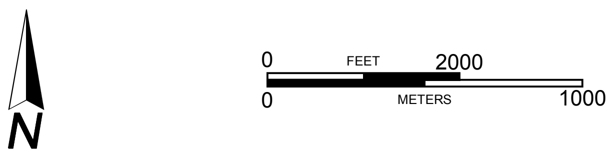
FIGURES

FIGURE 1: Vicinity Map
FIGURE 2: Site Plan
FIGURE 3: Regional Geologic Map (Diblee)
FIGURE 4: Topographic Map
FIGURE 5: Regional Faulting and Seismicity Map





BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP DEL HOMBRE SITE WALNUT CREEK, CALIFORNIA	PROJECT NO.: 14813.000.000		FIGURE NO. 1
		SCALE: AS SHOWN		
		DRAWN BY: SRP	CHECKED BY: TTB	

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14813\000\000-GEX\14813000000-GEX-1-VicinityMap-0318.dwg Plot Date: 4-05-18 spatters

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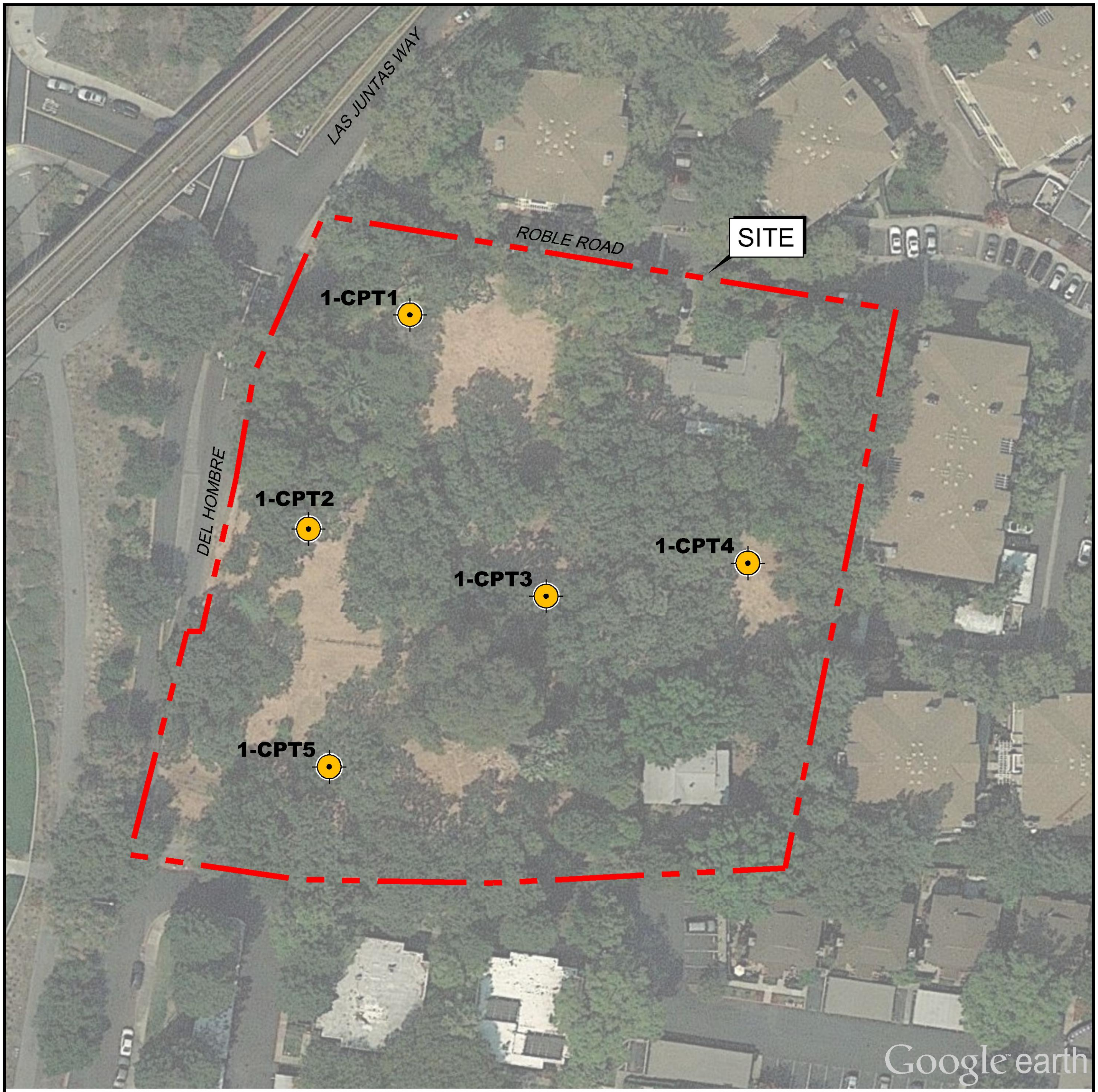

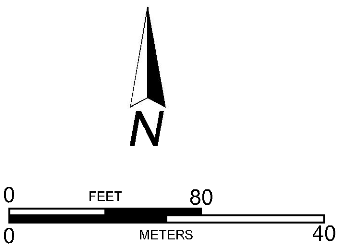

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT5 CONE PENETRATION TEST (ENGEO, 2018)

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	SITE PLAN DEL HOMBRE SITE WALNUT CREEK, CALIFORNIA	PROJECT NO.: 14813.000.000 SCALE: AS SHOWN DRAWN BY: SRP / CHECKED BY: TTB	FIGURE NO. 2

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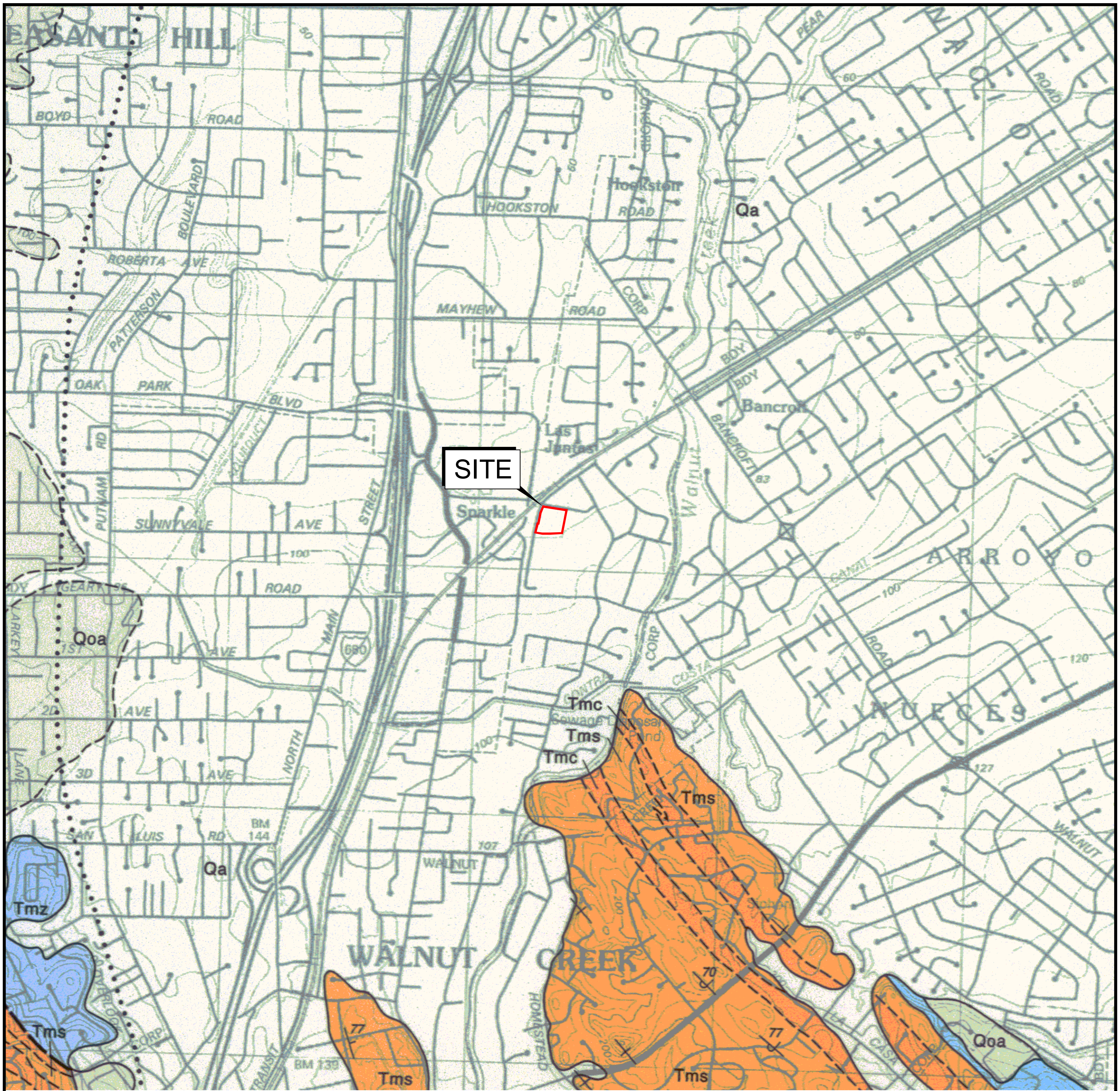


N

0 FEET 2000

0 METERS 1000

EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

Qa	ALLUVIAL GRAVEL
Qoa	OLDER SURFICIAL SEDIMENTS
Tms	MONTEREY FORMATION, SANDSTONE
Tmc	MONTEREY FORMATION, CLAY SHALE/SILTSTONE

STRIKE AND DIP OF STRATA

INCLINED VERTICAL OVERTURNED

BASE MAP SOURCE: DIBBLEE, 2005

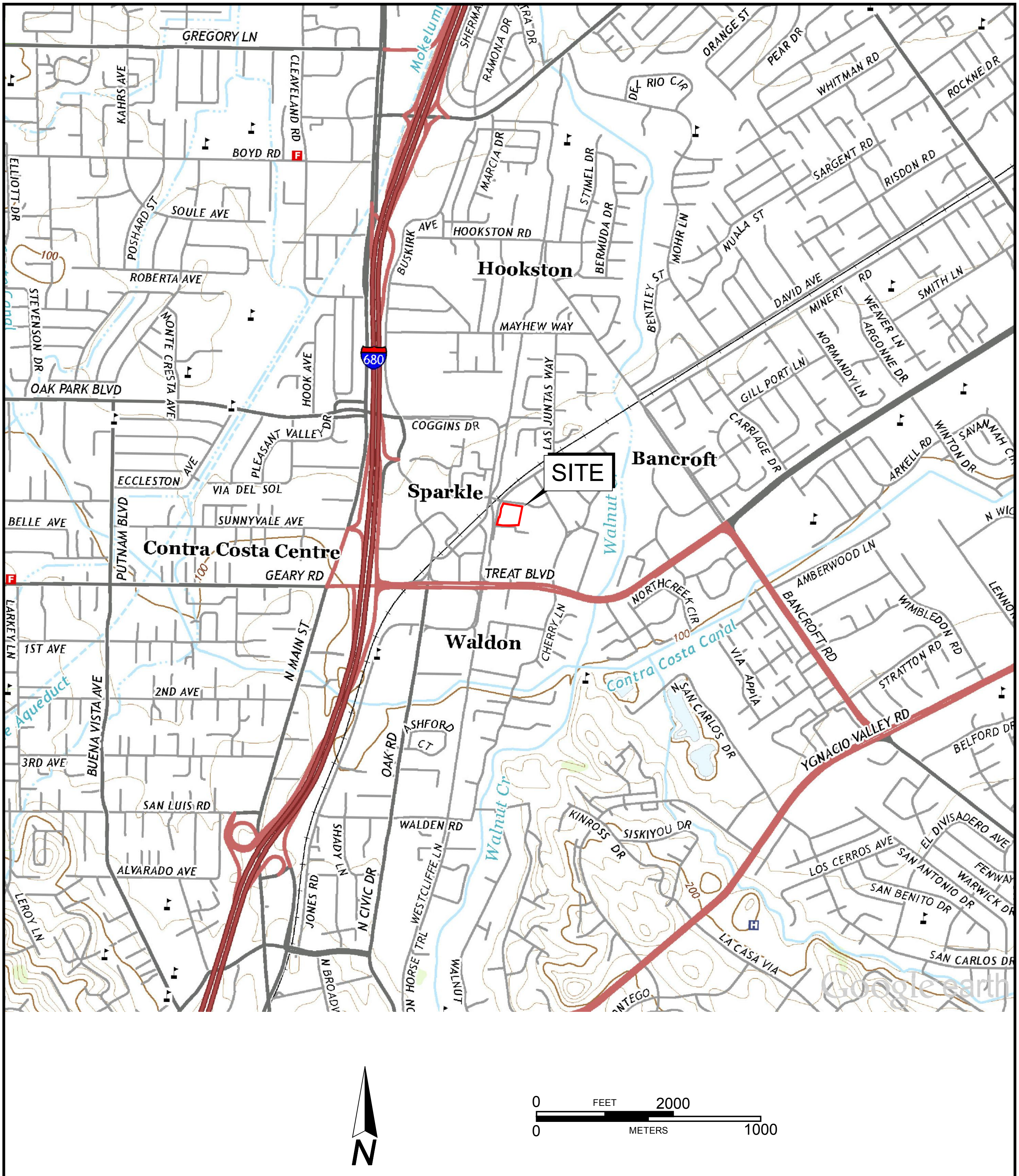
ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
DEL HOMBRE SITE
WALNUT CREEK, CALIFORNIA

PROJECT NO.: 14813.000.000	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: TTB

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14813\000\000-GEX\14813000000-GEX-3-GeologicMap-0318.dwg Plot Date: 4-05-18 spatters

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE: USGS, 2015

ENGEO Expect Excellence	TOPOGRAPHIC MAP DEL HOMBRE SITE WALNUT CREEK, CALIFORNIA	PROJECT NO.: 14813.000.000 SCALE: AS SHOWN DRAWN BY: SRP CHECKED BY: TTB	FIGURE NO. 4

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14813\000\000-GEX\14813000000-GEX-4-TopograpphicMap-0318.dwg Plot Date: 4-05-18 spatters

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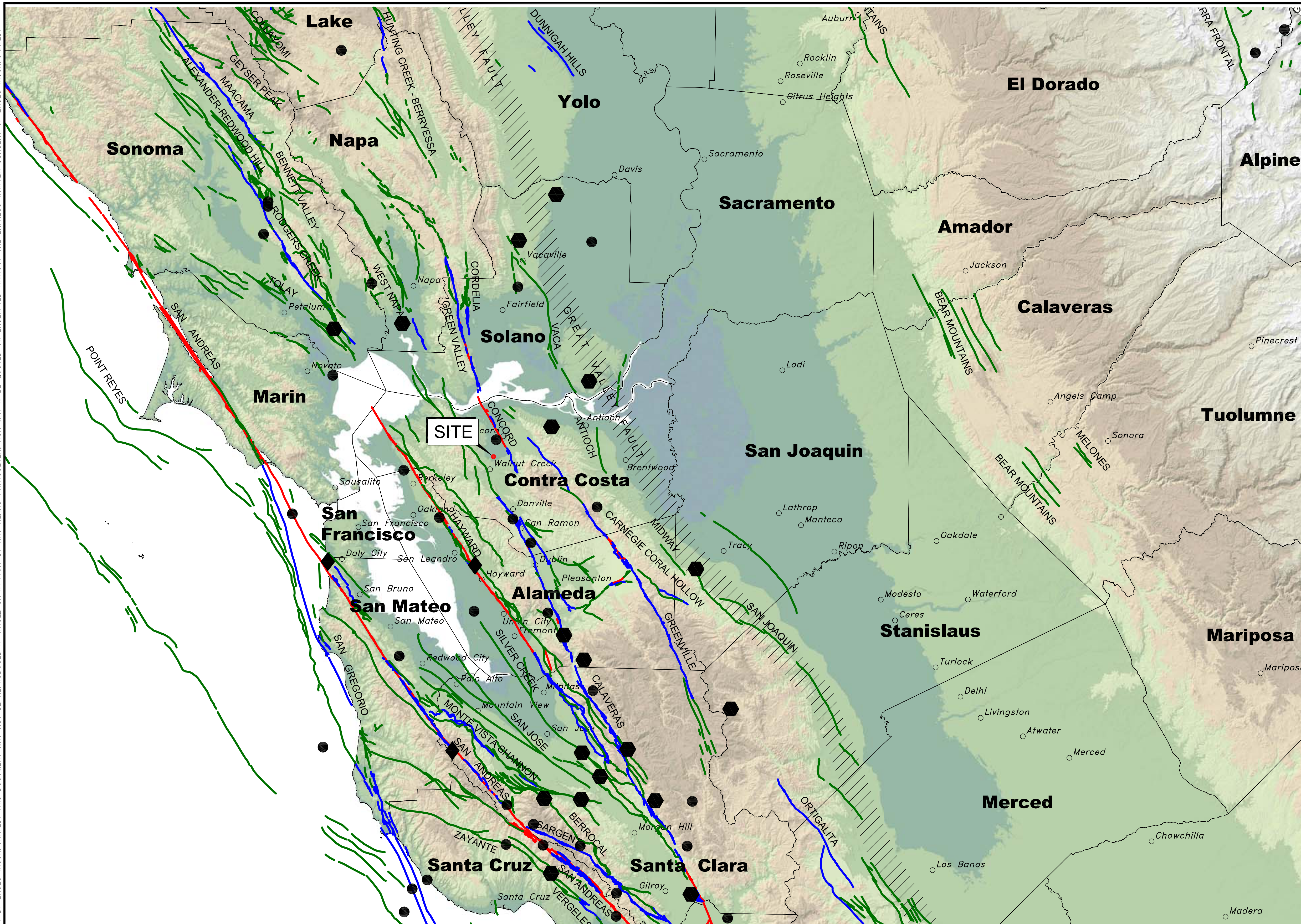

EXPLANATION

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- HISTORIC BLIND THRUST FAULT ZONE

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
DEL HOMBRE SITE
WALNUT CREEK, CALIFORNIA

PROJECT NO.: 14813.000.000	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: TTB

ORIGINAL FIGURE PRINTED IN COLOR

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APPENDIX A

CPT DATA

GREGG DRILLING, INC.
www.greggdrilling.com

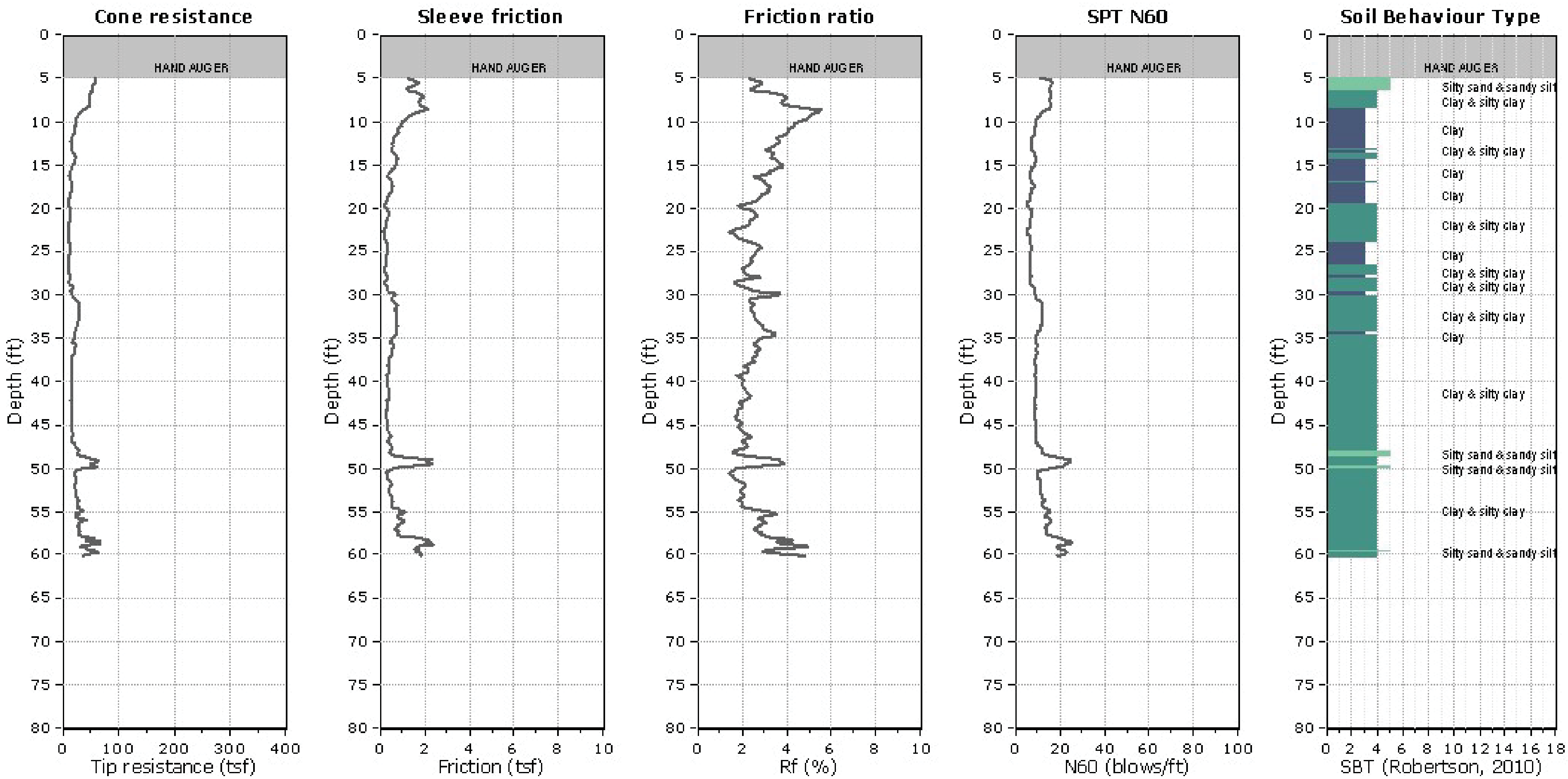
CPT: 1-CPT1

CLIENT: ENGEO

SITE: DEL HOMBRE SITE

Field Rep: TAWNY

Total depth: 60.04 ft, Date: 3/16/2018



SBTn legend

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

GREGG DRILLING, INC.
www.greggdrilling.com

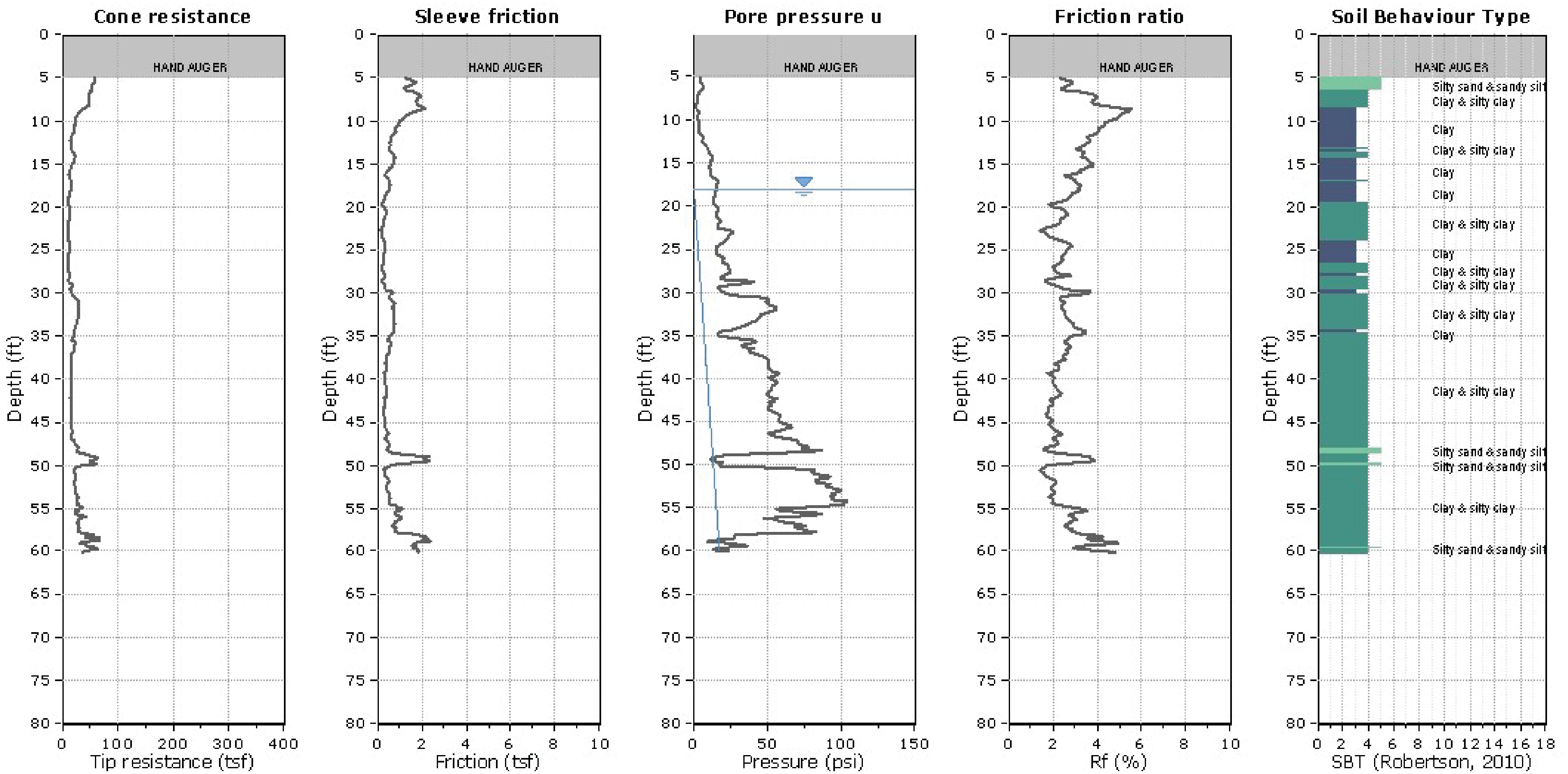
CPT: 1-CPT1

CLIENT: ENGEO

SITE: DEL HOMBRE SITE

Field Rep: TAWNY

Total depth: 60.04 ft, Date: 3/16/2018



SBTn legend

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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

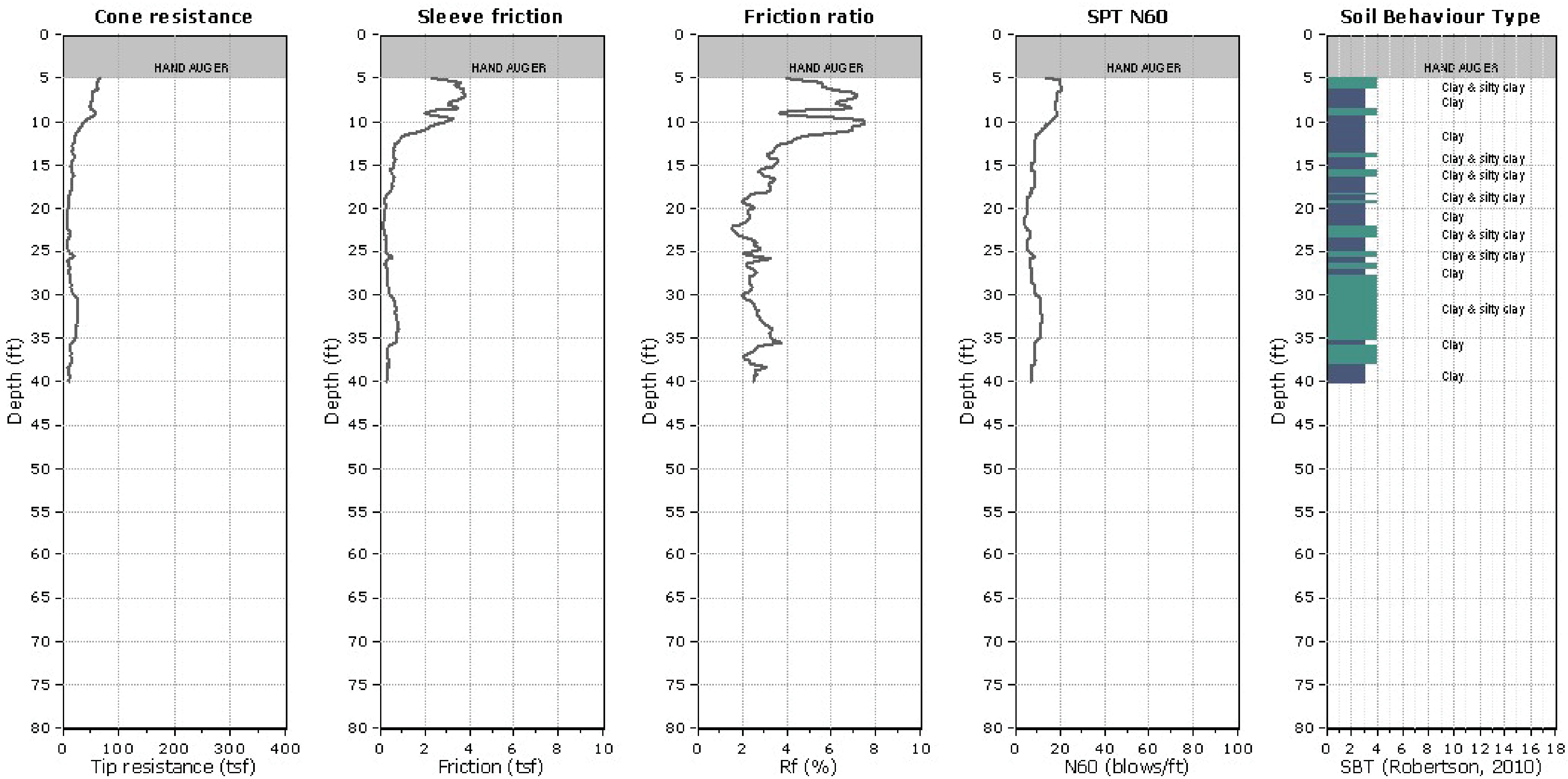
CPT: 1-CPT2

CLIENT: ENGEO

SITE: DEL HOMBRE SITE

Field Rep: TAWNY

Total depth: 40.03 ft, Date: 3/16/2018



SBTn legend

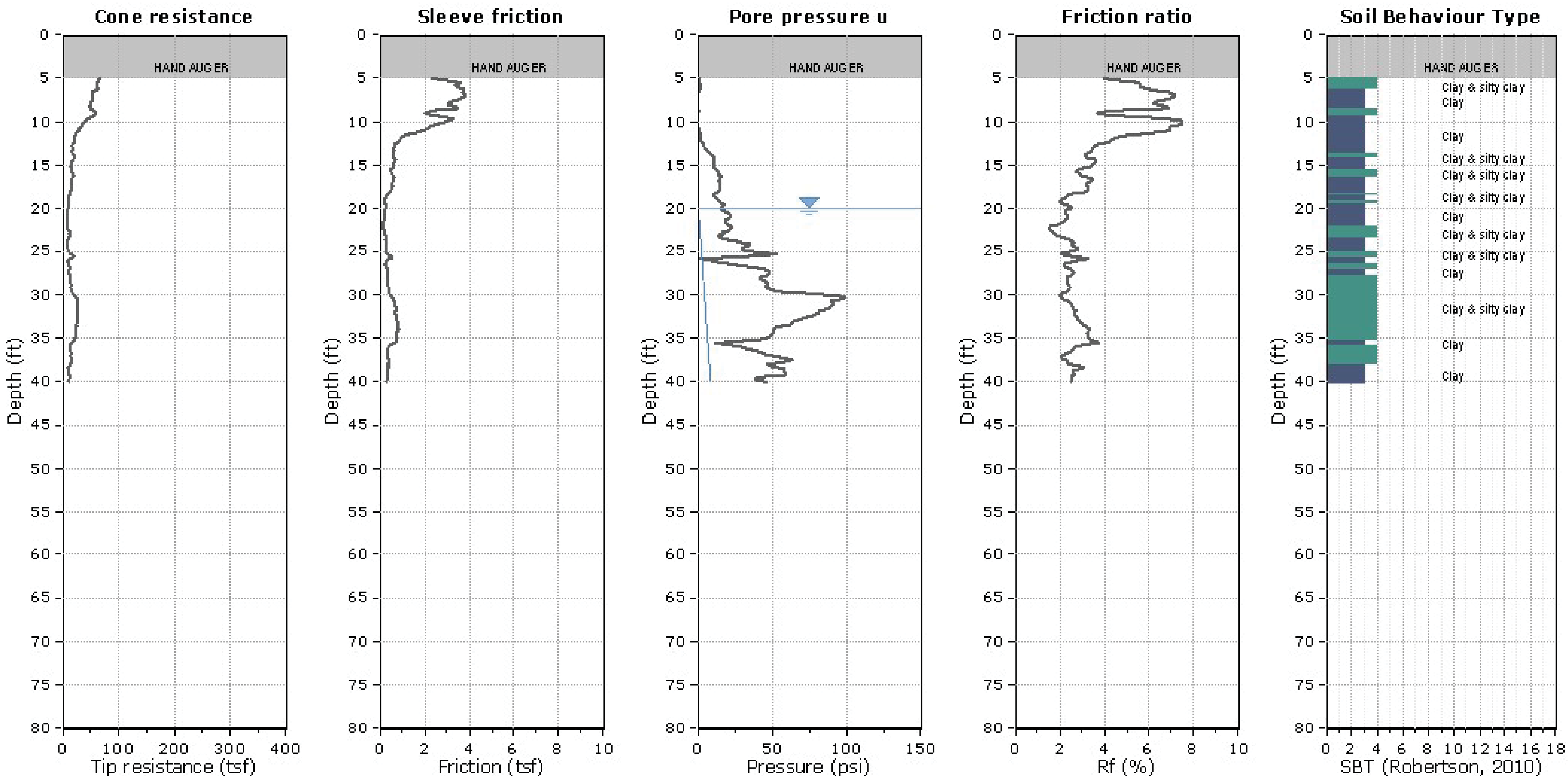
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: 1-CPT2

CLIENT: ENGEO
SITE: DEL HOMBRE SITE

Field Rep: TAWNY
Total depth: 40.03 ft, Date: 3/16/2018



SBTn legend

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

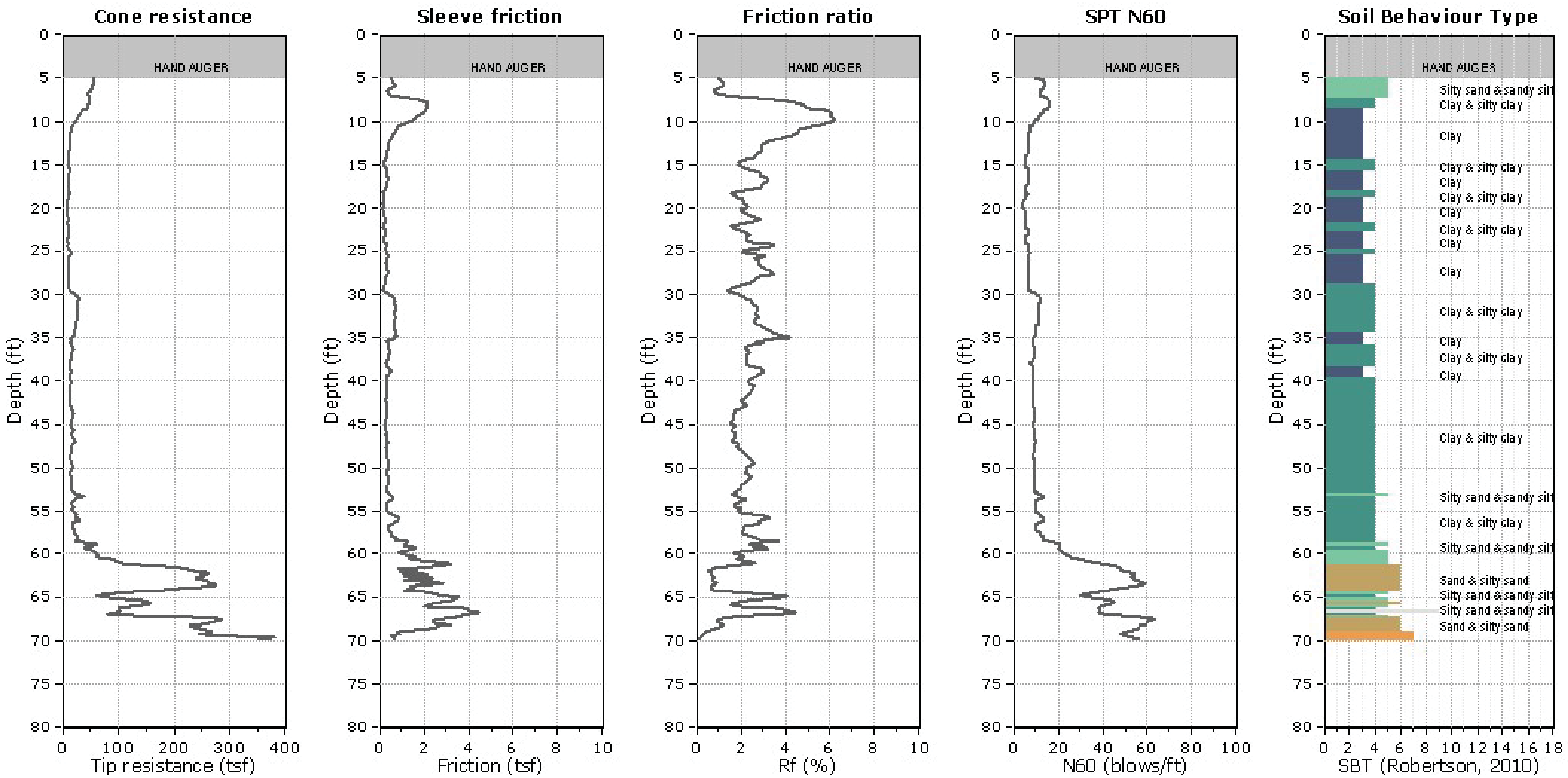
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: 1-CPT3

CLIENT: ENGEO
SITE: DEL HOMBRE SITE

Field Rep: TAWNY
Total depth: 69.72 ft, Date: 3/16/2018



SBTn legend

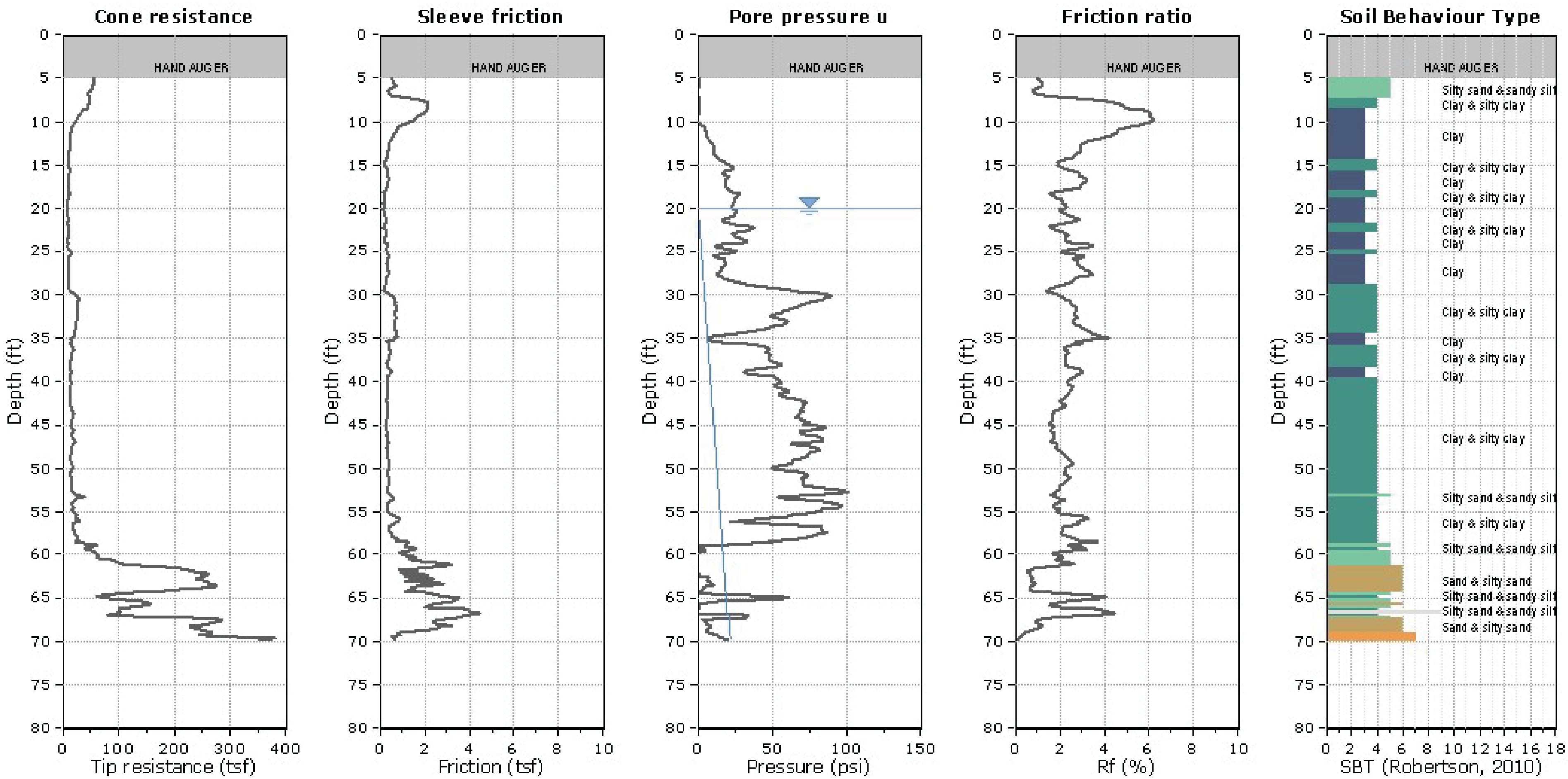
1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: 1-CPT3

CLIENT: ENGEO
SITE: DEL HOMBRE SITE

Field Rep: TAWNY
Total depth: 69.72 ft, Date: 3/16/2018



SBTn legend

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

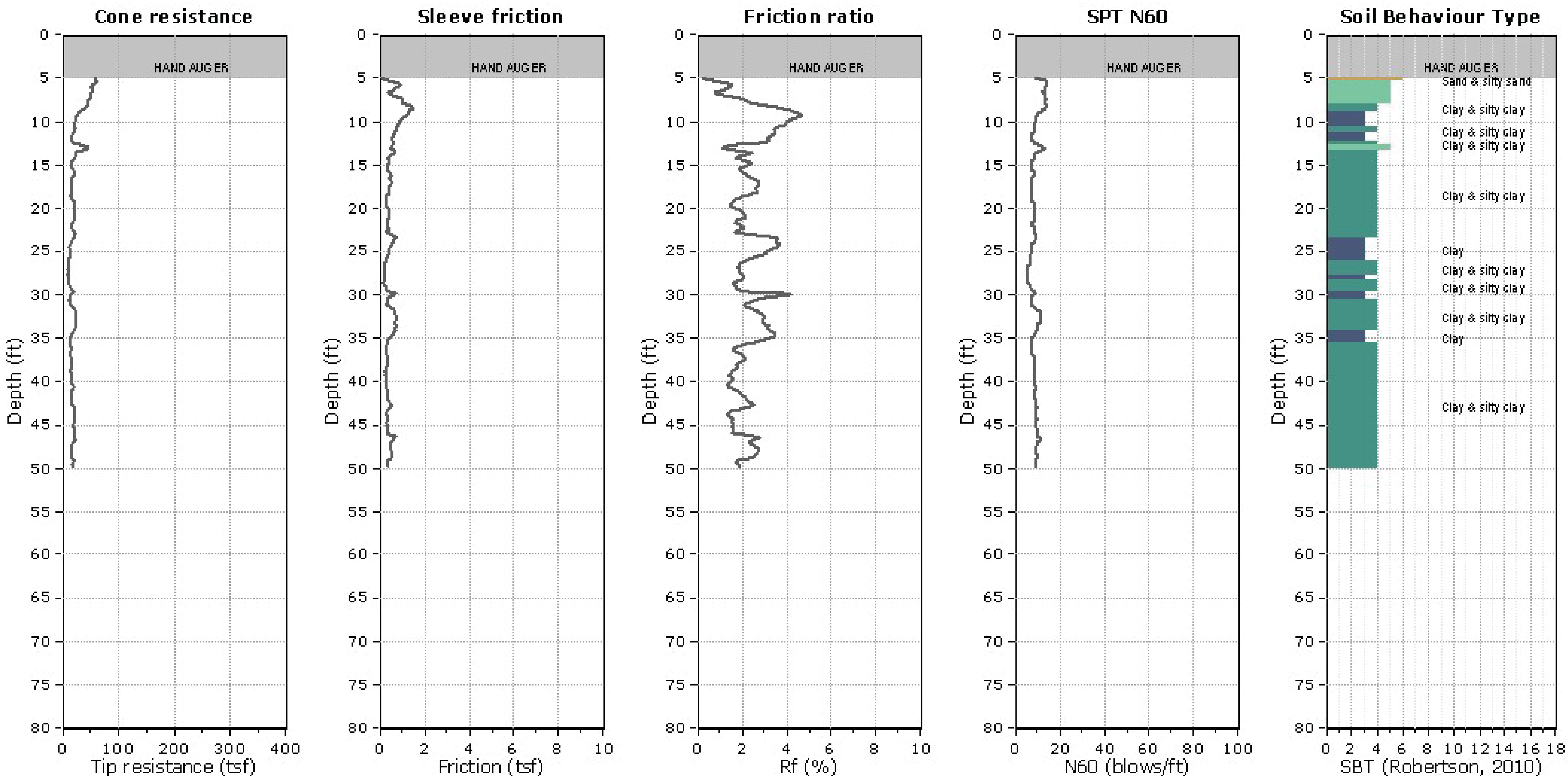
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: 1-CPT4

CLIENT: ENGEO
SITE: DEL HOMBRE SITE

Field Rep: TAWNY
Total depth: 49.87 ft, Date: 3/16/2018



SBTn legend

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

GREGG DRILLING, INC.
www.greggdrilling.com

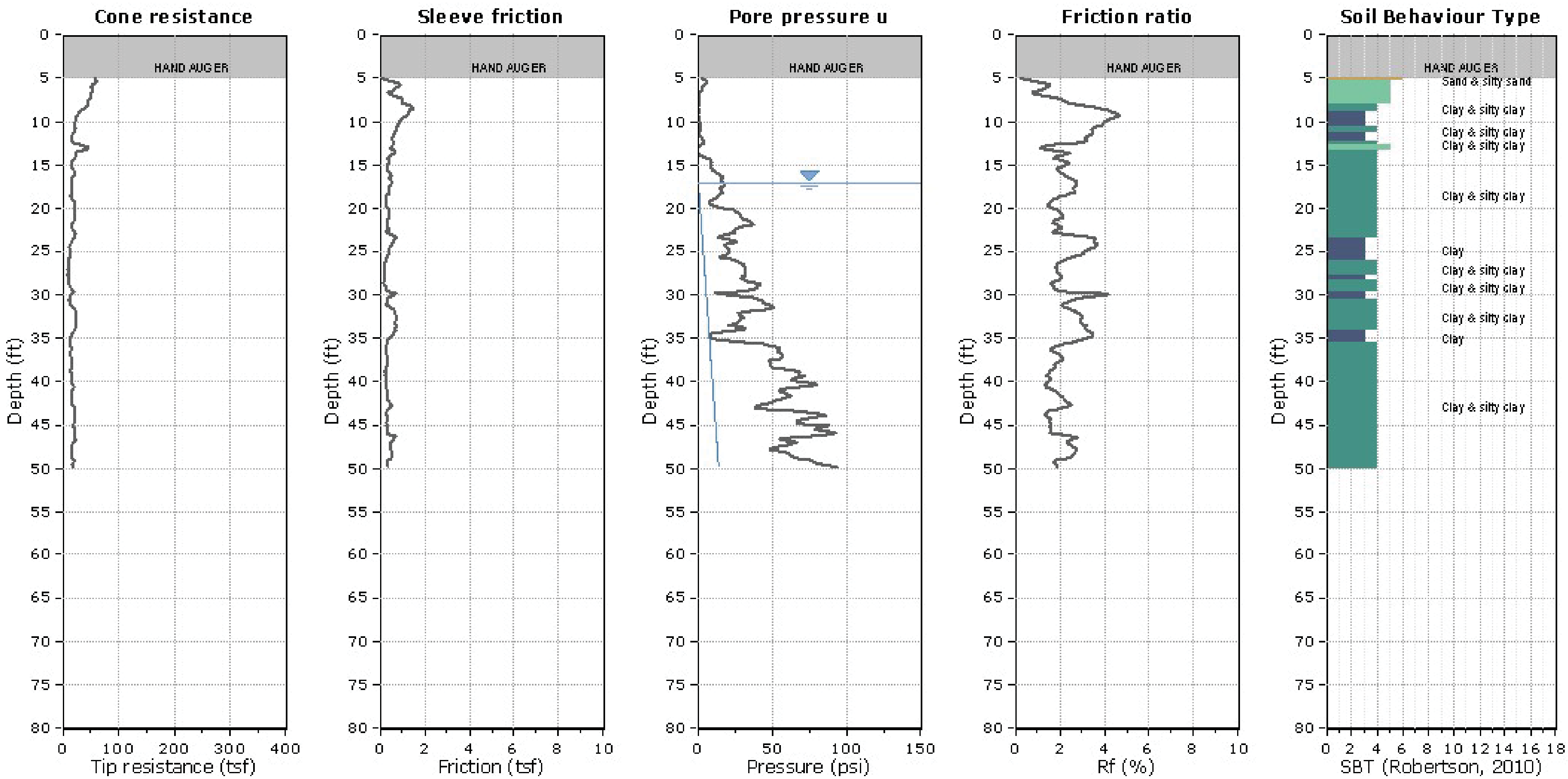
CPT: 1-CPT4

CLIENT: ENGEO

SITE: DEL HOMBRE SITE

Field Rep: TAWNY

Total depth: 49.87 ft, Date: 3/16/2018



SBTn legend

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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

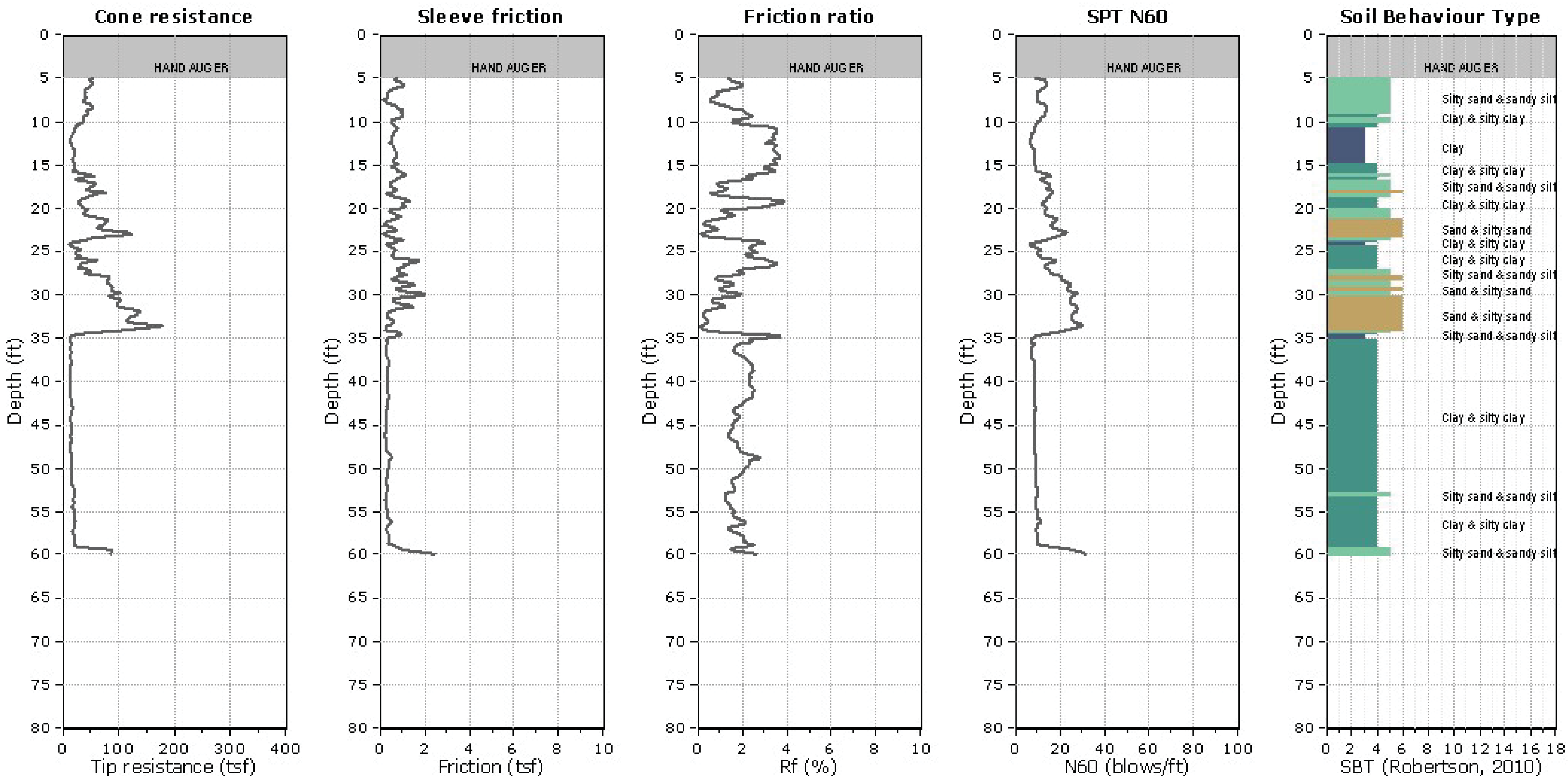
CPT: 1-CPT5

CLIENT: ENGEO

SITE: DEL HOMBRE SITE

Field Rep: TAWNY

Total depth: 59.88 ft, Date: 3/16/2018



SBTn legend

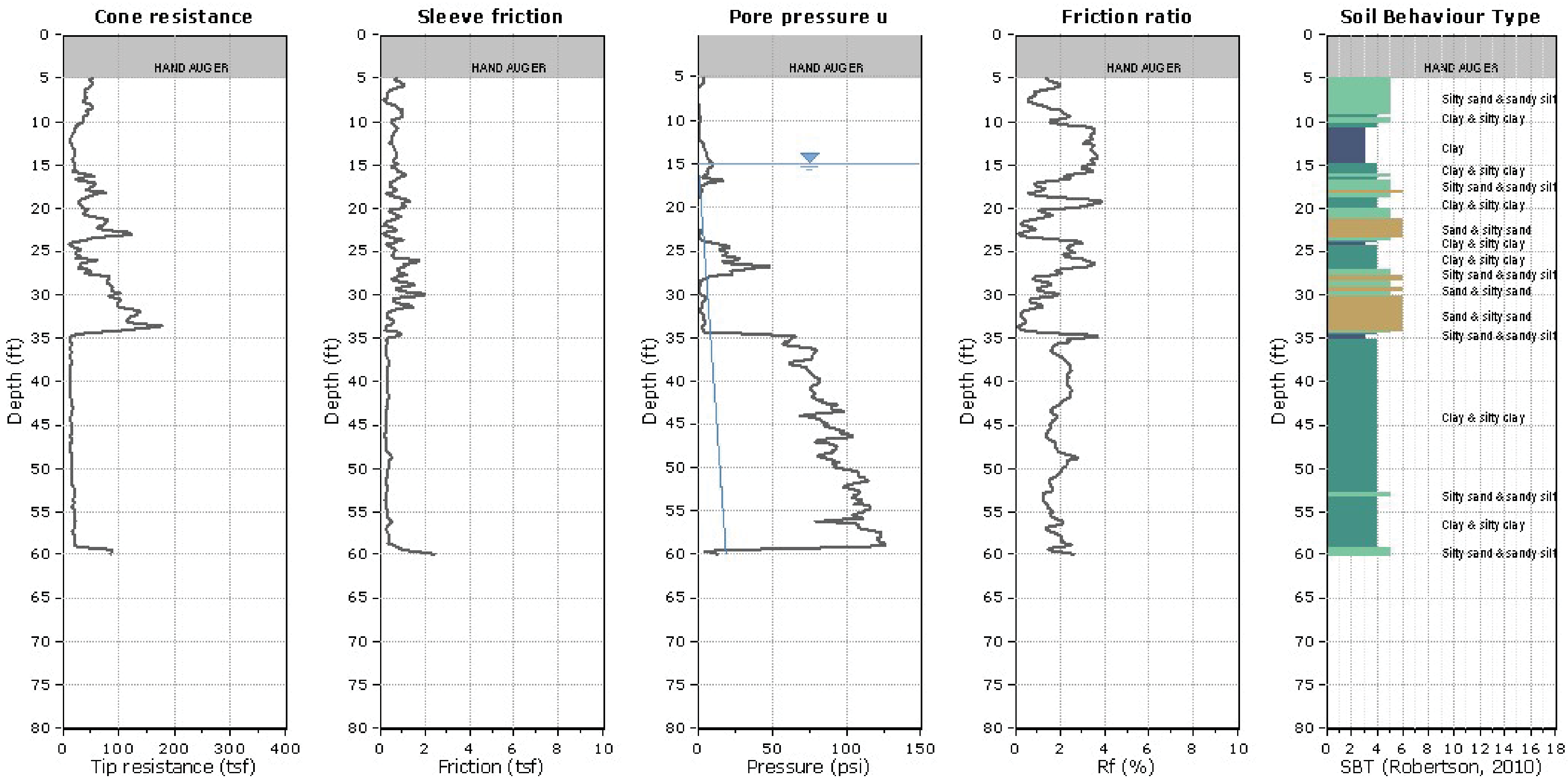
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: 1-CPT5

CLIENT: ENGEO
SITE: DEL HOMBRE SITE

Field Rep: TAWNY
Total depth: 59.88 ft, Date: 3/16/2018



SBTn legend

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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

APPENDIX B

LIQUEFACTION ANALYSIS

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

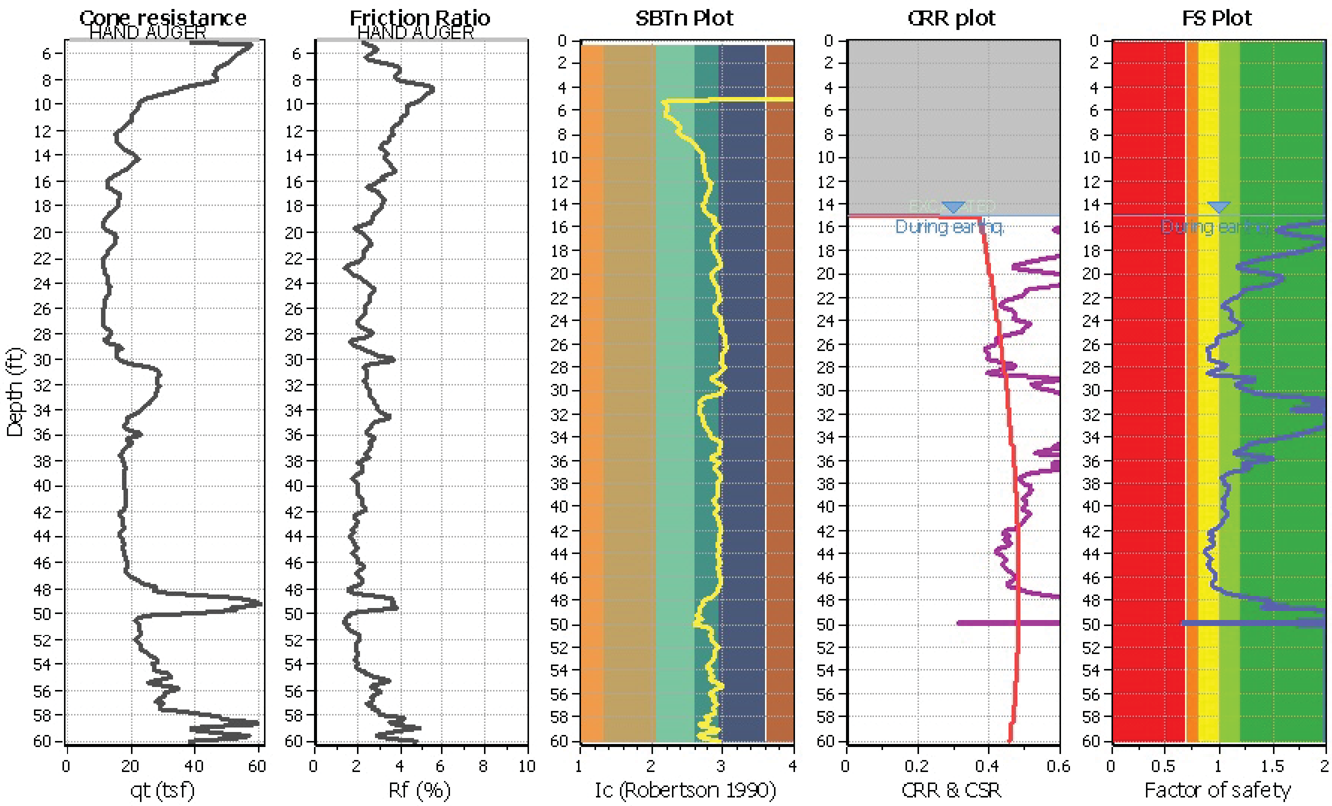
LIQUEFACTION ANALYSIS REPORT

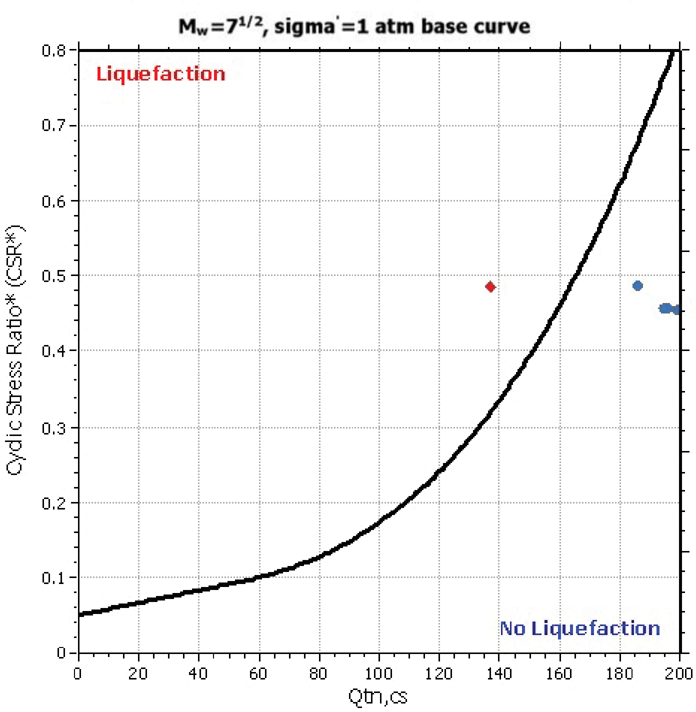
Project title : Del Hombre Site **Location : Walnut Creek**

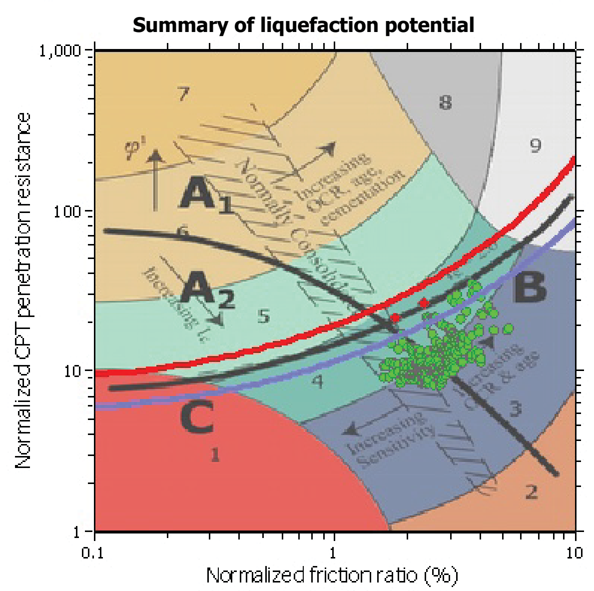
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	15.00 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	1.50 tsf	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_o applied:	No		

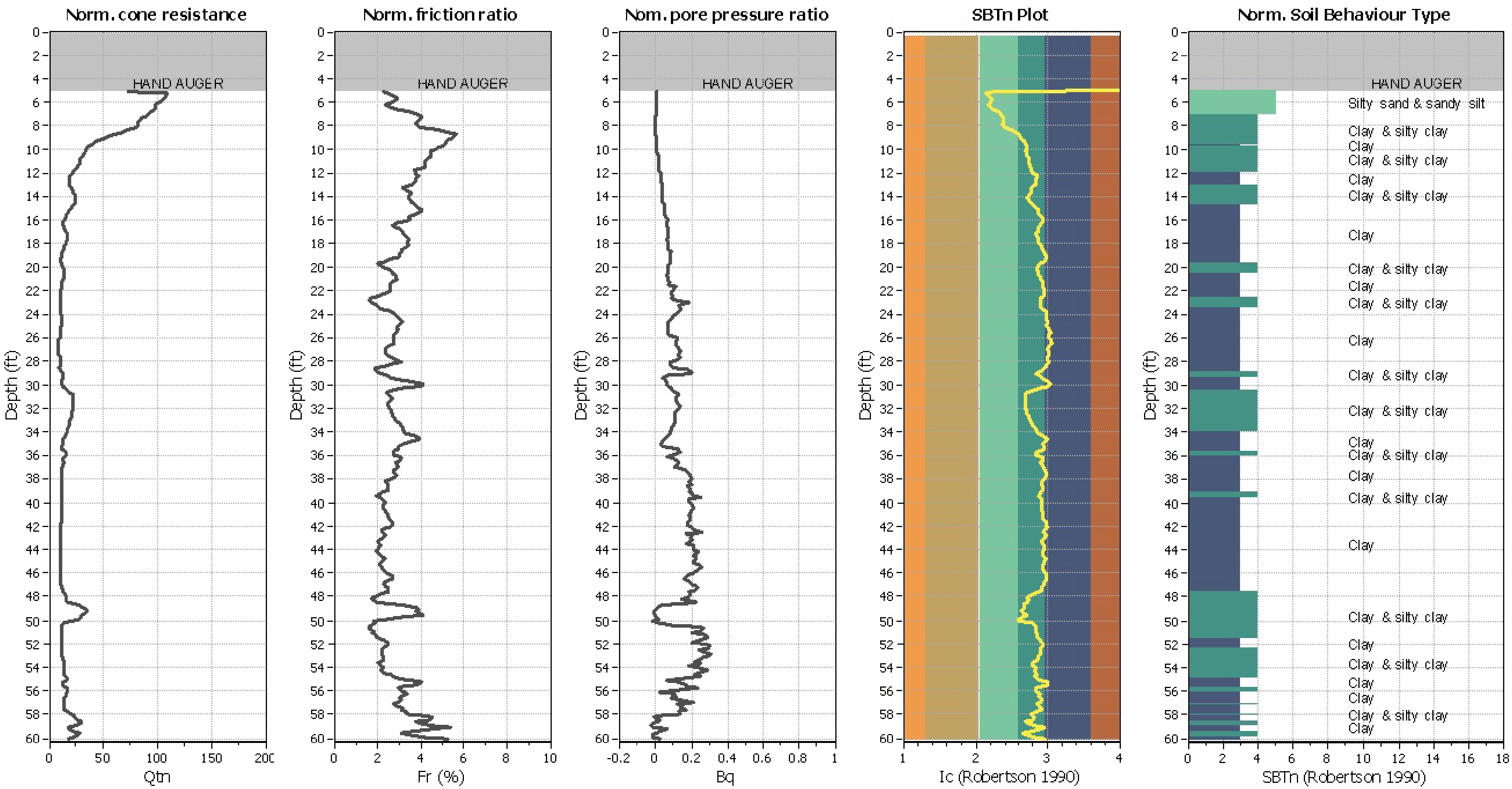






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normaliz



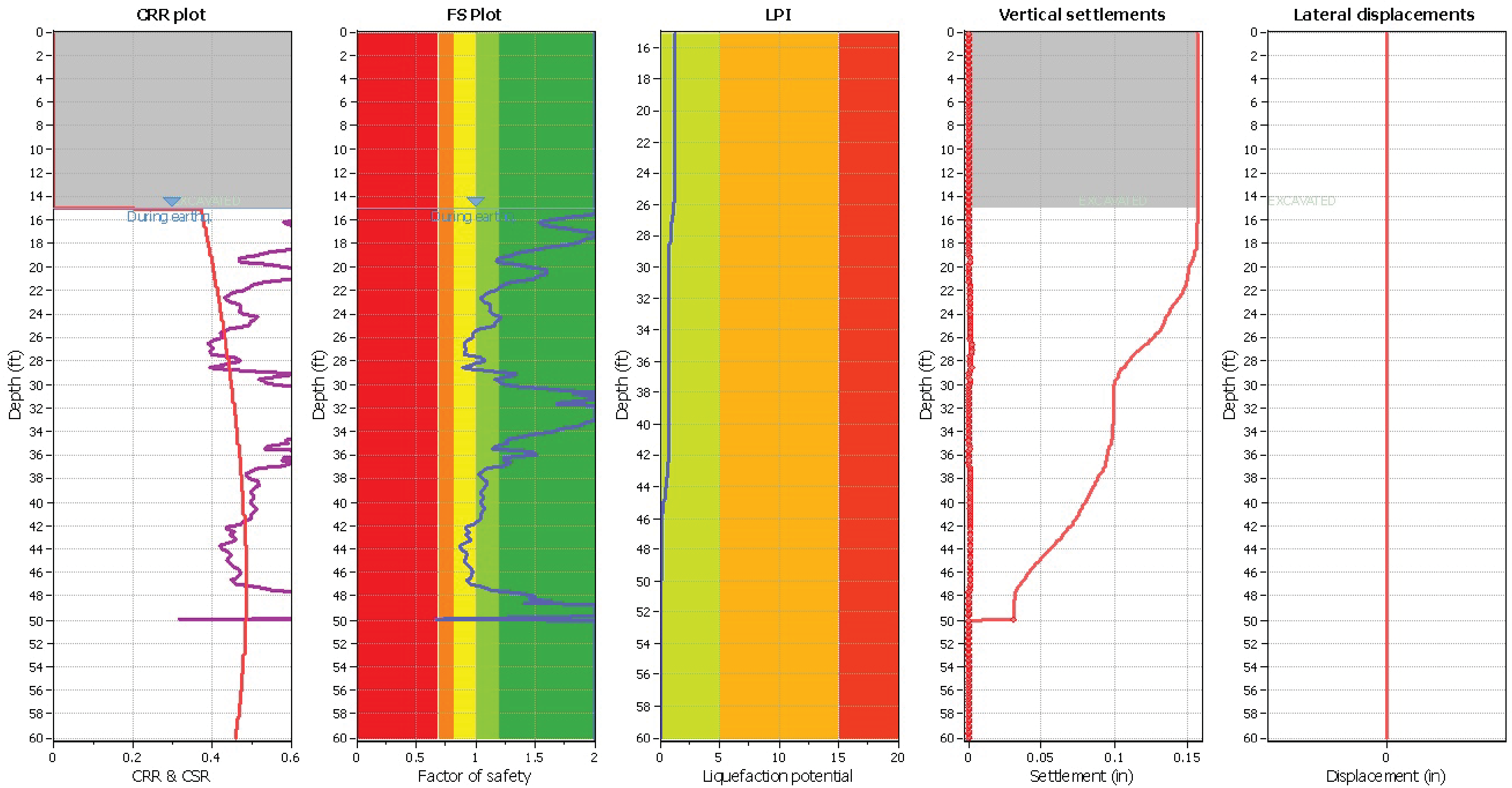
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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 plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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

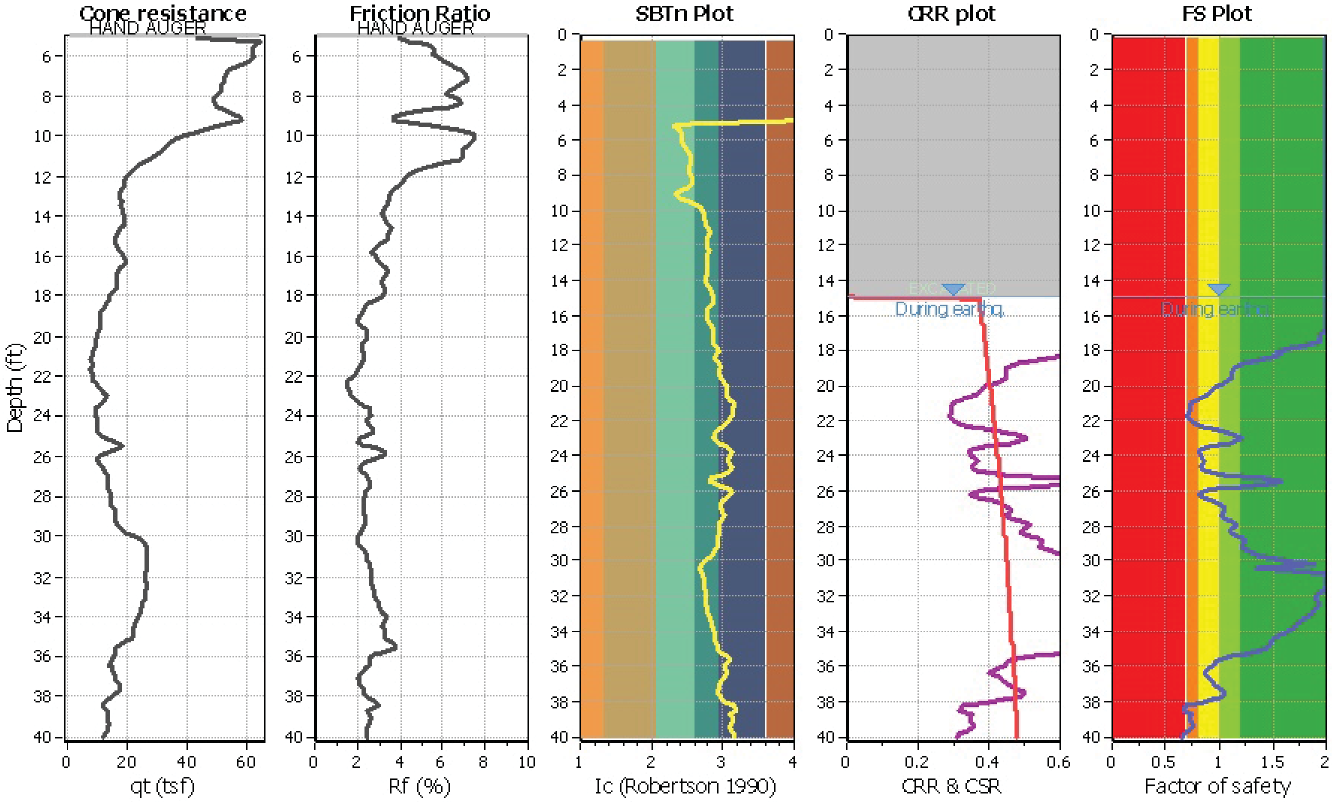
LIQUEFACTION ANALYSIS REPORT

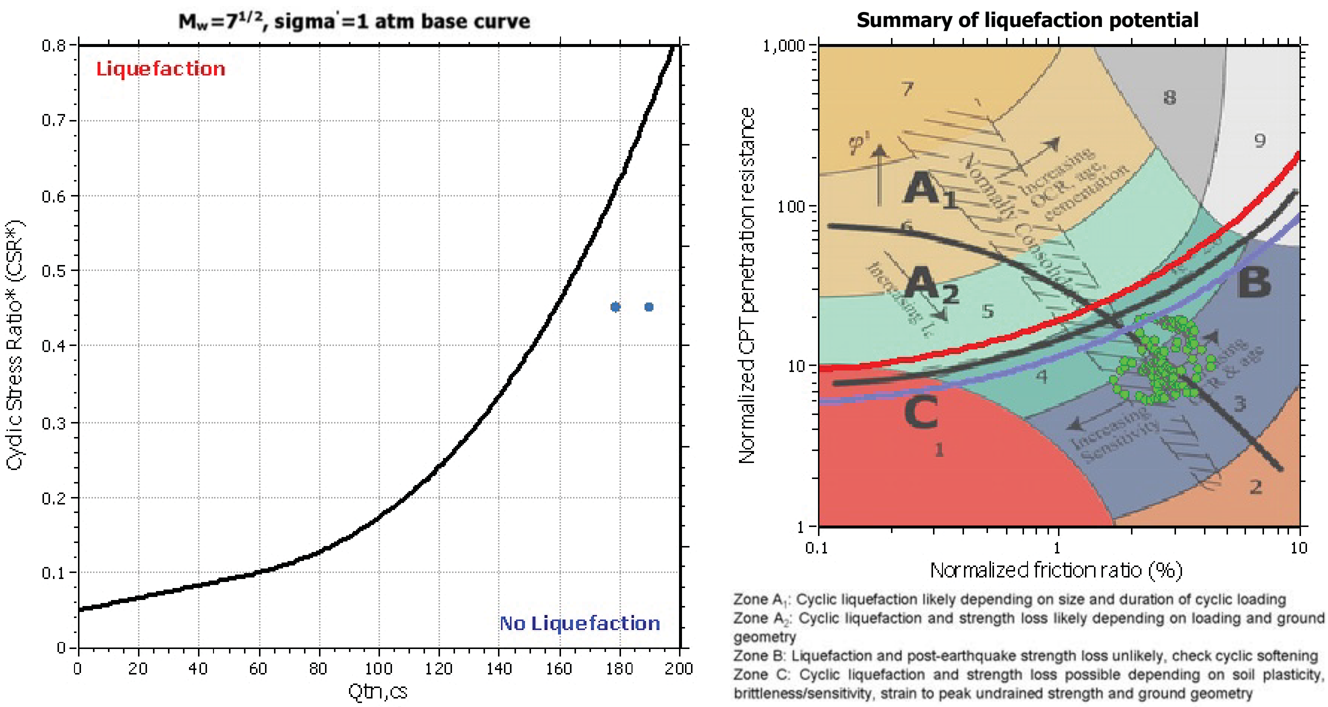
Project title : Del Hombre Site **Location : Walnut Creek**

CPT file : CPT-2

Input parameters and analysis data

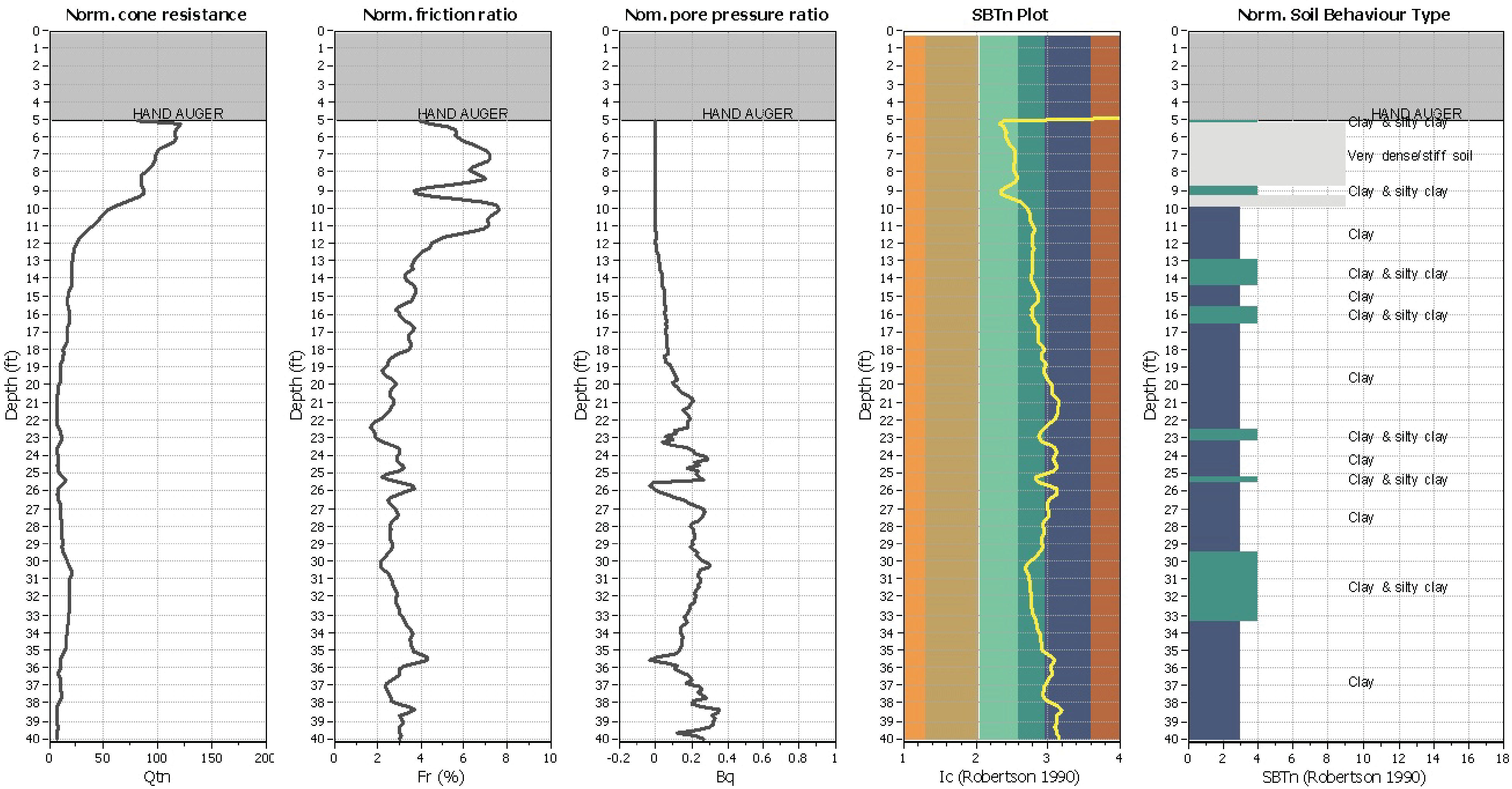
Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	15.00 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	1.50 tsf	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	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 (normaliz



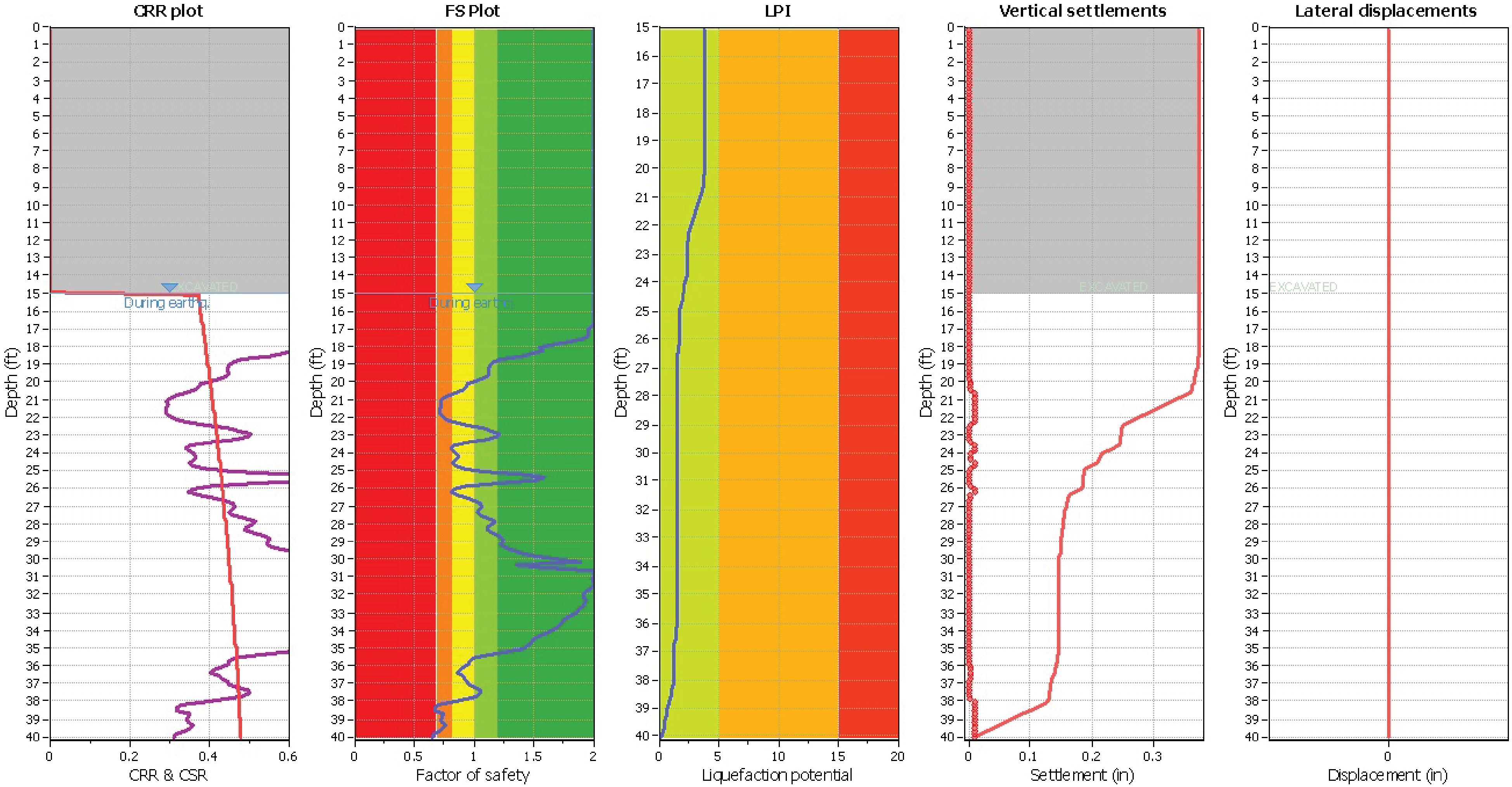
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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_o applied:	No
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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 plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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

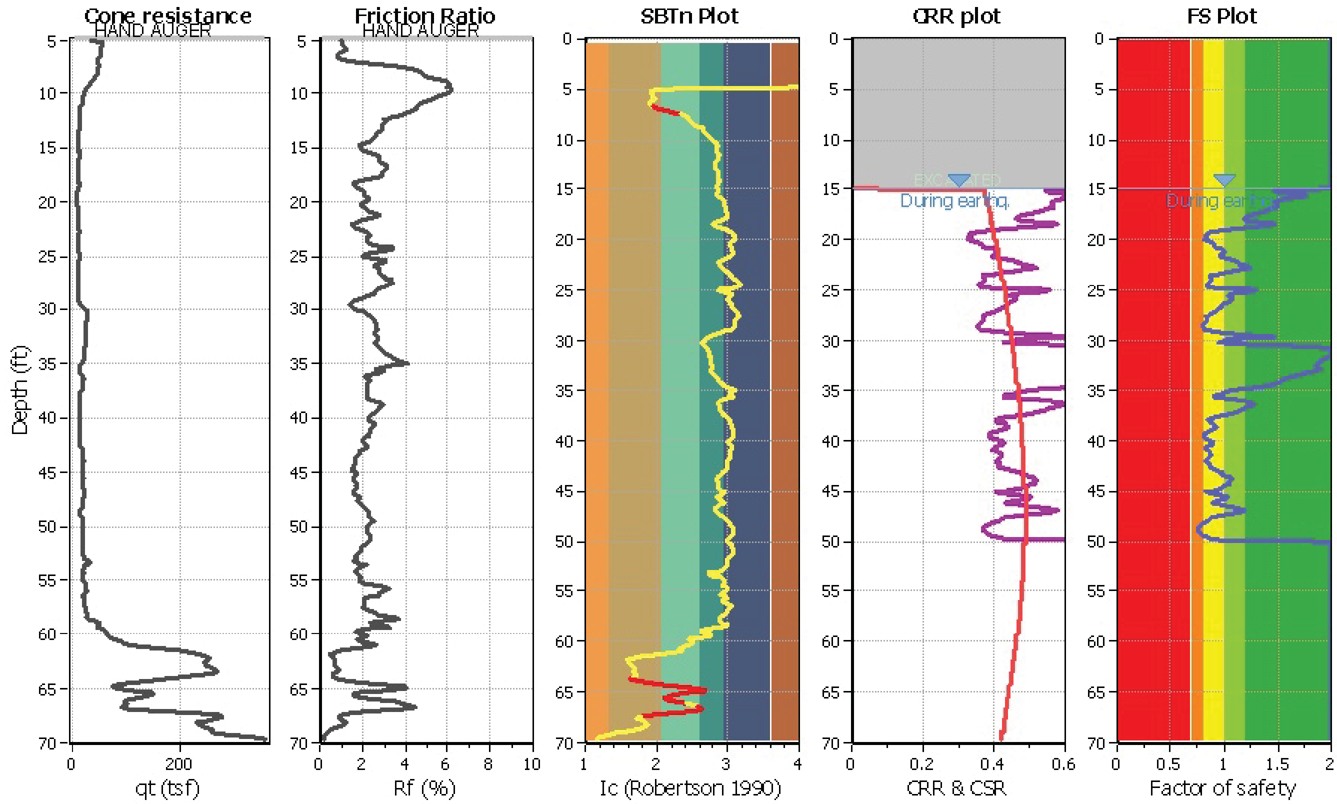
LIQUEFACTION ANALYSIS REPORT

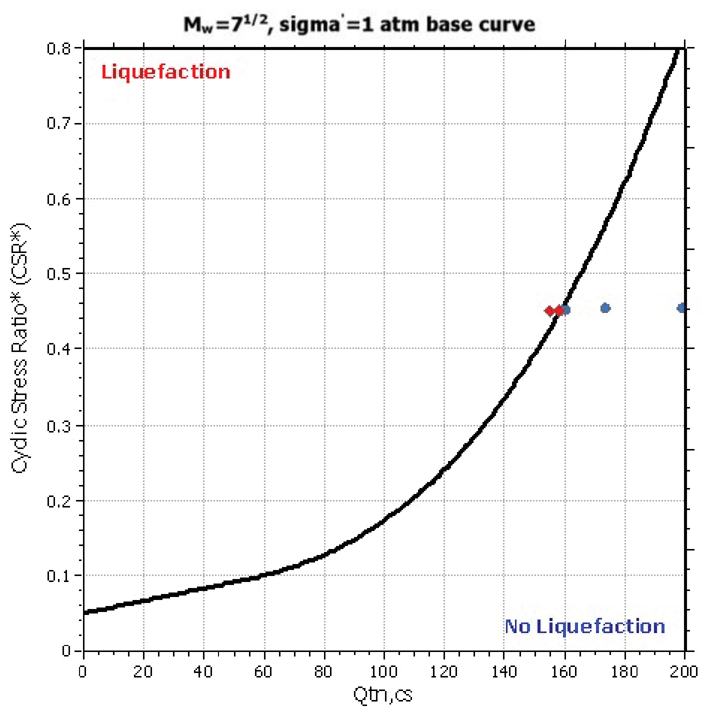
Project title : Del Hombre Site **Location : Walnut Creek**

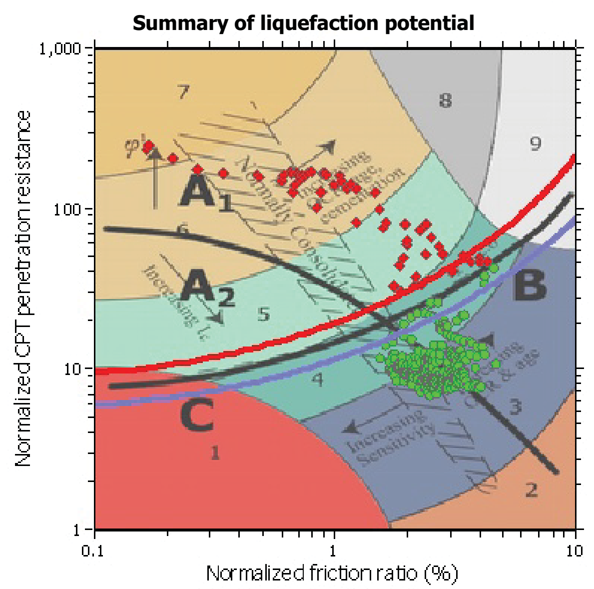
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	15.00 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	1.50 tsf	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	No		

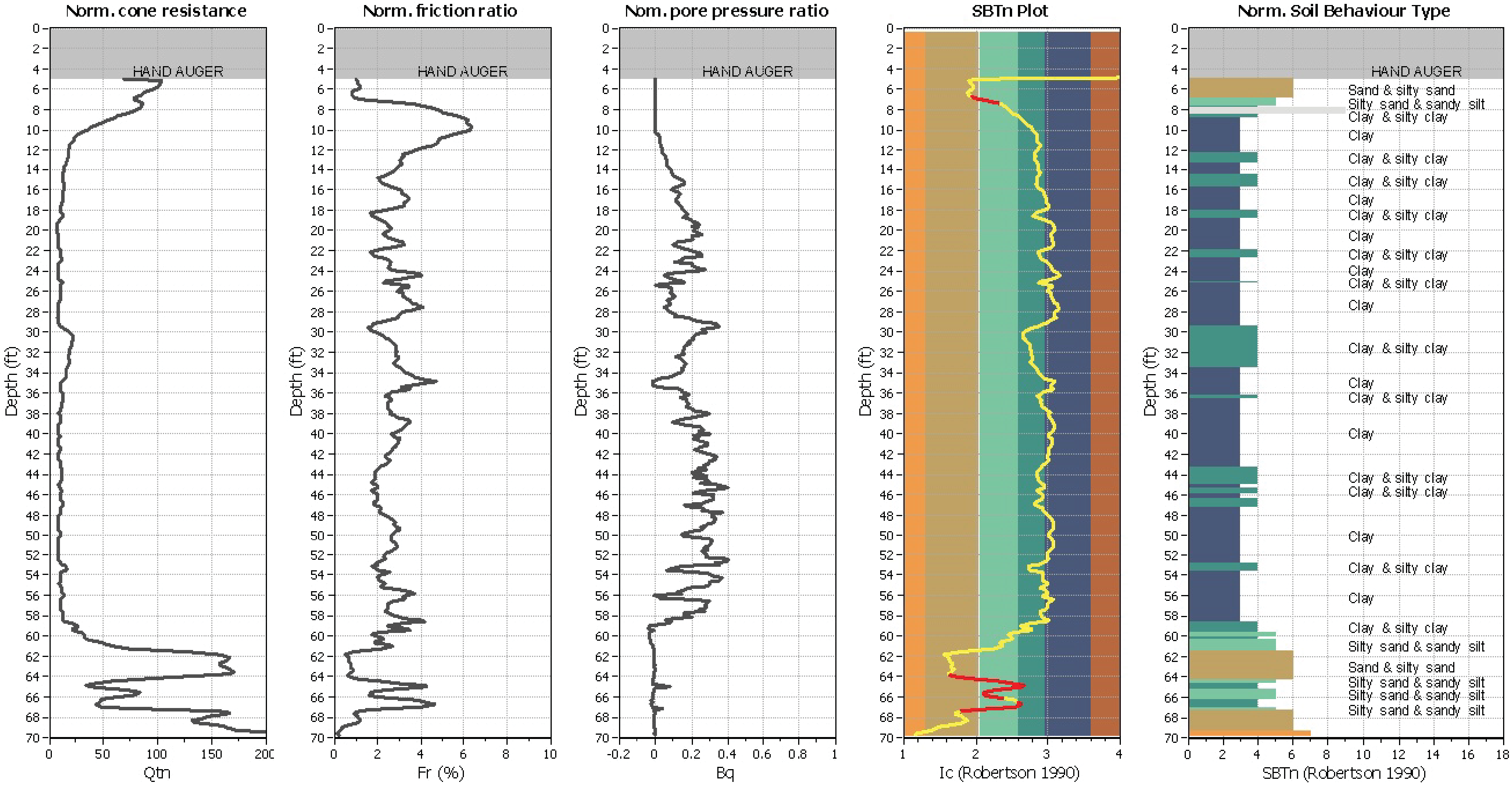






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normaliz



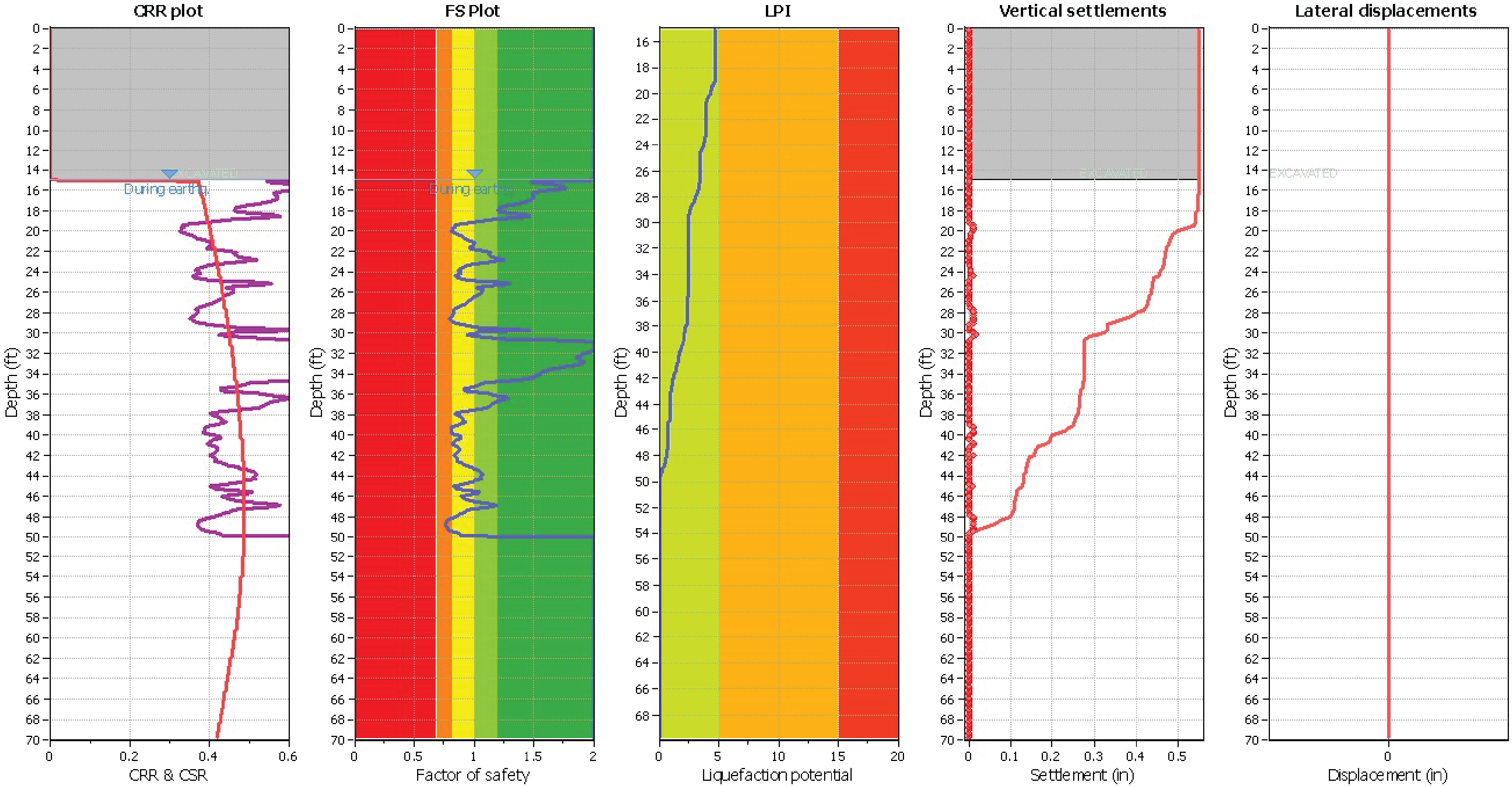
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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 plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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

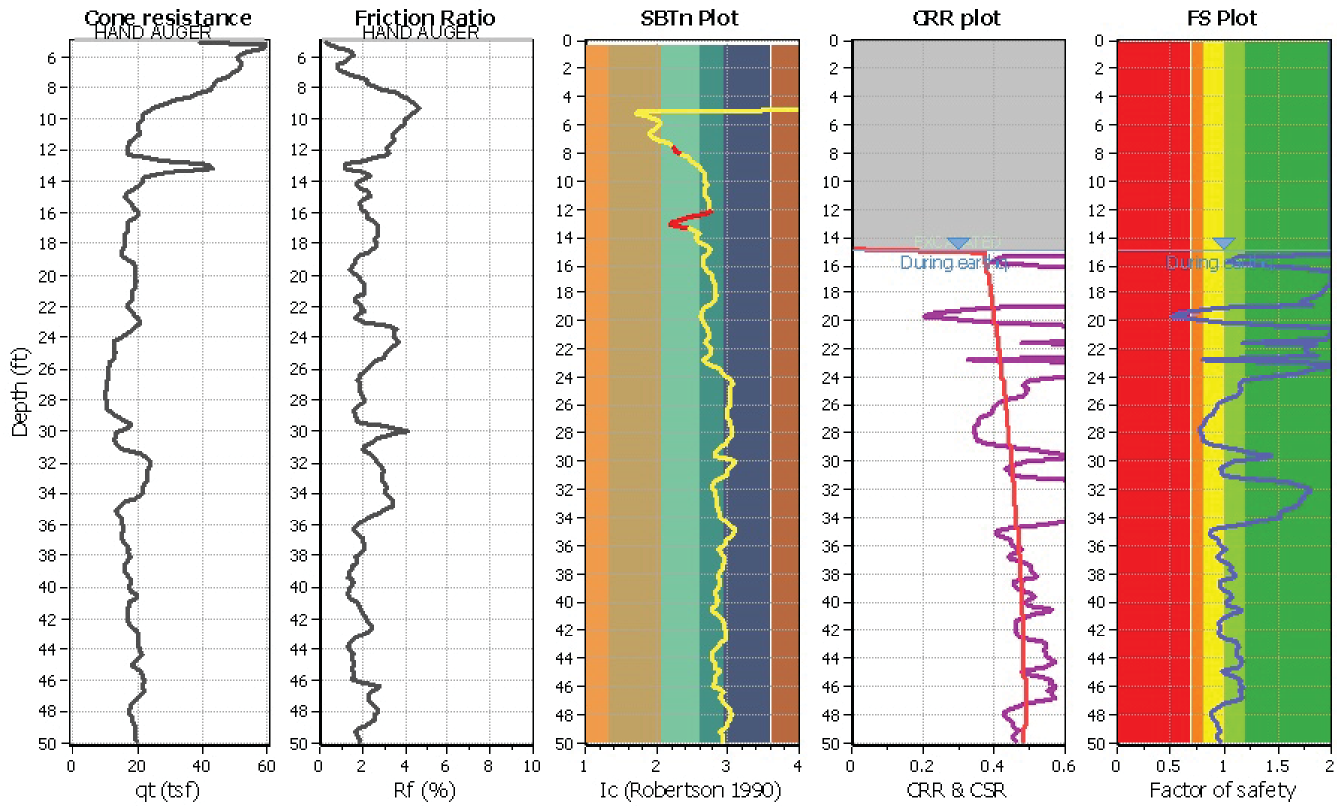
LIQUEFACTION ANALYSIS REPORT

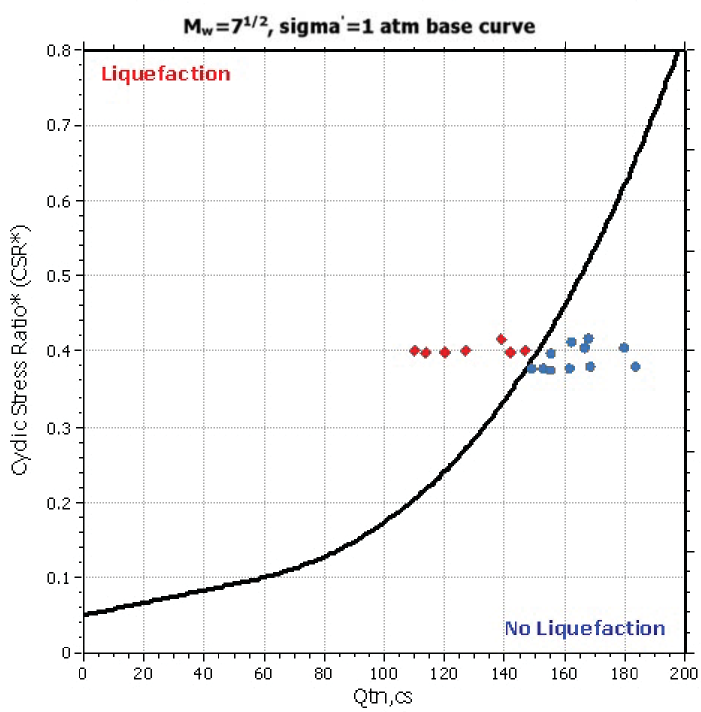
Project title : Del Hombre Site **Location : Walnut Creek**

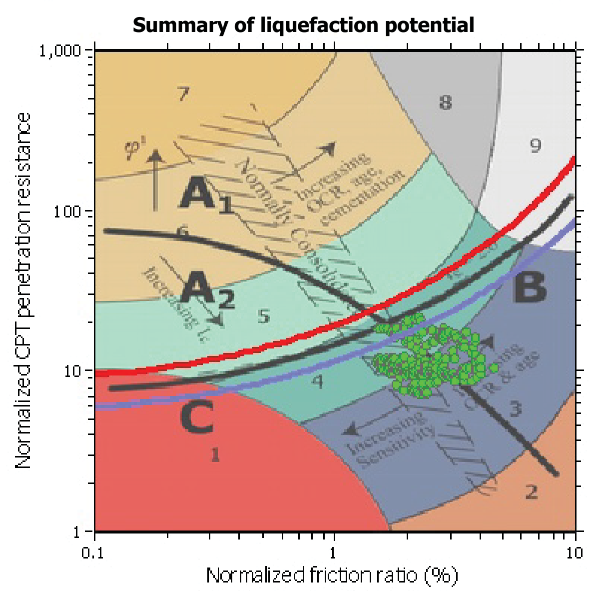
CPT file : CPT-4

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	15.00 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	1.50 tsf	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	No		

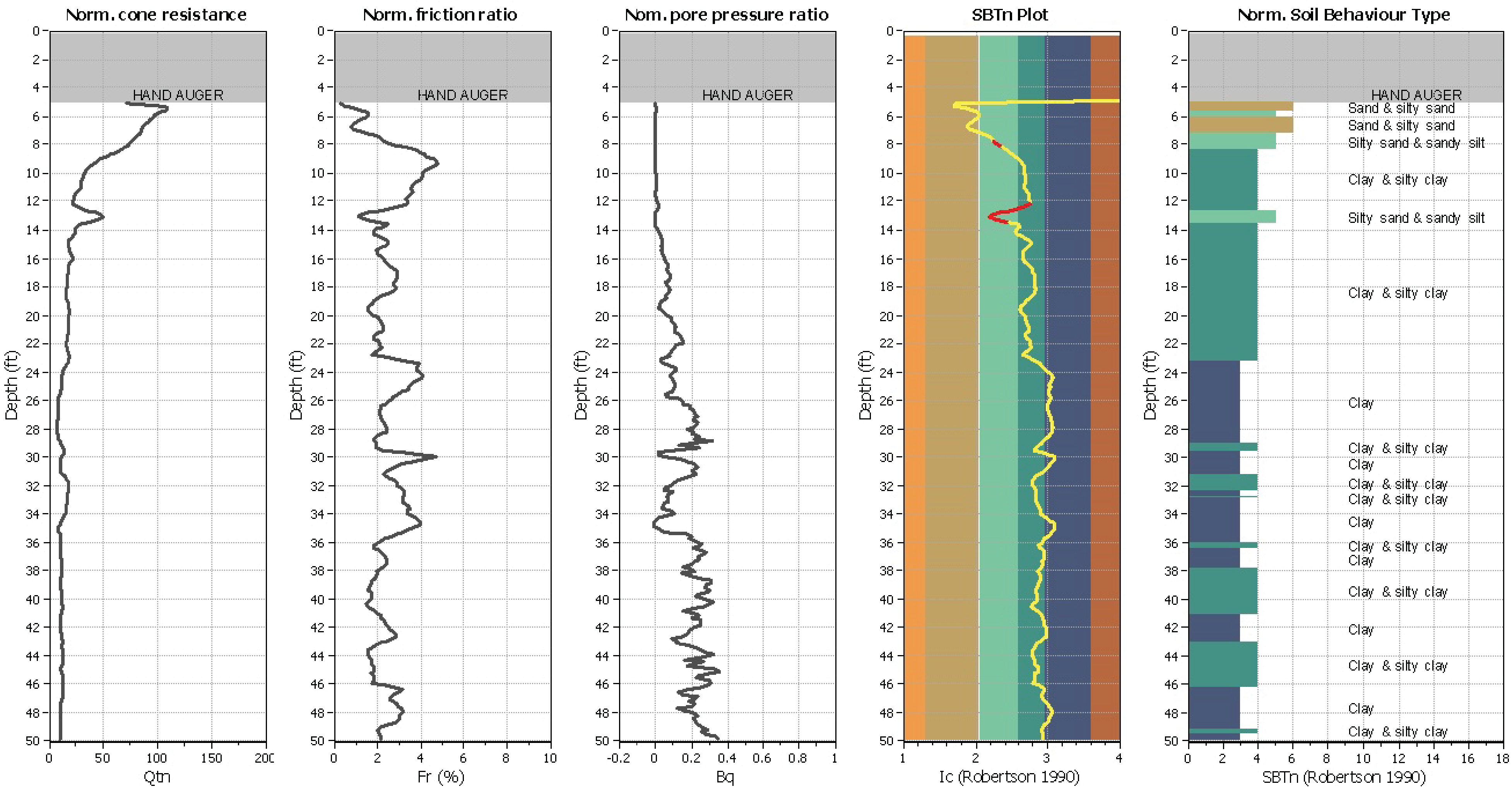






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normaliz



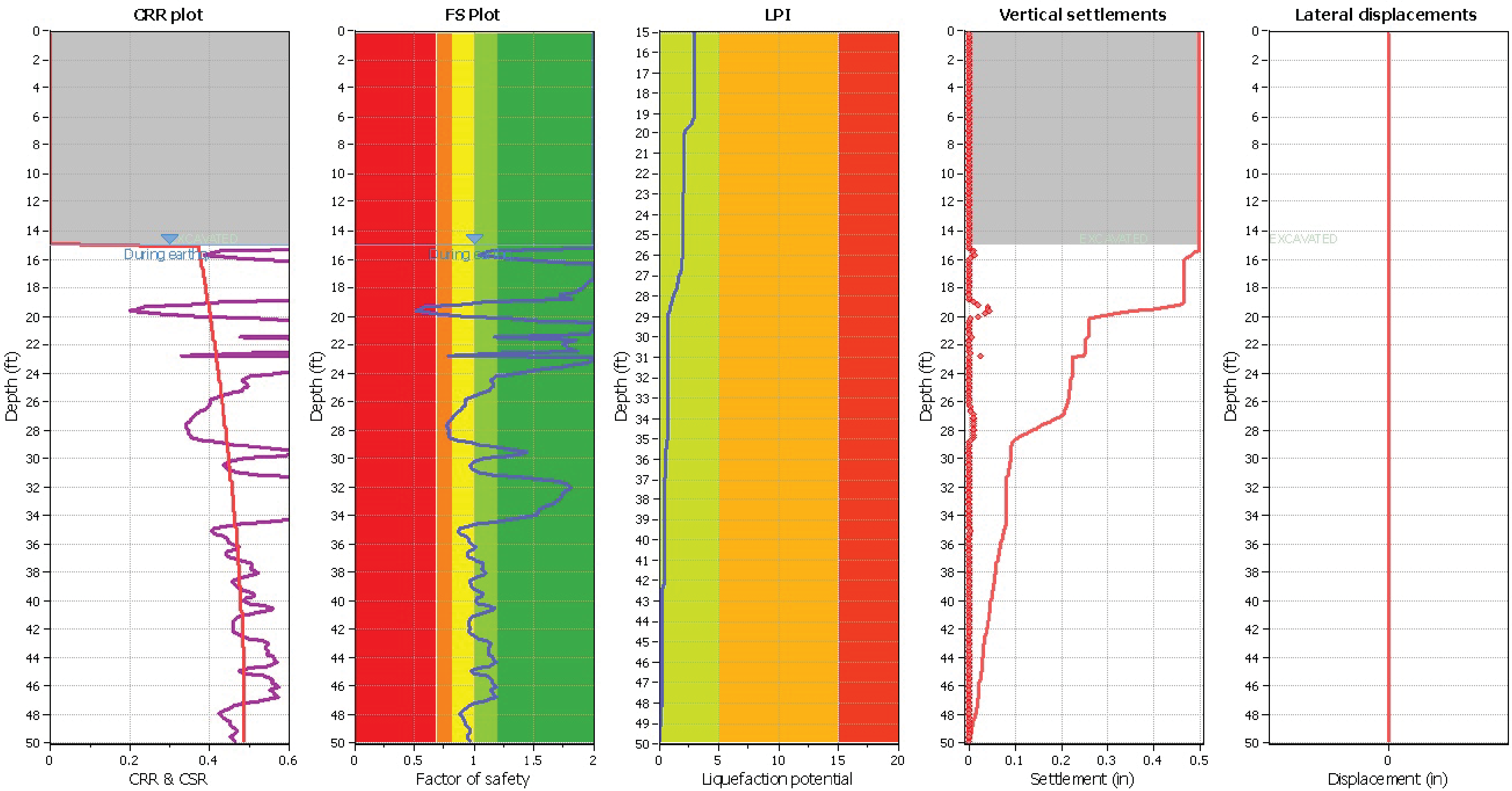
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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 plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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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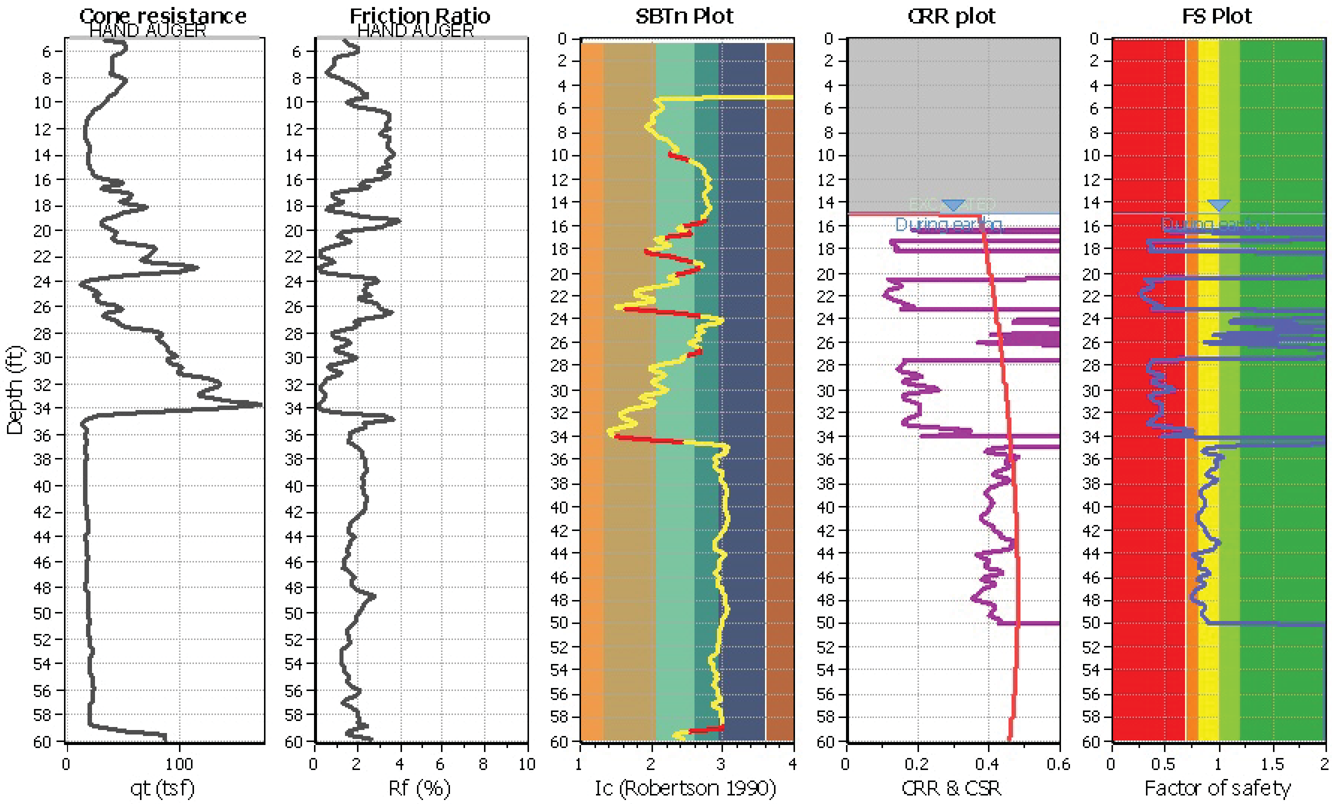
LIQUEFACTION ANALYSIS REPORT

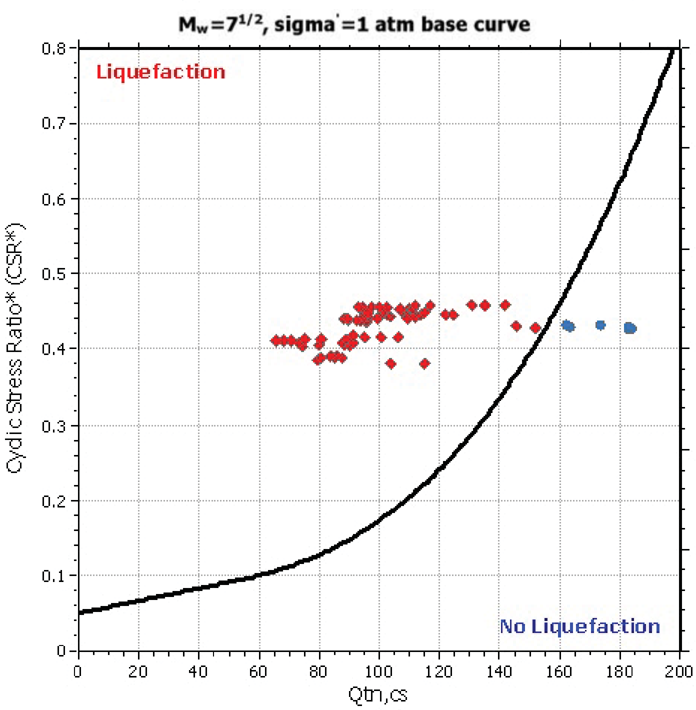
Project title : Del Hombre Site **Location : Walnut Creek**

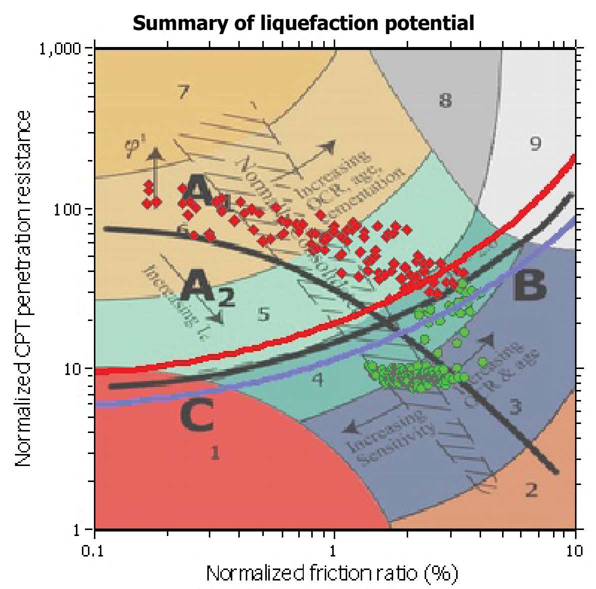
CPT file : CPT-5

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	15.00 ft	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	1.50 tsf	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_{σ} applied:	No		

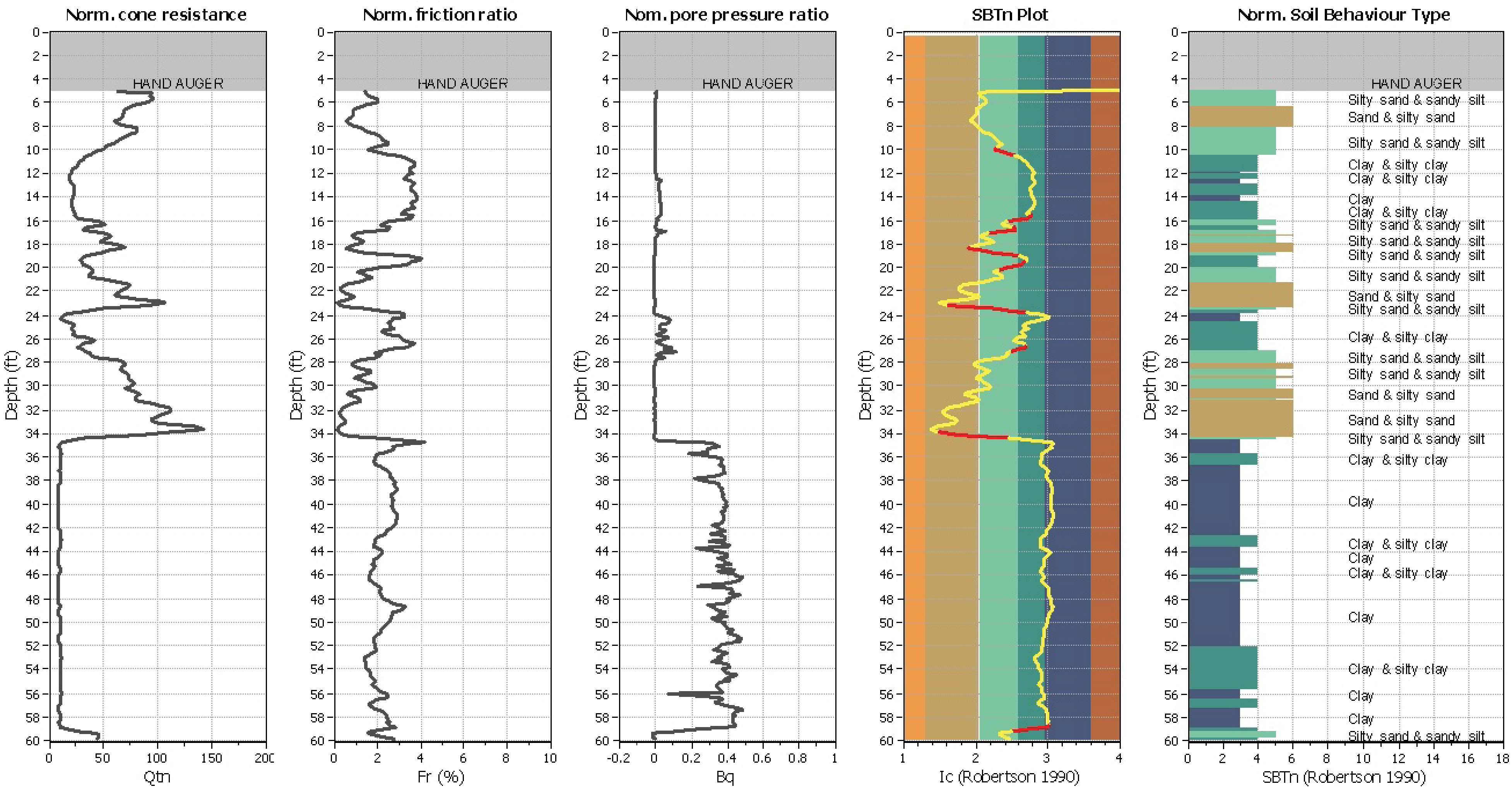






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normaliz



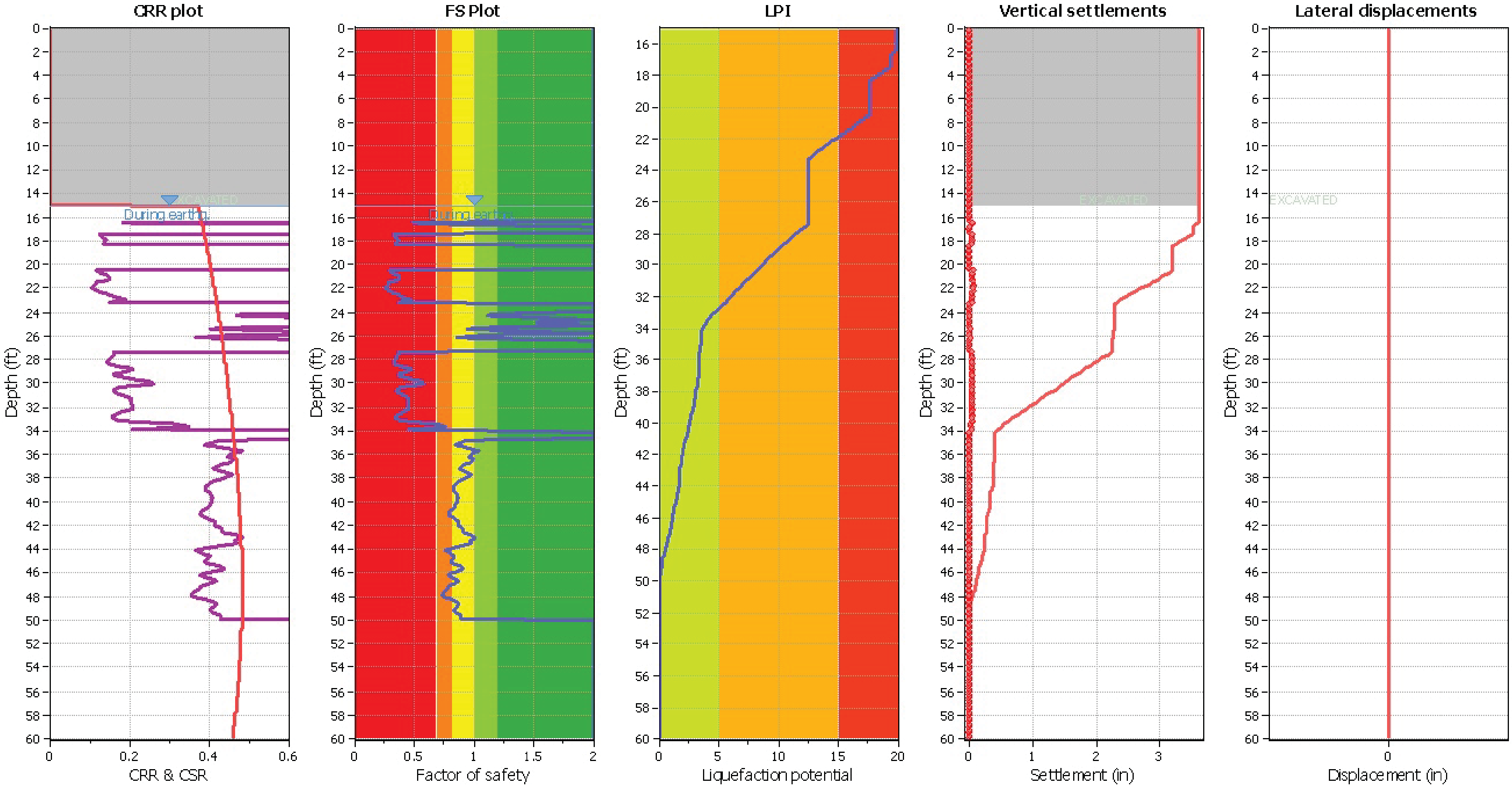
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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 plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	1.50 tsf
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.68	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	15.00 ft	Excavation depth:	15.00 ft	Limit depth:	50.00 ft

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

LATHROP

RENO

ROCKLIN

SANTA CLARITA

IRVINE

CHRISTCHURCH

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