Appendix E:
Geology and Soils Supporting Information

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E.1 - Geotechnical Report

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PALMER
WALNUT CREEK, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Marshall Torre
SummerHill Homes
3000 Executive Parkway, Suite 450
San Ramon, CA 94583

PREPARED BY
ENGEO Incorporated

September 1, 2020

PROJECT NO.
17567.000.000

ENGEO
Expect Excellence

Project No.
17567.000.000

September 1, 2020

Mr. Marshall Torre
SummerHill Homes
3000 Executive Parkway, Suite 450
San Ramon, CA 94583

Subject: Palmer
Walnut Creek, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Torre:

With your authorization, we completed a preliminary geotechnical exploration for your proposed residential project (Palmer) at 2740 Jones Road in Walnut Creek, California, as outlined in our agreement dated July 27, 2020. The accompanying report presents our field exploration data with our conclusions and preliminary recommendations regarding the proposed residential project.

Based on our preliminary assessments, it is our opinion that the proposed residential development is feasible from a geotechnical standpoint. A design-level geotechnical exploration report should be conducted to develop design-level recommendations once the final detailed land plans 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 or comments regarding this preliminary report, please call and we will be glad to discuss them with you.

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

Sincerely,

ENGEO Incorporated

Anne Robertson, EIT

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical report for your proposed residential development at 2740 Jones Road in Walnut Creek, California. We performed the following scope of services:

- Review of published geologic maps, aerial photographs, historic topography, and publically available geologic and groundwater information in the area.
- Limited subsurface explorations.
- Data analysis and development of preliminary geotechnical recommendations.
- Report preparation.

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

1.2 PROJECT LOCATION AND DESCRIPTION

The site location is presented on Figure 1, the Vicinity Map. The site address is 2740 Jones Road in Walnut Creek and is associated with Assessor's Parcel Number (APN) 172-012-020-1. The parcel is approximately 5.5 acres in size and is currently occupied by a private school and its associated facilities such as sport field and courts, a swimming pool, and surface parking lots. As shown on the Site Plan, Figure 2, the site is bounded by Jones Road on the west, Oak Road on the east, two apartment buildings and one condominium complex to the north, and the Oak Road Villas condominium to the south.

Based on conversations with you and review of the preliminary site plans and renderings, we understand that the proposed development will consist of three-story multi-family townhome/condominium development, associated parking, and landscape areas. We anticipate the development to consist of three-story at grade structures with no below-grade levels. Grading plans were not available at the time of this report preparation, but we anticipate minor cuts and fills will be conducted to accommodate the development. We anticipate that the structures will be of wood-frame construction. Therefore, in our opinion, the building loads are estimated to be light to moderately light.

2.0 FINDINGS

2.1 HISTORICAL AERIAL PHOTOGRAPH REVIEW

Review of historical aerial photographs found the property associated with parcel 172-012-020-1 was primarily used for agricultural purposes prior to 1939. Historical records indicate that the property has been occupied by a private school, Palmer School for Boys and Girls, since 1939.

Review of historical aerial photographs from the period 1939 through 2019 show that the site has remained relatively unchanged from current conditions since 1968, aside from the removal of two additional pools on the property. Between the years of 1939 and 1968, the property transitioned from a sparsely developed parcel with clusters of small buildings, amidst open space and orchards, to the current development.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Regional Geology

The site is located within the Coast Ranges geomorphic province of California. The Coast Ranges geomorphic province is characterized 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 topography 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.

2.2.2 Geology

More specifically, the site is located within the west portion of Ygnacio Valley. Ygnacio Valley represents an area of low relief, between Mount Diablo within the Diablo Range to the east and the Briones Hills within the East Bay Hills to the west. Both Witter (2006) and Helley (1997) map the geology at the site as alluvial fan deposits; however, Witter interprets the deposits as Holocene and Helley interprets them as Pleistocene. Dibblee (2005) interprets the map surficial deposits as a combination of Holocene and Pleistocene. The alluvial deposits are commonly unconsolidated, heterogeneous, poorly to moderately sorted, irregularly interbedded clays and silts containing discontinuous lenses of sand, silty clay, and gravel. According to Witter (2006), the alluvial deposits underlying the site are considered of moderate liquefaction susceptibility. Our relevant experience in the area indicates that the alluvium may consist of moderately to highly expansive clay to sandy clay. Bedrock exposed in the Briones Hill directly west of the site generally comprises units of the Monterey Formation and Martinez Group.

2.2.3 Seismicity

The Bay Area area contains numerous active earthquake faults. Nearby active faults include the Contra Costa (Larkey) fault, which has a nearest rupture distance of approximately 1.3 miles from the project site and the Franklin fault, which has a nearest rupture distance of approximately 1.6 miles west of the project site. An active fault is defined by the California Geologic Survey as one that has had surface displacement within Holocene time (about the last 11,000 years) (Bryant and Hart, 2007).

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

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded within the San Francisco Bay Region. The Uniform California Earthquake Rupture Forecast (UCERF 3)

(Field et al, 2015) estimates the 30-year probability for a magnitude 6.7 or greater earthquake in Southern California at approximately 72 percent, considering the known active seismic sources in the region.

To determine nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the USGS Unified Hazard Tool* and disaggregated the hazard at the peak ground acceleration (PGA) and at period of 0.5 seconds for 2475-year return period, with the resulting faults listed below in Table 2.2.3-1.

TABLE 2.2.3-1: Active Faults Capable of Producing Significant Ground Shaking at the Site, Latitude: 37.924022° **Longitude:** -122.058775°

SOURCE	R_{RUP}		MOMENT MAGNITUDE M_W
	(KM)	(MILES)	
Contra Costa (Larkey) [1]	2.02	1.26	6.29
Franklin [1]	2.54	1.58	7.10
Mount Diablo Thrust North CFM [1]	3.62	2.25	7.15
Contra Costa Shear Zone (connector) [4]	4.93	3.06	7.10
Concord [2]	5.08	3.16	6.65
Contra Costa (Lafayette) [1]	5.12	3.18	7.02
Concord [1]	8.81	5.47	6.57
Calaveras (No) [0]	9.83	6.11	7.03
Clayton [0]	10.43	6.48	6.92
Hayward (No) [1]	16.99	10.56	7.33

*USGS Unified Hazard Tool - Edition: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

2.3 FIELD EXPLORATION

We retained a truck-mounted rig to advance two cone penetration tests (CPTs) and one seismic cone penetration test (SCPT) to a maximum depth of approximately 78 feet below the ground surface (bgs). Figure 2 presents these exploration locations. The CPT equipment has a 20-ton compression-type cone with a 10-square-centimeter (cm^2) base area and a friction sleeve with a surface area of 150 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 standards (D3441). Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and dynamic pore pressure (U).

Pore pressure dissipation tests were conducted at all three locations. The CPT cone was halted at select depths, and the variation of the penetration pore pressure with time was measured until the pore pressure stabilized. Shear wave velocity (Vs) tests were conducted at 1-meter intervals in 1-SCPT2. The SCPT cone was halted at select depths, and the time needed for shear waves to travel from the ground to a geophone in the SCPT cone was recorded and used to calculate Vs.

Appendix A presents the CPT data. The CPT and SCPT holes were permitted and backfilled with cement grout upon completion per requirements of Contra Costa County Health Services.

2.4 SURFACE CONDITIONS

According to published topographic maps and Google Earth elevations, the Property is relatively level at an elevation of approximately 93 to 95 feet, based on the North American Vertical Datum of 1988 (NAVD 88). The project site contains several structures, including school facilities, a tennis court, a basketball court, a swimming pool, a field, and a parking lot. The site is vegetated, with several large deciduous trees throughout the Property and additional smaller trees and bushes along the southern and western edges of the site.

2.5 SUBSURFACE CONDITIONS

Alluvial deposits were found in 1-CPT1, 1-SCPT2, and 1-CPT3, and the granular materials in the upper 40 feet appear to be discontinuous layers at different elevations across the site. Between 40 and 50 feet bgs, a dense to very dense sand and gravelly sand layer was found in all three borings.

Based on CPT soil behavior type correlations (SBT) by Robertson (2009, 2016), the native material encountered in 1-CPT1 was primarily composed of layers between approximately 1 and 12 feet thick of medium-dense to loose sand and medium stiff to very soft silt-like and clay-like material. A medium dense to very dense sand layer was found between 18 and 24 feet bgs, and at approximately 48 feet bgs.

The material encountered in 1-SCPT2 was mostly composed of discontinuous layers up to 6 feet in thickness of medium stiff to very soft silty and clayey mixtures with interbedded pockets of loose to very dense sand deposits. A lens of dense to very dense sand was found between depths of 41 and 49 feet bgs.

The native material found in the upper 40 feet of 1-CPT3 was primarily a continuous deposit of soft to very soft silty clay and clayey silt. A lens of dense to very dense sand and gravelly sand was found between depths of 41 and 49 feet bgs.

Consult the Site Plan and exploration logs for specific subsurface conditions at each location. We include the CPT exploration report in Appendix A. The report contains the soil behavior type classification, calculated using measurements of cone tip resistance, skin friction, and excess pore pressure. The report graphically depicts the subsurface condition interpretation at the time of the exploration.

2.6 GROUNDWATER CONDITIONS

During The CPT explorations, we performed pore pressure dissipation (PPD) tests, as described in Appendix A, to infer approximate groundwater table elevations, summarized in the table below. Elevations are based on NAVD88.

TABLE 2.6-1: Groundwater Observations

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)
1-CPT1	18.8	75.2
1-SCPT2	14.5	79.5
1-CPT3	10.6	84.4

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. Future irrigation may cause an overall rise in groundwater levels.

3.0 DISCUSSION AND CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the preliminary geotechnical recommendations in this report are properly addressed.

The primary geotechnical concerns that could affect development on the site are liquefaction-induced settlement, potential consolidation of compressible material, potential expansive soil, and areas of shallow ground water. The preliminary recommendations included in this report should be utilized for project planning purposes and are intended for the areas of the site that will be developed with structural improvements. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls. Prior to development, we should be retained to provide a design-level geotechnical report for the development, which would include additional CPTs, borings and laboratory testing to provide data for preparation of specific recommendations regarding site grading, foundations, and drainage for the proposed development.

We evaluated the site was with respect to known geologic and other hazards common to the area. The primary hazards and the risks associated with these hazards with respect to the planned development are discussed in the following sections of this report.

3.1 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 and ground lurching. The following sections present information regarding these hazards as they apply to the site. Based on site observations, topographic and lithologic data, subsurface data, and regional geology, the risk of regional subsidence or uplift, lateral spreading, landslides, tsunamis, flooding or seiches is considered low to negligible at the site.

3.1.1 Ground Rupture

The site is not located within a State of California Earthquake Fault Hazard Zone and no known faults cross the site (California Geologic Survey Walnut Creek Quadrangle, 1993). Therefore, it is our opinion that ground rupture is unlikely at the subject property.

3.1.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the Northern California region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the 2019 California Building Code (CBC) requirements, as a minimum.

3.1.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained 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 cause liquefaction of susceptible soil.

Review of the US Geologic Survey (USGS) liquefaction susceptibility map (Knudsen et al, 2000) for this area indicates that the site is located within an area with moderate susceptibility to liquefaction (Figure 5). We assessed liquefaction potential at the site by performing liquefaction analyses utilizing data obtained from the CPT probes. We assigned a design groundwater level of 10 feet below the existing ground surface, a peak ground acceleration (PGA) of 0.88g, and a maximum moment magnitude (M_w) of 7.0. Our analyses were based on guidelines provided in DMG Special Publication 117A (2008) and methods developed by the National Center for Earthquake Engineering Research (1998), Moss et al. (2006), Idriss and Boulanger (2008), and Boulanger and Idriss (2014). We calculated the vertical settlements based on the procedure recommended by Zhang et al (2002).

Based on our limited subsurface explorations and liquefaction analysis (Appendix B), we estimate that a maximum of 4¾ inches of total liquefaction-induced settlement may occur during a maximum considered event (MCE) earthquake. This amount of total liquefaction-induced settlement corresponds to less than 2½ inches of differential settlement over a horizontal distance of 40 feet. Based on our experience, it is our understanding that this amount of differential settlement can be accommodated by the structural engineer in the foundation design. Additional subsurface exploration, collection of soil samples, and laboratory testing during the design-level study will better delineate the areas with a potential for liquefaction, and may help to optimize estimates of liquefaction-induced settlement magnitude.

3.1.4 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 diminish with distance from the slope. Based on site topography and subsurface conditions, it is our opinion that the risk of lateral spreading at the site is low.

3.2 COMPRESSIBLE SOIL

Soil is subject to consolidation settlement when a new loading scenario is introduced by structures, earthworks or equipment. The amount of consolidation settlement is dependent on the magnitude and duration of the applied load, the shape and size of the applied load area, the depth, thickness and the stress history of the compressible soil. The time required for primary consolidation settlement to occur is highly dependent on the permeability of the deposit. Consequently, sandy soil will settle almost immediately, whereas clayey soil will settle much more slowly.

Based on review of the CPT data, it is our opinion that the subsurface clay and silt mixture material at depths of 0 to 27 feet and 31 to 37 feet are very soft to soft and may undergo consolidation under loads from additional fill required to grade the site and proposed buildings. Based on our knowledge and experience, it is our opinion that a portion of the consolidation will occur during construction, and that the remaining differential consolidation-induced settlement can be accommodated in the structural foundation design.

Laboratory testing and additional analysis should be performed in the design-level study to confirm the magnitude and extent of the potentially compressible material and the potential consolidation-induced settlement.

3.3 EXISTING NON-ENGINEERED FILL

We cannot determine the extent of the existing non-engineered fill based on present exploration data. Based on our review of aerial photos and historical records, it appears as though the site has had a history of agricultural activities prior to 1960s. From the review of historic aerial photos (https://www.historicaerials.com) and historic topographic maps dating to the 1890s, the topography of the site does not appear to have changed significantly at any point. Therefore, we expect to encounter minor (less than 5 feet) of non-engineered fill across most of the site. There may be localized areas of deeper fill beneath existing and previous structure foundations, such as under the three swimming pools that were constructed on the site since the 1940s. We recommend that a design-level geotechnical exploration with borings be conducted to evaluate the existence and extent of potential non-engineered fill.

Non-engineered fill can undergo excessive settlement, especially under new fill or building loads. In general, undocumented fill should be excavated, and if deemed suitable for reuse, replaced as engineered soil fill. The extent and quality of existing fill should be evaluated at the time of design-level study and mitigated during remedial grading activities. Based on the available data, it is our opinion that significant amounts of undocumented fill will not be present across the majority of the site.

3.4 EXPANSIVE SOIL

Sampling and testing for expansiveness potential was not performed as part of this preliminary study. While we did not observe potentially expansive soil during the site exploration, the CPT data suggests clayey soil is present at the site, which may exhibit expansive potential based on their flood plain origin. The presence of potentially expansive soil should be further evaluated during the design-level geotechnical exploration.

Expansive soil 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 soil 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 lime treatment in the upper 18 inches of the building pad to reduce the expansion potential of the onsite soil.

To mitigate potential damage from expansive soil, selective grading or blending would be necessary to create relatively low expansion potential surface conditions. Selective grading typically involves careful planning of cut and fill along with blending and disking to mix soil types and create low expansion soil conditions. Creating low expansion conditions can necessitate more complicated cut/fill operations and multiple blending operations that can increase earthwork costs.

3.5 CORROSIVITY CONSIDERATIONS

Sampling and testing for corrosion potential was not performed as part of this preliminary study. Representative samples of the foundation grade soil should be obtained 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 the 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 2019 CBC. 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.

3.6 SHALLOW GROUNDWATER

We summarized groundwater conditions at this site in Table 2.6-1. The groundwater table was found at a depth of ranging from 10 to 19 feet below grade depending on location. Based on the groundwater data and our experience, we believe the groundwater level may rise in the future. As such, we recommend that a groundwater level of 10 feet below ground surface be considered for preliminary design. Based on the groundwater levels interpreted in the CPTs and historical groundwater depth for the site, it appears that shallow groundwater beneath the site could potentially affect the proposed development. Shallow groundwater can:

- Impede grading activities.
- Require construction dewatering during grading and improvement.
- Cause moisture damage to sensitive floor coverings.
- Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.

Based on the groundwater measurements observed during our CPT explorations, we believe that the foundation construction process should be just above the existing static groundwater; however, site utilities may extend below those depths and encounter static groundwater during construction.

Existing fill removal and any deep utility trench excavation may encountered groundwater. Shallow groundwater condition should be considered during design of utilities, site grading, and excavation of the utility trenches and foundation. The project contractor should evaluate the site conditions and selected properly designed dewatering, shoring systems, and other as necessary during site grading and construction. Seasonal fluctuations can potentially raise and lower the groundwater level from the depths observed during the time of our explorations.

3.7 FLOODING

Flood Insurance Map by FEMA (Figure 6) indicates that the project site is outside of mapped flood zones within its boundaries. Therefore, it is our opinion that the risk of flooding is low at this site. The Civil Engineer should review the pertinent information relating to flood levels for the project site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

3.8 2019 CBC SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions encountered and shear wave velocity measurements, we characterized the site as Site Class D in accordance with the 2019 CBC. We provide the 2019 CBC seismic design parameters in Table 3.8-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.8-1: 2019 CBC Seismic Design Parameters,
Latitude: 37.924022° **Longitude:** -122.058775°

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.971
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.640
Site Coefficient, F_a	1.00
Site Coefficient, F_v	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.971
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.314
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.804
Site Coefficient, F_{PGA}	1.10
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.885

*Requires site-specific ground motion hazard analysis per ASCE 7-16 Section 11.4.8

Considering the proposed residential development, we estimate the fundamental periods of the proposed structures to be less than $1.5T_s$ (where T_s is 0.55 seconds for this project). Therefore, the structural engineer may consider exception(s) of Section 11.4.8 of ASCE 7-16 as follows:

> "A ground motion hazard analysis is not required for structures… where, structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) of ASCE 7-16 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with Eq. (12.8-3) of ASCE 7-16 for $1.5T_s < T \leq T_L$."

However, based on our experience, a site-specific seismic hazard analysis can optimize the spectral values at the short period range. We recommend that we collaborate with the structural engineer of record to further evaluate the effects of taking the exceptions on the structural design and identify the need for performing a site-specific seismic hazard analysis. We can provide a scope for site-specific seismic hazard analysis and ground motion study under separate cover, if needed.

3.9 PRELIMINARY FOUNDATION RECOMMENDATIONS

We anticipate the proposed residential development can be supported by a post-tensioned mat foundation. Based on our understanding of the presence of expansive soil in the area, as well as the potential for liquefaction-induced settlements of up to 4¾ inches, we do not recommend the use of conventional footings with a concrete slab-on-grade. We provide preliminary recommendations for the post-tensioned mat foundation below.

3.9.1 Post-tension Mat Foundation

We recommend that the proposed residential structures be supported on post-tensioned (PT) mat foundations bearing on engineered fill. 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 below. 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.

3.9.2 Slab Moisture Vapor Reduction

When buildings are constructed with a concrete slab-on-grade, including post-tensioned mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we typically recommend a moisture retarder system to reduce, but not stop, water vapor transmission upward through the slab-on-grade. This generally involves installing a Class A vapor retarder membrane (ASTM E1745, latest edition), underlain by 4 inches of clean crushed rock. The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane. Lastly, we typically recommend a concrete water-cement ratio for slabs-on-grade of no more than 0.50, special inspections during concrete placement, and moist curing slabs for a minimum of 3 days (or other equivalent curing specified by the structural engineer).

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

4.1 GENERAL SITE CLEARING AND DEMOLITION

After demolition of the existing buildings, paving, and associated improvements, the site should be cleared of all obstructions, including existing foundations, and debris. Any existing underground utilities within the proposed development area should be identified and removed entirely including pipes and their backfill. Depressions resulting from the removal of underground obstructions extending below the proposed finish grades should be cleared and backfilled with suitable material compacted to the recommendations presented in Section 5.3.

Areas containing surface vegetation or organic laden topsoil within the areas to be improved should be stripped to an appropriate depth to remove these materials. The amount of actual stripping and tree root removal should be determined in the field by the Geotechnical Engineer at the time of construction. Subject to approval by the Landscape Architect, strippings and organically contaminated soil can be used in landscape areas. Otherwise, such soil 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.

Stripping and demolition below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be cleaned, scarified, moisture conditioned, and backfilled with suitable material compacted to the recommendations presented in the Fill Compaction section. No loose or uncontrolled backfilling of depressions resulting from demolition and stripping should be permitted.

4.1 NON-ENGINEERED FILL

As described previously, we expect the presence of non-engineered fill at the site. Where applicable, existing fill, existing utility trench backfill, existing foundation backfill, and existing landscape materials are considered undocumented and should be subexcavated to expose underlying competent native soil that is approved by the Geotechnical Engineer. If in a fill area, the base of the excavations should be processed, moisture conditioned, as needed, and compacted in accordance with the recommendations for engineered fill.

4.2 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 soil (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.

4.3 FILL COMPACTION

We recommend removal of existing fills (if encountered during grading), stripping of organics, scarification, moisture conditioning, and compaction of the soil prior to fill placement, following cutting operations, and in areas left at grade. For land planning and cost estimating purposes, the following compaction control requirements should be anticipated for general fill areas.

- Test Procedures: ASTM D-1557.
- Required Moisture Content: Not less than 3 percentage points above optimum moisture content.
- Minimum Relative Compaction: Not less than 90 percent.

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

4.4 FLEXIBLE PAVEMENT

Based on our experience with nearby developments, we judged an R-value of 5 to be appropriate for preliminary pavement design. Using a preliminary design R-value of 5 and Procedure 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety), we developed the following pavement sections presented in Table 4.4-1 below.

TABLE 4.4-1: Preliminary Pavement Sections

TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)
5	3	10
6	3½	13
7	4	16

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

4.5 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 CBC 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.

5.0 DESIGN-LEVEL GEOTECHNICAL REPORT

This report presents geotechnical feasibility findings and considerations for the planned residential development. A design-level geotechnical exploration should be performed when development plans are finalized. The purpose of the design-level exploration is to further evaluate the liquefaction, liquefaction-induced settlement, potential for excessive amounts of fill, compressible soil and other geotechnical hazards. Specific recommendations for site grading, ground improvement, and the design and construction of foundations and utilities should be included in the design-level geotechnical report.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This preliminary report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this preliminary report are solely professional opinions.

The professional staff of ENGEO strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This preliminary report is based upon field and other conditions discovered at the time of preparation of ENGEO's report. This document must not be subject to unauthorized reuse that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time. Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-study area 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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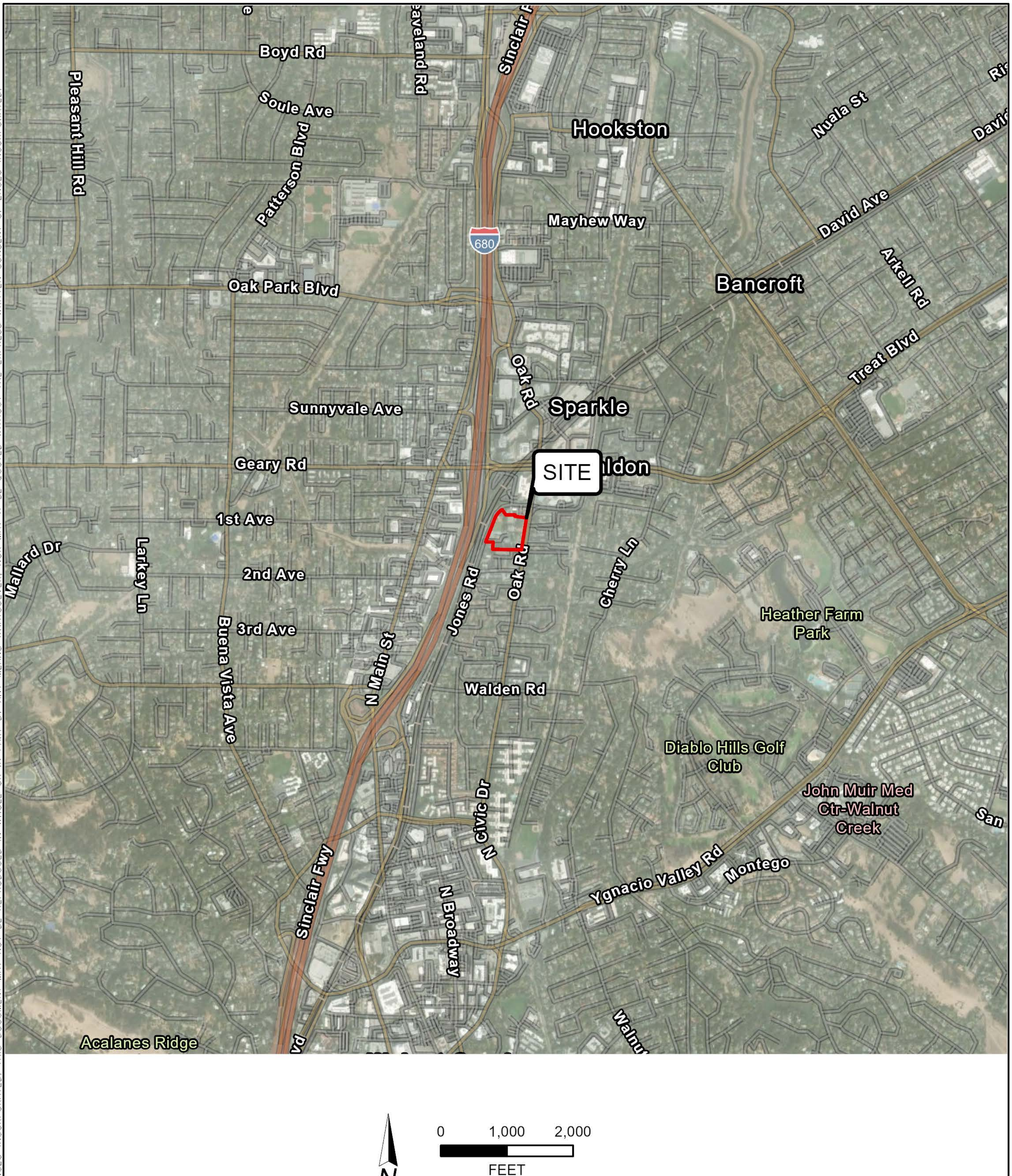
FIGURES



BASEMAP SOURCE: ESRI MAPPING SERVICE 2017

ENGEO Expect Excellence	VICINITY MAP PALMER WALNUT CREEK, CALIFORNIA	PROJECT NO.: 17567.000.000		FIGURE NO. 1
		SCALE: AS SHOWN		
		DRAWN BY: JV	CHECKED BY: RHB	

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\17567\GEOTECH\PGEX\17567.APRX
LAYOUT: VICINITY USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT3 CONE PENETRATION TEST (ENGEO, 2020)

N

0 50 100

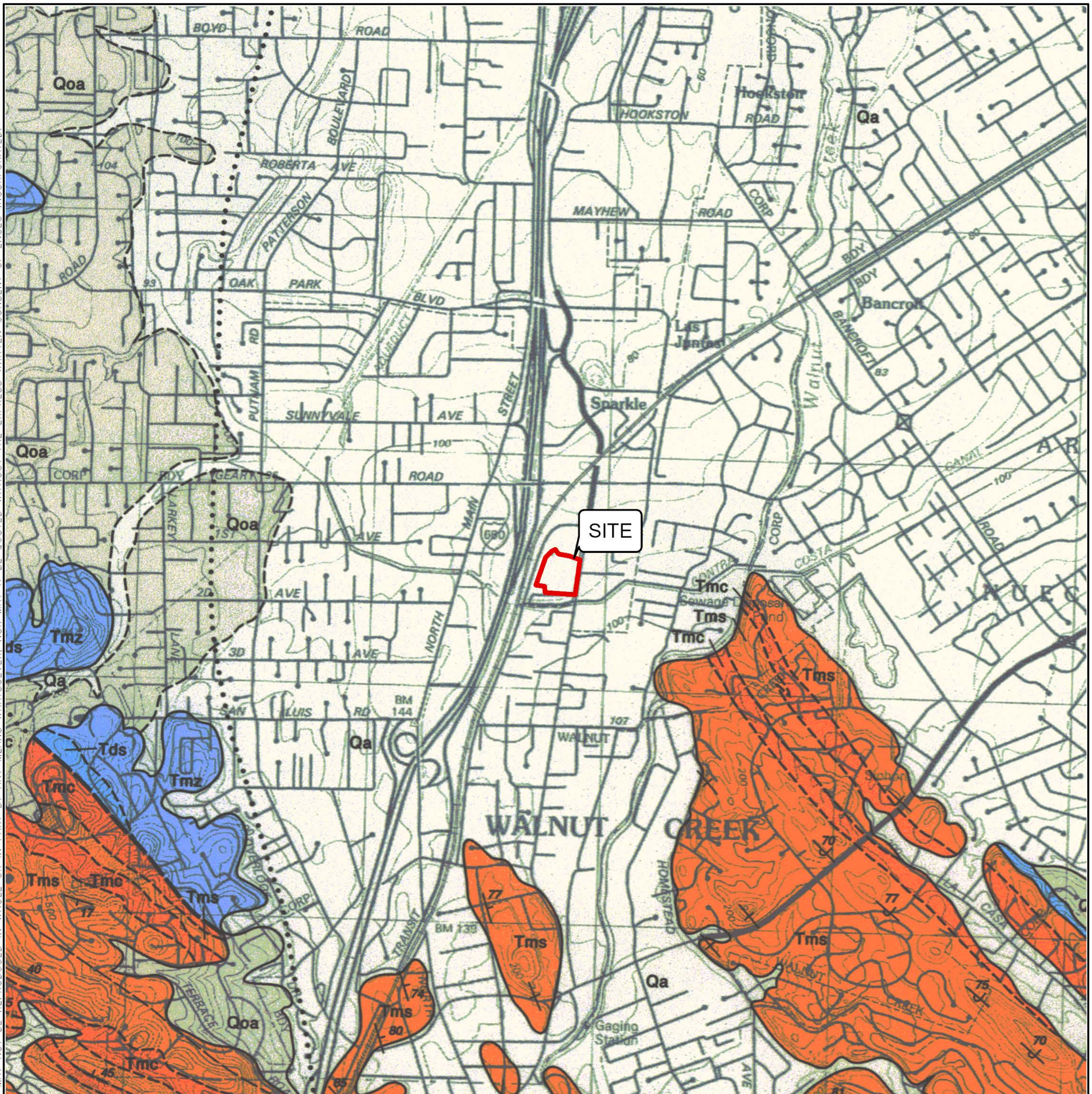
FEET

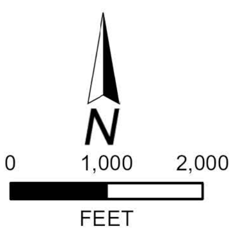
BASEMAP SOURCE: ESRI MAPPING SERVICE 2017

ENGEO Expect Excellence	SITE PLAN PALMER WALNUT CREEK, CALIFORNIA	PROJECT NO. : 17567.000.000 SCALE: AS SHOWN DRAWN BY: JV CHECKED BY: RHB	FIGURE NO. 2

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\17567\GEOTECH\PGEX\17567.APRX
LAYOUT: SITE PLAN USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR





EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qa	ALLUVIAL GRAVEL
Qoa	ALLUVIAL GRAVEL AND SAND
Tms	SANDSTONE
Tmc	CLAY SHALE/SILTSTONE
Tmz	CLAY SHALE/SILTSTONE
Tds	MUIR SANDSTONE

BASEMAP SOURCE: DIBBLEE, 2005

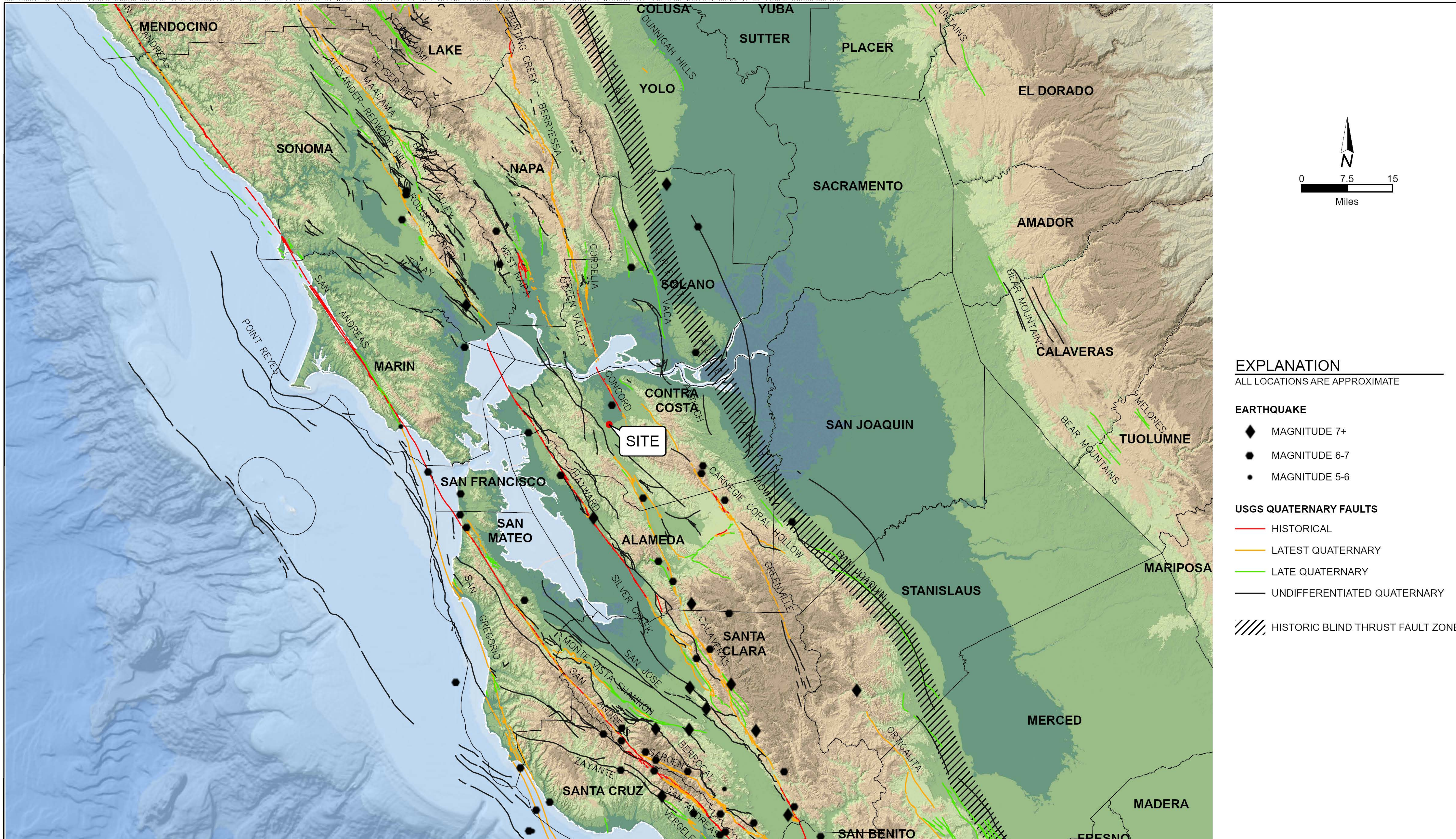
ENGEO Expect Excellence

REGIONAL GEOLOGIC MAP
PALMER
WALNUT CREEK, CALIFORNIA

PROJECT NO.: 17567.000.000	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: RHB

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\17567\GEOTECH\PGEX\17567.APRX
LAYOUT: GEOLOGIC USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR

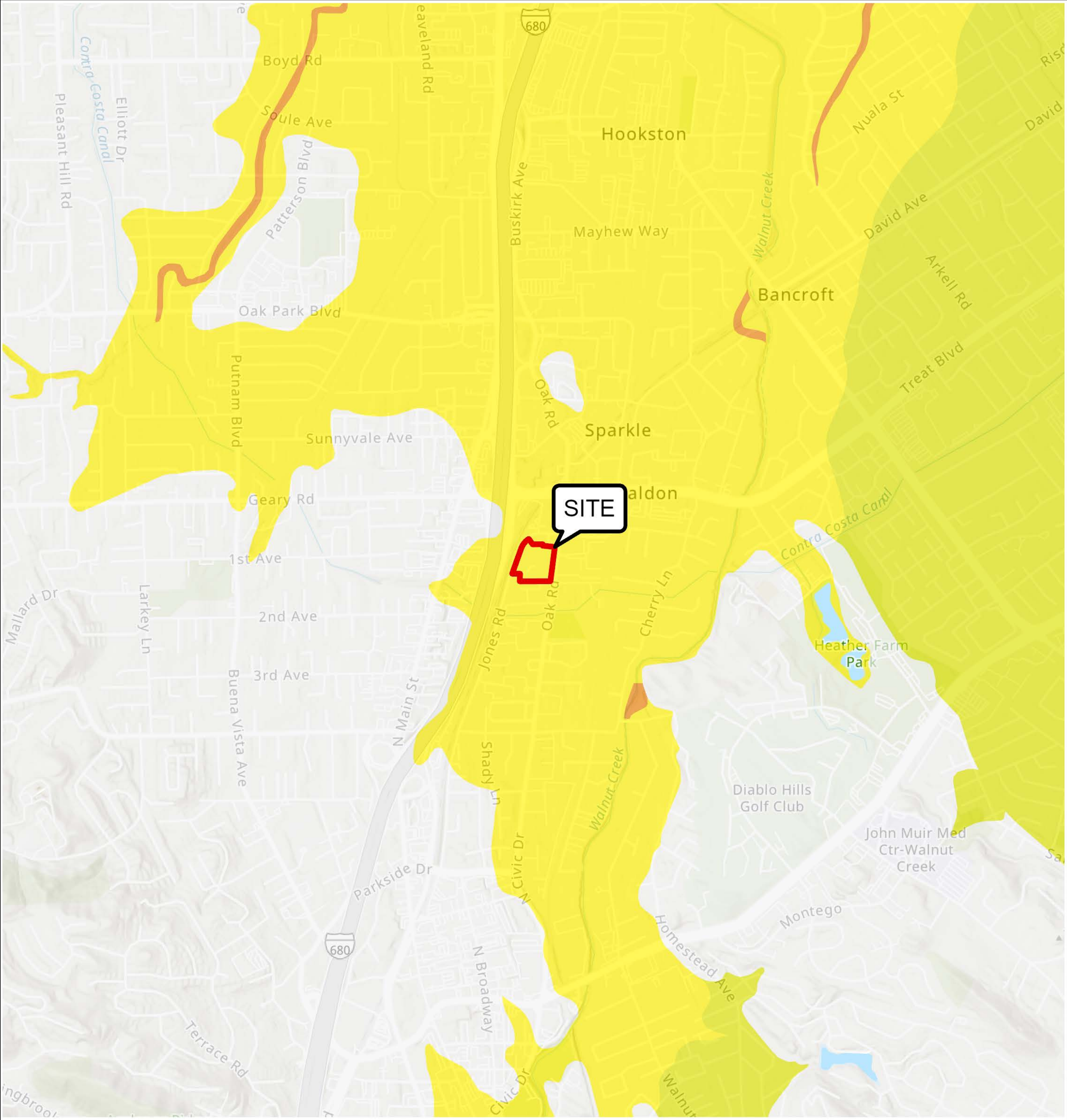


BASE MAP SOURCE
ESRI, GEBCO, DELORME, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

ENGEO Expect Excellence	REGIONAL FAULTING AND SEISMICITY PALMER WALNUT CREEK, CALIFORNIA	PROJECT NO.: 17567.000.000 SCALE: AS SHOWN DRAWN BY: JV CHECKED BY: RHB	FIGURE NO. 4

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\17567\GEOTECH\PGEX\17567.APRX
LAYOUT: FAULT&SEISMICITY USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

LIQUEFACTION SUSCEPTIBILITY

- VERY HIGH
- HIGH
- MODERATE
- LOW
- VERY LOW

BASEMAP SOURCE: ESRI MAPPING SERVICE AND USGS, 2000

ENGEO
Expect Excellence

LIQUEFACTION SUSCEPTIBILITY MAP	PROJECT NO.: 17567.000.000	FIGURE NO. 5
PALMER	SCALE: AS SHOWN	
WALNUT CREEK, CALIFORNIA	DRAWN BY: JV / CHECKED BY: RHB	

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\17567\GEOTECH\PGEX\17567.APRX
LAYOUT: LIQUEFACTION USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1% ANNUAL CHANCE FLOOD HAZARD

0.2% ANNUAL CHANCE FLOOD HAZARD

N

0 250 500

FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE 2017 AND FEMA

ENGEO Expect Excellence	FEMA FLOOD INSURANCE MAP PALMER WALNUT CREEK, CALIFORNIA	PROJECT NO.: 17567.000.000 SCALE: AS SHOWN DRAWN BY: JV CHECKED BY: RHB	FIGURE NO. 6

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\17567\GEOTECH\PGEX\17567.APRX
LAYOUT: FEMA USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

CPT REPORT AND LOGS

PRESENTATION OF SITE INVESTIGATION RESULTS

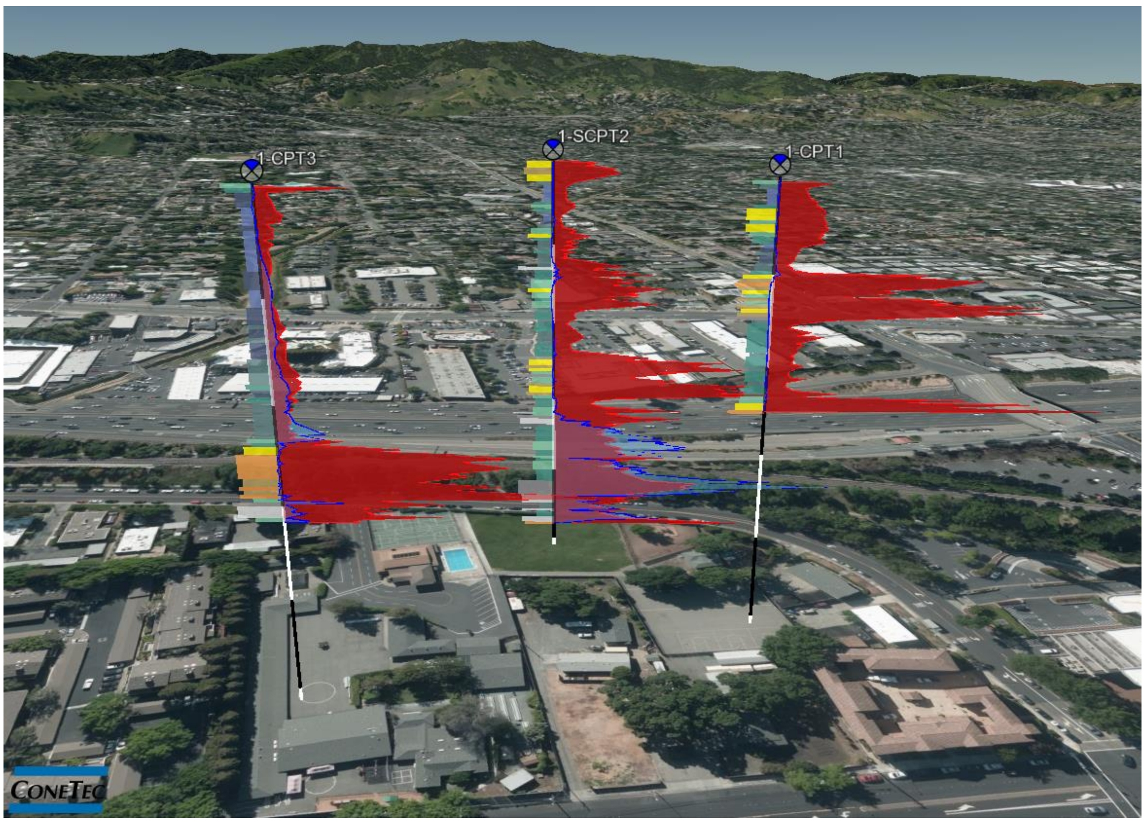
Palmer, Walnut Creek

Prepared for:

ENGEO Incorporated

ConeTec Job No: 20-56-21232

Project Start Date: 14-Aug-2020
Project End Date: 14-Aug-2020
Report Date: 17-Aug-2020



Prepared by:

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Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for ENGEO Incorporated of San Ramon, CA. The program consisted of cone penetration testing (CPTu) at three (3) locations. Shear wave velocities were recorded in one (1) sounding.

Project Information

Project	
Client	ENGEO Incorporated
Project	Palmer, Walnut Creek
ConeTec Project #	20-56-21232

An aerial overview from Google Earth including the CPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30-ton truck mounted cylinder	CPTu/SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu/SCPTu	Consumer grade GPS	32610

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
499:T1500F15U1K	499	15	225	1500	15	1000
Cone 499 was used on all soundings.						

Cone Penetration Test	
Depth reference	Depths are referenced to the existing ground surface at the time of test.
Tip and sleeve data offset	0.1 Meter This has been accounted for in the CPT data files.
Additional Comments	Advanced plots with Ic, Phi, Su(Nkt), and N1(60)Ic, Seismic plots, as well as Soil Behavior Type (SBT) Scatter plots have been included in the data release package.

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of ENGEO Incorporated (Client) for the project titled "Palmer, Walnut Creek". The report's contents may not be relied upon by any other party without the express written permission of ConeTec, Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meet or exceed those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

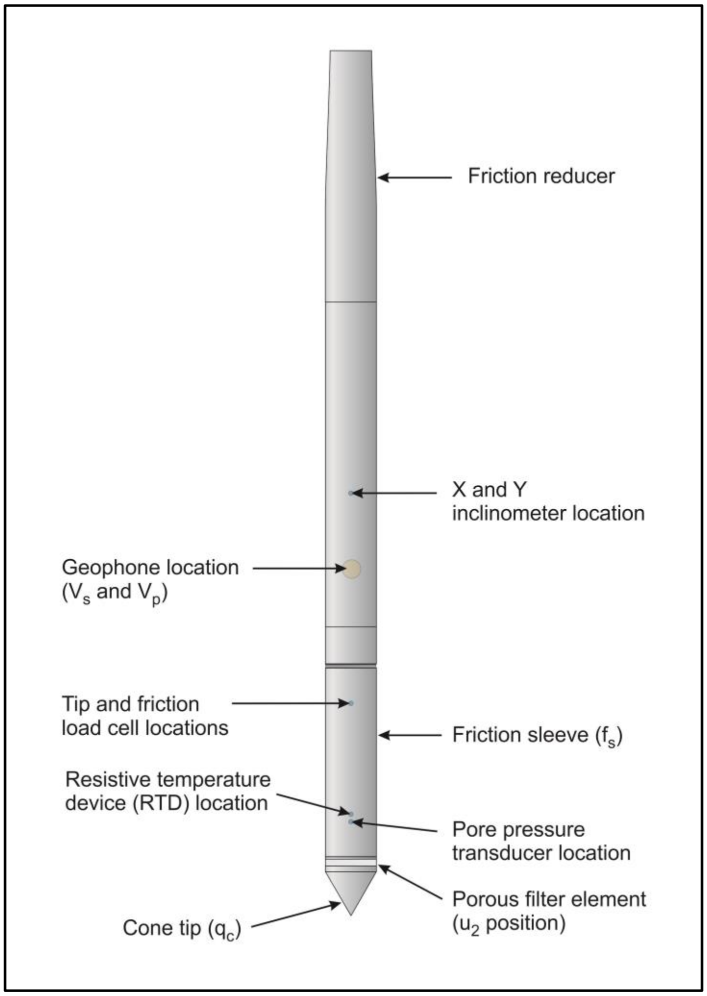

Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerin or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerin under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1-a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (Vp) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

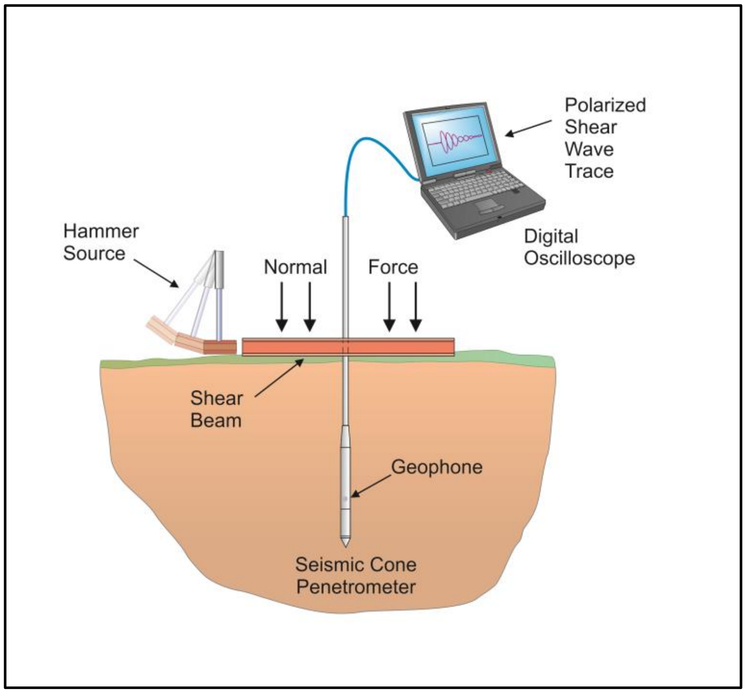

Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

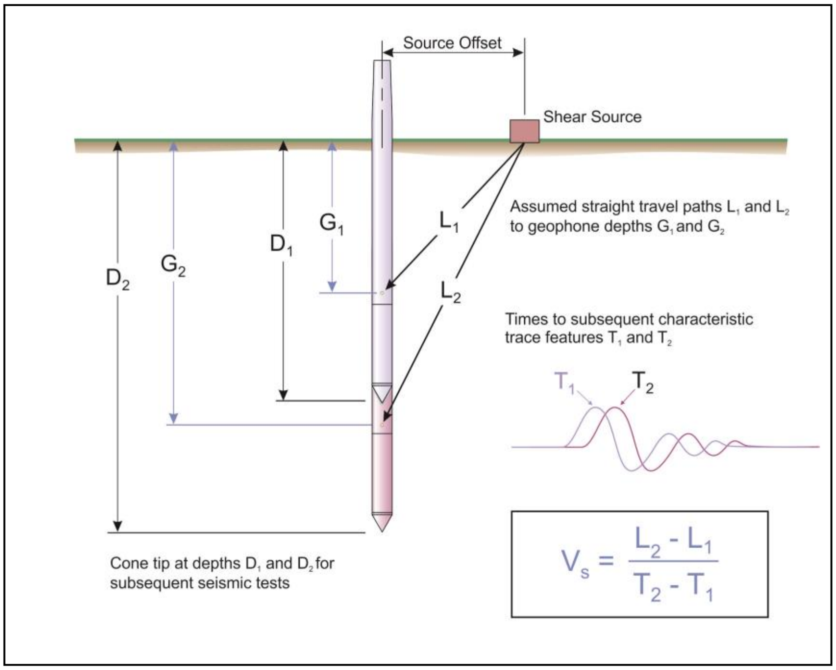

Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 100 feet (30 meters) ($\bar{v}_s$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE, 2010.

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_{s100} or V_{s30}.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

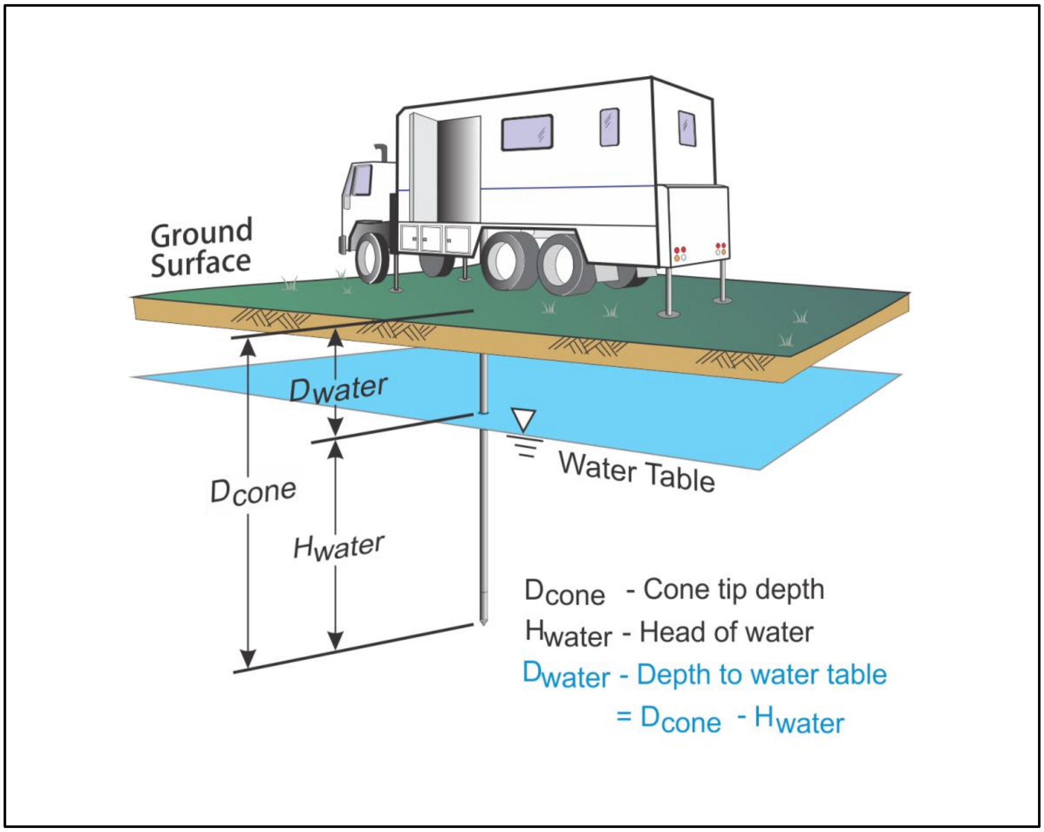

Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

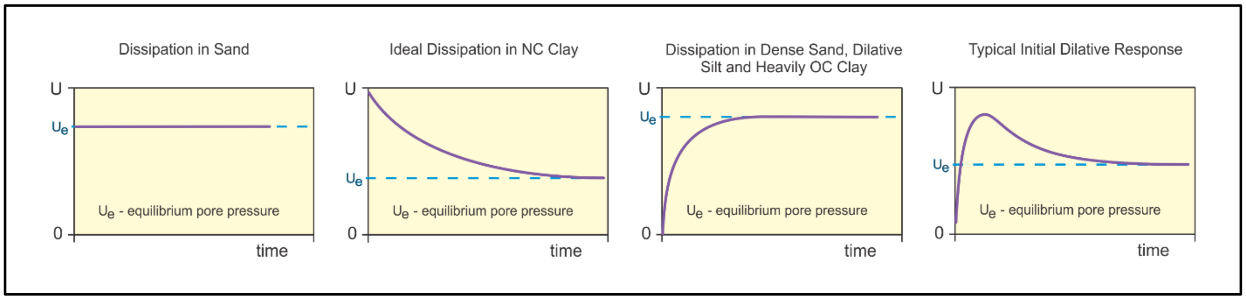

Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

- T* is the dimensionless time factor (Table Time Factor)
- a is the radius of the cone
- I_r is the rigidity index
- t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

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Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Ic, Phi, Su(Nkt), and N1(60)Ic
- Soil Behavior Type (SBT) Zone Scatter Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Shear Wave (Vs) Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

ConeTec

Job No:	20-56-21232
Client:	ENGEO Incorporated
Project:	Palmer, Walnut Creek
Start Date:	14-Aug-2020
End Date:	14-Aug-2020

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting[2] (m)	Elevation[3] (ft)	Refer to Notation Number
1-CPT1	20-56-21232_1CP01	14-Aug-2020	499:T1500F15U1K	18.8	40.85	4197844	582726	94	
1-SCPT2	20-56-21232_1SP02	14-Aug-2020	499:T1500F15U1K	14.5	78.25	4197801	582661	94	
1-CPT3	20-56-21232_1CP03	14-Aug-2020	499:T1500F15U1K	10.6	51.84	4197717	582733	95	

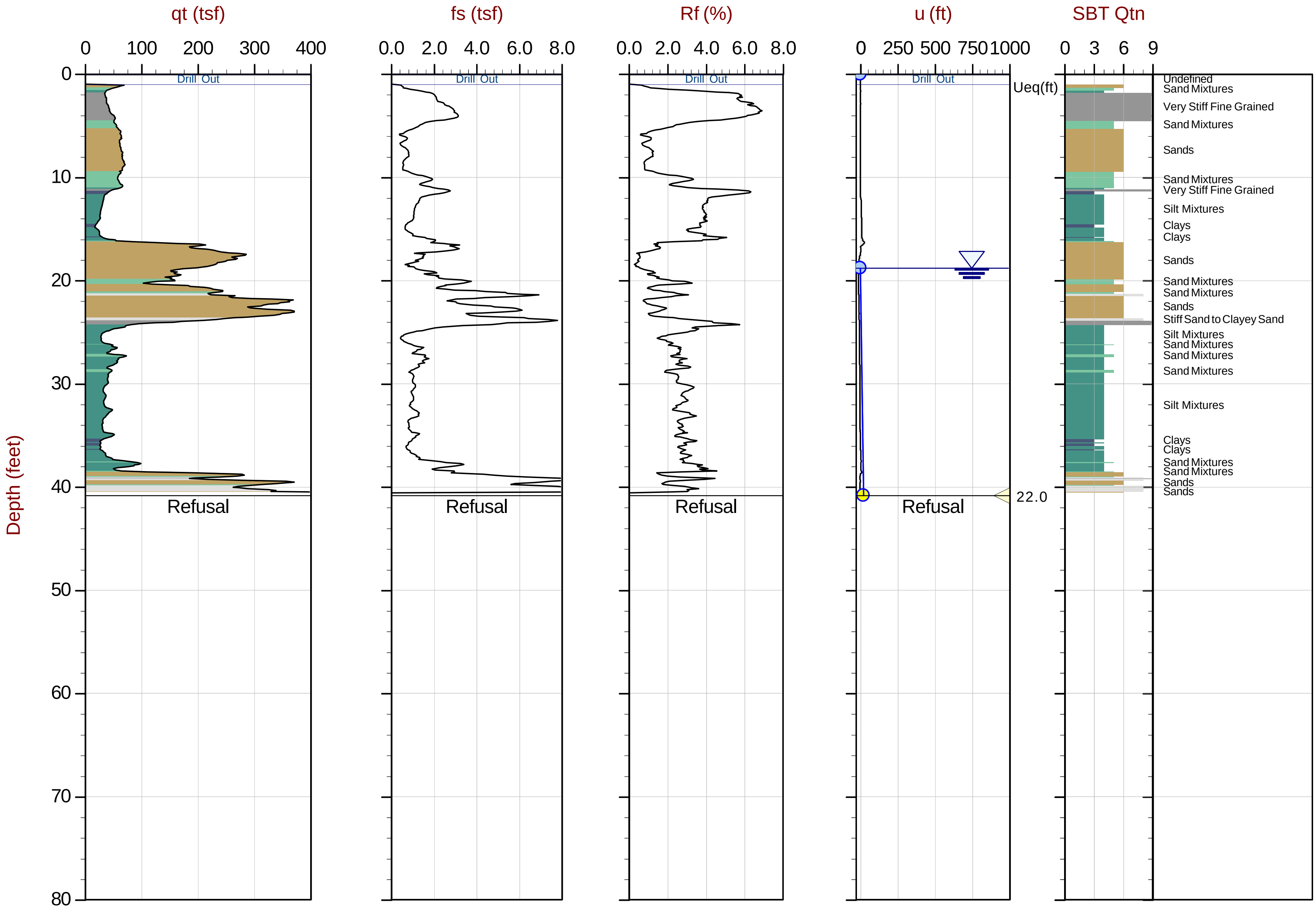
1. The assumed phreatic surface was based on the shallowest pore pressure dissipation tests performed within the sounding. Hydrostatic conditions are assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10 North.
3. Elevations are refrenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.

CONETEC

ENGEO

Job No: 20-56-21232
Date: 2020-08-14 08:43
Site: Palmer, Walnut Creek

Sounding: 1-CPT1
Cone: 499:T1500F15U1K



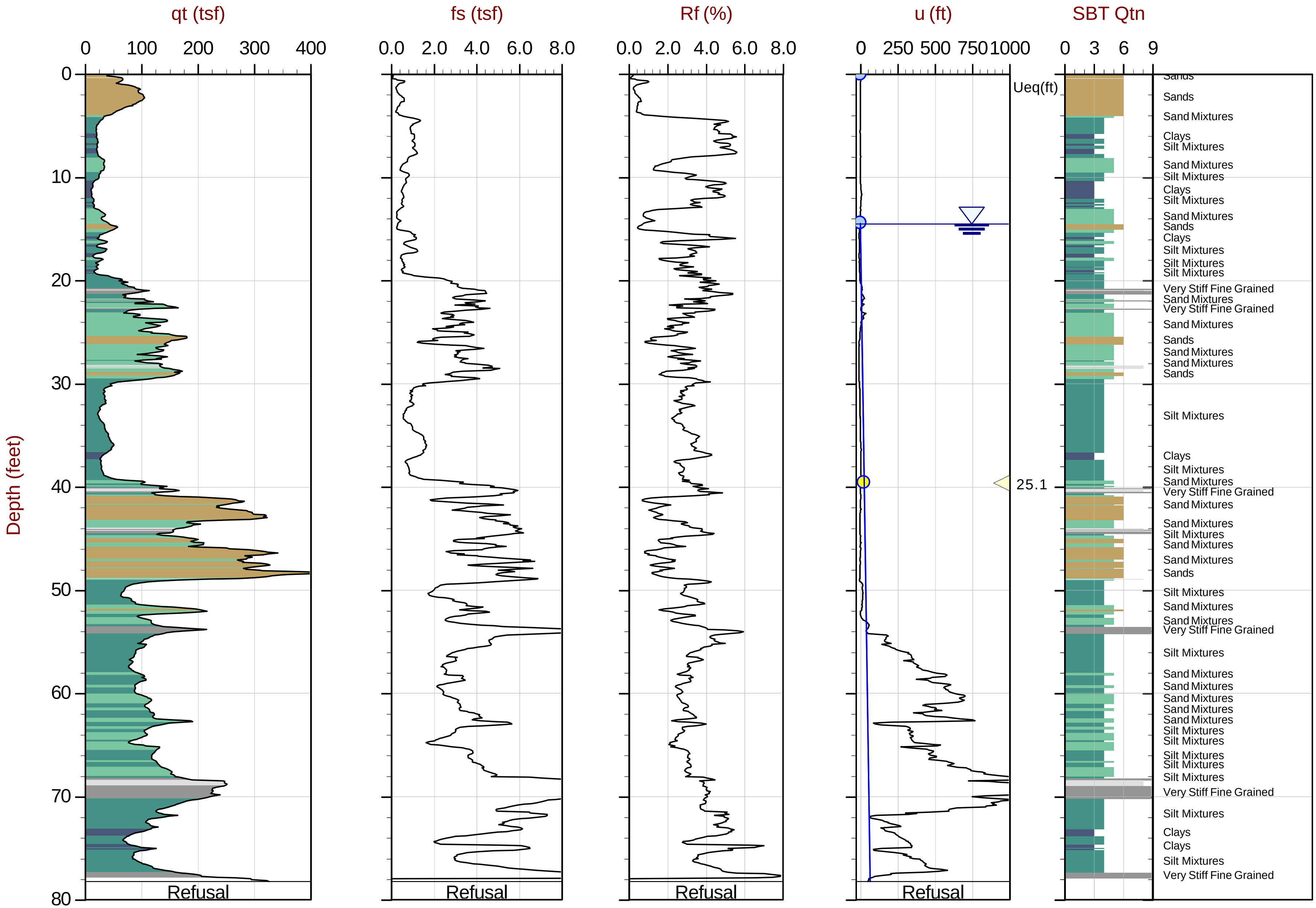
Max Depth: 12.450 m / 40.85 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21232_1CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4197844m E: 582726m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Max Depth: 23.850 m / 78.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

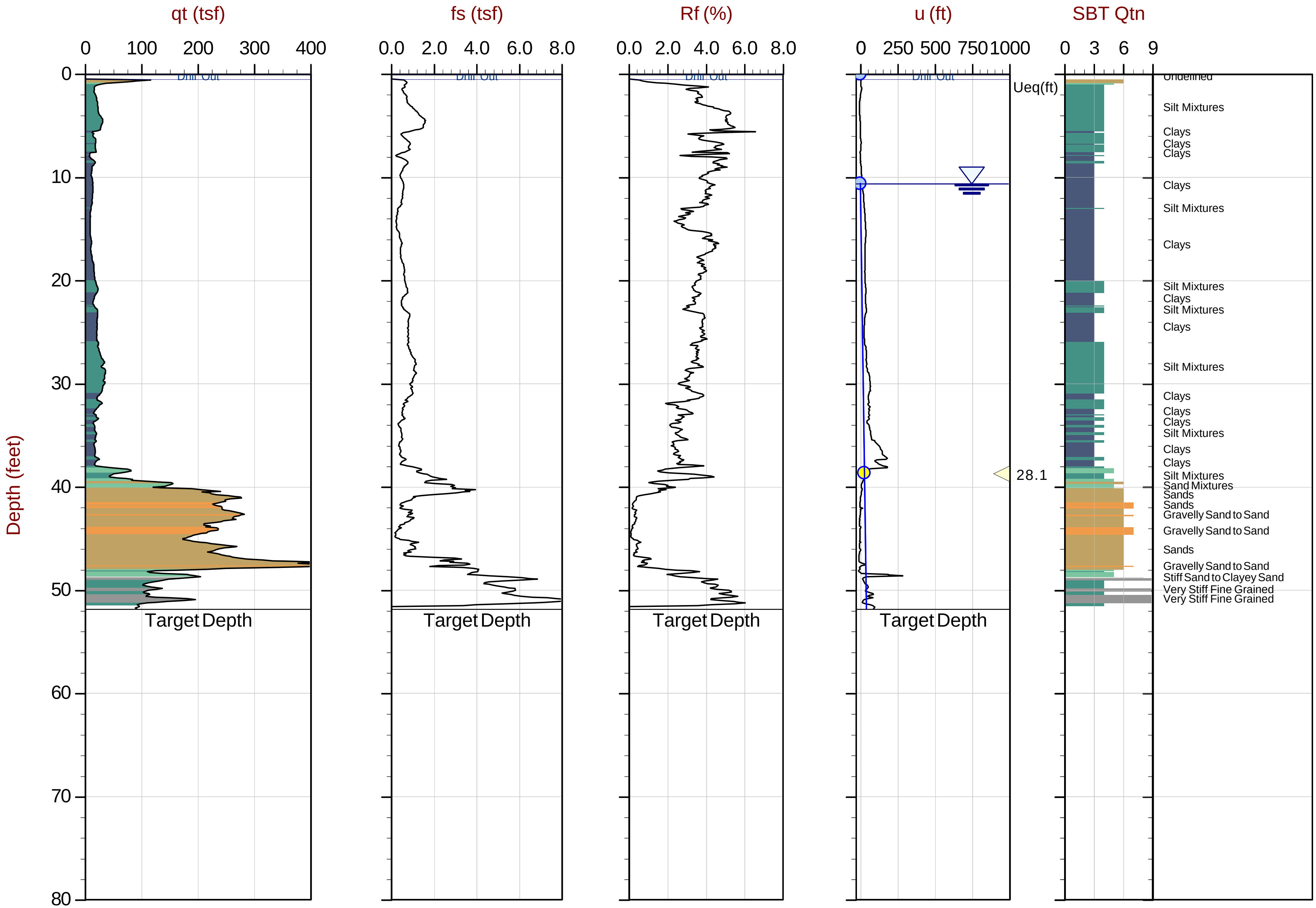
File: 20-56-21232_1SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4197801m E: 582661m

Equilibrium Pore Pressure (Ueq) · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ENGEO	Job No: 20-56-21232	Sounding: 1-CPT3
	Date: 2020-08-14 11:58	Cone: 499:T1500F15U1K
	Site: Palmer, Walnut Creek	



Max Depth: 15.800 m / 51.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21232_1CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4197717m E: 582733m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

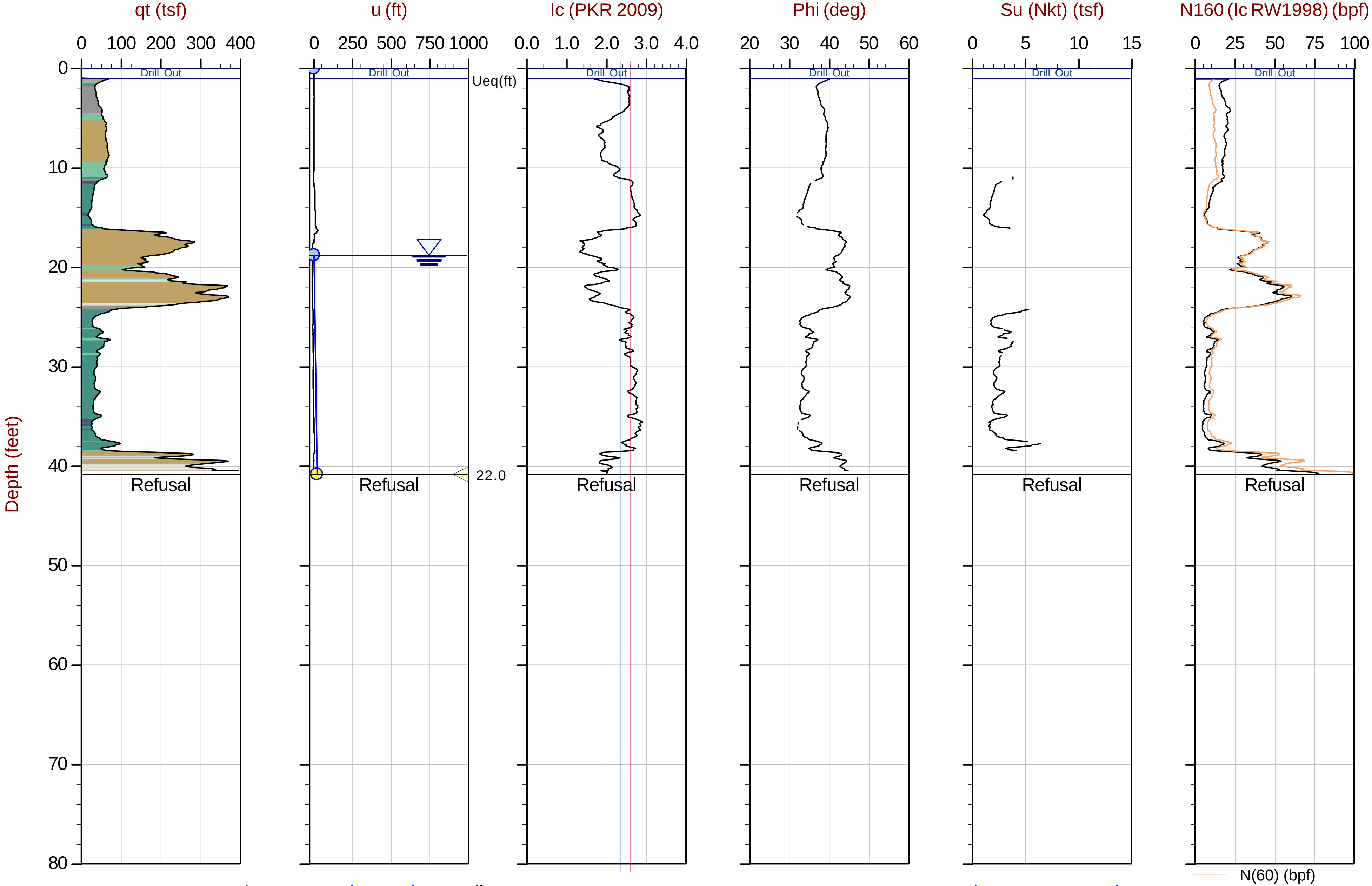
Advanced Cone Penetration Test Plots with Ic, Phi, Su(Nkt), and N1(60)Ic

CONETEC

ENGEO

Job No: 20-56-21232
Date: 2020-08-14 08:43
Site: Palmer, Walnut Creek

Sounding: 1-CPT1
Cone: 499:T1500F15U1K



Max Depth: 12.450 m / 40.85 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21232_1CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4197844m E: 582726m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

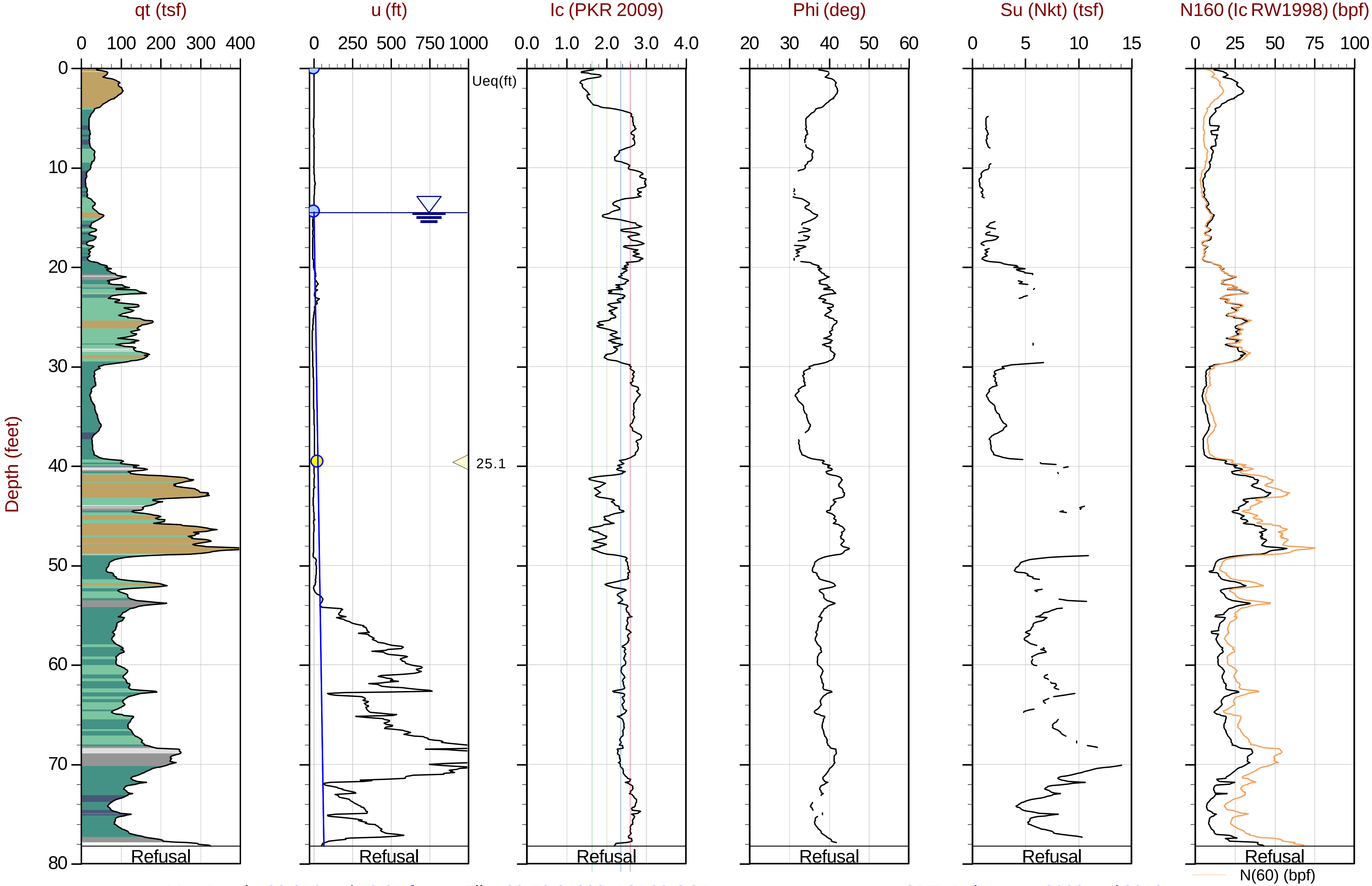
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-21232
Date: 2020-08-14 10:07
Site: Palmer, Walnut Creek

Sounding: 1-SCPT2
Cone: 499:T1500F15U1K



Max Depth: 23.850 m / 78.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21232_1SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4197801m E: 582661m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

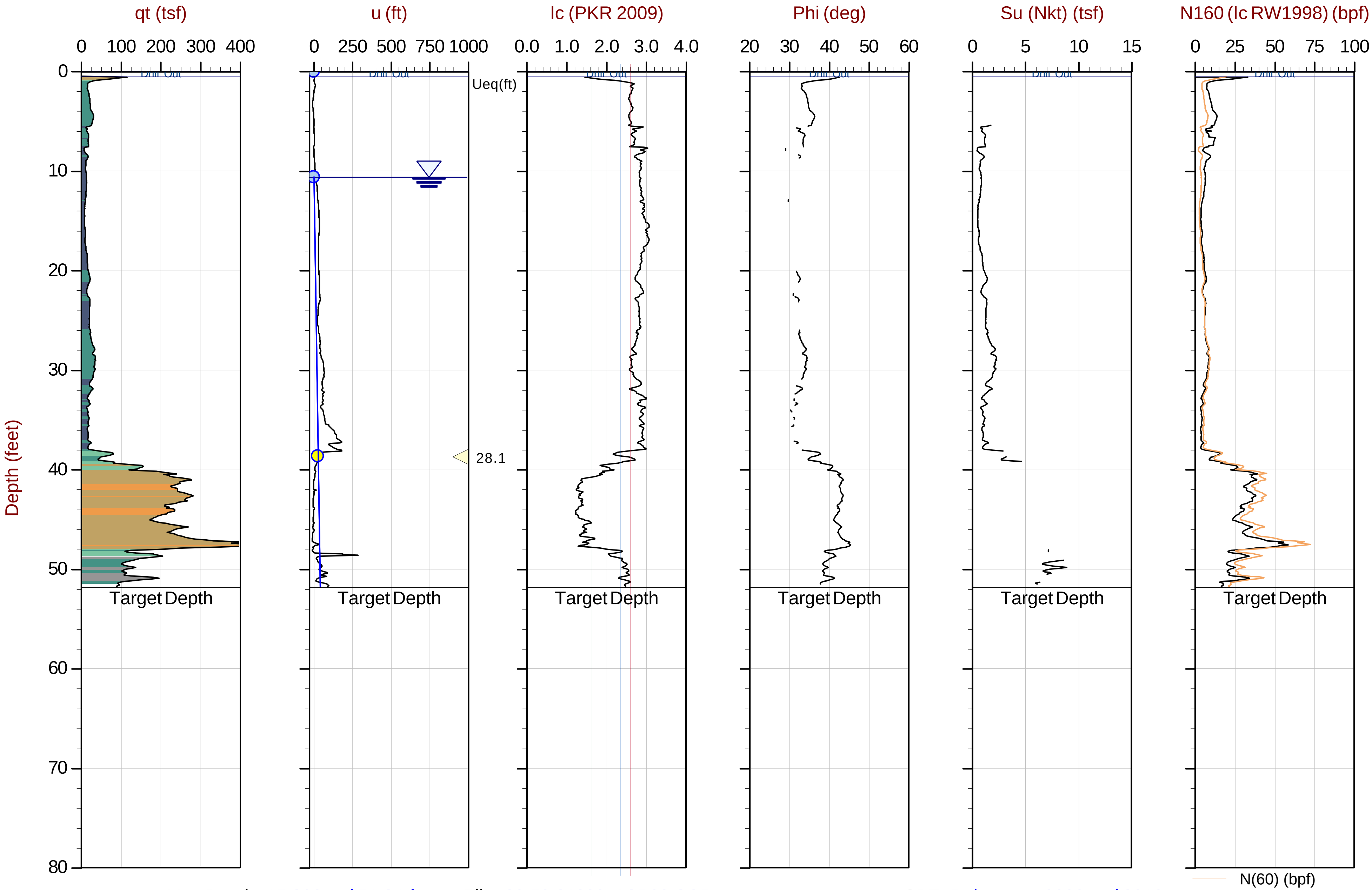
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 20-56-21232
Date: 2020-08-14 11:58
Site: Palmer, Walnut Creek

Sounding: 1-CPT3
Cone: 499:T1500F15U1K



Max Depth: 15.800 m / 51.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21232_1CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4197717m E: 582733m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

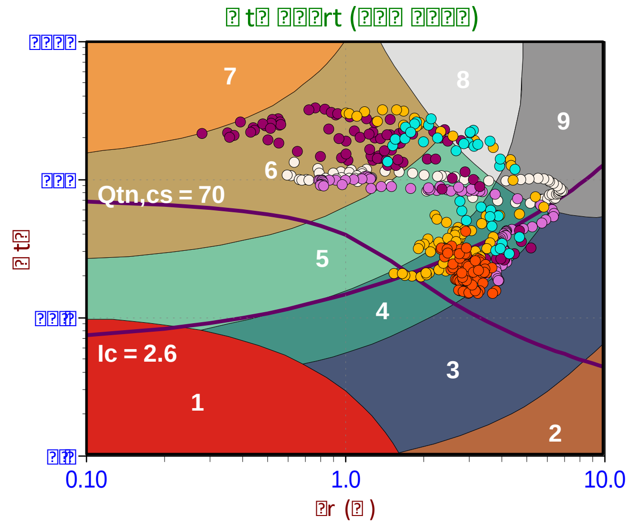
Soil Behavior Type (SBT) Scatter Plots

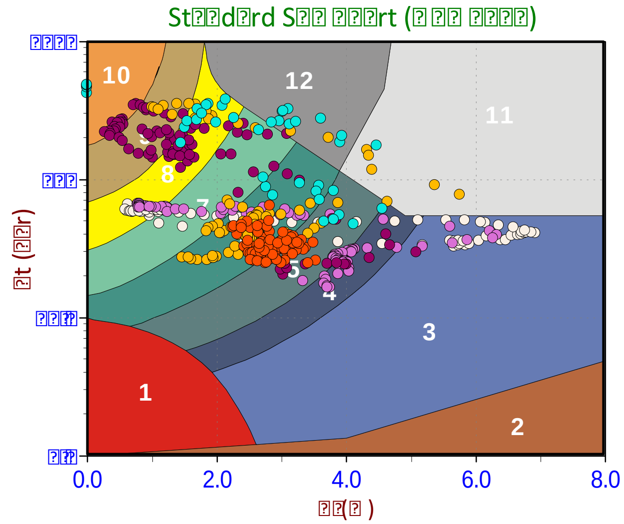
CONETEC

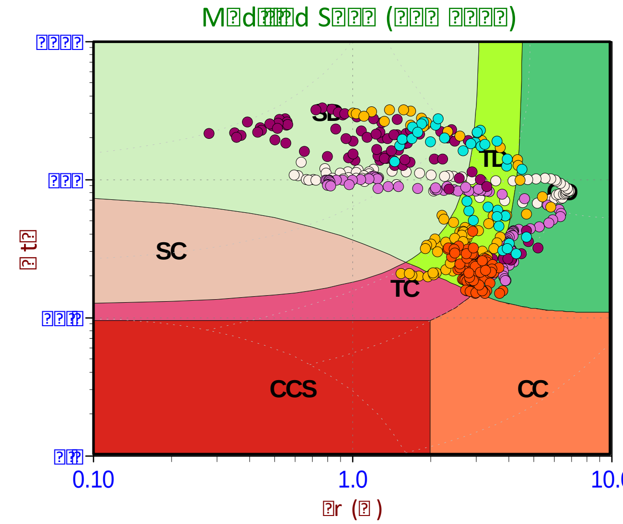
ENGEO

Job No: 20-56-21232
Date: 2020-08-14 08:43
Site: Palmer, Walnut Creek

Sounding: 1-CPT1
Cone: 499:T1500F15U1K







Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend

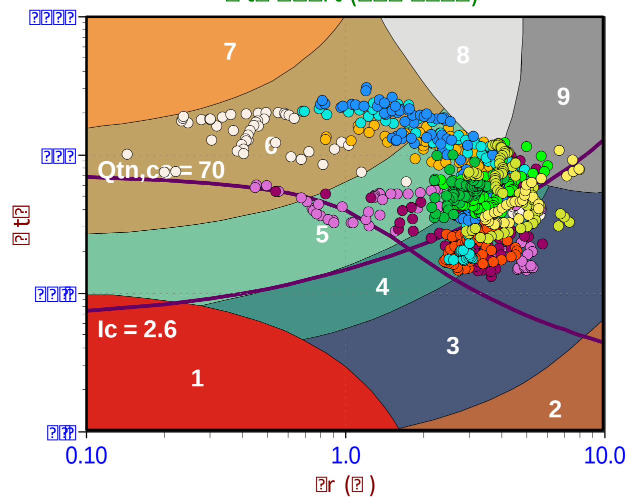
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

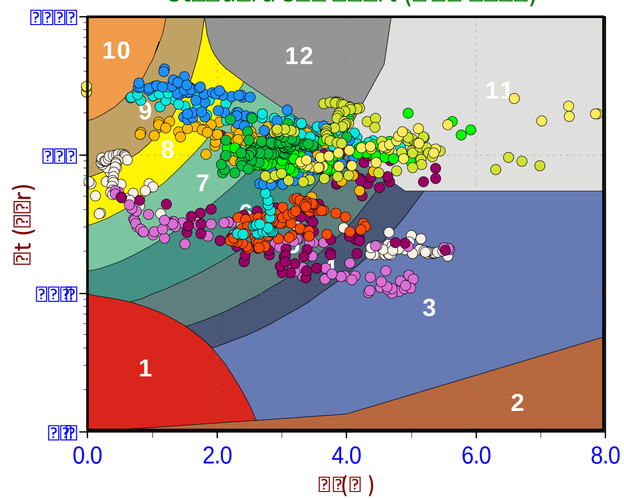
CONETEC

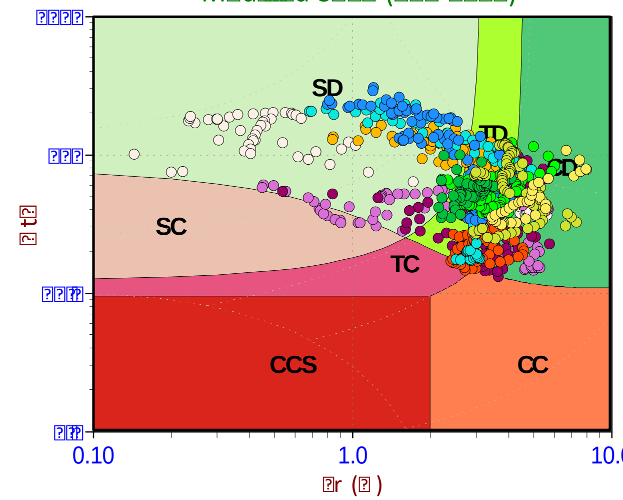
ENGEO

Job No: 20-56-21232
Date: 2020-08-14 10:07
Site: Palmer, Walnut Creek

Sounding: 1-SCPT2
Cone: 499:T1500F15U1K







Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

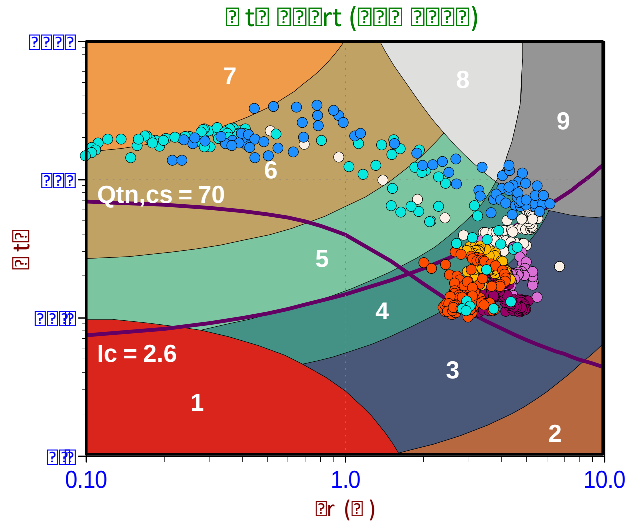
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

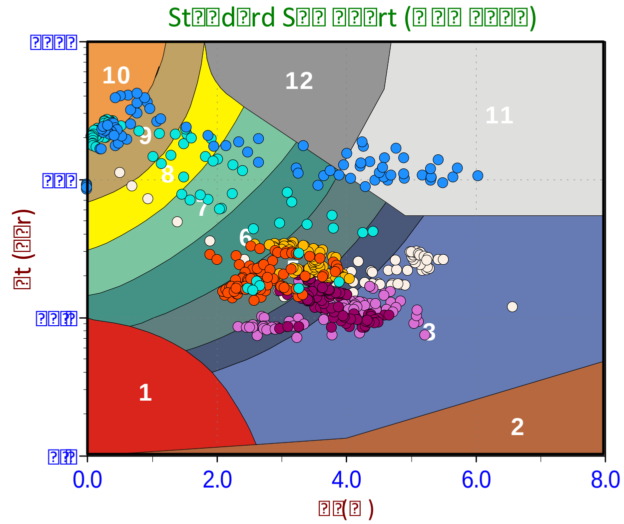
CONETEC

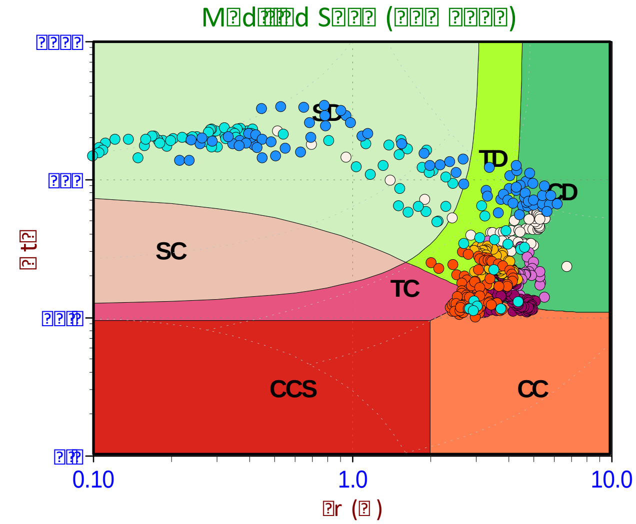
ENGEO

Job No: 20-56-21232
Date: 2020-08-14 11:58
Site: Palmer, Walnut Creek

Sounding: 1-CPT3
Cone: 499:T1500F15U1K







Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Seismic Cone Penetration Test Plots

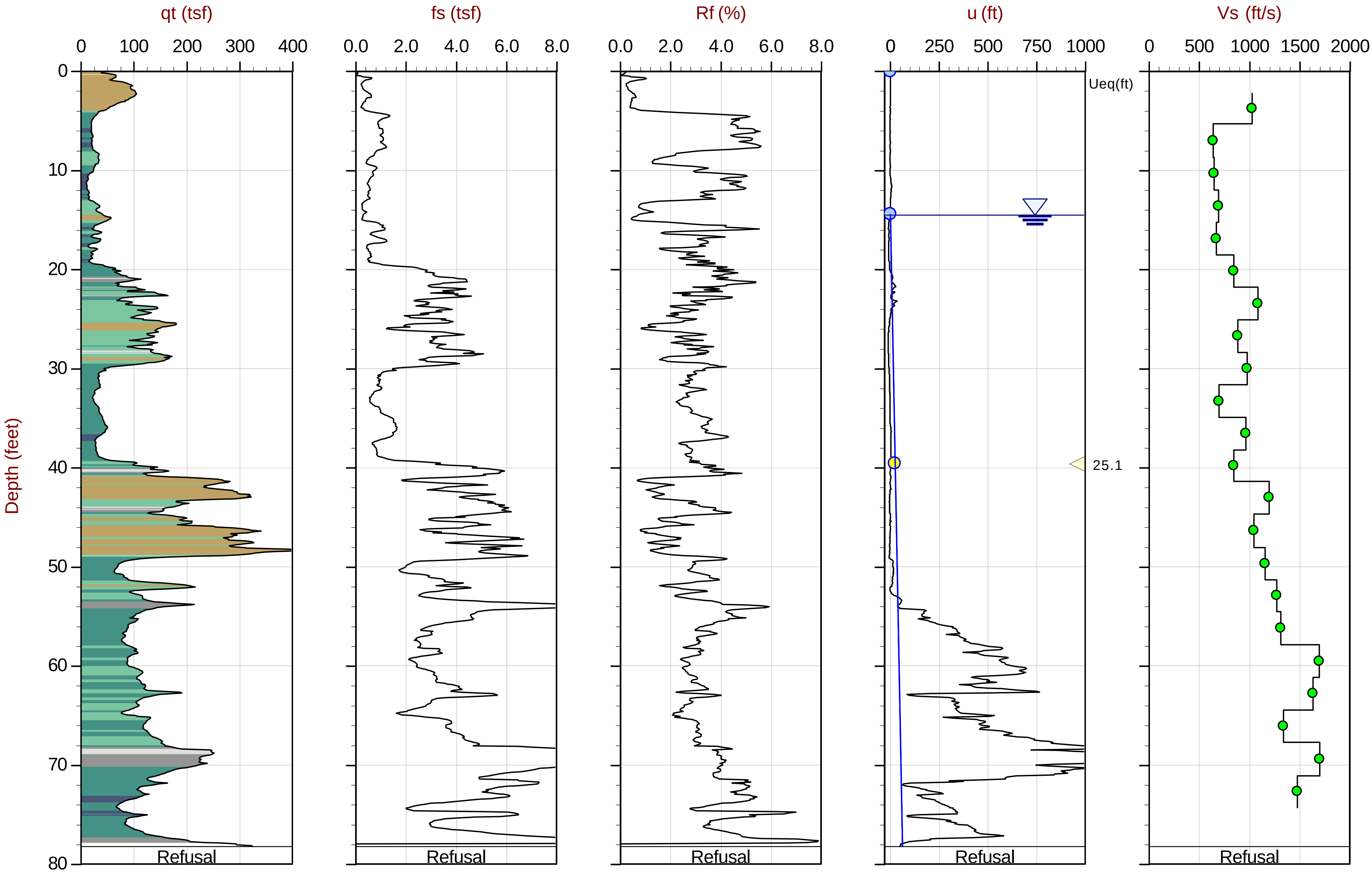
ConeTec

CONETEC

ENGEO

Job No: 20-56-21232
Date: 2020-08-14 10:07
Site: Palmer, Walnut Creek

Sounding: 1-SCPT2
Cone: 499:T1500F15U1K



Max Depth: 23.850 m / 78.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21232_1SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4197801m E: 582661m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Tabular Results

Job No: 20-56-21232
Client: ENGEO
Project: Palmer, Walnut Creek
Sounding ID: 1-SCPT2
Date: 08:14:20 10:07

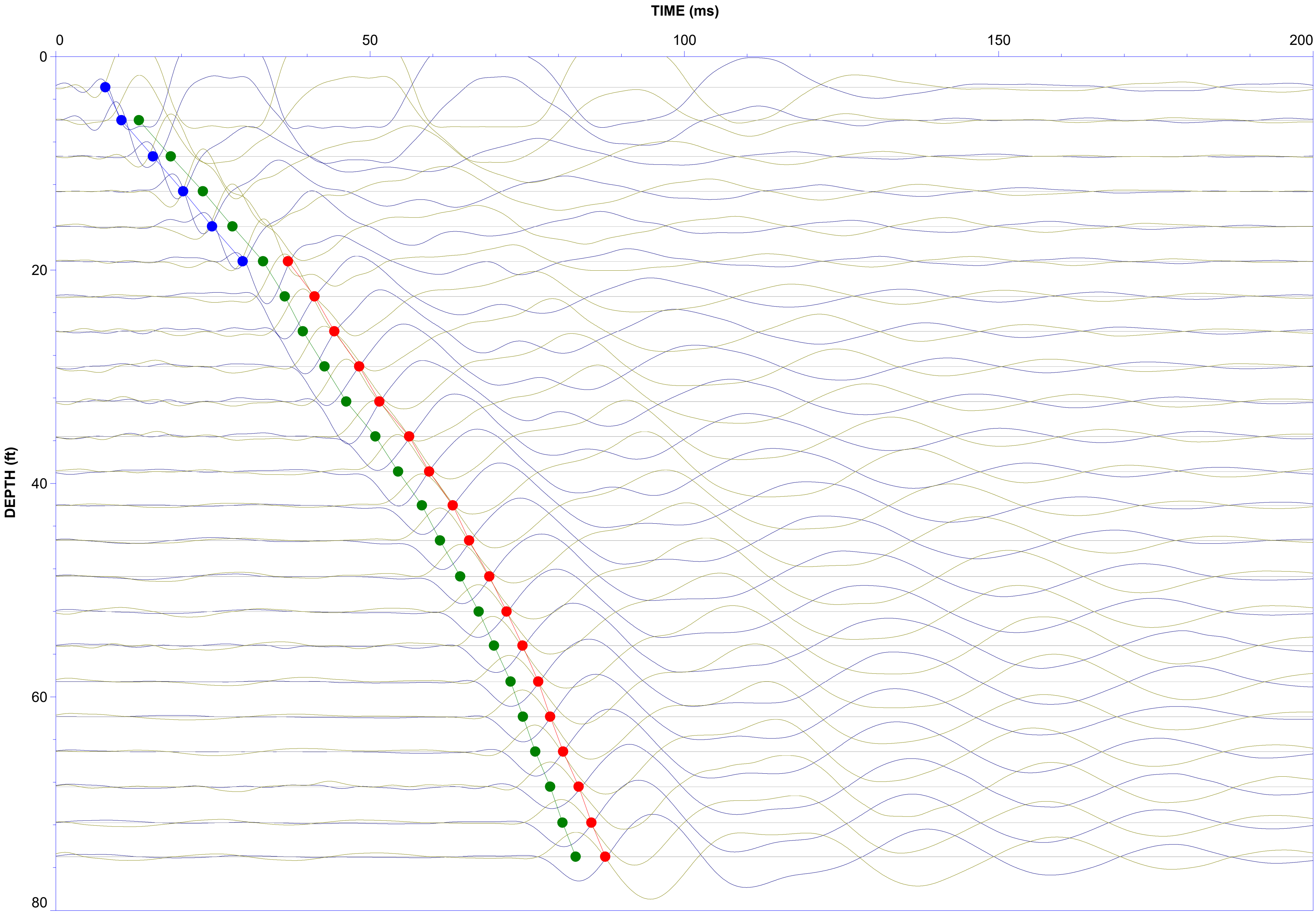
Seismic Source: Beam
Seismic Offset (ft): 2.10
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
2.89	2.23	3.06			
5.97	5.32	5.72	2.65	2.57	1030
9.35	8.69	8.94	3.23	5.05	639
12.63	11.98	12.16	3.21	4.95	649
15.91	15.26	15.40	3.24	4.66	695
19.19	18.54	18.66	3.26	4.86	670
22.47	21.82	21.92	3.26	3.85	848
25.75	25.10	25.19	3.27	3.01	1085
29.04	28.38	28.46	3.27	3.69	887
32.32	31.66	31.73	3.27	3.35	978
35.60	34.94	35.00	3.27	4.69	698
38.88	38.22	38.28	3.28	3.39	966
42.06	41.40	41.46	3.18	3.76	845
45.34	44.69	44.73	3.28	2.74	1198
48.72	48.06	48.11	3.38	3.23	1047
52.00	51.35	51.39	3.28	2.84	1156
55.18	54.53	54.57	3.18	2.49	1275
58.56	57.91	57.95	3.38	2.57	1317
61.84	61.19	61.22	3.28	1.93	1700
65.12	64.47	64.50	3.28	2.00	1636
68.41	67.75	67.78	3.28	2.44	1343
71.78	71.13	71.16	3.38	1.98	1705
74.97	74.31	74.34	3.18	2.15	1481

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 20-56-21232 Client: ENGEO Project Title: Palmer, Walnut Creek Filter: BP 0-200 Hz Hole: 1-SCPT2 Date: 08:14:20 10:07
Cone: 499:T1500F15U1K



Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No:	20-56-21232
Client:	ENGEO Incorporated
Project:	Palmer, Walnut Creek
Start Date:	14-Aug-2020
End Date:	14-Aug-2020

CPTu PORE PRESSURE DISSIPATION SUMMARY

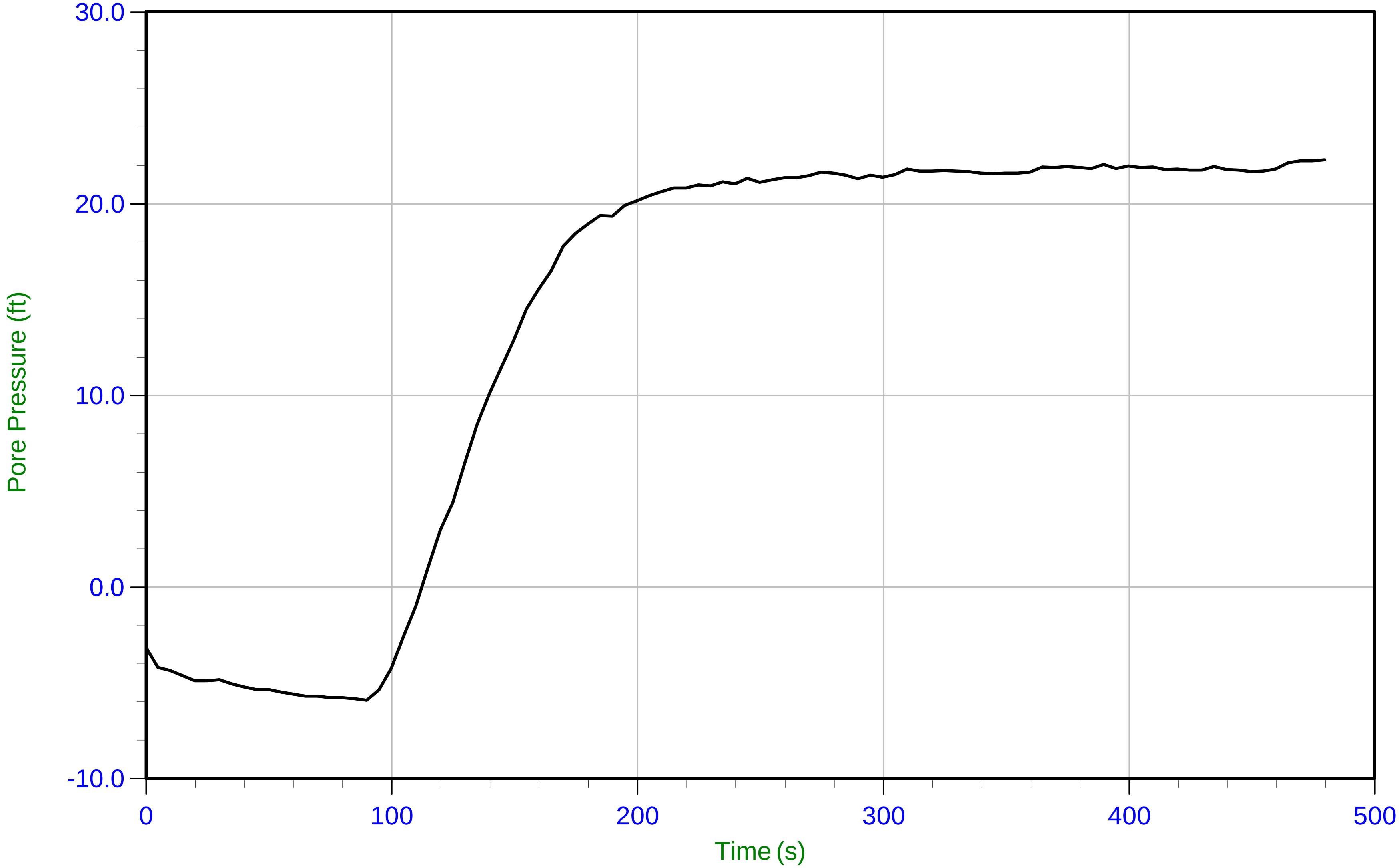
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
1-CPT1	20-56-21232_1CP01	15	480	40.85	22.0	18.8
1-SCPT2	20-56-21232_1SP02	15	320	39.62	25.2	14.5
1-CPT3	20-56-21232_1CP03	15	340	38.71	28.1	10.6

ConeTec

ENGEO

Job No: 20-56-21232
Date: 08/14/2020 08:43
Site: Palmer, Walnut Creek

Sounding: 1-CPT1
Cone: 499:T1500F15U1K Area=15 cm²



Trace Summary:

Filename: 20-56-21232_1CP01.PPF
Depth: 12.450 m / 40.846 ft
Duration: 480.0 s

u Min: -5.9 ft
u Max: 22.3 ft
u Final: 22.3 ft

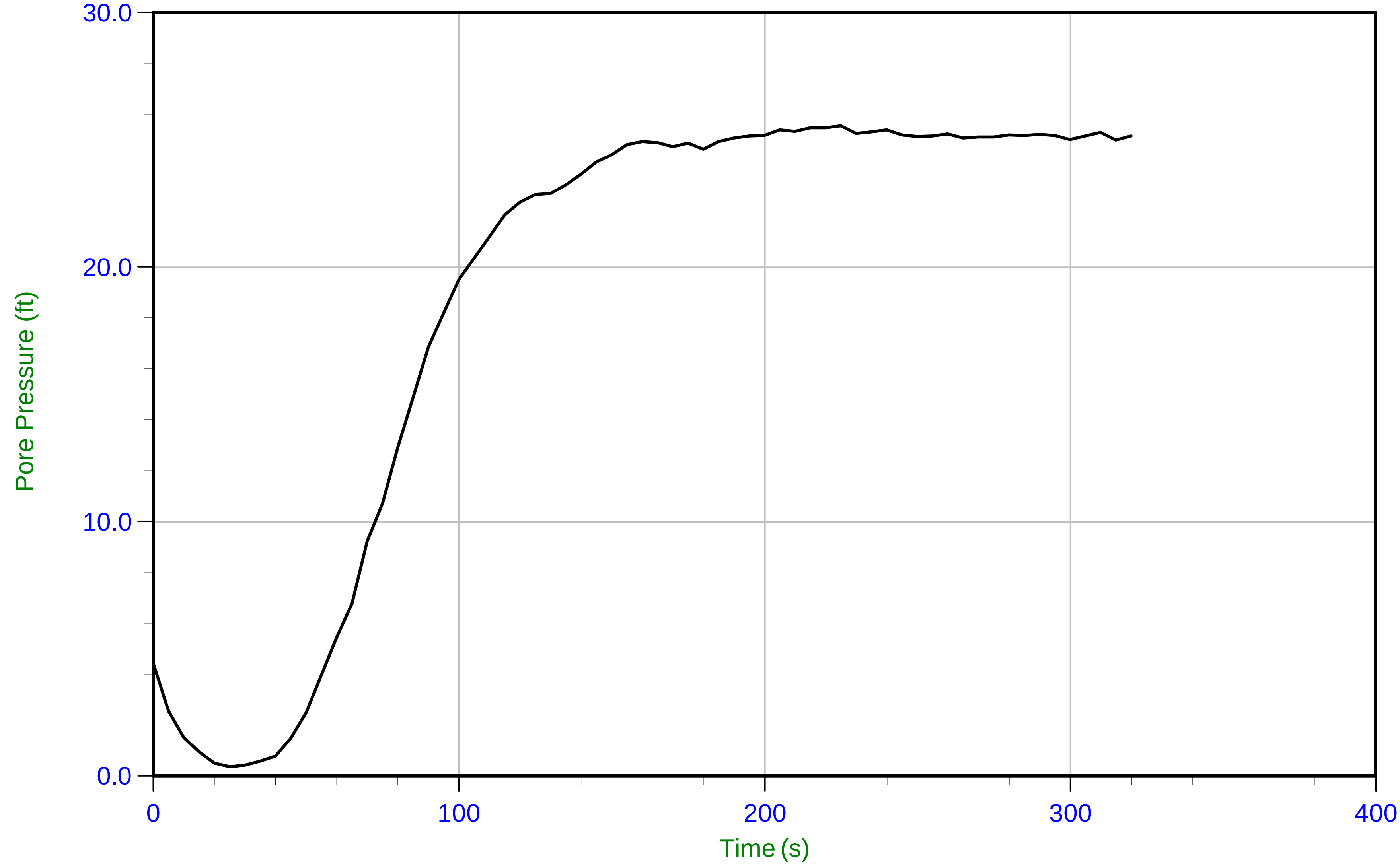
WT: 5.739 m / 18.829 ft
Ueq: 22.0 ft

CONETEC

ENGEO

Job No: 20-56-21232
Date: 08/14/2020 10:07
Site: Palmer, Walnut Creek

Sounding: 1-SCPT2
Cone: 499:T1500F15U1K Area=15 cm²



Trace Summary:

Filename: 20-56-21232_1SP02.PPF
Depth: 12.075 m / 39.616 ft
Duration: 320.0 s

u Min: 0.4 ft
u Max: 25.6 ft
u Final: 25.2 ft

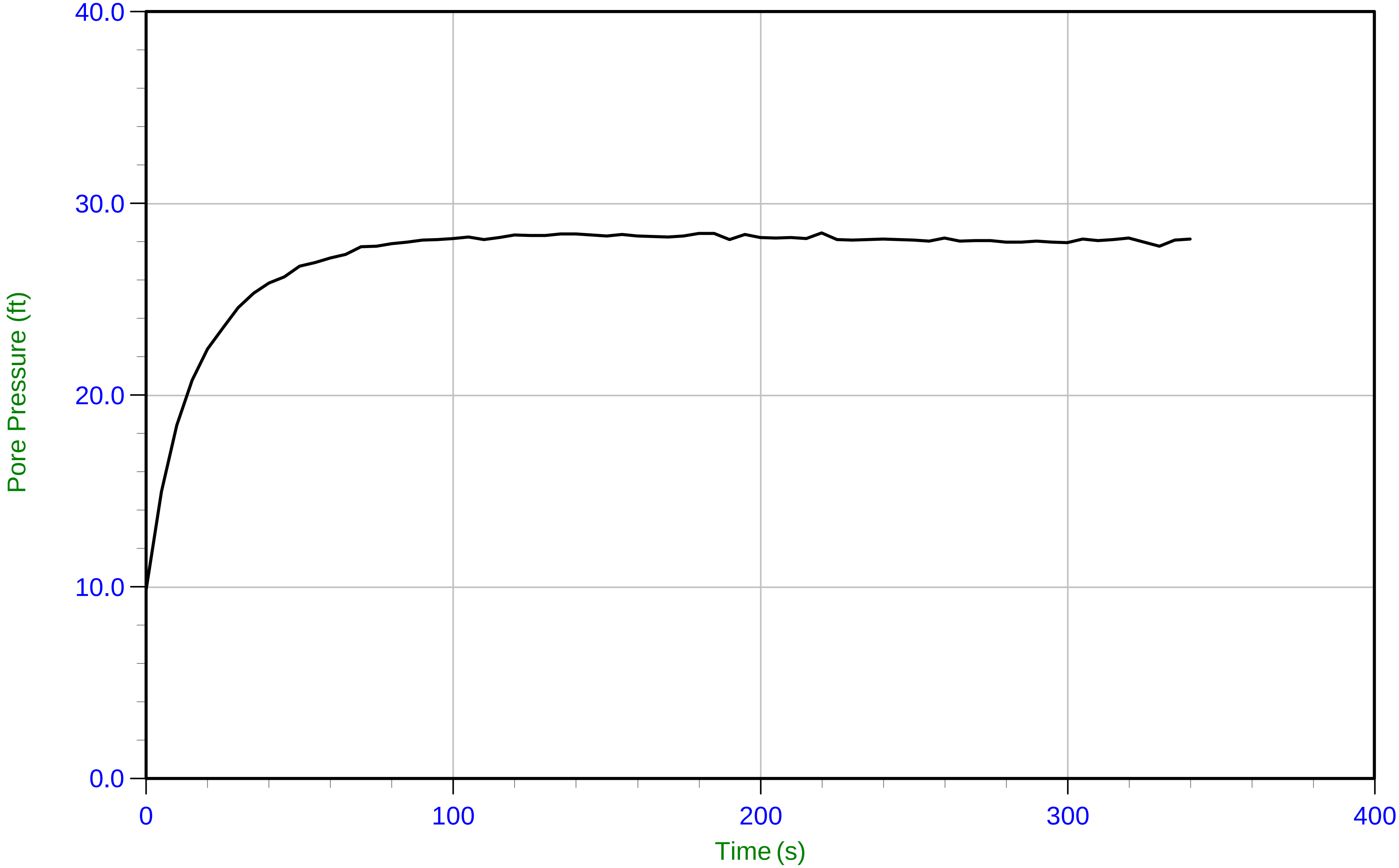
WT: 4.406 m / 14.455 ft
Ueq: 25.2 ft

CONETEC

ENGEO

Job No: 20-56-21232
Date: 08/14/2020 11:58
Site: Palmer, Walnut Creek

Sounding: 1-CPT3
Cone: 499:T1500F15U1K Area=15 cm²



Trace Summary:

Filename: 20-56-21232_1CP03.PPF
Depth: 11.800 m / 38.713 ft
Duration: 340.0 s

u Min: 9.8 ft
u Max: 28.5 ft
u Final: 28.2 ft

WT: 3.238 m / 10.623 ft
Ueq: 28.1 ft

APPENDIX B

LIQUEFACTION ANALYSIS

ENGEO Expect Excellence

2010 Crow Canyon Pl
Suite 250
San Ramon, CA 94583
www.engeo.com

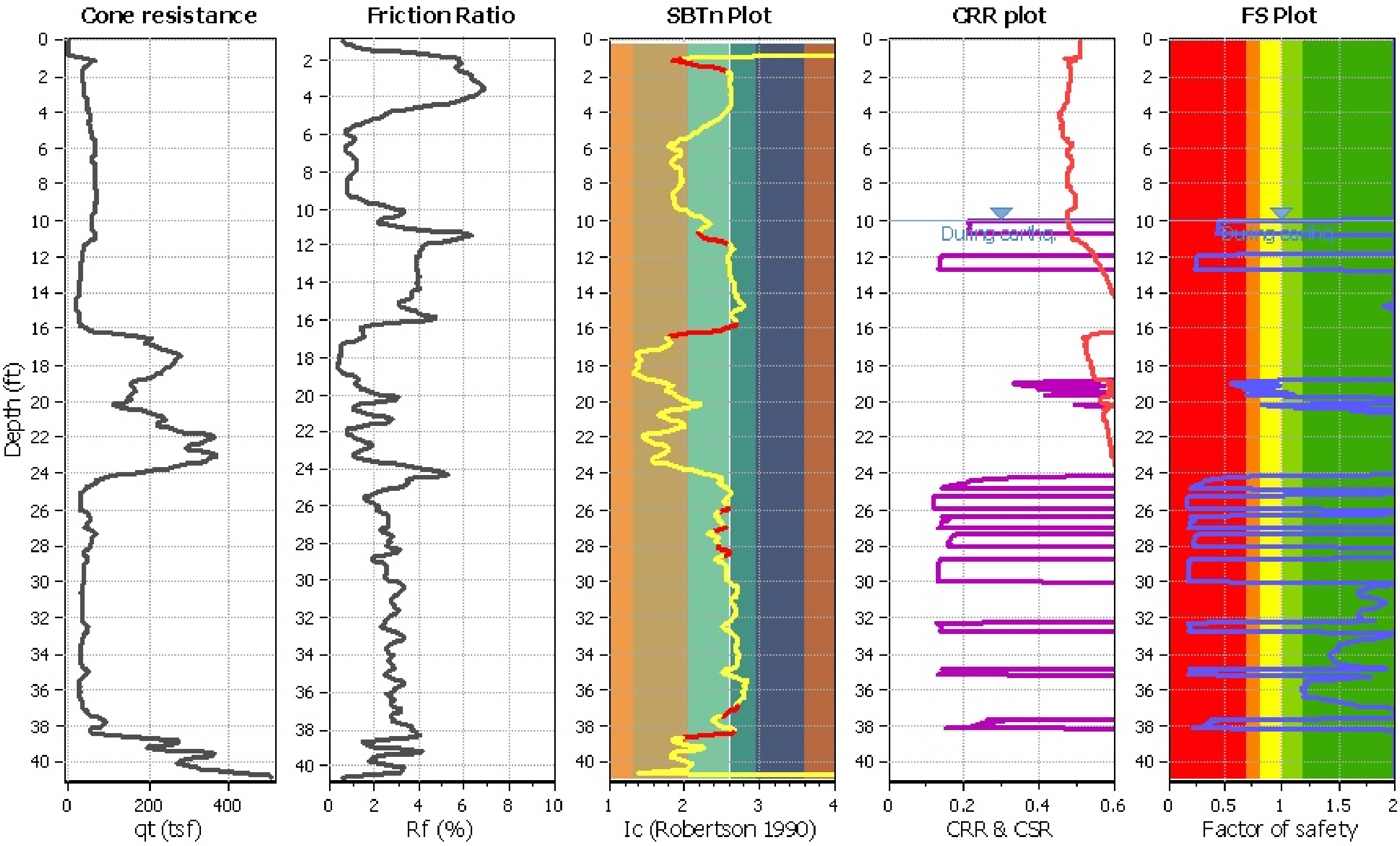
LIQUEFACTION ANALYSIS REPORT

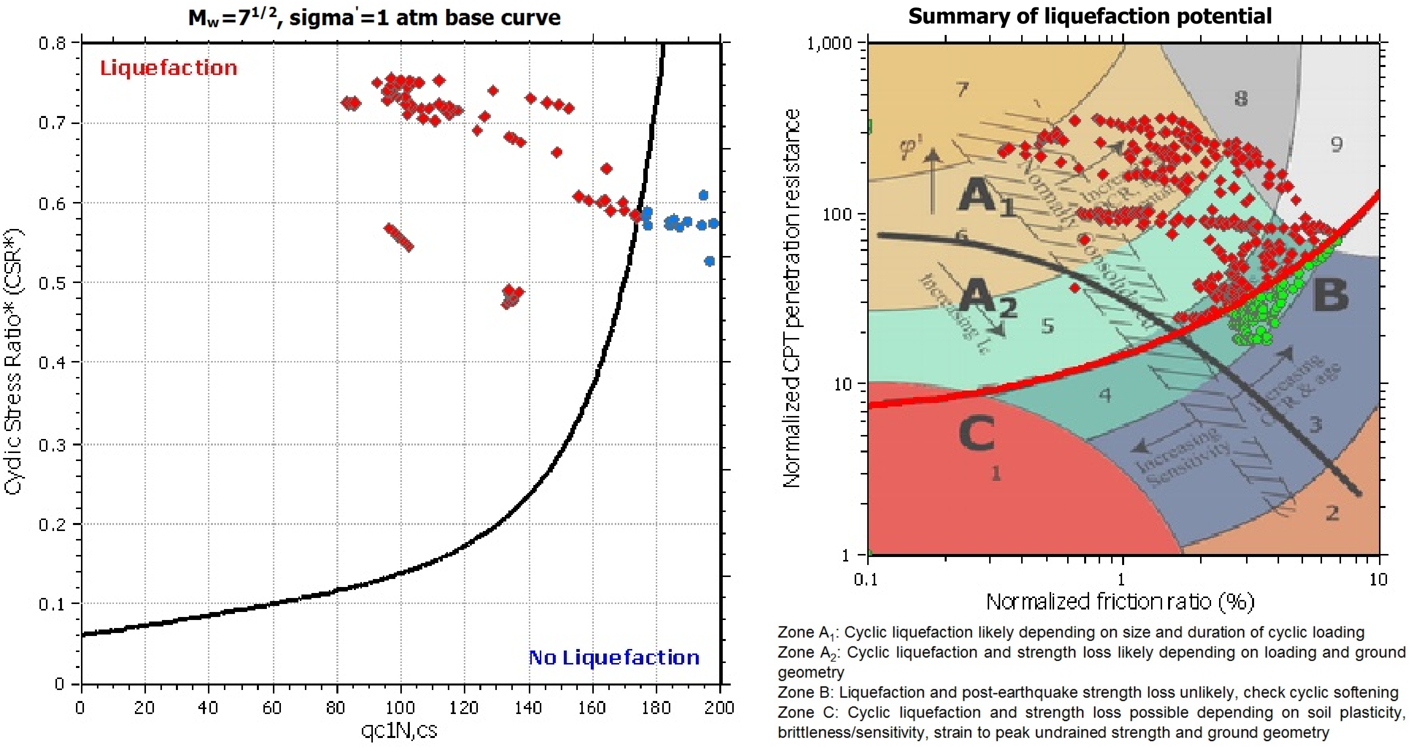
Project title : Palmer **Location : Walnut Creek, CA**

CPT file : 1-CPT1

Input parameters and analysis data

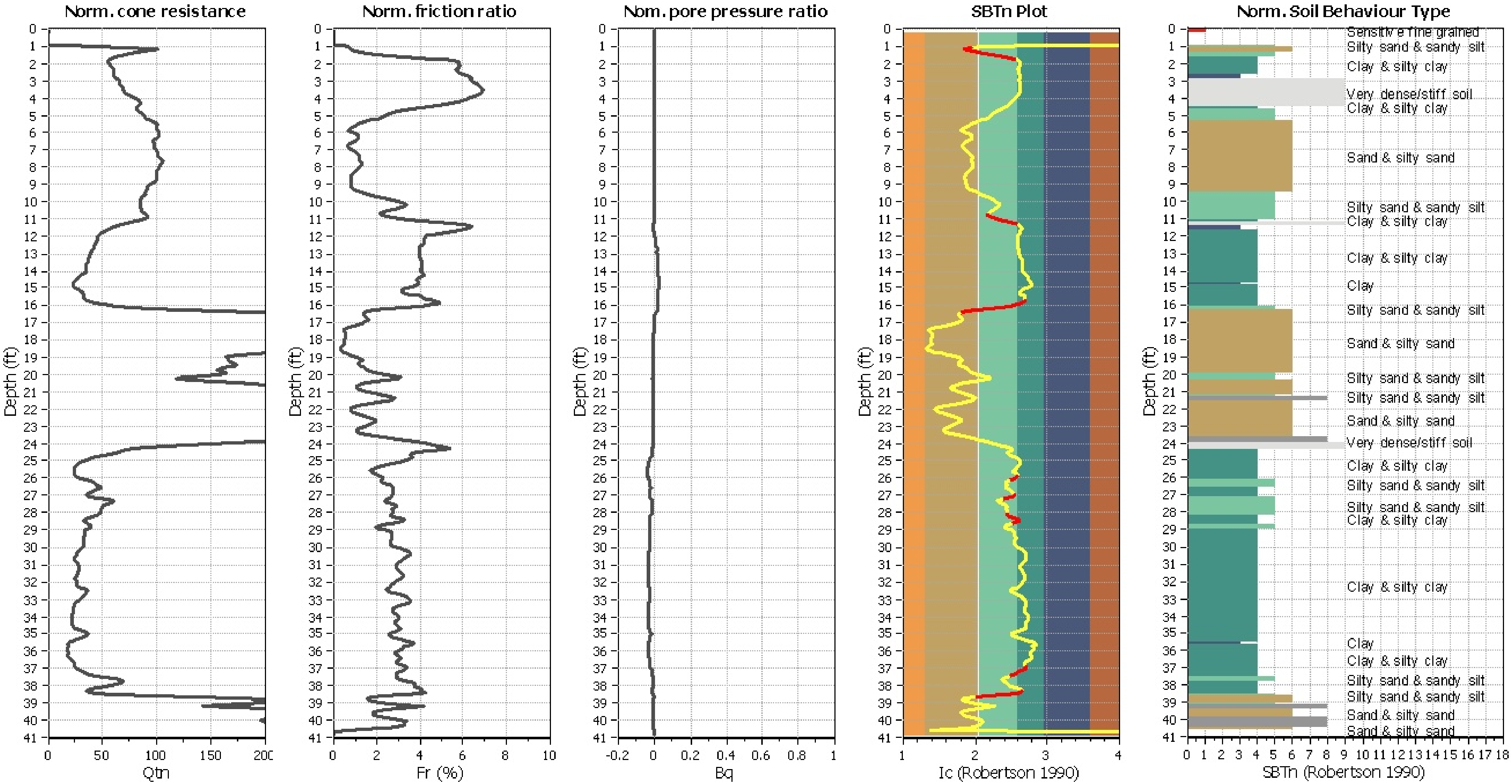
Analysis method:	B&I (2014)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.88	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		





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

CPT basic interpretation plots (normaliz



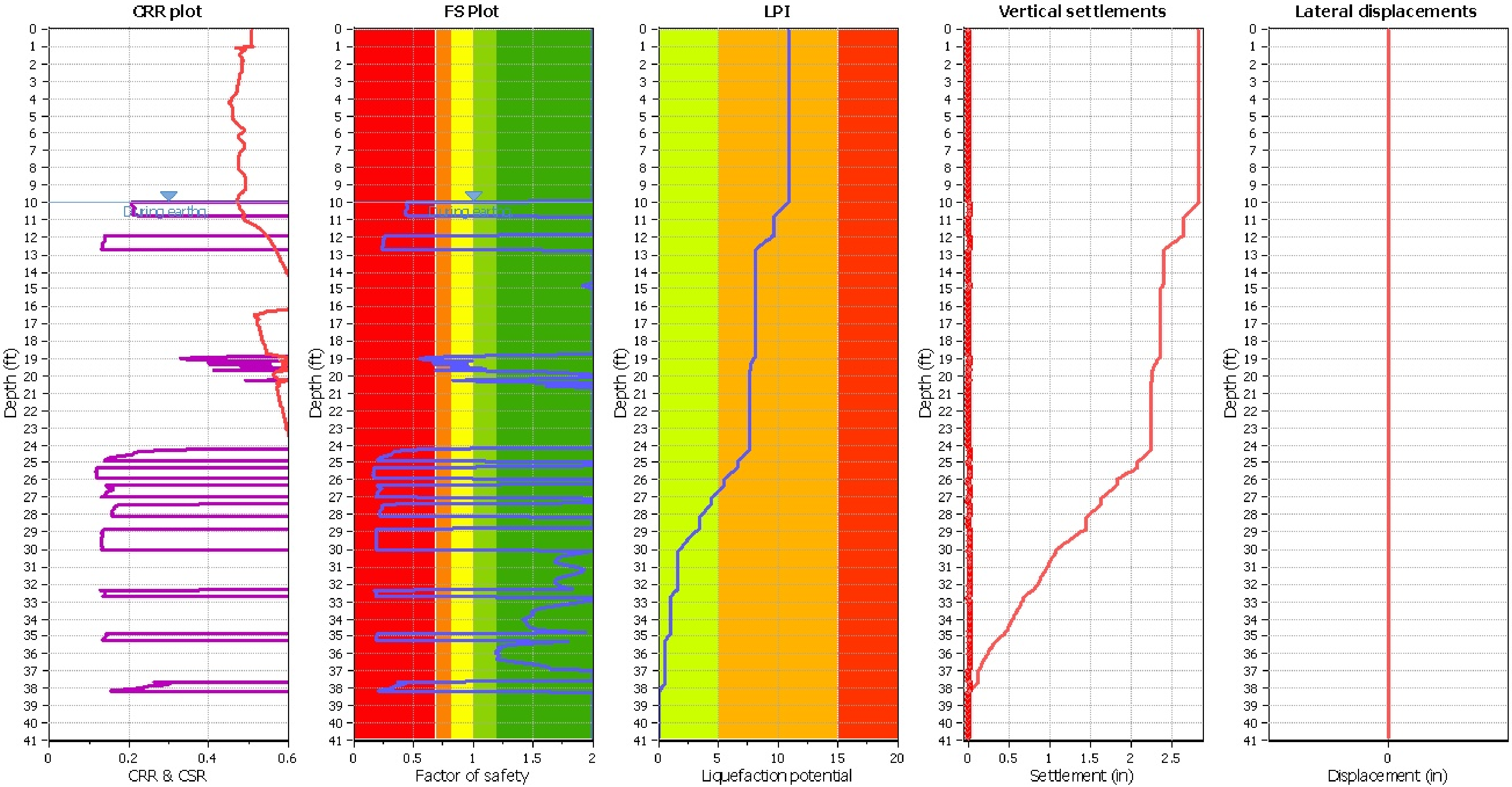
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.88	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 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:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.88	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 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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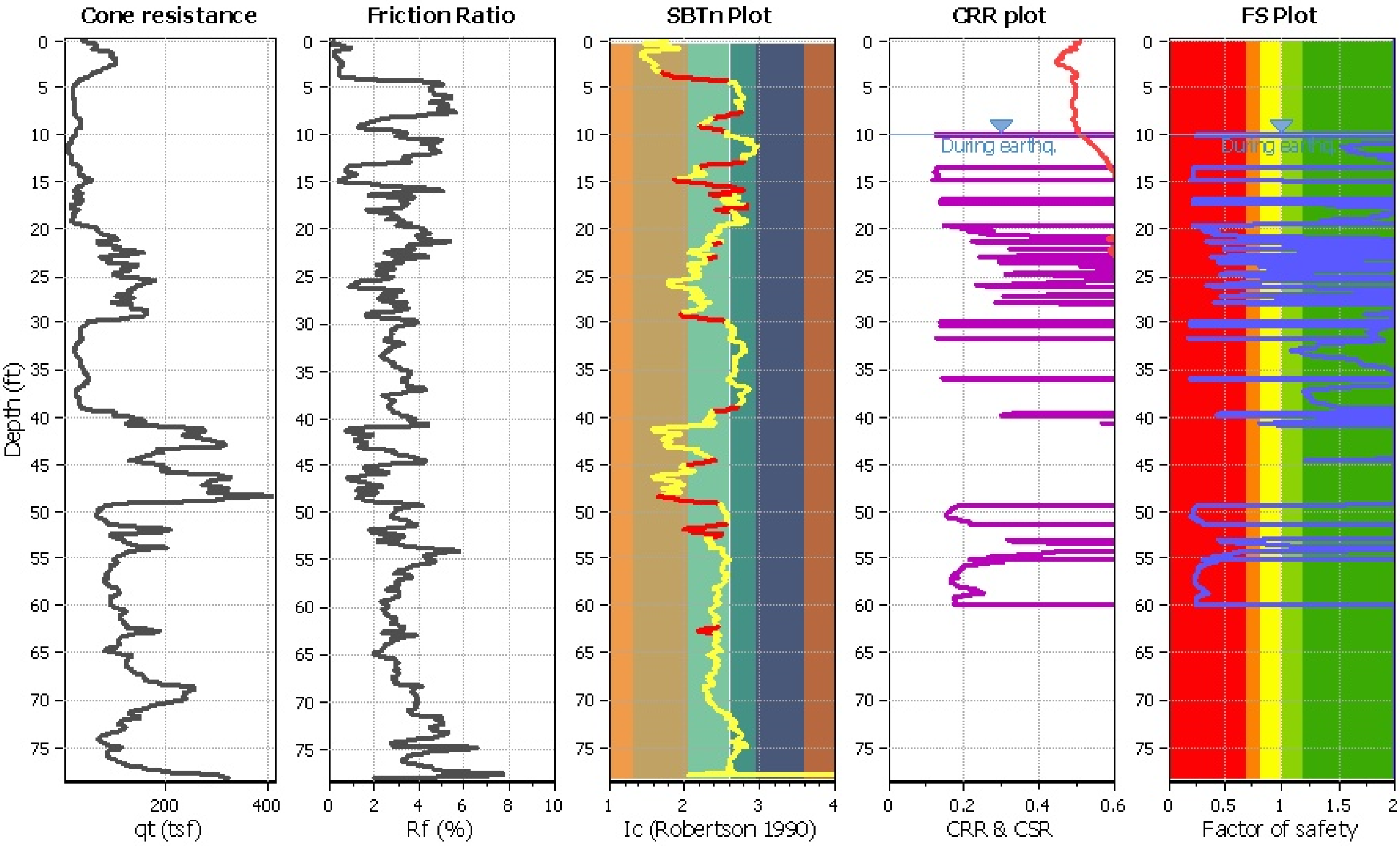
LIQUEFACTION ANALYSIS REPORT

Project title : Palmer **Location : Walnut Creek, CA**

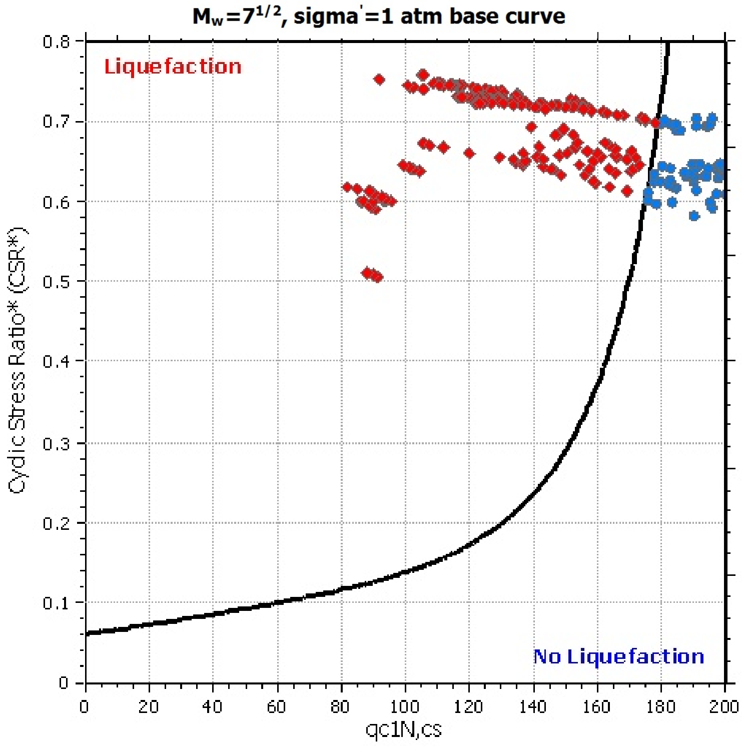
CPT file : 1-SCPT2

Input parameters and analysis data

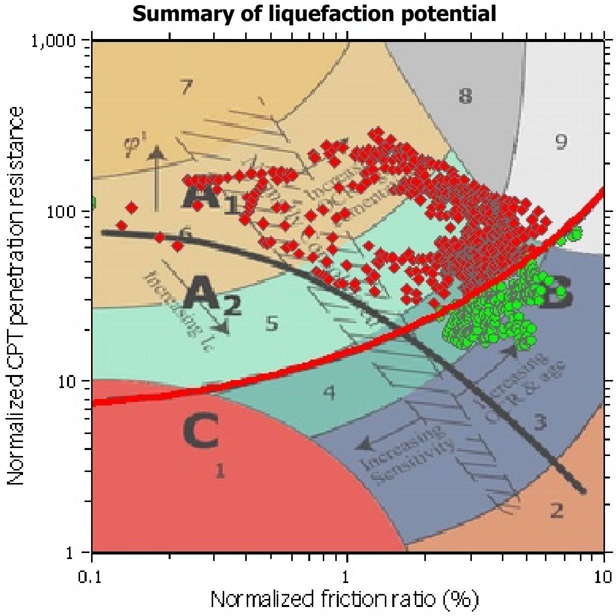
Analysis method:	B&I (2014)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.88	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

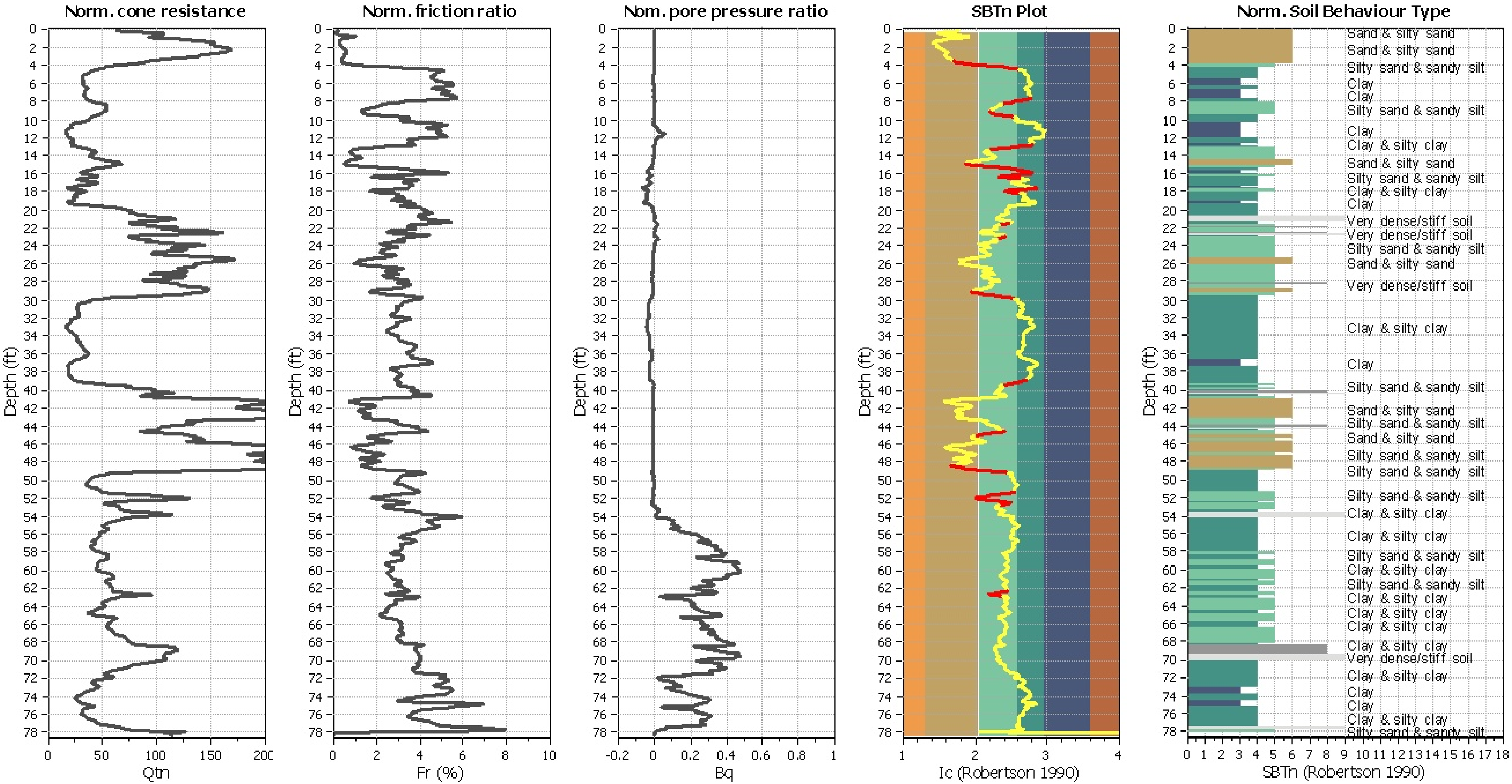


Summary of liquefaction potential



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



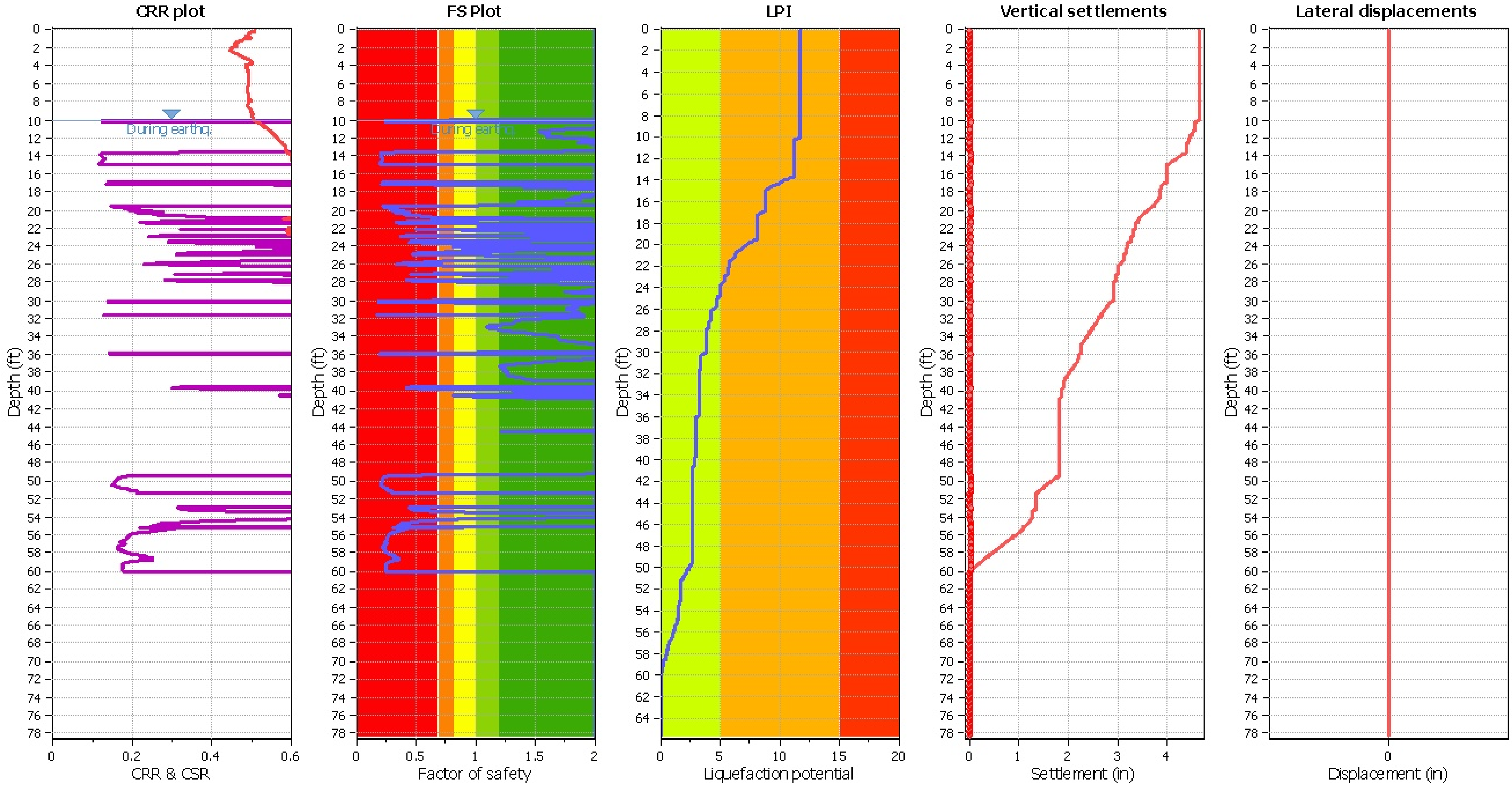
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.88	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 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: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.00
Peak ground acceleration: 0.88
Depth to water table (insitu): 10.00 ft

Depth to GWT (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sand & Clay
Limit depth applied: Yes
Limit depth: 60.00 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

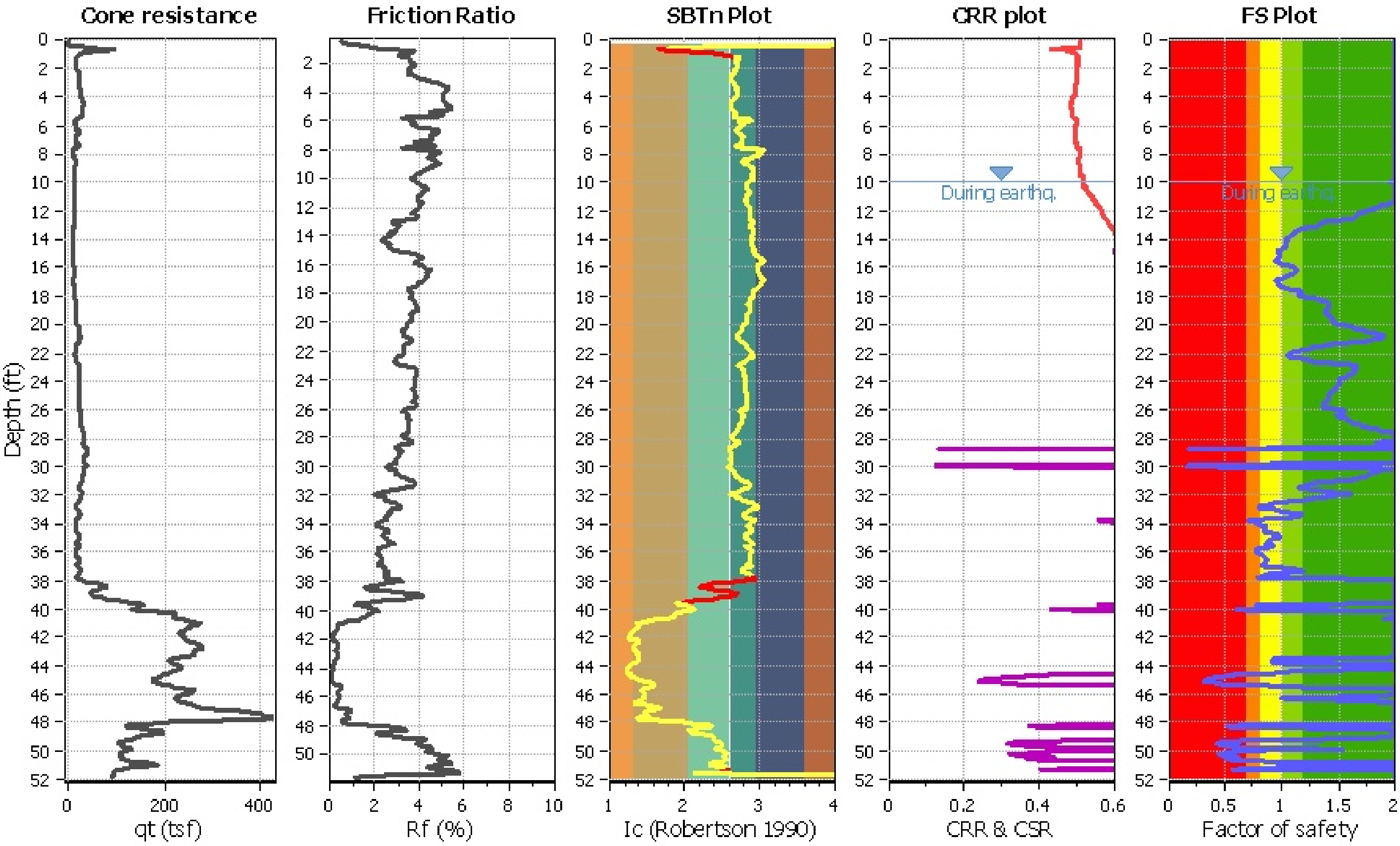
LIQUEFACTION ANALYSIS REPORT

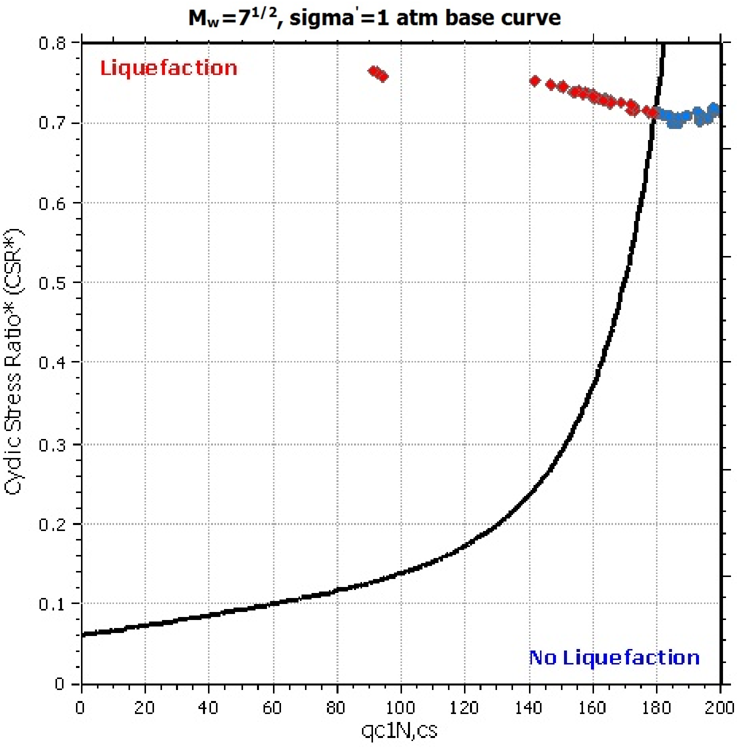
Project title : Palmer **Location : Walnut Creek, CA**

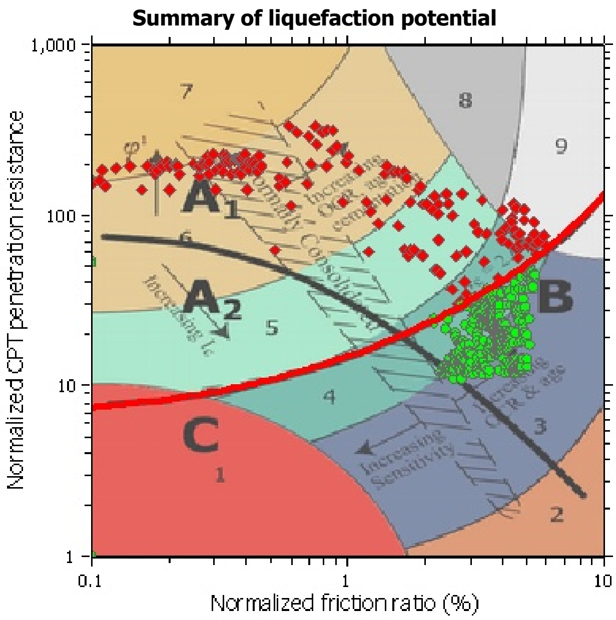
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.88	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
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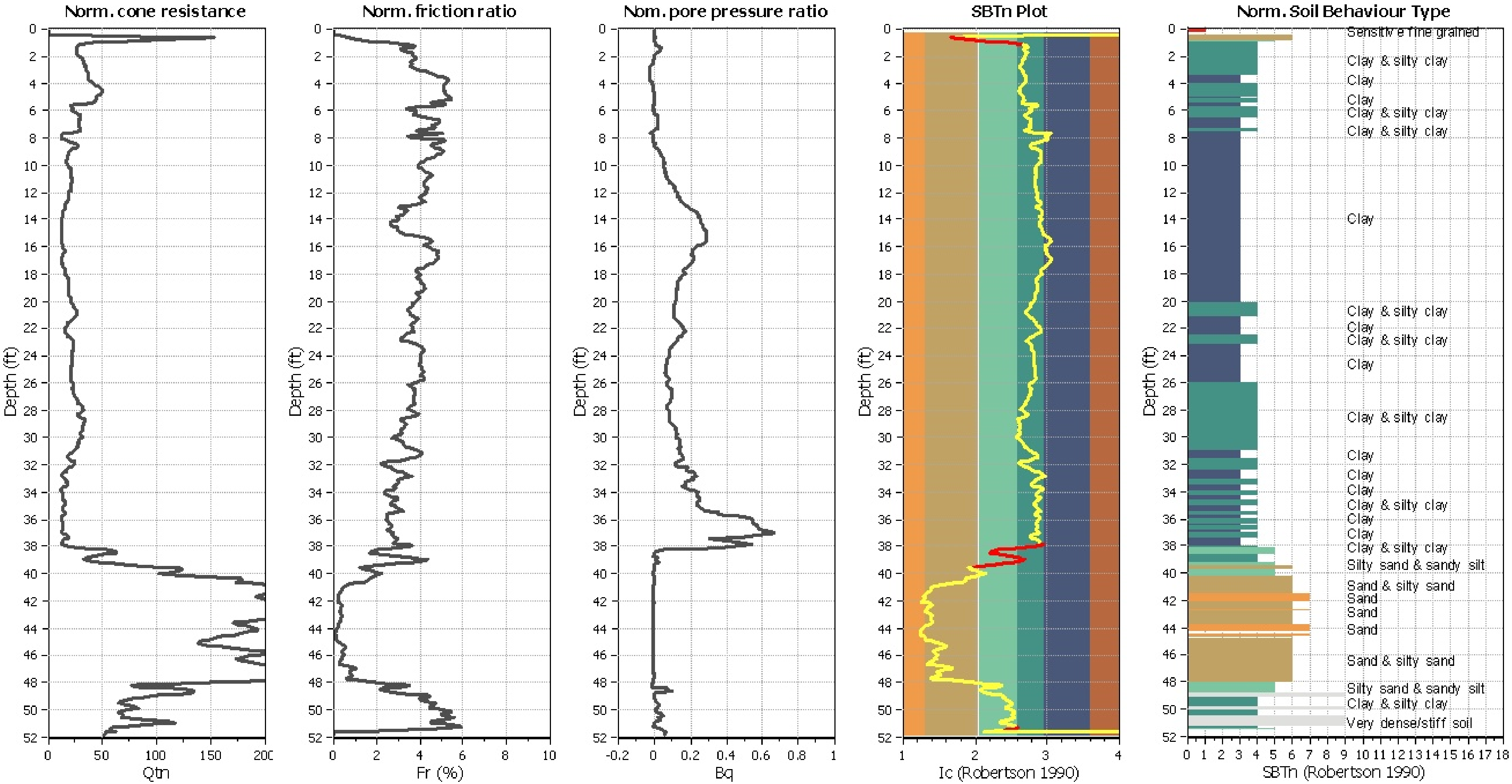
CPT basic interpretation plots (normaliz



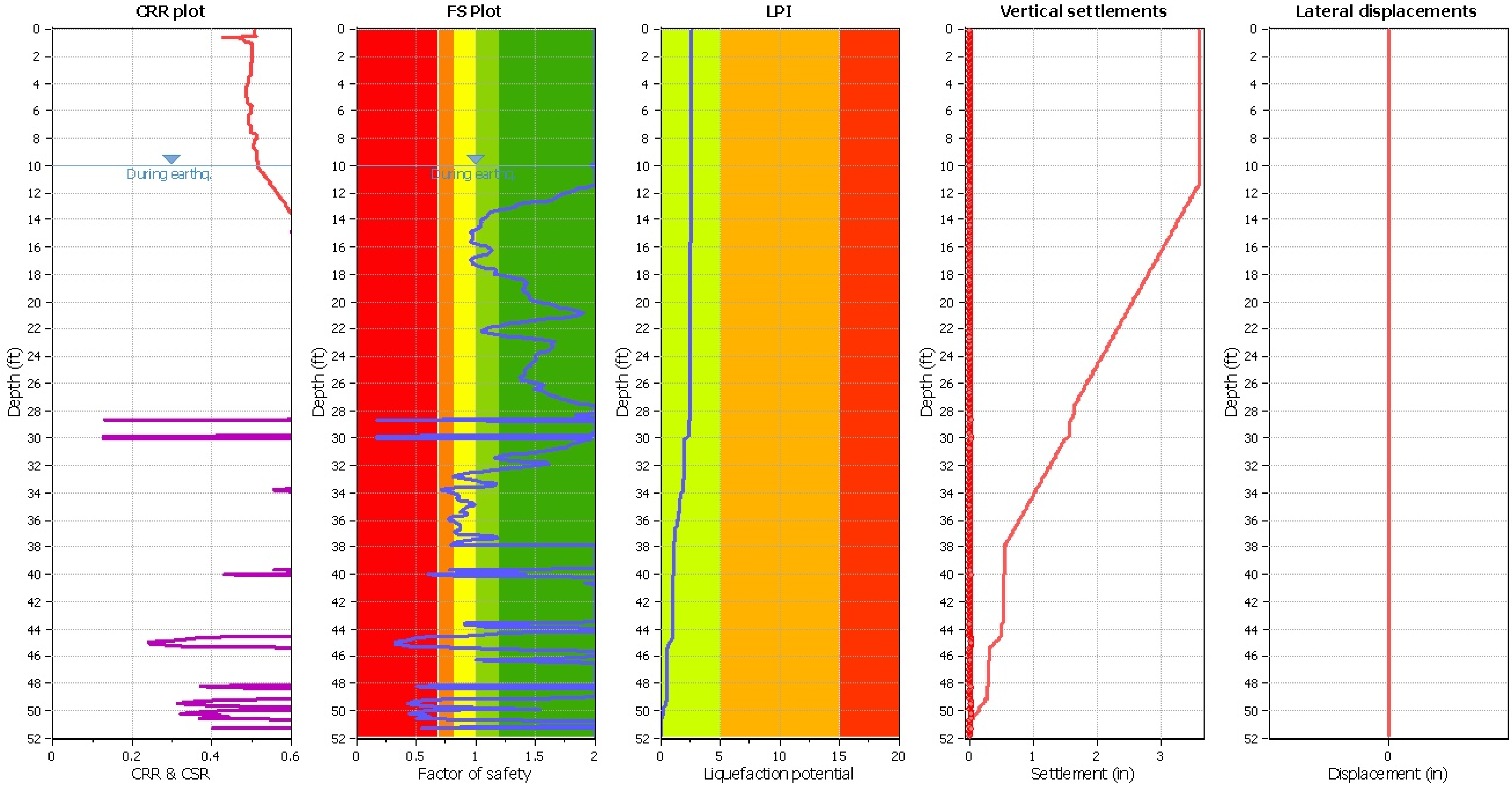
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.88	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 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:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.88	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

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- 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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E.2 - County Geologist Comments

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DARWIN MYERS ASSOCIATES
ENVIRONMENTAL RESEARCH ■ ENGINEERING GEOLOGY

March 19, 2021

Jennifer Cruz, Senior Planner
Contra Costa County
Department of Conservation & Development
Community Development Division
30 Muir Road
Martinez, CA 94553

Subject: **Geologic Peer Review/ 30-Day Comments**
RZ21-3258, SD21-9559 & DP21-3001
Oak Road Townhouse Condominiums (125 units / 5.94 ac.)
APN 172-012-001, -007, -008, -020, -021, -023, -025 & -026
SummerHill Homes (applicant) / Palmer School (owner)
Walnut Creek Area, Contra Costa County
DMA Project #3010.21

Dear Jenn,

Based on your authorization, we have reviewed the application materials submitted by the SummerHill Homes in support of the captioned project. This letter is organized to first outline the purpose and scope of our review. We then provide background information on the geologic and seismic setting of the site before commenting on the geotechnical report. We then present, our evaluation and recommendations.

Purpose

The purpose of our review is to provide a professional opinion on the adequacy of published geologic and soils reports and maps issued by public agencies and professional organizations, in combination with the geologic and geotechnical report prepared by the applicant's consultants, along with the grading- and drainage-related drawings prepared by the project civil engineers to allow full processing of the pending applications. Prior to deeming the application complete, the County requires sufficient data on site conditions to allow: *(i)* delineation the potential geologic hazards based on adequate subsurface data, and *(ii)* the data must be sufficient to serve as the primary basis for preparation of the "Geology and Soils" chapter of the CEQA document. Appendix G of the CEQA Guidelines issued by the State of California identifies the potential geologic and seismic hazards that must be evaluated by the CEQA document (see Table 1 for a list of the potential hazards that must addressed by the CEQA document.

Understanding of the Project

The application is a request to rezone the project to the Planned Unit district, approval of a Vesting Tentative Subdivision Map and Final Development Plan that would allow 125 residential units on the 5.94 acre site, consisting of 3-story town house buildings, with 2-car garages and 30 additional onsite parking spaces. Additionally the applicant is requesting approval of a tree permit that would allow for the removal of 75 trees and the relocation of 1 tree.

1308 PINE STREET ■ MARTINEZ, CA 94553 ■ 925/209-9994

The geotechnical report submitted with the application was prepared by Engeo Inc. [1] The Vesting Tentative Map, as well as the civil engineering drawings showing preliminary grading and drainage plans, and utility plans were prepared by Ruggeri-Jensen-Azar, the project civil Engineers. [2]

Table 1
Appendix G of State CEQA Guidelines

7. GEOLOGY AND SOILS – *Would the project:*				
a) Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury or death involving:				
i) Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault?	☐	☐	☐	☐
ii) Strong seismic ground shaking?	☐	☐	☐	☐
iii) Seismic-related ground failure, including liquefaction?	☐	☐	☐	☐
iv) Landslides?	☐	☐	☐	☐
b) Result in substantial soil erosion or the loss of topsoil?	☐	☐	☐	☐
c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse?	☐	☐	☐	☐
d) Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial direct or indirect risks to life or property?	☐	☐	☐	☐
e) Have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater?	☐	☐	☐	☐
f) Directly or indirectly destroy a unique paleontological resource or site or unique geologic feature?	☐	☐	☐	☐

Scope

The scope of our review included *(i)* geologic analysis of vertical angle aerial photographs using a mirror stereoscope equipped with 3x and 8x binoculars, [3] *(ii)* review of pertinent published geologic reports and maps, *(iii)* review of the Soil Survey of Contra Costa County, *(iv)* review of Safety Element geologic hazard maps and geologic-related Safety Element policies. With that background we *(v)* reviewed the geotechnical report and project plans submitted with the applications, *(vi)* evaluated the data gathered, and

[1] Engeo, Inc., 2020, *Preliminary Geotechnical Exploration, Palmer, Walnut Creek, California,* Engeo, Inc. Job #17567.000.000 (report dated September 1, 2020; date stamped received by DCD on February 1, 2021).

[2] Ruggeri-Jensen-Azar, BKF Engineers, 2021 *Vesting Tentative Map – Sub 9559, Oak Road Townhome and Condominium, City of Walnut Creek, Contra Costa County, CA,* RJA Job #201069 (plans dated January 29, 2021).

[3] Pacific Aerial Surveys, 1973, Aerial Photographs #CC3526-2-199 & -200; scale 1:12,000 (flight date May 7, 1973).

(vii) prepared the 30-Day comment letter presented herein. In summary, the concern of the County at this point in the processing of the application is evaluation of potential geologic, seismic and geotechnical hazards. Detailed technical data on the design of planned improvements is not required by CEQA.

Background

1. Active Faults

Figure 1 presents a Vicinity Map. The project site is outlined in red and is within a red bullseye. The base map shows city boundaries, creeks, freeways, parklands and the local road network. The nearest fault that is considered active by California Geological Survey (CGS) is the northwest trending Concord fault, which passes near the southwest toe of Lime Ridge, approximately 3 mi. northeast of the site. The A-P zones encompassing the recently active and potentially active traces of the Calaveras and Hayward faults pass approximately 6 and 10½ mi. south and southwest of the site, respectively. According to the CGS, recently active and potentially active traces of the active faults may be present anywhere in the A-P zones. The location of future surface rupture generally can be assumed to be along an active major fault traces. Because the subject property is not within the A-P zone, the probability of the project experiencing surface rupture can be considered very low.

It should be recognized that the CGS does not delineate an A-P zone unless it determines there is clear evidence of surface fault rupture during Holocene time (i.e. during the last 11,700 years±). In the case of the Calaveras fault, review of technical data by CGS geologists determined that the Calaveras fault has no proven Holocene offset north of Danville. So, although geologic maps have confirmed that the ancestral traces of the Calaveras fault pass through the Walnut Creek area, it has not been placed in an A-P Zone (i.e. no proven surface fault rupture within Holocene time). Nevertheless, ancestral traces of the Calaveras fault are a potential seismic source. Specifically, a 1998 report prepared by Geomatrix found evidence of activity during the Late Quaternary on this fault system within the Walnut Creek area (minor offset with a right-normal-oblique sense of displacement). The alluvium that was offset was dated 31,410 radio-carbon years before present.[4] This is evidence of seismic activity and at least limited fault rupture on the northern branch of the Calaveras fault during the Late Quaternary. Also shown on Figure 1 are bedrock faults mapped in the site vicinity by the U.S. Geological Survey (USGS), which are represented by green lines. Two of these bedrock faults are shown to be within the western portion of the bullseye. There is no manifestation of these faults on the floor of the Diablo Valley. The locations shown are based chiefly on deep geophysical survey data and explorations wells of petroleum companies, who made their mapping available to the USGS.

2. Bedrock Geologic Map

In 1994 the U.S. Geological Survey (USGS) issued a digitized bedrock geology map of Contra Costa County.[5] Figure 2 presents a portion of this map. The base map shows the local road network, parcels, topography and creeks. It also shows the bedrock faults that were previously shown in Figure 2. The project site is outlined in red, and fronts on both Oak Road and Jones Road. The geologic map indicates the site located within the broad, relatively level, floor of the Diablo Valley. The nearest bedrock is indicated to be approximately 2,000 ft. southeast of the site. They are bedrock formations on Miocene age. Approximately 1 mi. southwest of the site are hills that expose rock of Paleocene age.

---

[4] Geomatrix, 1998. *Final Report, Walnut Creek Water Treatment Plant Expansion, Seismic Study - Phase II.* Geomatrix Job #3970 (report dated October 30, 1998).

[5] Graymer, R., D.L. Jones & E.E. Brabb, 1994. *Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California.* U.S. Geological Survey Open File Report 94-622.

3. Quaternary Geologic Map

In 1997 the U.S. Geological Survey issued a map that divided Quaternary deposits of Contra Costa County into nine categories that vary in age, depositional environment and engineering properties.[6] A portion of this USGS map is presented in Figure 3, where five (5) different surficial deposits are mapped on the valley floor. Note that the legend for Figure 3 divides these units into groups according to age (i.e. those of Holocene age, those of Pleistocene age, and older alluvial deposits). Table 2 presents a brief description of these units. According to this map the project site is located within an area mapped as "alluvial fan and fluvial deposits of Pleistocene age" (Qpaf).

Table 2
Quaternary Deposits Mapped in the Saranap Area

Stream channel deposits (Qhsc). These are deposits of Holocene age (<11,000 years before present), and consist of poorly to well-sorted sand, silt, silty sand or sandy gravel with minor cobbles. Those mapped north of the site are modern stream channel deposits of Las Trampas Creek.
Floodplain Deposits (Qhfp). These deposits are of Holocene age and tend to be medium to dark gray, dense, sandy to silty clay, with lenses of coarser material (silt, sand, pebbles). Floodplain deposits usually occur between levee deposits (Qhl) and basin deposits (Qhb), and are prevalent on the valley floor in the Concord-Walnut Creek area.
Alluvial fan deposits (Qhaf). These deposits are of Holocene age and tend to be brown to tan and medium dense (never reddish) that generally grade upward to sandy or silty clay.
Alluvial Fan deposits (Qpaf). These are deposits of Pleistocene age and tend to be brown, dense, gravelly and clayey sand that fines upward to sandy clay. All Qpaf deposits can be related to modern stream courses, and can be distinguished from younger alluvial deposits by higher topographic position, greater degree of dissection, and stronger soil profile development. They are less permeable than Holocene deposits. In some locations in the San Francisco Bay Region, Qpaf deposits contain fresh water mollusks and extinct late Pleistocene vertebrate fossils.
Undifferentiated continental gravels" (QTu). These deposits are of Plio-Pleistocene age, and are described as semi-consolidated and poorly sorted. They consist of irregularly interbedded gravel, sand, silt and clay. The USGS report states that theses deposits are (a) unrelated to modern drainages, (b) thickness is variable but locally ranges up to 50 meters, and (c) they are regarded as evidence of the late Cenozoic uplift of the Coast Ranges. Other surficial deposits shown on Figure 3 are all younger in age than QTu.
Bedrock (br). This symbol denotes the rocky upland areas that overlook the valley floor area.
Source: USGS Open File Report 97-98

4. Landslide Deposits Map

In 1975 the USGS issued quadrangle maps of Contra Costa County that provide an interpretation of surficial deposits (including landslide deposits). These USGS maps were presented to the County at a scale of 1 in.= 2,000 ft. The map showing the interpretation of the site and vicinity is presented in the Walnut Creek 7.5-Minute Quadrangle.[7] (This set of USGS landslides map was included in a hazard map presented on Page 10-24 of the Safety Element of the County General Plan.) The mapping was performed by an unusually well qualified USGS geologist, and the entire County was mapped during a one year period. The interpretation shown on these maps was based solely on geologic interpretation of aerial photographs, without the benefit of a site visit or any subsurface data. The landslides are not classified on

[6] Helley E.J. and R.W. Graymer, 1997. *Quaternary Geology of Contra Costa County and Surrounding Parts of Alameda, Marin, Sonoma, Solano, Sacramento and San Joaquin Counties, California.* A Digital Database. U.S. Geological Survey, Open File Report 97-98.

[7] Nilsen, T.H., 1975. *Preliminary Photointerpretative Map of Landslides and Other Surficial Deposits of the Walnut Creek 7.5-Minute Quadrangle, Contra Costa County, California.* U.S. Geological Survey Open File Map 75-277-55.

the basis of the activity status (i.e. active or dormant), depth of slide plane (shallow or deep seated), or type of landslide deposit. Nevertheless, the map fulfills its function, which is to *red flag* sites that may be at risk of landslide damage. According to the USGS map there are no landslide deposits within 1 mi. of the project site.

5. Seismic Hazard Zone Mapping Act

The provisions of the Seismic Hazard Mapping Act can be found in the California Public Resources Code, Chapter 7.8, Sections 2690-2699.6. This law is similar in many respects to the Alquist-Priolo Earthquake Fault Zone Mapping Act, which has been implemented by the County for the past 40+ years. However, the official Seismic Hazard Zone (SHZ) maps issued by the California Geological Survey (CGS) identify areas that are at risk of earthquake triggered landslides and earthquake triggered liquefaction. Since 1990, the Safety Element of the Contra Costa County General Plan has included hazard maps for liquefaction and landslide potential (see pages 10-15 & 10-24, respectively), along with adopted General Plan goal statements and policies. We consider those policies statements to be comprehensive. They are intended to mitigate hazards posed by liquefaction and landslides, and we do not expect those policies to be invalidated by the SHZ maps. However, as SHZ maps are issued to the County, they will be utilized in place of the mapping that is currently presented in the Safety Element. In 2018 the CGS commenced issuing SHZ maps of Contra Costa County. To data the adopted maps are of the East County and a portion of the North Coast. To date the SHZ map of the Walnut Creek Quadrangle has not yet been issued. County is required to implement the provisions of this state law. Unless the SummerHill Homes project is built-out prior to issuance of the official SHZ map of the Walnut Creek Quadrangle, the project will be subject to the provisions of this state law.

For projects that fall under the authority of the SHZ Mapping Act, the required investigations must be prepared by a certified engineering geologist and/or geotechnical engineer registered in the State of California. A copy of each consultant prepared report, along with evidence of peer review by the local jurisdiction, must be forwarded to the CGS within 30 days of County approval of the report. (This requirement provides the CGS with a basis for modifying the boundary of the hazard zone in the future, as detailed studies define locations within the delineated hazard zone that are free of hazards.) The CGS has adopted guidelines for the required reports, and there are guidelines for the peer review of the reports that are triggered by the SHZ mapping of hazardous areas.

6. Soils

According to the Soil Survey of Contra Costa County,[8] the soil series mapped on the site is the Clear Lake clay (Cc, 0-2% slopes). Permeability is slow and the available water holding capacity is 8 to 10 inches. The typical soils profile is 60 inches deep. The A horizon extends from the ground surface to a depth of 30 inches below the ground surface, and is described as a dark gray, very dark gray or black clay. The ACca horizon extends from 30 to 46 inches below the ground surfave. It is a dark gray or very dark gray clay. During the summer desiccation cracks ½ to 2 inches wide extend to the C-horizon (46 inches below the ground surface. The C horizon, which extends from 46 to 60 inches below the ground surface is olive, light olive brown or grayish brown and is mottled in places. It is a clay loam, silty clay loam or light clay. clay loam, With regard to engineering properties, the Clear Lake clay *highly* expansive and *very highly* corrosive.

Expansive soils expand when water is added and shrink when they dry out. This continuous change in soils volume causes homes and other structures to move unevenly and crack. Corrosive soils tend to damage concrete and/or uncoated steel that is in contact with the ground. Testing of the soils on the site is

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[8] Welch, L.E. et. al., 1977, *Soil Survey of Contra Costa County, California,* USDA Soil Conservation Service.

needed to confirm foundation conditions. Corrosivity is typically measured after rough grading is completed to ensure that corrosivity testing is based on pad conditions. Design-level geotechnical reports routinely provide specific criteria and standards to avoid/ minimize damage from expansive and corrosive soils.

7. Seismicity

The San Francisco Bay Region is considered one of the most seismically active regions of the United States. Consequently, it can be assumed that the proposed improvements will be subject to one or more major earthquakes during their useful life. Earthquake intensities vary depending on numerous factors, including *(i)* earthquake magnitude, *(ii)* distance of the site from the causative fault, *(iii)* geology of the site, *(iv)* duration of earthquake shaking, and other factors. The USGS has stated that there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the Bay Region between 2014 and 2043.[9]

The Safety Element includes a figure titled "Seismic Ground Response" (General Plan, page 10-13). This map classifies the site as *moderately low damage susceptibility.* Buildings in this zone that are conservatively design and properly constructed typically perform satisfactorily, provided foundation materials and critical slopes are stable. The risk of structural damage from earthquake ground shaking is strongly influenced by building and grading regulations. According to the California Building Code (CBC), structures requiring building permits require that the design take into account both foundation conditions and proximity of active faults and their associated ground shaking characteristics. Design-level geotechnical reports must include CBC seismic design parameters. Those parameters are used by the structural engineer in the design of civil engineering structures. All grading on the site must comply with the provisions of the County Grading Ordinance. Compliance with building and grading regulations can be expected to keep risks within generally accepted limits.

Safety Element

1. Liquefaction

The liquefaction potential map presented in the Safety Element of the General Plan divides Contra Costa County into three categories: "generally high", "generally moderate to low", and "generally low" liquefaction potential. This map was prepared by a geotechnical consulting firm that used available data on soil conditions, depth of the ground water table and limited review of geotechnical reports in County files during the late 1980s. The product of the consultant's investigation was incorporated into the County General Plan in 1990. Figure 4 presents an enlargement of the portion of liquefaction map presented in the General Plan that shows the site an adjacent area.According to this map, the northeast half of the project site ic classified Generally High liquefaction potential, and the southwest half of the site is rated *Generally Moderate to Low* liquefaction susceptibility. During the processing of land development applications, the County requires rigorous evaluation of liquefaction potential in areas of *Generally High* liquefaction potential, and less comprehensive investigations are demanded in the *Moderate to Low* category. For project sites classified *Generally Moderate to Low* liquefaction potential, the expectations of the County are minimal, except perhaps for critical facilities.

The classification *Generally High* liquefaction potential does not imply the presence of liquefiable sands on a parcel. The map attempts to be conservative of the side of safety. Where geologically recent fluvial

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[9] Aagaard, Blair, Boatwright, Garcia, Harris, Michael, Schwartz, and De Leo, 2016, *Earthquake Outlook for the San Francisco Bay Region, 2014-204M3*, USGS Fact Sheet 2016-3020, revised August 2016; ver. 1.1)

deposits or sand bars could exist in the subsurface, the map places such areas in the *Generally High* category. Site specific investigations are needed to determine if liquefiable sands are present and to provide stabilization measures where liquefiable sands are confirmed. It should also be recognized that a 1997 USGS Quaternary Geologic Map classifies the surficial deposits on the site as Late Pleistocene alluvium (see Figure 3). In the experience of the County peer review geologist, project sites that are underlain by Pleistocene alluvium are not candidates for liquefiable sands. The Safety Element includes a number of policies indicating that at-risk areas require evaluation of liquefaction potential and effective mitigation of the hazard posed to new development. Operative General Plan policies are presented in Table 2.

Table 2
General Plan Liquefaction Policies

Policy 10-18. This General Plan shall discourage urban or suburban development in areas susceptible to high liquefaction dangers and where appropriate subject to the policies of 10-20 below, unless satisfactory mitigation measures can be provided, while recognizing that there are low intensity uses such as water-related recreation and agricultural uses that are appropriate in such areas.
Policy 10-19. To the extent practicable, the construction of critical facilities, structures involving high occupancies, and public facilities shall not be sited in areas identified as having a high liquefaction potential, or in areas underlain by deposits classified as having a high liquefaction potential.
Policy 10-20. Any structures permitted in areas of high liquefaction damage shall be sited, designed and constructed to minimize dangers from damage due to earthquake-induced liquefaction.
Policy 10-21. Approvals to allow the construction of public and private development projects in areas of high liquefaction potential shall be contingent on geologic and engineering studies which define and delineate potentially hazardous geologic and/or soils conditions, recommend means of mitigating these adverse conditions, and on proper implementation of the mitigation measures.

Engeo Investigation

1. Purpose and Scope

The purpose of the investigation was to provide a preliminary assessment of geotechnical and seismic hazards, and provide preliminary recommendations for initial land planning and cost estimating purposes). In summary, the report was intended to identify the primary concerns associated with the residential use of the site. At the time of the investigation, Engeo was provided with a preliminary plans for the SummerHill Homes residential project. Their scope of work included: *(i)* site reconnaissance data; *(ii)* review of pertinent geologic maps and reports; *(iii)* limited subsurface exploration of the project site; *(iv)* evaluation of the data gathered; and *(v)* preparation of a report intended document the investigation and present Engeo's evaluation of potential hazards, preliminary earthwork recommendations, comments on the scope of the recommended design level report, along with a statement of limitations. Clearly the scope and intent of the 2020 Engeo investigation was to provide a preliminary characterization of potential geologic and seismic hazards, what will require further investigation to confirm/ modify their preliminary assessment. Additionally the design level report will provide a full range of geotechnical recommendations for the project based on review of construction drawings.

2. Historical Aerial Photograph Review

Engeo reviewed aerial photographs that covered 80 year period (1939-2019). Prior to 1939 the site served as agricultural land with orchards, small buildings with vacant land. In 1939 the Palmer School was established on the site and minor development activity began occurring in the vicinity. From 1968 to the present, the project site has remained relatively unchanged. Engeo does report removal of two pools on the site. There is no evidence that backfilling of the pools was supervised by an engineer or that the work was done under a permit. Hence the suitability of the backfill for the support of the planned improvements is undocumented.

3. Subsurface Exploration

Field exploration was performed on August 14, 2020. The approach was to utilize a Cone Penetration Test (identified as 1-CPT1 in the report), was located near the north P/L, and positioned midway between the east and west P/Ls. A second CPT (identified in the report as 1-CPT3) was located in the southeast quadrant of the site. Additionally, one seismic cone penetration test (identified as 1-SCPT2) was located near the western P/L of the project site, (approximated 250 ft. southwest of 1-CPT1; and 330 ft. northwest of 1-CPT3). Engeo report on page 3 provides background information of the methodology and data collected, and Figure 2 of the Engeo report shows the location of these subsurface data points. Table 3 of our peer review letter provides information of the depth of the probes, water table depth and description of the material penetrated. The probes were advanced up to depths of 70 ft. Figure 2 of the Engeo report show the approximate location of the CPT probes. A key finding of the investigation was that the water table is relatively shallow.

Table 3

Summary of Subsurface Data Gathered by Engeo, Inc.

CPT #	Location of CPT on Project Site	Total Depth of CPT Probe	Est. Water Table Depth Below Ground Surface	Est. Elevation of Ground Surface
1-CPT1	Near North P/L	40.85 ft.	18.8 ft.	94 ft.
1-SCPT2	Near West P/L	75.25 ft.	14.5 ft.	94 ft.
1-CPT2	Within SE Quadrant	51.84 ft.	20 ft.	95 ft.

The alluvial penetrated by the probes is described by Engeo as follows:

(i) 1-CPT. The alluvial deposits penetrated are primarily composed of layers ranging from 1 to 12 ft. in thickness. These deposits are characterized as irregularly interbedded medium-dense to loose sand, medium stiff to v. soft silt-like and clay-like deposits. A 6 ft. thick bed of medium dense to v. dense sand was penetrated approx.18 to 24 ft. below the ground surface (bgs).

(ii) 1-SCPT2. The alluvial deposits penetrated were composed chiefly of discontinuous layers up to 6 ft. in thickness, described by Engeo as medium stiff to very soft clay and clayey silt. A lens of dense to very dense sand was penetrated between depths of 41 and 49 ft. bgs.

(iii) 1-CPT3. The alluvial deposits penetrated from the ground surface to 40 ft. bgs are described by Engeo as continuous deposit of soft to very soft silty clay and clayey silt. A lense of dense to very-dense sand and gravelly sand was encountered from 41 to 49 ft. bgs.

4. Hazards Evaluation

Engeo considers the project feasible rom a geologic/ geotechnical standpoint. The primary concerns are *(i)* areas of relatively shallow groundwater, *(ii)* liquefaction induced settlement, *(iii)* potential for consolidation of compressible material, *(iv)* expansive soils, *(iv)* compressibility of relatively soft alluvial clay at depth. Other potential hazards include *(v)* earthquake ground shaking, and *(vi)* and an unknown potential for corrosive soils. The discussion presented in Table 4 is intended to highlight and summarize (not supersede) the evaluation of the project geotechnical engineers. Not addressed by Table 4 is the potential for direct or indirect potential for destruction of a unique paleontological resource or unique geologic feature. Based on the engineering properties of the alluvial deposits within 20 ft. of the ground surface, it is our opinion that these deposits are of Holocene age. Deposits of the age are generally considered to be too young to possess unique fossil resources, and there is no expectation of unique geologic features on the site. Consequently, it is our conclusion that item f) in Table 1 can be considered to have a less-than-significant impact and does not require further evaluation.

Table 4
Engeo Evaluation of Potential Hazards
SummerHill Homes Residential Project

- **Ground Rupture.** The site is not within an Alquist-Priolo Earthquake Fault Zone. On that basis the risk of surface fault rupture within the site is low.
- **Ground Shaking.** The site is within the seismically active San Francisco Bay Region area, where a moderate to high magnitude earthquake is a foreseeable event. The risk of damage from ground shaking is controlled by using sound engineering judgement and compliance with the latest provisions of the California Building Code (CBC), as a minimum. The seismic design provisions of the CBC prescribe minimum lateral forces applied statistically to the structure(s), combined with the gravity forces and 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. The intent of the code is to enable structures to *(i)* resist minor earthquakes without damage, *(ii)* resist moderate earthquakes without structural damage but with some non-structural damage, and *(iii)* resist major earthquakes without collapse but with some structural as well as non-structural damage.
- **Liquefaction/ Cyclic Softening.** This hazard is characterized as a phenomena in which a saturated, relatively clay free sand is subject to a temporary loss of shear strength because of a buildup of pore pressure. Engeo references a 2000 map published by the USGS.[10] According to this publication, the site is within an area rated *Moderate Susceptibility to Liquefaction*. Based on data gathered during the subsurface investigation, Engeo assigned the following parameters: (i) depth of the water table 10 ft., (ii) peak ground acceleration 0.88g's and a maximum earthquake moment magnitude of 7.0. Using the methodology required for projects in the official Seismic Hazard Zone, Engeo calculated the vertical settlements based on procedures recommended by Zhang et.al. (2002).[11] Engeo's concludes that the analysis found the following: *(i)* a maximum of 4¾ inches of total liquefaction induced settlement (max.) may occur during the design earthquake; and *(ii)* differential settlement of 2⅓ inches over a horizontal distance of 40 ft. These estimates are not for final design purposes. The geotechnical design-level report shall include soil borings and laboratory testing of the sand layer(s) to more accurately provide estimate liquefaction-related total and differential settlement.
- **Lateral Spreading.** Lateral spreading is a failure within weak soils, typically due to liquefaction, which causes the soil mass to move toward an open channel or down even a gentle slope. The Engeo report indicates that the risk of a lateral spreading failure is anticipated to be low.
- **Compressible Soil.** Soil is subject to consolidation settlement when a new load is introduced. The time required for settlement is highly dependent on the permeability of the soils. Other factors include the depth, thickness and stress history of the compressible soil, as well as the magnitude/ shape/ size of the applied load. Based on the CPT data, Engeo estimates that the alluvial deposits from 0 to 27 ft. bgs and 31 to 37 ft. bgs. Are relatively soft and may undergo consolidation. Engeo states that some consolidation will occur during the construction period, and with appropriate structural design, the remaining consolidation can be accommodated by the foundation.
- **Existing Non-Engineered Fill.** Based on review of historic aerial photographs and historic topographic maps, site topography has not changed significantly. Nevertheless, three swimming pools were

[10] Knudsen, K.L. et.al., 2000, *Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California: a Digital Database*, USGS Open File Report 00-444

[11] Zhang, G. et.al., 2002, *Estimating Liquefaction-Induced Ground Settlements from CPT for Level Ground*, Canadian Geotechnical Journal, Vol. 39(5), pgs. 1168-1180.

constructed on the site and there may be other areas of weak fills on the site associated with the previous uses It is the opinion of Engeo that other fill areas shallow undocumented fill may be present elsewhere on the site. Engeo recommends that this is a subject that will need to be evaluated at the time of the design level geotechnical report. It is anticipated that all undocumented fills on the site will be over-excavated during site grading and replaced with engineered fill.

- **Expansive Soil.** No laboratory testing has been performed for the 2020 Engeo investigation. Nevertheless, the CPT data confirms the presence of clayey alluvial deposits in the subsurface. Based on their origin (i.e. floodplain deposits) Engeo concludes that the fine-grained alluvial deposits and clay matrix material of sandy lenses are likely to be expansive, and will need to be evaluated during the design-level geotechnical report. Special engineering approaches identified by Engeo, include to mitigate the potential damage of expansive soils, Engeo recommends *(i)* selective grading or blending to create relatively low expansion potential engineered fill at the ground surface, *(ii)* using a rigid mat foundation system, *(iii)* deepening foundations below the zone of moisture fluctuation, and/or *(iv)* using lime treatment in the upper 18 inches of the building pad soil to reduce its expansion potential.
- **Soil Corrosion Potential.** The scope of the Engeo investigation did not include laboratory testing to evaluate the corrosion potential of soils on the site (Use of CPT probes, does not allow for sampling of soils). Engeo does recommend that the scope of the design-level report include laboratory testing of foundation grade soils. Depending on the corrosion potential of the soils, Engeo states that recommendations can be provided to protect soil and steel that is in contact with the ground. If subsurface transformers are proposed within the project (for underground utilities), the consultant recommends that subsurface samples be obtained and tested in accordance with recommendations that have been set forth by Pacific Gas & Electric.
- **Shallow Groundwater.** It the time of the Engeo investigation, groundwater was encountered at 10 to 20 ft. below the ground surface. Based on their experience, Engeo states there is potential for shallow groundwater depths on the project site to potentially affect the proposed development. Specifically shallow groundwater can *(i)* impede grading operations, *(ii)* require dewatering during grading and construction of improvements, *(iii)* cause moisture damage to sensitive floor coverings, and *(iv)* transmit moisture vapor through slabs, causing buildup of mold/mildew, fogging of windows and damage to computers and other sensitive equipment. Based on the existing data groundwater depths on site, removal of existing fill on the site and deep utility trench excavation may encounter groundwater. The project contractor should evaluate site conditions and select properly designed dewatering and shoring systems. Engeo also cautions that groundwater levels may fluctuate seasonally, and may be significantly influenced by, irrigation of plantings, water and/or sewer leaks and other factors.
- **Flooding.** On page 8 of their report, Engeo indicates that the Flood Insurance Rate Map issued by FEMA indicates that the site is located in Zone X (i.e. the 500–yr. flood zone). Therefore, it is Engeo's preliminary assessment that the risk of flooding is low. The discussion in the geotechnical report goes on to suggests that the project civil engineer should review pertinent information on the elevation of the flood zone and provide appropriate design measures for the proposed project.
- **Seismic Design Parameters.** On page 9 of their report, Engeo provides seismic design parameters that are based on the 2019 CBC. Engeo goes on to suggest that a site-specific seismic hazard analysis can optimize the spectral values at the short period range, and Engeo offers to collaborate with the project structural engineer to further evaluate the effects of taking the advantage of the exceptions on the structural design; and identify the potential advantages of performing a site-specific seismic hazard analysis. (In our opinion, this approach warrants support of the County.)

5. Preliminary Recommendations

Engeo provides recommendations that are intended for initial land planning and preliminary estimating purposes that commence on page 9 of their report. Final recommendations are to be provided after a future design-level investigation has been performed, which includes subsurface exploration (borings), laboratory testing of selected samples, and engineering analysis of the data gathered. The resulting recommendations to be presented in the design level report. Nevertheless, the 2020 report provides preliminary recommendations that address *(i)* foundation design, *(ii)* earthwork (including clearing, demolition, removal of existing fill and fill compaction), *(iii)* pavement design, *(iv)* surface drainage, and *(v)* scope of the design-level geotechnical report, along with *(vi)* a limitations statement that includes the proper use of the report by the project proponent, and the limitations of the investigation methods. This statement is followed of a list of selected reference and six maps that that provide background data.

Grading and Drainage Plans

1. Grading

The civil engineers for the project are Ruggeri-Jensen-Azar, who have prepared preliminary grading and drainage plans (Sheet TM4.0), along with other civil engineering drawings. The information provided does not include earthwork quantities, but the site is nearly level and earthwork volumes are expected to be limited. Note that earthwork volumes will be affected by shrinkage, swelling or foundation elements. Additionally they will be influenced by the volume of existing fill material that the Engeo may determine to be unsuitable for use in engineered fill.

The Grading & Drainage Plan indicate that the finished floor elevations for proposed buildings range from+94.4 ft. to 96.7 ft. Sheet TM4.0 provides typical sections for the internal roads. The internal roads intended to carry heavier volumes of traffic (labels "streets" are 25 ft. in width; minor streets (labeled "Courts" are 20 ft. in width. The gradient of internal roads is to range from 0.5% to 1.3%.

2. Drainage

Sheet TM4.0 also show the drainage plan for the project, which includes 12 in. diameter on-site storm drains which ultimately outfall to existing storm drainage facilities in Jones Road and Oak Road. Sheet TM6.0 presents the Preliminary Stormwater Control Plan. It indicates a series of small water quality basins that are distributed throughout the project. The intent of the plan is to direct roof gutter water to culverts that will outfall into a water quality basin for treatment prior to exiting the site. Most basins are very near the foundations of residential buildings and/or curbs and pavement. The primary concerns with bio-retention structures are *(i)* providing suitable support for foundations and curbs constructed near the bio-retention facilities, and *(ii)* potential for subsurface water from the bio-retention areas to migrate (and possibly build up) beneath pavements and the proposed buildings. Even if the basins are designed with impervious materials to preclude lateral migration of water, they must be properly maintained to function as designed. The design and sizing of the basins must satisfy the C.3 requirements of the Regional Water Quality Control Board. Review of the Stormwater Control Plans is performed by the professional staff of the Public Works Department. Our comments are limited to the engineering geologic aspects of the basins: *(i)* setbacks of bio-retention basins from improvements, and *(ii)* importance of requiring long-term commitment to inspection and maintenance of these facilities by competent authority.

DMA Evaluation

The immediate need of the Department of Conservation & Development is to determine if there is sufficient data to allow the processing of the pending applications, including preparation of the California Environmental Quality Act (CEQA) document. The provisions of CEQA and associated case law acknowledge that final design studies are not needed for the purposes of CEQA compliance. However, there must be sufficient information on the extent of potential geologic and geotechnical hazards, and guidance must be provided to the project designers pertaining to the layout of the planned improvements.

Therefore, the type of data needed at this stage of the land development process is limited to the following

i. Evaluation of the project plans by the geotechnical engineers to ensure the layout is sensitive to geologic and geotechnical constrains.

ii. The assessment of hazards identified by Engeo addresses the gamut of potential geologic, seismic and geotechnical hazards identified in Appendix G of the CEQA Guidelines issued by the State of California (see Table 1). In our experience, the expectation of the County is that the project geologists and geotechnical engineers provide at least a preliminary evaluation of potential geologic hazards, and provide recommendations to mitigate any significant hazards that are confirmed to be present. We believe that threshold has been satisfied by the Engeo report (Table 4 presents a summary of impacts and preliminary mitigations). We note that preliminary assessment of potential hazards and assoicaated recommendation intended as guidance on geotechnical constrains that will require further evaluation in the design-level geotechnical report. The purpose of that aspect of the design-level report will confirm (or modify) Engeo's preliminary assessment and add needed specificity to the mitigation measures. The future geotechnical report will also provide specific standards and criteria for site grading, drainage and foundation design that are based on the specific approach to development.

In summary, it is our opinion there is sufficient available data available from the Engeo report, in combination with reconnaissance data presented herein, to deem the application complete.

DMA Recommendation

The following are recommended mitigation measures and/or conditions of approval.

GEO-1 At least 60 days prior to recording the final Subdivision Map, requesting issuance of construction permits or installation of utility improvements, the project proponent shall submit a design-level recommendations for the project, based on adequate subsurface exploration, laboratory testing and engineering analysis.

The scope of the geotechnical investigation should address to fully evaluated the following potential hazards: *(i)* grading, including removal of existing undocumented fill that is deemed to be unsuitable for use in engineered fills, preparation to receive fill, compaction standards for fill, etc., *(ii)* consolidation settlement, *(iii)* analysis of liquefaction potential, including estimating total settlement and differential settlement, and surface manifestation of liquefaction, *(iv)* foundation design, *(v)* measures to protect improvements from the relatively shall water table, *(vi)* laboratory testing to evaluate the expansive and corrosion potential soils, and measures designed to protect improvement that are in contact with the ground from these hazard, including the building foundation, parking garage slabs, flatwork, pavement and utilities, *(vii)* exploration/ testing/ and engineering analysis aimed at providing recommendations pertaining to foundation design, including foundation retaining walls, and pavement design *(viii)* evaluation of the drainage design, including the proposed bio-retention facilities and their effect on planned improvements, *(ix)* address temporary shoring and support of excavations, *(x)* provide updated California Building Code seismic parameters, and *(xi)* outline the recommended geotechnical monitoring, commencing with clearing and demolition, extending through final grading, installation of drainage improvements, and including the monitoring of foundation related work.

We strongly support the suggestion of Engeo that they work with the project structural engineers to optimize the spectral values at the short range period waves associated with earthquake ground shaking,

GEO-2 The geotechnical report shall be subject to review by the County's peer review geologist, and review/approval of the Zoning Administrator. Improvement, grading and building plans shall carry out the recommendations of the approved report.

GEO-3 The geotechnical report required by GEO-1 routinely includes recommended geotechnical observation and testing services during construction. These services are essential to the success of the project. They allow the geotechnical engineer to *(i)* ensure geotechnical recommendations for the project are properly interpreted and implemented by contractors, *(ii)* allow the geotechnical engineer to view exposed conditions during construction to ensure that field conditions match those that were the basis of the design recommendations in the approved report, and *(iii)* provide the opportunity for field modifications of geotechnical recommendations (with BID approval), based on exposed conditions. The monitoring shall commence during clearing, and extend through grading, placement of engineered fill, installation of recommended drainage facilities, and foundation related work. A ***hard hold*** shall be placed on the "final" grading inspection, pending submittal of a report from the project geotechnical engineer that documents their observation and testing services to that stage of construction, including monitoring and testing of backfilling required for utility and drainage facilities.

Similarly, a ***hard hold*** shall be placed on the final building inspection apartment building, pending submittal of a letter-report from the geotechnical engineer documenting the monitoring services associated with implementation of final grading, drainage, and foundation-related work. The geotechnical monitoring shall include documentation of conformance of retaining wall, pier hole drilling/ foundation preparation work and installation of drainage improvements.

GEO-4 All grading, excavation and filling shall be conducted during the dry season (April 15 through October 15) only, and all areas of exposed soils shall be revegetated to minimize erosion and subsequent sedimentation. After October 15, only erosion control work shall be allowed by the grading permit. Any modification to the above schedule shall be subject to review by the Grading Inspector, and the review / approval of the Zoning Administrator.

Limitations

This review has been performed to provide technical advice to assist the Community Development Division with discretionary permit decisions. Our services have been limited to review of the documents identified in this peer review letter in combination with geologic analysis of historic aerial photographs. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the engineering geology profession.

We trust this letter provides the evaluation and comments that you requested. Please call if you have any questions.

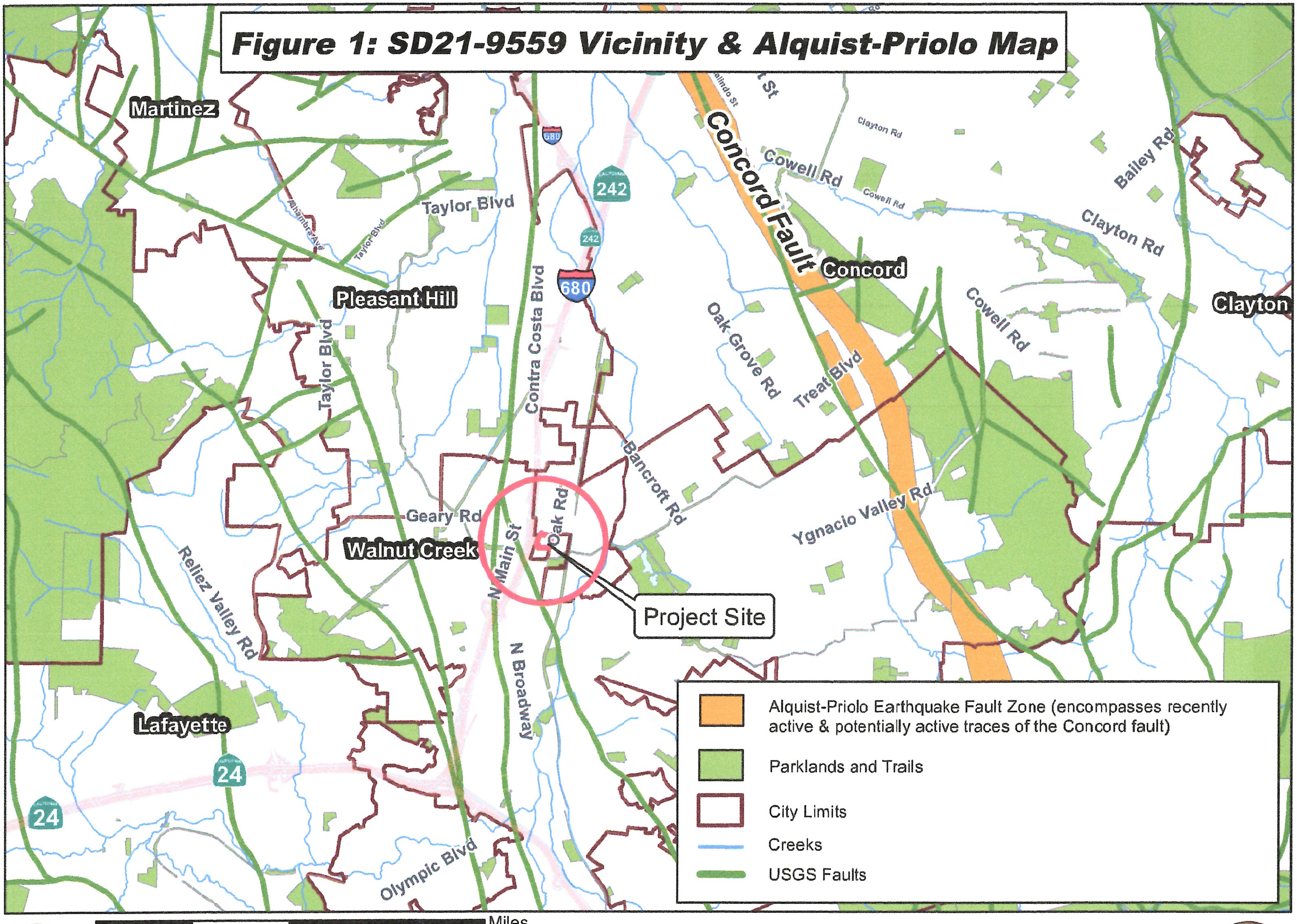
Sincerely,
DARWIN MYERS ASSOCIATES

Darwin Myers

REGISTERED GEOLOGIST
W. DARWIN
MYERS
No. 946
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

Darwin Myers, CEG 946
Principal

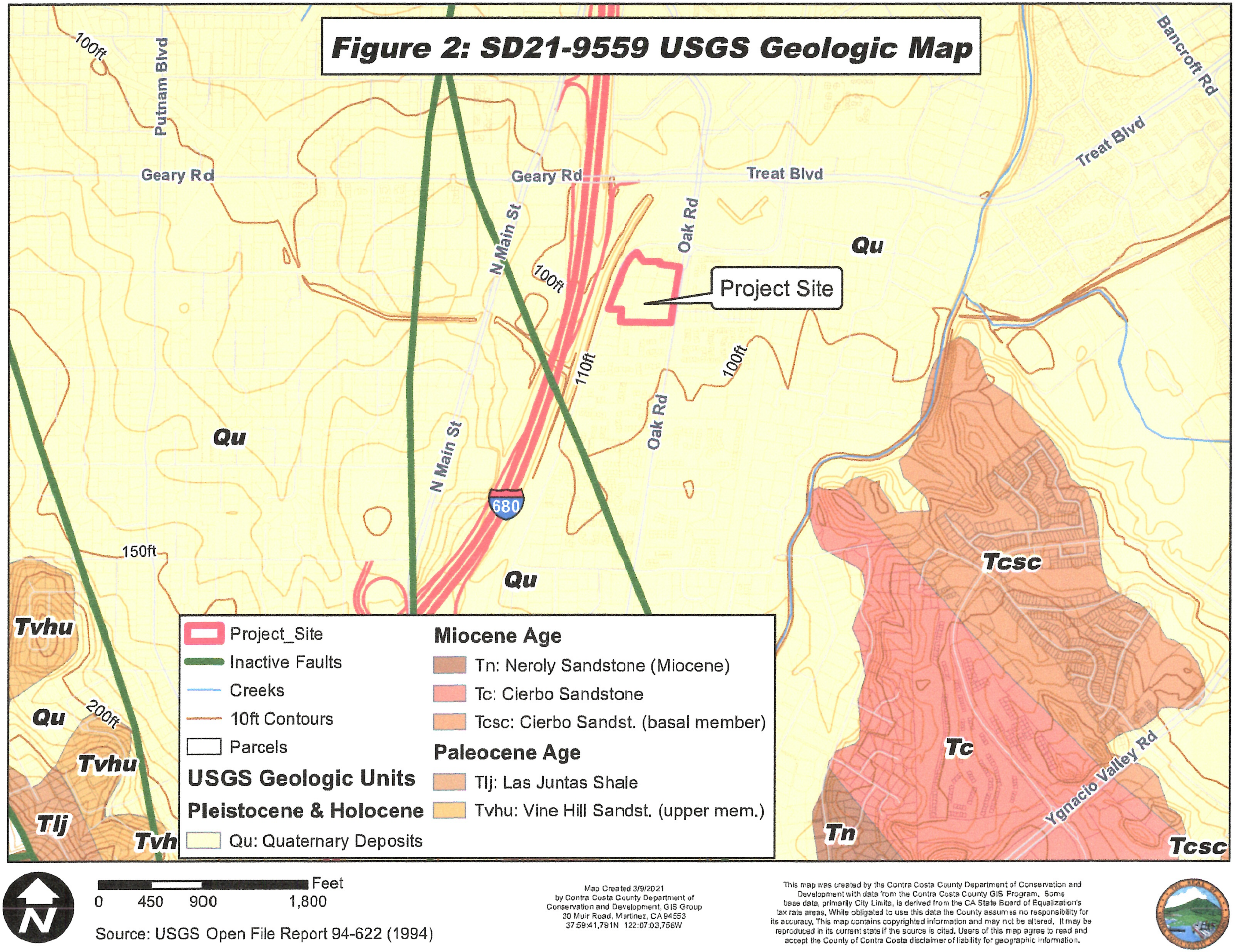
Figure 1: SD21-9559 Vicinity & Alquist-Priolo Map



source CGS, Special Publication 42 (revised in 2018) & USGS Open File Report 94-622 (1994)

Map Created 3/9/2021 by Contra Costa County Department of Conservation and Development, GIS Group 30 Muir Road, Martinez, CA 94553 37:59:41.791N 122:07:03.756W

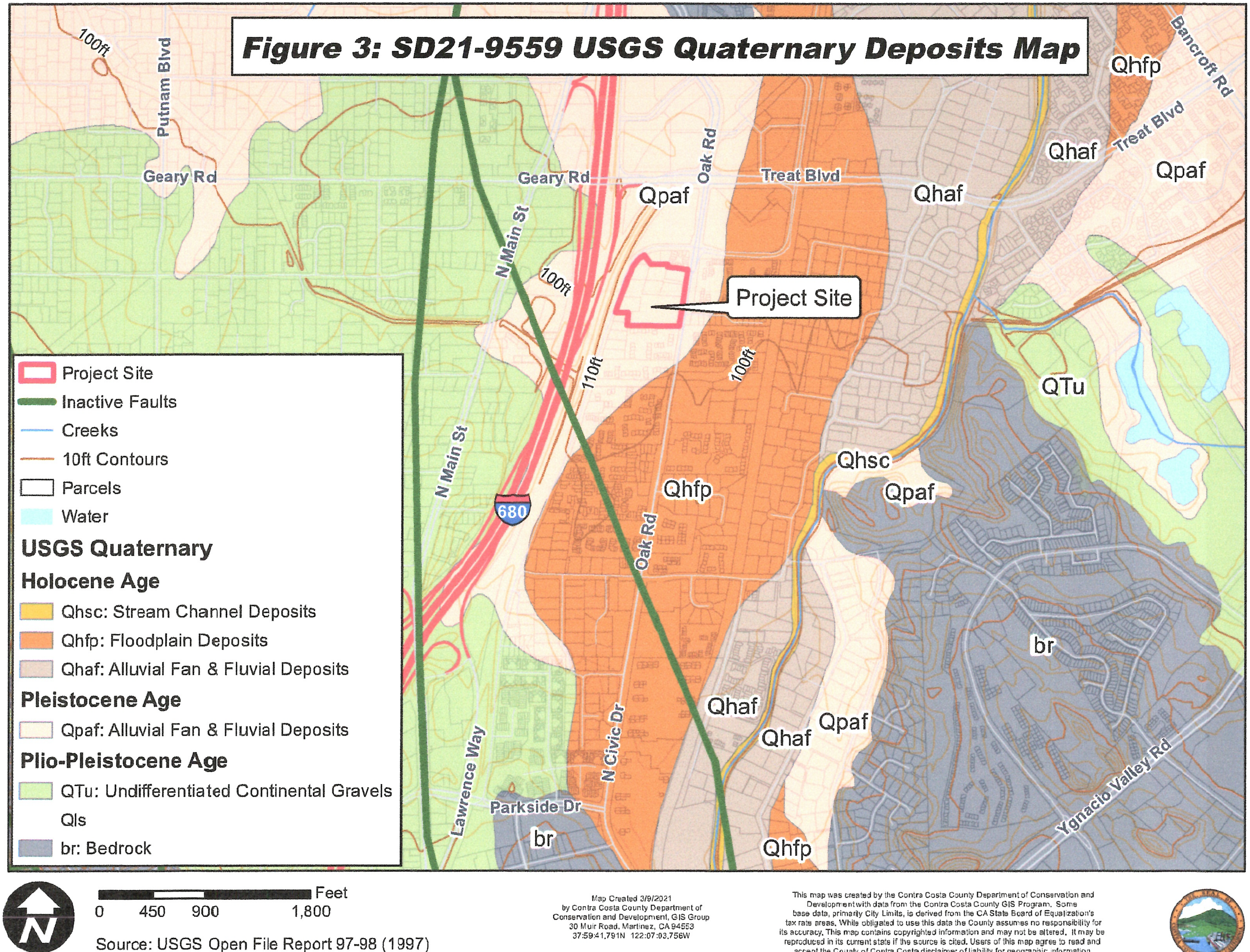
Figure 2: SD21-9559 USGS Geologic Map



Source: USGS Open File Report 94-622 (1994)

Map Created 3/9/2021
by Contra Costa County Department of
Conservation and Development, GIS Group
30 Muir Road, Martinez, CA 94553
37:59:41.791N 122:07:03.756W

Figure 3: SD21-9559 USGS Quaternary Deposits Map



Source: USGS Open File Report 97-98 (1997)

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Conservation and Development, GIS Group
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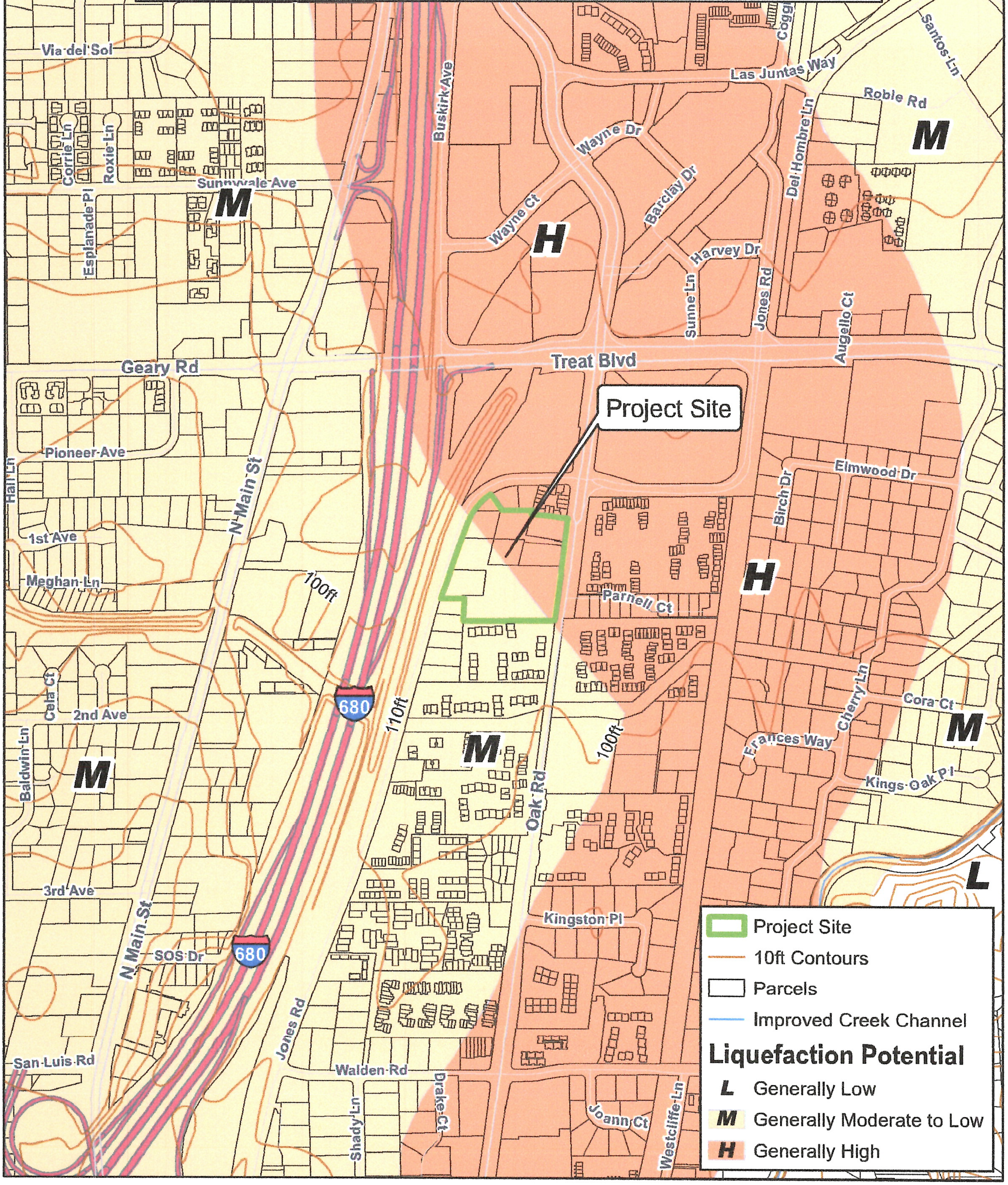
Figure 4: SD21-9559 Liquefaction Potential Map



Map Created 3/9/2021 by Contra Costa County Department of Conservation and Development, GIS Group 30 Muir Road, Martinez, CA 94553 37:59:41.791N 122:07:03.756W

This map was created by the Contra Costa County Department of Conservation and Development with data from the Contra Costa County GIS Program. Some base data, primarily City Limits, is derived from the CA State Board of Equalization's tax rate areas. While obligated to use this data the County assumes no responsibility for its accuracy. This map contains copyrighted information and may not be altered. It may be reproduced in its current state if the source is cited. Users of this map agree to read and accept the County of Contra Costa disclaimer of liability for geographic information.

Source: Saftey Element, page 10-15

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E.3 - Oak Road Paleontological Records Search

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Dana DePietro
FirstCarbon Solutions
1350 Treat Boulevard, Suite 380
Walnut Creek, CA 94597

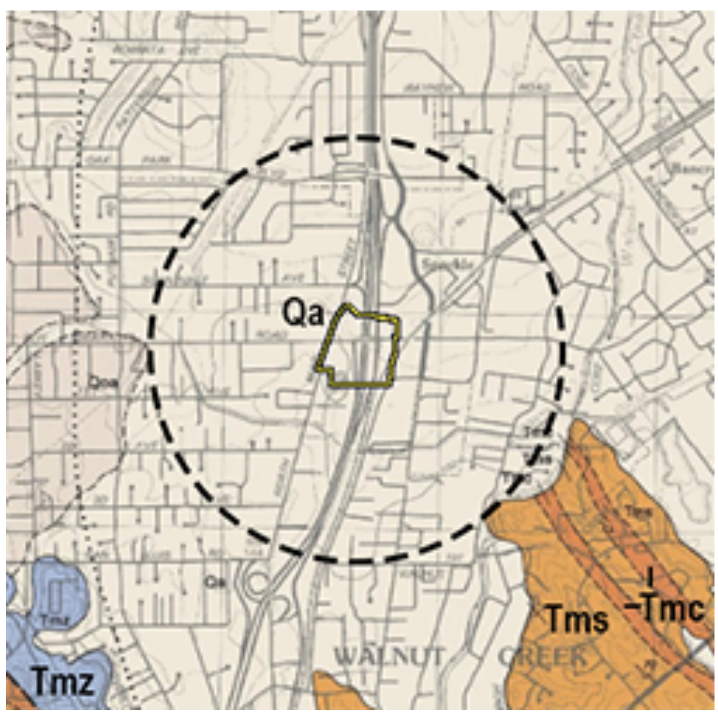
Re: Paleontological Records Search: Oak Road Project (2648.0017), Unincorporated Contra Costa County

Dear Dr. DePietro:

As per the request of Madelyn Dolan, I have performed a records search on the University of California Museum of Paleontology (UCMP) database for the proposed Oak Road project just west of Walnut Creek in unincorporated Contra Costa County. The 5.94-acre project site comprises eight parcels located at 2740 Jones Road, southeast of the intersection of Interstate 680 and Treat Boulevard. Its PRS location is SW, SW¼, SW¼, Sec. 14, T1N, R2W, Walnut Creek quadrangle (USGS 7.5'-series topographic map). Google Earth imagery shows this flat terrain has been disturbed by prior development. The client proposes constructing 19 three-story townhome condominium buildings on this site.

Geologic Mapping

As shown here on part of the geologic map of Dibblee and Minch (2005), both the surface of the project site (outline at center) and half-mile search area (larger dashed black outline) consist solely of Holocene alluvium (Qa). Just outside the search area are the late to middle Miocene Monterey Formation shale (Tmc) and sandstone (Tms), and the Paleocene Martinez Formation (Tmz). Holocene deposits are too young to be fossiliferous, while the two Tertiary units are of marine origin and potentially fossiliferous.



Paleontological Records Search

The paleontological records search of the UCMP database initially focused on the Miocene and Martinez formations in

Contra Costa County, even though either is unlikely to be present in the shallow subsurface of the project site. The results for the Monterey Formation are one vertebrate locality (V4616, Tormey B), which yielded a Barstovian-aged (upper Miocene) cetacean vertebra. In addition, there are three other Barstovian localities in unidentified geological units, and their yield consists of a pelvis fragment of *Desmostylus* (extinct marine hippo-like mammal), a *Carcarocles megalodon* (megalodon shark) tooth, and ribs of an unidentified marine mammal. Of these four Barstovian localities, the closest to the project site is V68104 (Bellamy) in Pleasant Hill, mapped at the north-northwest edge of the search area and which yielded the megalodon tooth, while the other three localities are about 15 miles to the northwest. The recorded location of V68104, however, is questionable because the area is mapped as Holocene, the geologic map shows the Monterey Formation nearly one mile to the southeast, and there is no indication that the tooth was found in the subsurface. For the Martinez Formation, there are three vertebrate localities, all approximately five miles east of Mount Diablo, and each yielded a single element of fish. No plant localities are recorded for either the Monterey or Martinez formations in the County.

Paleontological Assessment and Mitigation Recommendations

A preconstruction paleontological walkover survey of the proposed Oak Road project site is not recommended because its surface is flat and disturbed. I also do not recommend paleontological monitoring of project-related earth-disturbing construction activities because the surficial deposits are Holocene, and the nearest older deposits are mapped nearly a mile away. Hence, it is highly unlikely that any significant paleontological resources will be encountered during project-related construction activities. This report therefore concludes the paleontological mitigation for this project in accordance with CEQA guidelines.

Sincerely,

Ken Finger

Reference Cited

Dibblee, T.W., Jr., and Minch, J.A., 2005. Geologic map of the Walnut Creek quadrangle, Contra Costa County, California. Dibblee Foundation Map DF-149, scale 1:24,000.