# *Appendix 4.10-1:*

# Preliminary Geotechnical Exploration, Greentree, Solano County, California

# GREENTREE
SOLANO COUNTY, CALIFORNIA

# PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. William M. Messana
Greentree Development Group, Inc.
2301 Napa Valley Highway
Napa, CA 94558

PREPARED BY
ENGEO Incorporated

June 6, 2019

PROJECT NO.
16018.000.000



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Project No.
**16018.000.000**

June 6, 2019

Mr. William M. Messana
Greentree Development Group, Inc.
2301 Napa Valley Highway
Napa, CA 94558

Subject: Greentree
Solano County, California

**PRELIMINARY GEOTECHNICAL EXPLORATION**

Dear Mr. Messana:

This report summarizes geotechnical and geological conditions and constraints at the site and provides preliminary geotechnical recommendations for conceptual planning of Greentree project located in Solano County, California. The preliminary conclusions and recommendations of this report are based on geotechnical and geologic studies completed to date.

Based on the results of this study, we identified the following geotechnical and geologic considerations to incorporate in the project planning:

- Disturbed near-surface soil and existing undocumented fill.
- Expansive soil.
- Compressible soil.
- Seismically induced settlement of potentially liquefiable soil.
- Shallow groundwater.

It is our opinion that the proposed Greentree project is feasible from a geotechnical standpoint provided the recommendations by ENGEO, as summarized in this document, are incorporated into project planning.

We trust that this document provides geotechnical guidance appropriate for the current planning process. Please contact us if you have any questions regarding this document.

Sincerely,

ENGEO Incorporated

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# 1.0 INTRODUCTION

## 1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical exploration, as described in our proposal dated March 24, 2019, is to provide an assessment of the potential geotechnical and geologic concerns associated with the use of the site for residential neighborhoods, commercial development, open space, and associated improvements. The scope of our services included a site visit, review of published geologic maps and readily available geotechnical reports for the site, advancing eight cone penetration tests (CPTs) to a depth of up to approximately 50 feet below existing grade, observation and logging of eight shallow test pits, and preparation of this report to summarize our findings and discuss potential geotechnical and geological hazards.

This report was prepared for your exclusive use and your 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 determine whether modifications are necessary. 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 project is approximately 164 acres in area and is identified as Assessor's Parcel Numbers (APN) 134-020-380, 134-480-110, 133-120-190, 134-020-360, 134-020-240, 134-020-450, 134-020-180, 134-020-460, and 133-120-340. Two known addresses are associated with the site: 640 and 999 Leisure Town Road. The project consists of a former golf course. It is located along the east edge of the Vacaville city limits, just south of Interstate 80 (Figure 1). Orange Drive and Leisure Town Road border the northwest and east edges of the site, respectively, while the remaining perimeter is bordered by existing light-commercial and residential structures. The northernmost tip of the site is bordered by a channelized waterway, Horse Creek. A small natural stream, Ulatis Creek, borders the southernmost portion of the project site. Based on a review of available topographic maps, the existing topography is relatively level, decreasing from approximately elevation 95 feet in the southwest section to approximately elevation 80 feet in the northeast section (WGS84). The site is bisected by Sequoia Drive.

## 1.3 PROJECT DESCRIPTION

Based on the Conceptual Site Plan, dated April 19, 2019, prepared by CBG Civil Engineers, and the undated Greentree Concept Plan/General Plan Amendment and Zoning document you provided, the subject site will be comprised of two major subdivisions: one north of Sequoia Drive and one south of Sequoia Drive. The northern portion will consist of a mixed-used development with high-density residential blocks, commercial blocks, and a large park. The southern portion will consist of a single-family home residential community and park space. In total, we understand the site is being planned for between approximately 1,100 and 1,500 residential units, 500,000 square feet of commercial space, and 32 acres of park and trail space. In addition to the above-mentioned improvements, we anticipate the development will include minor ancillary structures, street and sidewalk paving, underground utilities, retaining structures, stormwater detention basins, and landscaping. Conceptual grading plans were not available for our review, but we anticipate minor cuts and fills to accommodate practical foundation construction, accessible roadways and sidewalks, and possible flood mitigation.

# 2.0 SITE GEOLOGY AND SEISMICITY

## 2.1 REGIONAL GEOLOGY

### 2.1.1 Regional Geology

The Greentree site is located within the Coast Ranges physiographic province of California. The Coast Ranges physiographic province is typified by a system of northwest-trending, fault-bounded mountain ranges and intervening alluvial 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.

### 2.1.2 Site Geology

More specifically, the site is located in the valley approximately 3 miles east of the English Hills. According to mapping by Graymer (2002) (Figure 3), most of the proposed development area is underlain by Holocene-aged alluvial fan deposits (Qhf), with the southernmost section of the site mapped as underlain by Holocene-aged natural levee deposits (Qhl). Our experience in the area suggests the surficial Qhf and Qhl may be underlain by Holocene-aged basin deposits (Qhb) and Delta mud deposits (Qhdm). These soils are generally characterized as high plasticity, normally consolidated to slightly over-consolidated clay or mud. The site is not currently mapped within a California Geologic Survey (CGS) Seismic Hazard Zone for liquefaction, landslides, or surface ruptures.

## 2.2 REGIONAL SEISMICITY

The San Francisco Bay Area contains numerous active faults. Figure 4 shows the approximate location of active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region. An active fault is defined by the State as one that has had surface displacement within Holocene time, about the last 11,000 years (Bryant and Hart, 1997). Based on the 2008 USGS National Seismic Hazard Maps, the nearest active fault is the Great Valley fault, which is a blind thrust fault with areal limits overlapping with the project site. This fault is considered capable of a moment magnitude earthquake of 6.8. However, a blind thrust fault poses a low risk of surface fault rupture. Other active faults located near the site include the Hayward fault, located approximately 33.5 miles west of the site, considered capable of a moment magnitude earthquake of 7.3; the West Napa fault, located approximately 21.4 miles west of the site, considered capable of a moment magnitude earthquake of 6.7; and the Green Valley fault, located approximately 12.9 miles west of the site, considered capable of a moment magnitude earthquake of 6.6.

## 2.3 SITE CONDITIONS

The site was previously used as a golf course. The site is currently densely vegetated on the north section of the site, and the areas immediately south of Sequoia Drive is sparsely vegetated due to possibly cuts and ripping of disturbed native soil. A channel runs east of the site along the boundary with Leisure Town Road. During our site visits on May 1 and 2, 2019, we noted minor landscape ponds and dirt mounds, as well as gravel and dirt roads, scattered throughout the site. Various fences also exist on site. The southern portion of the site appears to have been previously used as a disposal area for tree cuttings and landscaping debris. One large clubhouse structure, a storage structure, and large concrete parking lot currently exist on the eastern edge of the site along Leisure Town Road.

## 2.4 FIELD EXPLORATION

We performed field exploration on May 1 and 2, 2019. The exploration included advancing eight cone penetration tests (CPTs) at various accessible locations at the site. Additionally, we observed and logged eight shallow test pits.

We retained a CPT truck rig to push the cone penetrometer to a maximum depth of approximately 50 feet below ground surface (bgs). The CPT has a 25-ton compression-type cone with a 15-square-centimeter ($cm^2$) base area, an apex angle of 60 degrees, and a friction sleeve with a surface area of 225 $cm^2$. The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D5778. Measurements include the tip resistance to penetration of the cone (qt), the resistance of the surface sleeve (fs), and pore pressure (u) (Robertson and Campanella, 1988). CPT logs are presented in Appendix A.

For the test pits, we employed a track-mounted Yanmar mini-excavator with a 36-inch bucket. We logged the test pits in the field. The test pits were excavated to depths between about 4 and 7 feet bgs. The exploratory test pits were backfilled with the excavated soil with nominal compactive effort and is considered non-engineered fill.

The exploration locations are shown in Figure 2. The location of our explorations are approximately located and were estimated using consumer-grade global positioning system (GPS) and their proximity to existing site features; therefore, the locations shown should be considered accurate only to the degree implied by the method used.

## 2.5 LIMITED LABORATORY TESTING

We retrieved bulk samples of near-surface material at select test pit locations. These samples were submitted to our laboratory for testing. We performed plasticity index tests on these samples. Laboratory test results are included in Appendix C.

## 2.6 SUBSURFACE CONDITIONS

Fill, approximately ½ to 3¼ feet thick, was encountered in all of our test pits except Test Pit T-7. The fill materials typically consisted of clay. In some locations, the fill material consisted of layers of sand approximately 1 to 2 feet thick followed by a layer of pea gravel approximately 4 to 6 inches thick. These areas were likely previously used as a sand bunker in the golf course. The native soils encountered consisted of fat clay to lean clay. According to the laboratory test results, the near-surface soils have Plasticity Indices ranging from 6 to 42. This is an indication that these surficial soils have a moderate to high expansion potential. Below the fill and disturbed soil, the CPT data indicated the site was predominantly underlain by fine-grained soil materials with the occasional dense sand layers between around 12 and 30 feet bgs. Sandy layers were not encountered in 1-CPT5 and 1-CPT8 at the northernmost area of the site.

The CPT logs and test pit logs include the specific subsurface conditions at the location of the probes. We include our CPT and test pit logs in Appendices A and B, respectively.

### 2.7 GROUNDWATER CONDITIONS

Groundwater was encountered during excavation of Test Pit T-7 at a depth of approximately 7 feet below ground surface. We performed pore pressure dissipation tests as part of several of the CPT explorations, which indicated groundwater is approximately 10 feet bgs. Similarly, review of literature shows groundwater varying between 7 and 15 feet below ground surface.

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

## 3.0 DISCUSSION AND PRELIMINARY CONCLUSIONS

Based upon this preliminary study, it is our opinion that the project site is feasible for the proposed mixed-used developments from a geotechnical standpoint provided that the preliminary recommendations contained in this report and future design-level geotechnical studies are incorporated into the development plans. A more comprehensive site-specific geotechnical exploration should be performed as part of the design process. The exploration would include borings and laboratory soil testing to provide data for preparation of specific recommendations regarding grading and foundation design 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.

Based upon our field exploration and review of readily available published maps for the site, the main geotechnical concerns for the proposed site development may include:

- Disturbed near-surface soil and existing undocumented fill.
- Expansive soil.
- Compressible soil.
- Seismically induced settlement of potentially liquefiable soil.
- Shallow groundwater.

### 3.1 DISTURBED NEAR-SURFACE SOIL AND EXISTING UNDOCUMENTED FILL

As previously mentioned, much of the development area has been used as a golf course, resulting in disturbed near-surface soil. Disturbed near-surface soil and undocumented fill may undergo excessive settlement, especially when subjected to new loads from grading and the planned building. Based on the results from our test pits (Appendix B), the site appears to be underlain by up to approximately 3 feet of disturbed soil or undocumented fill. Further field exploration may be conducted to further determine the areal extent and depth of the undocumented fill.

We present disturbed soil and fill treatment recommendations in Section 4.1.

## 3.2 EXPANSIVE SOIL

Based on our laboratory testing, potentially expansive clay was found near surface in the test pits. Our laboratory test results indicate that these soils exhibit moderate to high shrink/swell potential when subjected to variations in moisture content.

Successful performance of structures on expansive soils requires special attention during construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soils without excavation, moisture conditioning, and recompaction.

We have also provided preliminary grading recommendations for compaction of clay soil at the site. 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 amount of compaction. Expansive soil compaction recommendations are presented in Section 4.1 of this report.

Expansive soils 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. Building damage due to volume changes associated with expansive soils can be reduced by: (1) using a rigid mat foundation that is designed to resist the settlement and heave of expansive soil, (2) deepening the foundations to below the zone of moisture fluctuation, i.e. by using deep footings or drilled piers, and/or (3) using footings at normal shallow depths but bottomed on a layer of select fill having a low expansion potential.

Post-tensioned mat foundations are the preferred foundation system for the residential structures, and mat slabs  or footings with slab-on-grade for commercial structures. Design criteria for these foundation types are presented in Section 4.2.

## 3.3 COMPRESSIBLE SOIL

Our CPT data indicated soft to medium stiff clay-like soils were found at various depths between the ground surface and 20 feet bgs throughout the site as evidenced by low tip resistance of the cone. This soil is likely the previously mentioned basin deposits or Delta Mud deposits and is potentially susceptible to immediate and long-term settlement associated with additional loads of the proposed structures or/and additional fill. Further exploration and laboratory tests are required to characterize the potential compressibility of the subsurface soils. Once the structural loads are provided, total static settlement can be further evaluated and provided in the design-level geotechnical report.

## 3.4 SHALLOW GROUNDWATER

As described in the previous section, groundwater may vary between 7 and 15 feet below ground surface. Any excavation or utility installation at or below this depth should anticipate encountering groundwater.

## 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. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections.

Based on topographic and lithological data, the risk of regional subsidence or uplift, tsunamis, landslides and seiches is considered low at the site.

### 3.5.1 Ground Rupture

As illustrated in Figure 4, the project site is located within the Great Valley Fault blind thrust fault zone. However, in general, the risk of surface ruptures resulting from blind thrust faults are considered low. As previously discussed, the site is not located within a State of California Earthquake Fault Hazard Zone (1982) for known active faults. Therefore, fault rupture is not anticipated within the limits of the project.

### 3.5.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region, similar to those that have occurred in the past, could cause considerable ground shaking at the site. 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 substantially smaller than the expected peak 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 Liquefaction/Clay Soil Softening

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands. Empirical evidence indicates that loose to medium-dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable.

We performed a preliminary liquefaction-potential screening of the CPT soundings using the computer software CLiq Version 2.2 developed by GeoLogismiki. The procedure used in the software is based on the methodology by Robertson (2009) with consideration of cyclic softening of clay-like soils. The Cyclic Stress Ratio (CSR) was estimated for a peak ground acceleration (PGA) of 0.59g as outlined in the ASCE 7-10 and moment magnitude of 7.3 based on the nearby Hayward fault. We evaluated the liquefaction potential for the soils using the estimated groundwater level during CPT testing of 10 feet below ground surface.

Our preliminary liquefaction analysis results indicated intermittent layers of the clay and silty clay encountered in our explorations has potential for liquefaction. However, based on the previously described depositional environment and the likely presence of the high plasticity basin and Delta mud deposits, the hazard from liquefaction-induced settlement is considered low.

Additional sampling should be done during the design-level field exploration to further characterize the liquefaction susceptibility.

### 3.5.4 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sands or weak soils. Ulatis Creek along the southeast edge of the site has an approximate embankment height of 5 feet. Horse Creek, an unlined channelized creek along the northern edge of the site has an approximate embankment height of 15 to 17 feet. Further liquefaction analysis and lateral spread screening should be incorporated into the design-level geotechnical report.

## 3.6 FLOOD ZONE

The northernmost section of the project site is mapped within Zone A and a majority of the southern section of the project site is mapped within Zone X on the Federal Emergency Management Agency (FEMA 2015) Flood Hazard Map for City of Vacaville (Figure 5). The project Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

## 3.7 2016 BUILDING CODE SEISMIC DESIGN

We provide the 2016 California Building Code (CBC) seismic parameters in Table 3.7-1 below.

**TABLE 3.7-1: 2016 CBC Seismic Design Parameters
Latitude: 38.380276, Longitude: -121.937234**

| PARAMETER | VALUE |
|---|---|
| Site Class | D |
| Mapped $MCE_R$ Spectral Response Acceleration at Short Periods, $S_S$ (g) | 1.60 |
| Mapped $MCE_R$ Spectral Response Acceleration at 1-second Period, $S_1$ (g) | 0.55 |
| Site Coefficient, $F_A$ | 1.00 |
| Site Coefficient, $F_V$ | 1.50 |
| $MCE_R$ Spectral Response Acceleration at Short Periods, $S_{MS}$ (g) | 1.60 |
| $MCE_R$ Spectral Response Acceleration at 1-second Period, $S_{M1}$ (g) | 0.83 |
| Design Spectral Response Acceleration at Short Periods, $S_{DS}$ (g) | 1.07 |
| Design Spectral Response Acceleration at 1-second Period, $S_{D1}$ (g) | 0.55 |
| Mapped MCE Geometric Mean ($MCE_G$) Peak Ground Acceleration, PGA (g) | 0.59 |
| Site Coefficient, $F_{PGA}$ | 1.00 |
| $MCE_G$ Peak Ground Acceleration adjusted for Site Class effects, $PGA_M$ (g) | 0.59 |

# 4.0 PRELIMINARY RECOMMENDATIONS

## 4.1 GRADING

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 more detailed land plans have been prepared and additional field exploration conducted during our design-level geotechnical exploration.

### 4.1.1 Demolition and Stripping

Site development will commence with the demolition of existing structures and improvements, including structure foundations, abandoned utilities, irrigation lines, and basements, if any exist. All debris should be removed from any location to be graded, from areas to receive fill or structures, or those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

Existing vegetation and pavements (asphalt concrete/concrete and underlying aggregate base) should be removed from areas to receive fill, or structures, or those areas to serve for borrow. Strip organics from the ground surface to a depth of at least 2 to 3 inches below the surface. Tree roots should be removed down to a depth of at least 3 feet below existing grade. The actual depth 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 soils should be removed from the project site. 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.

### 4.1.2 Disturbed Near-Surface Soil and Undocumented Fill Removal

All disturbed near-surface soil, existing fill, and soft material (such as beneath the existing landscape ponds) should be excavated to expose firm native soils. Excavated material may be used as fill material if it meets the requirements of Section 4.1.3.

### 4.1.3 Selection of Materials

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 2 percent organic content by weight), and environmentally impacted soils (if any), we anticipate the site soils are suitable for use as engineered fill provided they are broken down to particles with diameter of 6 inches or less. Other materials and debris, including trees with their root balls, should be removed from the project site.

Imported fill materials should meet the above requirements and have a plasticity index less than 15. ENGEO should sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

### 4.1.4 Fill Placement

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

- Test Procedures: ASTM D1557.
- Required Moisture Content: Not less than 4 percentage points above optimum moisture content for soil with PI of 15 or greater.

  Not less than 3 percentage points above optimum moisture content for soil with PI of less than 15.
- Minimum Relative Compaction: 87 to 92 percent for soil with PI of 15 or greater.
  90 percent for soil with PI of less than 15.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material.

Additional compaction requirements may be required for deeper fills and retaining wall backfill. These additional requirements will be developed during our detailed exploration.

## 4.2 SPECIAL SUBGRADE PREPARATION

For commercial buildings that will be supported on a conventional footing system with slab-on-grade floor, the upper 18 inches of the building pad subgrade soils should consist of non-expansive soil material. As an alternative to importation of select fill, the upper 18 inches of building pad subgrade soils can be lime treated. The special treatment area should include the building footprint and an area extending 5 feet out from the building perimeters or to adjacent curb where walkways are planned.

### 4.2.1 Non-Expansive Selected Fill

The non-expansive selected fill should be compacted to a relative compaction of at least 95 percent and a moisture content of at least 2 percentage points above the optimum.

### 4.2.2 Lime-Treated Subgrade Soils

The lime mix should consist of 3 to 5 percent lime. The lime mix should be approved by ENGEO. Prior to lime treating the subgrade soils, testing should be performed to determine the actual percentage of lime required.

1. The soil should be moisture conditioned to at least 3 percentage points above the optimum moisture content before mixing. The mixing should be performed in accordance with the current version of Caltrans Standard Specifications with the following exceptions:

2. Following mixing, the treated soils should be allowed to fully hydrate at least 24 hours prior to compaction.

3. Following hydration, the treated soil should be compacted according to ASTM D1557 to not less than 95 percent relative compaction at a moisture content at least 3 percentage points above the optimum to a non-yielding surface.

## 4.3 PRELIMINARY FOUNDATION DESIGN

We developed preliminary foundation recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. The main considerations in foundation design for this project are expansive soil or differential movement due to the swell potential of the site's foundation soils. The preliminary recommendations for three foundation options are provided below:

- Structural mat foundation.
- Post-tensioned mat.
- Footings with slab-on-grade floor.

### 4.3.1 Structural Mat Foundations

For preliminary planning purposes, structural mat foundations may be considered for commercial structures or mixed-use podiums. The mat foundations should bear on properly compacted engineered fill. The upper 18 inches of building area foundation subgrade should consist of non- to low-expansive select fills (Plasticity Index less than 15), or alternatively, lime treatment foundation subgrade extending a minimum of 18 inches below building foundation subgrade level and laterally 5 feet beyond the building footprint.

The mat may be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live loads. This allowable bearing pressure may be increased to 2,000 psf in areas of loading concentration. The allowable bearing pressure can be increased by one-third for short-term loading that includes wind or seismic load combinations. For structures where moisture intrusion through the slab-on-grade would be a performance issue, we recommend they be underlain with a moisture reduction system as recommended in Section 4.3.

### 4.3.2 Post-Tensioned Mat Foundations

For preliminary purposes, post-tensioned (PT) slab foundations on properly prepared compacted fill may be considered for supporting the proposed single-family and townhome structures. On a preliminary basis, we recommend that PT mats be a minimum of 10 inches thick or greater and have a thickened edge at least 2 inches greater than the mat thickness. The Structural Engineer should determine the actual PT mat thickness using the geotechnical recommendations in the design-level report. We recommend that the thickened edge be at least 12 inches wide.

PT mats are typically underlain by a moisture reduction system as recommended in Section 4.3. In addition, the building pad subgrade is typically moisture conditioned such that the subgrade soil is at a moisture content at least 3 percentage points above optimum immediately prior to foundation construction. The subgrade should not be allowed to dry prior to concrete placement.

### 4.3.3 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 below the vapor retarder membrane.

### 4.3.4 Footings with Slab-on-Grade

Structures may be supported by a conventional continuous and interconnected strip footing foundation system. Footings should be embedded a minimum depth of 36 inches below the lowest adjacent pad grade. The upper 18 inches of slab subgrade should be underlain by either non- to low-expansive material or lime treated soil material as provided in previous section.

Design foundations recommended above for a maximum allowable bearing pressure of 2,000 pounds per square foot (psf) for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading. Lateral loads can be resisted by a friction coefficient between soil and bottom of footing of 0.30. An equivalent fluid passive pressure of 250 pounds per cubic foot can be used on the side of the footings, but should neglect the top one foot. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

### 4.3.5 Interior Concrete Floor Slabs

Provided the upper 18 inches of subgrade soil will consist of non- to low-expansion soil material or lime-treated soils, the following can be incorporated in the slab design for the conventional footing system:

1. Provide a minimum concrete thickness of 5 inches.

2. Place minimum steel reinforcing of No. 3 rebar on 18 inches on center each way within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures.

3. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following:
   a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs". The vapor retarder should be **underlain by**

   b. 6 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.

The structural engineer should provide final design thickness and additional reinforcement, as necessary, for the intended structural loads.

## 4.4 PRELIMINARY PAVEMENT DESIGN

The following preliminary pavement sections have been determined for an assumed Resistance Value (R-value) of 5 and in accordance to the design methods contained in Chapter 630 of Caltrans Highway Design Manual.

**TABLE 4.4-1: Preliminary Pavement Section**

| TRAFFIC INDEX | AC (INCHES) | AB (INCHES) |
|---|---|---|
| 5.0 | 3.0 | 10.0 |
| 6.0 | 3.5 | 13.0 |
| 7.0 | 4.0 | 16.0 |

Notes: AC – Asphalt Concrete
AB – Caltrans Class 2 aggregate base (R-value of 78 or greater)

The above preliminary pavement sections are provided for estimating only. We recommend the actual subgrade material should be tested for R-value, and the Traffic Index and minimum pavement section(s) should be confirmed by the Civil Engineer and the City of Vacaville.

## 4.5 RETAINING WALLS

### 4.5.1 Lateral Soil Pressures

Design proposed retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, walls restrained from movement at the top, such as basement walls, should be designed to resist an equivalent fluid pressure of 80 pounds per cubic foot (pcf) for level backfill. Unrestrained retaining walls should be designed with adequate drainage to resist an equivalent fluid pressure of 50 pcf for level backfill. In addition, design all walls to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining wall.

### 4.5.2 Retaining Wall Drainage

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.

2. Extend rock drains from the wall base to within 12 inches of the top of the wall.

3. Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down.

4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

### 4.5.3 Backfill

Backfill behind retaining walls should be placed and compacted in accordance with Section 4.1. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

### 4.5.4 Site Retaining Wall Foundations

For preliminary design purposes, retaining walls may be supported on continuous footings designed in accordance with recommendations presented in Section 4.2.3. Minimum embedment depth should be 24 inches below lowest adjacent soil grade.

### 4.6 DRAINAGE

The building pads must be positively graded at all times to provide for rapid removal of surface water runoff from the foundation systems and to prevent ponding of water under floors or seepage toward the foundation systems at any time during or after construction. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Ponding of stormwater must not be permitted on the building pad during prolonged periods of inclement weather. All surface water should be collected and discharged into the storm drain system. Landscape mounds must not interfere with this requirement.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should be directed to a solid pipe that discharges to the street or to an approved outlet or onto an impervious surface, such as pavement that will drain at a 2 percent slope gradient.

## 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 should include supplemental borings and laboratory soil testing to provide additional data for evaluation of liquefaction and seismic induced settlement, consolidation of compressible soil, extend and depth of disturbed soil and existing fill, and corrosion potential of site soils. Further test pits may also be conducted to refine the areal extent of disturbed soil and existing fill. The design-level report will also provide specific recommendations regarding grading, foundation design, retaining wall design, and drainage for the proposed development.

## 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 Greentree 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 preliminary 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; 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, ENGEO must be notified 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, 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, the proper regulatory officials must be notified 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.

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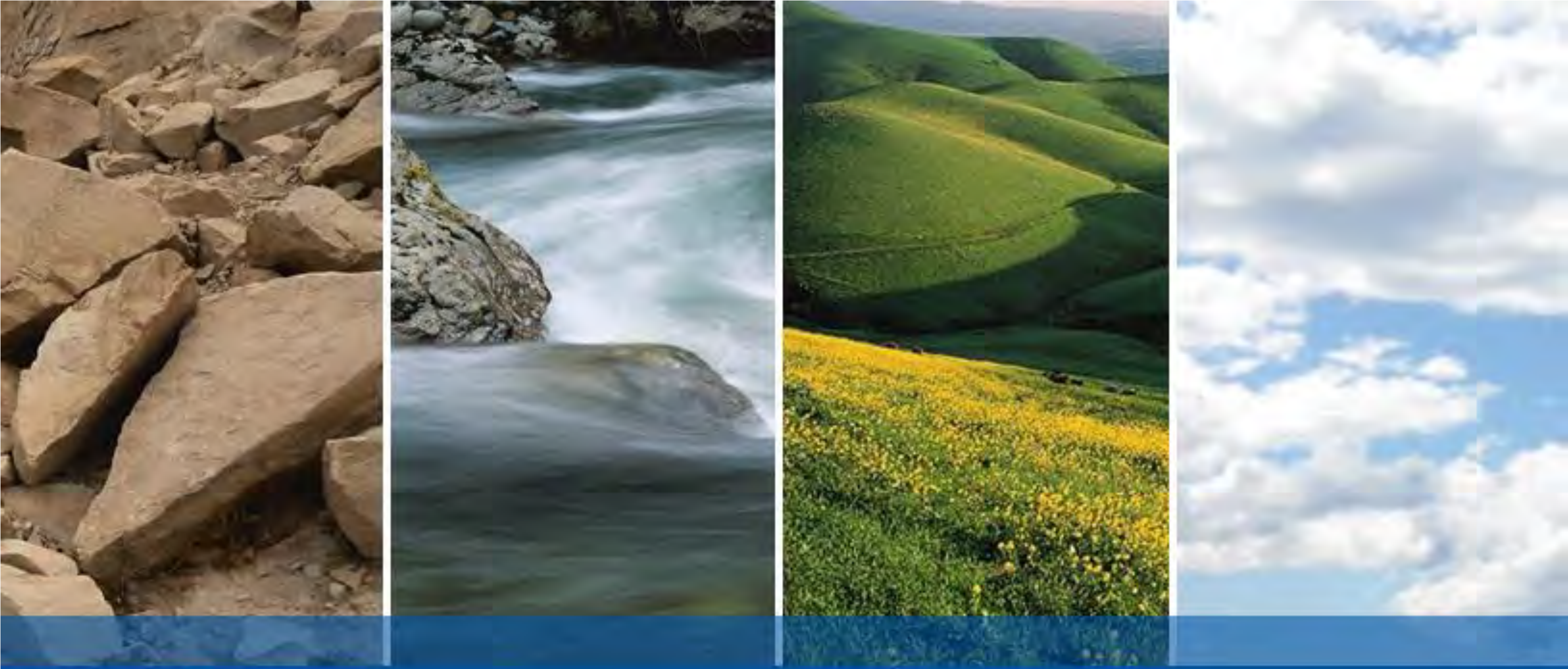


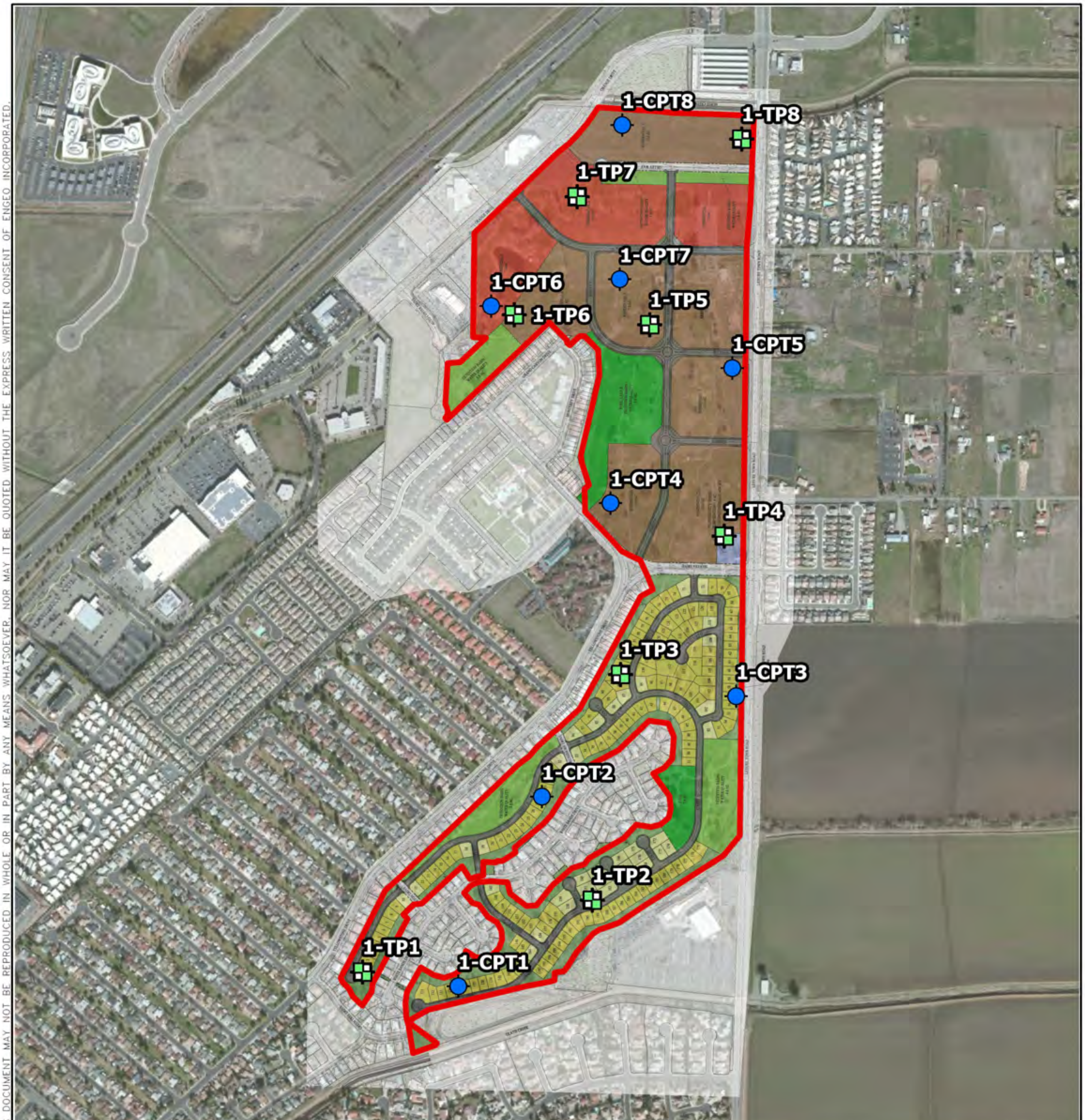
# FIGURES


BASEMAP SOURCE: ESRI MAPPING SERVICE

| ENGEO Expect Excellence | VICINITY MAP<br>GREENTREE<br>VACAVILLE, CALIFORNIA | PROJECT NO.: 16018.000.000<br>SCALE: AS SHOWN<br>DRAWN BY: QRL CHECKED BY: TTB | FIGURE NO.<br>1 |
|---|---|---|---|

PATH: G:\DRAFTING\DRAFTING2_DWG_13000 PLUS\16018\000\GREEN TREE.APRX
USER: OLIANG

ORIGINAL FIGURE PRINTED IN COLOR




EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROJECT SITE

1-CPT8 CPT (ENGEO, 2019)

1-TP8 TEST PIT (ENGEO, 2019)

N

0 500 1,000

FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE, CBG



| ENGEO Expect Excellence | SITE PLAN<br>GREENTREE<br>VACAVILLE, CALIFORNIA | PROJECT NO.: 16018.000.000 | | FIGURE NO. 2 |
|---|---|---|---|---|
| | | SCALE: AS SHOWN | | |
| | | DRAWN BY: QRL | CHECKED BY: TTB | |

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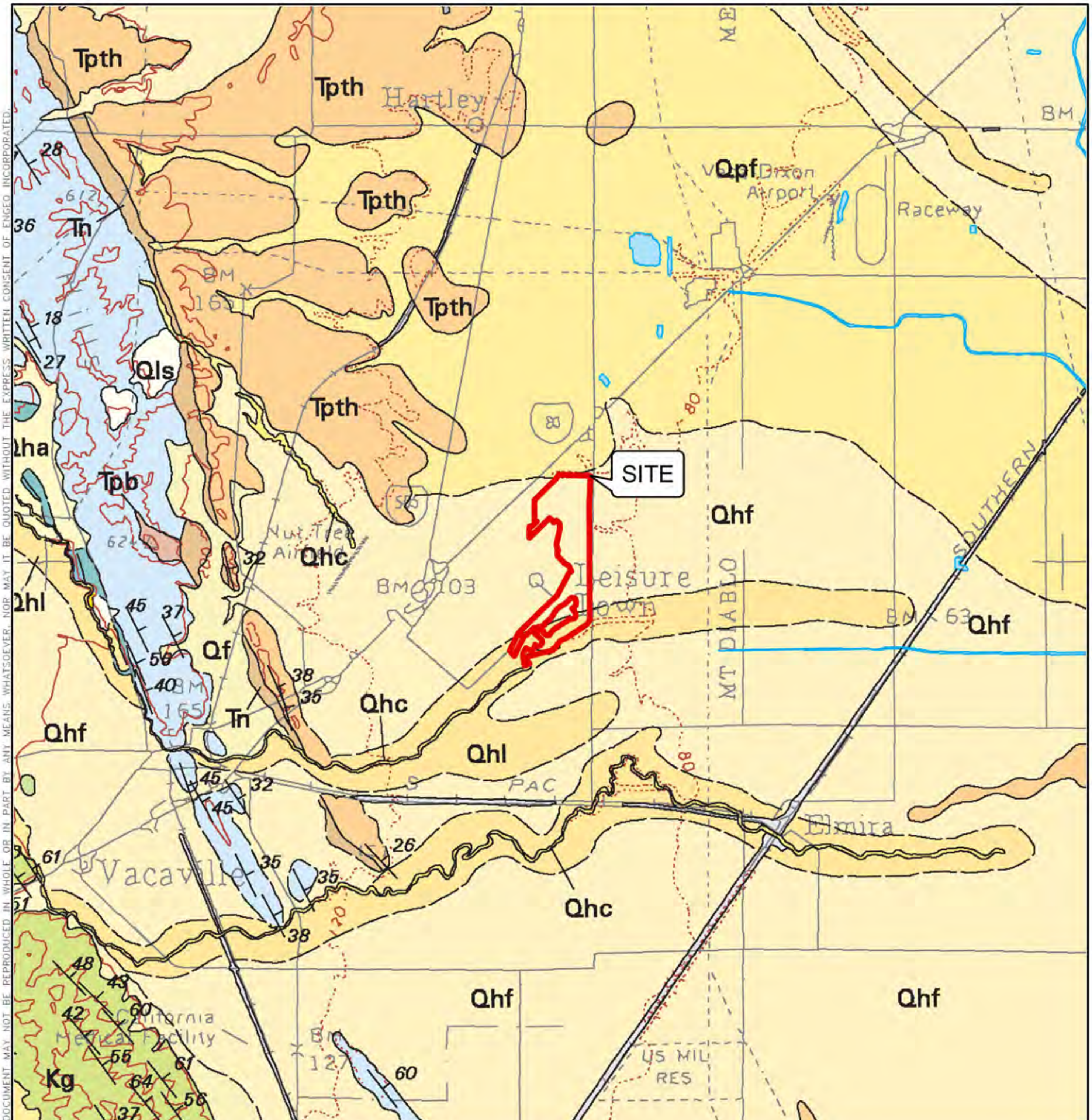
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR




N

0 2,500 5,000

FEET

## EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

| | |
|---|---|
| Qhf | ALLUVIAL FAN DEPOSITS (HOLOCENE) |
| Qhc | STREAM CHANNEL DEPOSITS (HOLOCENE) |
| Qhl | NATURAL LEVEE DEPOSITS (HOLOCENE) |
| Qf | ALLUVIAL FAN DEPOSITS (HOLOCENE AND LATE PLEISTOCENE) |
| Qls | LANDSLIDE DEPOSITS (HOLOCENE AND PLEISTOCENE) |
| Qpf | ALLUVIAL FAN DEPOSITS (LATE PLEISTOCENE) |
| Tn | NEROLY SANDSTONE (LATE MIOCENE) |
| Tpth | TEHAMA FORMATION (PLIOCENE) |
| Tpb | PUTNAM PEAK BASALT (MIOCENE) |

BASEMAP SOURCE: GRAYMER 2002

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
GREENTREE
VACAVILLE, CALIFORNIA

| | |
|---|---|
| PROJECT NO.: 16018.000.000 | FIGURE NO. 3 |
| SCALE: AS SHOWN | |
| DRAWN BY: QRL | CHECKED BY: TTB |

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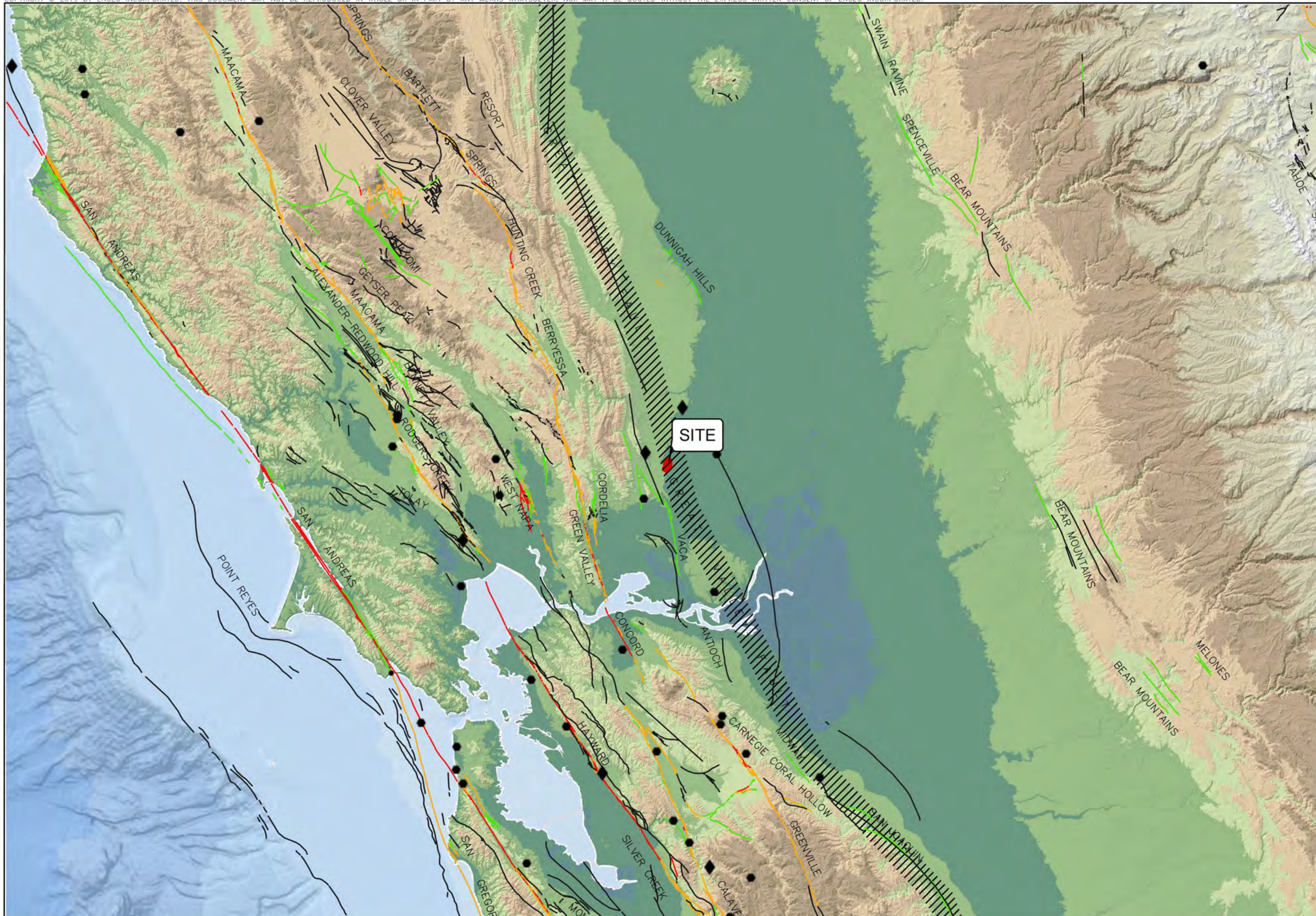
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR




## EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

**EARTHQUAKE**

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

**USGS QUATERNARY FAULTS**

- HISTORICAL
- LATEST QUATERNARY
- LATE QUATERNARY
- UNDIFFERENTIATED QUATERNARY

HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE
ESRI, GARMIN, GEBCO, NOAA NGDC, AND OTHER CONTRIBUTORS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

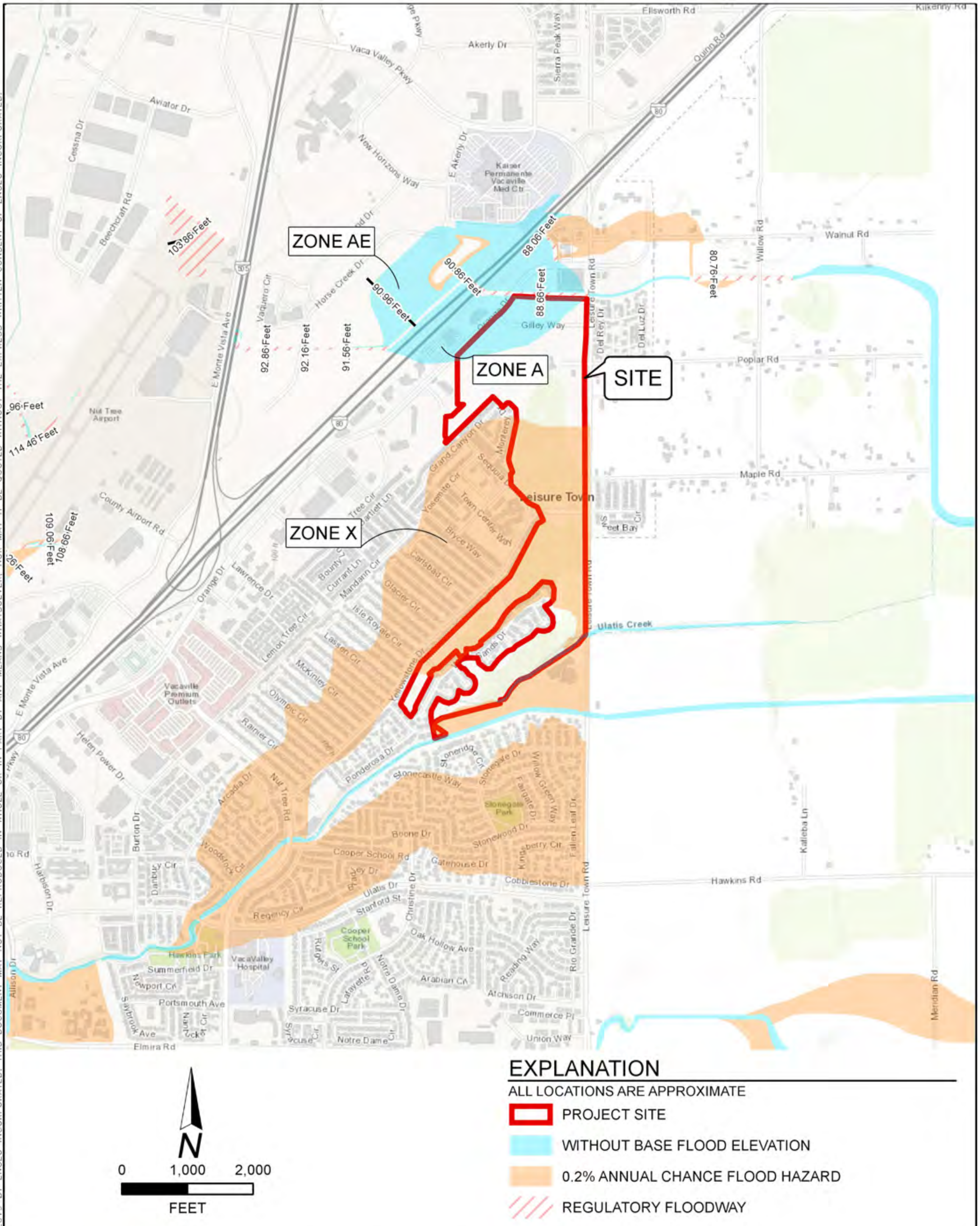
ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY
GREENTREE
VACAVILLE, CALIFORNIA

| | |
|---|---|
| PROJECT NO.: 16018.000.000 | FIGURE NO. 4 |
| SCALE: AS SHOWN | |
| DRAWN BY: QRL | CHECKED BY: TTB |

PATH: G:\DRAFTING\DRAFTING2_DWG_13000 PLUS\16018\000\GREEN TREE.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR




BASEMAP SOURCE: ESRI MAPPING SERVICE, FEMA

| | | | |
|---|---|---|---|
| ENGEO Expect Excellence | FEMA FLOOD MAPS<br>GREENTREE<br>VACAVILLE, CALIFORNIA | PROJECT NO.: 16018.000.000<br>SCALE: AS SHOWN<br>DRAWN BY: QRL CHECKED BY: TTB | FIGURE NO.<br>5 |

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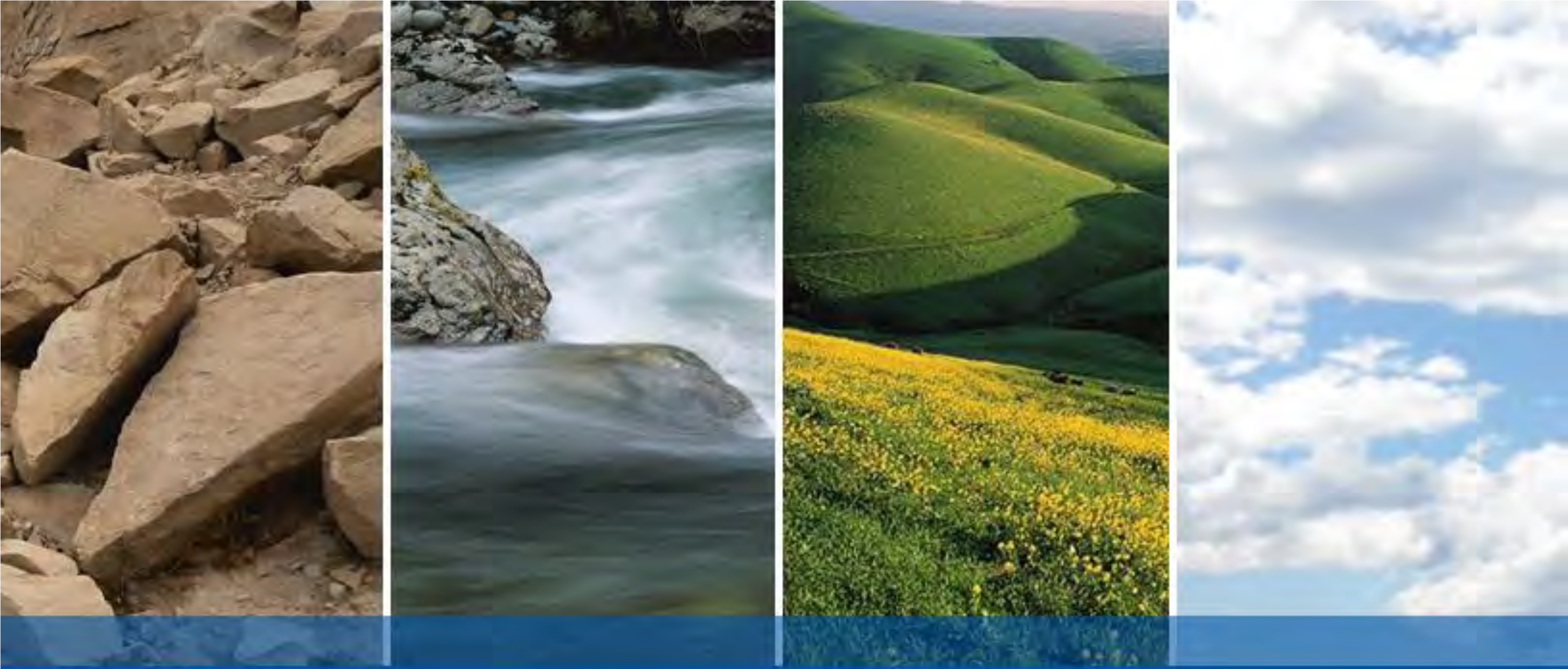


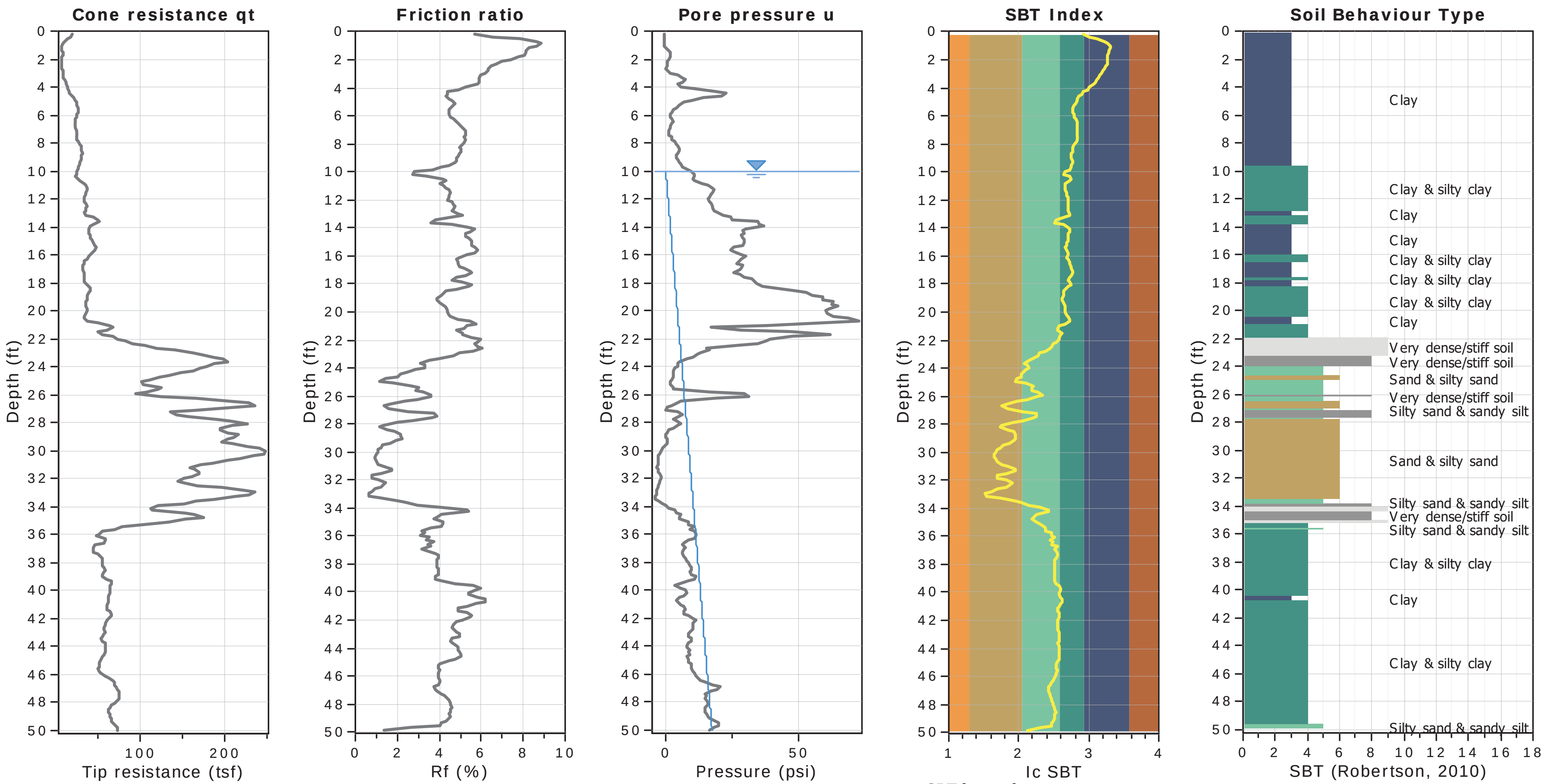
# APPENDIX A

## Cone Penetration Test Logs

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

**CPT: 1-CPT-1**
Total depth: 50.03 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

**Project:**
**Location:**



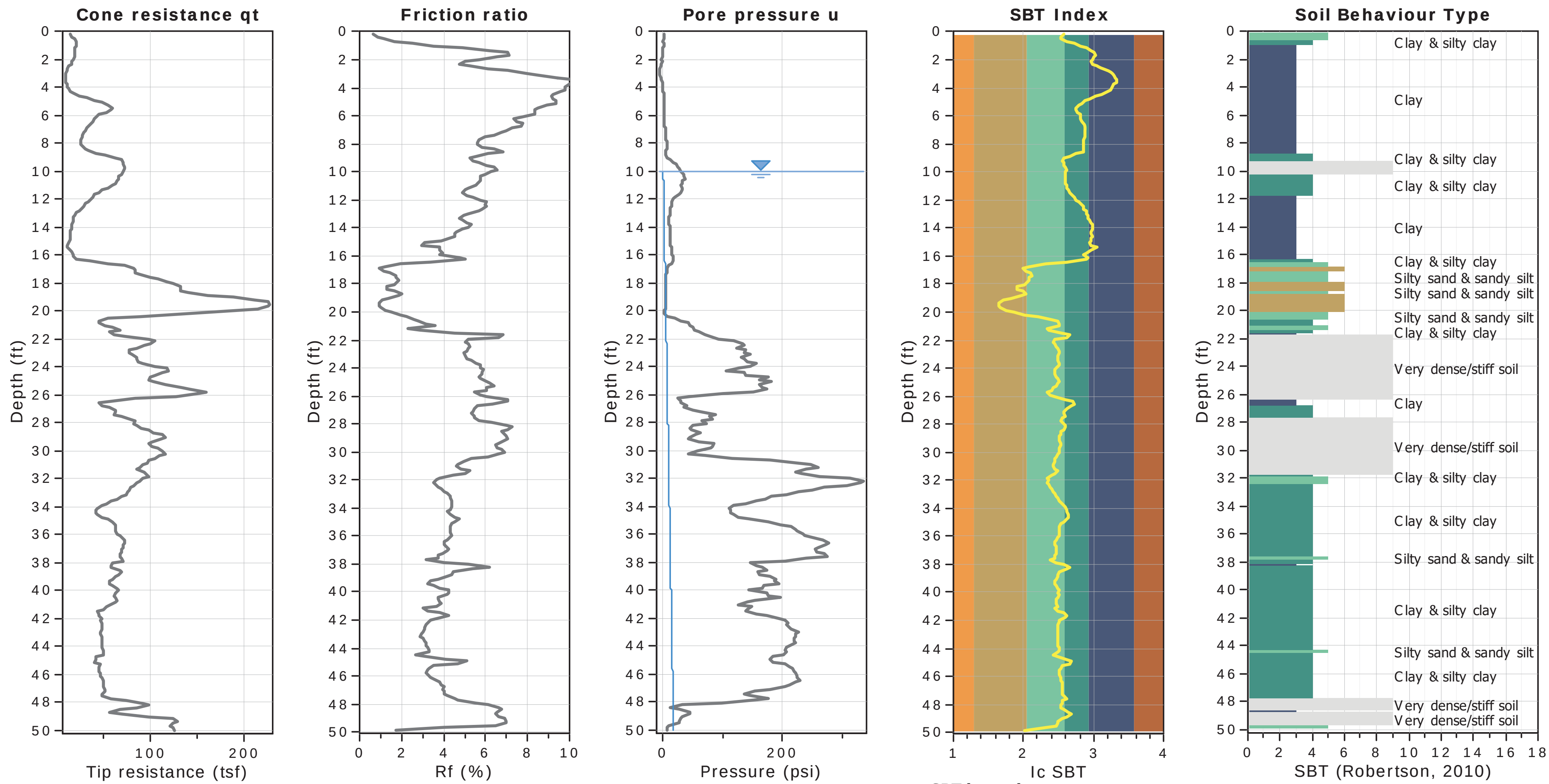

**SBT legend**

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

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

**CPT: 1-CPT-2**
Total depth: 50.03 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

**Project:**
**Location:**



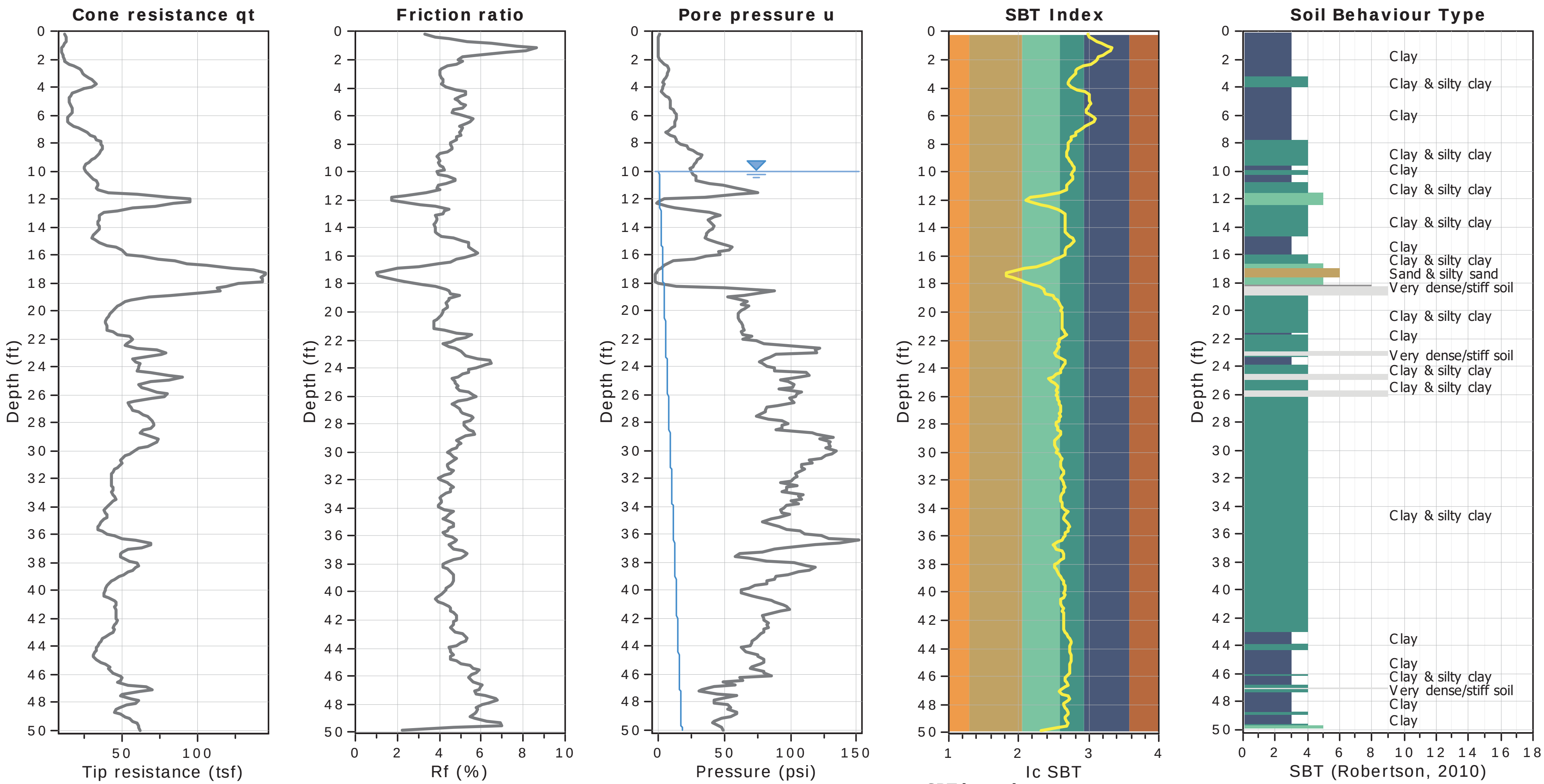

**SBT legend**

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

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

**CPT: 1-CPT-3**
Total depth: 50.03 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

**Project:**
**Location:**



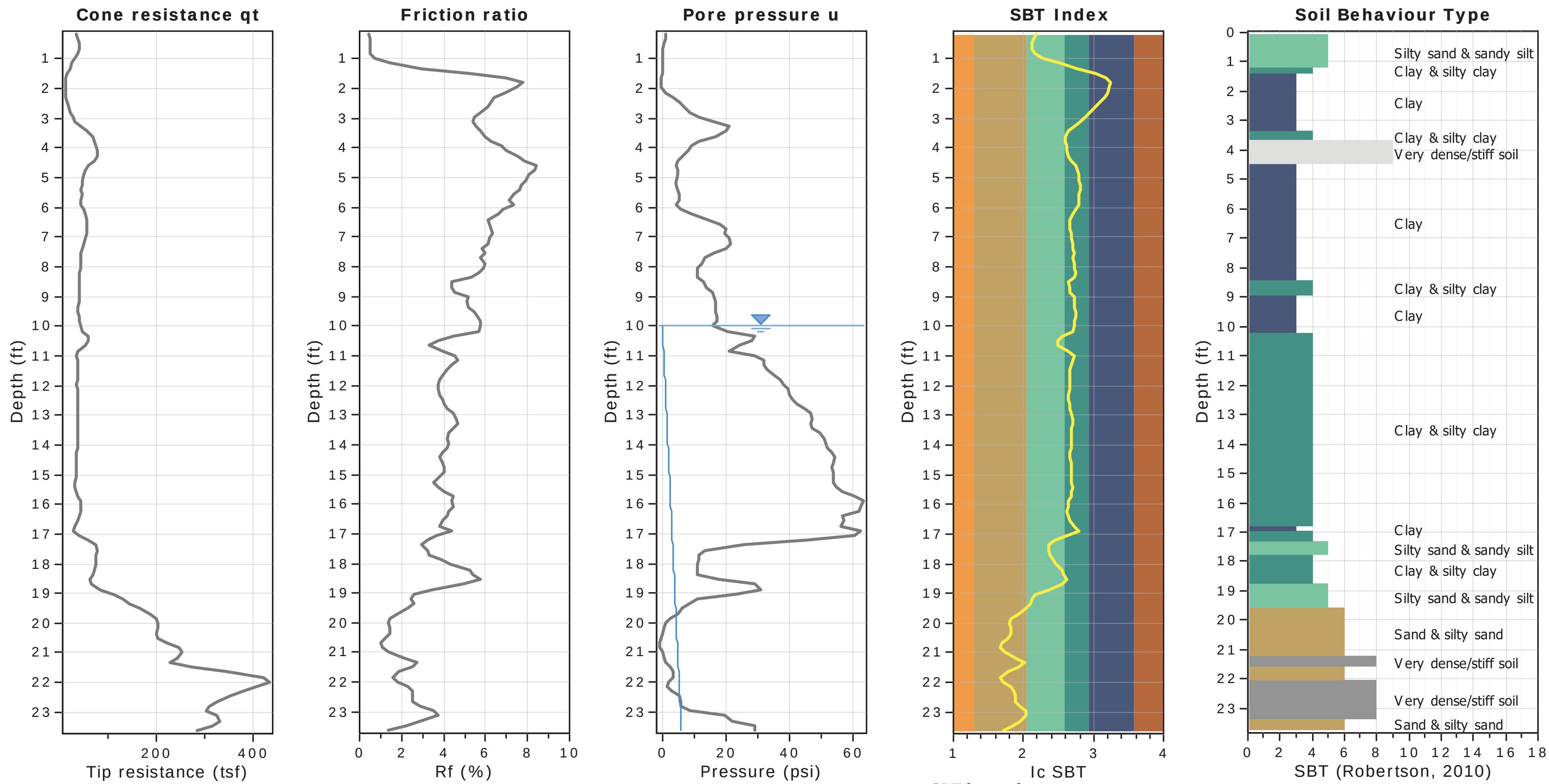

**SBT legend**

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

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

**CPT: 1-CPT-4**
Total depth: 23.62 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

**Project:**
**Location:**




**SBT legend**

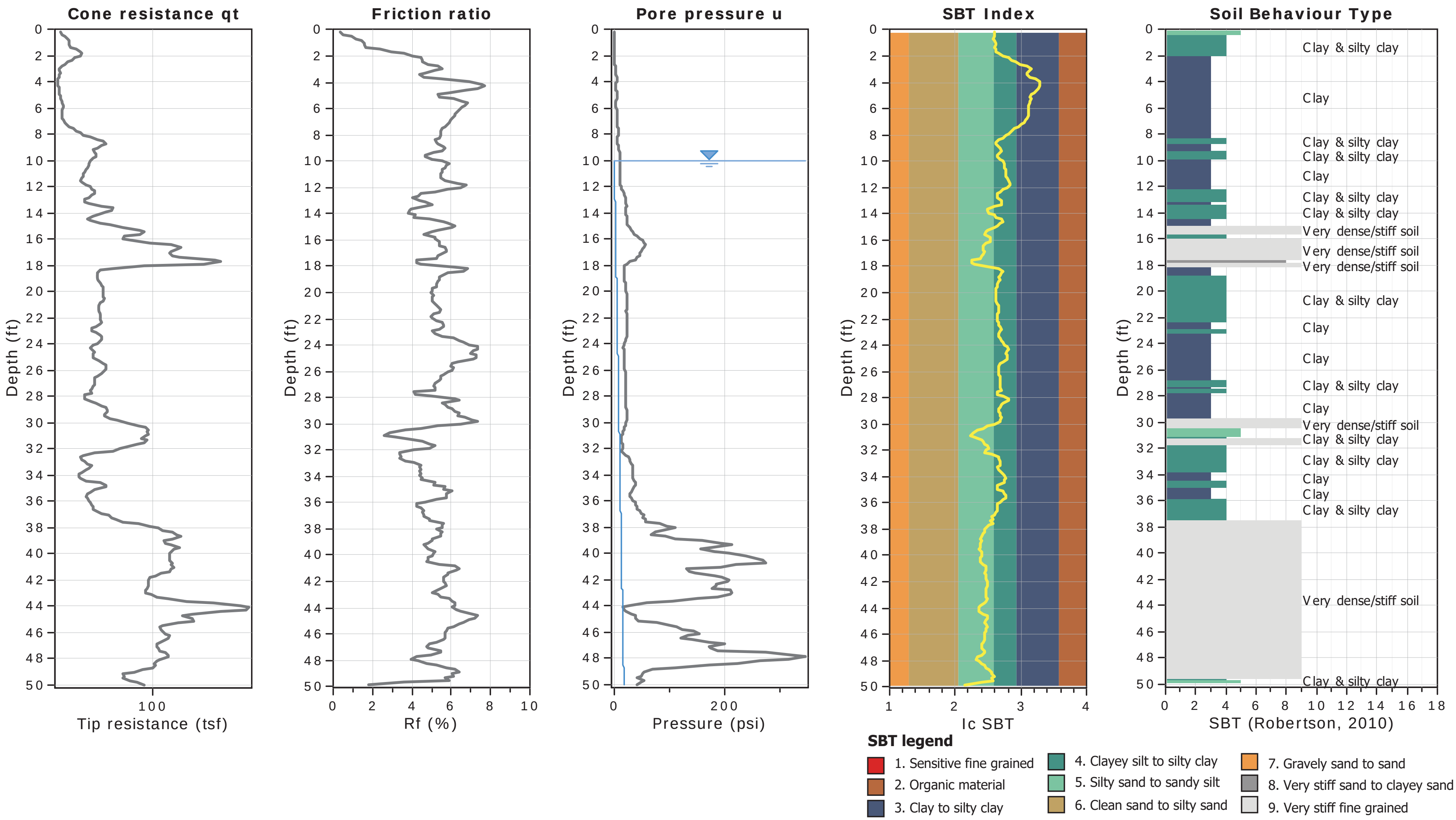
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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

**Project:**

**Location:**

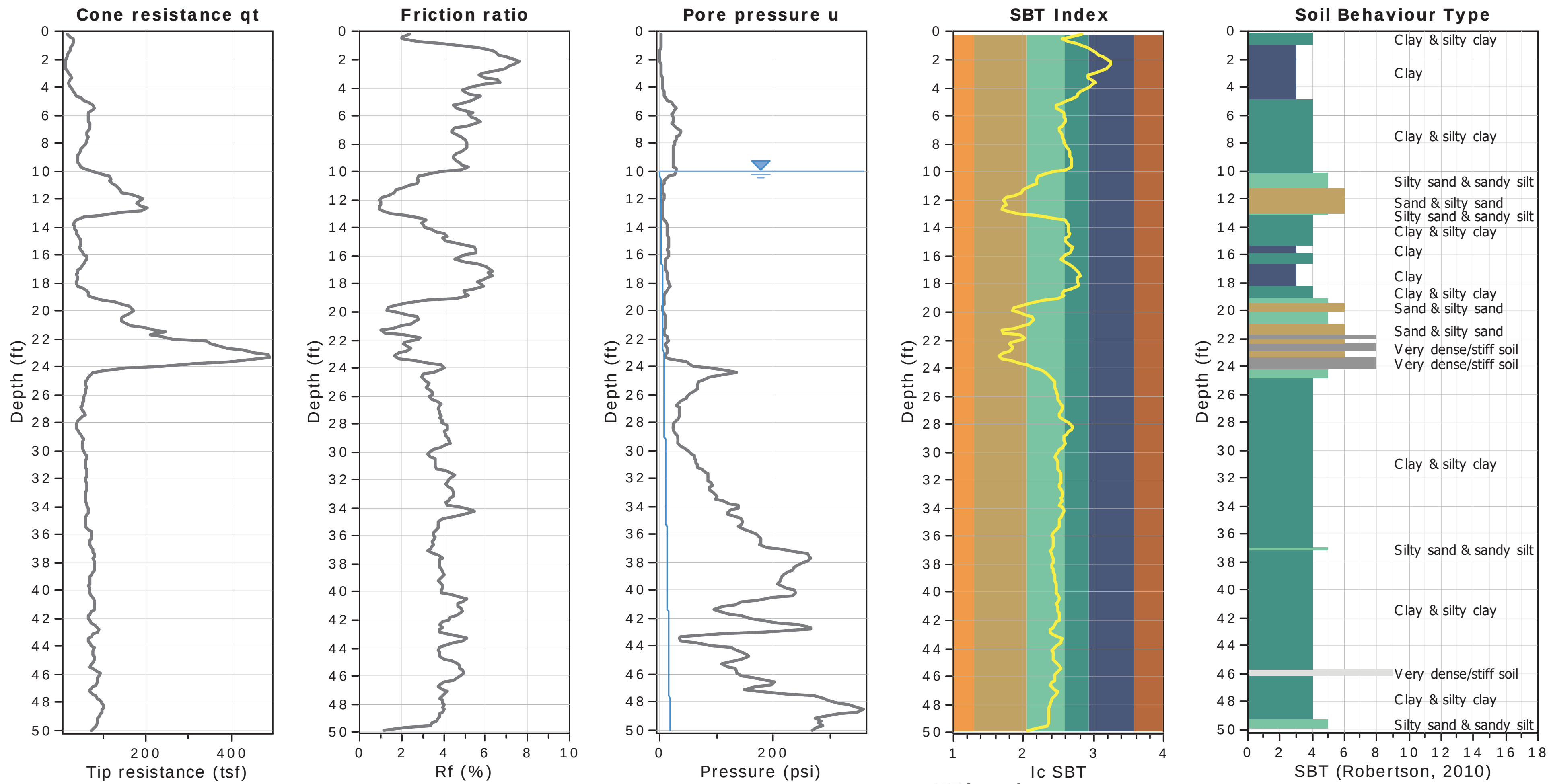
**CPT: 1-CPT-5**
Total depth: 50.03 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



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

**CPT: 1-CPT-6**
Total depth: 50.03 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

**Project:**
**Location:**




**SBT legend**

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

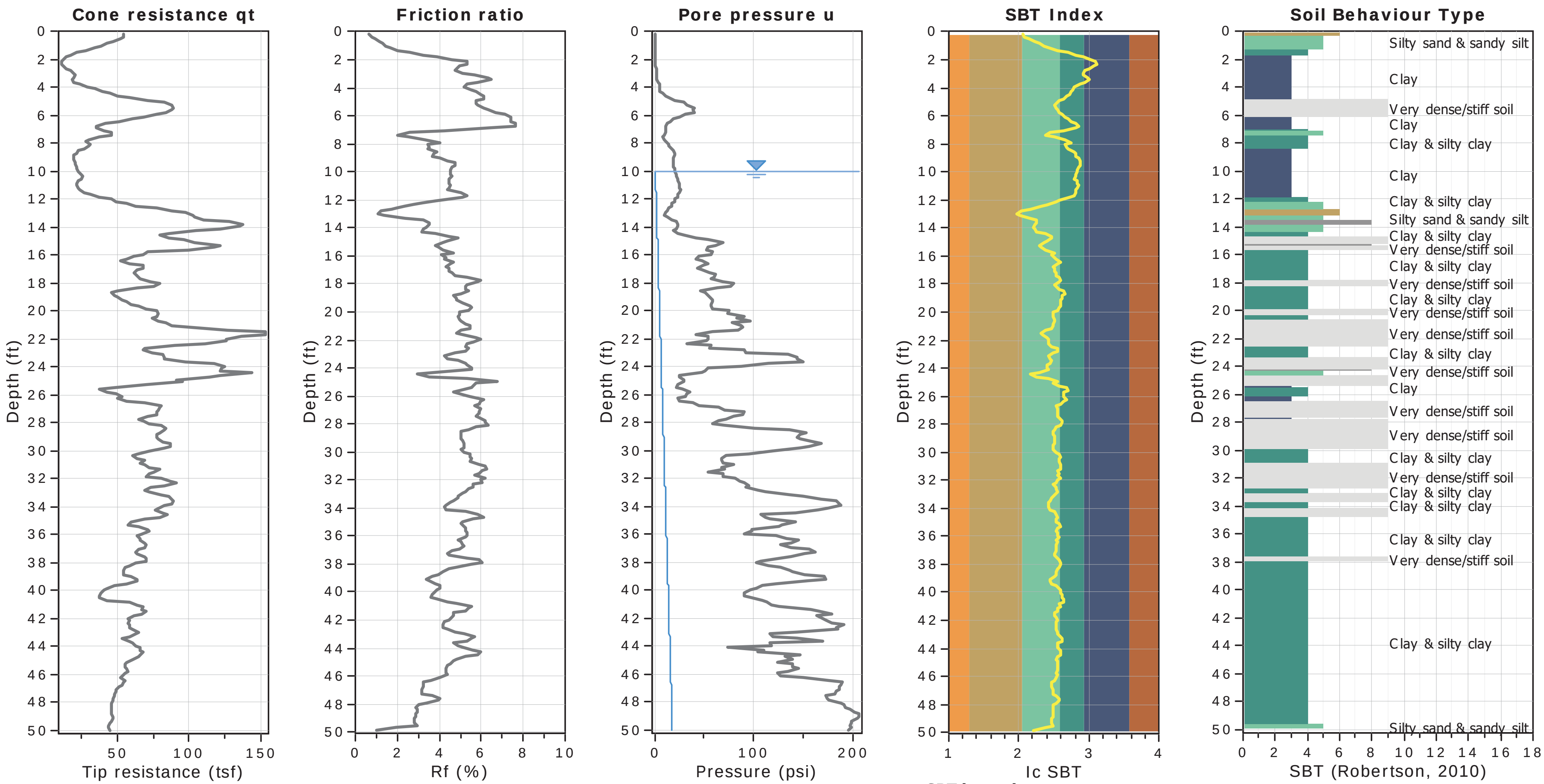
GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

**Project:**

**Location:**

**CPT: 1-CPT-7**

Total depth: 50.03 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



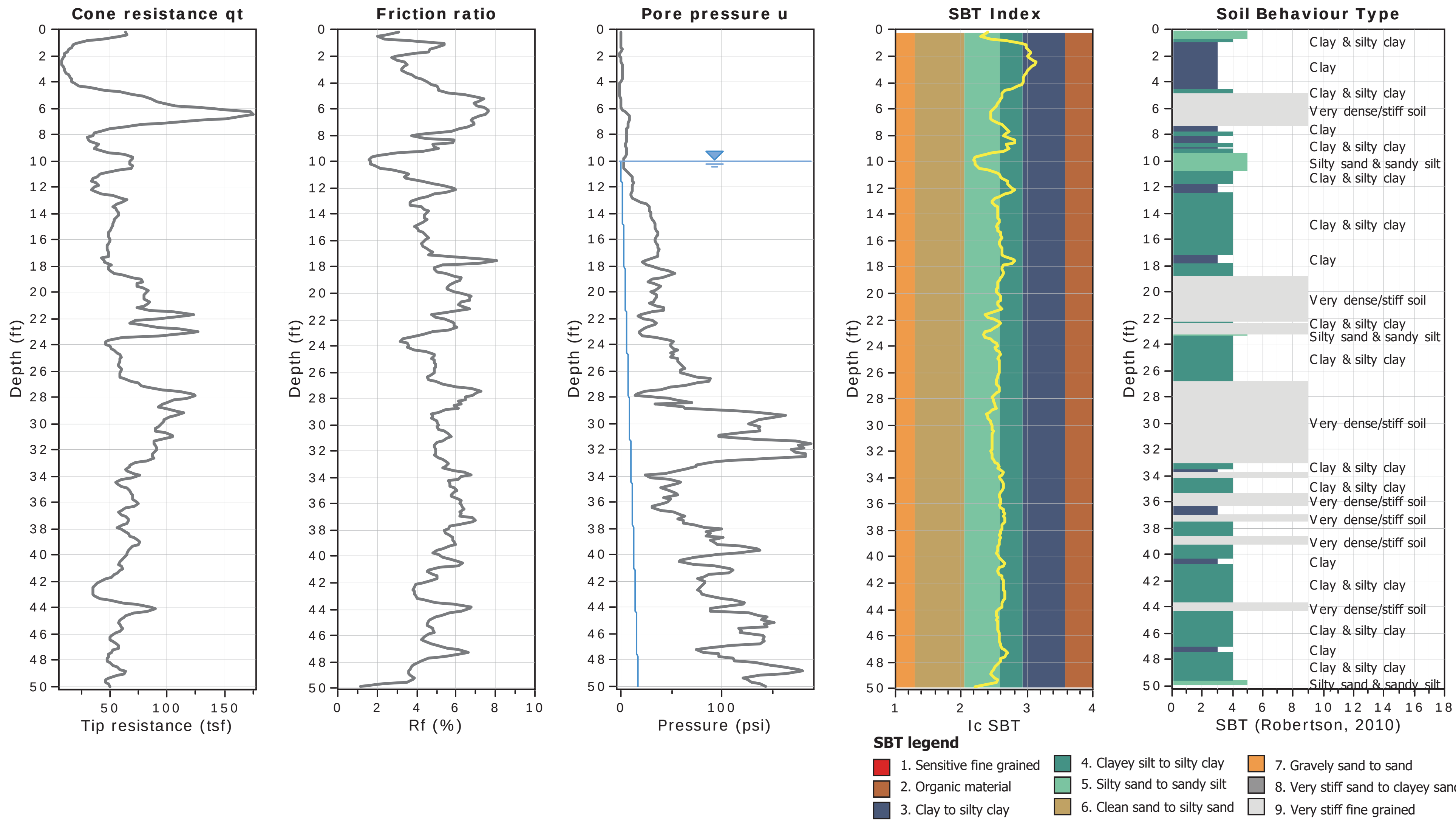

**SBT legend**

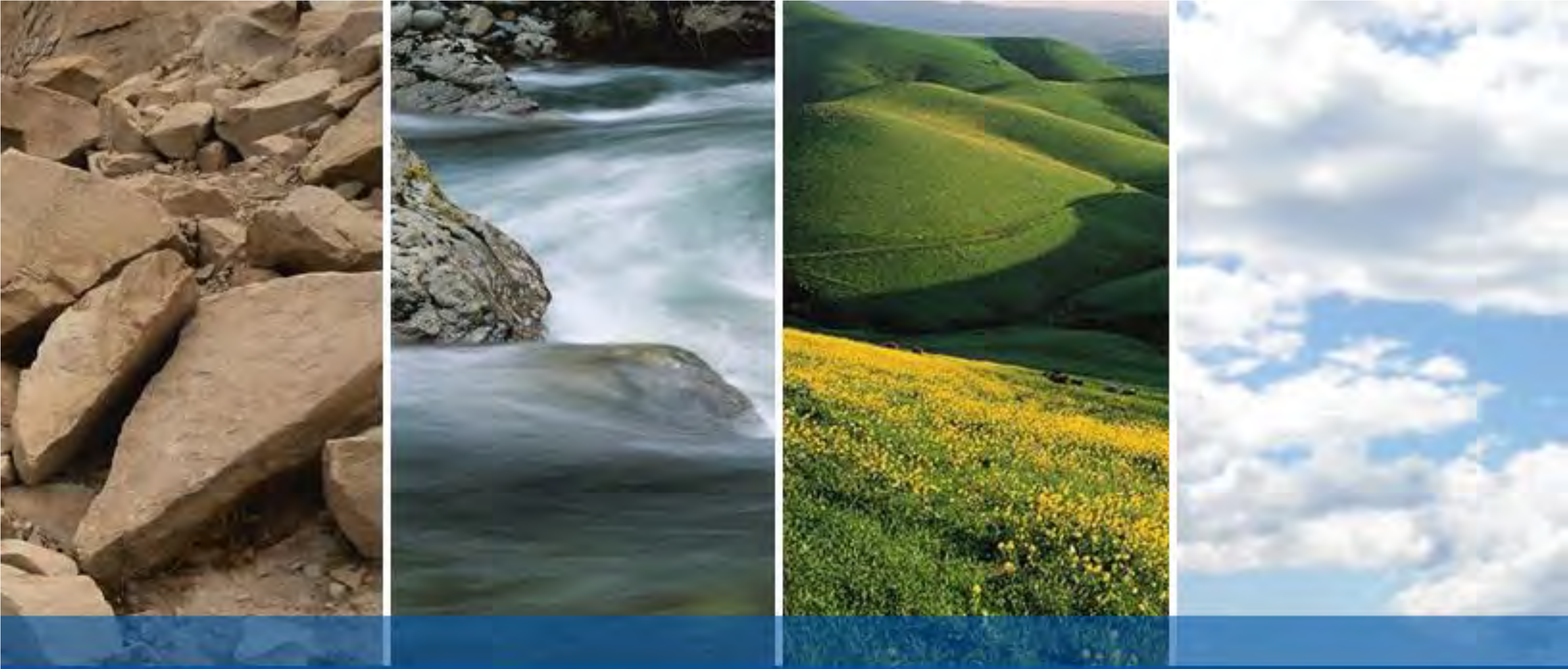
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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

**Project:**
**Location:**

**CPT: 1-CPT-8**
Total depth: 50.03 ft, Date: 5/14/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown





# APPENDIX B

## Test Pit Logs

# TEST PIT LOG

Green Tree
999 Leisure Town Road
Vacaville, CA.
16018.000.000

Logged By: Jerry Chen
Logged Date: May 2, 2019

| Test Pit Number | Depth (ft.) | Description |
|---|---|---|
| T-1 | 0 – 1½ | SAND (SP), light yellowish brown, moist, fine-grained, clean [FILL] |
| | 1½ –1¾ | Pea GRAVEL (GP), clean, fine, rounded [FILL] |
| | 1¾– 4¾ | LEAN CLAY (CL), dark brown (10YR3/3), very stiff, moist, PP = 3.75 tsf [NATIVE] |
| | 4¾ – 6½ | FAT CLAY (CH), very dark greenish gray (GREY1,3/10Y), medium stiff, moist, PP = 1.25 tsf |
| | | -Groundwater was not encountered at time of test pits |

| ENGEO Expect Excellence | TEST PIT LOG | |
|---|---|---|
| Green Tree<br>999 Leisure Town Road<br>Vacaville, CA.<br>16018.000.000 | Logged By: Jerry Chen<br>Logged Date: May 2, 2019 | |
| **Test Pit Number** | **Depth (ft.)** | **Description** |
| T-2 | 0 – 2 | CLAY (CL), yellowish brown (10YR5/6), stiff, moist, PP = 1.5 tsf, some medium-grained sand, many tree branches and logs [FILL] |
| | 2 – 4 | FAT CLAY (CH), gray (10YR4/1), stiff, moist, PP = 1.5 tsf, clean [NATIVE] |
| | | -Groundwater was not encountered at time of test pits |

# TEST PIT LOG

| Green Tree<br>999 Leisure Town Road<br>Vacaville, CA.<br>16018.000.000 | | Logged By: Jerry Chen<br>Logged Date: May 2, 2019 |
|---|---|---|
| **Test Pit Number** | **Depth (ft.)** | **Description** |
| T-3 | 0 – 1 | FAT CLAY (CH), dark gray (10YR4/1), stiff to very stiff, dry, PP = 1.25 – 2.5 tsf, rootlets [FILL] |
| | 1 – 4 | FAT CLAY (CH), dark gray (10YR4/1), very stiff, moist, PP = 2.5 tsf, few rootlets decreasing with depth [NATIVE] |
| | 4 – 6 | CLAY (CL), dark grayish brown (10YR4/2), hard, moist, PP > 4.5 tsf |
| | | -Groundwater was not encountered at time of test pits |

# TEST PIT LOG

| Green Tree<br>999 Leisure Town Road<br>Vacaville, CA.<br>16018.000.000 | | Logged By: Jerry Chen<br>Logged Date: May 2, 2019 |
|---|---|---|
| Test Pit Number | Depth (ft.) | Description |
| T-4 | 0 – 1½ | FAT CLAY (CH), dark gray (10YR4/1), dry, rootlets [FILL] |
| | 1½ – 7 | FAT CLAY (CH), dark gray (10YR4/1), stiff to very stiff, moist, PP = 1.5 – 2.25 tsf [NATIVE] |
| | | -Groundwater was not encountered at time of test pits |

# TEST PIT LOG

| Green Tree<br>999 Leisure Town Road<br>Vacaville, CA.<br>16018.000.000 | | Logged By: Jerry Chen<br>Logged Date: May 1, 2019 |
|---|---|---|
| Test Pit Number | Depth (ft.) | Description |
| T-5 | 0 – 1 | SANDY loam (SP), topsoil, dry [FILL] |
| | 1 – 2½ | SILTY CLAY (CL-ML), dark brown, hard, dry, PP > 4.5 tsf, with clay and pea gravel [FILL] |
| | 2½ – 3 | POORLY GRADED SAND (SP), light brown, slightly moist [FILL] |
| | 3 – 3½ | Pea GRAVEL (GP) [FILL] |
| | 3½ – 4 | FAT CLAY (CH), dark brown, medium stiff, moist, PP = 2.25 tsf [NATIVE] |
| | | -Groundwater was not encountered at time of test pits |

# TEST PIT LOG

| Green Tree<br>999 Leisure Town Road<br>Vacaville, CA.<br>16018.000.000 | | Logged By: Jerry Chen<br>Logged Date: May 1, 2019 |
|---|---|---|
| Test Pit Number | Depth (ft.) | Description |
| T-6 | 0 – ½ | CLAYEY SILT (ML), topsoil, dark brown [FILL] |
| | ½ – 2 | SILT (ML), dark brown (10YR3/1), hard, slightly moist, PP > 4.5 tsf, A-horizon, with sand and clay [NATIVE] |
| | 2 – 3½ | LEAN CLAY (CL), very dark grayish brown (10YR3/2), hard, slightly moist, PP = 4.25 tsf, B-horizon, laminated blocky structure |
| | 3½ – 5 | LEAN CLAY (CL), dark gray (10YR4/2), hard, moist, PP > 4.5 tsf, indurated, late Holocene/early Pleistocene |
| | | -Groundwater was not encountered at time of test pits |

# TEST PIT LOG

| Green Tree<br>999 Leisure Town Road<br>Vacaville, CA.<br>16018.000.000 | | Logged By: Jerry Chen<br>Logged Date: May 1, 2019 |
|---|---|---|
| Test Pit Number | Depth (ft.) | Description |
| T-7 | 0 – 2 | CLAY/SILTY SAND (SC), very dark grayish brown (10YR3/2) [NATIVE] |
| | 2 – 7 | POORLY GRADED SAND (SP), dark brown (7.5YR3/3), slightly moist, fine-grained sand with few clay and gravel, grades to medium- to coarse-grained at approximately 4 feet [NATIVE] |
| | | -Groundwater encountered at approximately 7 feet below ground surface |

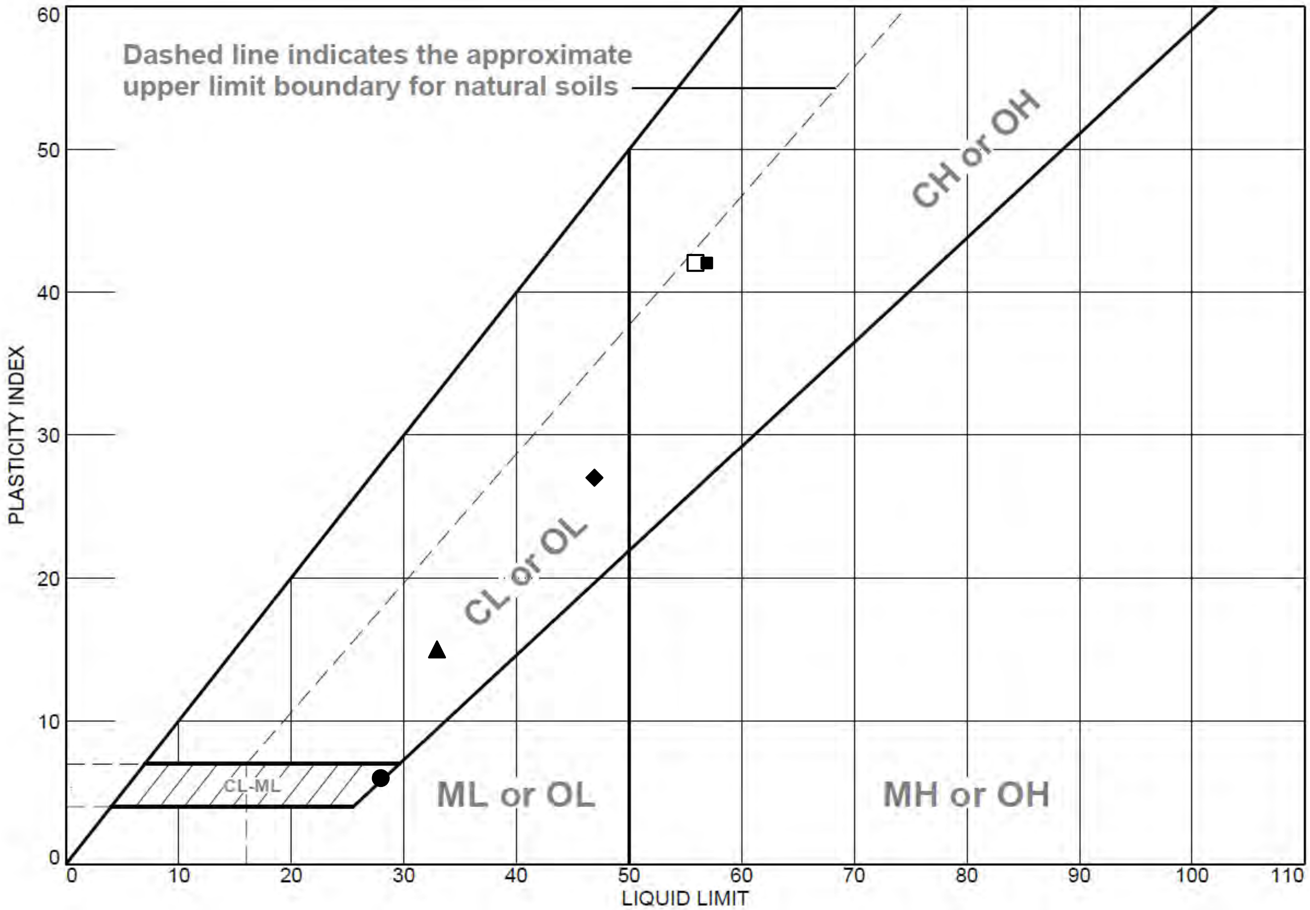
# TEST PIT LOG

| Green Tree<br>999 Leisure Town Road<br>Vacaville, CA.<br>16018.000.000 | | Logged By: Jerry Chen<br>Logged Date: May 1, 2019 |
|---|---|---|
| Test Pit Number | Depth (ft.) | Description |
| T-8 | 0 – 1½ | SILTY SAND (SM), very dark grayish brown, significant gravel content [FILL] |
| | 1½ – 1¾ | CLAYEY GRAVEL (GC), fine to medium, rounded to subrounded gravel, some medium plasticity fines [FILL] |
| | 1¾ – 3 | LEAN CLAY (CL), dark brown (10YR3/3), hard, slightly moist, PP > 4.5 tsf, A-horizon [NATIVE] |
| | 3 – 6 | LEAN CLAY (CH), gray (10YR5/1) mottled with brown, stiff to very stiff, slightly moist, PP = 1.5 – 3.0 tsf |
| | | -Groundwater was not encountered at time of test pits |

# APPENDIX C

## Laboratory Test Data

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ▲ | See exploration logs | 33 | 18 | 15 | | | |
| ◆ | See exploration logs | 47 | 20 | 27 | | | |
| □ | See exploration logs | 56 | 14 | 42 | | | |
| ● | See exploration logs | 28 | 22 | 6 | | | |
| ■ | See exploration logs | 57 | 15 | 42 | | | |

**Project No.:** 16018.000.000 **Date:** 5/13/2019

**Project Name:** Green Tree

**Project location:** Vacaville, California

**Client:** Greetree Development Group

▲ **Sample Number:** TP1@1.7-4.7 **Depth:** 1.7-4.7 ft

◆ **Sample Number:** TP2@0-2 **Depth:** 0-2 ft

□ **Sample Number:** TP4@1.5-7 **Depth:** 1.5-7 ft

● **Sample Number:** TP5@1-2.5 **Depth:** 1-2.5 ft

■ **Sample Number:** TP5@3.25-4 **Depth:** 3.25-4 ft

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**Remarks:**

▲ PI: ASTM D4318, Wet Method

◆ PI: ASTM D4318, Wet Method

□ PI: ASTM D4318, Wet Method

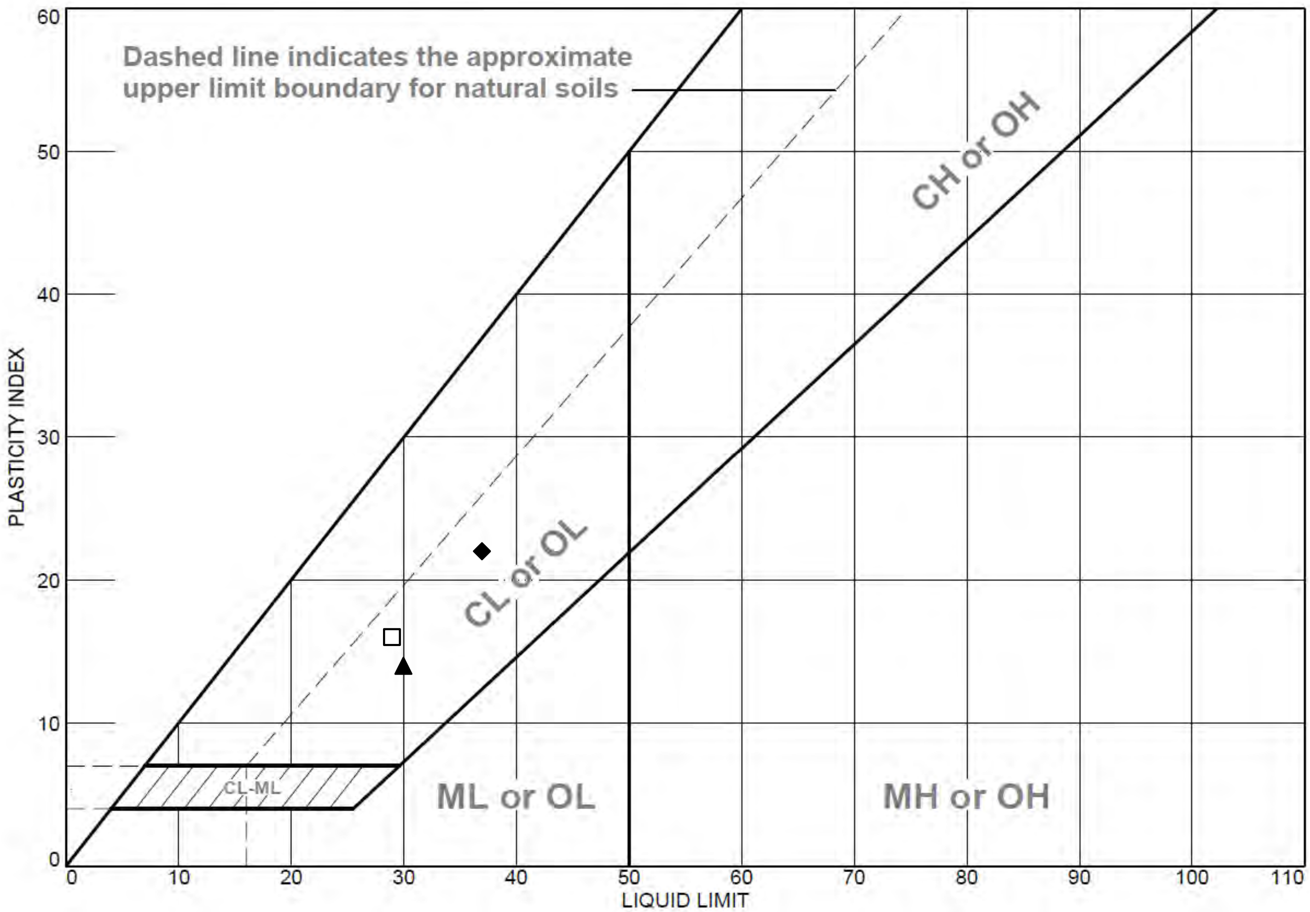
● PI: ASTM D4318, Wet Method

■ PI: ASTM D4318, Wet Method

Tested By: M. Quasem Checked By: G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ▲ | See exploration logs | 30 | 16 | 14 | | | |
| ◆ | See exploration logs | 37 | 15 | 22 | | | |
| □ | See exploration logs | 29 | 13 | 16 | | | |
| | | | | | | | |
| | | | | | | | |

**Project No.:** 16018.000.000 **Date:** 5/13/2019

**Project Name:** Green Tree

**Project location:** Vacaville, California

**Client:** Greentree Development Group, Inc.

▲ **Sample Number:** TP6@3.5-5 **Depth:** 3.5-5 ft

◆ **Sample Number:** TP8@1.7-3 **Depth:** 1.7-3 ft

□ **Sample Number:** TP8@3"-5" **Depth:** 3-5 in

**Remarks:**

▲ PI: ASTM D4318, Wet Method

◆ PI: ASTM D4318, Wet Method

□ PI: ASTM D4318, Wet Method

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Tested By: M. Quasem Checked By: G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

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