407 SPRECKELS AVENUE
MANTECA, CALIFORNIA

GEOTECHNICAL EXPLORATION

Submitted to
Ms. Terri Allen
DCT Industrial
12 Corporate Plaza, Suite 150
Newport Beach, CA 92660

Prepared by
ENGEO Incorporated

January 24, 2017

Project No.
13618.000.000

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
13618.000.000

January 24, 2017

Ms. Terri Allen
DCT Industrial
12 Corporate Plaza, Suite 150
Newport Beach, CA 92660

Subject: 407 Spreckels Avenue
Manteca, California

GEOTECHNICAL EXPLORATION

Dear Ms. Allen:

ENGEO is pleased to present this geotechnical report for the Spreckels Avenue project as outlined in our agreement dated December 9, 2016. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

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

Sincerely,

ENGEO Incorporated

Christopher Stouffer, EIT
cs/sh/jf

Steve Harris, GE

REGISTERED PROFESSIONAL ENGINEER
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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical exploration report is to provide geotechnical recommendations for the design of the proposed logistics facility and associated improvements in Manteca, California.

The scope of our services included:

- Reviewing available literature, geologic maps, and previous available reports pertinent to the site.
- Advancing 2 cone penetrometer tests (CPTs), drilling 6 brings and performing 27 test pits.
- Perform 2 standpipe percolation tests and one double ring infiltrometer percolation test.
- Preforming laboratory analysis.
- Analyzing the geotechnical data.
- Reporting our findings and recommendations.

This report was prepared for the exclusive use of DCT Industrial and its design team consultants. In the event that any changes are made in the character, design or layout of the development, the conclusions and recommendations contained in this report should be reviewed by ENGEO to determine whether modifications to the report are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

1.2 PROJECT LOCATION

The subject site is located on the west side of Spreckels Avenue, south of the intersection of Phoenix Drive and Spreckels Avenue in Manteca, California as shown on Figure 1. The site is accessible from Spreckels Avenue.

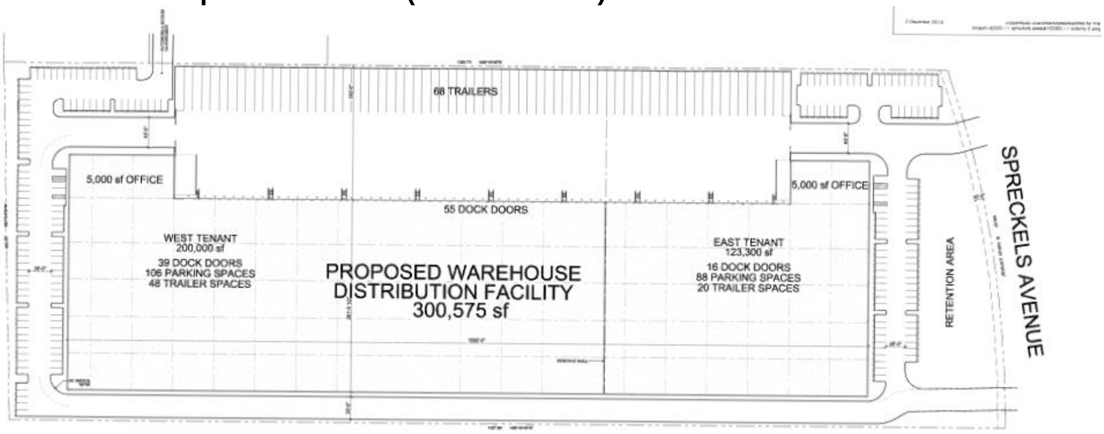
The site is bounded by commercial buildings to the north, industrial buildings to the south, residential structures to the west, and the Manteca Tidewater Bikeway to the east, parallel to Spreckels Avenue, as shown on Figure 2. The approximately 14-acre property currently consists of undeveloped land.

1.3 PROJECT DESCRIPTION

Based on our review of the information provided and discussions with you, we understand the proposed project will include:

- An approximately 300,000-square-foot distribution facility.
- One retention basin.
- Asphalt and Portland Cement Concrete paved parking and driveways.
- Underground utilities.

EXHIBIT 1: Proposed Site Plan (DCT Industrial)



Structural loads are yet to be determined; however, we assume that structural loads and maximum allowable differential settlements will be representative of this type of construction.

2.0 FINDINGS

2.1 SITE BACKGROUND

As shown below, historical images from Google Earth indicate the site was previously occupied by a sugar refinery and associated structures. The northern and southern portions of the site were previously developed with industrial buildings. The remainder of the site appears to contain smaller buildings, stockpiles, a tank, or is undeveloped. Imagery indicates that these structures were removed around 2003 and has since remained undeveloped. The majority of the site appears to have undergone various levels of construction and grading. Below is an aerial image of the site conditions in 1993. The project boundaries are outlined in teal.

EXHIBIT 2: Site conditions in 1993 (Google Earth)

2.2 FIELD EXPLORATION

Our field exploration included drilling 6 borings, advancing 2 CPT soundings and performing 27 exploratory test pits. We performed our field exploration between January 5 and January 6, 2017. The approximate locations of the explorations are shown on the site plan, Figure 2.

2.2.1 Borings

We retained a truck-mounted Soil Test Ranger drill rig and crew to advance the borings using 4-inch-diameter solid flight augers. The borings were advanced to depths ranging from approximately 16½ to 27 feet below existing grade. An ENGEO representative logged the borings in the field and collected soil samples using either a 3 inch O.D. Modified California-type split-spoon sampler fitted with 6-inch-long stainless steel liners or a 2-inch O.D. Standard Penetration Test (SPT) split-spoon sampler. The samplers were advanced with a 140-pound hammer with a 30-inch drop, employing a rope-and-cathead hammer system. The penetration of the samplers into the native materials was field recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. Blow count results on the boring logs were recorded as the number of blows required for the last 1 foot of penetration.

We used field logs to develop the boring logs included in Appendix A. The boring logs depict subsurface conditions within the borings at the time the exploration was conducted. Subsurface conditions at other locations may differ from conditions occurring at these boring locations and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types; the transitions may be gradual or gradational.

2.2.2 Cone Penetration Tests

We retained a CPT rig to perform 2 cone penetration tests advanced approximately 50 feet below existing grade. The soundings were performed with a 10-square-centimeter end area 10-ton subtraction digital cone with a pore pressure and seismic transducer. The area of the friction sleeve is 150 square centimeters and the average unequal end area ratio is 0.8. 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 D-3441. Measurements include the tip resistance to penetration of the cone (Q_c), the resistance of the surface sleeve (F_s), and pore pressure (U) (Robertson and Campanella, 1988). CPT logs are presented in Appendix A.

2.2.3 Test Pits

We retained a backhoe to perform 27 exploratory test pits throughout the site. The pits were approximately 3 feet wide and up to 10 feet long. They were excavated to depths ranging from approximately 2½ feet to 7½ feet below the existing ground surface. Logs of the test pits are attached in Appendix A.

Test pit excavations were loosely backfilled with the excavated material. During site grading, the loosely backfilled soils within our exploratory test pits should be removed and re-compacted in accordance with Section 5.0. The depth of removal of these materials should be determined by ENGEO in the field at the time of grading. The test pits were geocoded at the time of excavation in order to be located at time of construction.

2.3 REGIONAL AND SITE GEOLOGY

We present the following discussion of site geology based on our field reconnaissance and review of the CGS *Geologic Map of the San Francisco-San Jose Quadrangle* (Wagner, Bortugno, and McJunkin 1991).

The site is located in the Great Valley geomorphic province. The Great Valley is an elongate, northwest-trending structural trough bound by the Coast Range on the west and the Sierra Nevada on the east. The Great Valley has been and is presently being filled with sediments primarily derived from the Sierra Nevada.

Our site reconnaissance and previously referenced geologic map indicate that the underlying geologic formation at the site is Dune Sand (Qs) consisting of interbedded silt, sand, and gravel. The regional geologic map is included on Figure 3.

2.4 SITE SEISMICITY

The site is located in an area of moderate to high seismicity. No known active[1] faults cross the property and the site is not located within an Earthquake Fault Special Study Zone; however, large (greater than Moment Magnitude 7) earthquakes have historically occurred in the region and many earthquakes of low magnitude occur every year. Figure 4, Regional Faulting and Seismicity, shows the approximate locations of nearby faults and significant earthquakes recorded within the region. The two nearest earthquake faults zoned as active by the State of California Geological Survey are the Great Valley 7 fault located approximately 15 miles to the southwest and the Greenville fault, located about 26 miles to the southwest.

The Great Valley fault is a blind thrust fault with no known surface expression; the postulated fault location has been based on historical regional seismic activity and isolated subsurface information. Portions of the Great Valley fault are considered seismically active thrust faults; however, since the Great Valley fault segments are not known to extend to the ground surface, the State of California has not defined Earthquake Fault Hazard Zones around the postulated traces. The Great Valley fault is considered capable of causing significant ground shaking at the site, but the recurrence interval is believed longer than for more distant, strike-slip faults. Recent studies by Eaton 1986, Moores 1991 and Wong 1989 suggest that this boundary fault may have been the cause of the Vacaville-Winters earthquake sequence of April 1892.

Further seismic activity can be expected to continue along the western margin of the Central Valley.

[1] 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). The State of California has prepared maps designating zones for special studies that contain these active earthquake faults.

Other active faults capable of producing significant ground shaking at the site include the Ortigalita fault, 36 miles south; Calaveras fault, 40 miles southwest; the Hayward fault, 43 miles southwest; the Green Valley Connected fault, 44 miles northwest; and the San Andreas fault, 61 miles southwest of the site. Any one of these faults could generate an earthquake capable of causing strong ground shaking at the subject site. Earthquakes of Moment Magnitude 7 and larger have historically occurred in the nearby Bay Area and numerous small magnitude earthquakes occur every year.

2.5 SURFACE CONDITIONS

According to Google Earth, site grades range from elevation 40 feet to 44 feet (Datum WGS84). During our field reconnaissance, we observed the following site conditions:

- Excessive growth of grasses and weeds across the site.
- Varying amounts of concrete, brick and debris across surface of the site with the largest concentration along the northern portion.
- Large debris such as mattresses and bikes along the north and east perimeter.
- A metallic pipe with a diameter of approximately 4 inches extending approximately 4 feet above the ground surface on the eastern portion of the site.

Please refer to the Site Plan, Figure 2, for more information on site features.

2.6 SUBSURFACE CONDITIONS

We encountered varying amounts of undocumented fill within the majority of our test pits and boring explorations. The fill contained concrete debris, bricks, asphalt, and non-native rock, all of varying diameters. Test pits on the southwestern portion of the site uncovered undocumented fill identified as a black, low plastic sandy lean clay at a depth of 3 to 6½ feet below the surface. The depths to native material varied from approximately the surface to 6¾ feet below existing grade. The native soils encountered in our explorations generally consisted of loose to medium dense silty sand and clayey sand to a depth ranging between 2½ to 5 feet. Across the site, a relatively continuous layer of medium dense silty sand extended to a depth ranging from 8 to 10½ feet. Beneath the silty sand stratum was a continuous layer of medium dense poorly graded sand to a depth ranging from 16 to 20 feet. The sand layer was underlain by a lean clay and sandy lean clay to the total depth of the explorations. Data from the CPT explorations found the clay layer to be underlain by a sand and gravelly sand to the total depth of the explorations.

Refer to Figure 2 and exploration logs included in Appendix A for specific subsurface conditions at each location. The logs graphically depict the subsurface conditions encountered at the time of the exploration. The boring logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System (USCS).

2.7 GROUNDWATER CONDITIONS

We did not observe static groundwater in any of the borings or test pits to the maximum depth explored of 27 feet. 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.

2.8 LABORATORY TESTING

We performed laboratory tests on selected soil samples to determine their engineering properties. For this project, we performed moisture content, plasticity index, #200 wash, and resistance value. Corrosion testing was performed by Sunland Analytical. Selected soil properties are recorded on the boring logs in Appendix A. All laboratory data is included in Appendix B.

2.9 PERCOLATION TESTING

2.9.1 Standpipe Percolation Test

We installed two percolation test holes to a depth of 5 feet in the approximate location of the proposed retention basin. Percolation Holes P-1 and P-3 were installed as standpipe percolation tests. We drilled the percolation test holes using a 4-inch-diameter solid flight auger. Preparation of the percolation test holes began by placing a 2-inch-thick layer of open-graded gravel in the bottom of the holes, then placing a 3-inch-diameter plastic pipe in the test holes and ¾-inch-diameter drain rock surrounding the pipe up to the ground surface. We presoaked the holes with municipal drinking water the day prior to performing the percolation test. It is our opinion that the percolation rate of drinking water should be similar to storm water.

ENGEO performed the percolation testing on January 6, 2017. At the start of the test, we filled the holes with water to approximately 12 inches above the gravel placed at the bottom of the holes. The water was then measured until the percolation rate stabilized. At the end of each interval, additional water was added, as needed, to reset the water level to approximately 12 inches above the gravel.

2.9.2 Double Ring Infiltrometer Testing

One 5-foot-deep trench was excavated in the center of the proposed basin. During excavation, the trench's subsurface material was identified and logged as TP-27. Percolation Hole P-2 was installed as a double-ring percolation test in accordance with ASTM D3385. A double-ring infiltrometer consisting of two 20-inch-high open cylinders with diameters of 12 inches and 24 inches were concentrically driven 4 and 6 inches into the ground, respectively. A competent seal between the soil and the cylinders was ensured. Each ring was then filled with water to no more than a depth of 6 inches and the differential depth between the inner and outer cylinders no more than ¼ inch. Two Mariotte tubes of approximately 3,000 ml for the inner cylinder and 10,000 ml for the outer cylinder were used to maintain a constant and even water level in the two tubes.

ENGEO performed the percolation testing for the double-ring test method on January 6, 2017. The apparatus was installed and a field representative monitored the water levels in both the cylinders and the Mariotte tubes. The test was performed and monitored until the infiltration rate in both the inner and outer cylinders remained constant.

2.9.3 Percolation Testing Results

After performing the standpipe and double-ring percolation tests, the most conservative rate recorded was 60 gallons per square foot per day. This percolation rate is only applicable to the proposed design basin location and depth of 5 feet. If the basin is to be moved or dimensions altered, further testing is suggested. Additional factors of safety should be applied as seen fit by the design civil engineer.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications. The primary geotechnical issues that could affect development is undocumented fill. We summarize this and our other conclusions below.

3.1 UNDOCUMENTED FILL

Our borings and test pits indicate that the majority of the site is underlain by undocumented fill. The site had an average fill depth of 3 to 5 feet with the deepest fill depth of 6¾ feet. Although explorations were extensive, the depth of fill is variable and may fluctuate outside these averages and limits.

Non-engineered fills can undergo excessive settlement, especially under new fill or building loads. Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. We present fill removal recommendations in Section 5.0.

3.2 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 a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, soil liquefaction, dynamic densification, lateral spreading, landslides, tsunamis, flooding or seiches is considered low to negligible at the site.

3.2.1 Ground Rupture

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

3.2.2 Ground Shaking

A potential seismic hazard at the site is strong ground shaking from a nearby moderate to major seismic event. The degree of shaking experienced at a site is dependent on the magnitude of the event, the distance to its epicenter, and the nature of the underlying soils. Based on the probabilistic seismic data provided by the Unites States Geological Survey (USGS), we recently utilized the online 2008 Interactive Deaggregations tool to determine that a horizontal ground surface acceleration of 0.44g is predicted to have a 2 percent probability of being exceeded in a 50-year design life at the site.

To mitigate the ground shaking effects, all structures are to be designed using sound engineering judgment and the latest California Building Code (CBC) requirements as a minimum. The 2016 CBC Seismic Design Parameters are provided below in a subsequent section of this report.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures are to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.2.3 Liquefaction and Cyclic Softening

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands. Empirical evidence indicates that loose to medium-dense gravels, silty sands, and low- to moderate-plasticity silts and clays may be susceptible to liquefaction. In addition, sensitive high-plasticity soils may be susceptible to significant strength loss (cyclic softening) as a result of significant cyclic loading. The silts and clays encountered are not sensitive and, therefore, not subject to cyclic softening. We summarize the results of our liquefaction analysis below.

According to Bray and Sancio 2006, fine-grained soils with PI less than or equal to 12 and moisture content and liquid limit ratio of greater than 0.85 can undergo cyclic mobility. Based on our laboratory results, we found site soils to have a plasticity index of 14, and less than a ratio of 0.85.

We evaluated the liquefaction potential of the site soil with CPT data using methods published by Robertson (2009). The Cyclic Stress Ratio (CSR) was estimated for a Peak Ground Acceleration (PGA_M) value of 0.44g, based on probabilistic seismic data provided by USGS as discussed in Section 3.2.2. We also used a moment magnitude (M_w) of 6.7 in our analysis, which corresponds to the maximum magnitude for the Great Valley 7 fault based on the United States USGS national seismic hazard maps. We considered a design groundwater elevation of approximately 28 feet in our analysis.

The results of our liquefaction analyses indicate relatively thin and discontinuous sand layers approximately 2 feet in thickness below a depth of 34 feet as potentially liquefiable. Consequences of liquefaction could include surface disruption, settlement, and downdrag on deep foundations. Based on the results of our analysis and the relative thickness of non-liquefiable surface soils and potentially liquefiable soil, the risk of surface disruption is low to moderate. We estimate approximately ¾ inch of total liquefaction-induced settlement in a design-level seismic event based on the results of our CPT liquefaction analysis. Appendix C includes the results of our CPT-based liquefaction analysis.

3.2.4 Densification Due to Earthquake Shaking

Densification of loose granular soils above and below the groundwater level can cause settlement due to earthquake-induced vibrations. Due to the density of the granular materials sampled in the boring, the potential for densification of granular layers due to earthquake shaking is considered low at the site.

3.2.5 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Since the potential for liquefaction is considered low and the site is relatively flat, it is our opinion that the potential for lateral spreading is low.

3.2.6 Flooding

Based on site elevation and distance from water sources, flooding is not expected at the subject site; however, the Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

3.3 2016 CBC SEISMIC DESIGN PARAMETERS

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

TABLE 3.3-1: 2016 CBC Seismic Design Parameters (Latitude: 37.79153° Longitude: -121.19926°)

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	0.97
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.35
Site Coefficient, F_A	1.11
Site Coefficient, F_V	1.71
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.08
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.59
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	0.72
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.40
Mapped MCE Geometric Mean Peak Ground Acceleration (g)	0.37
Site Coefficient, F_{PGA}	1.15
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	0.41
Long period transition-period, T_L	8 sec

3.4 SOIL CORROSION POTENTIAL

As part of this study, we obtained two representative soil samples to determine their pH, resistivity, sulfate, and chloride. Two near-surface samples were combined for the testing. The results are presented in the table below and provided in Appendix B.

TABLE 3.4-1: Corrosivity Test Results

SAMPLE LOCATION	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
1-B3 @3.5’ and 1-B5 @ 2.5’	8.04	1,450	55.0	63.2

The 2016 CBC references the 2011 American Concrete Institute Manual, ACI 318-11, Chapter 4, Sections 4.2.1 for structural concrete requirements. ACI Table 4.2.1 provides the following exposure categories and classes, and concrete requirements in contact with soil based upon the exposure risk.

TABLE 3.4-2: ACI Table 4.2.1: Exposure Categories and Classes

CATEGORY	SEVERITY	CLASS	CONDITION	
F Freezing and thawing	Not Applicable	F0	Concrete not exposed to freezing-and-thawing cycles	
	Moderate	F1	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture	
	Severe	F2	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture	
	Very Severe	F3	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals	
S Sulfate	Not applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 \leq 1{,}500$ seawater
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1{,}500 \leq SO_4 \leq 10{,}000$
	Very severe	S3	$SO_4 > 2.00$	$SO_4 > 10{,}000$
P Requiring low permeability	Not applicable	P0	In contact with water where low permeability is not required.	
	Required	P1	In contact with water where low permeability is required.	
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture	
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides	
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources	

*Percent sulfate by mass in soil determined by ASTM C1580
**Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

In accordance with the criteria presented in the above table, these soils are categorized as Not Applicable, and are within the F0 freeze-thaw class, S0 sulfate exposure class, P0 exposure class and C0 corrosion class. Cement type, water-cement ratio, and concrete strength, are not specified for these ranges.

Considering a 'Not Applicable' sulfate exposure, there is no requirement for cement type or water-cement ratio; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code.

If desired to investigate this further, we recommend a corrosion consultant be retained to determine if specific corrosion recommendations are necessary for the project.

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to determine whether our recommendations have been implemented, and to provide additional or modified recommendations, if necessary. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. All earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

5.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as determined by an ENGEO representative.

As used in this report, the term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define "structural areas" in a subsequent section of this report as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

5.1 GENERAL SITE CLEARING

Areas to be developed should be cleared of all surface and subsurface deleterious materials, including existing building foundations, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in the subsequent Earthwork Recommendations sections of this report. ENGEO should be retained to observe and test all backfilling.

Following clearing, mow and remove as much of the near surface vegetation that is feasible.

5.2 UNDOCUMENTED FILL REMOVAL

As previously discussed, a majority of the site is underlain by undocumented fill. All undocumented fill will need to be removed to expose competent native soil. Figure 2 shows the approximate location and depth of nonengineered fill that was encountered in our test pits. The actual lateral extent and depth of fill is expected to vary. ENGEO will need to be present during the subexcavation of the non-engineered fill to confirm that it is all removed. The non-engineered fill may be placed back as an engineered fill provided it meets the recommendations in Section 5.4 below.

5.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

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

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

Options 3 and 4 should be evaluated and approved by ENGEO prior to implementation.

5.4 ACCEPTABLE FILL

Onsite soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension.

Portions of the site identified potentially expansive near-surface soils. During excavation, if an expansive clay material is encountered, the soil should be removed or mixed with other non-expansive soil onsite. Soil with a plasticity index greater than 12 inches should not be placed within the upper 24 inches of the building pad.

Imported fill materials should meet the above requirements and have a plasticity index less than 12. Allow ENGEO to sample and test proposed imported fill materials at least one week prior to delivery to the site.

5.5 FILL COMPACTION

5.5.1 Grading in Structural Areas

Once all non-engineered fill is removed, compact the exposed subgrade and surface of areas without non-engineered fill as follows.

1. Scarify to a depth of at least 12 inches.

2. Moisture condition lifts to at least 1 percentage point above the optimum moisture content for soil with a plasticity index less than 12 and at least 3 percentage points above the optimum moisture content for soil with a plasticity index greater than 12.

3. Compact the subgrade to at least 90 percent relative compaction. Compact the upper 6 inches of finish pavement subgrade to at least 95 percent relative compaction prior to aggregate base placement.

After the subgrade soil has been compacted, place and compact acceptable fill as follows:

1. Spread fill in loose lifts that do not exceed 8 inches.

2. Moisture condition lifts to at least 1 percentage point above the optimum moisture content for soil with a plasticity index less than 12 and at least 3 percentage points above the optimum moisture content for soil with a plasticity index greater than 12.

3. Compact fill to a minimum of 90 percent relative compaction; Compact the upper 12 inches of fill in pavement areas and building pads to 95 percent relative compaction prior to aggregate base placement.

Additional testing may need to be performed once non-engineered fill has been removed to identify proper moisture and compaction specifications.

Compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

5.5.2 Underground Utility Backfill

Recommendations for fill compaction of underground utility backfill within structural areas are provided in this section. Jetting of backfill is not an acceptable means of compaction.

The contractor is responsible for conducting all trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe bedding materials.

Place and compact trench backfill as follows:

1. Trench backfill should have a maximum particle size of 6 inches.

2. Moisture condition lifts to at least 1 percentage point above the optimum moisture content for soil with a plasticity index less than 12 and at least 3 percentage points above the optimum moisture content for soil with a plasticity index greater than 12. Moisture condition backfill outside the trench.

3. Place fill in loose lifts not exceeding 12 inches.

4. Compact fill to a minimum of 90 percent relative compaction (ASTM D1557).

5.6 SLOPE GRADIENTS

Construct final slope gradients less than 10 feet high to 2:1 (horizontal:vertical) or flatter. Slopes taller than 10 feet high should be constructed as a 3:1. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

5.7 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. 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 systems to reduce overland surface drainage across the site.

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

6.0 FOUNDATION AND SLAB-ON-GRADE RECOMMENDATIONS

We developed structural improvement recommendations using our field exploration and laboratory test results and engineering analysis. The proposed building can be supported on continuous or isolated spread footings bearing in competent native soil or compacted fill, in conjunction with slab-on-grade floors.

6.1 FOOTING DIMENSIONS AND ALLOWABLE BEARING CAPACITY

Provide minimum footing dimensions as follows in the Table 6.1-1 below.

TABLE 6.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (INCHES)	MINIMUM WIDTH (INCHES)
Continuous	24 (Perimeter Footings) 18 (Interior Footings)	12
Isolated	24 (Perimeter Footings) 18 (Interior Footings)	24

*below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade. The cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent exterior grade. Design foundations recommended above for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

6.2 FOUNDATION LATERAL RESISTANCE

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of footings bearing in competent native soil or compacted fill. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design:

- Passive Lateral Pressure: 300 pcf
- Coefficient of Friction: 0.30

Increase the above values by one-third for the short-term effects of wind or seismic loading. Passive lateral pressure should not be used for footings on or above slopes.

6.3 SETTLEMENT

While we were not provided any structural loads for evaluating potential foundation settlements, we anticipate that total and differential foundation settlements will be less than approximately ½ and ¼ inch, respectively, over 50 feet, provided the above report recommendations are followed. Once the foundation layout and structural loads are known, we should be retained to review the information and update or revise the above total and differential settlement estimates.

As noted in Section 3.2.3, total earthquake-induced settlements of up to ¾ inch can be expected under the maximum considered earthquake (MCE) as a result of liquefaction. However, due to the relatively thick cap of non-liquefiable soils at the surface of the site, we anticipate differential settlements to be negligible under the MCE. The foundation should be designed to accommodate the cumulative static and seismically induced settlement without collapse of the structure.

6.4 INTERIOR CONCRETE SLAB-ON-GRADE FLOORS

We anticipate that the operation of the distribution facility will include forklift and rack loads on the interior concrete floor slab. When the types and sizes of forklifts and rack loads are known, we recommend that we be retained to review and update these recommendations, as needed.

Interior concrete floors that will support forklift or rack loads should be underlain by 6 inches of granular base having an R-value of at least 50, a plasticity index less than 12, and no more than 10 percent passing the No. 200 sieve. The base should be compacted to at least 95 percent relative compaction (ASTM D1557) to provide firm, uniform support for the slab-on-grade. Prior to construction of the slab, the surface should be proof-rolled with heavy equipment to check that the base material is uniformly compacted and does not deflect under equipment loads. Prior to placing the base material, the building subgrade should be prepared in accordance with the Earthwork Recommendations.

When buildings are constructed with concrete slab-on-grade, 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, such as in any designated office areas where floor coverings may be applied, for example, we recommend installation of a durable vapor retarder beneath the concrete floor. The vapor retarder should be sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders should conform to Class A vapor retarders in accordance with ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs".

6.5 TRENCH BACKFILL

Backfill and compact all trenches below building slabs-on-grade and to 5 feet laterally beyond any edge in accordance with the Underground Utility Backfill recommendations in a previous section of this report.

7.0 RETAINING WALLS

7.1 LATERAL SOIL PRESSURES

Unrestrained drained walls, such as site retaining walls, up to 10 feet in height should be designed for active lateral earth pressures. For drained and restrained retaining walls, such as loading dock walls, at-rest lateral earth pressures should be considered. Table 7.1-1 provides lateral earth pressures for retaining wall design with level backfill conditions.

TABLE 7.1-1: Lateral Earth Pressures for Drained Retaining Walls with Level Backfill

ACTIVE PRESSURE (PCF)	AT-REST PRESSURE (PCF)
40	60

In accordance with 2016 California Building Code requirements, foundation walls and retaining walls supporting more than 6 feet of backfill height are to be designed for dynamic seismic lateral earth pressures corresponding to design earthquake ground motions. We recommend a dynamic seismic lateral earth pressure corresponding to 20H, where H is the height of the retaining wall and the seismic earth pressure has a triangular distribution. When considering seismic earth pressures for retaining walls, the recommended seismic earth pressure increment should be added to the active earth pressures provided above.

Appropriate surcharge loads from buildings, hardscape, and vehicles should be incorporated when the surcharge loading is situated above a 1:1 (horizontal:vertical) line of projection extending up the rear base edge of the bottom of the footing. A uniform horizontal surcharge load of 50 percent of the vertical surcharge load should be assumed to act over the height of the wall.

If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Passive pressures acting on foundations and keyways may be assumed as 400 pounds per cubic foot (pcf) provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. The friction factor for sliding resistance may be assumed as 0.30. The upper 1 foot of soil should be excluded from passive pressure computations unless it is confined by pavement or a concrete slab.

7.2 RETAINING WALL DRAINAGE

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

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-1.025) placed directly behind the wall, or

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

For both types of rock drains:

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

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

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

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

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

7.3 BACKFILL

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

7.4 FOUNDATIONS

Retaining walls may be supported on continuous footings designed for an allowable bearing pressure of 2,500 psf embedded to a minimum depth of 24 inches. Subgrade treatment of retaining wall foundations that are not within the building pad footprints should follow the recommendations in Existing Fill Removal and Expansive Soil Mitigation sections of this report.

8.0 EXTERIOR FLATWORK RECOMMENDATIONS

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 4 inches of concrete over 4 inches of aggregate base. In addition:

1. Compact the aggregate base to at least 90 percent relative compaction (ASTM D1557).

2. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

9.0 PAVEMENT DESIGN

9.1 FLEXIBLE PAVEMENTS

We obtained one representative bulk sample of the native soil and performed one R-value tests to provide data for pavement design. The results of the tests are included in Appendix B and indicate an R-value of 34, based on the site variability we judge an R-value of 20 to be to be appropriate for design. Additional R-Value testing should be performed on the actual pavement subgrade material to verify the following recommendations are applicable.

Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Topic 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the Table 9.1-1 below.

TABLE 9.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	ASPHALT CONCRETE (INCHES)	CLASS 2 AB (INCHES)
5	3	8
6	3½	10
7	4	12
8	5	14
9	5½	16
10	6½	18
11	7	20
12	8	22
13	9	24

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

9.2 RIGID PAVEMENTS

We developed the rigid Portland Cement Concrete Pavement (PCCP) pavement section in accordance with ACI 330R-08 “Guide for the Design and Construction of Concrete Parking Lots”.

At the time we performed this analysis, no traffic data was available and no serviceability information was provided. Therefore, the design is based on ACI 330-08 Traffic Category D for the distribution of traffic with varying average daily truck traffic volumes (ADTT), a 550 psi modulus of rupture for the concrete, a serviceability index of 2.5, a reliability index of 95 percent, and a 20-year design life. These assumptions correspond to a rigid pavement section designed to have five percent of the slabs cracked at the end of the design life; if the design team and yourself would like these assumptions revised, we can provide supplemental pavement sections.

We provide jointed plane concrete pavement (JPCP) recommendations below for R-value 20 due to the variability of the site. We calculated the following pavement section in accordance with the Portland Cement Association assuming edge support is provided by a tied concrete shoulder or curb and gutter. We confirmed our pavement section design using the commercially available software program *StreetPave12*. Additional R-Value testing should be performed on the actual pavement subgrade material to verify the following recommendations are applicable.

TABLE 9.2-1: Recommended Concrete Pavement Sections

ADTT	MINIMUM JPCP (INCHES)	MAXIMUM JOINT SPACING (FEET)
300	7½	15
1,400	8½	15
2,300	8½	15

Note: Calculations are based on the presence of a concrete shoulder or curbs

9.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with the Fill Compaction section of this report. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

9.4 CUT-OFF CURBS

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

10.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Project Description section of this report for 407 Spreckels Avenue. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

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

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

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

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

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

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

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FIGURES

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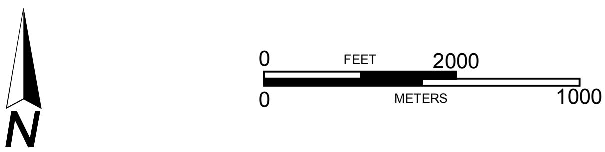
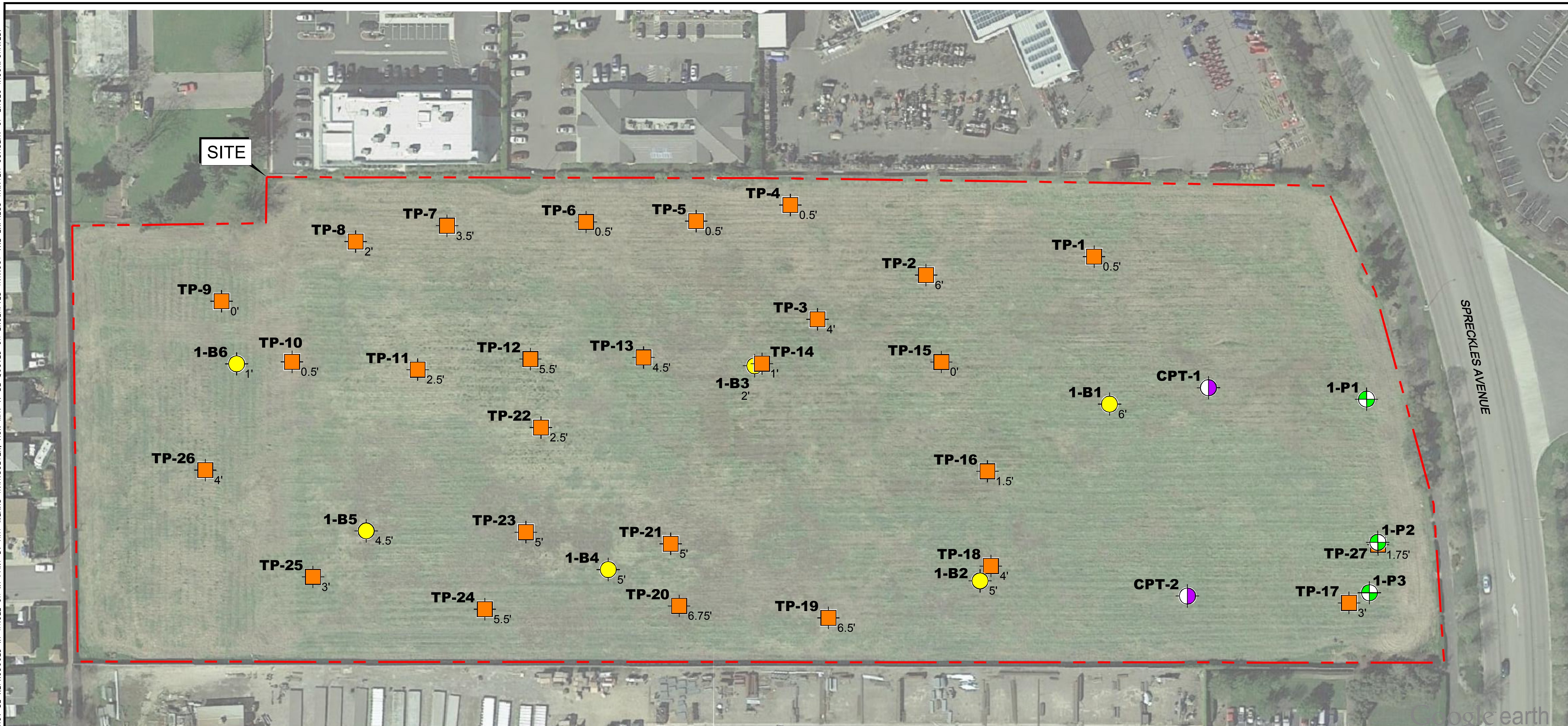


BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP 407 SPRECKELS AVENUE MANTECA, CALIFORNIA	PROJECT NO.: 13618.000.000 SCALE: AS SHOWN DRAWN BY: SRP / CHECKED BY: SDH	FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg_13000 Plus\13618\000\GEX\13618000000-1-VicinityMap-0117.dwg Plot Date: 1-17-17 spatters ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B6 1'	BORING SHOWING NON-ENGINEERED FILL DEPTH (ENGEO, JANUARY 2017)
TP-27 1.75'	TEST PIT SHOWING NON-ENGINEERED FILL DEPTH (ENGEO, JANUARY 2017)
CPT-2	CONE PENETRATION TEST (ENGEO, JANUARY 2017)
1-P3	PERCOLATION TEST (ENGEO, JANUARY 2017)

N

0 FEET 100

0 METERS 50

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	SITE PLAN 407 SPRECKELS AVENUE MANTECA, CALIFORNIA	PROJECT NO.: 13618.000.000 SCALE: AS SHOWN DRAWN BY: SRP CHECKED BY: SDH	FIGURE NO. 2

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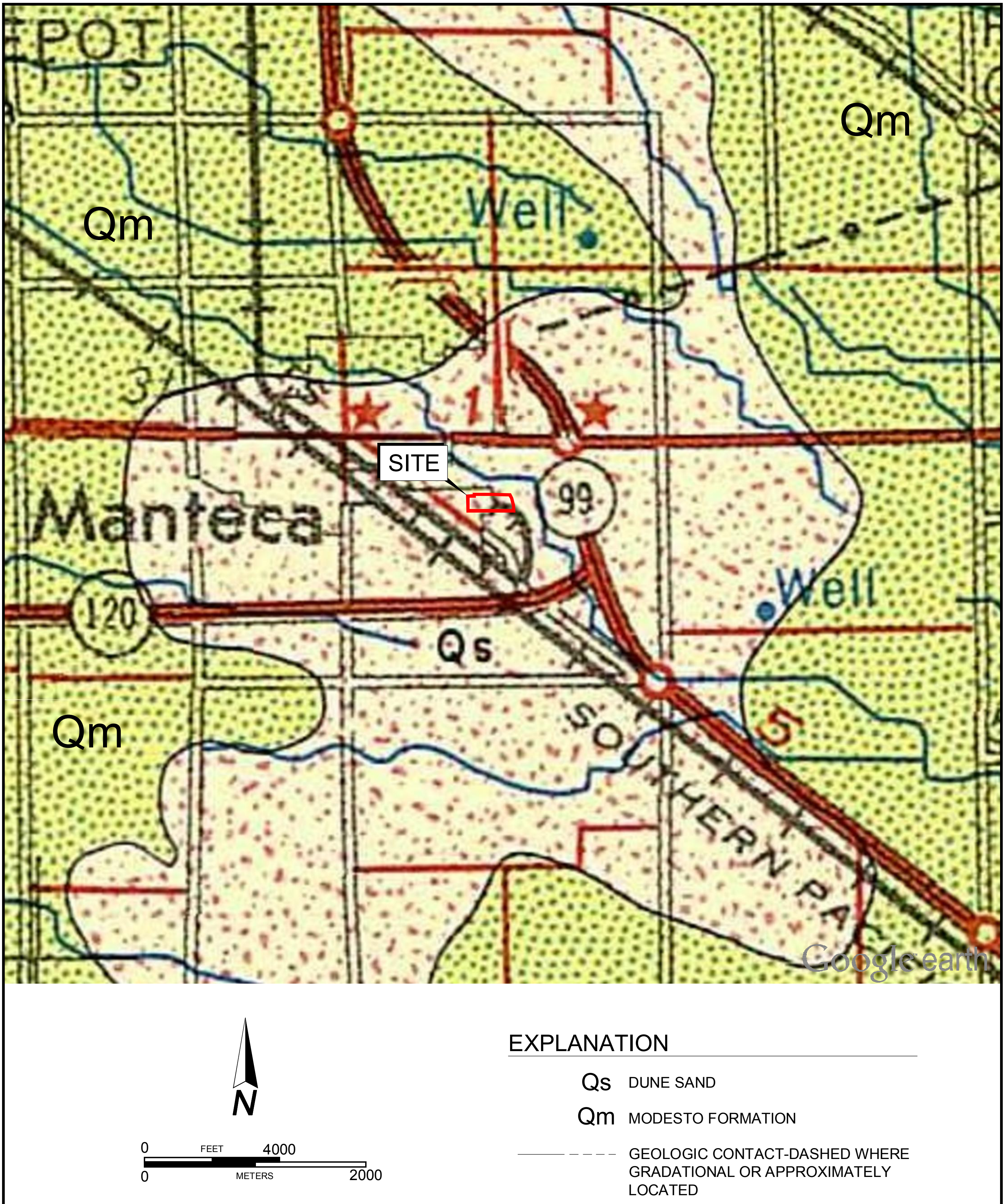

EXPLANATION

Qs DUNE SAND

Qm MODESTO FORMATION

——— – – – – GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

BASE MAP SOURCE: WAGNER, 1991

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
407 SPRECKELS AVENUE
MANTECA, CALIFORNIA

PROJECT NO.: 13618.000.000		FIGURE NO. 3
SCALE: AS SHOWN		
DRAWN BY: SRP	CHECKED BY: SDH	

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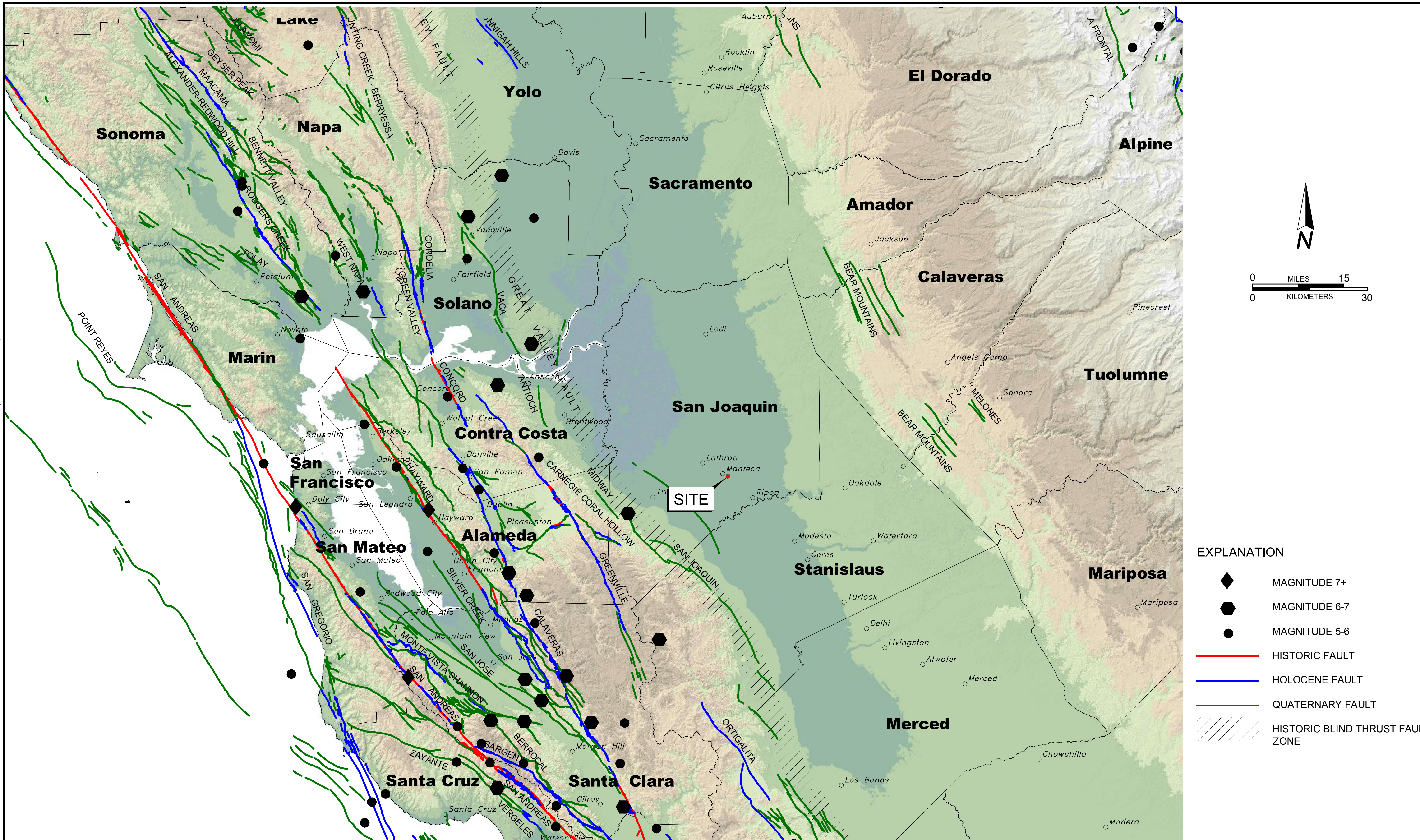
ORIGINAL FIGURE PRINTED IN COLOR



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

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REGIONAL FAULTING AND SEISMICITY
407 SPRECKELS AVENUE
MANTECA, CALIFORNIA

PROJECT NO.: 13618.000.000	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: SDH

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ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

KEY TO BORING LOGS
EXPLORATION LOGS
TEST PITS
CONE PENETRATION TESTS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

Dry	Dusty, dry to touch
Moist	Damp but no visible water
Wet	Visible freewater

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

Expect Excellence

LOG OF BORING 1-B1

ENGEO INCORPORATED

Geotechnical Exploration
407 Spreckels Ave
Manteca, CA
13618.000.000

DATE DRILLED: 1/5/2017
HOLE DEPTH: Approx. 18 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 43 ft.

LOGGED / REVIEWED BY: C. Stouffer / ZC
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SILTY SAND WITH GRAVEL (SM), very dark grayish brown, loose, moist, low plasticity, fine-grained sand, 35-45% fines, contains organics, miscellaneous rock fragments[undocumented fill]												
			SILTY SAND WITH GRAVEL (SM), dark brown, medium dense, moist, 15-25% gravel, fine-grained sand, 20-30 % fines[undocumented fill]			20					9				
	40		SILTY SAND WITH GRAVEL (SM), dark yellowish brown, loose to very loose, moist, 15-25% fine- to coarse-grained gravel, fine-grained sand, 25-35% fines [undocumented fill]			6									
5			contains miscellaneous non-native rock approximately 1.5" diameter			9					6				
10	35		SILTY SAND (SM), yellowish brown, medium dense, moist, non plastic, fine-grained sand, 10-15% silt fines [Native]			27					9				
15	30		POORLY GRADED SAND (SP), yellowish brown, medium dense to loose, moist, fine- to medium-grained sand, <5% fines (grades to fine- to coarse-grained sand)			20					21				
			SANDY LEAN CLAY (CL), light brownish gray, stiff, moist, medium plasticity, 10-20% fine-grained sand			16									
	25		Bottom of boring at approximately 18 feet. Groundwater not observed during drilling.												

LOG OF BORING 1-B2

ENGEO INCORPORATED

Geotechnical Exploration
407 Spreckels Ave
Manteca, CA
13618.000.000

DATE DRILLED: 1/5/2017
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 42 ft.

LOGGED / REVIEWED BY: C. Stouffer / ZC
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND (SC), very dark grayish brown, loose, moist, low to medium plasticity, fine-grained sand, 35-45% fines, organics, contains miscellaneous non-native rock and concrete up to 8" diameter [undocumented fill]												
	40		SILTY SAND (SM), dark brown, loose, moist, low plasticity, fine-grained sand, 30-40% fines, contains rock fragments approximately 1/2" diameter [undocumented fill]			14									
5			SILTY SAND (SM), dark yellowish brown, loose, moist, fine-grained sand, 20-30% fines [native]			11									
	35		(grades to fine- to medium-grained sand)												
			POORLY GRADED SAND WITH SILT (SP-SM), dark yellowish brown, medium dense, fine-grained sand, 5-10% fines			38									
10			POORLY GRADED SAND (SP), yellowish brown, medium dense, fine- to medium-grained sand												
	30		grades fine- to coarse-grained sand			24									
15						16									
			LEAN CLAY WITH SAND (CL), light brownish gray, very stiff, medium plasticity, <20% fine-grained sand											2.5*	PP

Bottom of boring at approximately 16 1/2 feet.
Groundwater not observed during drilling.

SHEAR AND UNCONF STRENGTH W/ ELEV BORINGS_GINT_13618.GPJ ENGEO INC.GDT 1/24/17

ENGEO INCORPORATED

LOG OF BORING 1-B3

Geotechnical Exploration
407 Spreckels Ave
Manteca, CA
13618.000.000

DATE DRILLED: 1/5/2017
HOLE DEPTH: Approx. 27 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 41 ft.

LOGGED / REVIEWED BY: C. Stouffer / ZC
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	40		SILTY SAND (SM), very dark grayish brown, loose, moist, low plasticity, fine-grained sand, 30-40% fines, contains organics, contains non-native rock fragments and concrete [undocumented fill]												
			SILTY SAND (SM), dark yellowish brown, loose to medium dense, moist, fine-grained sand, 15-25% fines [native]			18					5				
5						18									
	35		POORLY GRADED SAND (SP), yellowish brown, loose to medium dense, moist, fine- to medium-grained sand, <5% fines			8				5					
10	30					19					3				
15	25		LEAN CLAY (CL), light brownish gray, stiff, moist, medium plasticity, <5% fine-grained sand			18					4				
20	20		(grades to soft)			8					37				
25			LEAN CLAY (CL), light brownish gray, medium plasticity, <15% fine-grained sand			28					34	87			
	15		LEAN CLAY WITH SAND low to medium plasticity, <30 fine-grained sand			27									
			Bottom of boring at approximately 27 feet. Groundwater not observed during drilling.												

ENGEO
INCORPORATED

LOG OF BORING 1-B4

Geotechnical Exploration
407 Spreckels Ave
Manteca, CA
13618.000.000

DATE DRILLED: 1/5/2017
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 43 ft.

LOGGED / REVIEWED BY: C. Stouffer / ZC
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SILTY SAND (SM), very dark grayish brown, loose, moist, low plasticity, fine-grained sand, 35-45% fines, organics, contains non-native rock fragments approximately 1/2" diameter [undocumented fill]												
	40		SILTY SAND (SM), light grayish brown, medium dense, moist, low plasticity, fine-grained sand, 20-30% fines, contains non-native rock fragments and concrete [undocumented fill]			40									
5			SILTY SAND (SM), dark yellowish brown, medium dense, moist, fine-grained sand [native]												
	35		SILT (ML), gray mottled with yellowish brown, hard, moist, non plastic, fine-grained sand			65				79				>4.5*	PP
			SILTY SAND (SM), yellowish brown, medium dense, moist, fine- to medium-grained sand, <15% fines			27				23					
10			POORLY GRADED SAND (SP), yellowish brown, medium dense, moist, fine- to medium-grained sand, <5% fines			20									
15	30		(grades to medium-grained sand, very pale brown)			40									
	25														
20			LEAN CLAY (CL), light brownish gray, very stiff, moist, medium plasticity, <5% fine-grained sand			27								3.0*	PP
			Bottom of boring at approximately 21 1/2 feet. Groundwater not observed during drilling.												

ENGEO INCORPORATED

LOG OF BORING 1-B5

Geotechnical Exploration
407 Spreckels Ave
Manteca, CA
13618.000.000

DATE DRILLED: 1/5/2017
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 44 ft.

LOGGED / REVIEWED BY: C. Stouffer / ZC
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND (SC), very dark grayish brown, loose, moist, low to medium plasticity, fine-grained sand, contains organics and non-native rock fragments approximately 1/2 inch diameter [undocumented fill]												
			CLAYEY SAND (SC), brown, medium stiff to soft, moist, low plasticity, 30-40% fines, contians concrete and brick fragments [undocumented fill]			21					10				
						9									
	40														
5			SILTY SAND (SM), brown, medium dense, moist, [native]			11				34	8				
			(grades to less fines)												
	35														
10															
			POORLY GRADED SAND WITH SILT (SP-SM), yellowish brown, medium dense, moist, fine- to coarse-grained sand			14				9	3				
	30		(grades to fine- to medium-grained sand)												
15															
			(grades to fine- to coarse-grained sand)			29					3				
			Bottom of boring at approximately 16 1/2 feet. Groundwater not observed during drilling.												

LOG OF BORING 1-B6

ENGEO INCORPORATED

Geotechnical Exploration
407 Spreckels Ave
Manteca, CA
13618.000.000

DATE DRILLED: 1/5/2017
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 43 ft.

LOGGED / REVIEWED BY: C. Stouffer / ZC
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		CLAYEY SAND (SC), very dark grayish brown, loose, moist, low to medium plasticity, 20-30% fine-grained sand, contains organics, non-native rock fragments approximately 1/2" diameter, and concrete [undocumented fill]	65	27	13	14	44					
	40	CLAYEY SAND (SC), dark brown mottled with dark yellowish brown, medium dense, moist, low to medium plasticity, fine-grained sand	11									
5		SILTY SAND (SM), dark yellowish brown, loose, moist, low plasticity, fine-grained sand, 20-30% fines	17									
	35	POORLY GRADED SAND WITH SILT (SP), dark yellowish brown, medium dense, moist, fine-grained sand, <12% fines										
10		POORLY GRADED SAND (SP), pale brown, medium dense, moist, fine- to coarse-grained sand, <5% fines	16									
	30											
15			22									
	25											
20		LEAN CLAY (CL), light brownish gray, stiff, moist, medium plasticity, <15% fine-grained sand	18									

Bottom of boring at approximately 21 1/2 feet.
Groundwater not observed during drilling.

SHEAR AND UNCONF STRENGTH W/ ELEV BORINGS_GINT_13618.GPJ ENGEO INC.GDT 1/24/17

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000		Logged By: Christopher Stouffer Logged Date: 1/6/2016
Test Pit Number	**Depth (Feet)**	**Description**
TP-1	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics, rock fragments, and concrete debris with maximum diameter of 2” [UNDOCUMENTED FILL]
	½ – 3	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand [NATIVE]
TP-2	0 – 3	SILTY SAND (SM) – very dark grayish brown, loose to medium dense, moist, fine-grained sand, contains organics and large chunks of concrete footing observed to a depth of 3 feet [UNDOCUMENTED FILL]
	3-6	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 8” [UNDOCUMENTED FILL]
	6 – 6 ½	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000		Logged By: Christopher Stouffer Logged Date: 1/6/2016
Test Pit Number	**Depth (Feet)**	**Description**
TP-3	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics, brick, concrete, and rock fragments with maximum diameter of 4” [UNDOCUMENTED FILL]
	½ - 1 ½	SILTY SAND (SM) – dark brown, medium dense, moist, fine-grained sand, contains concrete and brick with maximum diameter of 8” [UNDOCUMENTED FILL]
	1 ½ - 4	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 2” [UNDOCUMENTED FILL]
	4 – 4 ½	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]
TP-4	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics, rock fragments, concrete debris with maximum diameter of 2” [UNDOCUMENTED FILL]
	½ - 3	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000		Logged By: Christopher Stouffer Logged Date: 1/6/2016
Test Pit Number	Depth (Feet)	Description
TP-5	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, low to medium plasticity, fine-grained sand, contains organics, rock fragments, concrete debris with maximum diameter of 2” [UNDOCUMENTED FILL]
	½ - 3	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand [NATIVE]
TP-6	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, low to medium plasticity, fine-grained sand, contains organics, rock fragments, concrete debris with maximum diameter 4” and metal pipe with diameter of 4” [UNDOCUMENTED FILL]
	½ - 2	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand [NATIVE]
	2 - 3	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand

ENGEO Expect Excellence	TEST PIT LOG	
407 Spreckels, Ave GEX Manteca, CA 13618.000.000	Logged By: Christopher Stouffer Logged Date: 1/6/2016	

Test Pit Number	Depth (Feet)	Description
TP-7	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics, rock fragments and concrete debris with maximum diameter of 6” [UNDOCUMENTED FILL]
	½ - 2 ½	SILTY SAND (SM) – brown, to medium dense to dense, moist, fine-grained sand, contains rock with maximum diameter of 4” and concrete debris with maximum diameter of 3” [UNDOCUMENTED FILL]
	2 ½ - 3 ½	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand, contains large concrete footing [UNDOCUMENTED FILL]
	3 ½ - 5 ½	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]
TP-8	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics, rock fragments, and concrete debris with maximum diameter of 2” [UNDOCUMENTED FILL]
	½ - 2	SILTY SAND (SM) – dark brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 4” [UNDOCUMENTED FILL]
	2 – 4	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000		Logged By: Christopher Stouffer Logged Date: 1/6/2016
Test Pit Number	Depth (Feet)	Description
TP-9	0 – ½	CLAYEY SAND (SC) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [NATIVE]
	½ - 2 ½	CLAYEY SAND (SC) – dark brown, loose to medium dense, moist, fine-grained sand
TP-10	0 – ½	CLAYEY SAND (SC) – very dark grayish brown, loose, moist, fine-grained sand, contains organics, and rock fragments with maximum diameter of 2” [UNDOCUMENTED FILL]
	½ - 1 ½	CLAYEY SAND (SC) – dark brown, loose to medium dense, moist, fine-grained sand [NATIVE]
	1 ½ - 3	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000	Logged By: Christopher Stouffer Logged Date: 1/6/2016

Test Pit Number	Depth (Feet)	Description
TP-11	0 – 1 ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	1 ½ - 2 ½	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand, concrete debris and brick fragments with maximum diameter 3” [UNDOCUMENTED FILL]
	2 ½ - 3 ½	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]
TP-12	0 – 1 ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	1 ½ - 5.5	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand, contains large concrete with maximum diameter of 8” to 5.5 feet and metal pipe of 4” diameter at 3.5’ [UNDOCUMENTED FILL]
	5 ½ - 6 ½	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000		Logged By: Christopher Stouffer Logged Date: 1/6/2016
Test Pit Number	Depth (Feet)	Description
TP-13	0 – 1	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics and rock fragments with maximum diameter of 2” [UNDOCUMENTED FILL]
	1 – 4 ½	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand, contains concrete footing encountered at 1 ½ feet and debris with maximum diameter of 4” [UNDOCUMENTED FILL]
	4 ½ - 6	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]
TP-14	0 – 1	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics and rock fragments with maximum diameter of 2” [UNDOCUMENTED FILL]
	1 - 3	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000		Logged By: Christopher Stouffer Logged Date: 1/6/2016
Test Pit Number	**Depth (Feet)**	**Description**
TP-15	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [NATIVE]
	½ - 2	SILTY SAND (SM) – dark yellowish brown, loose to medium dense, moist, fine-grained sand
	2 – 3	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand
TP-16	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics and rock fragments with maximum diameter 2" [UNDOCUMENTED FILL]
	½ - 1 ½	SILTY SAND (SM) –brown, loose to medium dense, moist, fine-grained sand, contains organics, rock fragments with maximum diameter 2", and asphalt and cement debris with maximum diameters of 4" [UNDOCUMENTED FILL]
	1 ½ - 3	SILTY SAND (SM) – dark yellowish brown, medium dense, moist, fine-grained sand [NATIVE]

ENGEO Expect Excellence	TEST PIT LOG	
407 Spreckels, Ave GEX Manteca, CA 13618.000.000	Logged By: Christopher Stouffer Logged Date: 1/6/2016	

Test Pit Number	Depth (Feet)	Description
TP-17	0 – 2	SILTY SAND (SM) – very dark grayish brown, loose to medium dense, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	2 – 3	SILTY SAND (SM) – dark brown, medium dense, moist, fine-grained sand, contains concrete and brick debris with maximum diameter of 2” [UNDOCUMENTED FILL]
	3 – 6 ¾	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]
TP-18	0 – ½	CLAYEY SAND (SC) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	½ - 1 ½	CLAYEY SAND (SC) – dark brown, loose to medium dense, moist, fine-grained sand, contains large concrete debris with maximum diameter of 10” to depth of 1 ½ feet [UNDOCUMENTED FILL]
	1 ½ - 4	SILTY SAND (SM) – dark brown, dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 6” to depth of 1 ½ feet and asphalt with maximum diameter of 6” to depth of 4 feet [UNDOCUMENTED FILL]
	4 - 5	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000	Logged By: Christopher Stouffer Logged Date: 1/6/2016

Test Pit Number	Depth (Feet)	Description
TP-19	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics, rock fragments, and concrete debris with a maximum diameter of 2” [UNDOCUMENTED FILL]
	½ - 3	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains rock fragments, concrete chunks and asphalt with maximum diameter of 4” [UNDOCUMENTED FILL]
	3 – 6 ½	SANDY LEAN CLAY (CL) – black, medium stiff, low plasticity [UNDOCUMENTED FILL]
	6 ½ - 7 ½	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]
TP-20	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	½ - 4	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains concrete footing at depth of 1 foot, asphalt and concrete debris with maximum diameter of 6” and tree root with diameter of 4” [UNDOCUMENTED FILL]
	4 – 6 ¾	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 3” [UNDOCUMENTED FILL]
	6 ¾ - 7 ½	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000		Logged By: Christopher Stouffer Logged Date: 1/6/2016
Test Pit Number	Depth (Feet)	Description
TP-21	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	½ - 3	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 4” [UNDOCUMENTED FILL]
	3 – 5	SILTY SAND (SM) – black, medium stiff, low plasticity [UNDOCUMENTED FILL]
	5 - 7	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]
TP-22	0 – 1	SILTY SAND (SM) – very dark grayish brown, loose to medium dense, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	1 – 2 ½	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 4” [UNDOCUMENTED FILL]
	2 ½ - 4	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000	Logged By: Christopher Stouffer Logged Date: 1/6/2016

Test Pit Number	Depth (Feet)	Description
TP-23	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose to medium dense, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	½ - 3	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 4” [UNDOCUMENTED FILL]
	3 – 4	SANDY LEAN CLAY (CL) – black, medium stiff, low plasticity, contains concrete and asphalt debris with maximum diameter of 3” [UNDOCUMENTED FILL]
	4 – 5	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 3” [UNDOCUMENTED FILL]
	5 – 6	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]

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TEST PIT LOG

407 Spreckels, Ave GEX
Manteca, CA
13618.000.000

Logged By: Christopher Stouffer
Logged Date: 1/6/2016

Test Pit Number	Depth (Feet)	Description
TP-24	0 – ½	SILTY SAND (SM) – very dark grayish brown, loose to medium dense, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	½ - 3 ½	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 4” [UNDOCUMENTED FILL]
	3 ½ - 5 ½	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 4” [UNDOCUMENTED FILL]
	5 ½ - 7	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]
TP-25	0 – 1 ½	CLAYEY SAND (SC) – very dark grayish brown, loose to medium dense, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	1 ½ - 3	CLAYEY SAND (SC) – dark yellow brown, medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 6” [UNDOCUMENTED FILL]
	3 – 5	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]

TEST PIT LOG

407 Spreckels, Ave GEX Manteca, CA 13618.000.000	Logged By: Christopher Stouffer Logged Date: 1/6/2016

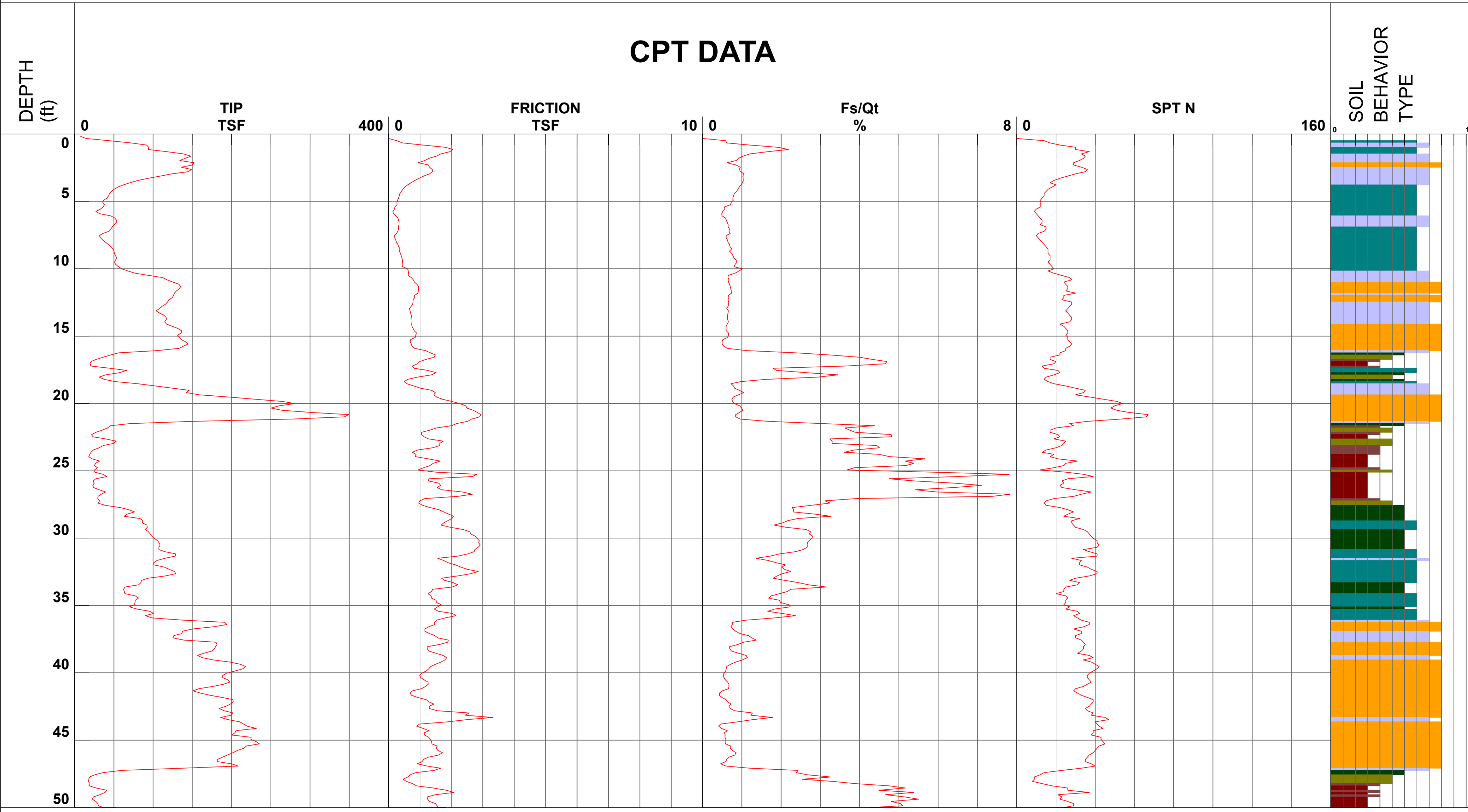
Test Pit Number	Depth (Feet)	Description
TP – 26	0 – ½	CLAYEY SAND (SC) – very dark grayish brown, loose, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	½ - 3	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains concrete debris with maximum diameter of 3” [UNDOCUMENTED FILL]
	3 – 4	SANDY LEAN CLAY (CL) – black, medium stiff, low plasticity [UNDOCUMENTED FILL]
	4 - 6	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]
TP-27	0 – 1	SILTY SAND (SM) – very dark grayish brown, loose to medium dense, moist, fine-grained sand, contains organics [UNDOCUMENTED FILL]
	1 – 1 ¾	SILTY SAND (SM) – dark yellow brown, loose to medium dense, moist, fine-grained sand, contains asphalt and rock fragment debris [UNDOCUMENTED FILL]
	1 ¾ - 5	SILTY SAND (SM) – dark yellow brown, medium dense, moist, fine-grained sand [NATIVE]

Engeo Inc

Project	Spreckels	Operator	KK-RB	Filename	SDF(583).cpt
Job Number	TBD	Cone Number	DDG1333	GPS	
Hole Number	CPT-02	Date and Time	1/6/2017 8:05:22 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

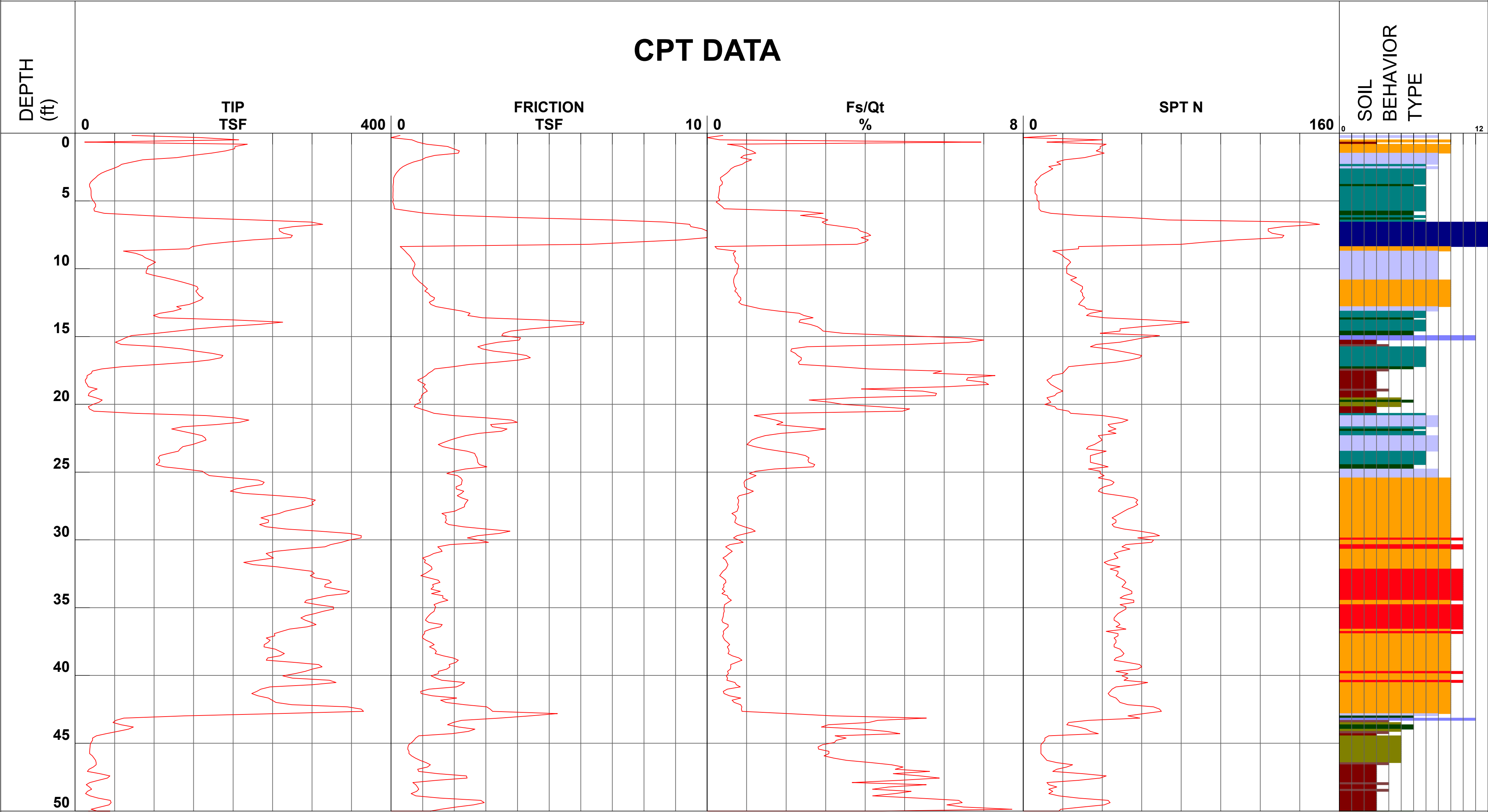
S*Soil behavior type and SPT based on data from UBC-1983

Engeo Inc

Project	Spreckels	Operator	KK-RB	Filename	SDF(582).cpt
Job Number	TBD	Cone Number	DDG1333	GPS	
Hole Number	CPT-01	Date and Time	1/6/2017 7:18:06 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

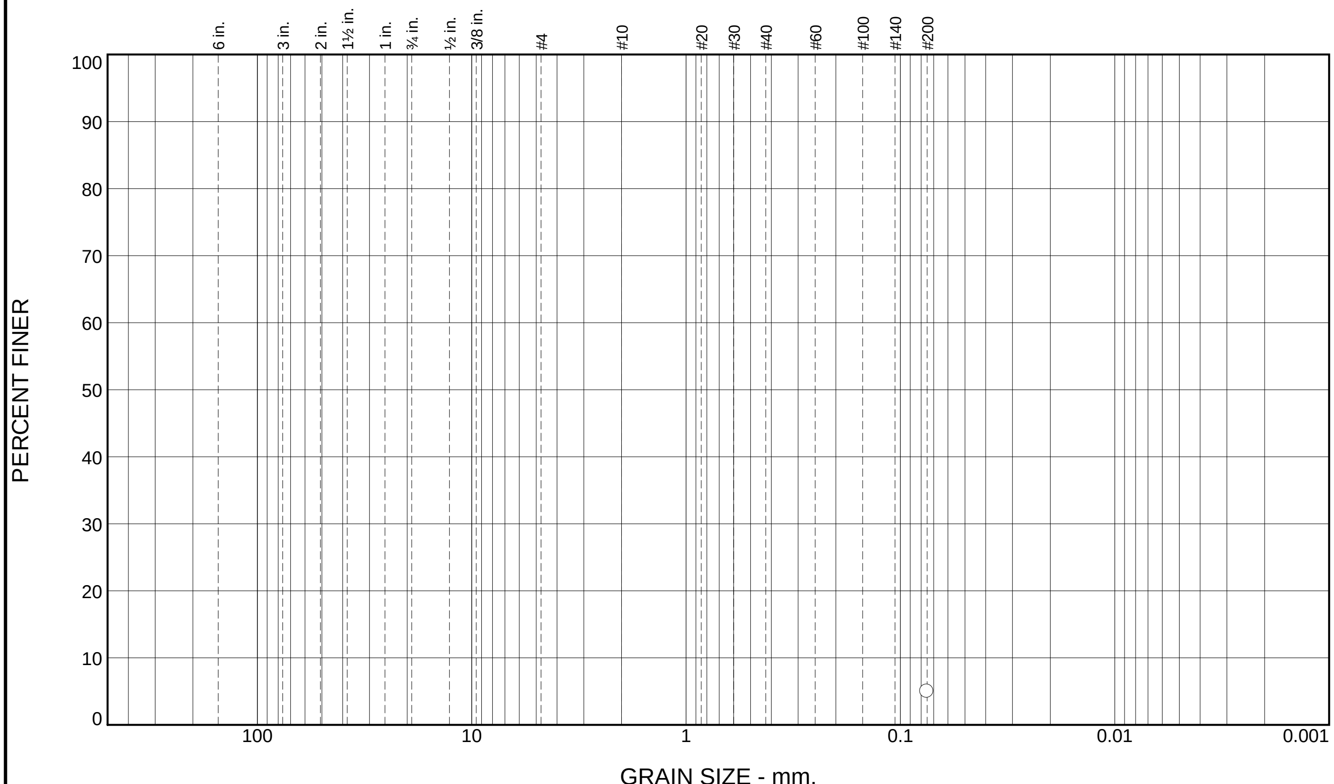
S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX B

LABORATORY TEST DATA

Particle Size Distribution Report
Liquid and Plastic Limits Test Report
R-Value Test Report
Sunland Analytical Test Report

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						5.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	5.0		

* (no specification provided)

Soil Description

See Exploration Log

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Source of Sample: GEX **Depth:** 6
Sample Number: 1-B3 @ 6

Date: 1-12-17

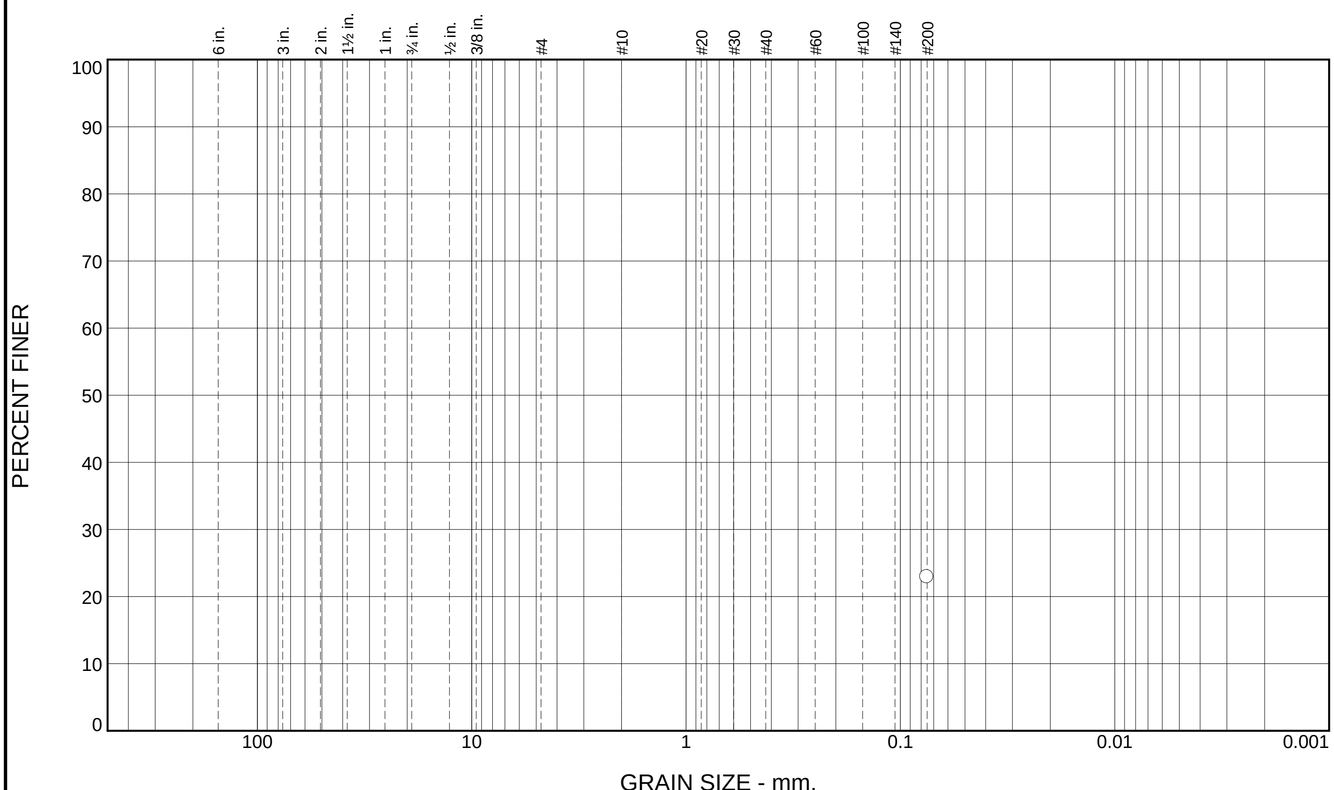
ENGEO INCORPORATED

Client: DCT Industrial
Project: 407 Spreckles Ave.

Project No: 13618.000.000 **Figure**

Tested By: I. McCauley **Checked By:** K. Lecce

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						22.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	22.9		

* (no specification provided)

Soil Description

See Exploration Log

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Source of Sample: GEX **Depth:** 8.5
Sample Number: 1-B4 @ 8.5

Date: 1-12-17

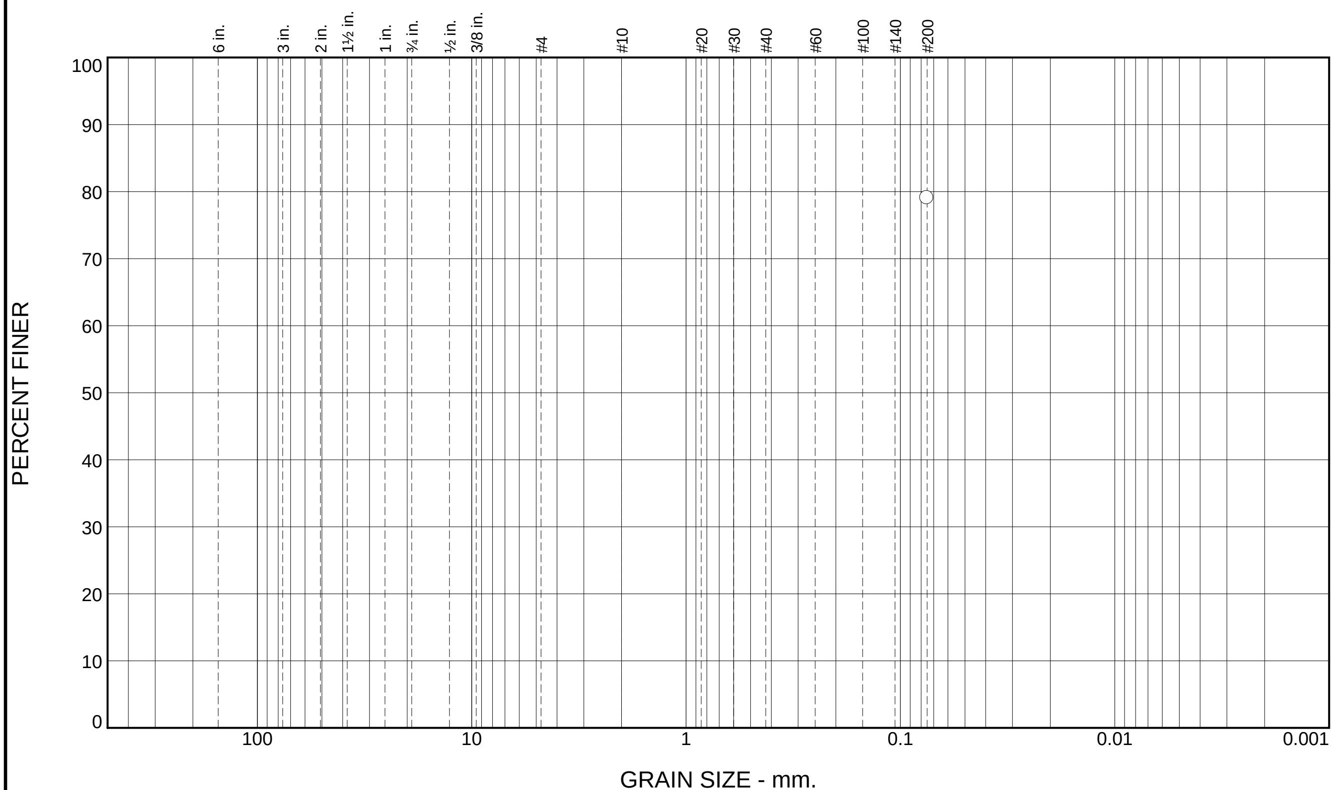
ENGEO INCORPORATED

Client: DCT Industrial
Project: 407 Spreckles Ave.

Project No: 13618.000.000 **Figure**

Tested By: I. McCauley **Checked By:** K. Lecce

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						79.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	79.0		

* (no specification provided)

Soil Description

See Exploration Log

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Source of Sample: GEX **Depth:** 8
Sample Number: 1-B4 @ 8

Date: 1-12-17

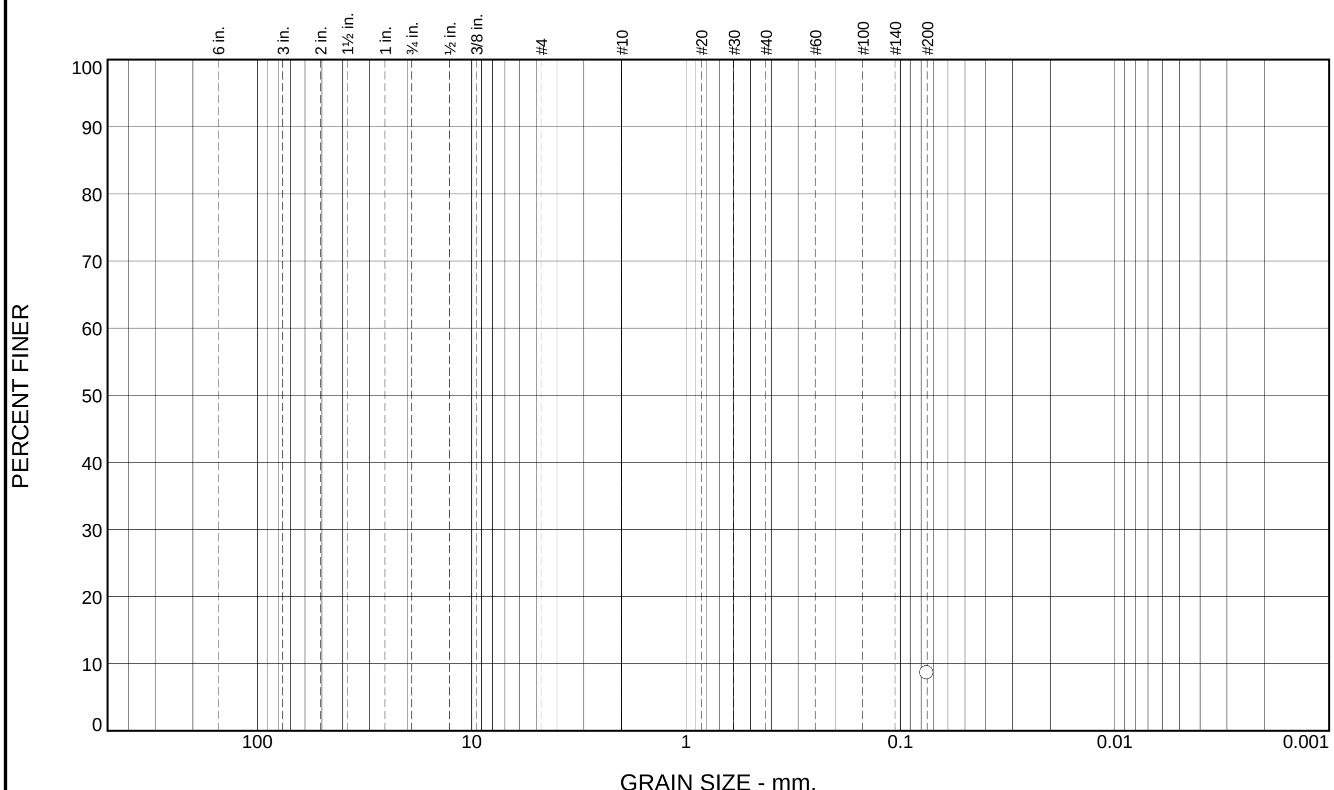
ENGEO INCORPORATED

Client: DCT Industrial
Project: 407 Spreckles Ave.

Project No: 13618.000.000 **Figure**

Tested By: I. McCauley **Checked By:** K. Lecce

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						8.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	8.6		

* (no specification provided)

Soil Description

See Exploration Log

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Source of Sample: GEX **Depth:** 10
Sample Number: 1-B5 @ 10

Date: 1-12-17

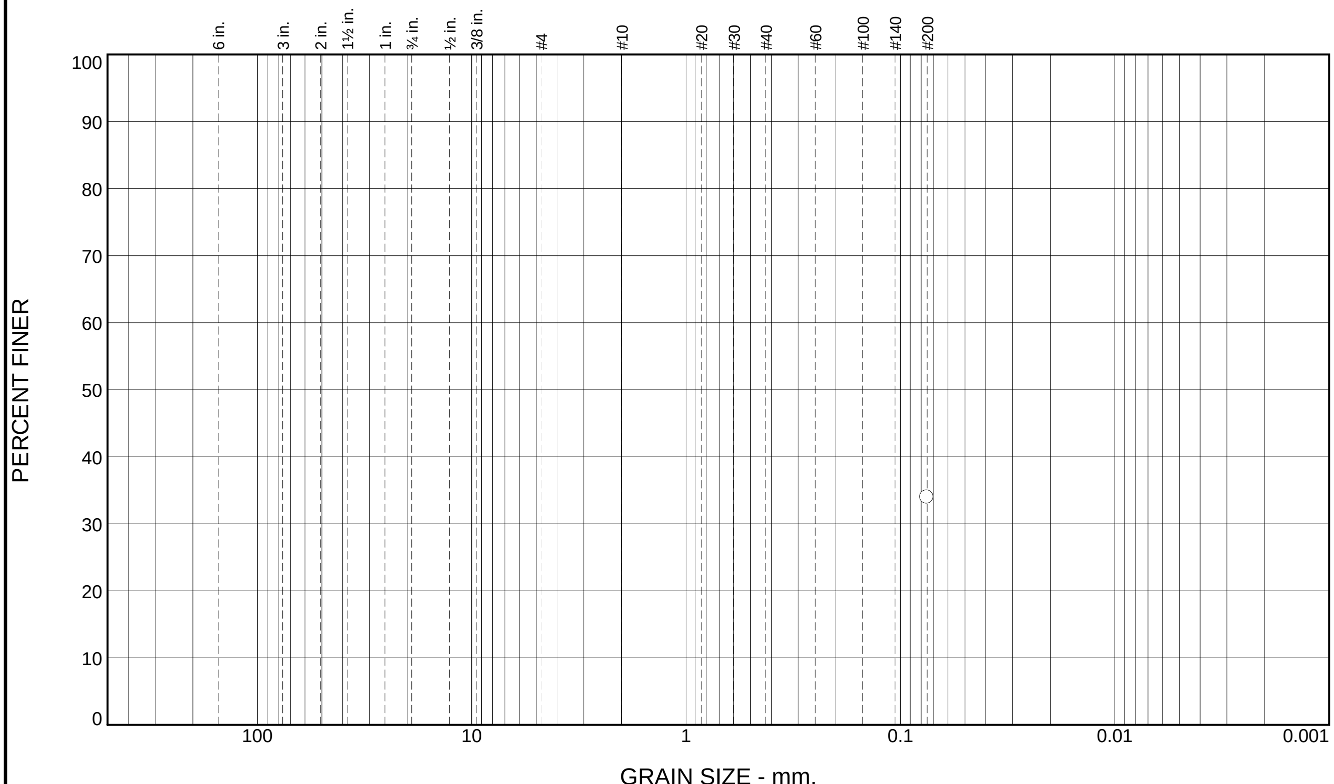
ENGEO INCORPORATED

Client: DCT Industrial
Project: 407 Spreckles Ave.

Project No: 13618.000.000 **Figure**

Tested By: I. McCauley **Checked By:** K. Lecce

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						33.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	33.9		

* (no specification provided)

Soil Description

See Exploration Log

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Source of Sample: GEX **Depth:** 5
Sample Number: 1-B5 @ 5

Date: 1-12-17

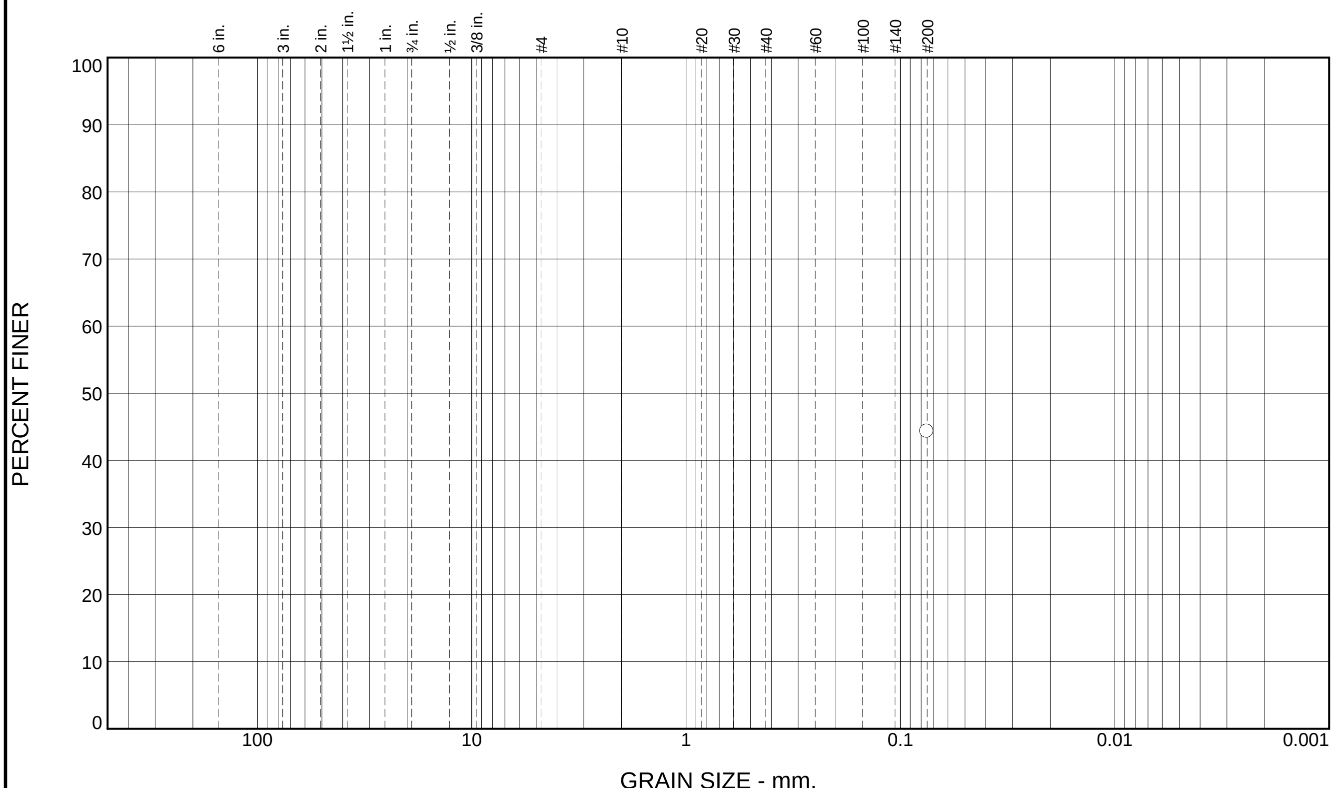
ENGEO INCORPORATED

Client: DCT Industrial
Project: 407 Spreckles Ave.

Project No: 13618.000.000 **Figure**

Tested By: I. McCauley **Checked By:** K. Lecce

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						44.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	44.3		

* (no specification provided)

Soil Description

See Exploration Log

Atterberg Limits

PL= 13 LL= 27 PI= 14

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140
ASTM D4318, wet method

Source of Sample: GEX **Depth:** 2
Sample Number: 1-B6 @ 2 **Date:** 1-12-17

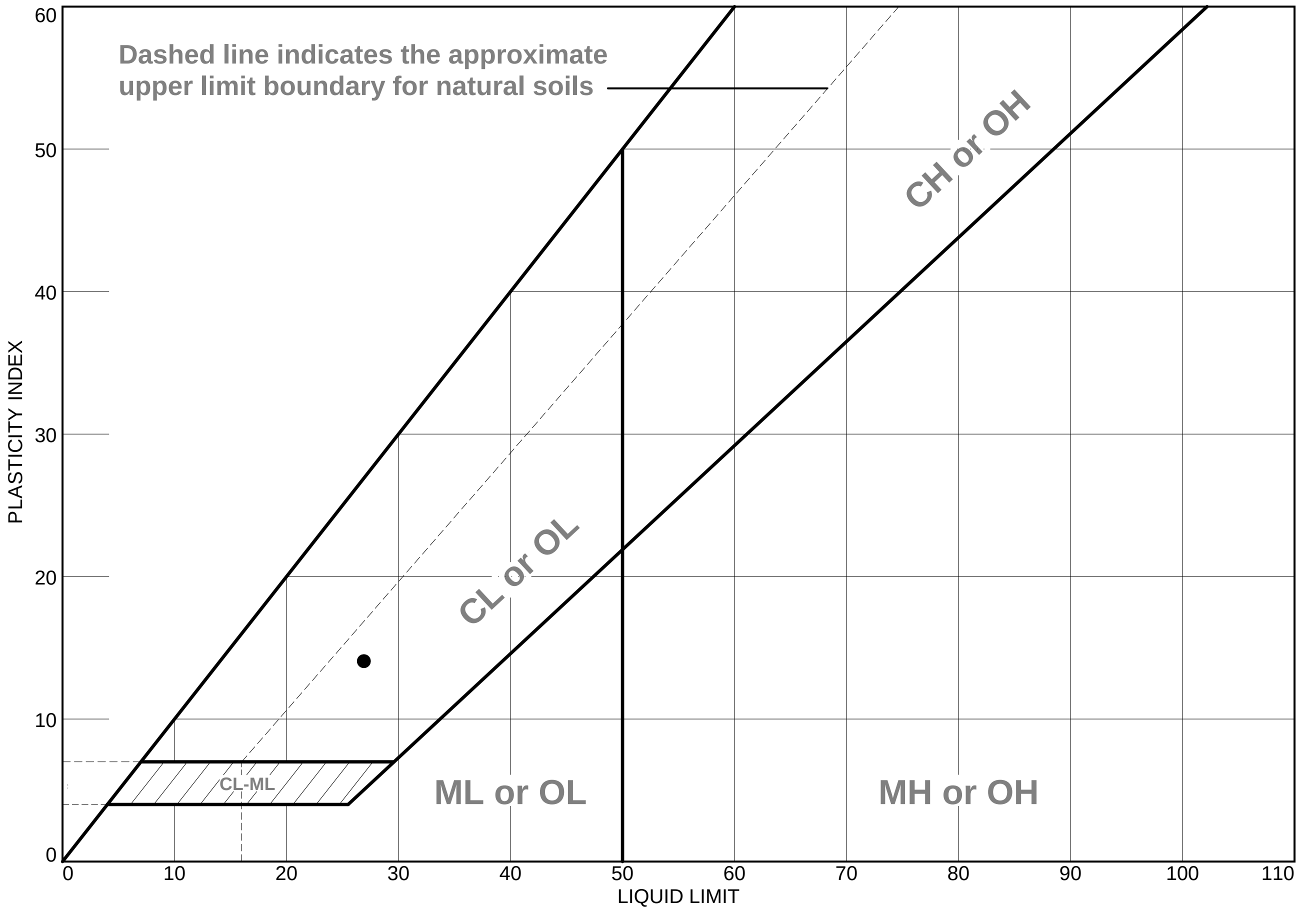
ENGEO INCORPORATED

Client: DCT Industrial
Project: 407 Spreckles Ave.

Project No: 13618.000.000 **Figure**

Tested By: I. McCauley **Checked By:** K. Lecce

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See Exploration Log	27	13	14		44.3	

Project No. 13618.000.000 **Client:** DCT Industrial

Project: 407 Spreckles Ave.

●**Source of Sample:** GEX **Depth:** 2 **Sample Number:** 1-B6 @ 2

Remarks:

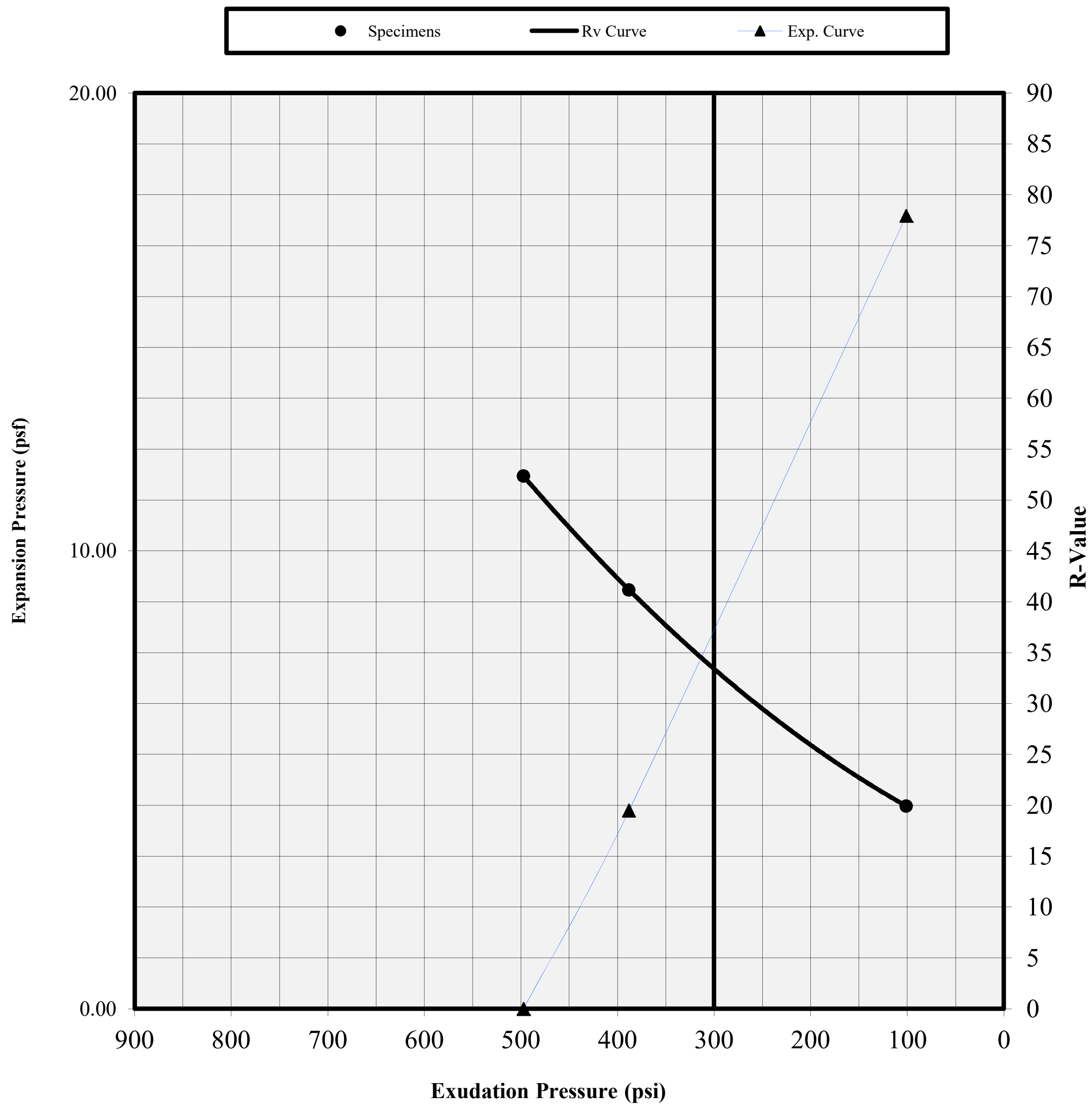
●ASTM D4318, wet method
ASTM D1140

ENGEO INCORPORATED

Figure

Tested By: I. McCauley **Checked By:** K. Lecce

R VALUE TEST REPORT
CTM-301



Sample ID/Location: Spreckles Ave. 0-5

Description: Dark grayish brown silty SAND

Test remarks:

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	497	388	101
Expansion dial (0.0001")	0	1	4
Expansion Pressure (p.s.f.)	0	4	17
Resistance Value, "R"	52	41	20
% Moisture at Test	10.3	11.0	11.8
Dry Density at Test, p.c.f.	121.8	119.6	117.7
"R" Value at Exudation Pressure of 300 psi.	34		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	8		

PROJECT NAME: 407 Spreckels Ave
PROJECT NUMBER: 13618.000.000
CLIENT: DCT Industrial
PHASE NUMBER: 001

DATE: 01/12/16

ENGEO
Expect Excellence

Tested by: W. Miller

Reviewed by: D. Seibold

Lab Address : 3420 Fostoria Way, Suite E, San Ramon, CA 94583

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 01/18/2017
Date Submitted 01/12/2017

To: Chris Stouffer
Engeo Inc.
580 Golden Valley Pkwy
Lathrop CA 95330

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 1-B3@3.5F+1-B5@2.5F Site ID : 1/12/17.
Thank you for your business.

* For future reference to this analysis please use SUN # 73489-153310.

EVALUATION FOR SOIL CORROSION

Soil pH	8.04		
Minimum Resistivity	1.45 ohm-cm (x1000)		
Chloride	55.0 ppm	0.00550	%
Sulfate	63.2ppm	0.00632	%

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

APPENDIX C

LIQUEFACTION ANALYSIS RESULTS

LIQUEFACTION ANALYSIS REPORT

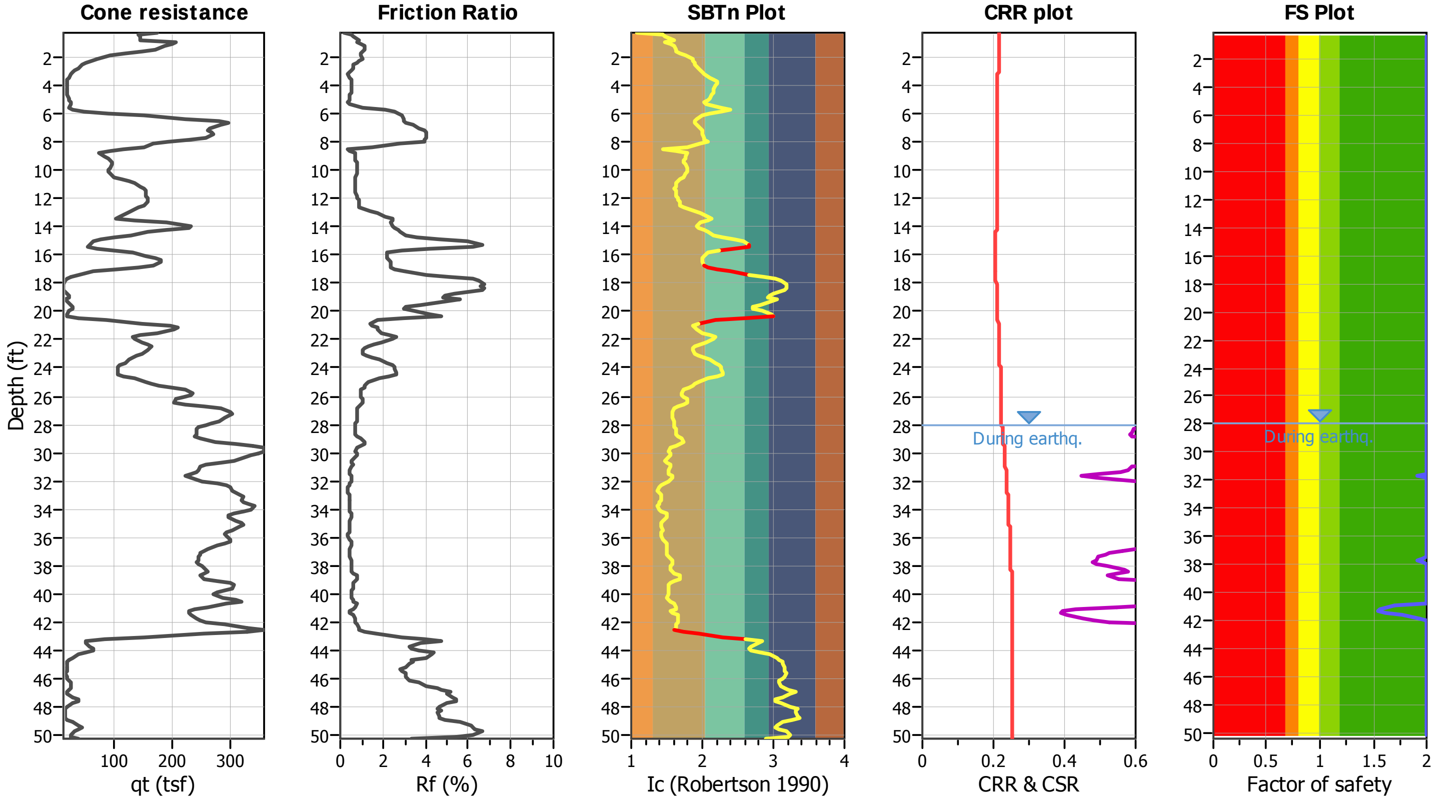
Project title : 407 Spreckels Ave **Location : Manteca, CA**

CPT file : CPT-01

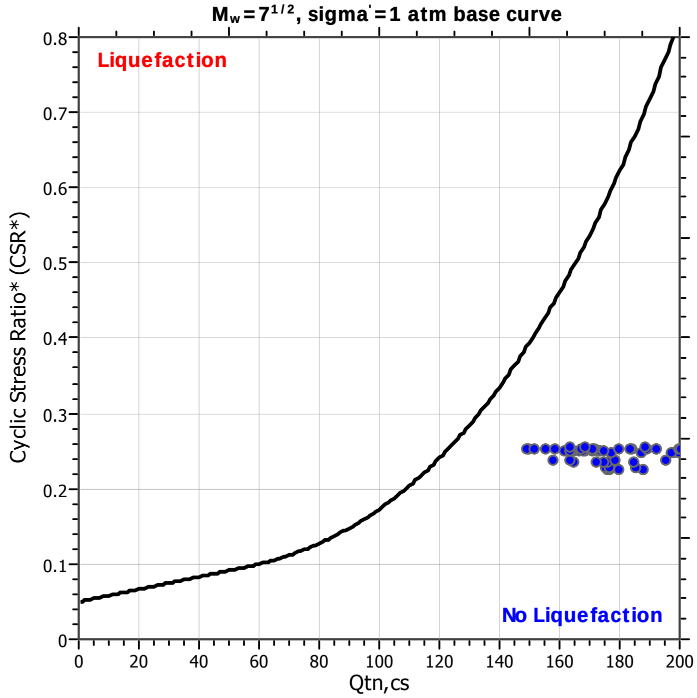
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	28.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	28.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

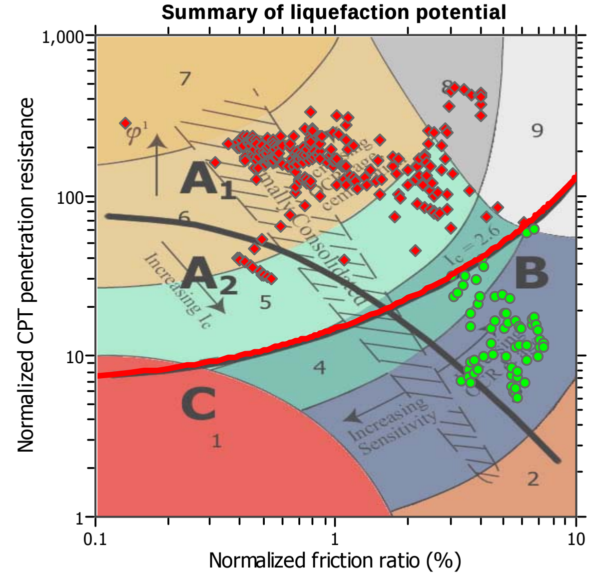
Cone resistance **Friction Ratio** **SBTn Plot** **CRR plot** **FS Plot**



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

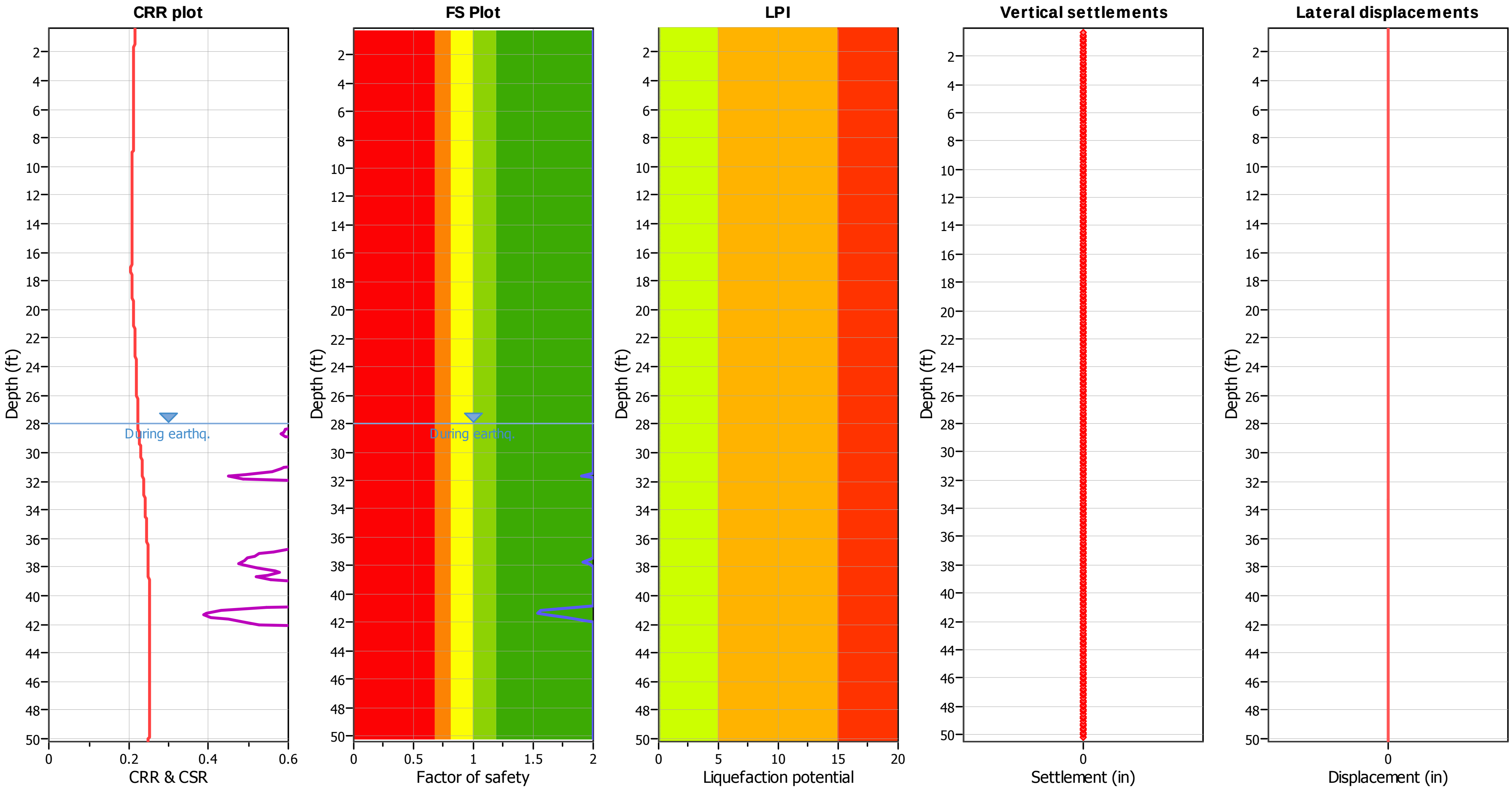


Summary of liquefaction potential



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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	28.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	28.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Inc.
17278 Golden Valley Pkwy
Lathrop, CA
www.engeo.com

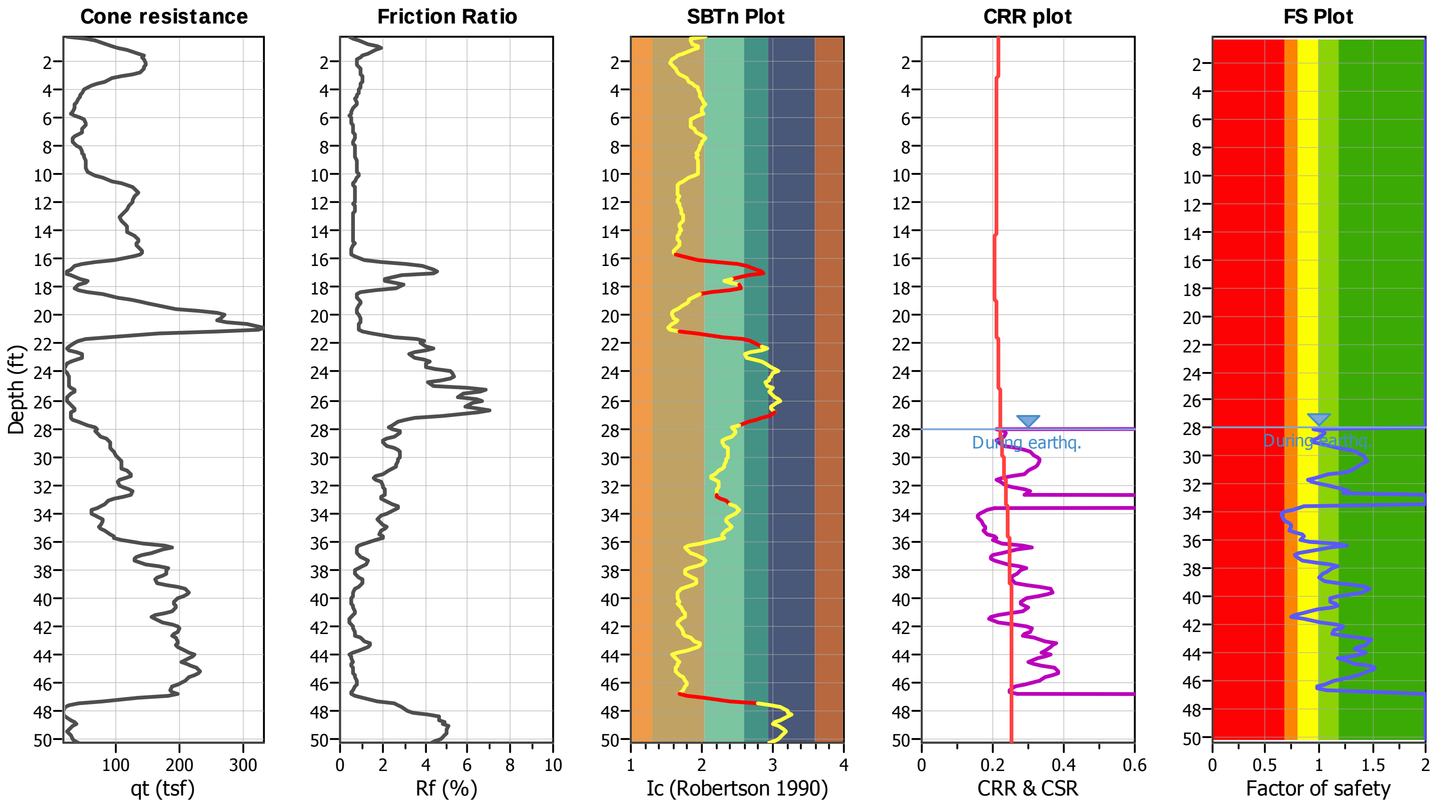
LIQUEFACTION ANALYSIS REPORT

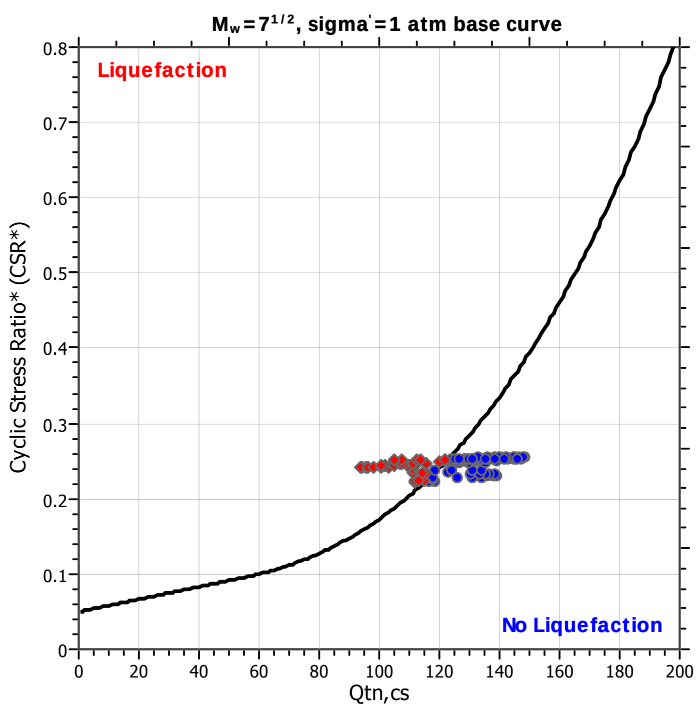
Project title : 407 Spreckels Ave **Location : Manteca, CA**

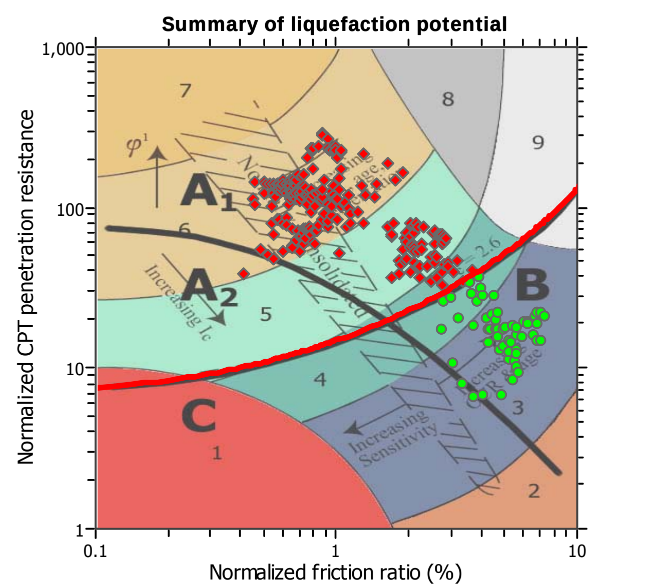
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	28.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	28.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

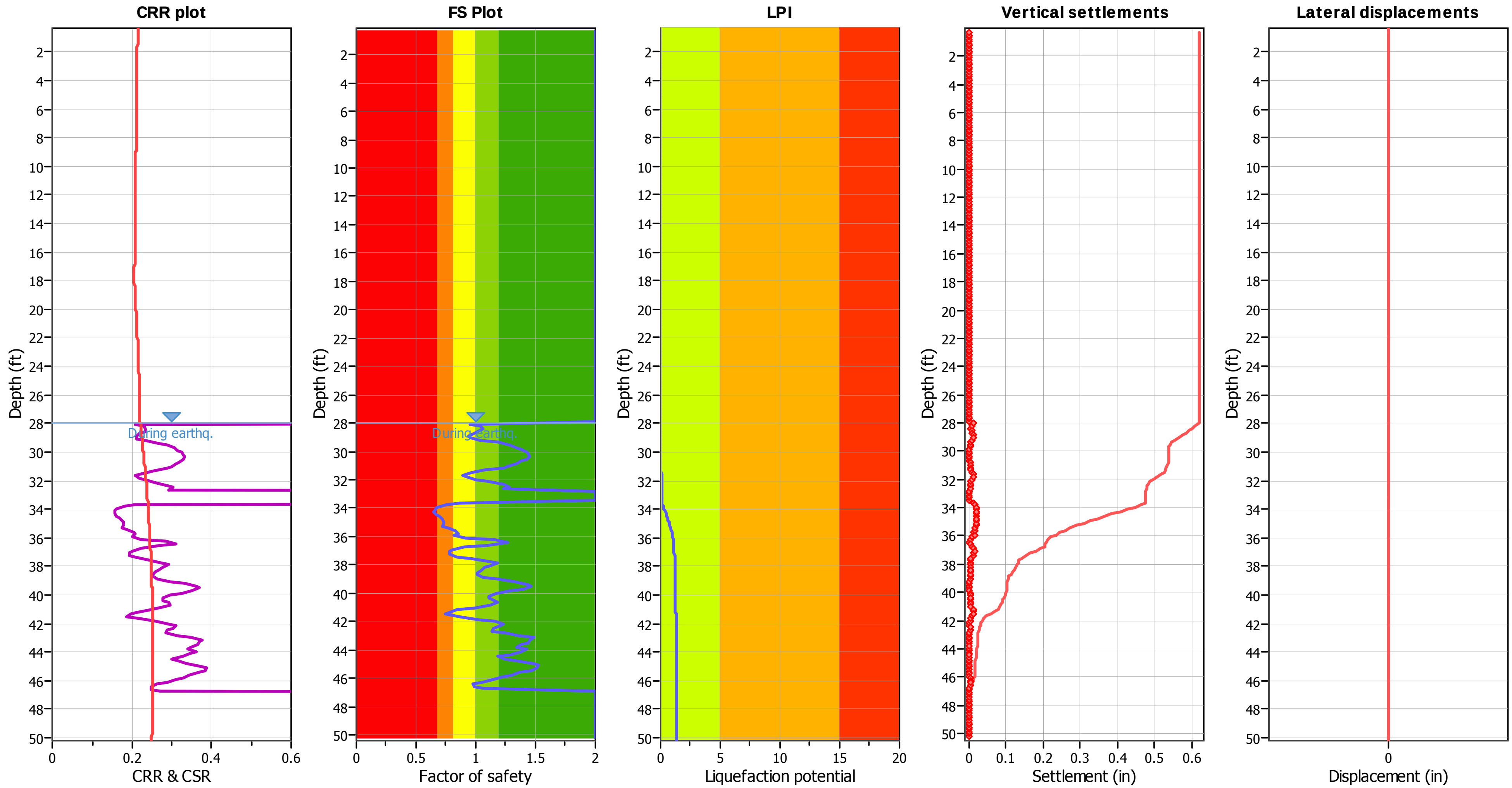






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	28.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	28.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

SAN RAMON

SAN FRANCISCO

SAN JOSE

OAKLAND

LATHROP

ROCKLIN

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