Appendix I

Geotechnical Report

HANOVER NORTH SAN JOSE
SAN JOSE, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Kristen Gates
Due Diligence Manager
Hanover R.S. Limited Partnership
1780 South Post Oak Lane
Houston, TX 77056

PREPARED BY
ENGEO Incorporated

February 23, 2021
Revised March 30, 2021

PROJECT NO.
18233.000.001

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Ms. Kristen Gates
Due Diligence Manager
Hanover R.S. Limited Partnership
1780 South Post Oak Lane
Houston, TX 77056

Subject: Hanover North San Jose
681 East Trimble Road
San Jose, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Ms. Gates:

This report summarizes geotechnical and geological existing conditions and constraints and provides preliminary geotechnical recommendations for the proposed Hanover North San Jose development, as outlined in our agreement dated January 15, 2021. The preliminary conclusions and recommendations of this report are based on geotechnical and geologic studies completed to date.

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

- Undocumented fill or compressible native soil that could undergo excessive settlement under new structural loads or additional fill.
- Surface manifestations of liquefaction

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

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

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical exploration, as described in our agreement dated January 15, 2021, and addendum, dated March 22, 2021, is to provide an assessment of the potential geotechnical and geologic concerns for the proposed mixed-use development and associated improvements. The scope of our services included a site visit, a review of published geologic maps, review of readily available geotechnical reports for the site, advancing six cone penetration test (CPT) soundings to evaluate subsurface conditions, advancing four hand auger holes to collect near-surface soil samples, advancing one direct push sampler to collect additional samples, laboratory testing of collected soil samples, and preparation of this report discussing potential geotechnical and geologic hazards.

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

1.2 PROJECT LOCATION

The approximately 11-acre site is located on 681 East Trimble Road, north of the intersection of Seely Avenue and Montague Expressway in San Jose, California as shown on the Vicinity Map, Figure 1 and is associated with Assessor's Parcel Number (APN) 097-15-033. Access to the site is provided by an unpaved driveway along Seely Avenue. The site is bounded by Seely Avenue to the southwest, Montague Expressway to the southeast, Coyote Creek to the northeast, and undeveloped land to the northwest.

1.3 PROJECT DESCRIPTION

Based on our review of the Site Density study (Figure 2) and discussions with you, we understand the proposed development will include construction of two phases of residential development and construction of an office complex. Phase I of the residential development will include a 5-story residential complex and a 5½-story parking garage in the western portion of the site. Phase II of the residential development will include a 5-story residential complex and a 6-story parking garage in the northern portion of the site. The office complex in the southern portion of the site will consist of construction of an 8-story structure with 4 levels of parking.

Structural loads and grading are yet to be determined; however, we assume that structural loads will be representative for this type of construction. According to the site density study, prepared by The Hanover Company (July 2020), approximately 720 residential units and roughly 200,000 square feet of commercial space are proposed at the site. Conceptual grading plans were not available for our review but we anticipate minor cuts, and fill to accommodate the development.

1.4 SITE BACKGROUND

We reviewed historical aerial photographs, and published geologic and hazard maps for the site and local vicinity. Based on aerial photographs dated between 1948 and 2016, as well as the existing conditions observed during our site visits, the site appears to be continuously used for agricultural purposes since the late 1930s. Currently, existing structures on the site include two single-family homes, barns, other storage structures, a fruit stand, and agricultural land (orchards, fruits, and vegetables) and multiple piles of debris.

2.0 SITE GEOLOGY AND SEISMICITY

2.1 REGIONAL AND SITE GEOLOGY

The region is within the North Coast Range Province of California, an area dominated by northwest-trending geologic features such as folds and faults. The San Francisco Bay is located in a fault bound, elongated structural trough that has been filled with a sequence of Quaternary age sedimentary deposits derived from the surrounding Coast Ranges. According to mapping by Dibblee (2005) shown in Figure 3, the subject site is located on Quaternary alluvial gravel, sand, and clay (Qa).

2.2 SITE SEISMICITY

The San Francisco Bay Area contains numerous active faults. Figure 4 shows the approximate location of active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region. An active fault is defined by the State as one that has had surface displacement within Holocene time, about the last 11,000 years (Hart and Bryant, 1997).

To determine nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the USGS Unified Hazard Tool* and deaggregated the hazard at the peak ground acceleration (PGA). The nearest active fault with a significant contribution to the overall seismic hazard at the site is the Silver Creek fault, approximately 0.7 mile away. This fault is considered capable of generating earthquakes with moment magnitudes up to 6.8. Other active faults located near the site include the Hayward fault, which is located approximately 4.5 miles away and considered capable of generating a moment magnitude earthquake of up to 7.1, the Calaveras fault, which is located approximately 6.8 miles away and considered capable of generated a moment magnitude of 7.3, and the San Andreas fault, which is located approximately 13.6 miles away and considered capable of generating a moment magnitude earthquake of 8.0.

TABLE 2.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site Latitude: 37.396461, Longitude: -121.916785

SOURCE	R_{RUP} (KM)	R_{RUP} (MILES)	MOMENT MAGNITUDE M_W
Silver Creek [6]	1.1	0.7	6.8
Hayward (So) [1]	7.3	4.5	6.8
Hayward (So) [0]	7.3	4.5	7.1
Hayward (So) [2]	10.6	6.6	6.9
Calaveras (No) [6]	11.0	6.8	7.3

SOURCE	R_{RUP} (KM)	R_{RUP} (MILES)	MOMENT MAGNITUDE M_W
Calaveras (Central)	11.1	6.9	6.8
San Andreas (Peninsula) [2]	21.9	13.6	8.0

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

These results represent sources contributing at least one percent to the seismic hazard at the site for the peak ground acceleration and for the given return period. Gridded or areal sources are not presented; however, these sources did not contribute more than one percent to the seismic hazard for the peak ground acceleration and for the given return period.

The Uniform California Earthquake Rupture Forecast (UCERF3, 2014) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF3 generated an overall probability of 72 percent for the San Francisco Region as a whole, a probability of 14.3 percent for the Hayward fault, 7.4 percent for the Calaveras fault, and 6.4 percent for the northern section of the San Andreas fault.

2.3 SURFACE CONDITIONS

This site gradually slopes from northeast to southwest. Based on preliminary topographic data obtained from Google Earth, the site elevation ranges from approximately 34 feet on the west end of the site to 44 feet on the east end of the site (WGS84). The site is currently occupied by agricultural land and several structures, including two single-family residences, barns, storage sheds, and a fruit stand.

2.4 FIELD EXPLORATIONS

We performed our field explorations between January 25 and January 29, and on March 16, 2021. Our field explorations included six cone penetration test (CPT) soundings, four hand auger holes, and one direct push probe (DP) hole at the locations on the site as shown on Figure 2. The exploratory points were roughly located by pacing from existing features and should be considered accurate only to the degree implied by the method used.

We retained a CPT rig to push the cone penetrometer to a maximum depth of about 51 feet, where it encountered practical refusal. The CPT has a 20-ton compression-type cone with a 15-square-centimeter (cm^2) base area, an apex angle of 60 degrees, and a friction sleeve with a surface area of 225 cm^2. The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). CPT logs are presented in Appendix A. Two of the CPTs, 1-CPT1 and 1-CPT3, included shear wave velocity measurements. During the advancement of 1-CPT4, electronic issues with the probe resulted in a loss of sleeve friction data. As such, data from 1-CPT4 were not used in o our analyses.

We retained a subcontractor to advance one direct push (DP) probe hole using a Geoprobe® direct push rig to a maximum depth of approximately 15 feet below the existing grade. The probe was advanced as a matched-pair in close proximity to 1-CPT3, and during the field exploration, two soil samples were collected from approximately 8 to 13 feet below the ground surface (bgs) and from 13 to 15 feet bgs, respectively.

In addition, we advanced four hand auger holes for collecting near-surface soil samples (Samples S-1 through S-4).

2.5 LABORATORY TESTING

We performed laboratory testing on four near-surface samples (S-1 through S-4) and two geoprobe samples. The laboratory testing included determination of Plasticity Index (PI) using the Atterberg limits method and particle size distribution as described in ASTM D4318 and ASTM D1140 (Method B), respectively. The results of the laboratory testing are provided in Appendix B.

2.6 SUBSURFACE CONDITIONS

The CPT data indicate the site soil composed predominantly of clay and silty clay. With the exception of 1-CPT6, which encountered clay and silt to the maximum depth explored, a sandy layer was encountered at several feet below grade in each exploration with a thickness varying from approximately 2 to 8 feet. Below this sandy material to about 35 feet below existing grade, the explorations encountered clay and silty clay. These clayey materials are underlain by dense sand or gravel to the maximum depths explored.

According to the laboratory test results, Plasticity Indices of near-surface samples ranges from 7 to 10, indicating low expansion potential of the near-surface soil tested.

2.7 GROUNDWATER CONDITIONS

Pore pressure dissipations testing performed as part of our CPT explorations indicate an in-situ groundwater level of approximately 13 feet below existing grade. Based on the Seismic Hazard Zone Report for the Milpitas Quadrangle (2004), the historically high groundwater level is approximately 8 feet below existing grade. Therefore, we recommend considering a design groundwater depth of 8 feet below existing ground surface. Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

3.0 DISCUSSION AND PRELIMINARY CONCLUSIONS

Based upon this preliminary study, it is our opinion that the project site is feasible for the proposed mixed-used development from a geotechnical standpoint provided the development plans incorporate the preliminary recommendations contained in this report and future design-level geotechnical studies. A more comprehensive site-specific geotechnical exploration should be performed as part of the design process. The exploration would include borings and laboratory soil testing to provide data for preparation of specific recommendations regarding grading and foundation design. The exploration will also allow for more detailed evaluations of the geotechnical issues discussed below and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

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

- Undocumented fill / disturbed near-surface soil
- Compressible soil

- Liquefaction or cyclic softening

3.1 UNDOCUMENTED FILL/DISTURBED NEAR-SURFACE SOIL

As previously mentioned, much of the development area has been used for agricultural purposes, which has likely resulted in disturbed near-surface soil.

Disturbed near-surface soil and undocumented fill may undergo excessive settlement, especially when subject to new loads from grading and the planned structures. Detailed mapping of subsurface fill materials should be performed at the time of our design-level study. We present fill removal recommendations in Section 4.1.

3.2 COMPRESSIBLE SOIL

The saturated clay materials that are present in the upper 35 feet will likely undergo consolidation settlement under the imposed structural loads. While structural loads are not available at the time of this report, we considered typical loads for the types of structures proposed in our preliminary settlement analyses. We anticipate that settlements will remain within tolerable levels for a post-tensioned mat foundation system. Evaluation of the settlement due to imposed structural load or fill material will be conducted during the final geotechnical exploration.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections.

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

3.3.1 Ground Rupture

The project site is not located within an Alquist-Priolo Earthquake Fault Zone. This indicates that there is a low potential for surface expression of fault rupture.

3.3.2 Ground Shaking

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

3.3.3 Liquefaction/Cyclic Softening

The project site is mapped within a zone of required investigation for liquefaction by the State of California as shown on Figure 5. Soil liquefaction results from loss of strength during cyclic loading, such as that imposed by earthquakes. Clean, loose, saturated, uniformly graded, fine sand below the water table is typically considered the most susceptible soil to liquefaction. Empirical evidence indicates that loose silty sand is also potentially liquefiable.

Seismically induced settlement can be generally subdivided into two categories for cohesionless (sand-like) soil: (1) settlement as a result of liquefaction of saturated or nearly saturated soil and (2) dynamic densification of non-saturated soil. Research has also shown that low-expansive cohesive (clay-like) soil can also undergo post-seismic settlement.

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting of the liquefied soil to the ground surface. Densification occurs as excess pore pressures dissipate, resulting in vertical settlement at the ground surface. In addition to the above analysis, we also evaluated the capping effect of any overlying non-liquefiable soil. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert sufficient force to break through the overlying soil and vent to the surface, resulting in sand boils or fissures.

Cohesive soil can also develop pore pressures during cyclic loading, but generally do not reach zero effective stress and are typically considered non-liquefiable (Robertson, 2009). However, cohesive soil can deform during cyclic earthquake loading and experience volumetric strains and post-earthquake reconsolidation. The volumetric strains for cohesive soil are generally small compared to cohesionless soil, since cohesive soil often retain some original soil structure.

We performed an analysis of liquefaction potential based on the CPT data using the computer software CLiq (Version 2.2.1.4) developed by GeoLogismiki to screen the soil profile for potentially liquefiable or cyclic softening soil. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Roberston (2009).

Preliminary screenings indicate that some portions of the silty clay materials, especially from approximately 8 to 15 feet bgs in the vicinity of 1-CPT1 and 1-CPT3, are susceptible to cyclic softening. To refine our liquefaction analysis, we utilized the screening criteria presented by Bray and Sancio (2006) to access the potential for liquefaction triggering of the fine-grained soil layers. Bray and Sancio observed that fine-grained soils with a plasticity index (PI) of greater than 18 and a water content to liquid limit ratio (w_c/LL) of less 0.8 were considered not susceptible to liquefaction. Based on the laboratory test results from the soil collected in 1-DP1, which was advanced as a matched-pair within close proximity to 1-CPT3, we determined that the soil deposits from approximately 13 to 15 feet bgs, which has a PI of 32, are not susceptible to liquefaction or cyclic softening. Our exploration and analyses indicate that the fine-grained soil between 8 and 13 feet bgs, which has a PI of 15, are transitional and may be susceptible to liquefaction. More rigorous and targeted design-level explorations should be conducted to evaluate the lateral extent of these transitional deposits.

We compared the calculated soil behavior type index (I_c) from the data of 1-CPT3 to the soil materials found in 1-DP1 at depth of 13 to 15 feet bgs, where laboratory testing and subsequent analysis indicates a low susceptible to liquefaction or cyclic softening. The I_c value of soil between 13 and 15 feet was determined to range between 2.4 and 2.6. From this comparison, we established that soil with an I_c greater than 2.4 have a low susceptibility of liquefaction. Due to the limited field exploration, we only applied our revised I_c cut-off value to the data of 1-CPT1 and 1-CPT3, which indicate similar soil deposits in the upper 15 feet. We recommend evaluating the applicability of the revised I_c cut-off value to the overall site during future design-level studies.

We performed our densification analysis using methods from Boulanger and Idriss (2014). The peak ground acceleration of 0.74g was taken from the mapped seismic design parameters for the site (Table 3.6-1). The value of predominant earthquake magnitude (6.9) was chosen based on the mean value of the deaggregation of the seismic hazard at the site. The groundwater was assumed to be at the historically high level at 8 feet below existing grade.

Our densification analysis indicates that seismically induced settlements of up to 1¼ inches may occur at the site. Additional field sampling, laboratory testing, and evaluation should be conducted during a design-level study to further characterize the soil layers with susceptibility to liquefaction and their lateral extent.

3.3.4 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sand or weak soil. Due to the proximity of the Coyote Creek channel and shallow depth of the soil susceptible to liquefaction, lateral spreading is a potential concern at the site. Additional evaluation should be conducted during a design-level study to further characterize potential impacts of lateral spreading.

3.4 FLOOD ZONE

The project site is mapped within Zone X on the Federal Emergency Management Agency (FEMA 2014) Flood Hazard Map for the City of San Jose, indicating that it is within an area determined to be outside the 0.2 percent annual chance floodplain. The project Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

3.5 CREEK SETBACK

As previously mentioned, Coyote Creek runs along the northeast of the project site. For preliminary planning purposes, we recommend maintaining a minimum setback of 50 feet from the top of the top of the slope to habitable structures. The final creek setback will be reviewed during the design-level geotechnical exploration. Additional slope stability analyses may be required if a closer setback is desired.

3.6 2019 CALIFORNIA BUILDING CODE SEISMIC DESIGN

We used the shear weave velocity measurements from two CPTs (1-CTP1 and 1-CPT3) to estimate the average shear wave velocity of the site soil. The CPT soundings were advanced to a maximum depth of 51 feet below ground surface before encountering refusal. We assume that shear wave velocity measurements will either increase or remain relatively constant when extrapolated to a depth of 100 feet below existing grade. Based on the shear weave velocity measurements from the CPTs, we estimate an average shear wave velocity between 230 and 360 meters per second, which are corresponding to Site Class D.

We provide the 2019 California Building Code (CBC) seismic parameters in Table 3.6-1 below.

TABLE 3.6-1: 2019 CBC Seismic Design Parameters
Latitude: 37.396461, Longitude: -121.916785

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.60
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.61
Site Coefficient, F_A	1.00
Site Coefficient, F_V	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.60
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.07
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.68
Site Coefficient, F_{PGA}	1.10
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.74

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

We estimate the fundamental period of the proposed structures may be less than 1.5 Ts. Therefore, the structural engineer may consider exception of Section 11.4.8 of ASCE 7-16 as follows.

> "A ground motion hazard analysis is not required for structures… where, structures on Site Class D sites with S1 greater than or equal to 0.2, provided the value of the seismic response coefficient Cs is determined by Eq. (12.8-2) of ASCE 7-16 for values of $T \le 1.5 TS$ and taken as equal to 1.5 times the value computed in accordance with Eq. (12.8-3) of ASCE 7-16 for $1.5Ts < T \le TL$."

If the noted exception is not used, a ground motion hazard analysis can be provided upon request under separate cover.

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

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

4.1 GENERAL DEMOLITION AND SITE CLEARING

Grading should begin with the removal of existing structures including their foundations. Underground structures such as buried pipes, septic tanks and leach fields, if any, which will be abandoned or are expected to deteriorate, should be removed from the project site entirely.

Based on our Phase I environmental site assessment, an underground storage tank (UST) and one water supply well were observed during the site reconnaissance and five water supply wells were identified during agency file review. These underground features should be properly removed in accordance with local and state regulations.

All existing unengineered fill, soft or compressible soil in areas to be graded should be removed as necessary for project requirements. The Geotechnical Engineer's representative should determine the depth of removal of these materials at the time of grading.

Areas containing surface vegetation or organic laden topsoil within the areas to be improved should be stripped to an appropriate depth to remove these materials. Tree roots should be removed to a depth of at least 3 feet below finished grade. The amount of actual stripping and depth of tree root removal should be determined in the field by the Geotechnical Engineer at the time of construction. Subject to approval by the Landscape Architect, strippings and organically contaminated soil can be used in landscape areas. Otherwise, such soil should be removed from the project site. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations. It may be feasible to cut and remove the existing vegetation as close to the ground surface as possible and then disk the remaining vegetation into the near-surface soil. This should be evaluated by ENGEO just prior to the commencement of grading.

Excavations resulting from demolition and stripping which extend below final grades should be cleaned to firm undisturbed soil as determined by the Geotechnical Engineer's representative. Once the surface of areas to be graded are prepared as discussed above, the surface should then be scarified, moisture conditioned, and backfilled with suitable material compacted to the recommendations presented in the Fill Placement section.

4.2 SELECTION OF MATERIALS

Re-use of any potentially contaminated material at the site should be evaluated by our project geotechnical and environmental consultant team. Any soil that is determined to be not contaminated and contains less than 3 percent organics is suitable for use as engineered fill. The acceptable fill materials should be determined and confirmed by the Geotechnical Engineer's representative in the field.

Imported fill materials should meet the above requirements and with a Plasticity Index of less than 12. ENGEO should sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

4.3 FILL PLACEMENT

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

- Test Procedures: ASTM D1557
- Required Moisture Content: Not less than 3 percentage points above optimum moisture content
- Minimum Relative Compaction: 90 percent

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material as determined by ASTM D1557.

Deeper fill and retaining wall backfill may have modified compaction control requirements. These additional requirements will be developed during our design-level exploration, as necessary.

4.4 SITE DRAINAGE

4.4.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.3 specifies minimum slopes of 5 percent away from foundations within 10 feet. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following:

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.
3. Do not allow water to pond near foundations, pavements, or exterior flatwork.

4.5 STORMWATER BIORETENTION AREAS

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

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

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

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area

excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

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

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

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

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

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

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

Primary considerations in foundation design for this project include static consolidation settlement, seismically induced settlement, and low bearing capacity of near-surface soil. The preliminary screenings indicated that some portions of the silty clay materials are susceptible to cyclic softening, resulting in total seismically induced settlements of up to 1¼ inches and differential settlements of ¾ inch over a lateral distance of 50 feet. In addition, the consolidation settlement estimates are provided in following sections.

The following preliminary foundation recommendations and estimated settlement, static and seismic, will be refined during the detailed geotechnical exploration.

5.1 STRUCTURAL MAT FOUNDATIONS

Structural mat foundations are a robust, economical option for support of mid-rise structures. A well-designed structural mat can tolerate significant post-construction movement and exerts relatively low pressures on the subgrade soil. The thickness of the mat foundation will be driven by the structural design.

We recommend a preliminary allowable bearing pressure of 1,500 pounds per square foot (psf) for a structural mat, which may result in overall consolidation settlements of up to 1½ inches and corresponding differential settlements of less than ¾ inch over a lateral distance of 50 feet.

5.2 SPREAD FOOTINGS

Spread footings with slabs-on-grade floor may be used to support the proposed structures. Continuous or isolated spread footings should bear in competent native soil or compacted fill. Near-surface tip resistance values in 1-CPT2 and 1-CPT6, located along the southwestern edge of the site, were found to be significantly lower than those encountered elsewhere. Limited ground improvement may be required on the near-surface soil if continuous or isolated footings are used. Further evaluation is required to more precisely delineate the areas requiring ground improvement. We recommend a preliminary allowable bearing pressure of 2,000 pounds per square foot for continuous or isolated footings, which may result in overall consolidation settlements of up to 1 inch and corresponding differential settlements of less than ½ inch over a lateral distance of 50 feet.

5.3 SLAB MOISTURE VAPOR REDUCTION

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

1. Install a vapor retarder membrane directly beneath the mat or slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E 1745 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.”
2. Concrete should have a concrete water-cement ratio of no more than 0.5.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water-cement ratio are used.
4. Consider and implement adequate moist cure procedures for mat foundations.
5. Protect foundation subgrade soil from seepage by providing impermeable plugs within utility trenches.

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

6.0 PRELIMINARY RETAINING WALL RECOMMENDATIONS

For preliminary purposes, unrestrained drained site retaining walls constructed on level ground may be designed using an active equivalent fluid weight of 50 pounds per cubic foot (pcf). The friction factor for sliding resistance may be assumed as 0.30.

Drainage facilities should be installed behind retaining walls to prevent the build-up of hydrostatic pressures on the walls. For planning purposes, wall drainage may be provided using 4-inch-diameter perforated (SDR 35 or approved equivalent) pipe encapsulated in either Class 2 permeable material, or a free-draining gravel surrounded by synthetic filter fabric (minimum 6-ounce). The width of the drainage medium should be at least 12 inches and should extend from base of the wall to about 1 foot below the finished soil subgrade. The upper 1 foot of wall backfill should consist of on-site compacted soil. If pre-fabricated drain panels are to be considered in lieu of the drainage medium above the pipe/rock bulb, the contractor should submit their materials packet for our review prior to order and delivery.

The Geotechnical Engineer should be consulted on wall design values where surcharge loads, such as from permanent structures and automobiles, are expected or where slopes exist above or below a proposed wall. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations. A specialty consultant should be consulted regarding retaining wall waterproofing.

7.0 PRELIMINARY PAVEMENT DESIGN

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

TABLE 7.0-1: Preliminary Pavement Sections

TRAFFIC INDEX	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	4	8
6	4½	10½
7	5	14

Notes: AC is asphaltic concrete
AB is aggregate base Class 2 Material with minimum R = 78

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

7.1 CUT-OFF CURBS

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

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

8.0 FUTURE STUDIES

As previously discussed, a site-specific design level geotechnical exploration should be performed as part of the design process. The exploration should include supplemental borings, soil sampling to determine the extent of non-engineered fill, and subsequent laboratory testing to provide additional data for evaluation of liquefaction susceptibility, test soil strength for foundation design, and consolidation tests to evaluate settlement susceptibility. The design-level report will also provide specific recommendations regarding grading, foundation design, retaining wall design, and drainage for the proposed development.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents preliminary geotechnical recommendations for design of the improvements discussed in Section 1.3 for the proposed Hanover North San Jose development. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and preliminary recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The preliminary conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance. We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks 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, 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, then 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.

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FIGURES

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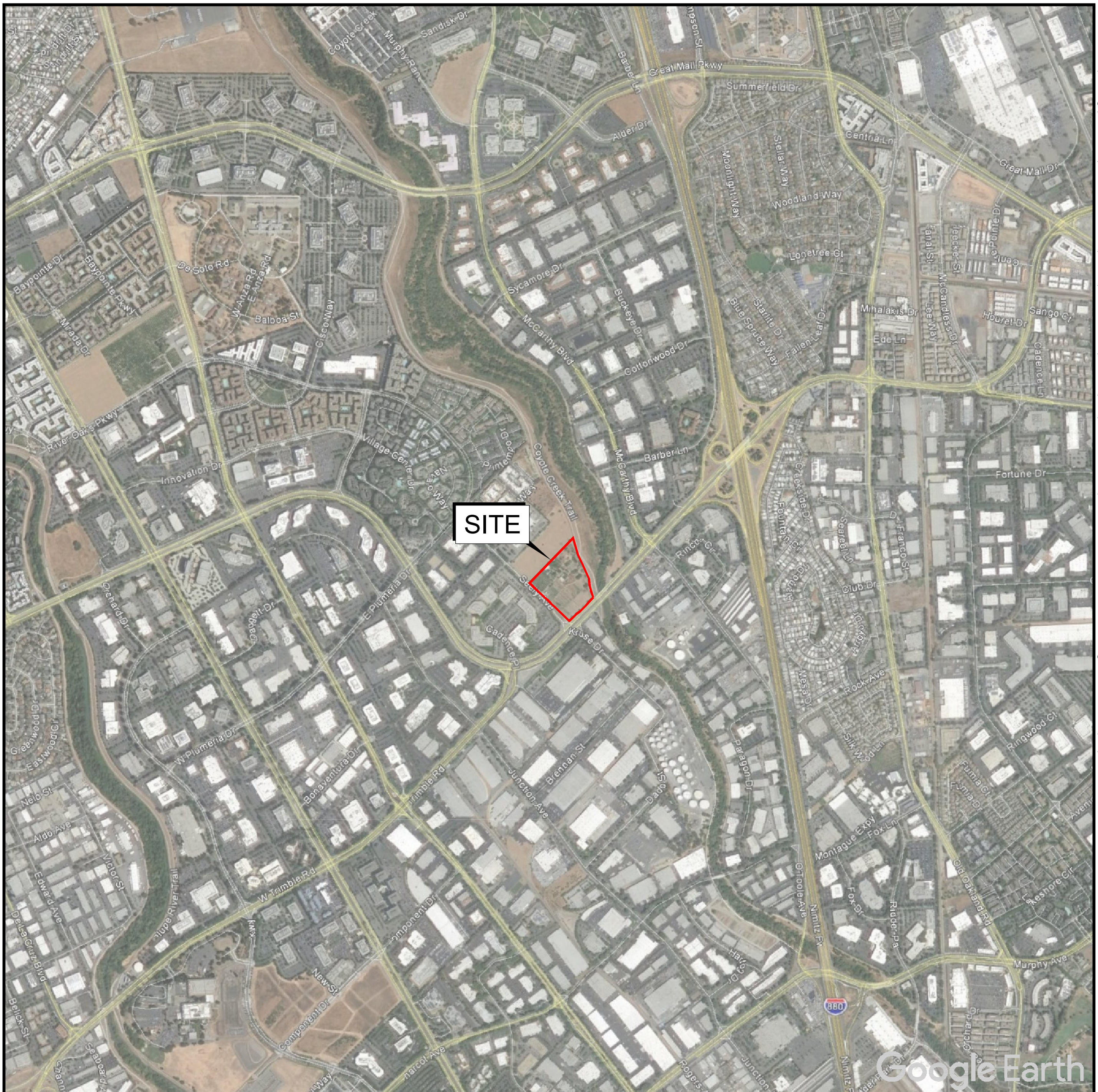

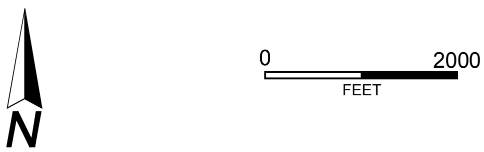

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP HANOVER NORTH SAN JOSE SAN JOSE, CALIFORNIA	PROJECT NO.: 18233.000.001 SCALE: AS SHOWN DRAWN BY: CC CHECKED BY: JFB	FIGURE NO. 1

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-DP1 DIRECT PUSH SAMPLER (ENGEO, 2021)

1-CPT6 CPT LOCATION (ENGEO, 2021)

S-4 SOIL SAMPLE LOCATION (ENGEO, 2021)

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FEET

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence

SITE PLAN
HANOVER NORTH SAN JOSE
SAN JOSE, CALIFORNIA

PROJECT NO.: 18233.000.001
SCALE: AS SHOWN
DRAWN BY: CC
CHECKED BY: JFB
FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR

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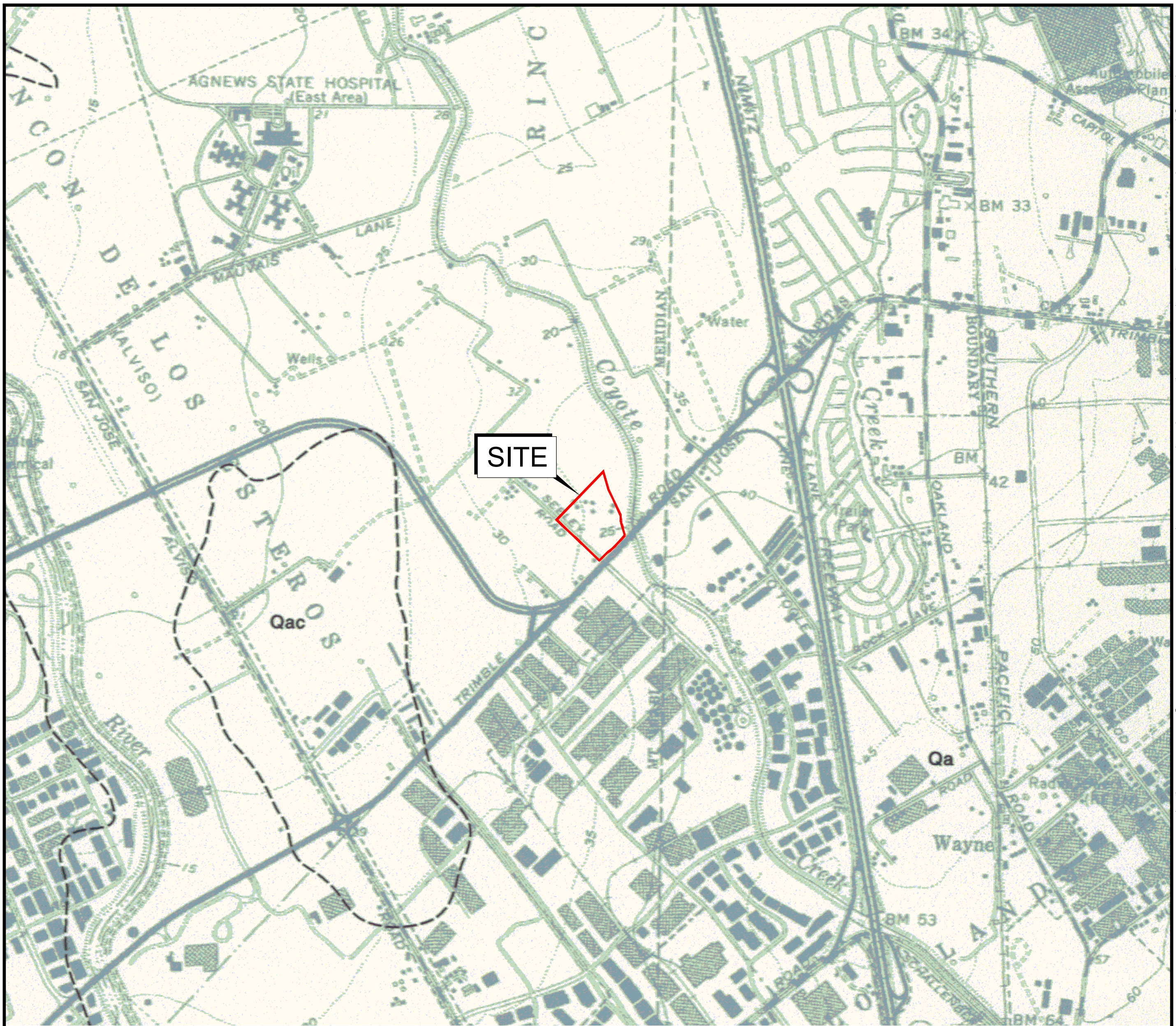

EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT

DIRECTION OF LANDSLIDE MOVEMENT

AXIS OF FOLD

ANTICLINE SYNCLINE

STRIKE AND DIP OF STRATA

INCLINED VERTICAL OVERTURNED

Qa ALLUVIUM

Qac ALLUVIAL CLAY SOIL, INCLUDES BAY MUD

N

0 2000
FEET

BASE MAP SOURCE: DIBLEE, 2005

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
HANOVER NORTH SAN JOSE
SAN JOSE, CALIFORNIA

PROJECT NO.: 18233.000.001	FIGURE NO. 3
SCALE: AS SHOWN	
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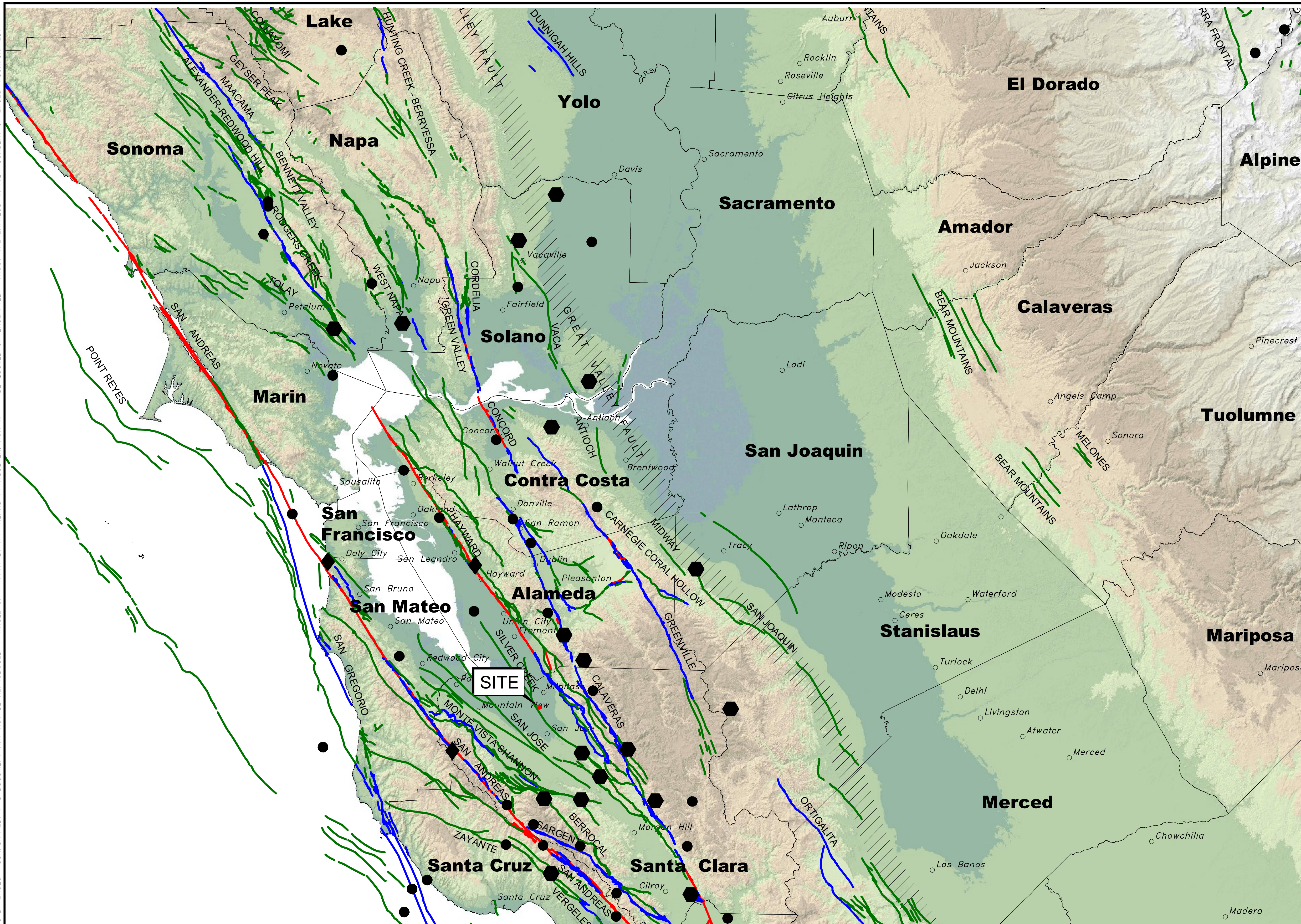

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0 MILES 15
0 KILOMETERS 30

EXPLANATION

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

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

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HANOVER NORTH SAN JOSE
SAN JOSE, CALIFORNIA

PROJECT NO.: 18233.000.001		FIGURE NO. 4
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EXPLANATION

LIQUEFACTION

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

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

ENGEO Expect Excellence	SEISMIC HAZARD ZONES MAP HANOVER NORTH SAN JOSE SAN JOSE, CALIFORNIA	PROJECT NO.: 18233.000.001 SCALE: AS SHOWN DRAWN BY: CC — CHECKED BY: JFB	FIGURE NO. 5

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

Cone Penetration Test Logs

PRESENTATION OF SITE INVESTIGATION RESULTS

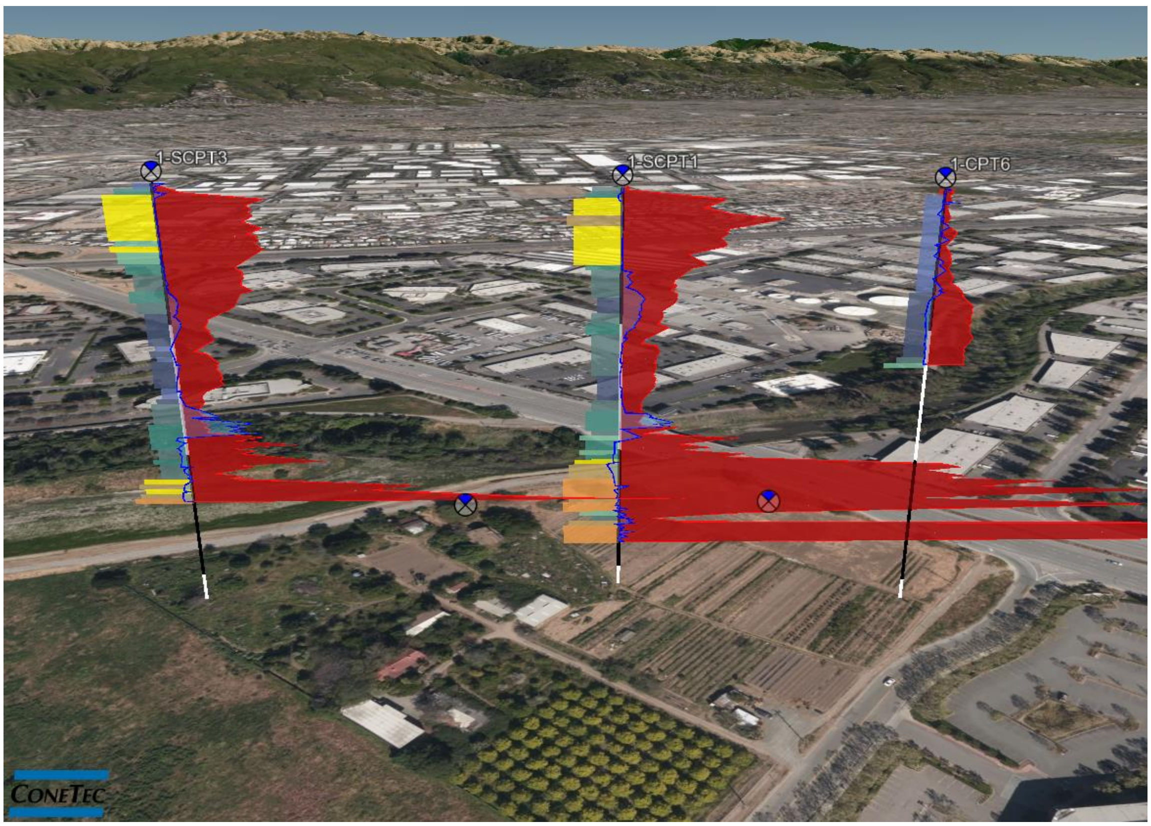
Hanover North San Jose

Prepared for:

ENGEO Incorporated

ConeTec Job No: 21-56-21887

Project Start Date: 29-Jan-2021
Project End Date: 29-Jan-2021
Report Date: 01-Feb-2021



Prepared by:

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Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for ENGEO Incorporated of San Ramon, California. The program consisted of cone penetration testing (CPTu) at six (6) locations. Shear wave velocities were recorded in two (2) soundings. The assumed phreatic surface used for the calculated parameters is based on the shallowest pore pressure dissipation test to reach equilibrium within or nearest to each sounding.

Project Information

Project	
Client	ENGEO Incorporated
Project	Hanover North San Jose
ConeTec Project #	21-56-21887

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

Rig Description	Deployment System	Test Type
CPT track rig (GPT2)	20-ton track mounted cylinder	CPTu/SCPTu

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

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

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

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

Limitations

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

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

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

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

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 cm^2, 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross-sectional area (typically forty-four millimeter diameter over a length of thirty-two millimeter with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

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

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

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

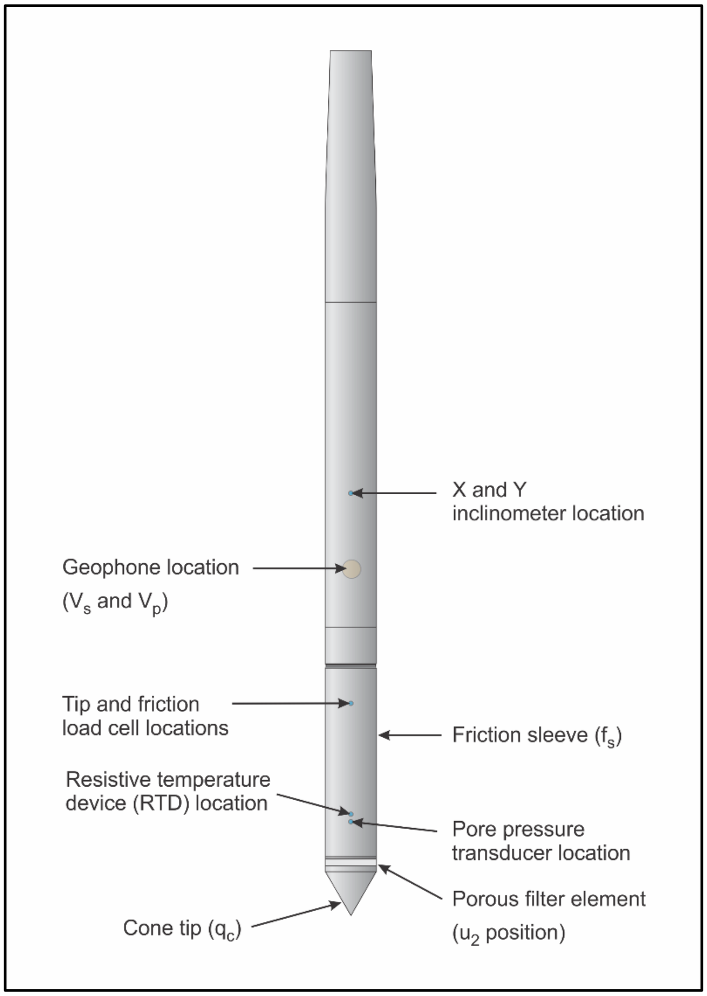

Figure CPTu. Piezocone Penetrometer (15 cm^2)

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

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

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

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

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically, one-meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

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

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

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

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

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

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

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

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

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

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

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

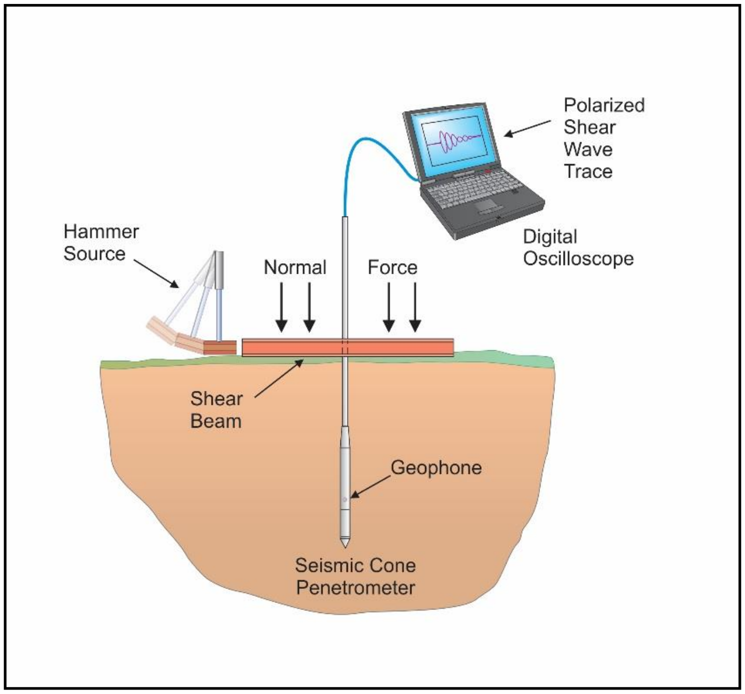

Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 and ASTM D7400 standards.

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

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

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

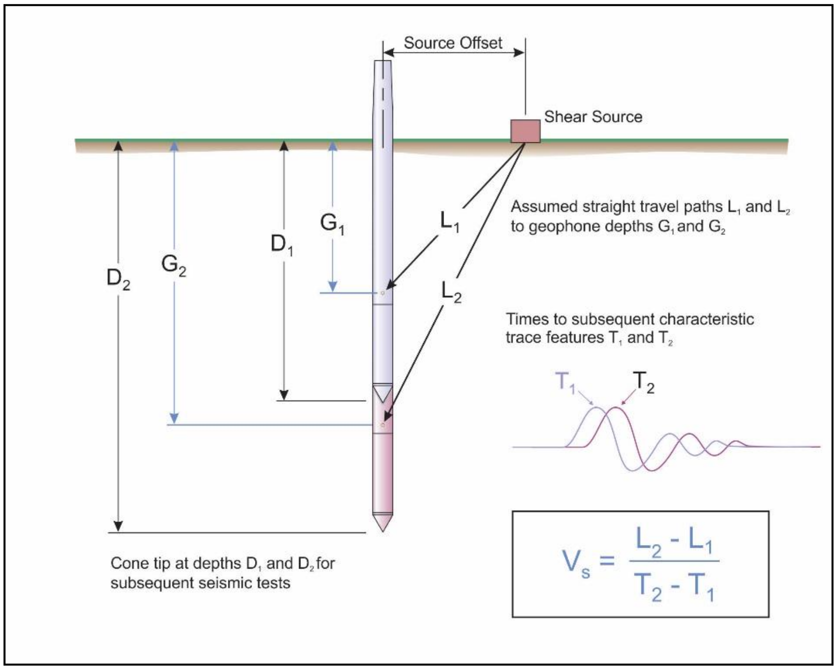

Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)

d_i = the thickness of any layer between 0 and 100 ft (30 m)

v_{si} = the shear wave velocity in ft/s (m/s)

$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

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

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

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

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

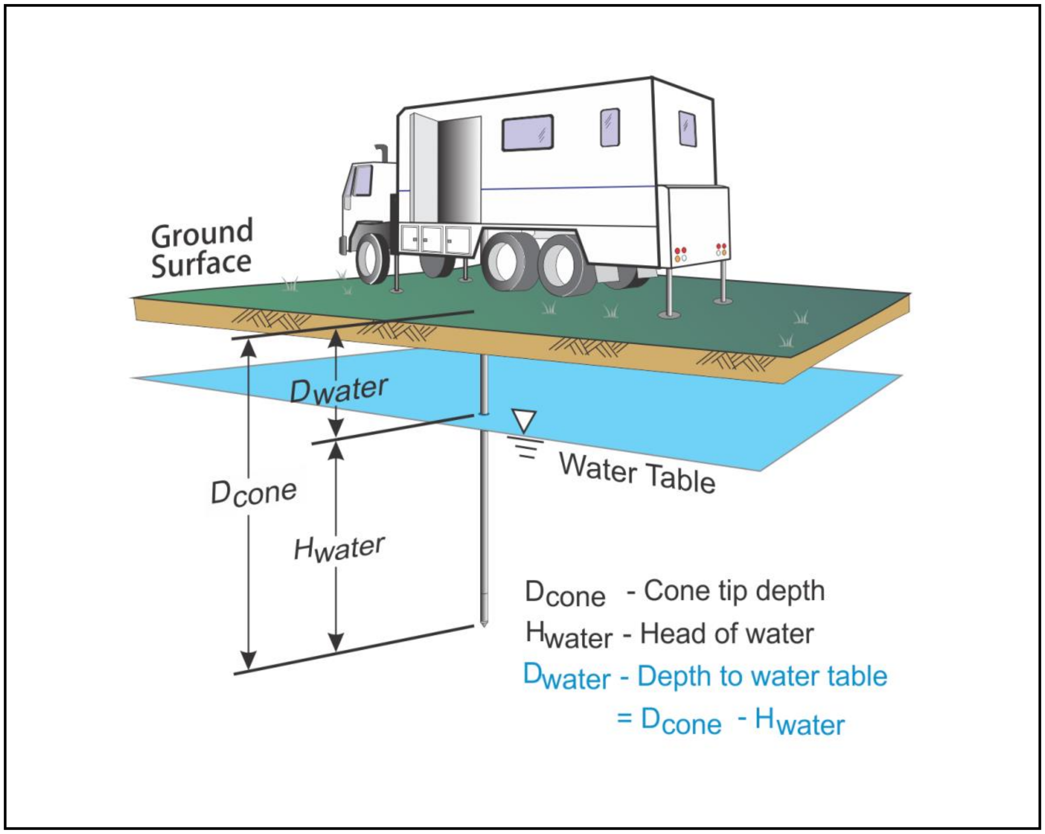

Figure PPD-1. Pore pressure dissipation test setup

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

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

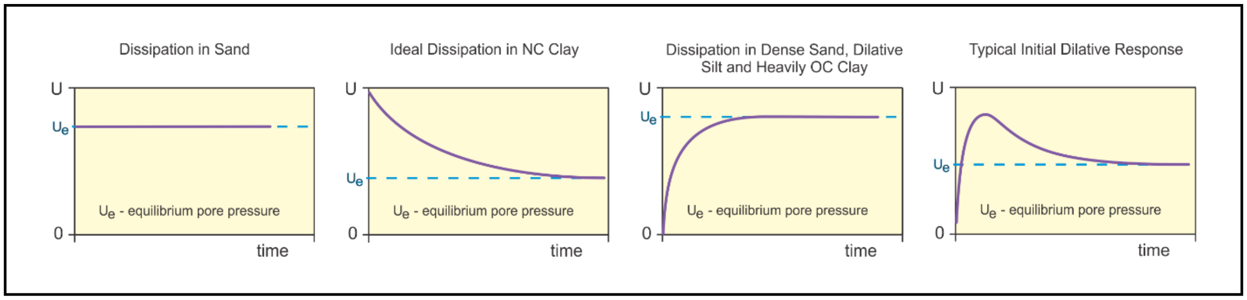

Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

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

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

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

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

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

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

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

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

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

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APPENDICES

The following appendices listed below are included in the report:

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

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No:	21-56-21887
Client:	ENGEO Incorporated
Project:	Hanover North San Jose
Start Date:	29-Jan-2021
End Date:	29-Jan-2021

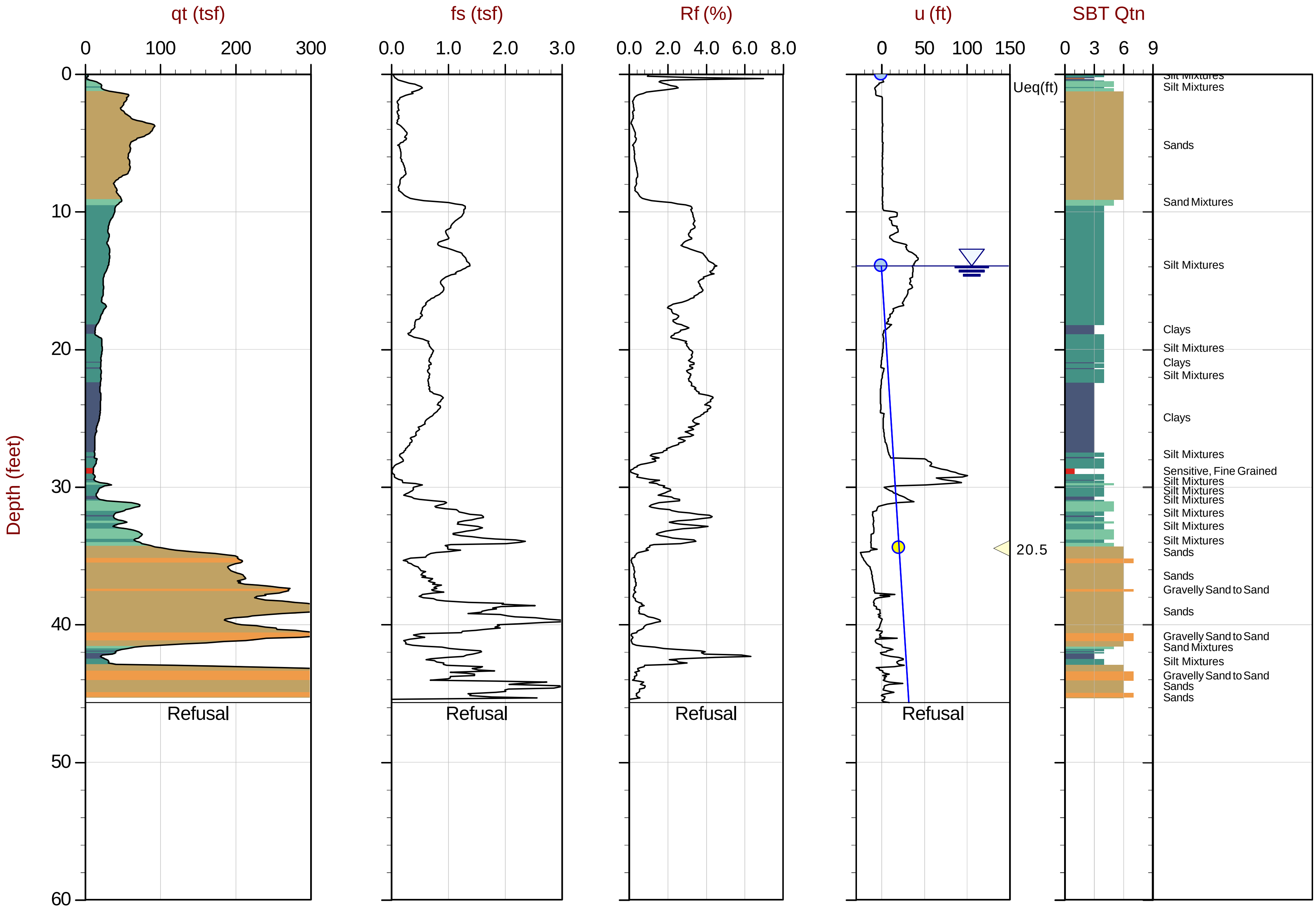
CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting[2] (m)	Elevation[3] (ft)	Refer to Notation Number
1-SCPT1	21-56-21887_SP01	29-Jan-2021	499:T1500F15U1K	13.9	45.69	4139409	595869	37	
1-CPT2	21-56-21887_CP02	29-Jan-2021	499:T1500F15U1K	13.5	20.51	4139425	595737	35	4
1-SCPT3	21-56-21887_SP03	29-Jan-2021	499:T1500F15U1K	13.9	39.04	4139556	595865	38	
1-CPT4	21-56-21887_CP04	29-Jan-2021	499:T1500F15U1K	13.5	20.51	4139468	595928	41	4,5
1-CPT5	21-56-21887_CP05	29-Jan-2021	499:T1500F15U1K	13.2	50.52	4139340	595926	39	
1-CPT6	21-56-21887_CP06	29-Jan-2021	499:T1500F15U1K	13.5	20.51	4139310	595853	37	4

1. The assumed phreatic surface was based on the shallowest pore pressure dissipation tests performed within the sounding. Hydrostatic conditions are assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10S.
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.
4. The assumed phreatic surface is based on the pore pressure dissipation at nearby soundings.
5. The cone malfunctioned on the sounding, which resulted in low to no sleeve friction values. The sleeve friction data was not reported for the sounding.

Job No: 21-56-21887
Date: 2021-01-29 14:28
Site: Hanover North San Jose
Sounding: 1-SCPT1
Cone: 499:T1500F15U1K

ENGEO



Max Depth: 13.925 m / 45.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139409m E: 595869m

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

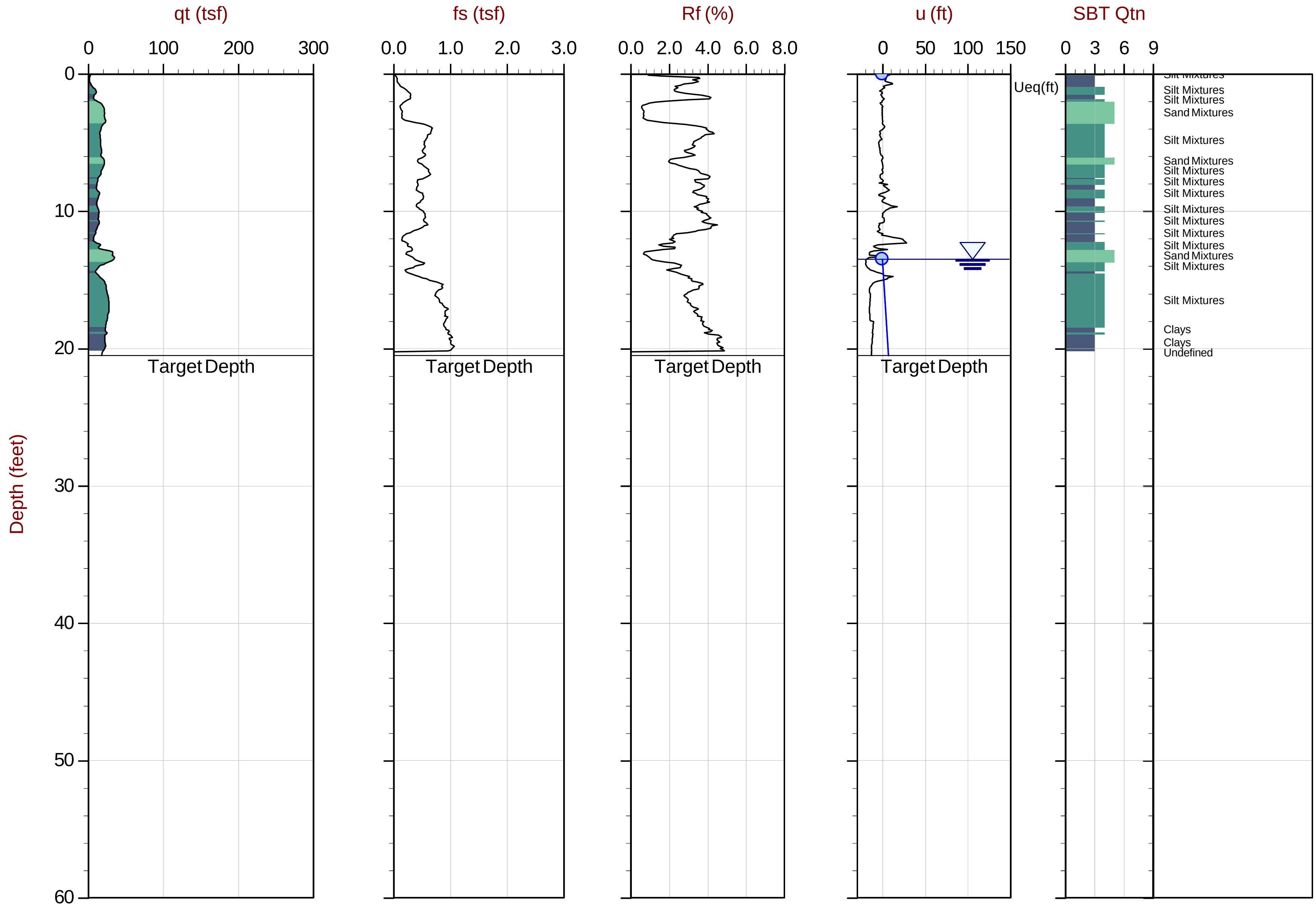
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 19:03
Site: Hanover North San Jose

Sounding: 1-CPT2
Cone: 499:T1500F15U1K



Max Depth: 6.250 m / 20.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139425m E: 595737m

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

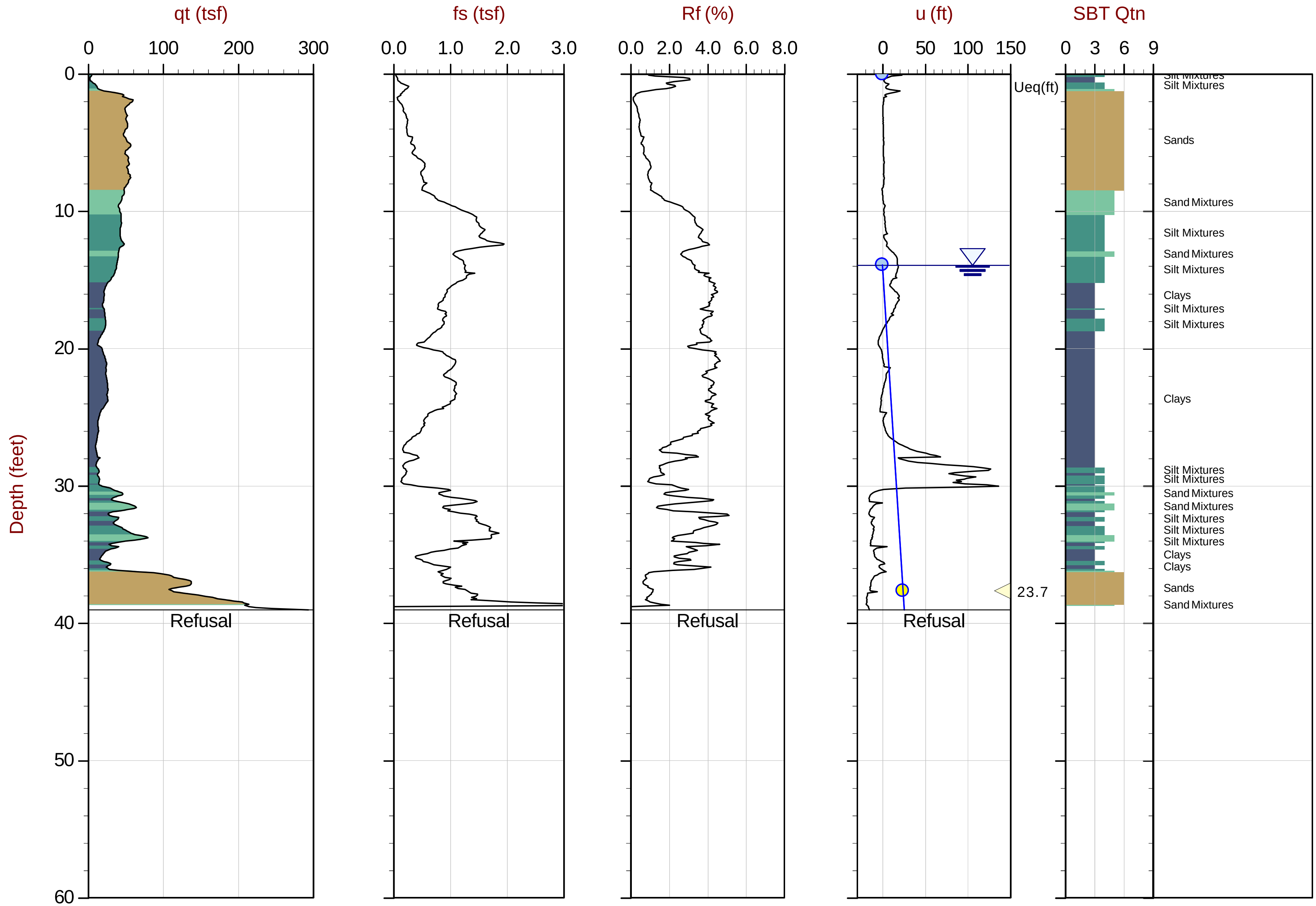
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 16:15
Site: Hanover North San Jose

Sounding: 1-SCPT3
Cone: 499:T1500F15U1K



Max Depth: 11.900 m / 39.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139556m E: 595865m

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

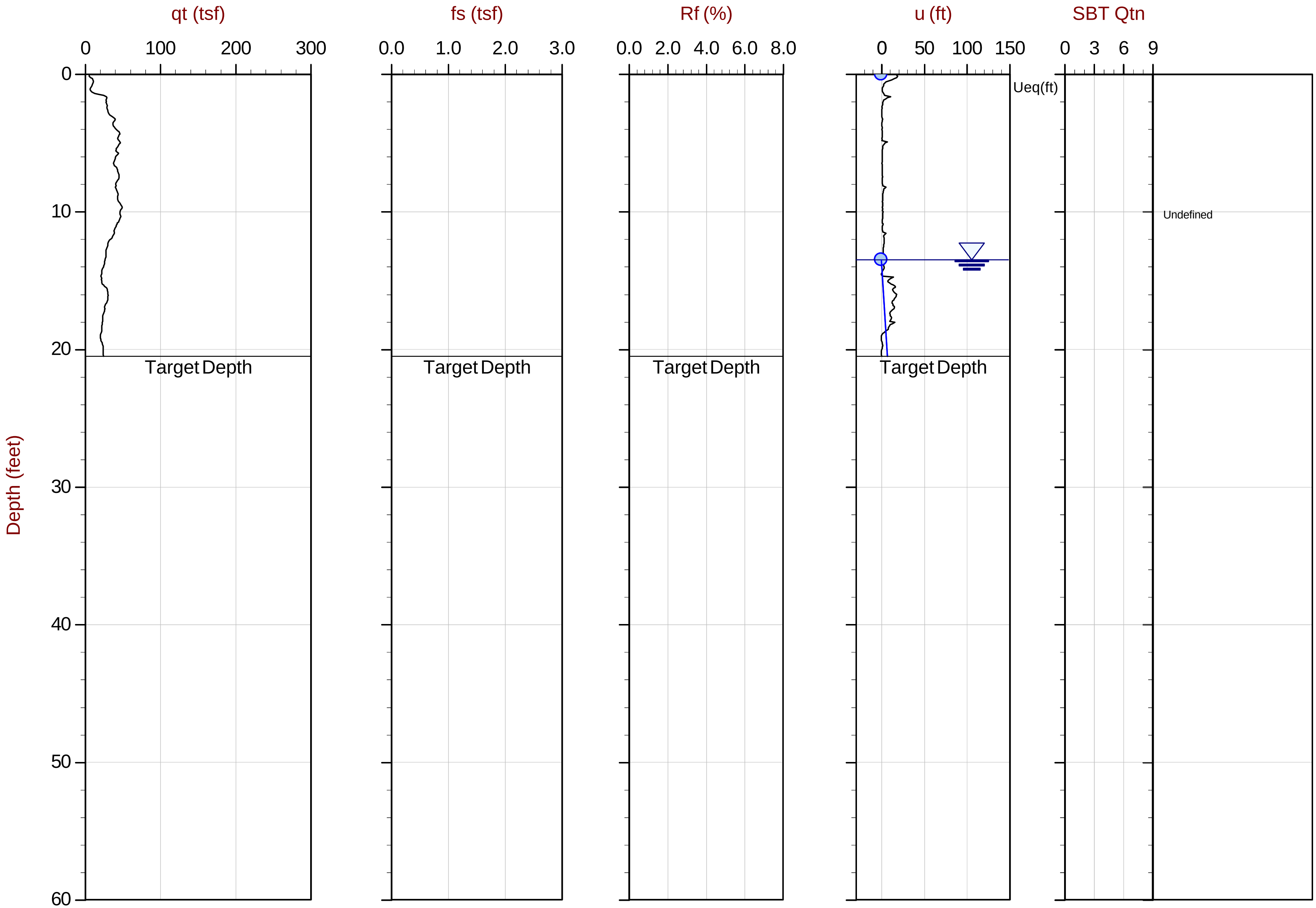
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 17:31
Site: Hanover North San Jose

Sounding: 1-CPT4
Cone: 499:T1500F15U1K



Max Depth: 6.250 m / 20.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139468m E: 595928m

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

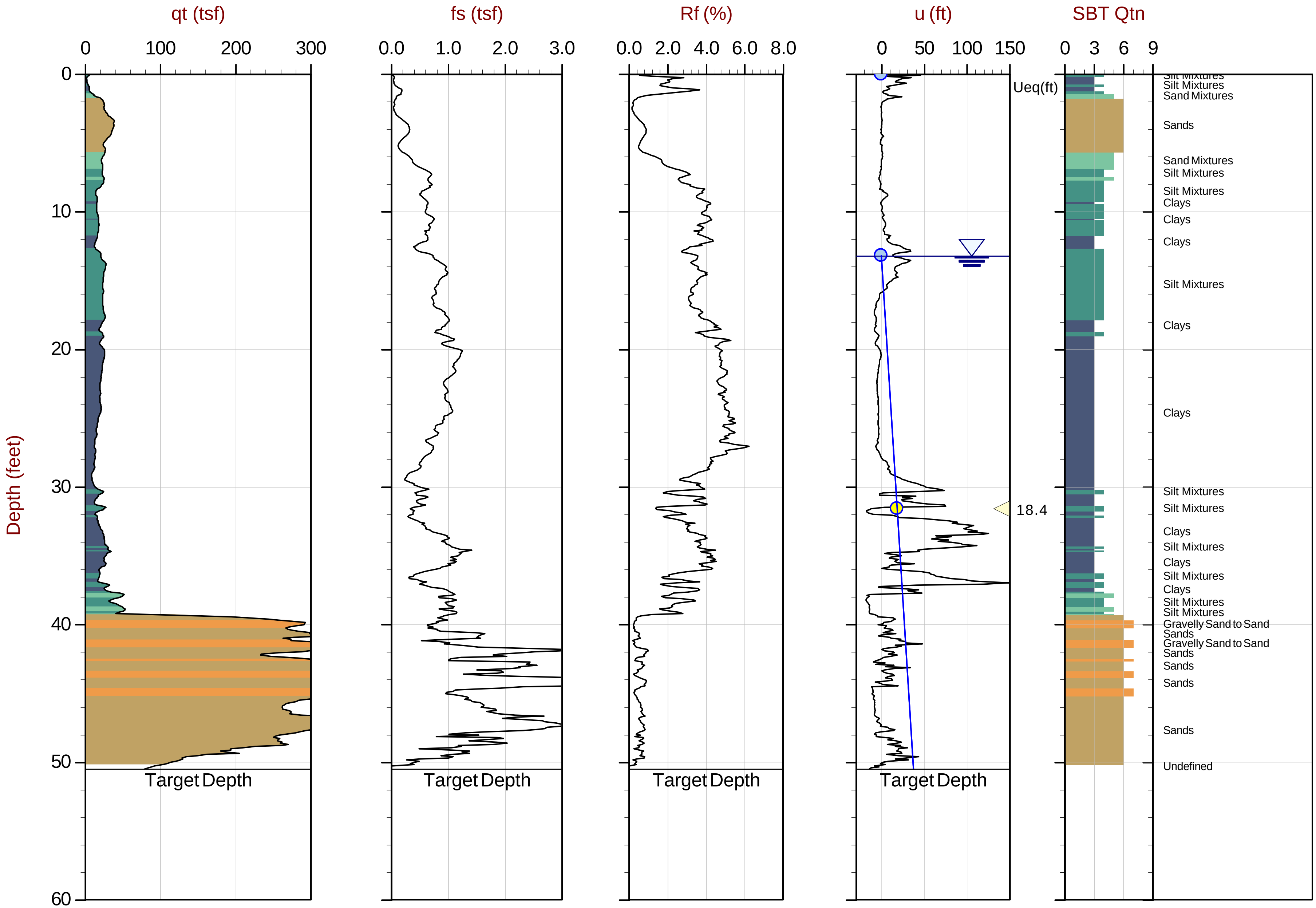
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 12:26
Site: Hanover North San Jose

Sounding: 1-CPT5
Cone: 499:T1500F15U1K



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139340m E: 595926m

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

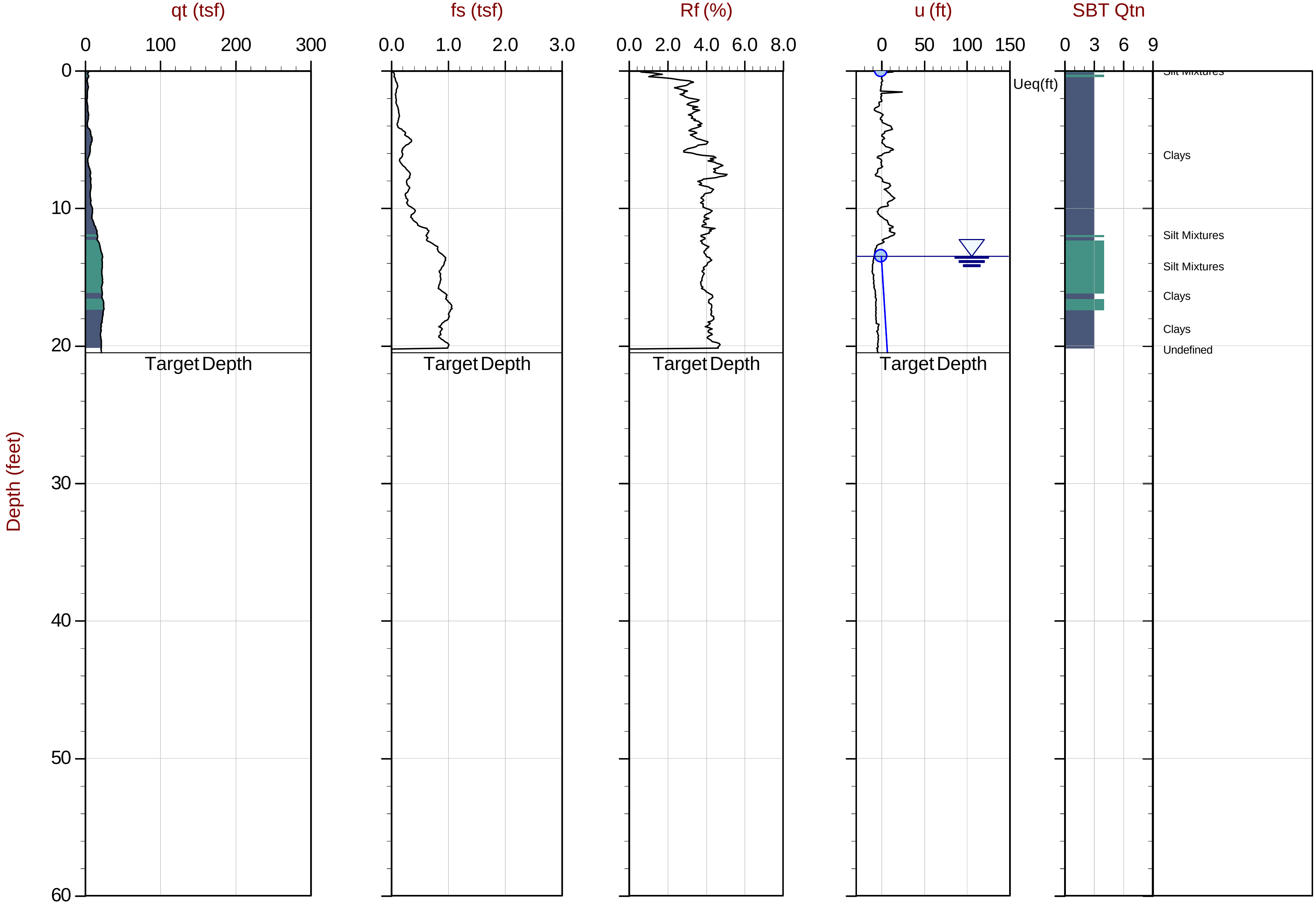
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 18:19
Site: Hanover North San Jose

Sounding: 1-CPT6
Cone: 499:T1500F15U1K



Max Depth: 6.250 m / 20.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139310m E: 595853m

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

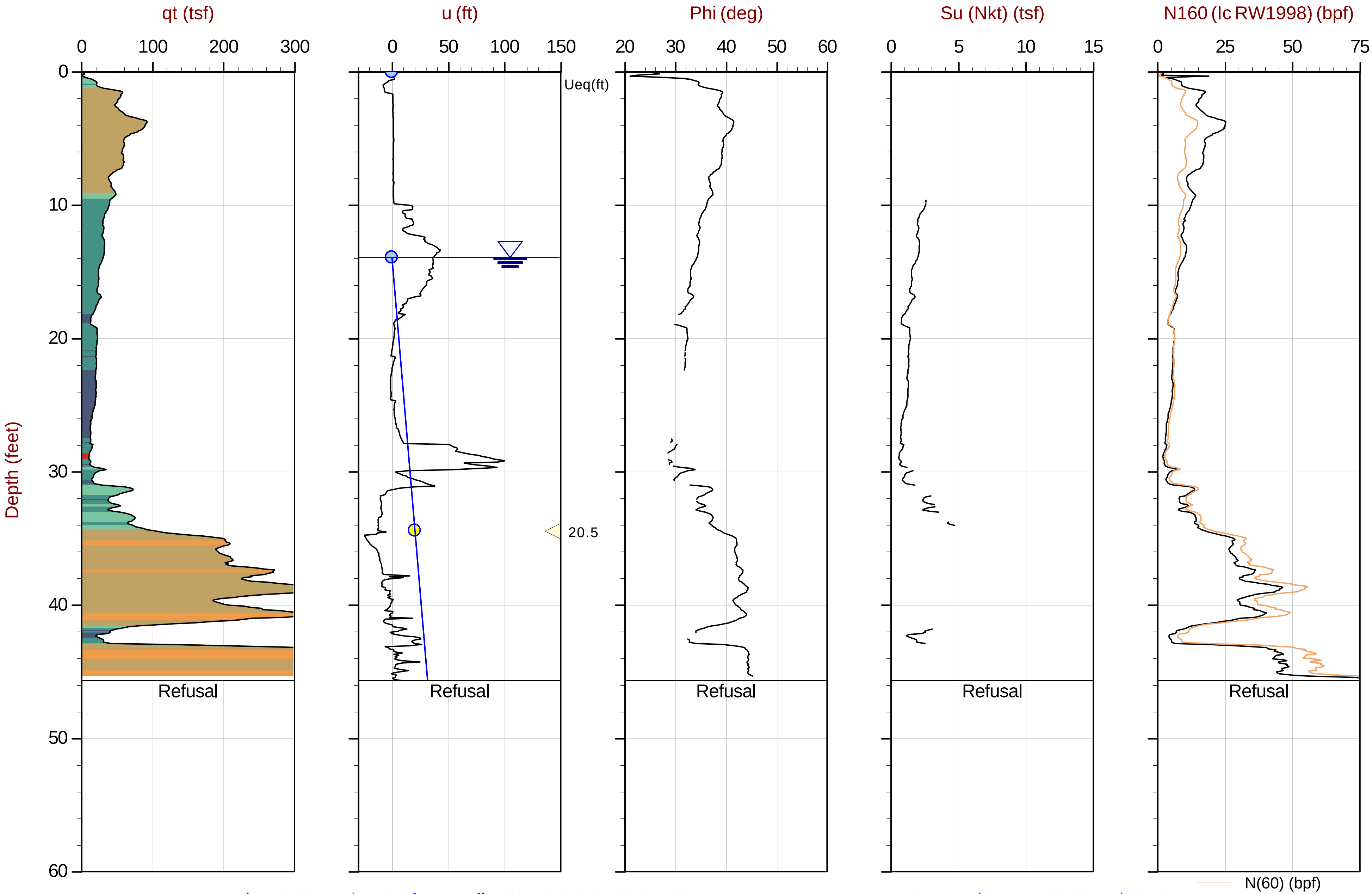
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 14:28
Site: Hanover North San Jose

Sounding: 1-SCPT1
Cone: 499:T1500F15U1K



Max Depth: 13.925 m / 45.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139409m E: 595869m

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

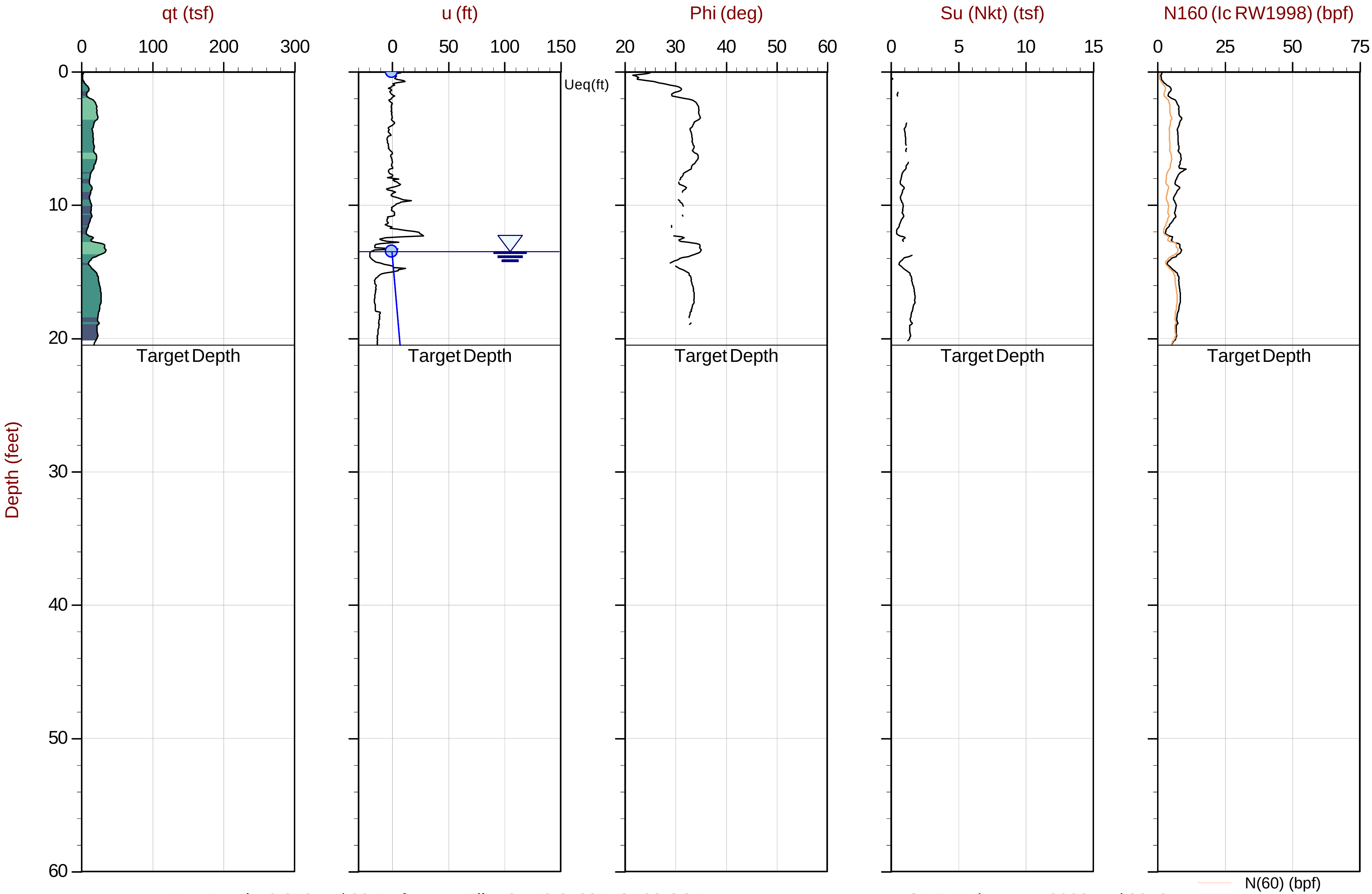
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 19:03
Site: Hanover North San Jose

Sounding: 1-CPT2
Cone: 499:T1500F15U1K



Max Depth: 6.250 m / 20.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139425m E: 595737m

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

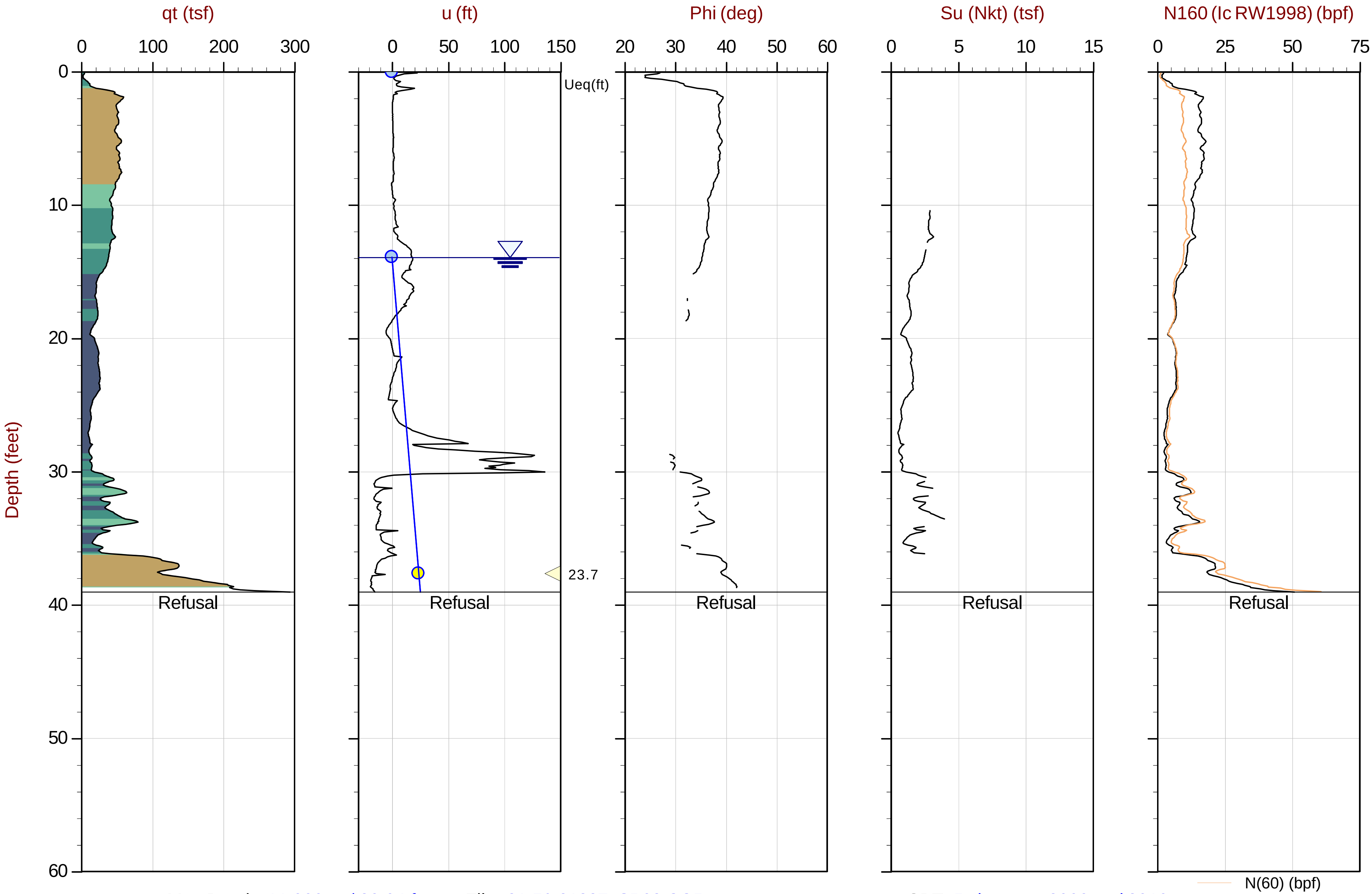
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 16:15
Site: Hanover North San Jose

Sounding: 1-SCPT3
Cone: 499:T1500F15U1K



Max Depth: 11.900 m / 39.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_SP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139556m E: 595865m

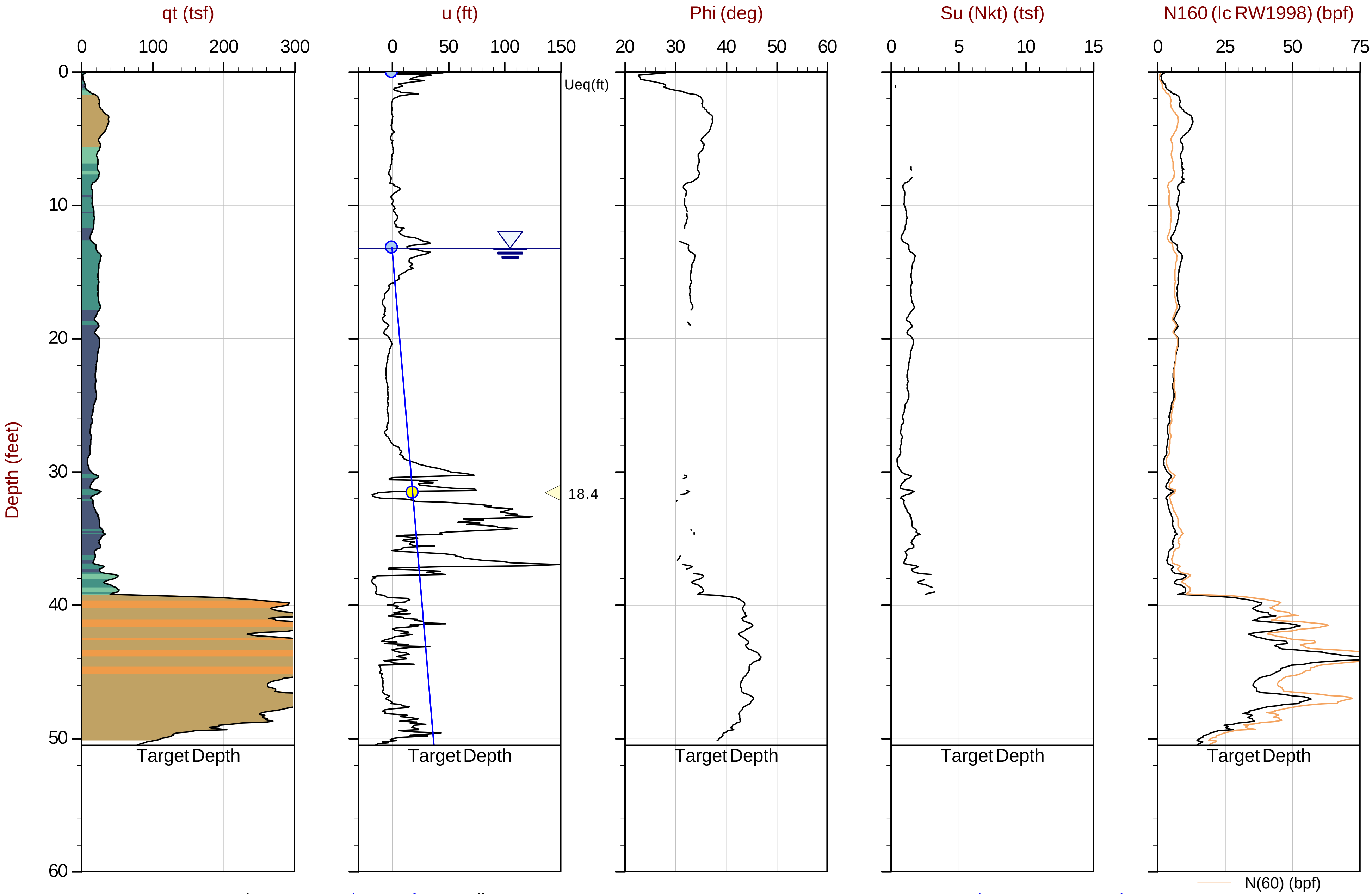
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 12:26
Site: Hanover North San Jose

Sounding: 1-CPT5
Cone: 499:T1500F15U1K



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139340m E: 595926m

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

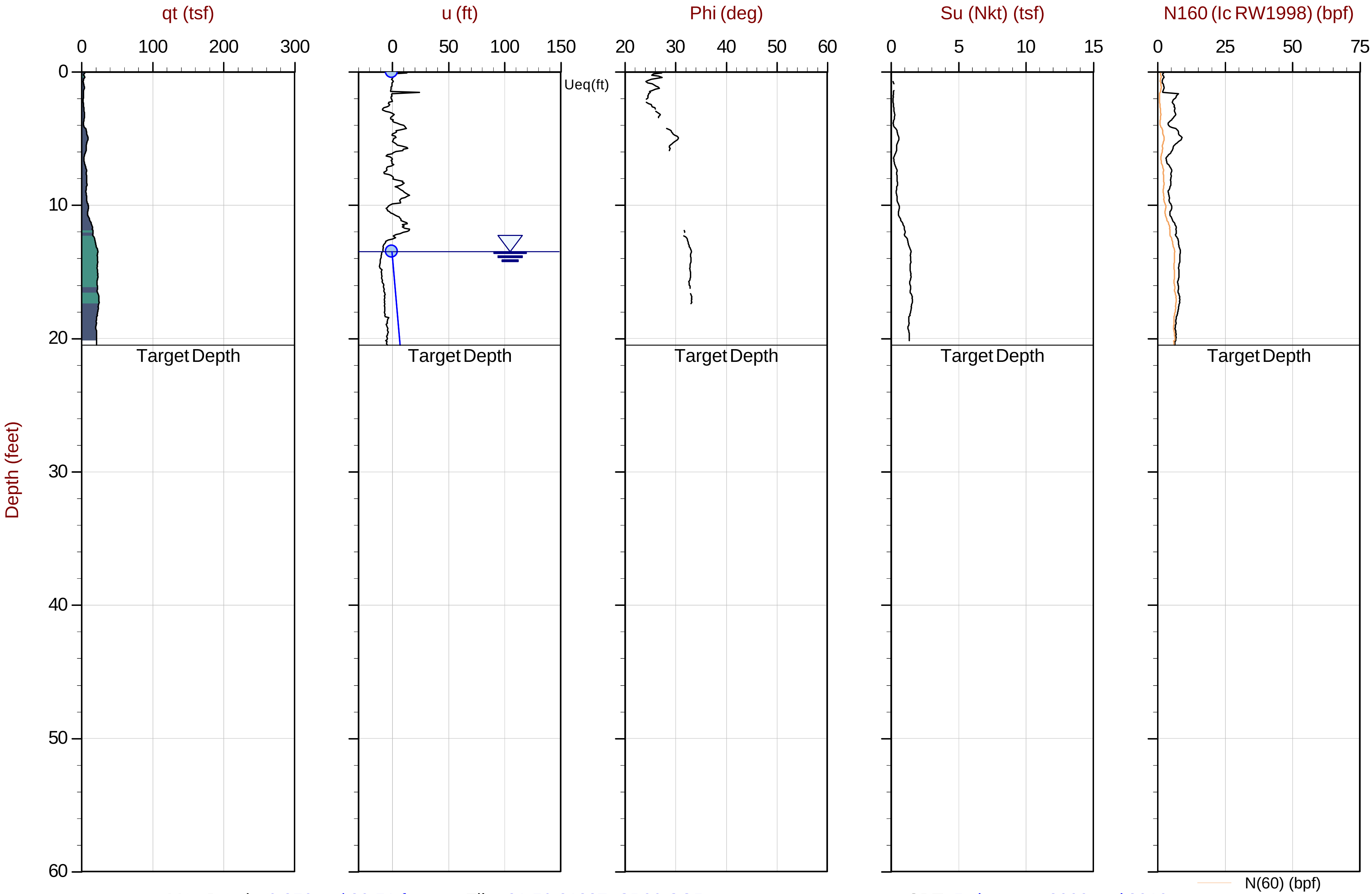
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 18:19
Site: Hanover North San Jose

Sounding: 1-CPT6
Cone: 499:T1500F15U1K



Max Depth: 6.250 m / 20.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139310m E: 595853m

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

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

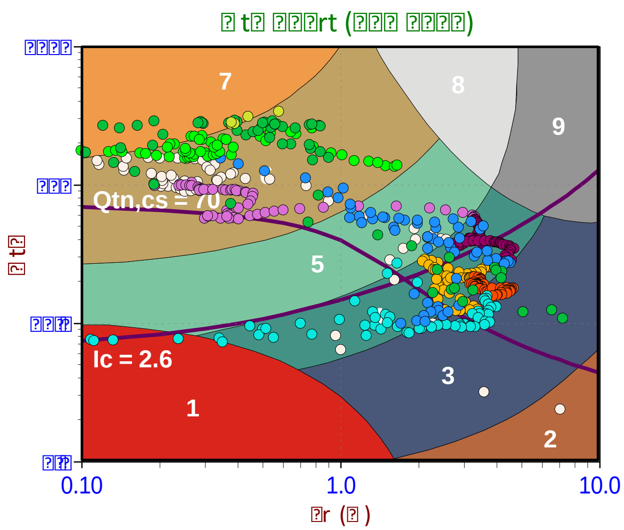
Soil Behavior Type (SBT) Scatter Plots

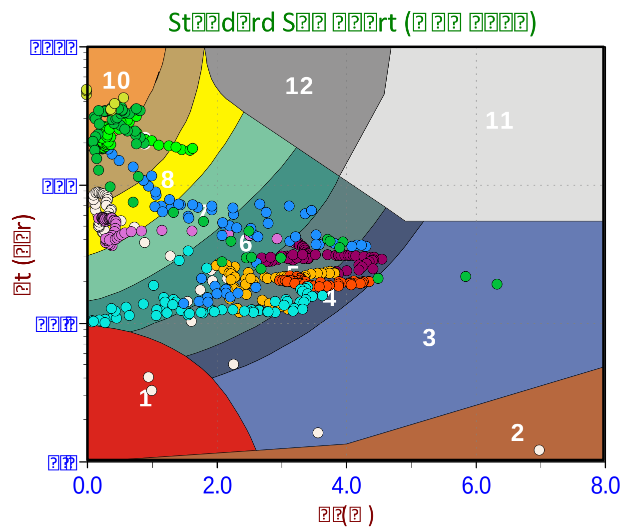
ConeTec

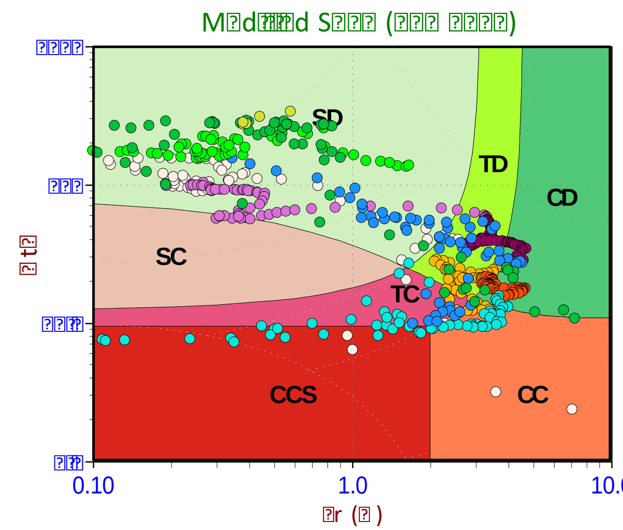
ENGEO

Job No: 21-56-21887
Date: 2021-01-29 14:28
Site: Hanover North San Jose

Sounding: 1-SCPT1
Cone: 499:T1500F15U1K







Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

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

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

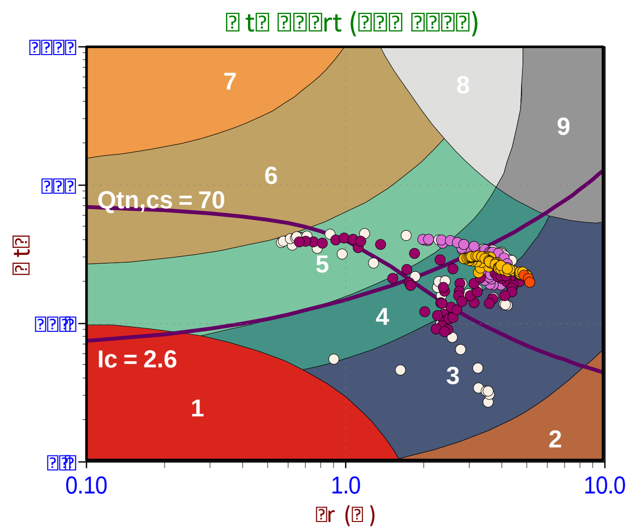
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

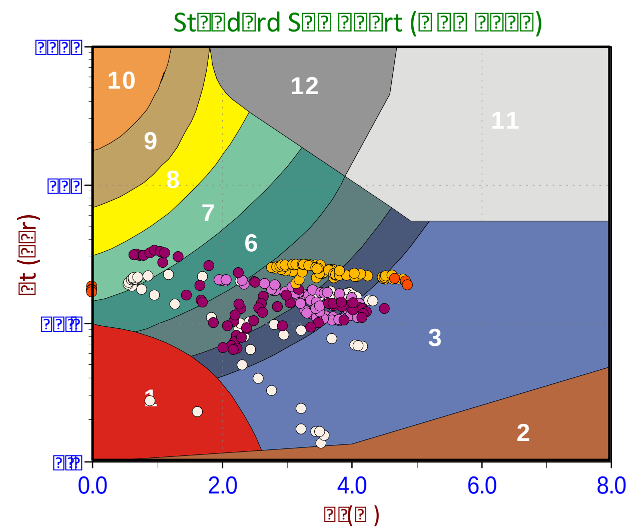
CONETEC

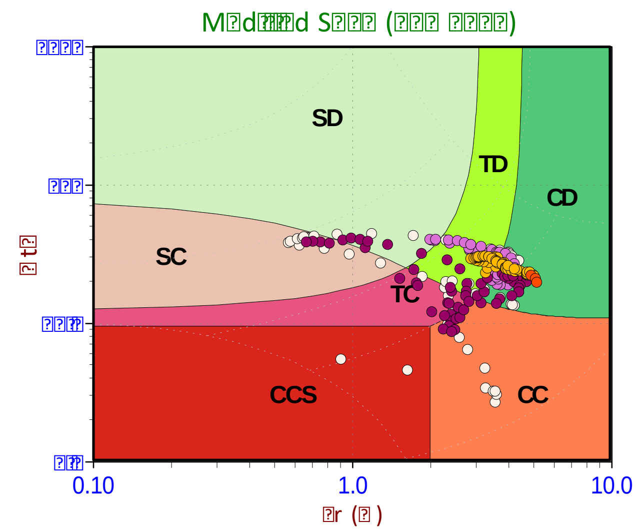
ENGEO

Job No: 21-56-21887
Date: 2021-01-29 19:03
Site: Hanover North San Jose

Sounding: 1-CPT2
Cone: 499:T1500F15U1K







Depth Ranges

- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend

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

Legend

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

Legend

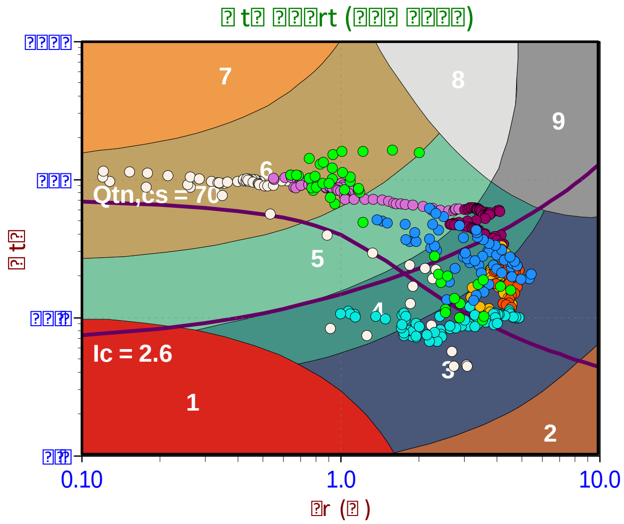
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

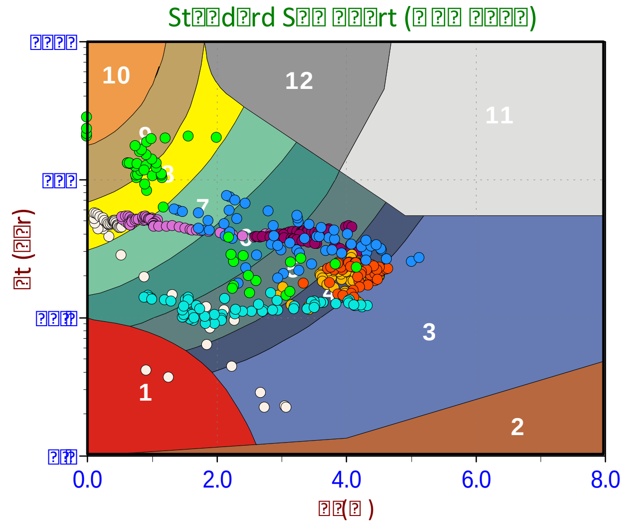
ConeTec

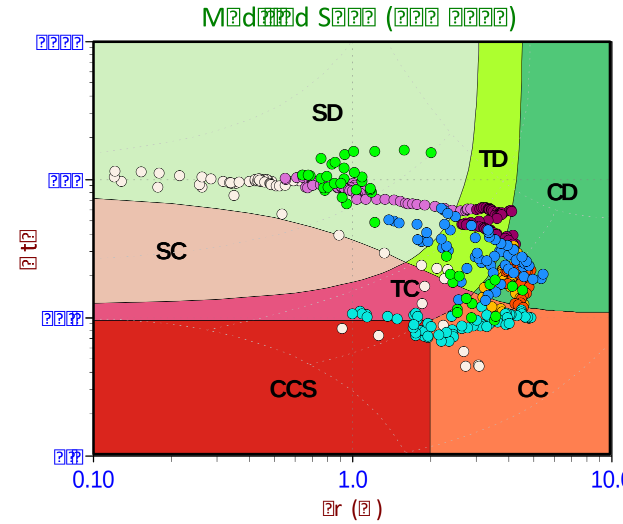
ENGEO

Job No: 21-56-21887
Date: 2021-01-29 16:15
Site: Hanover North San Jose

Sounding: 1-SCPT3
Cone: 499:T1500F15U1K







Depth Ranges

- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend

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

Legend

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

Legend

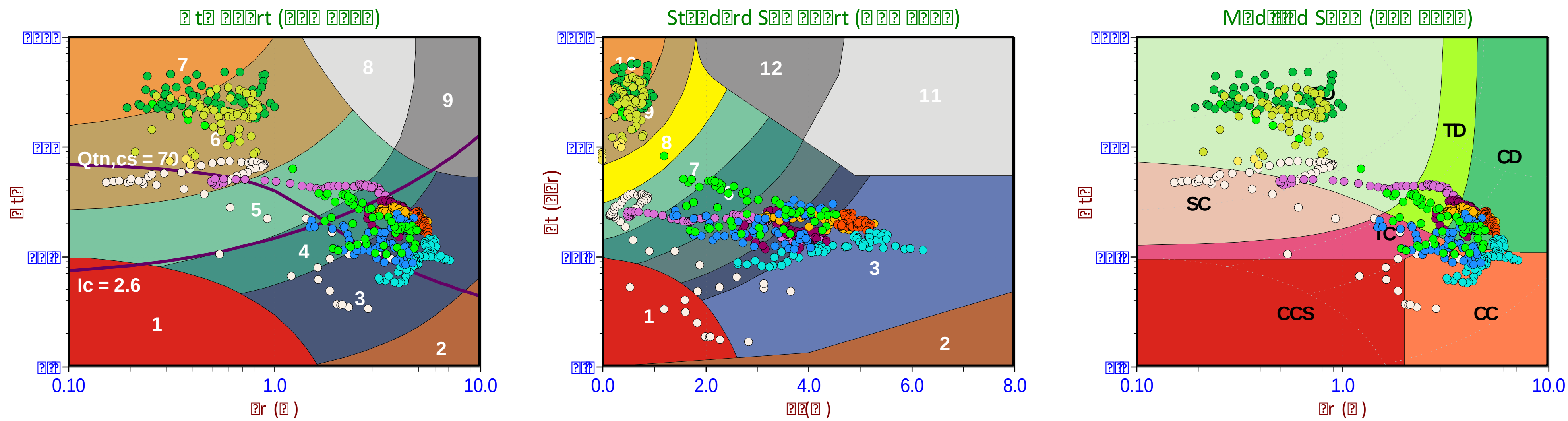
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

CONETEC

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 12:26
Site: Hanover North San Jose

Sounding: 1-CPT5
Cone: 499:T1500F15U1K



Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

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

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

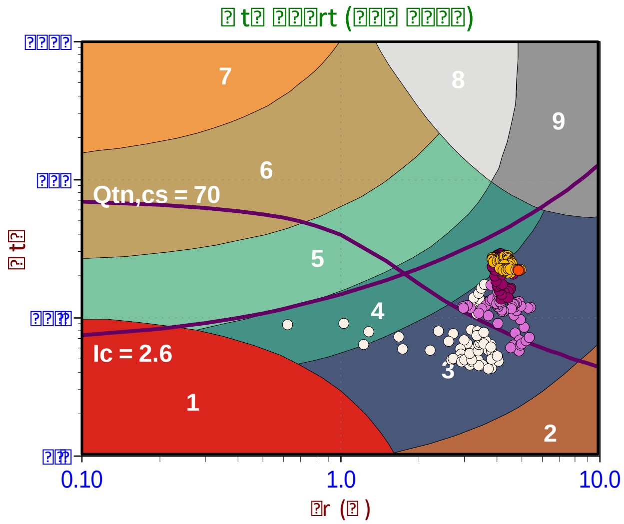
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

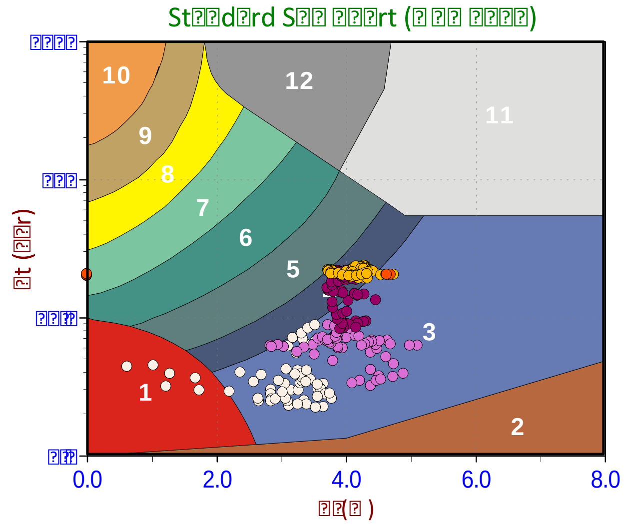
CONETEC

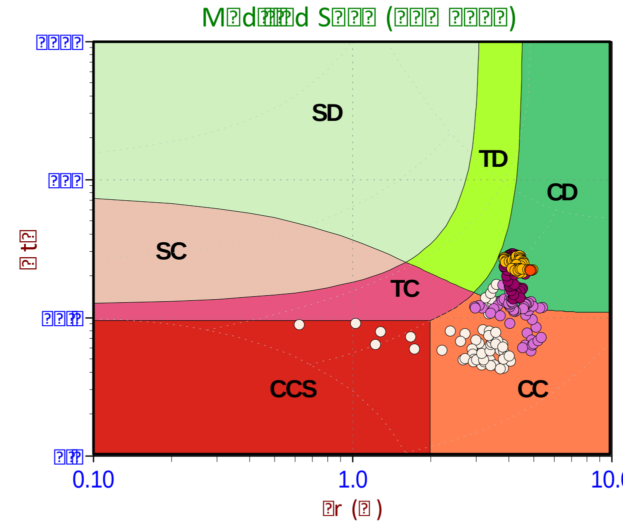
ENGEO

Job No: 21-56-21887
Date: 2021-01-29 18:19
Site: Hanover North San Jose

Sounding: 1-CPT6
Cone: 499:T1500F15U1K







Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

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

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

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

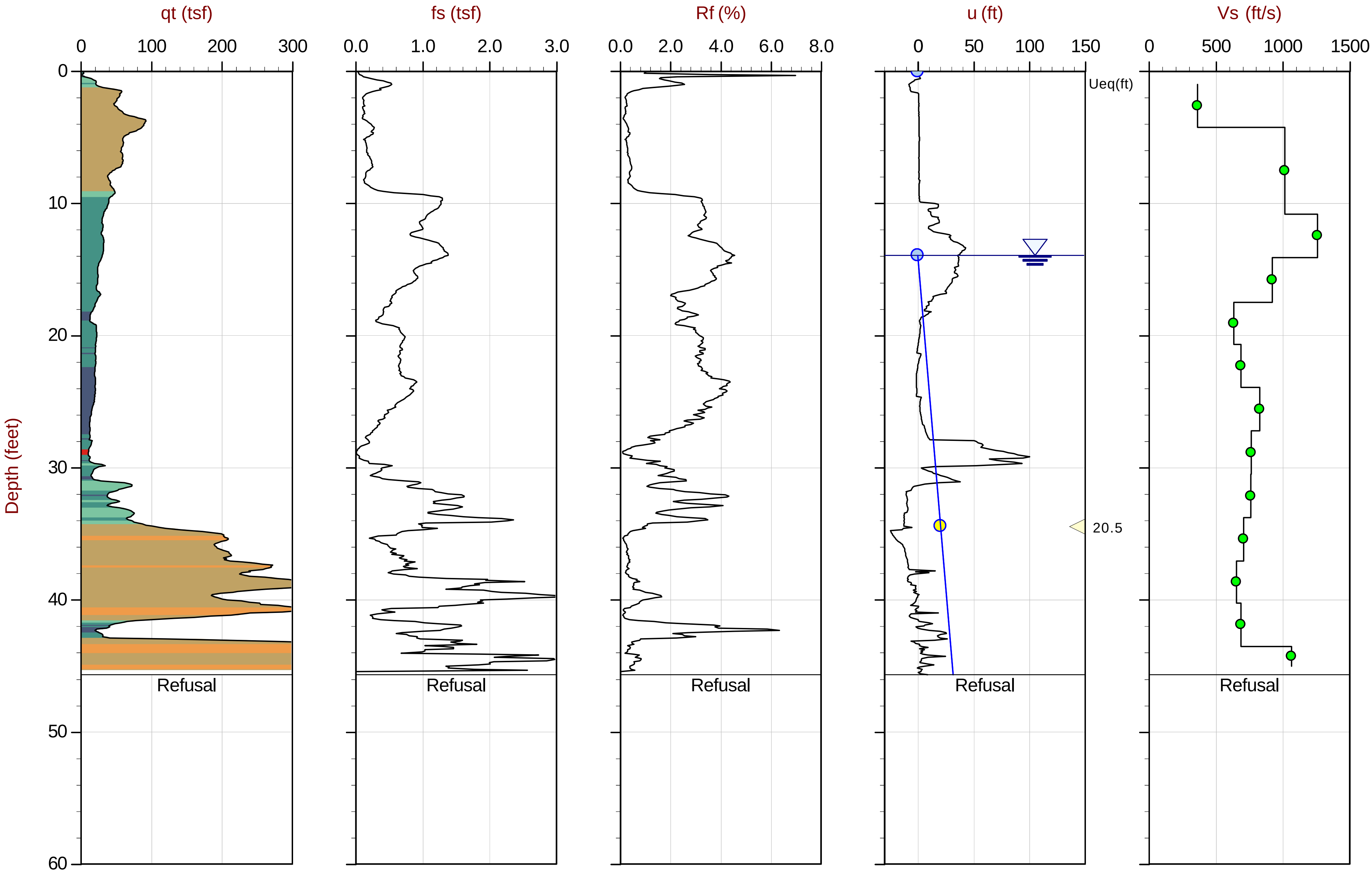
Seismic Cone Penetration Test Plots

ConeTec

ConeTec

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 14:28
Site: Hanover North San Jose
Sounding: 1-SCPT1
Cone: 499:T1500F15U1K



Max Depth: 13.925 m / 45.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139409m E: 595869m

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

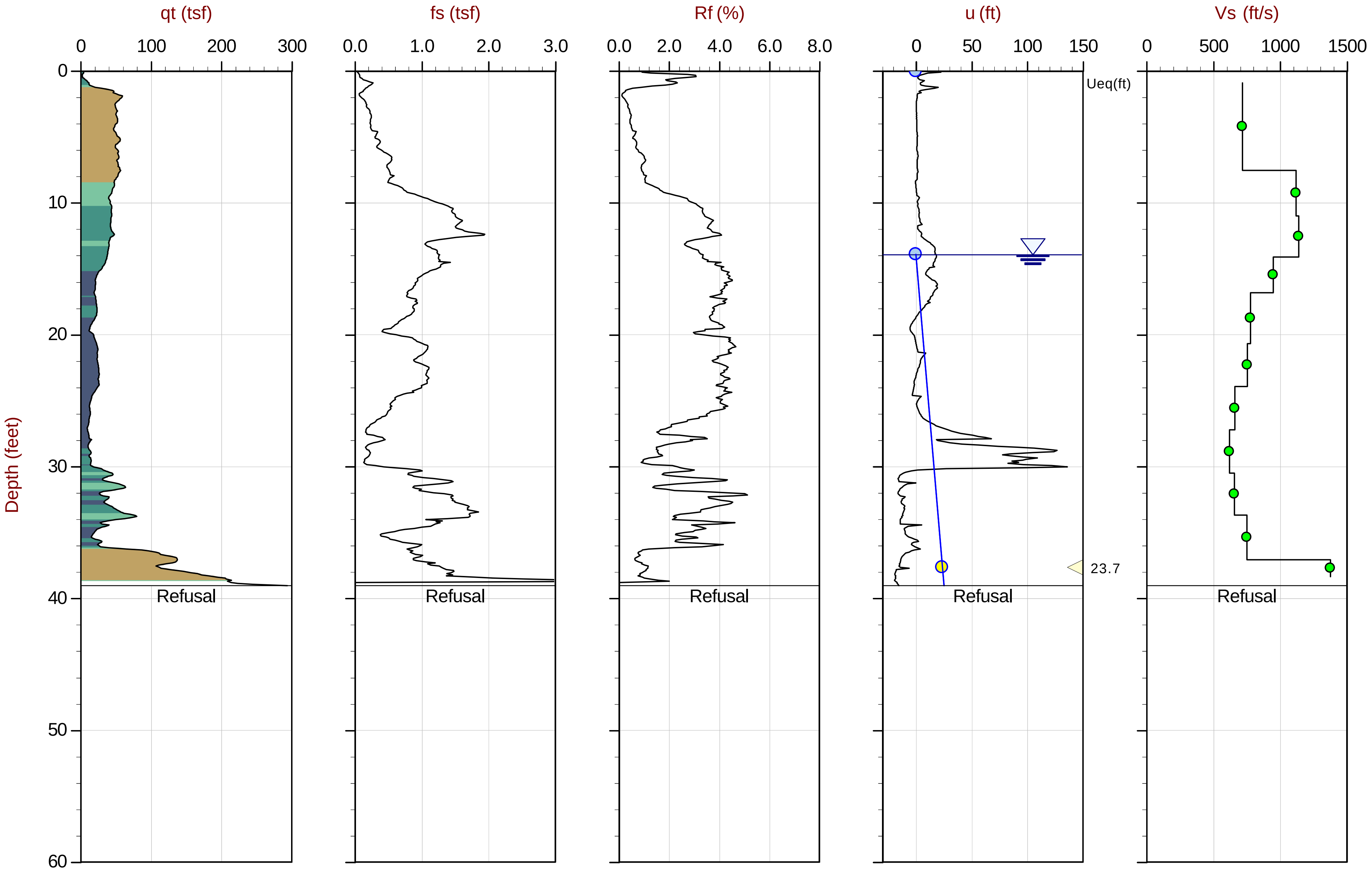
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 21-56-21887
Date: 2021-01-29 16:15
Site: Hanover North San Jose

Sounding: 1-SCPT3
Cone: 499:T1500F15U1K



Max Depth: 11.900 m / 39.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-21887_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4139556m E: 595865m

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

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

Seismic Cone Penetration Test Tabular Results

Job No: 21-56-21887
Client: ENGEO
Project: Hanover North San Jose
Sounding ID: 1-SCPT1
Date: 01:29:21 14:28

Seismic Source: Beam
Seismic Offset (ft): 10.24
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
1.64	0.98	10.29			
4.92	4.27	11.09	0.81	2.21	365
11.48	10.83	14.90	3.81	3.75	1016
14.76	14.11	17.43	2.53	2.01	1262
18.14	17.49	20.27	2.83	3.07	923
21.33	20.67	23.07	2.80	4.42	635
24.61	23.95	26.05	2.98	4.32	689
27.89	27.23	29.09	3.05	3.68	828
31.17	30.51	32.18	3.09	4.05	764
34.45	33.79	35.31	3.13	4.10	763
37.73	37.07	38.46	3.15	4.46	707
40.91	40.26	41.54	3.08	4.69	656
44.19	43.54	44.73	3.19	4.64	687
45.70	45.05	46.20	1.47	1.38	1066

CONETEC

Job No: 21-56-21887
Client: ENGEO
Project: Hanover North San Jose
Sounding ID: 1-SCPT3
Date: 01:29:21 16:15

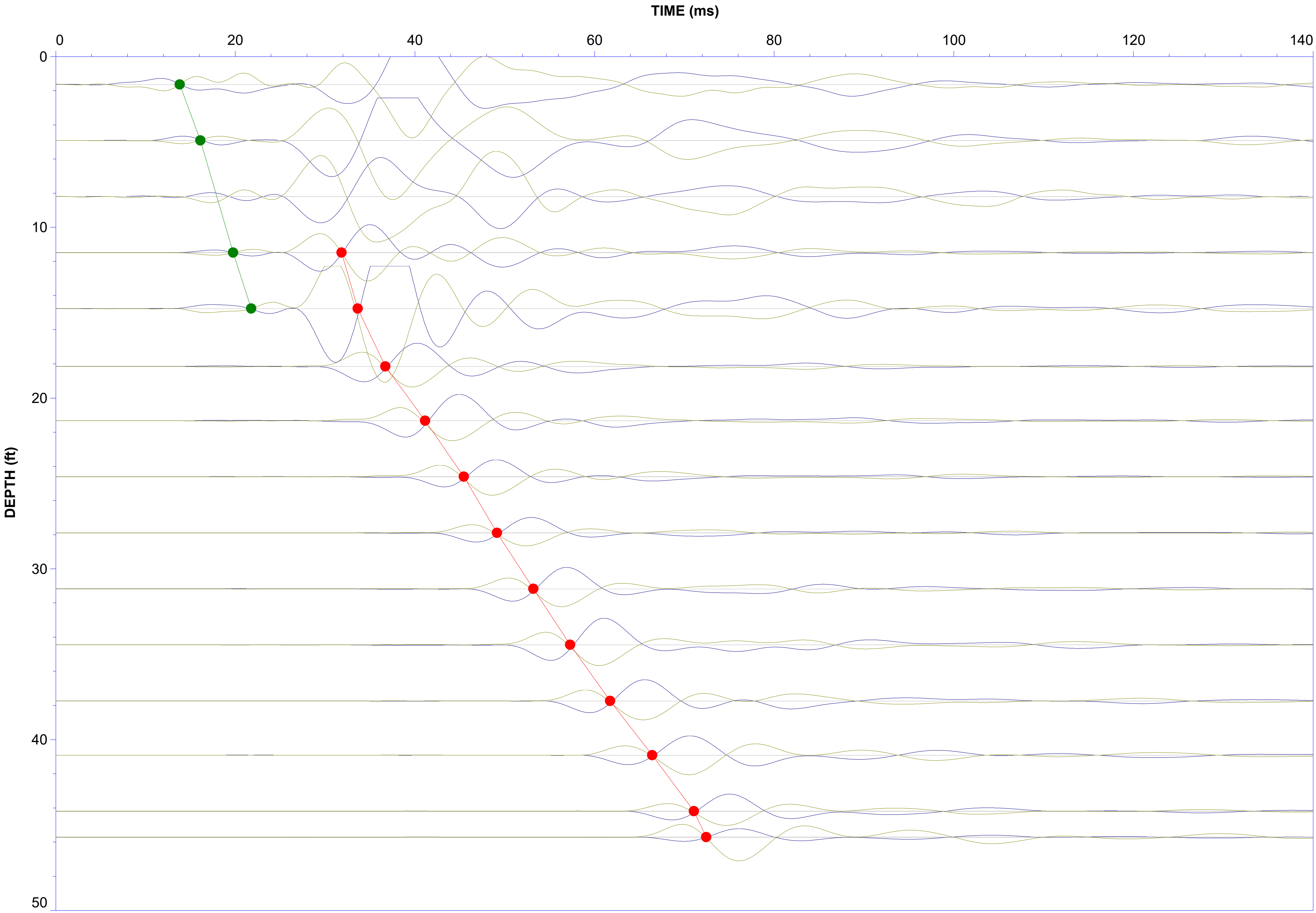
Seismic Source: Beam
Seismic Offset (ft): 10.24
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

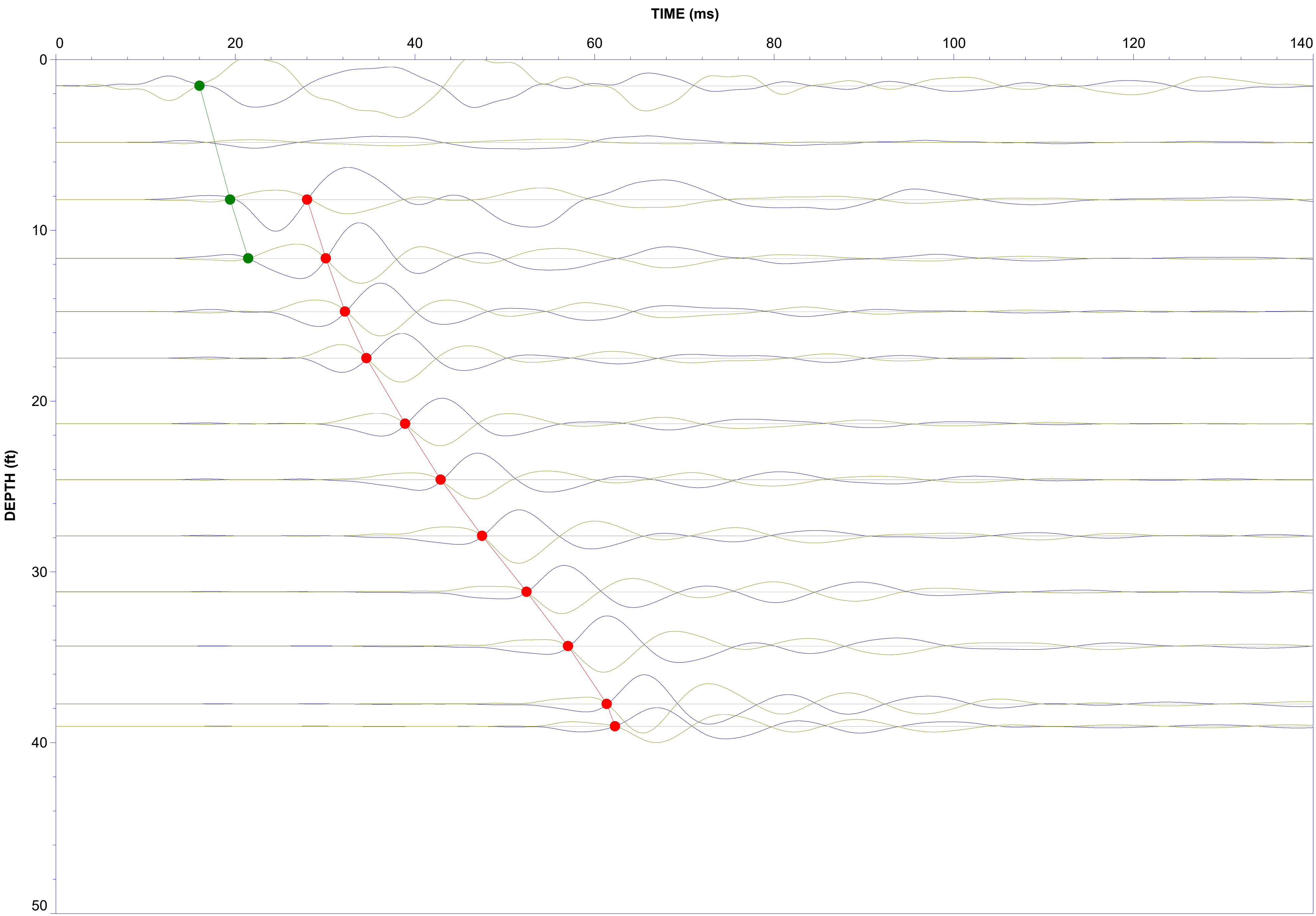
Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
1.54	0.89	10.28			
8.20	7.55	12.72	2.44	3.40	717
11.65	10.99	15.02	2.30	2.05	1121
14.76	14.11	17.43	2.41	2.12	1139
17.49	16.83	19.70	2.27	2.39	949
21.33	20.67	23.07	3.37	4.32	779
24.61	23.95	26.05	2.98	3.95	754
27.89	27.23	29.09	3.05	4.60	662
31.17	30.51	32.18	3.09	4.97	622
34.35	33.69	35.22	3.03	4.60	659
37.73	37.07	38.46	3.25	4.32	751
39.04	38.39	39.73	1.27	0.92	1377

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 21-56-21887 | Client: ENGEO | Project: Hanover North San Jose | Filter: BP 0-200 Hz | Sounding: 1-SCPT1 | Date: 01:29:21 14:28
Cone: 499:T1500F15U1K



Job No: 21-56-21887 Client: ENGEO Project: Hanover North San Jose Filter: BP 0-200 Hz Sounding: 1-SCPT3 Date: 01:29:21 16:15
Cone: 499:T1500F15U1K



Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No:	21-56-21887
Client:	ENGEO Incorporated
Project:	Hanover North San Jose
Start Date:	29-Jan-2021
End Date:	29-Jan-2021

CPTu PORE PRESSURE DISSIPATION SUMMARY

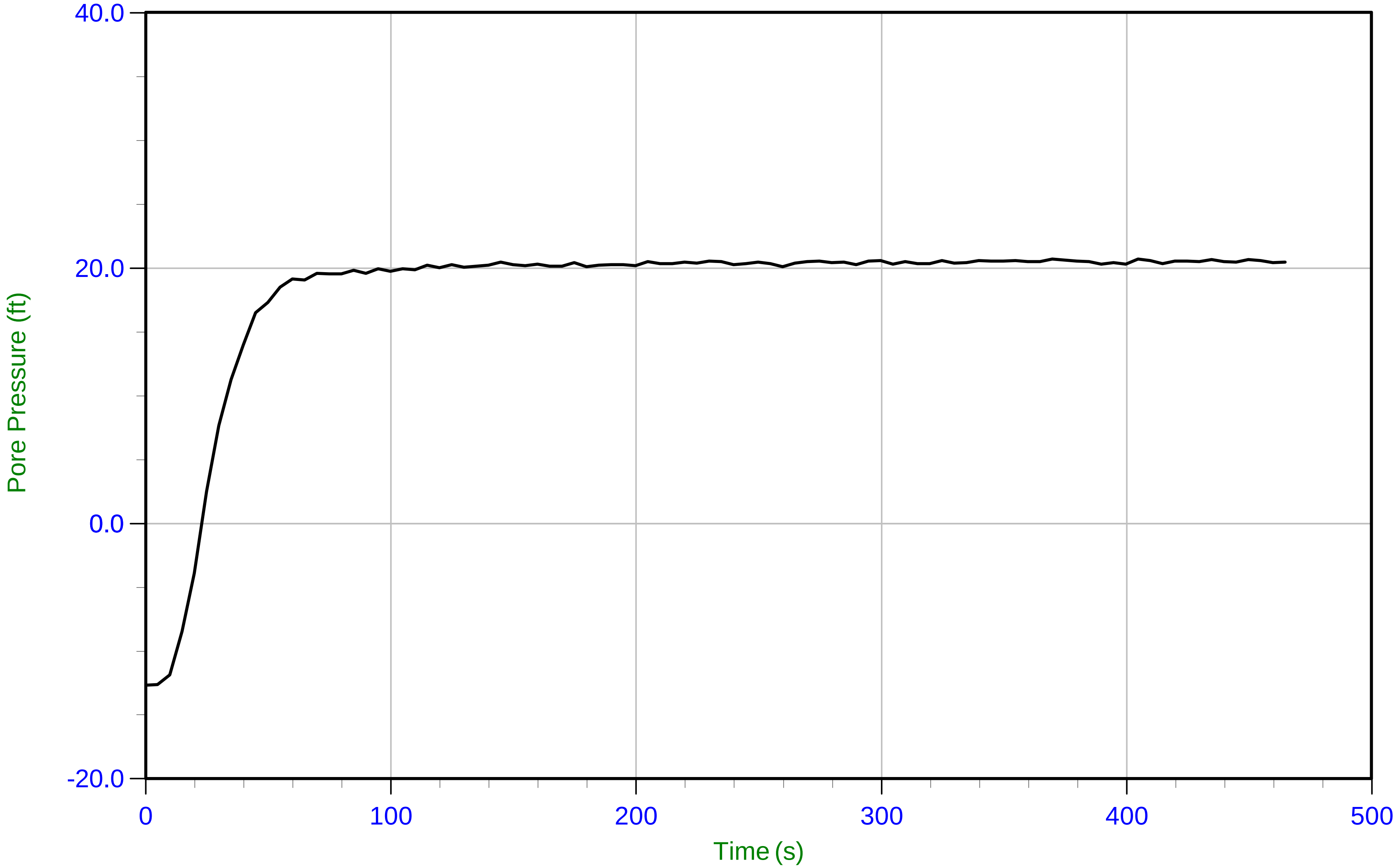
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
1-SCPT1	21-56-21887_SP01	15	465	34.45	20.5	13.9
1-SCPT3	21-56-21887_SP03	15	300	37.65	23.7	13.9
1-CPT5	21-56-21887_CP05	15	330	31.58	18.4	13.2

CONETEC

ENGEO

Job No: 21-56-21887
Date: 01/29/2021 14:28
Site: Hanover North San Jose

Sounding: 1-SCPT1
Cone: 499:T1500F15U1K Area=15 cm²



Trace Summary:

Filename: 21-56-21887_SP01.PPF
Depth: 10.500 m / 34.448 ft
Duration: 465.0 s

u Min: -12.7 ft
u Max: 20.7 ft
u Final: 20.5 ft

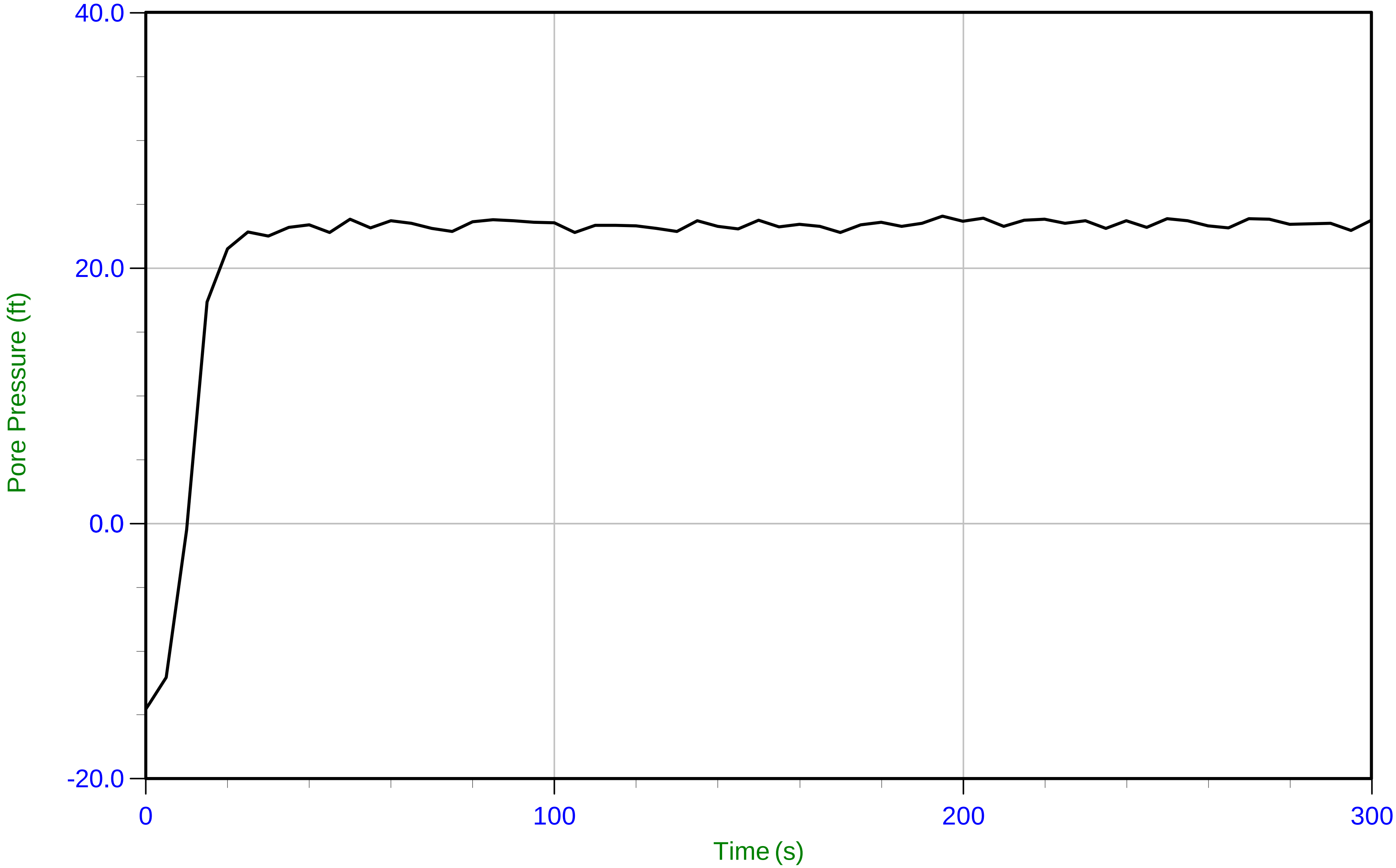
WT: 4.249 m / 13.939 ft
Ueq: 20.5 ft

CONETEC

ENGEO

Job No: 21-56-21887
Date: 01/29/2021 16:15
Site: Hanover North San Jose

Sounding: 1-SCPT3
Cone: 499:T1500F15U1K Area=15 cm²



Trace Summary:

Filename: 21-56-21887_SP03.PPF
Depth: 11.475 m / 37.647 ft
Duration: 300.0 s

u Min: -14.5 ft
u Max: 24.0 ft
u Final: 23.7 ft

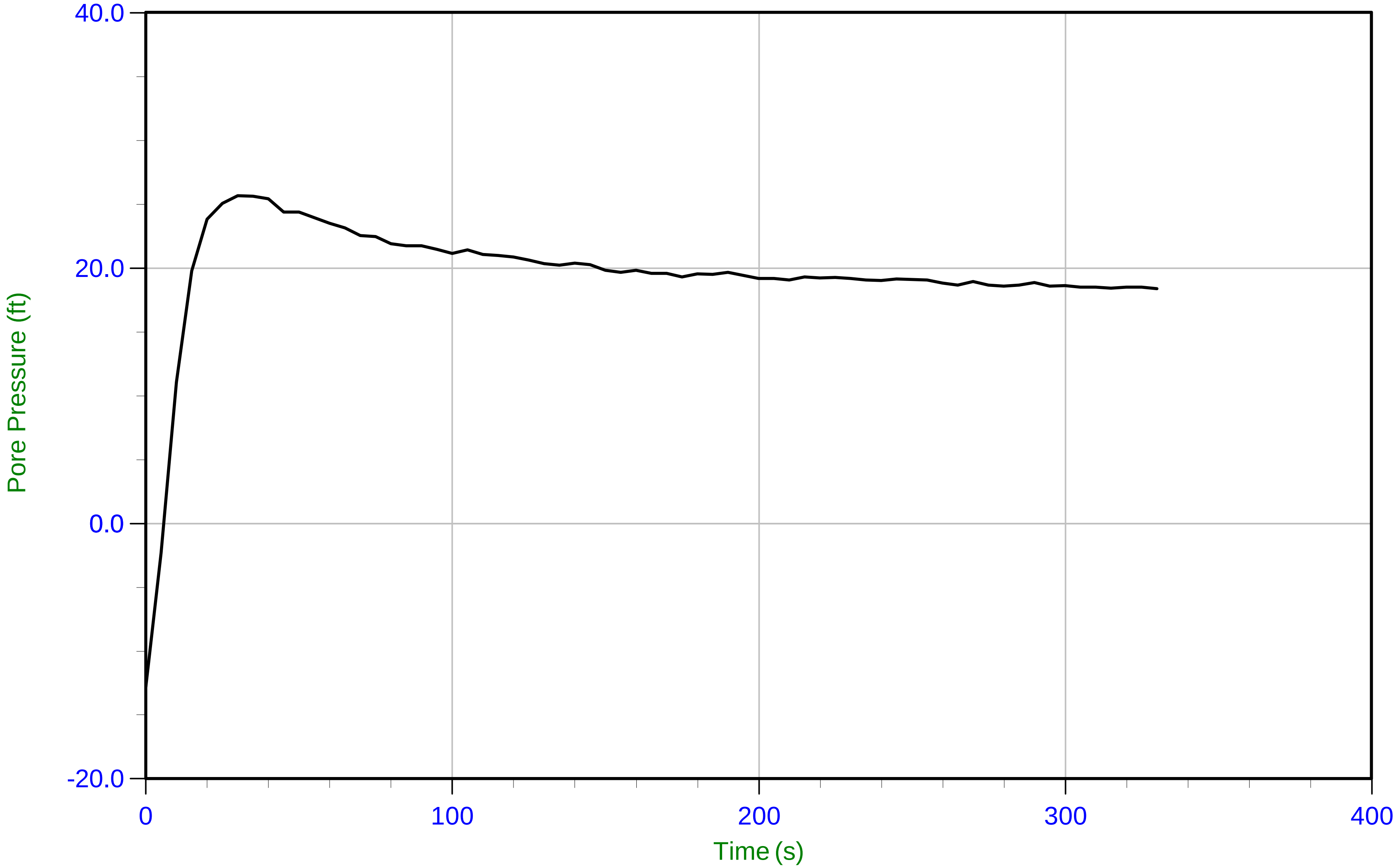
WT: 4.240 m / 13.911 ft
Ueq: 23.7 ft

CONETEC

ENGEO

Job No: 21-56-21887
Date: 01/29/2021 12:26
Site: Hanover North San Jose

Sounding: 1-CPT5
Cone: 499:T1500F15U1K Area=15 cm²



Trace Summary:

Filename: 21-56-21887_CP05.PPF
Depth: 9.625 m / 31.578 ft
Duration: 330.0 s

u Min: -12.8 ft
u Max: 25.7 ft
u Final: 18.4 ft

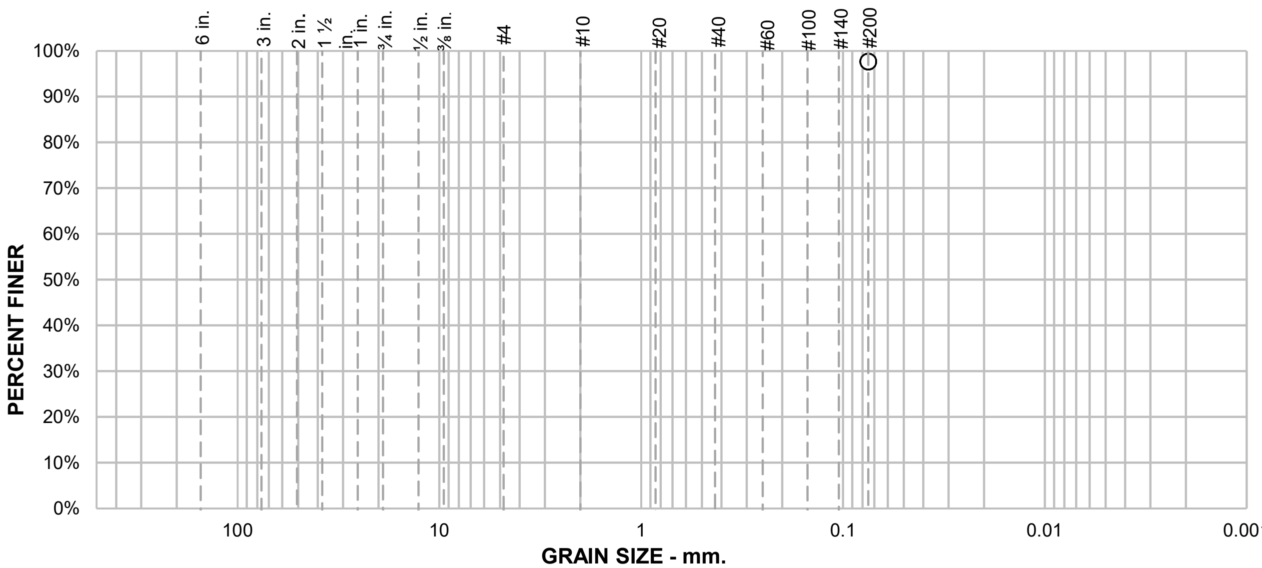
WT: 4.027 m / 13.213 ft
Ueq: 18.4 ft

APPENDIX B

Laboratory Test Results

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-DP1@13-15

DEPTH (ft): 13-15

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						97.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	97.6		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 25	LL = 57	PI = 32

COEFFICIENTS

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

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 94.56 g

ENGEO Expect Excellence

CLIENT: Hanover R.S. Limited Partnership

PROJECT NAME: 681 E Trimble

PROJECT NO: 18233.000.001 PH001

PROJECT LOCATION: San Jose, CA

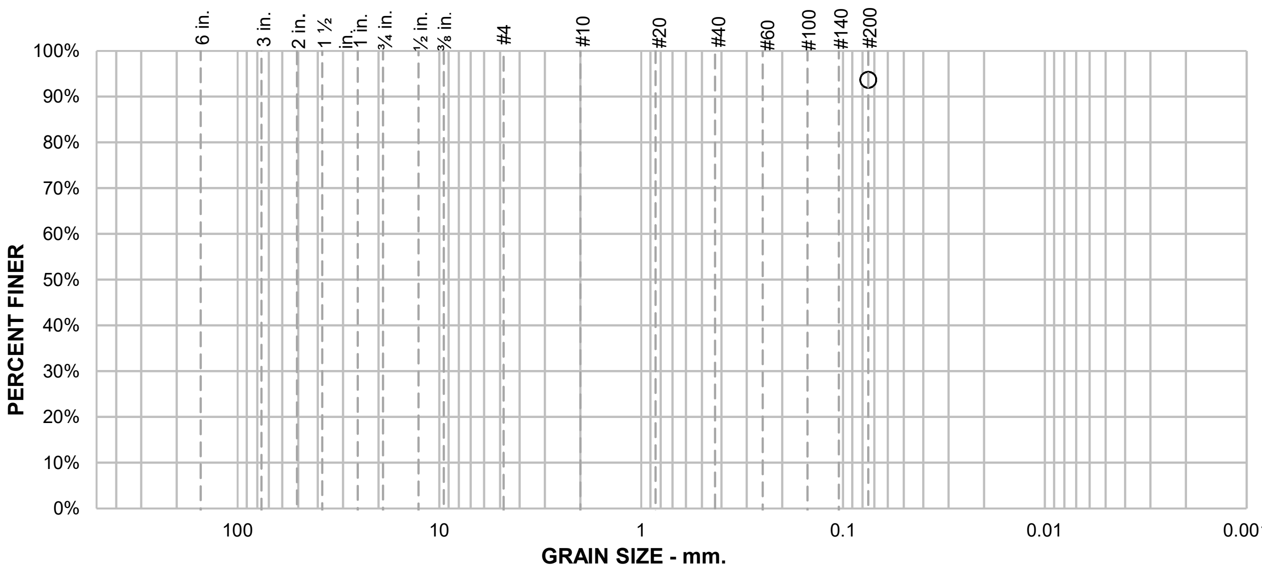
REPORT DATE: 3/19/2021

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-DP1@8-13

DEPTH (ft): 8-13

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						93.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	93.6		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 23 LL = 38 PI = 15

COEFFICIENTS

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

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 141.26 g

CLIENT: Hanover R.S. Limited Partnership

PROJECT NAME: 681 E Trimble

PROJECT NO: 18233.000.001 PH001

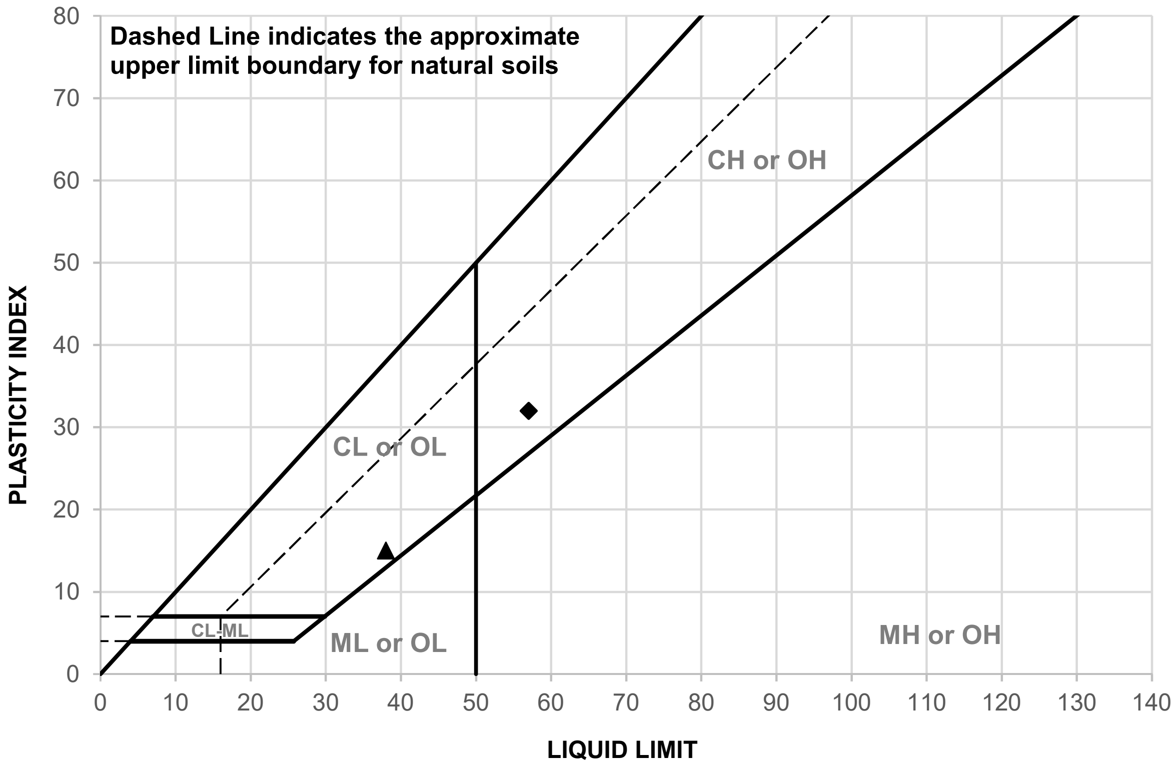
PROJECT LOCATION: San Jose, CA

REPORT DATE: 3/19/2021

TESTED BY: M. Quasem

REVIEWED BY: W. Miller

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-DP1@8-13	8-13 feet	See exploration logs	38	23	15
◆	1-DP1@13-18	13-15 feet	See exploration logs	57	25	32

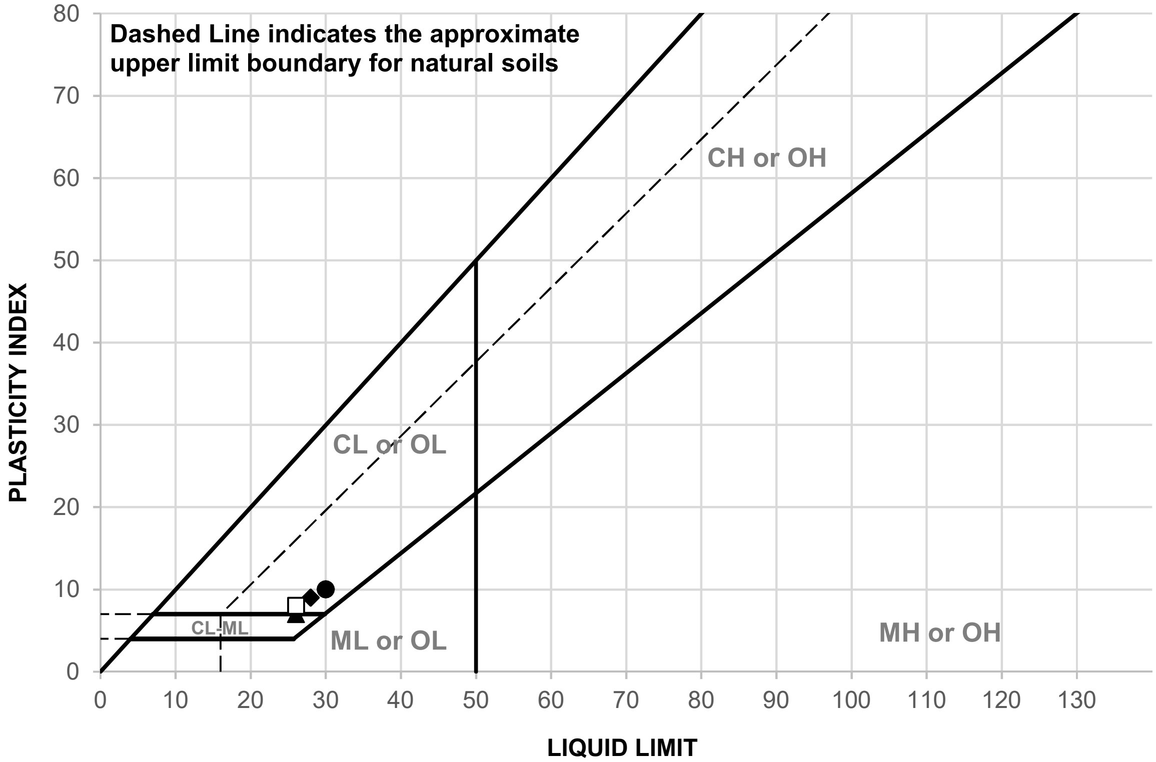
	SAMPLE ID	TEST METHOD	REMARKS
▲	1-DP1@8-13	PI: ASTM D4318, Wet Method	
◆	1-DP1@13-15	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Hanover R.S. Limited Partnership
PROJECT NAME: 681 E Trimble
PROJECT NO: 18233.000.001 PH001
PROJECT LOCATION: San Jose, CA
REPORT DATE: 3/23/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	S-1	0.5-1 feet	See exploration logs	26	19	7
◆	S-2	0.5 feet	See exploration logs	28	19	9
□	S-3	0.5 feet	See exploration logs	26	18	8
●	S-4	0.5 feet	See exploration logs	30	20	10

	SAMPLE ID	TEST METHOD	REMARKS
▲	S-1	PI: ASTM D4318, Wet Method	
◆	S-2	PI: ASTM D4318, Wet Method	
□	S-3	PI: ASTM D4318, Wet Method	
●	S-4	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Hanover R. S. Limited Partnership
PROJECT NAME: 681 E Trimble
PROJECT NO: 18233.000.001 PH001
PROJECT LOCATION: San Jose, CA
REPORT DATE: 1/27/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

APPENDIX C

Liquefaction Analysis

ENGEO
2010 Crow Canyon Place
San Ramon, CA 94583

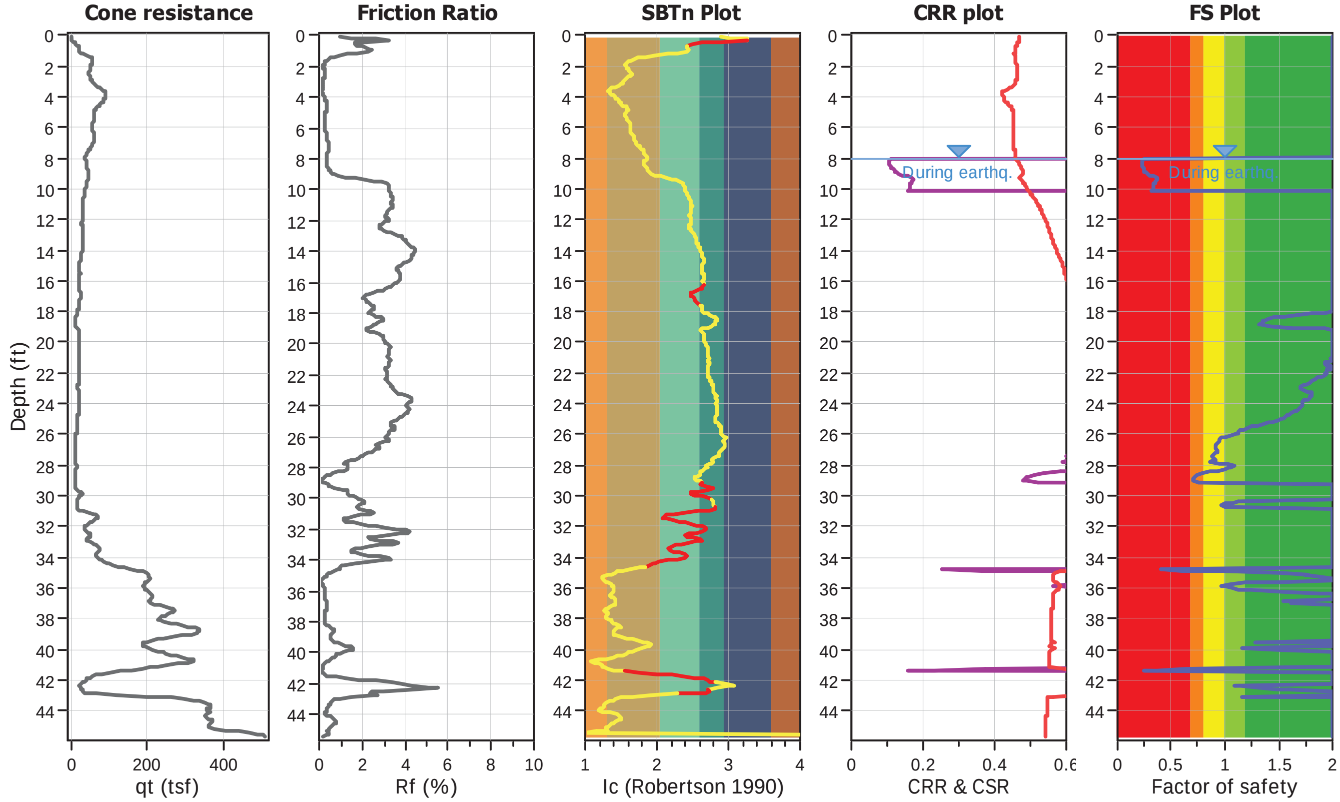
LIQUEFACTION ANALYSIS REPORT

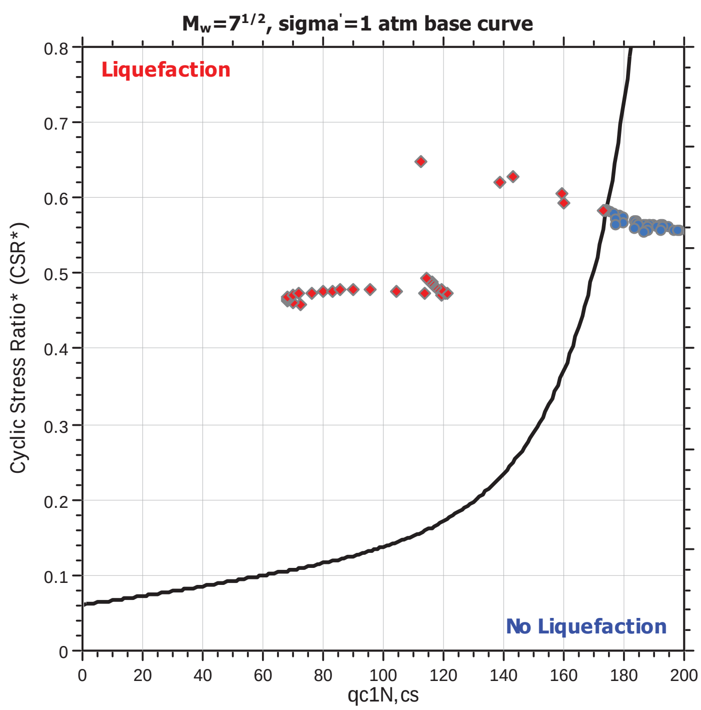
Project title : Hanover North San Jose **Location : 681 East Trimble Road, San Jose**

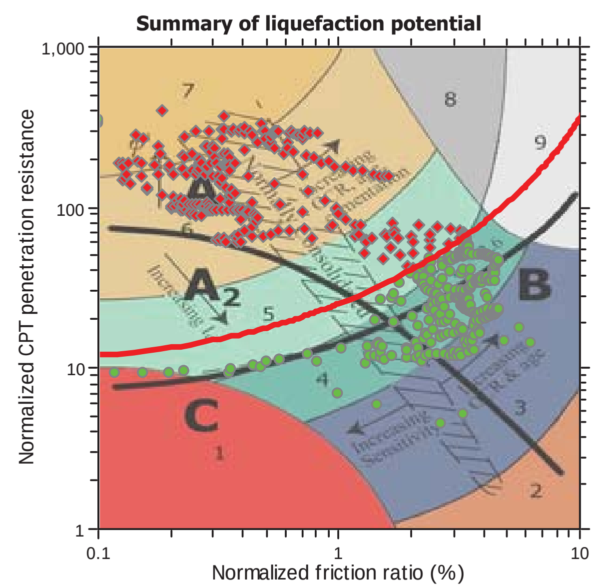
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 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:	6.90	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

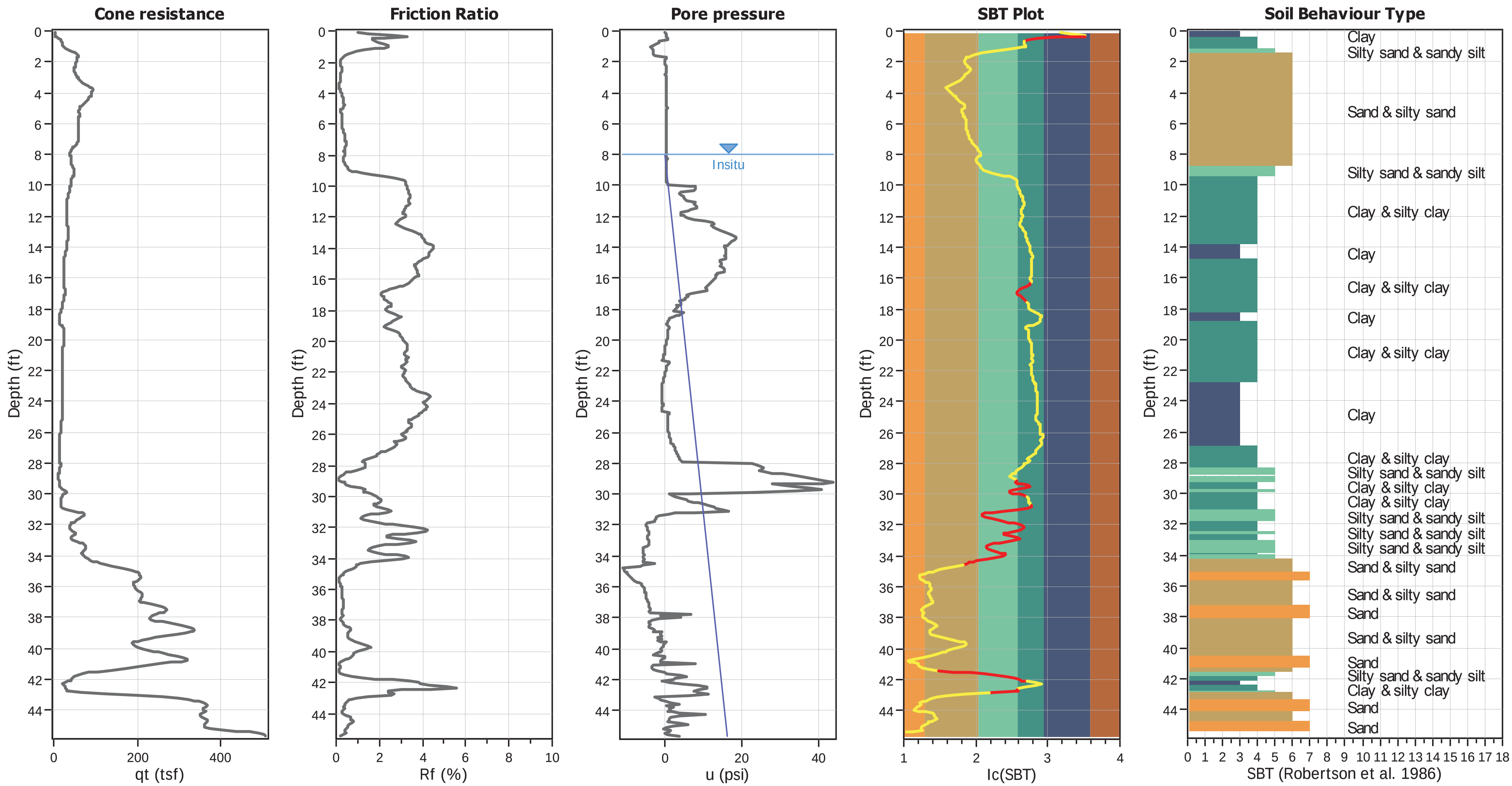






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



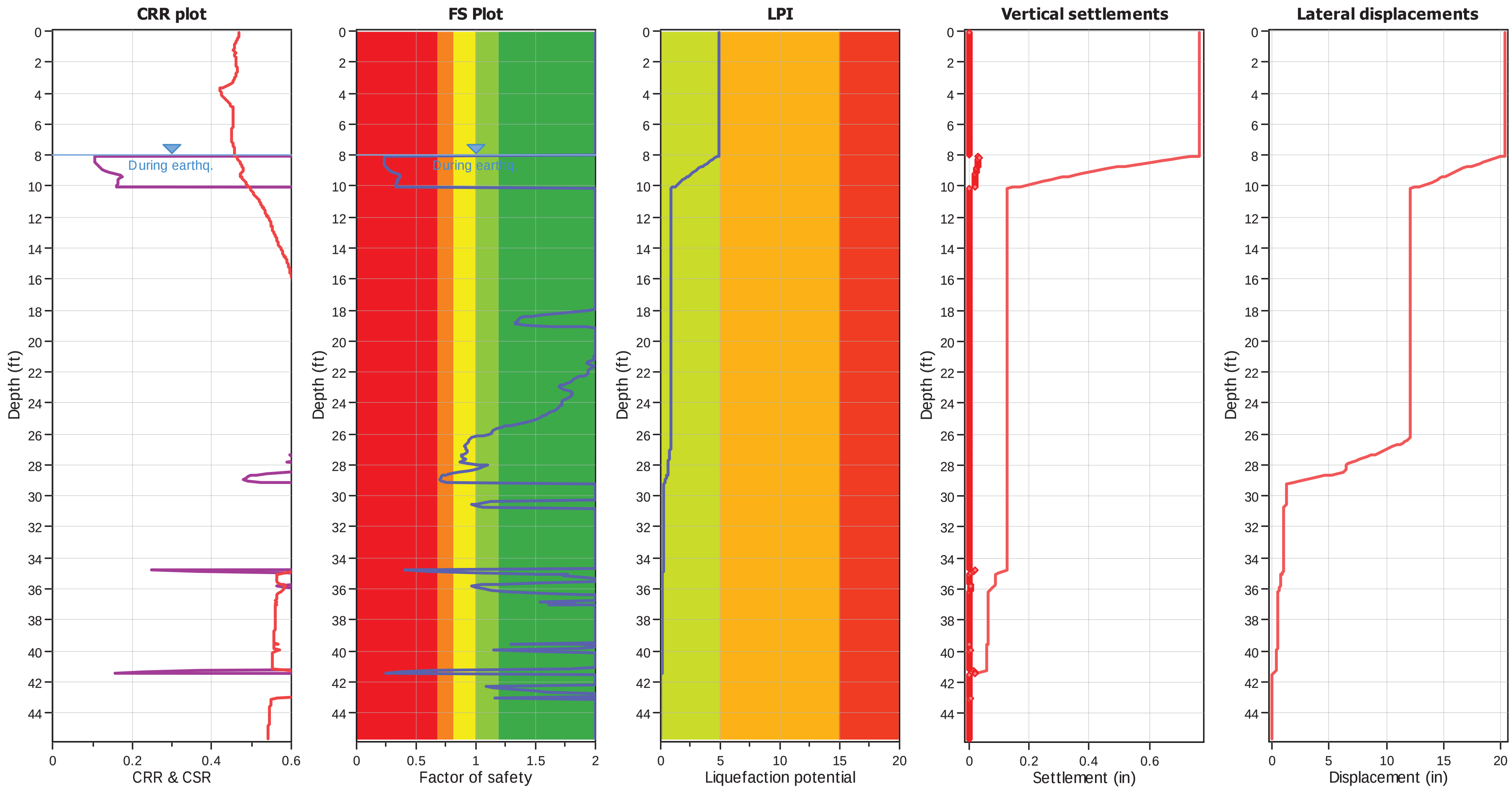
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.40	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.40	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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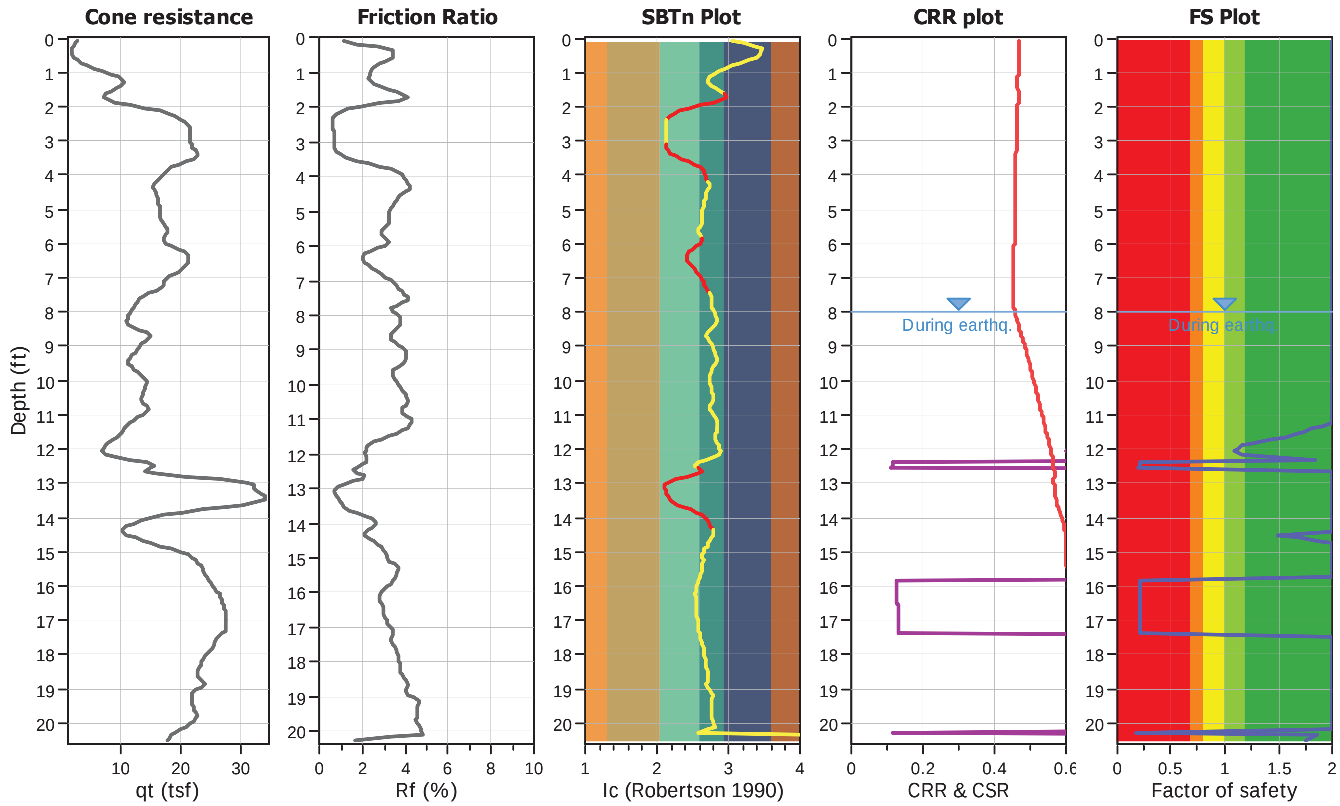
LIQUEFACTION ANALYSIS REPORT

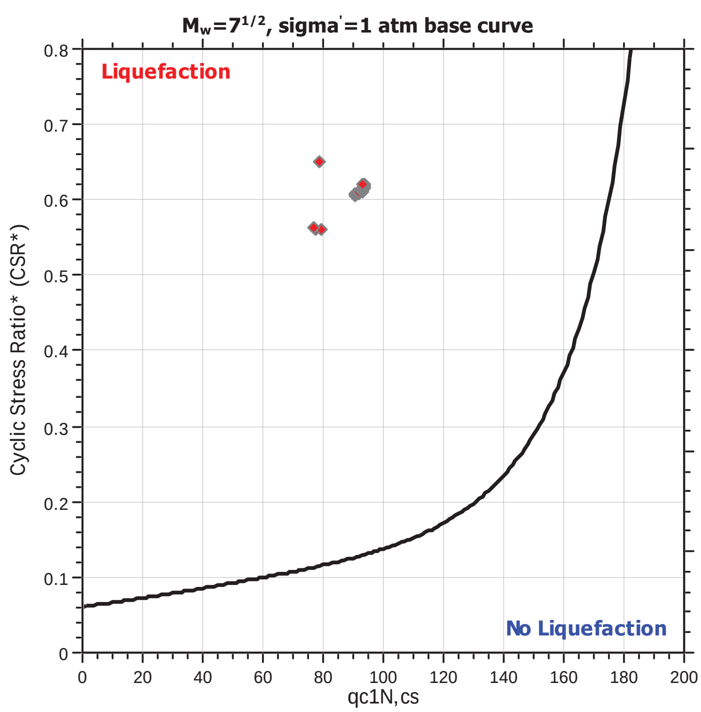
Project title : Hanover North San Jose **Location : 681 East Trimble Road, San Jose**

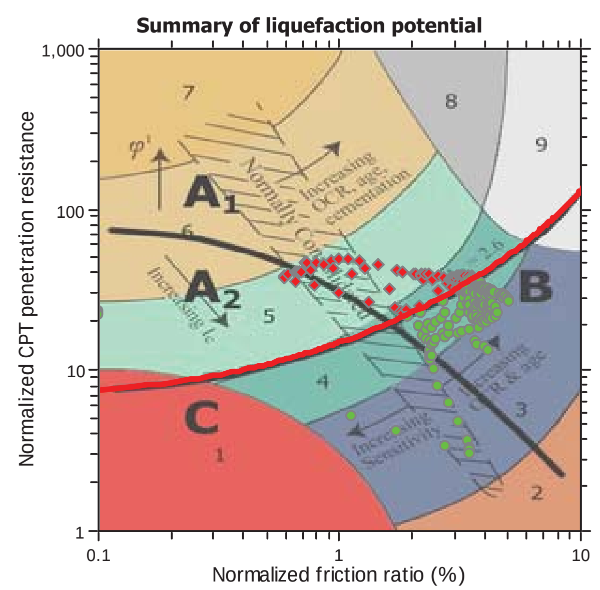
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 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:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

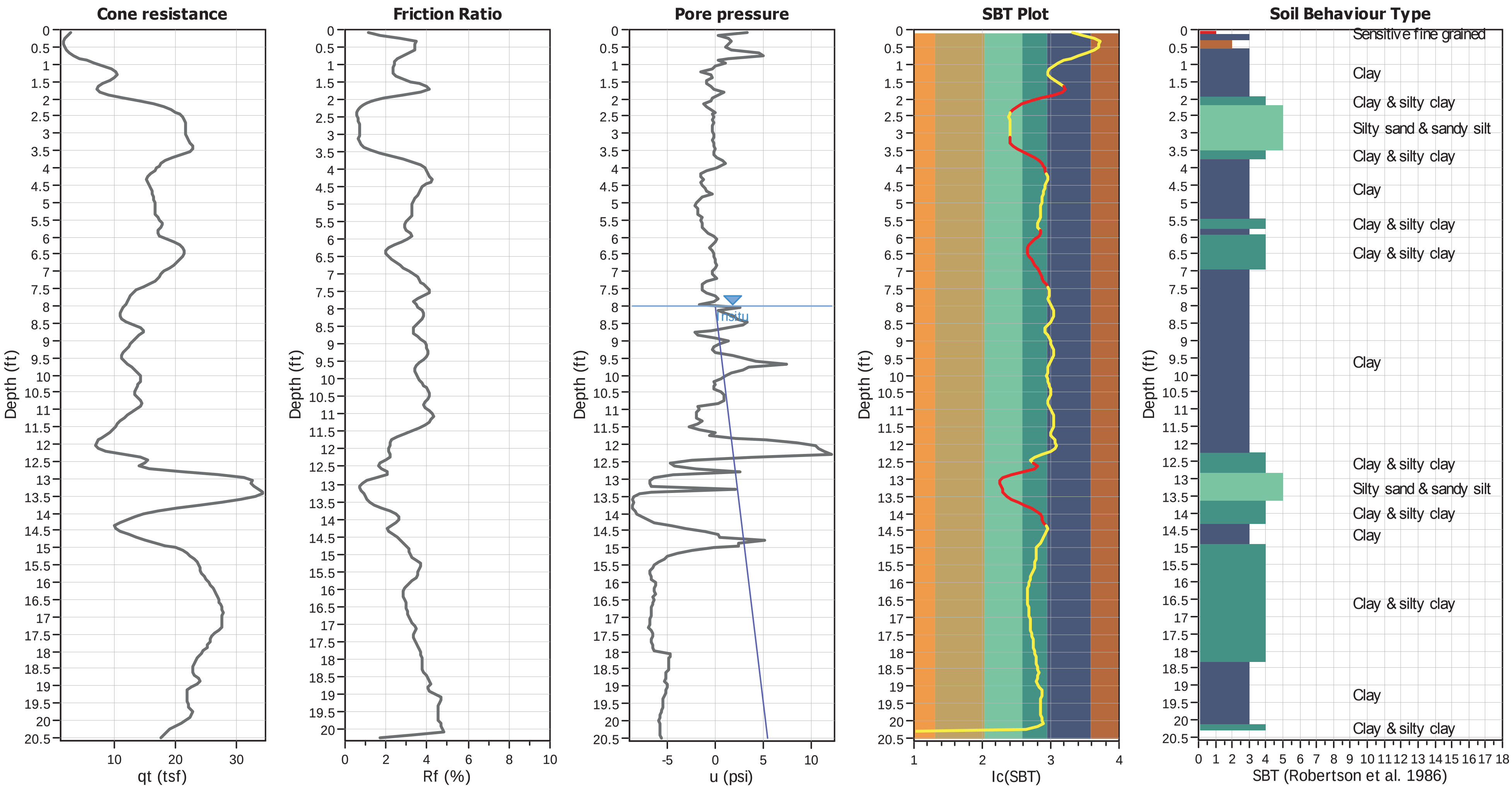






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



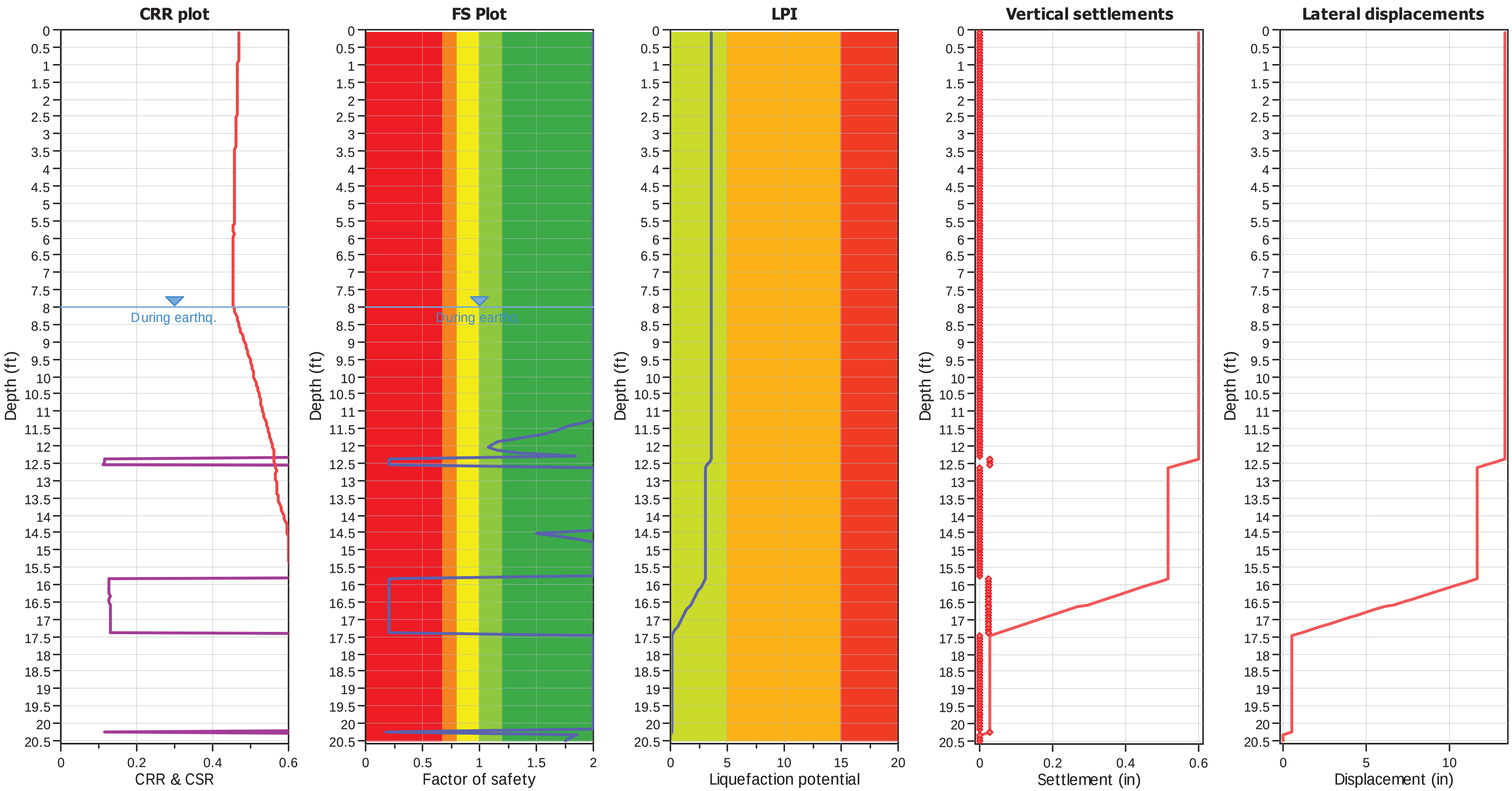
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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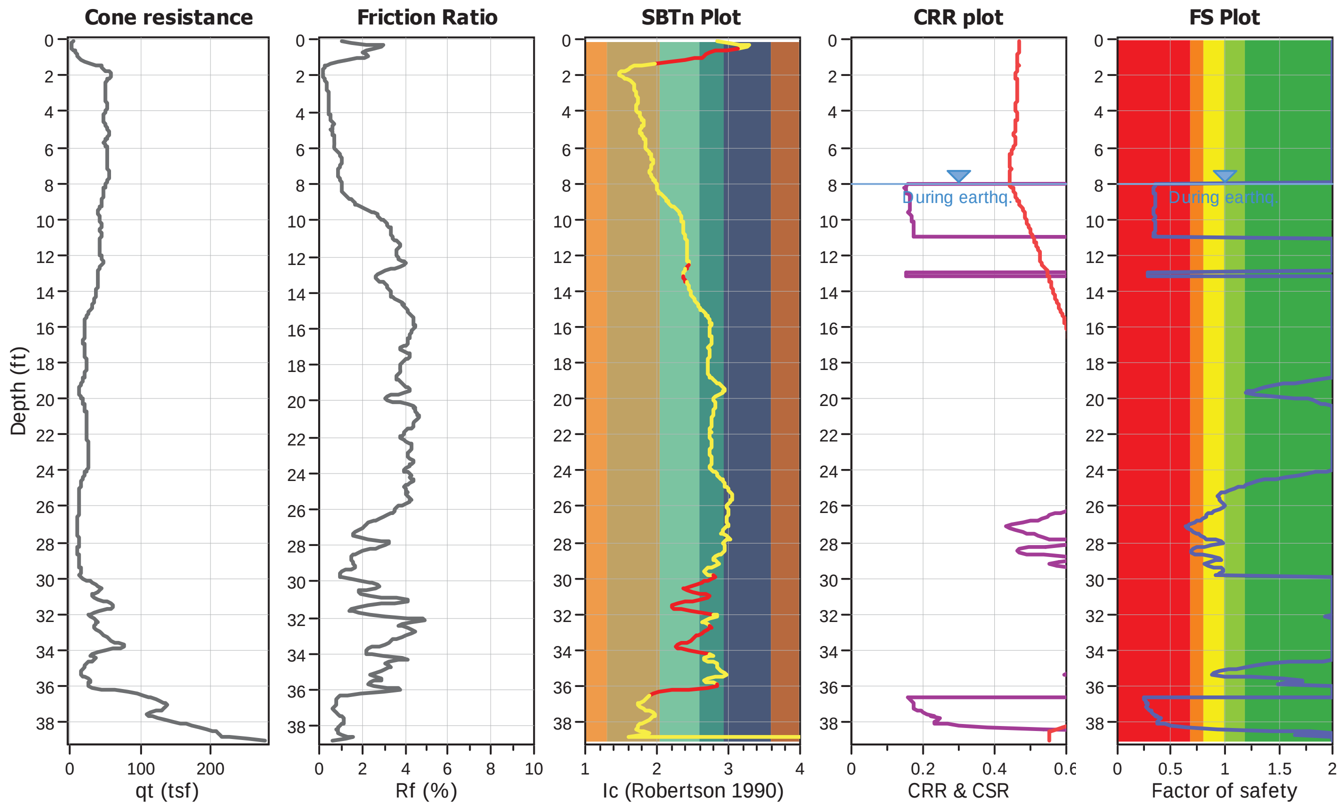
LIQUEFACTION ANALYSIS REPORT

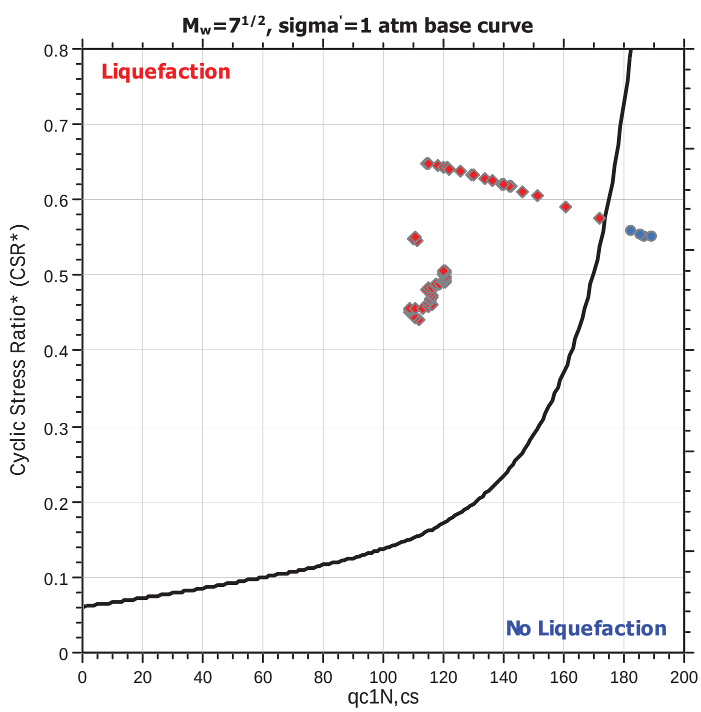
Project title : Hanover North San Jose **Location : 681 East Trimble Road, San Jose**

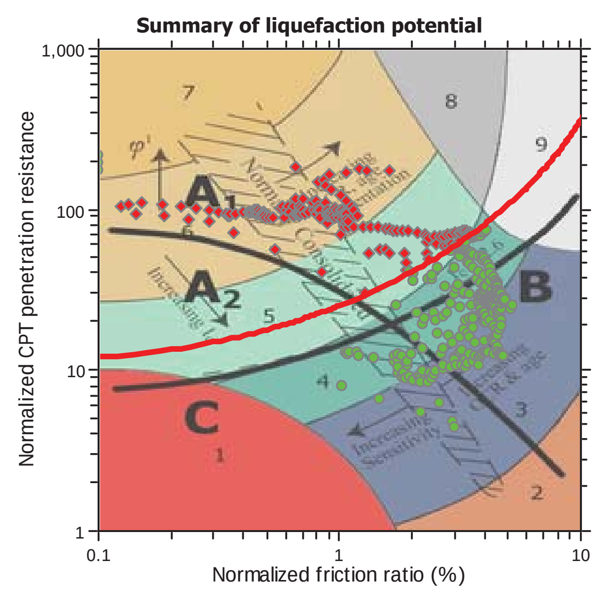
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	applied:	Sand & Clay
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method

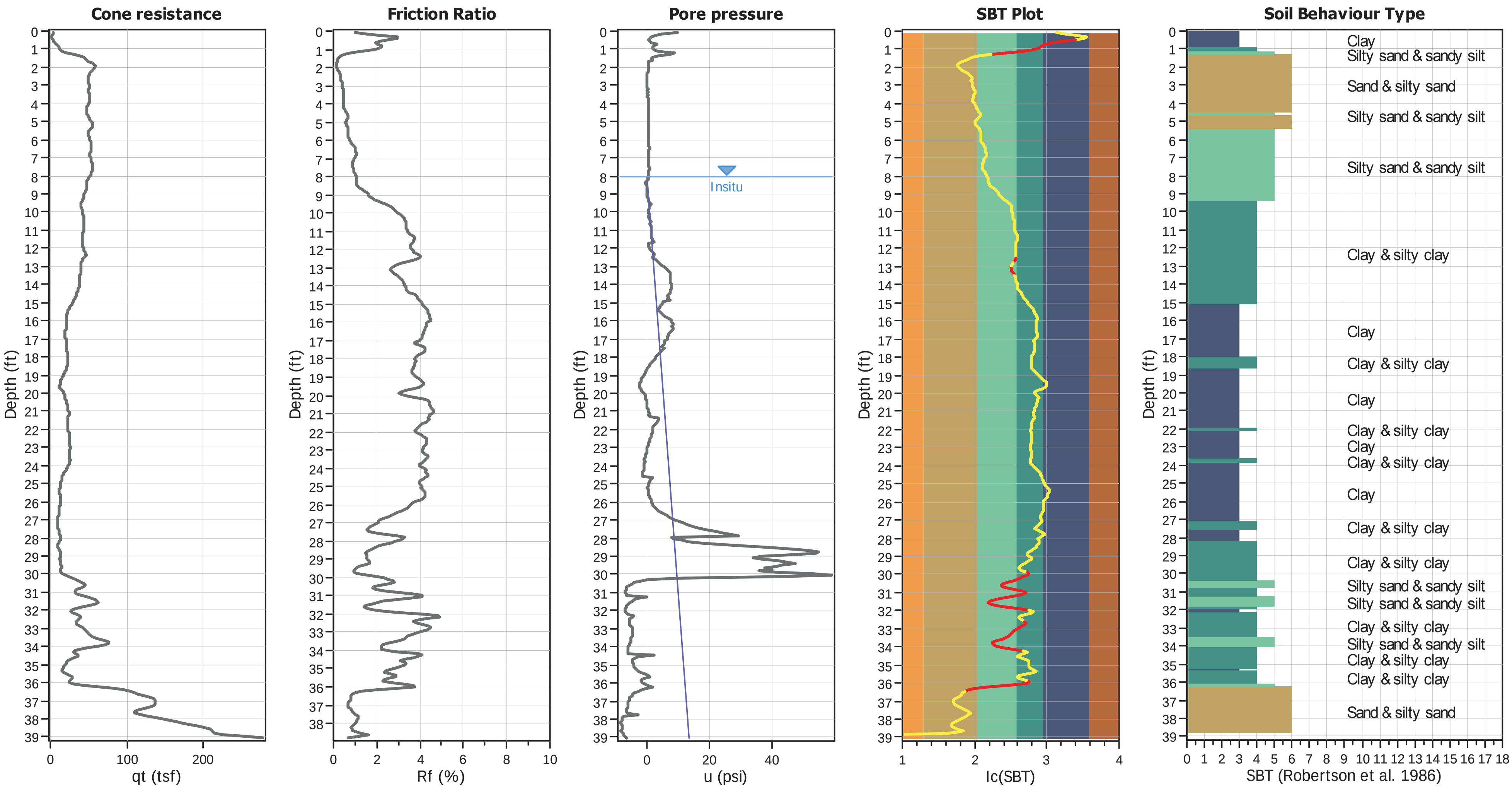






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



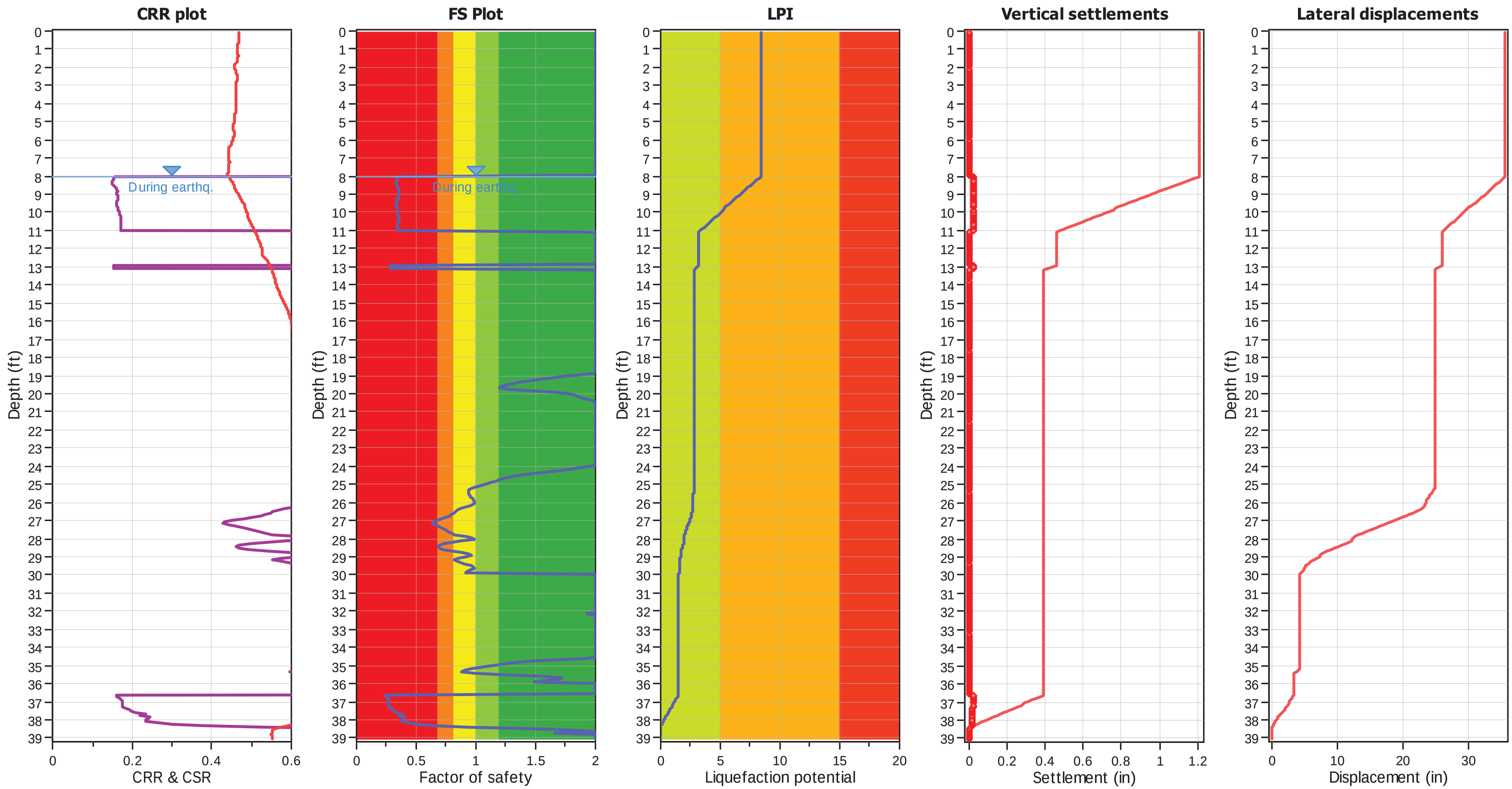
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.40	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.40	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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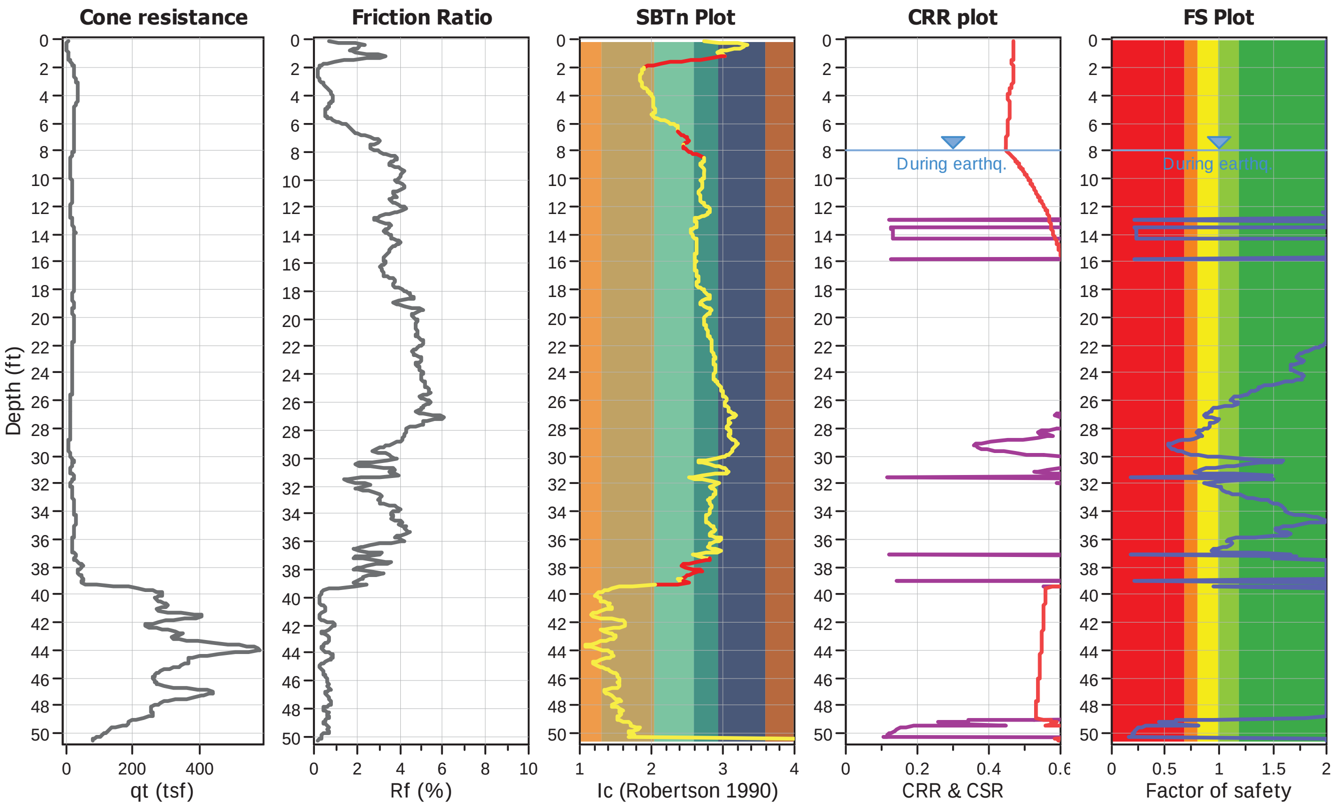
LIQUEFACTION ANALYSIS REPORT

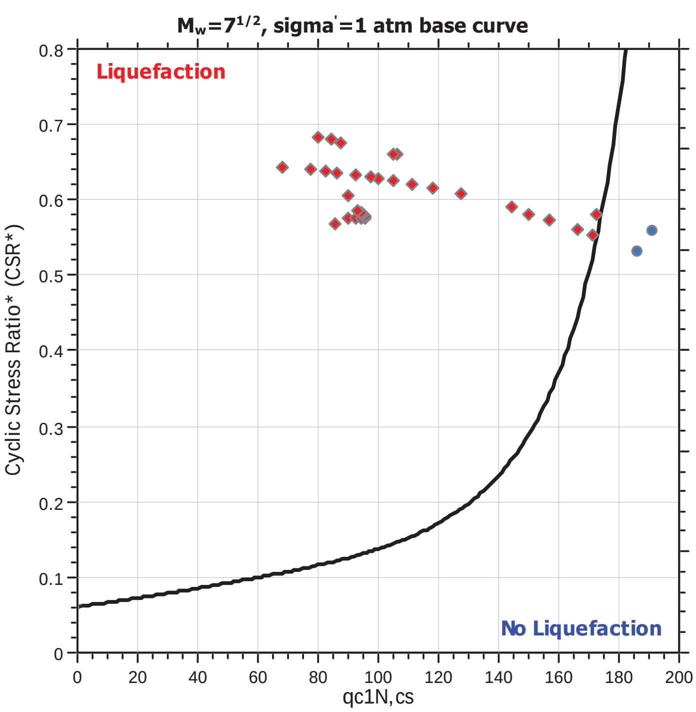
Project title : Hanover North San Jose **Location : 681 East Trimble Road, San Jose**

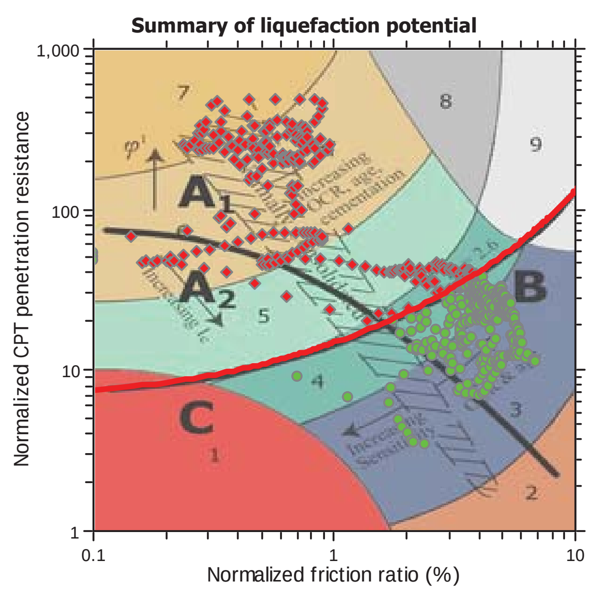
CPT file : 1-CPT5

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 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:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

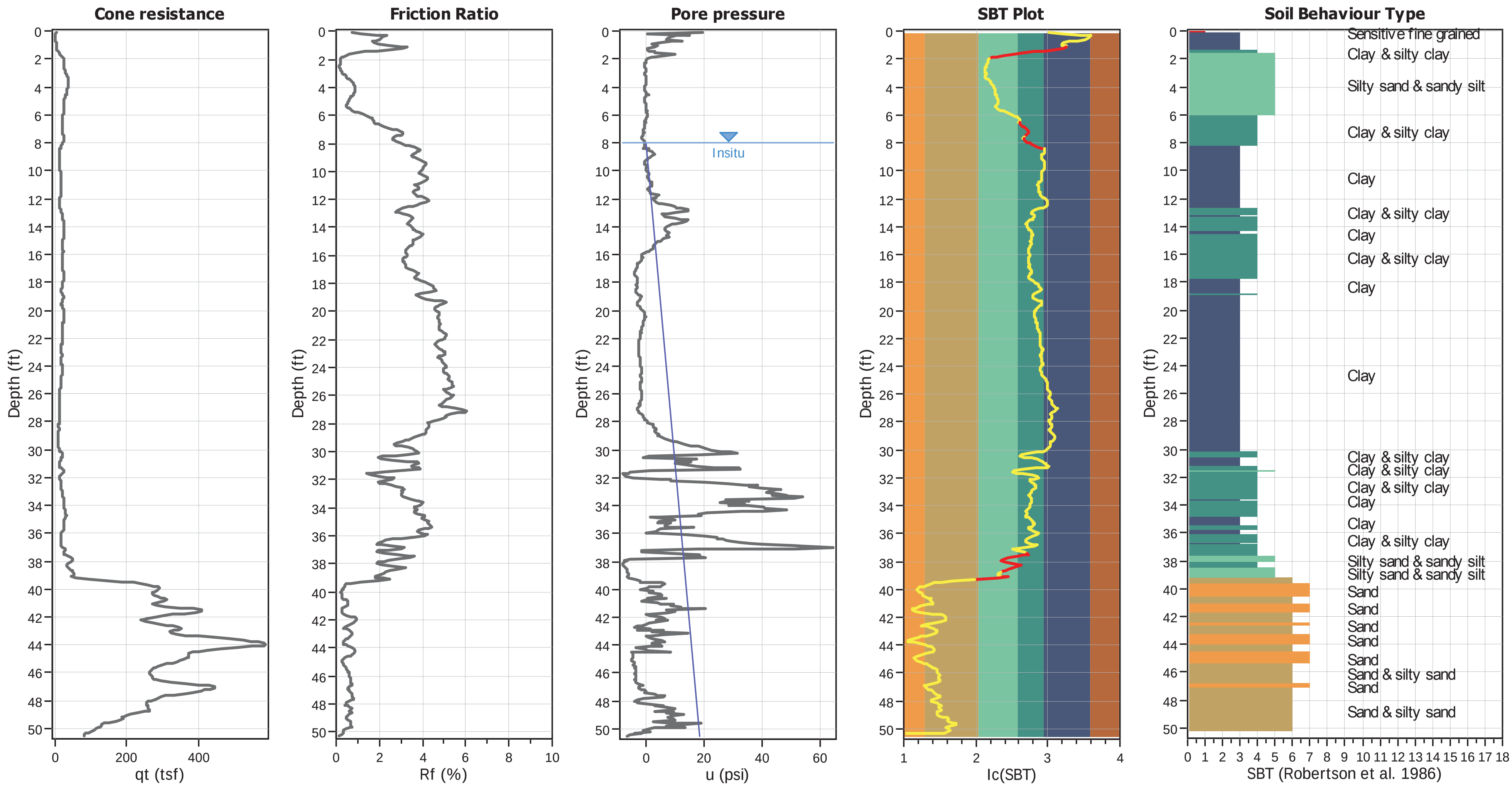






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



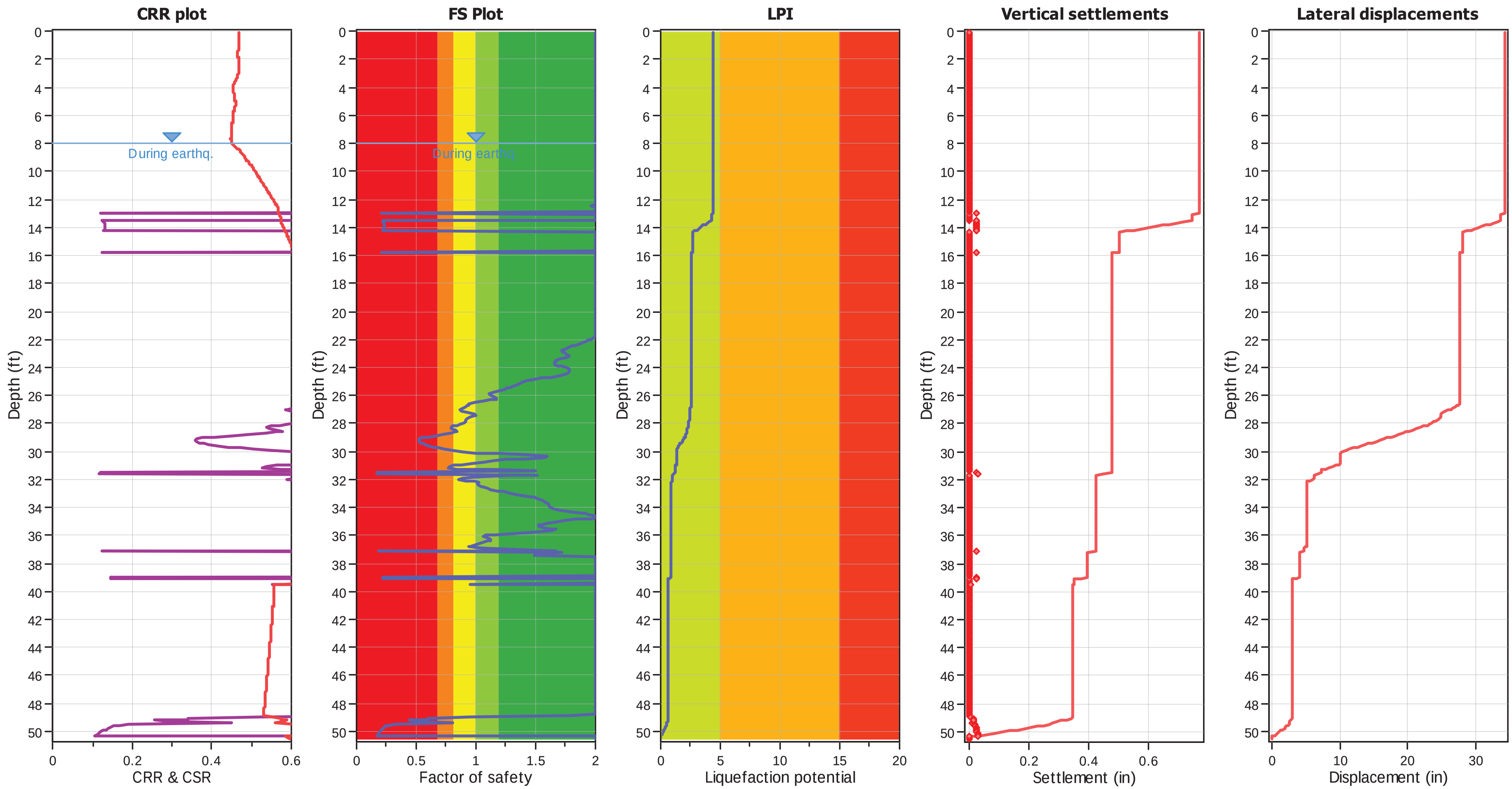
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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San Ramon, CA 94583

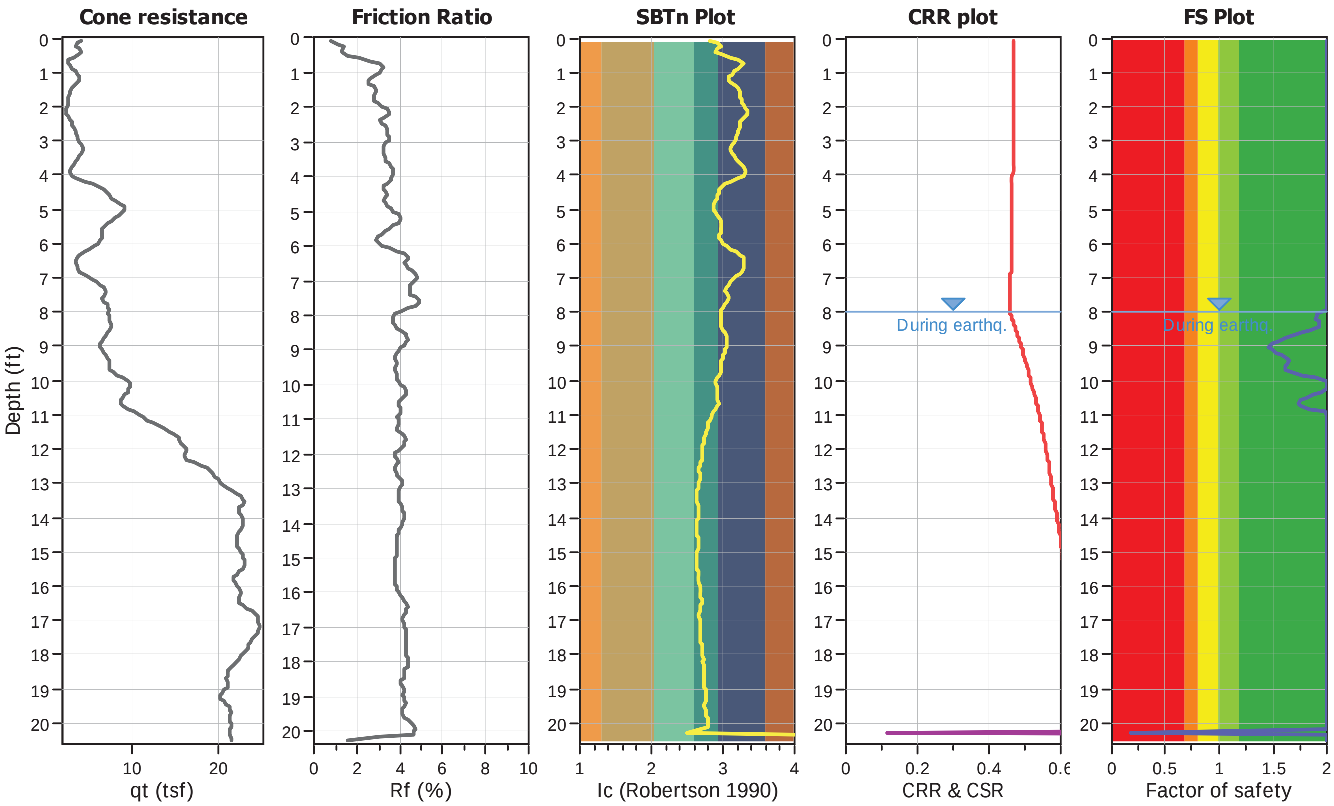
LIQUEFACTION ANALYSIS REPORT

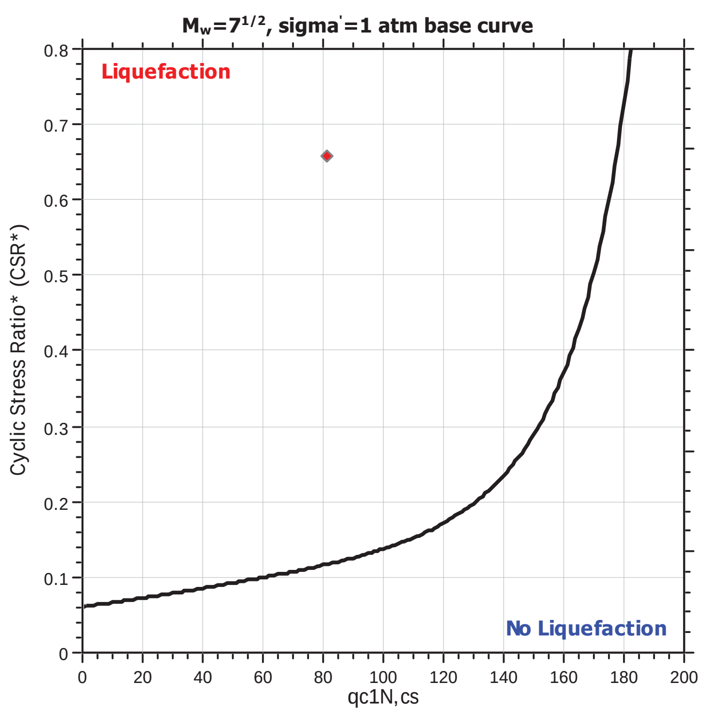
Project title : Hanover North San Jose **Location : 681 East Trimble Road, San Jose**

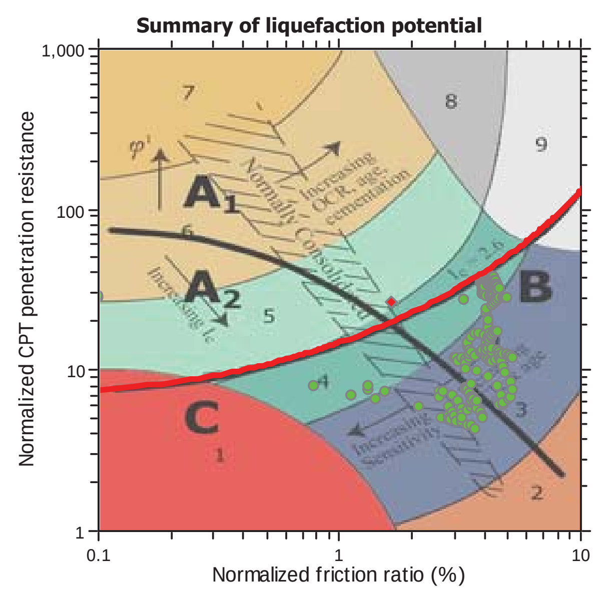
CPT file : 1-CPT6

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 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:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

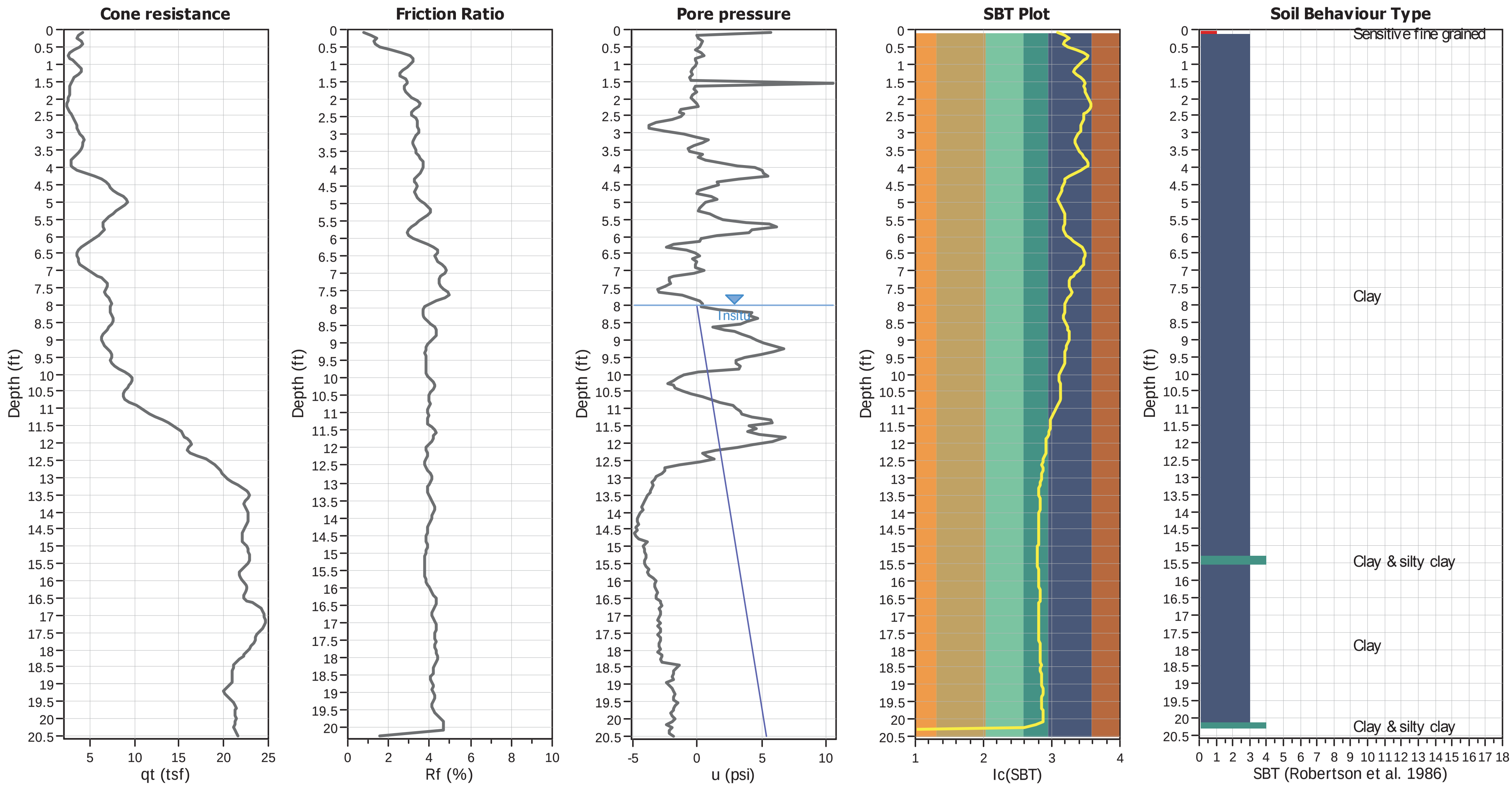






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



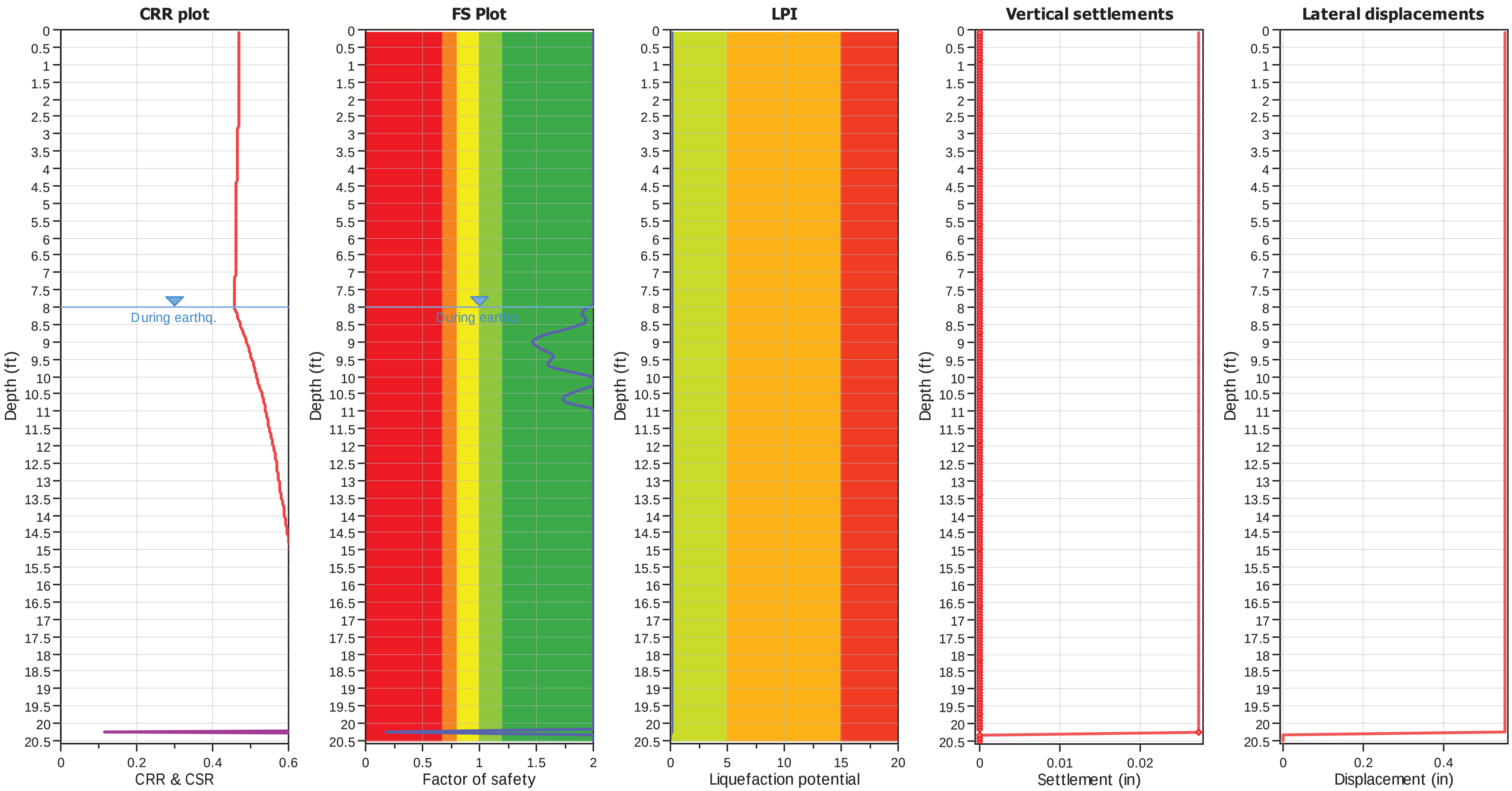
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
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F.S. color scheme

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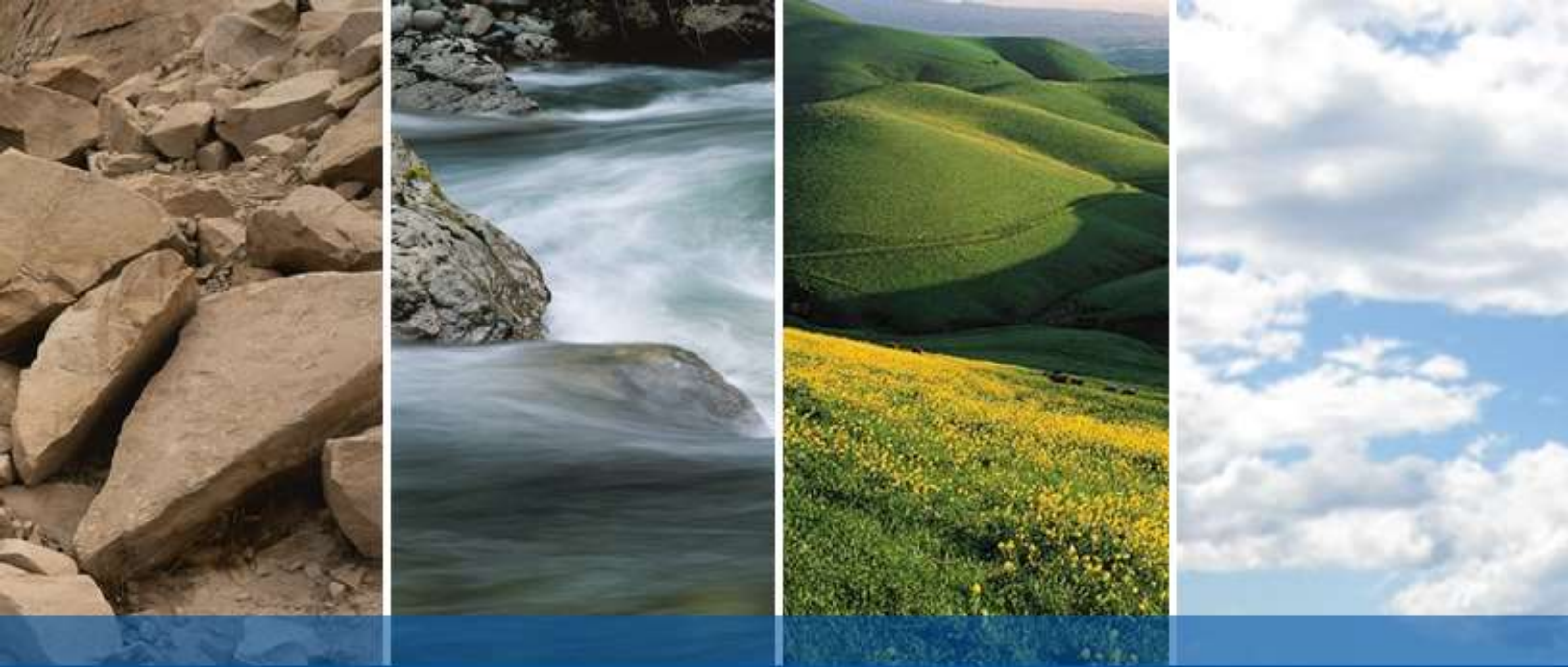
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk



HANOVER NORTH SAN JOSE
SAN JOSE, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Kristen Gates
Hanover R.S. Limited Partnership
1780 South Post Oak Lane
Houston, TX 77056

PREPARED BY
ENGEO Incorporated

August 9, 2021

PROJECT NO.
18233.000.002

ENGEO
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Project No.
18233.000.002

August 9, 2021

Ms. Kristen Gates, PE
Hanover R.S. Limited Partnership
1780 South Post Oak Lane
Houston, TX 77056

Subject: Hanover North San Jose
Seely Avenue
San Jose, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Ms. Gates:

ENGEO prepared this preliminary geotechnical report for the proposed Hanover North San Jose development, as outlined in our agreement dated June 23, 2021. We characterized the subsurface conditions at the site to provide the enclosed preliminary geotechnical recommendations for planning purposes.

Based upon our initial assessment, it is our opinion that the proposed development is feasible from a geotechnical standpoint provided that the recommendations contained in this report are incorporated into planning. A design-level exploration should be conducted prior to site development once more detailed land plans and structural loads have been prepared.

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

Wyatt Iwanaga

Todd Bradford, PE

REGISTERED PROFESSIONAL ENGINEER
TODD BRADFORD
No. 86636
CIVIL
STATE OF CALIFORNIA

Jonas F. Bauer, PE
wi/jb/tb/jam/jf

REGISTERED PROFESSIONAL ENGINEER
JONAS FABIAN BAUER
No. 92547
CIVIL
STATE OF CALIFORNIA

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical exploration, as described in our agreement dated June 23, 2021, is to provide an assessment of the potential geotechnical and geologic concerns for the western half of the overall Hanover North San Jose mixed-use development. The scope of our services included a site visit, a review of published geologic maps, review of readily available geotechnical reports for the site, advancing four borings to evaluate subsurface conditions, laboratory testing of select soil samples, and preparation of this preliminary report discussing potential geotechnical and geologic hazards.

This report was prepared the exclusive of use of Hanover R.S. Limited Partnership and its consultants for initial land planning of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the preliminary conclusions and recommendations contained in this report to determine whether modifications are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

The approximately 11-acre, L-shaped site is located along Seely Avenue, north of the intersection with Montague Expressway in San Jose, California. The site is associated with Assessor's Parcel Number (APN) 097-15-034 and is generally bounded by Seely Avenue to the southwest, agricultural land to the southeast, Coyote Creek to the northeast, and a residential development to the northwest (Figure 1, Vicinity Map).

1.3 PROJECT DESCRIPTION

On the basis of our review of the Site Density Study (Hanover, 2021) and discussions with you, we understand the proposed development will include construction of 149 townhomes, one mixed-use building, and associated paved roads, parking, and improvements. We assume that the townhomes will be up to three stories tall consisting of wood-frame construction. We understand the proposed mixed-use building will consist of a 7-story podium structure with five stories of wood-frame construction over three stories of reinforced concrete.

1.4 SITE BACKGROUND

We reviewed historical aerial photographs and published geologic maps for the site and local vicinity. Based on aerial photographs dated between 1948 and 2016, as well as the existing conditions observed during our site visits, the site appears to have been continuously used for agricultural purposes since the late 1940s. Based on the historical aerial photography, structures have not occupied the site dating back to 1948.

1.5 PREVIOUS GEOTECHNICAL EXPLORATIONS

We previously prepared a preliminary geotechnical report for the eastern half of the proposed Hanover North San Jose development, dated February 23, 2021, and revised March 30, 2021. Our exploration included six cone penetration tests (CPTs), four hand-auger holes, and one direct-push probe at various locations. We incorporate relevant findings of the previous explorations into this report.

2.0 SITE GEOLOGY AND SEISMICITY

2.1 REGIONAL AND SITE GEOLOGY

The region is within the North Coast Range Province of California, an area dominated by northwest-trending geologic features such as folds and faults. The San Francisco Bay is located in a fault-bound, elongated structural trough that has been filled with a sequence of Quaternary age sedimentary deposits derived from the surrounding Coast Ranges. According to mapping by Dibblee (2005) shown in Figure 3, the subject site is located on Quaternary alluvial gravel, sand, and clay (Qa).

2.2 SITE SEISMICITY

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

The San Francisco Bay Area contains numerous active faults. Figure 4 shows the approximate location of active[1] and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region. To determine nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the USGS Unified Hazard Tool[2] and disaggregated the hazard at the peak ground acceleration (PGA) for 2,475-year return period, with the resulting faults listed below in Table 2.2-1.

TABLE 2.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site Latitude: 37.398147, Longitude: -121.917809

SOURCE	R_{RUP} (KM)	R_{RUP} (MILES)	MOMENT MAGNITUDE M_W
Silver Creek [6]	1.1	0.7	6.8
Hayward (So) [1]	7.2	4.5	6.8
Hayward (So) [2]	10.6	6.6	6.9
Calaveras (No) [6]	10.9	6.8	7.3
Calaveras (Central) [9]	11.1	6.9	6.8
San Andreas (Peninsula) [2]	21.9	13.6	8.0

These results represent sources contributing at least one percent to the seismic hazard at the site for the peak ground acceleration and for the given return period. Gridded or areal sources are not presented; however, these sources did not contribute more than one percent to the seismic hazard for the peak ground acceleration and for the given return period.

The Uniform California Earthquake Rupture Forecast (UCERF3, 2014) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault

[1] An active fault is defined by the State as one that has had surface displacement within Holocene time, about the last 11,000 years (Hart and Bryant, 1997).

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

systems in the Bay Area. The UCERF3 generated an overall probability of 72 percent for the San Francisco Region as a whole, a probability of 14.3 percent for the Hayward fault, 7.4 percent for the Calaveras fault, and 6.4 percent for the northern section of the San Andreas fault.

2.3 SURFACE CONDITIONS

This site gradually slopes from northeast to southwest and is currently bare soil with disced grasses and some shrubs and small trees. Based on topographic data obtained from Google Earth, the site elevation ranges from approximately 33 feet on the southwest edge of the site along Seely Avenue to 39 feet on the northeast end of the site along Coyote Creek (WGS84). The site is currently vacant.

At the northeastern edge of the property is a downslope to Coyote Creek. The existing Coyote Creek bank is roughly 15 feet high and inclined at a gradient of approximately 5:1 (horizontal:vertical) and contains walking trails at the top of slope and at roughly mid-slope.

2.4 FIELD EXPLORATIONS

We performed our field exploration on July 2, 2021. Our field explorations included four borings at the approximate locations on the site shown on Figure 2. The exploratory points were roughly located using recreational-grade GPS and should be considered accurate only to the degree implied by the method used.

We retained the services of a subcontractor with a track-mounted drill rig to advance borings to a maximum depth of about 51½ feet below existing grade. One of our borings, 1-B1, was advanced using a combination of hollow-stem auger and mud-rotary methods. The remaining borings were advanced using hollow-stem auger methods.

We permitted and backfilled the borings with grout in accordance with the requirements of the Santa Clara Valley Water District. We observed the drilling and an ENGEO engineer logged the subsurface conditions at each location. We obtained soil samples at various depth intervals using a standard penetration test (SPT) sampler and a California Modified sampler. We obtained the SPT blow counts using an automatic-trip, 140-pound hammer with a 30-inch free fall. We drove the sampler 18 inches and recorded the number of blows for each 6 inches of penetration. The blow counts presented on the bore logs are actual field blow counts and have not been converted using any correction factors.

We used the field logs to develop the report logs in Appendix A. The logs show the soil type, color, consistency, and visual classification at the time of drilling in general accordance with the Unified Soil Classification System.

2.5 LABORATORY TESTING

We performed laboratory tests on select soil samples to determine select engineering properties. We summarize the laboratory testing and standard procedures in the Table 2.5-1. Laboratory test results are included in Appendix B.

TABLE 2.5-1: Laboratory Testing

TEST	DESIGNATION
Determination of Moisture Content by Mass	ASTM D2216
Determination of Density	ASTM D7263
Amount of Material in Soil Finer than No. 200 Sieve	ASTM D1140
Particle-Size Analysis	ASTM D422
Atterberg Limits	ASTM D4318
Unconfined Compression	ASTM D2166
Unconsolidated Undrained Triaxial Compression	ASTM D2850

2.6 SUBSURFACE CONDITIONS

The boreholes generally encountered silty and sandy deposits consisting of loose to medium dense silty sand, or soft to medium stiff sandy silt in the upper 7 to 10 feet below ground surface (bgs) in Borings 1-B2, 1-B3, and 1-B4, and in the upper 20 feet bgs in Boring 1-B1. Below the sandy and silty deposits, the boreholes encountered medium stiff to very stiff lean clay, up to roughly 20 feet thick in the deeper boreholes, which then transitioned into medium dense to dense sand below 40 feet in Boring 1-B1.

Consult the Site Plan and exploration logs for specific subsurface conditions at each location. We include our exploration logs in Appendix A. The logs indicate the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System.

2.7 GROUNDWATER CONDITIONS

Groundwater was encountered at approximately 12 feet below ground surface in Boring 1-B1. Based on the Seismic Hazard Zone Report for the Milpitas Quadrangle (2004), the historically high groundwater level is approximately 8 feet below existing grade. Therefore, we recommend considering a design groundwater depth of 8 feet below existing ground surface. Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

3.0 DISCUSSION AND PRELIMINARY CONCLUSIONS

Based upon this preliminary study, it is our opinion that the project site is feasible for the proposed mixed-used development from a geotechnical standpoint provided the development plans incorporate the preliminary recommendations contained in this report and future design-level geotechnical studies.

As noted, a comprehensive site-specific geotechnical exploration should be performed as part of the design process. The exploration should include additional borings, CPT soundings, and laboratory soil testing to provide data for preparation of specific recommendations regarding grading and foundation design. The exploration will also allow for more detailed evaluations of the geotechnical issues discussed below and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

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

- Disturbed near-surface soil
- Soft and loose native soil
- Liquefaction or cyclic softening
- Creek bank slope stability and structure setbacks

3.1 DISTURBED NEAR-SURFACE SOIL

As previously mentioned, the site has historically been used for agricultural purposes, which has resulted in disturbed near-surface soil. While not encountered in our explorations, localized regions of undocumented fill and potential buried structures or piping may be present from historical agricultural use.

Disturbed near-surface soil and undocumented fill may undergo excessive settlement, especially when subject to new loads from grading and the planned structures. Detailed mapping of subsurface fill materials and potential buried structures or piping should be performed at the time of our design-level study. We present fill and demolition removal recommendations in Section 4.1.

3.2 SOFT AND LOOSE SOIL

The sandy and silty materials encountered site wide in the upper 7 to 10 feet, and up to 20 feet in Boring 1-B1, were found to be relatively soft and loose based on the results of our laboratory testing and the blow counts recorded during our exploration. While structural loads are not available at the time of this report, our experience indicates the proposed structures will likely impose loads greater than the allowable bearing capacity of these relatively soft and loose materials. We anticipate that targeted ground improvement and remedial earthwork activities will be required to support the proposed structures and improvements.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections.

Other hazards considered included regional subsidence or uplift, tsunamis, and seiches. Based on topographic and lithological data, the risk of these hazards is considered low to negligible at the site.

3.3.1 Ground Rupture

The project site is not located within an Alquist-Priolo Earthquake Fault Zone. This indicates that there is a low potential for surface expression of fault rupture.

3.3.2 Ground Shaking

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

3.3.3 Liquefaction/Cyclic Softening

The project site is mapped within a zone of required investigation for liquefaction by the State of California as shown on Figure 5. Soil liquefaction results from loss of strength during cyclic loading, such as that imposed by earthquakes. Clean, loose, saturated, uniformly graded, fine sand below the water table is typically considered the most susceptible soil to liquefaction. Empirical evidence indicates that loose silty sand and sandy silt are also potentially liquefiable.

Seismically induced settlement can be generally subdivided into two categories for cohesionless (sand-like) soil: (1) settlement as a result of liquefaction of saturated or nearly saturated soil, and (2) dynamic densification of non-saturated soil. Research has also shown that low-expansive cohesive (clay-like) soil can also undergo post-seismic settlement.

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting of the liquefied soil to the ground surface. Densification occurs as excess pore pressures dissipate, resulting in vertical settlement at the ground surface. In addition to the above analysis, we also evaluated the capping effect of any overlying non-liquefiable soil. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert sufficient force to break through the overlying soil and vent to the surface, resulting in sand boils or fissures.

Cohesive (clay-like) soil can also develop pore pressures during cyclic loading, but generally do not reach zero effective stress, and are typically considered non-liquefiable (Robertson, 2009). However, cohesive soil can deform during cyclic earthquake loading and experience volumetric strains and post-earthquake reconsolidation. The volumetric strains for cohesive soil are generally small compared to cohesionless soil, since cohesive soil often retains some original soil structure.

To refine our liquefaction analysis, we utilized the screening criteria presented by Bray and Sancio (2006) to assess the potential for liquefaction triggering of the fine-grained soil layers. Bray and Sancio observed that fine-grained soil with a plasticity index (PI) of greater than 18 and a water content to liquid limit ratio (w_c/LL) of less 0.8 was considered not susceptible to liquefaction. Based on the laboratory test results from the soil collected in our exploratory borings, we identified potentially liquefiable materials in Boring 1-B1.

We performed an analysis of liquefaction potential in Boring 1-B1 based on the SPT data collected during our field exploration based on methods outlined by Youd et al. (2001), Seed et al. (2003), and Idriss and Boulanger (2008). The silty sand layer was found to be susceptible to liquefaction below design groundwater level of 8 feet to roughly 20 feet bgs, where the sand transitioned into clay.

3.3.4 Densification and Surface Manifestation

We performed a densification analysis using methods from Idriss and Boulanger (2008). The peak ground acceleration of 0.74g was taken from the mapped seismic design parameters for the site (Table 3.6-1). The predominant earthquake magnitude value of 6.9M was chosen based on the mean value of the disaggregation of the seismic hazard at the site, and the design groundwater level of 8 feet bgs is the assumed historic high level.

Our densification analysis indicates that seismically induced settlements of up to 4 inches may occur in the vicinity of Boring 1-B1. Localized ground improvement measures or earthwork remediation will likely be required to mitigate or reduce this liquefaction settlement. Additional field exploration, sampling, laboratory testing, and evaluation should be conducted during a design-level study to further characterize the soil layers with susceptibility to liquefaction and their lateral extent.

Considering the potential for liquefaction in Boring 1-B1, we performed a preliminary assessment for sand boils using methods by Ishihara (1985) and Youd and Garris (1995). The results indicate the layer of non-liquefiable material above the liquefiable layer is marginal and sand boils may result. Additional analysis will be performed during our design-level study and mitigation measures or site design elements can be developed if sand boils are possible.

3.3.5 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sand or weak soil. Due to the proximity of the Coyote Creek channel and shallow depth of the soil susceptible to liquefaction, lateral spreading is a potential concern at the site. Additional evaluation should be conducted during a design-level study to further characterize potential impacts of lateral spreading and develop remedial measures or setbacks.

3.4 FLOOD ZONE

The project site is mapped within Zone X on the Federal Emergency Management Agency (FEMA 2014) Flood Hazard Map for the City of San Jose, indicating that it is within an area determined to be outside the 0.2 percent annual chance floodplain. The project Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

3.5 CREEK SETBACK

As previously mentioned, Coyote Creek runs along the northeast edge of the project site. For preliminary planning purposes, we recommend maintaining a minimum setback of 50 feet from the top of the slope to habitable structures. The final creek setback will be reviewed during the design-level geotechnical exploration. Additional slope stability analyses may be required if a closer setback is desired, as well as more extensive remedial earthwork and/or pin wall installation.

3.6 2019 CALIFORNIA BUILDING CODE SEISMIC DESIGN

Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2019 CBC. We provide the 2019 CBC seismic design parameters in Table 3.6-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.6-1: 2019 CBC Seismic Design Parameters
Latitude: 37.398147, Longitude: -121.917809

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.60
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.61
Site Coefficient, F_A	1.00
Site Coefficient, F_V	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.60
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.07
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.68
Site Coefficient, F_{PGA}	1.10
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.74

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

For the proposed townhome structures, we estimate the fundamental period of the structures to be less than 1.5 T_S, where T_S is approximately 0.56 second for this site. Therefore, the structural engineer may consider the exception of Section 11.4.8 of ASCE 7-16 for the townhomes.

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

For the mixed-use podium structure, we recommend considering a site-specific ground motion hazard analysis to estimate seismic design parameters.

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

The following recommendations are for preliminary estimating and planning purposes. Final recommendations regarding site grading will be provided as part of the design-level geotechnical exploration.

4.1 GENERAL DEMOLITION AND SITE CLEARING

Grading should begin with the removal of existing structures including their foundations. Underground structures such as buried pipes, septic tanks and leach fields, if any, should also be removed from the project site entirely.

All debris or soft, compressible/loose soil should be removed from any location to be graded, from areas to receive fill or improvements, or those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer's representative in the field at the time of grading.

Areas containing surface vegetation or organic laden topsoil within the areas to be improved should be stripped to an appropriate depth to remove these materials. Tree roots should be removed to a depth of at least 3 feet below existing grade. The amount of actual stripping and depth of tree root removal should be determined in the field by the Geotechnical Engineer's representative at the time of construction. Subject to approval by the Landscape Architect, strippings and organically contaminated soil can be used in landscape areas that do not contain hardscape or improvements sensitive to settlement. Otherwise, such soil should be removed from the project site. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations. It may be feasible to cut and remove the existing vegetation as close to the ground surface as possible and then disc and mix the remaining vegetation into the near-surface soil. This should be evaluated by ENGEO just prior to the commencement of grading.

Excavations resulting from demolition and stripping which extend below final grades should be cleaned to firm undisturbed soil as determined by the Geotechnical Engineer's representative. Once the surface of areas to be graded is prepared as discussed above and below, the surface should then be scarified, moisture conditioned, and compacted.

Re-use of any potentially contaminated material at the site should be evaluated by the project environmental consultant and discussed with the geotechnical consultant. Any soil that is determined to be not contaminated, contains less than 3 percent organics, and is free of debris, is suitable for use as engineered fill. The acceptable fill materials should be determined and confirmed by the Geotechnical Engineer's representative in the field.

Imported fill materials should meet the above requirements and have a Plasticity Index of less than 12. ENGEO should sample and test proposed imported fill materials at least 10 days prior to delivery to the site. Environmental testing should be provided by the import source for our review and approval at least 10 days prior to delivery to the site.

4.2 EXISTING FILLS

Existing fills, including existing utility trench backfill, are considered undocumented and should be subexcavated to expose underlying competent native soil, as approved by the Geotechnical Engineer. The base of excavations should be processed, moisture conditioned, and, as needed, compacted in accordance with the recommendations for engineered fill.

4.3 SHALLOW SOIL TREATMENT

As noted above, the upper 7 to 10 feet of site materials encountered in the borings across the site is soft or loose. Soft and loose materials are susceptible to load-induced and seismically induced settlements.

For planning purposes, we recommend the existing upper 5 feet of site grades be overexcavated and replaced as engineered fill within at least the townhomes footprint (plus 5 feet beyond) prior to placing planned civil fill. If the townhome is in design civil cut, the shallow soil treatment should be a minimum of 5 feet below existing grade and at least 3 feet below pad grade, whichever is greater. The required depth of excavation should be reevaluated during design-level studies.

Extending the above treatment under the podium and into the street and parking lot areas is also recommended but not required if the podium is supported on a deep foundation or shallow foundation with ground improvements, and if minor vertical settlements are acceptable in the street and parking lot areas.

4.4 SLOPES

In general, the following preliminary slope gradient guidelines may be applied for design of both permanent cut and fill slopes.

TABLE 4.4-1: Slope Gradient Guidelines

ALLOWABLE SLOPE GRADIENT (horizontal:vertical)	MAXIMUM ALLOWABLE SLOPE HEIGHT (feet)	
	FILL	CUT
2:1	8	4
3:1	>8	>4

Geogrid reinforcement may be used for construction of 2:1 fill slopes greater than the maximum allowable height presented in the above table. Geogrid slope designs can be developed if this option is considered.

All cut slopes should be viewed by the Engineering Geologist during slope grading for adverse bedding, seepage, or bedrock conditions, which may affect slope stability. In the event that adverse geologic conditions are detected during grading of the cut slopes, overexcavation and reconstruction of these slopes may be necessary. Track rolling to compact faces of slopes is not sufficient. Slopes should be overbuilt and cut back to design grades.

4.5 FILL PLACEMENT

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

- Test Procedures: ASTM D1557
- Required Moisture Content: Not less than 2 percentage points above optimum moisture content
- Minimum Relative Compaction: 90 percent

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material as determined by ASTM D1557.

Soil material used for surficial pad treatments, utility trench backfill, street subgrade, and retaining wall backfill may have other compaction control requirements. These additional requirements will be developed during our design-level exploration.

4.6 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.3 specifies minimum slopes of 5 percent away from foundations within 10 feet. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following.

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.
3. Do not allow water to pond near foundations, pavements, or exterior flatwork.

4.7 STORMWATER INFILTRATION OPPORTUNITY

The near-surface site soil is expected to be remediated and graded as engineered fill and the sandy and silty soil is underlain by clay and denser material. In addition, the existing potential for liquefaction is not conducive with stormwater infiltration. As such, bioretention areas, grassy swales, or permeable pavers should be lined and have subdrains installed. Best Management Practices should assume that limited stormwater infiltration will occur at the site.

4.8 STORMWATER BIORETENTION AREAS

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

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted to between 85 and 90 percent relative compaction and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

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

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area

excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

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

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

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

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

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

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

Primary considerations for foundation design include elastic settlement, seismically induced settlement, and low bearing capacity of the shallow soil. Preliminary screenings indicate that materials susceptible to seismically induced liquefaction and settlement are present in Boring 1-B1, which is below the footprint of the podium structure. Our foundation recommendations are based on the latest site plan, dated June 4, 2021, and we should be informed of any changes in the site plan.

The following preliminary foundation recommendations will be refined during the design-level geotechnical exploration.

5.1 PODIUM STRUCTURE

The podium structure can be supported on a deep foundation such as drilled piers or driven piles. If a deep foundation is desired, foundation design criteria for the preferred system can be provided in the design-level report.

Alternatively, the proposed podium structure can be supported on a conventional shallow foundation system supported by ground improvement to increase bearing capacity, and mitigate static elastic settlement and potential liquefaction-induced settlement. Potential ground improvement options for consideration include drilled displacement columns, or rammed aggregate piers. Based on our experience with these types of ground improvement, a preliminary allowable bearing pressure of 4,000 pounds per square foot (psf) can be considered for a structural mat supported on the ground improvement element. However, further evaluation will be required by the design-build contractor to more precisely delineate the areas requiring ground improvement and assess allowable bearing values.

Soil improvement is typically procured as a design-build element of a project. The selection and design of the ground improvement system should be determined with input from an experienced general contractor. This allows consideration of individual specialty contractors' proprietary means and methods in selecting the most cost-effective approach that meets specific project performance and quality objectives.

5.2 TOWNHOMES

For preliminary purposes, post-tensioned (PT) mat foundations on properly prepared compacted engineered fill may be considered for supporting the proposed townhomes. On a preliminary basis, we recommend that PT mats be a minimum of 10 inches thick or greater and have a thickened edge at least 2 inches greater than the mat thickness. A preliminary allowable bearing pressure of 1,000 psf can be considered for this foundation type.

The actual mat thickness and reinforcement should be determined by the Structural Engineer using the geotechnical recommendations in the future design-level report, which may also include seismically induced settlements to incorporate into the final design.

5.3 SLAB MOISTURE VAPOR REDUCTION

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

1. Install a vapor retarder membrane directly beneath the mat or slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E 1745 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs."
2. Concrete should have a concrete water-cement ratio of no more than 0.5.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water-cement ratio are used.
4. Consider and implement adequate moist cure procedures for mat foundations.
5. Protect foundation subgrade soil from seepage by providing impermeable plugs within utility trenches.

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

6.0 PRELIMINARY RETAINING WALL RECOMMENDATIONS

For preliminary purposes, unrestrained drained site retaining walls constructed on level ground may be designed using an active equivalent fluid weight of 50 pounds per cubic foot (pcf). The friction factor for sliding resistance may be assumed as 0.30. We recommend a preliminary allowable bearing pressure of 1,000 psf in native soil, or 2,000 if supported on at least 3 feet of engineered fill.

The Geotechnical Engineer should be consulted on wall design values where surcharge loads, such as from permanent structures and automobiles, are expected or where slopes exist above or below a proposed wall. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations. A specialty consultant should be consulted regarding retaining wall waterproofing.

Drainage facilities should be installed behind retaining walls to prevent the build-up of hydrostatic pressures on the walls. For planning purposes, wall drainage may be provided using 4-inch-diameter perforated (SDR 35 or approved equivalent) pipe encapsulated in either Class 2 permeable material, or a free-draining gravel surrounded by synthetic filter fabric (minimum 6-ounce). The width of the drainage medium should be at least 12 inches and should extend from base of the wall to about 1 foot below the finished soil subgrade. The upper 1 foot of wall backfill should consist of on-site compacted soil. If pre-fabricated drain panels are to be considered, in lieu of the drainage medium above the pipe/rock bulb, the contractor should submit their materials packet for our review prior to order and delivery. The subdrain should be collected and discharged through a solid pipe to an outlet approved by the Civil Engineer, such as through the curb and into the street, into an area drain, or into a storm drain manhole.

7.0 PRELIMINARY PAVEMENT DESIGN

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

TABLE 7.0-1: Preliminary Pavement Sections

TRAFFIC INDEX	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	4	8
6	4½	10½
7	5	14

Notes: AC is asphaltic concrete
AB is aggregate base Class 2 Material with minimum R = 78

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

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

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

8.0 FUTURE STUDIES

As previously discussed, a site-specific design-level geotechnical exploration should be performed as part of the design process. The exploration should include supplemental borings and cone penetration tests, and subsequent laboratory testing to provide additional data for evaluation of liquefaction susceptibility. The design-level report will also provide specific recommendations regarding grading, foundation design, retaining wall design, and drainage for the proposed development.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents preliminary geotechnical recommendations for planning and preliminary design of the improvements discussed in Section 1.3 for the proposed Hanover North San Jose development. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and preliminary recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The preliminary conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance. We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is provided, either express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

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

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

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

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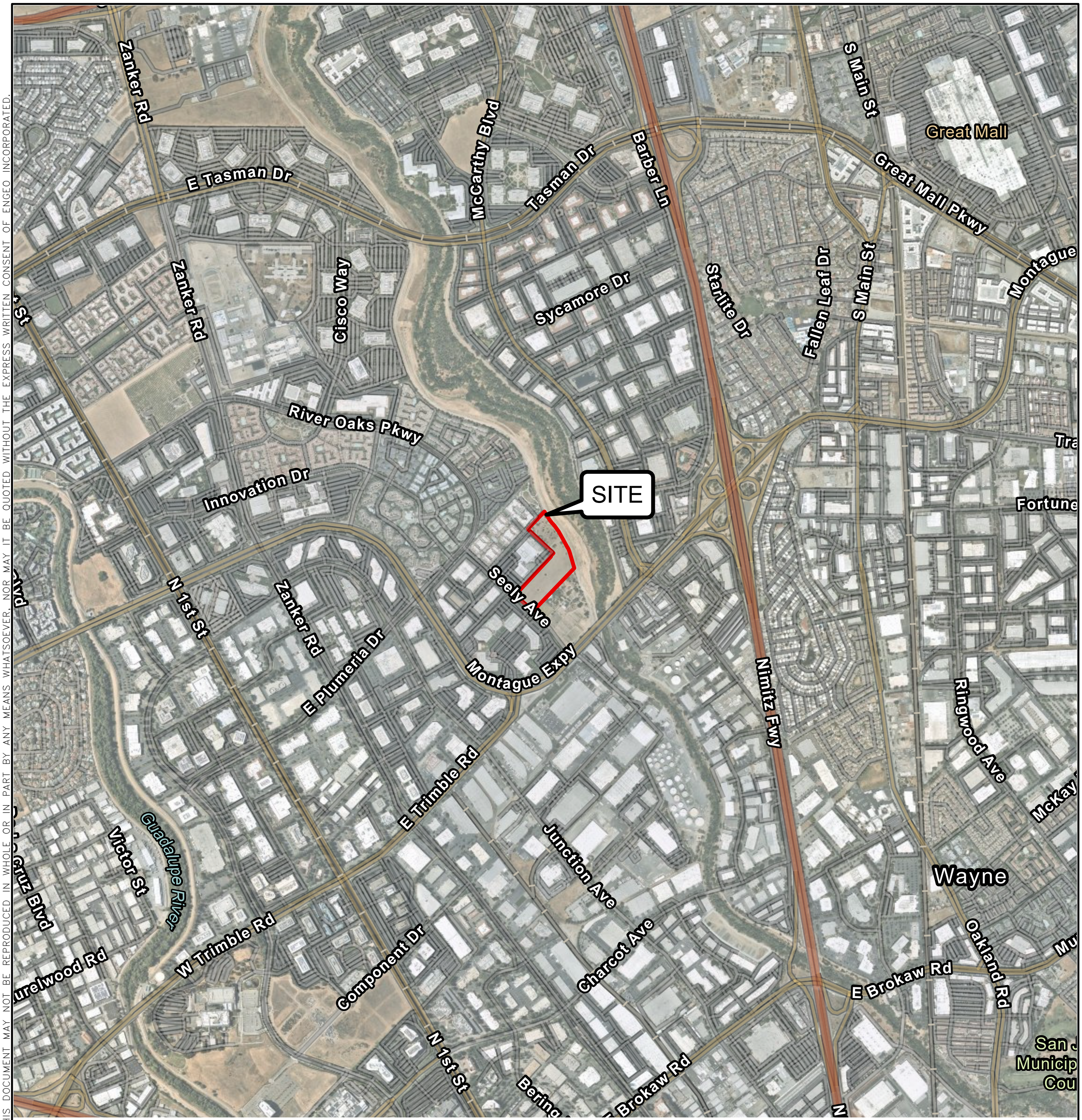
FIGURES



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FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE 5/22/2021

ENGEO
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VICINITY MAP

HANOVER NORTH SAN JOSE

SAN JOSE, CALIFORNIA

PROJECT NO. :	18233.000.002
SCALE:	AS SHOWN
DRAWN BY: QRL	CHECKED BY: JB

FIGURE NO.
1

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\18233\18233000002\GEO\GEX\HANOVER NORTH SAN JOSE.APRX
LAYOUT: VICINITY USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROJECT SITE

BORING (ENGEO, 2021)

N

0 100 200

FEET

BASEMAP SOURCE: NEARMAP MAPPING SERVICE 5/22/2021, TRIMBLE TOWNHOME 2021

ENGEO
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SITE PLAN

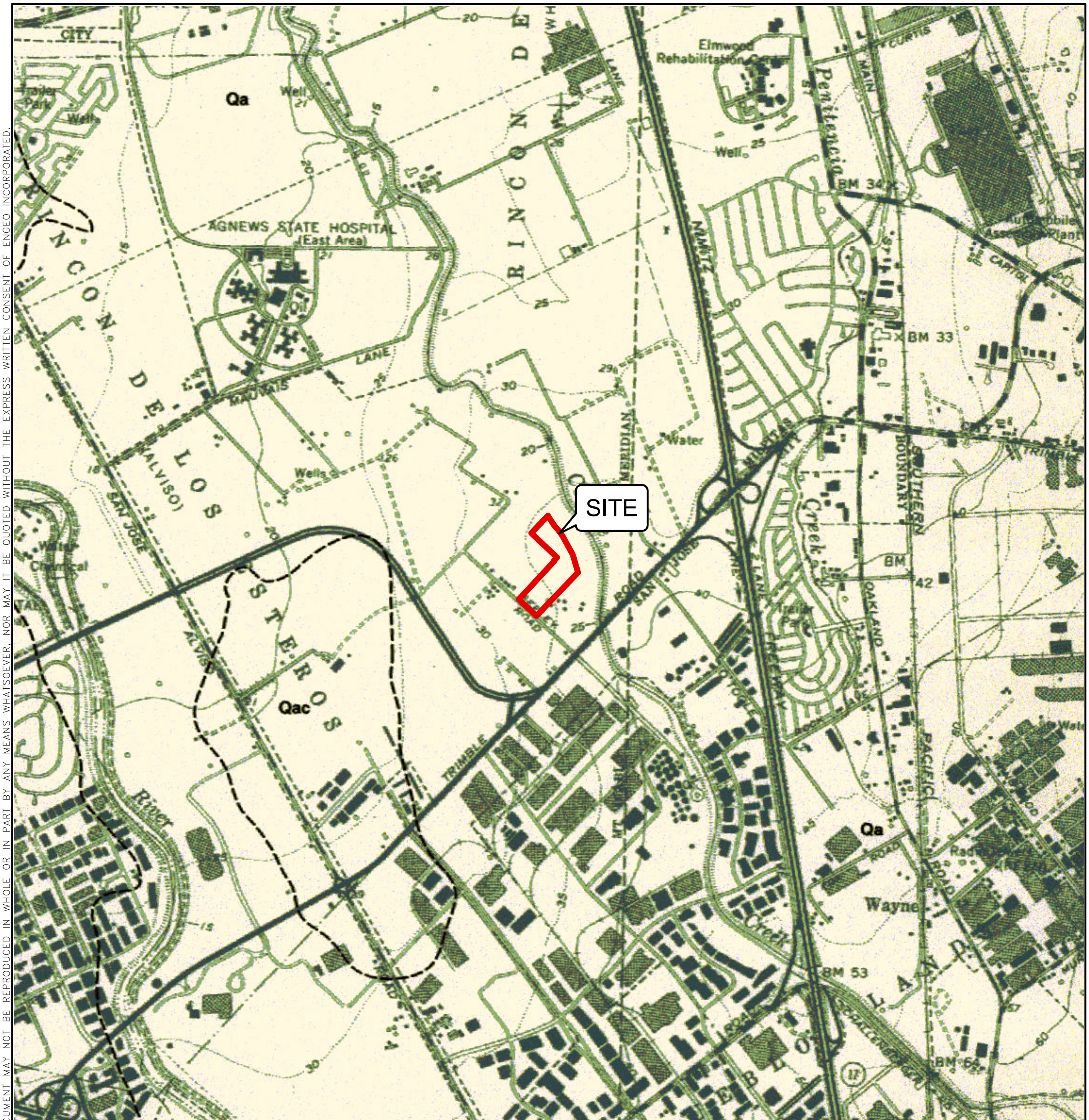
HANOVER NORTH SAN JOSE

SAN JOSE, CALIFORNIA

PROJECT NO. : 18233.000.002		FIGURE NO. 2
SCALE: AS SHOWN		
DRAWN BY: QRL	CHECKED BY: JB	

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\18233\18233000002\GEO\GEX\HANOVER NORTH SAN JOSE.APRX
LAYOUT: SITE PLAN USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qa	ALLUVIUM
Qac	ALLUVIAL CLAY SOIL,INCLUDES BAY MUD

N

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FEET

BASEMAP SOURCE: DIBLEE, 2005

ENGEO
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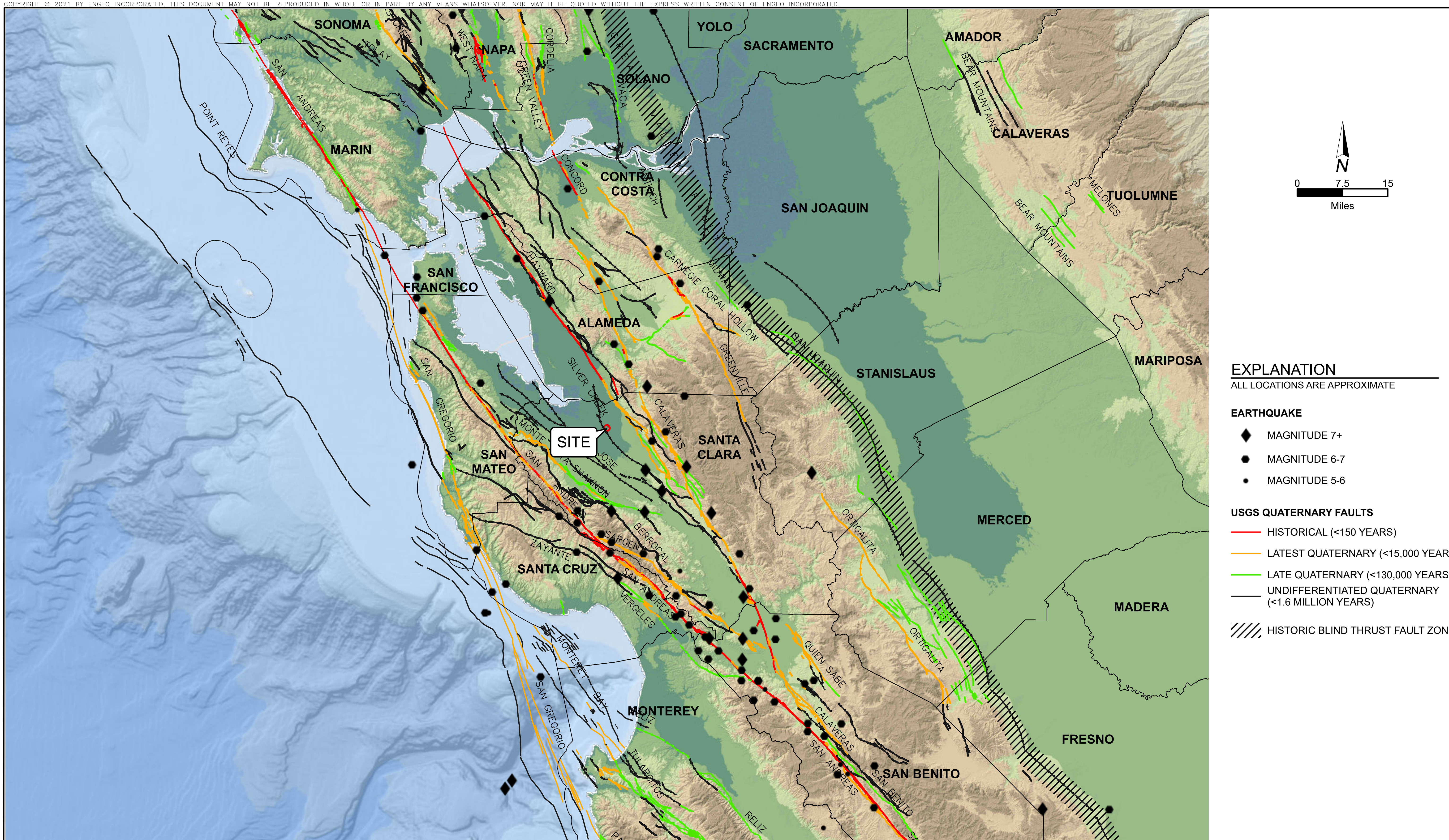
REGIONAL GEOLOGIC MAP HANOVER NORTH SAN JOSE SAN JOSE, CALIFORNIA	PROJECT NO. : 18233.000.002		FIGURE NO. 3
	SCALE: AS SHOWN		
	DRAWN BY: QRL	CHECKED BY: JB	

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\18233\18233000002\GEO\GEX\HANOVER NORTH SAN JOSE.APRX
LAYOUT: GEOLOGIC USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



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

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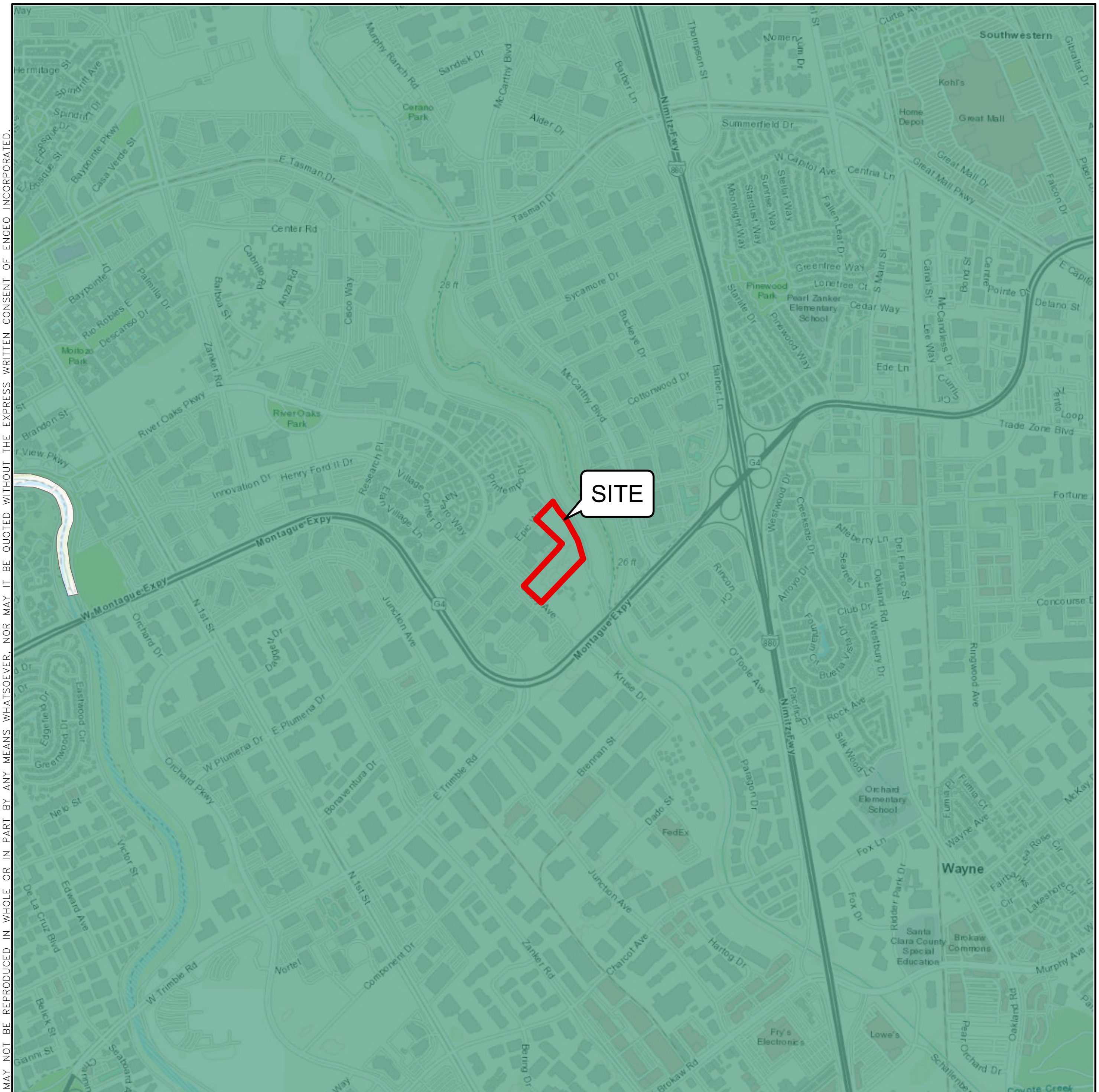
REGIONAL FAULTING AND SEISMICITY
HANOVER NORTH SAN JOSE
SAN JOSE, CALIFORNIA

PROJECT NO.: 18233.000.002	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: JB

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\18233\18233000002\GEO\GEX\HANOVER NORTH SAN JOSE.APRX
LAYOUT: FAULT&SEISMICITY USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



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FEET

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

LIQUEFACTION ZONE
AREAS WHERE THE HISTORICAL OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTEHNICAL AND GROUND WATER CONDITIONS INDICATE A POTENTIAL L FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

BASEMAP SOURCE: ESRI MAPPING SERVICE
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY

ENGEO Expect Excellence	SEISMIC HAZARDS ZONE MAP HANOVER NORTH SAN JOSE SAN JOSE, CALIFORNIA	PROJECT NO. : 18233.000.002 SCALE: AS SHOWN DRAWN BY: QRL CHECKED BY: JB	FIGURE NO. 5

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\18233\18233000002\GEO\GEX\HANOVER NORTH SAN JOSE.APRX
LAYOUT: SEISMIC HAZARD USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

Boring Logs

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

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

SAMPLER SYMBOLS

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

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

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LOG OF BORING 1-B1

LATITUDE: 37.3974 LONGITUDE: -121.91855

Prelim. Geotechnical Exploration
Hanover North San Jose
San Jose, CA
18233.000.002

DATE DRILLED: 7/2/2021
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS84): Approx. 37 ft.

LOGGED / REVIEWED BY: W.Iwanaga / J.Bauer
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: HSA/Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	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), yellowish brown, dry, fine- to medium-grained sand, loose											
	35												
				9	NP	NP	NP	31	3.7	96.3	369		UC
5													
				9					6.5	96			
		Grades to coarser sand, trace fine gravels											
	30												
		Becomes medium dense											
				14									
10													
				13									
		2 inch thick gravel lenses present											
			▽										
	25												
15													
		Grades to silty sand, becomes wet, loose											
				5	NP	NP	NP	39	22.1	108	767		UU
	20												
				2									
20		- - - - - - - - - - - - - - - -											

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LOG OF BORING 1-B1

LATITUDE: 37.3974 LONGITUDE: -121.91855

Prelim. Geotechnical Exploration
Hanover North San Jose
San Jose, CA
18233.000.002

DATE DRILLED: 7/2/2021
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS84): Approx. 37 ft.

LOGGED / REVIEWED BY: W.Iwanaga / J.Bauer
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: HSA/Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

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
			LEAN CLAY (CL), grayish brown mottled with brown, very stiff, moist, trace fines			18									
	15														
25			Grades to stiff to very stiff			16					22	108.1	1555		UU
			Becomes yellowish brown												
	10														
30			Becomes softer			6					26.2	100.8	721		UU
	5														
35			Becomes medium stiff to stiff			16					23.9	104.5	600*	2.0*	PP+TV
	0														
40															

LOG OF BORING 1-B1

ENGEO
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LATITUDE: 37.3974 LONGITUDE: -121.91855

Prelim. Geotechnical Exploration
Hanover North San Jose
San Jose, CA
18233.000.002

DATE DRILLED: 7/2/2021
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS84): Approx. 37 ft.

LOGGED / REVIEWED BY: W.Iwanaga / J.Bauer
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: HSA/Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

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), gray, medium dense, wet, fine- to medium-grained sand			31									
	-5														
45			POORLY GRADED SAND (SP), brownish gray, dense, wet, fine- to coarse-grained sand, fine to coarse gravel lenses			31									
	-10														
50			Becomes very dense			55									
			Boring terminated at 51.5 feet below ground surface (bgs). Groundwater encountered at 12 feet bgs. Boring backfilled with grout.												

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LOG OF BORING 1-B2

LATITUDE: 37.39792 LONGITUDE: -121.91722

Prelim. Geotechnical Exploration
Hanover North San Jose
San Jose, CA
18233.000.002

DATE DRILLED: 7/2/2021
HOLE DEPTH: 16.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS84): Approx. 38 ft.

LOGGED / REVIEWED BY: W.Iwanaga / J.Bauer
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

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
		SANDY CLAYEY SILT (CL-ML), brown, soft to medium stiff, dry										
	35		9	26	20	6	62	6.6	88	435		UC
5		Becomes moist, stiff	11									
	30	LEAN CLAY (CL), brown, stiff, moist	13	36	22	14	95	15.2				
10												
		Becomes very stiff	23	39	21	18	97	16.9	100.1	2942		UC
	25											
15			25									
		Boring terminated at 16.5 feet below ground surface (bgs). No groundwater encountered. Boring backfilled with grout.										

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LOG OF BORING 1-B3

LATITUDE: 37.39895 LONGITUDE: -121.91706

Prelim. Geotechnical Exploration
Hanover North San Jose
San Jose, CA
18233.000.002

DATE DRILLED: 7/2/2021
HOLE DEPTH: 26.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS84): Approx. 39 ft.

LOGGED / REVIEWED BY: W.Iwanaga / J.Bauer
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

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
		SANDY SILT (ML), brown, soft to medium stiff, dry, fine-grained sand										
	35		7	24	21	3	65	8	88.5	583		UC
5		Becomes moist	8								3.25*	PP
		LEAN CLAY (CL), brown, medium stiff, moist	5									
10	30		9					21.8	102.6	1709		UU
15	25	Becomes very stiff	21							2500*	2.25*	PP+TV
20	20											

LOG OF BORING 1-B3

ENGEO
Expect Excellence

LATITUDE: 37.39895 LONGITUDE: -121.91706

Prelim. Geotechnical Exploration
Hanover North San Jose
San Jose, CA
18233.000.002

DATE DRILLED: 7/2/2021
HOLE DEPTH: 26.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS84): Approx. 39 ft.

LOGGED / REVIEWED BY: W.Iwanaga / J.Bauer
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

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
			LEAN CLAY (CL), brown, very stiff, moist			26							3000*	3.5*	PP+TV
	15														
25						27							2500*	3.5*	PP+TV
			Boring terminated at 26.5 feet below ground surface (bgs). No groundwater encountered. Boring backfilled with grout.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 18233000002_BORING LOGS.GPJ ENGEO INC.GDT 8/5/21

LOG OF BORING 1-B4

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LATITUDE: 37.39988 LONGITUDE: -121.91792

Prelim. Geotechnical Exploration
Hanover North San Jose
San Jose, CA
18233.000.002

DATE DRILLED: 7/2/2021
HOLE DEPTH: 16.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS84): Approx. 35 ft.

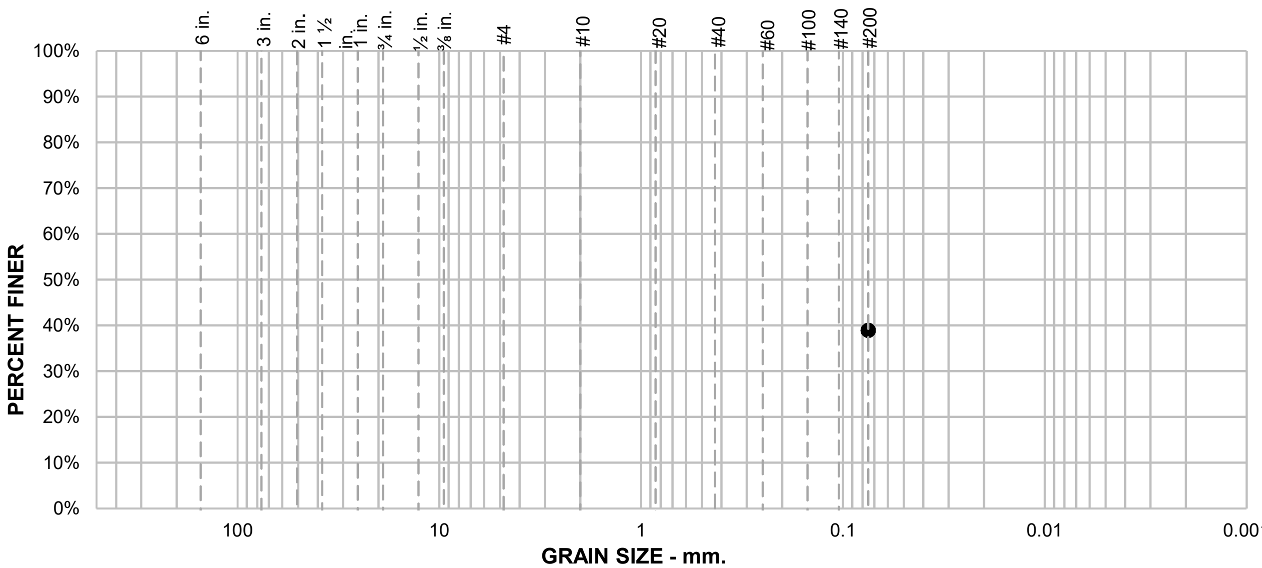
LOGGED / REVIEWED BY: W.Iwanaga / J.Bauer
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

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
		SANDY CLAYEY SILT (CL-ML), brown, stiff, dry										
			9	25	19	6		8.3	89.4	755		UC
5	30	Becomes dry to moist	12								3.25*	PP
			11									
10	25	LEAN CLAY (CL), brown mottled with gray, very stiff, moist										
			21	44	22	22		19.6			4.5*	PP
15	20											
			20								3.25*	PP
		Boring terminated at 16.5 feet below ground surface (bgs). No groundwater encountered. Boring backfilled with grout.										

APPENDIX B

Laboratory Test Results

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 1-B1@16'

DEPTH (ft): 16

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						38.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	38.9		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = NP	LL = NV	PI = NP

COEFFICIENTS

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

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Soak time = 190 min
Dry sample weight = 256.6 g

* (no specification provided)

CLIENT: Hanover R.S. Limited Partnership

PROJECT NAME: Hanover North San Jose - Phase I and II

PROJECT NO: 18233.000.002 PH002

PROJECT LOCATION: San Jose, CA

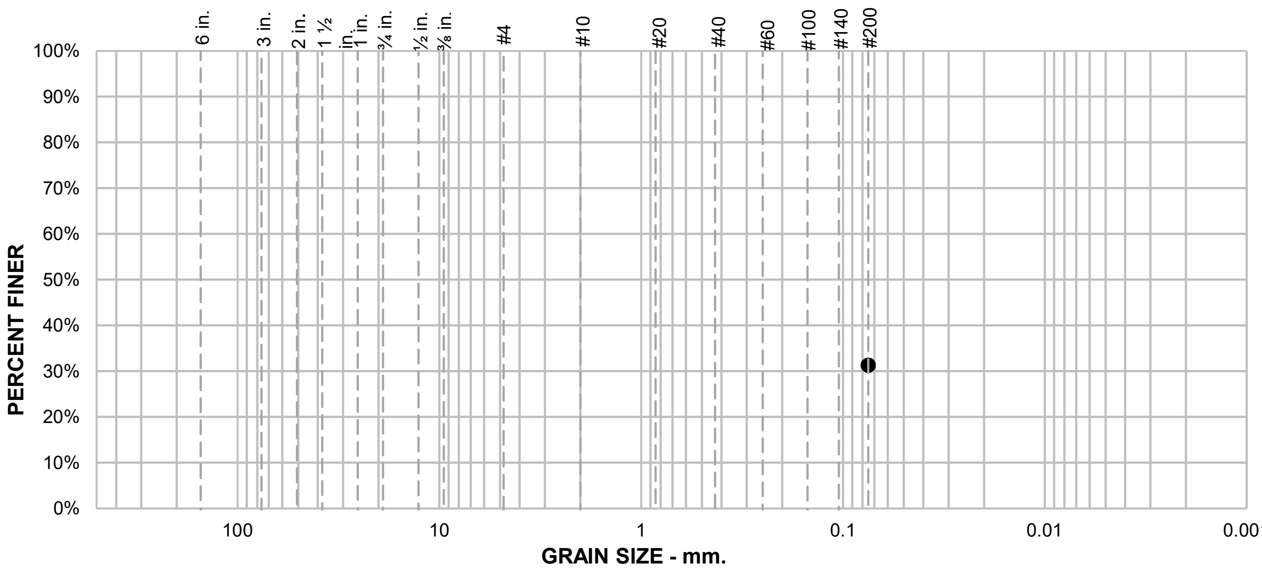
REPORT DATE: 7/20/2021

TESTED BY: V. Navarro

REVIEWED BY: K. Lecce

ENGEO Expect Excellence

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 1-B1@3.5'

DEPTH (ft): 3.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						31.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	31.3		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = NP   LL = NV   PI = NP

COEFFICIENTS

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

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Soak time = 180 min
Dry sample weight = 220.1 g

* (no specification provided)

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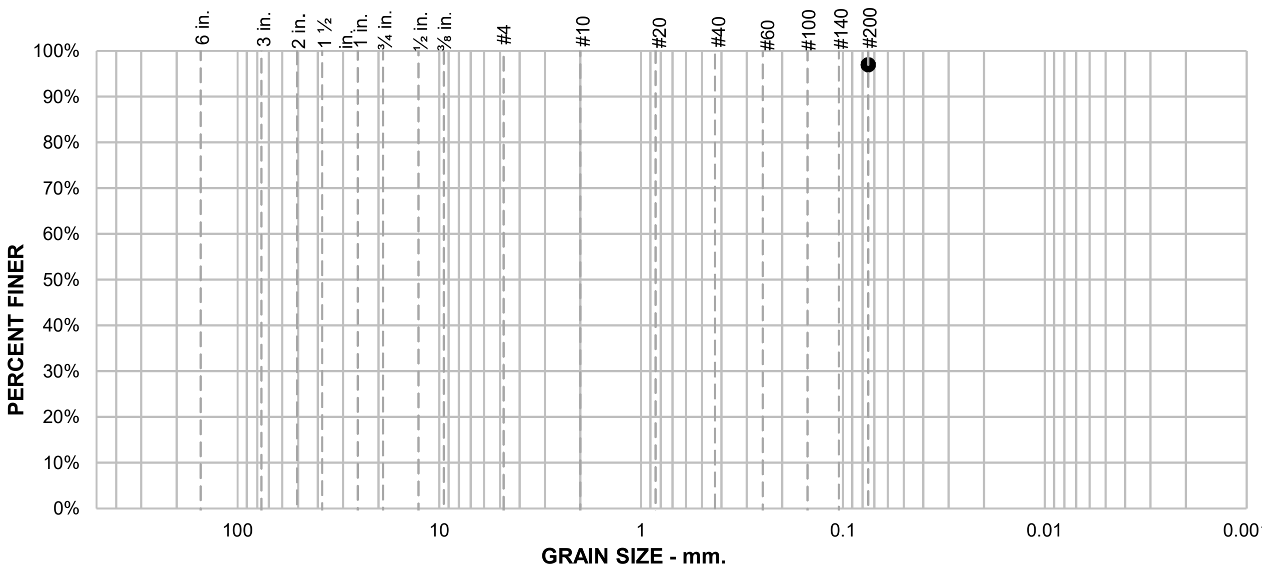
CLIENT: Hanover R.S. Limited Partnership
PROJECT NAME: Hanover North San Jose - Phase I and II
PROJECT NO: 18233.000.002 PH002
PROJECT LOCATION: San Jose, CA
REPORT DATE: 7/20/2021
TESTED BY: V. Navarro
REVIEWED BY: K. Lecce

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@12'

DEPTH (ft): 12

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						96.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	96.9		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 21 LL = 39 PI = 18

COEFFICIENTS

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

CLASSIFICATION

USCS = CL

REMARKS

PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Soak time = 220 min
Dry sample weight = 414.3 g

* (no specification provided)

CLIENT: Hanover R.S. Limited Partnership

PROJECT NAME: Hanover North San Jose - Phase I and II

PROJECT NO: 18233.000.002 PH002

PROJECT LOCATION: San Jose, CA

REPORT DATE: 7/20/2021

TESTED BY: V. Navarro

REVIEWED BY: K. Lecce

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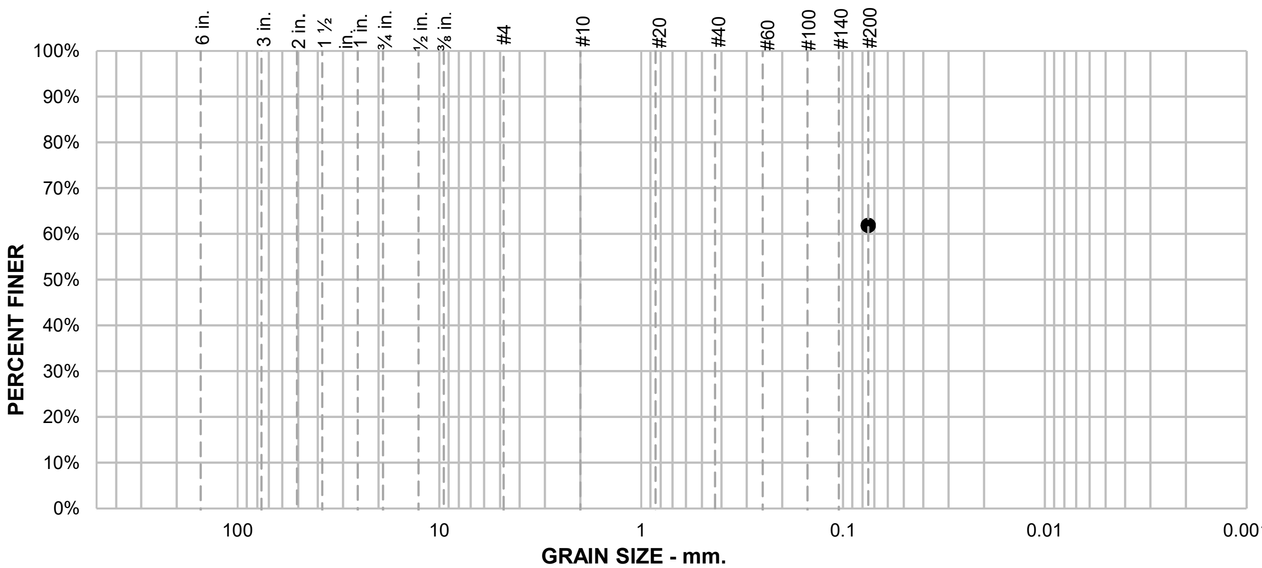
PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 1-B2@3.5'

DEPTH (ft): 3.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						61.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	61.8		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 20 LL = 26 PI = 6

COEFFICIENTS

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

CLASSIFICATION

USCS = CL-ML

REMARKS

PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Soak time = 200 min
Dry sample weight = 280.2 g

* (no specification provided)

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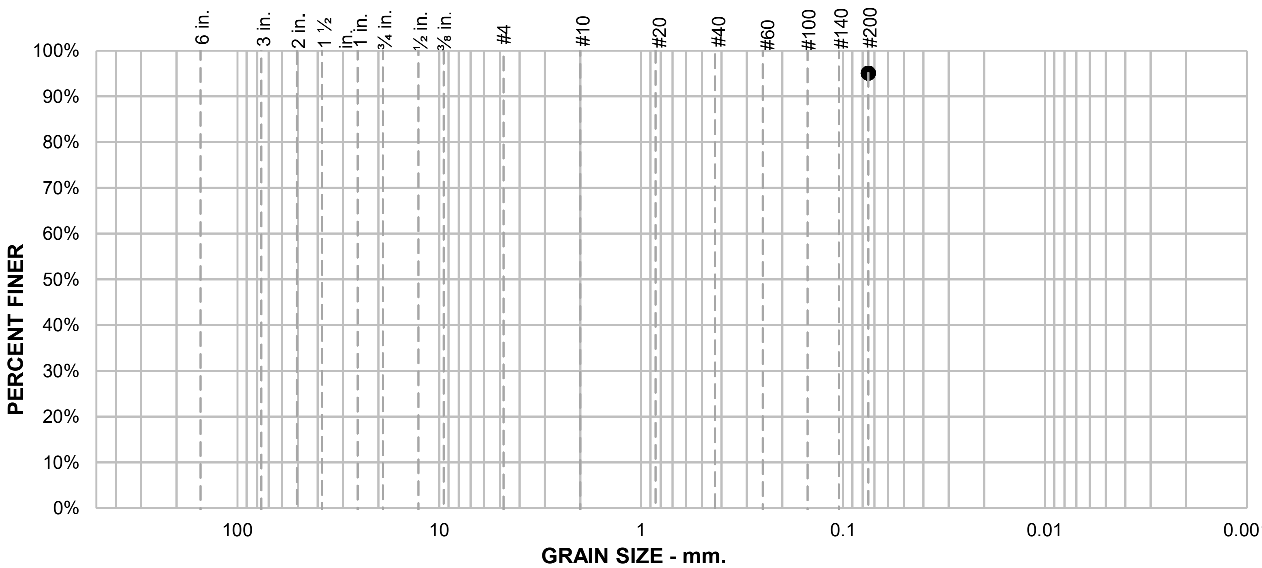
CLIENT: Hanover R.S. Limited Partnership
PROJECT NAME: Hanover North San Jose - Phase I and II
PROJECT NO: 18233.000.002 PH002
PROJECT LOCATION: San Jose, CA
REPORT DATE: 7/20/2021
TESTED BY: V. Navarro
REVIEWED BY: K. Lecce

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 1-B2@8'

DEPTH (ft): 8

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						95.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	95.0		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 22 LL = 36 PI = 14

COEFFICIENTS

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

CLASSIFICATION

USCS = CL

REMARKS

PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Soak time = 210 min
Dry sample weight = 201.2 g

* (no specification provided)

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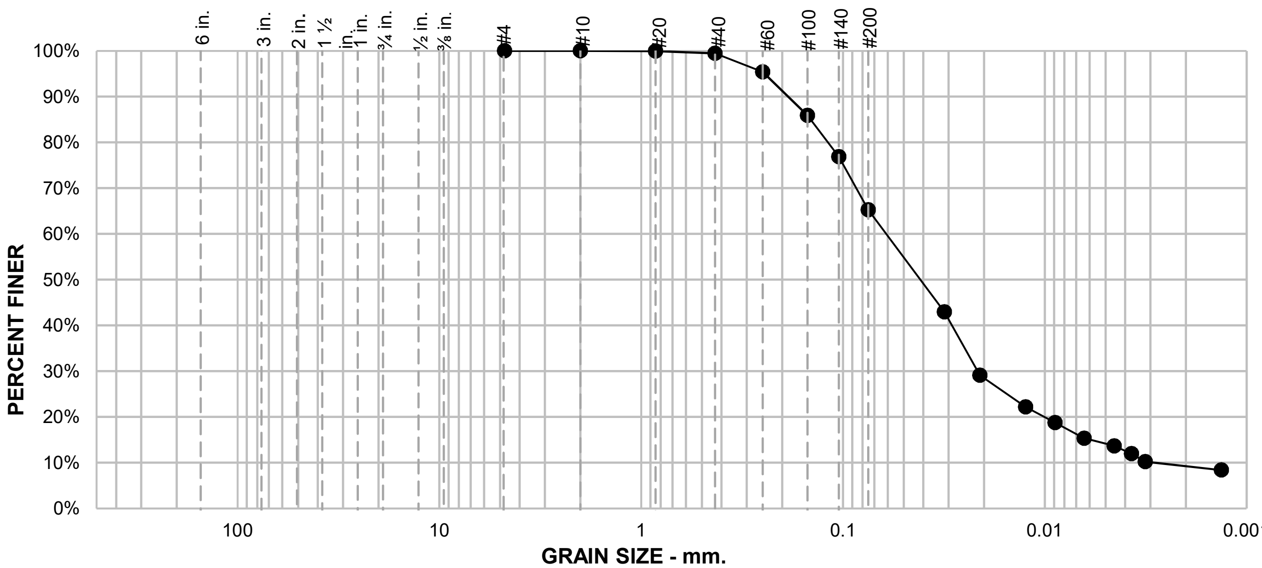
CLIENT: Hanover R.S. Limited Partnership
PROJECT NAME: Hanover North San Jose - Phase I and II
PROJECT NO: 18233.000.002 PH002
PROJECT LOCATION: San Jose, CA
REPORT DATE: 7/20/2021
TESTED BY: V. Navarro
REVIEWED BY: K. Lecce

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D422



SAMPLE ID: 1-B3@3.5'

DEPTH (ft): 3.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
				0.6	34.2	56.0	9.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	99.9		
#40	99.4		
#60	95.4		
#100	85.9		
#140	76.9		
#200	65.2		
0.0314 mm.	42.9		
0.0210 mm.	29.1		
0.0125 mm.	22.2		
0.0089 mm.	18.8		
0.0064 mm.	15.3		
0.0045 mm.	13.7		
0.0037 mm.	12.0		
0.0032 mm.	10.2		
0.0013mm.	8.4		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 21 LL = 24 PI = 3

COEFFICIENTS

D_{90} = 0.1870 mm D_{85} = 0.1447 mm D_{60} = 0.0612 mm
D_{50} = 0.0414 mm D_{30} = 0.0216 mm D_{15} = 0.0060 mm
D_{10} = 0.0029 mm C_u = 21.24 C_c = 2.64

CLASSIFICATION

USCS = ML

REMARKS

Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

* (no specification provided)

CLIENT: Hanover R.S. Limited Partnership
PROJECT NAME: Hanover North San Jose - Phase I and II
PROJECT NO: 182333.000.002 PH001
PROJECT LOCATION: San Jose, CA
REPORT DATE: 7/20/2021
TESTED BY: V. Navarro
REVIEWED BY: K. Lecce

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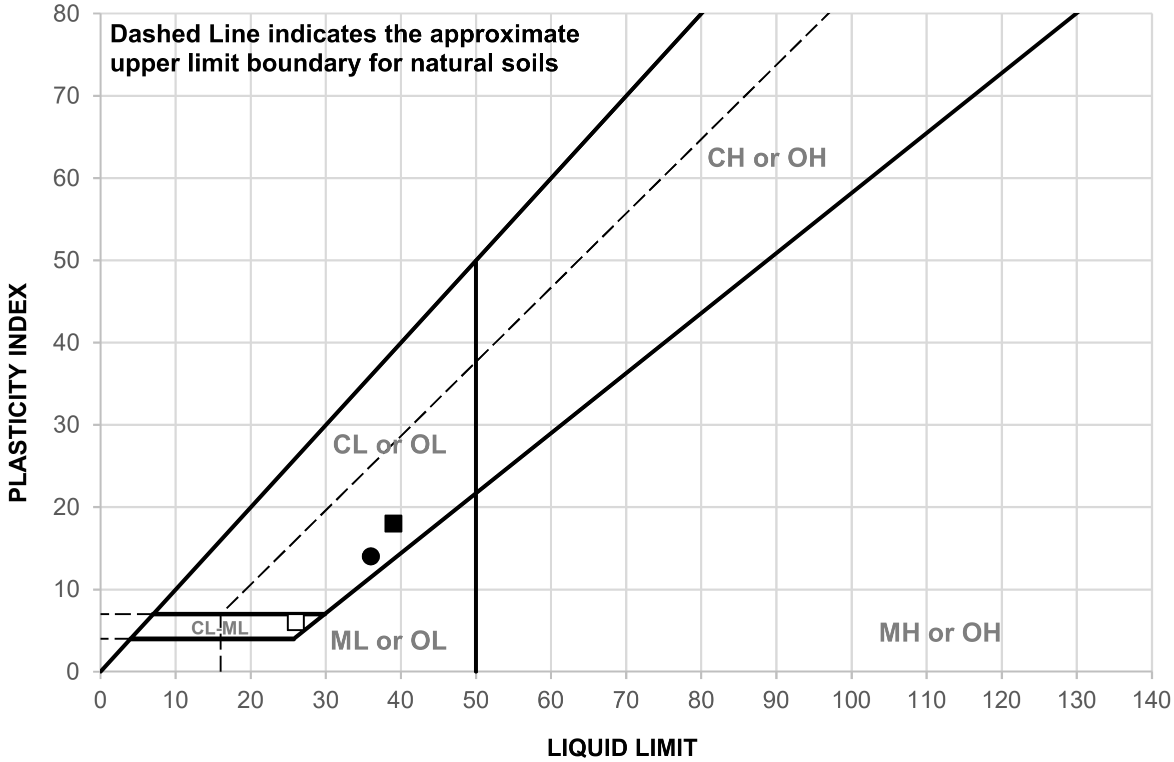
LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1@3.5'	3.5 feet	See exploration logs	NV	NP	NP
◆	1-B1@16'	16 feet	See exploration logs	NV	NP	NP
□	1-B2@3.5'	3.5 feet	See exploration logs	26	20	6
●	1-B2@8'	8 feet	See exploration logs	36	22	14
■	1-B2@12'	12 feet	See exploration logs	39	21	18

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1@3.5'	PI: ASTM D4318, Wet Method	
◆	1-B1@16'	PI: ASTM D4318, Wet Method	
□	1-B2@3.5'	PI: ASTM D4318, Wet Method	
●	1-B2@8'	PI: ASTM D4318, Wet Method	
■	1-B2@12'	PI: ASTM D4318, Wet Method	

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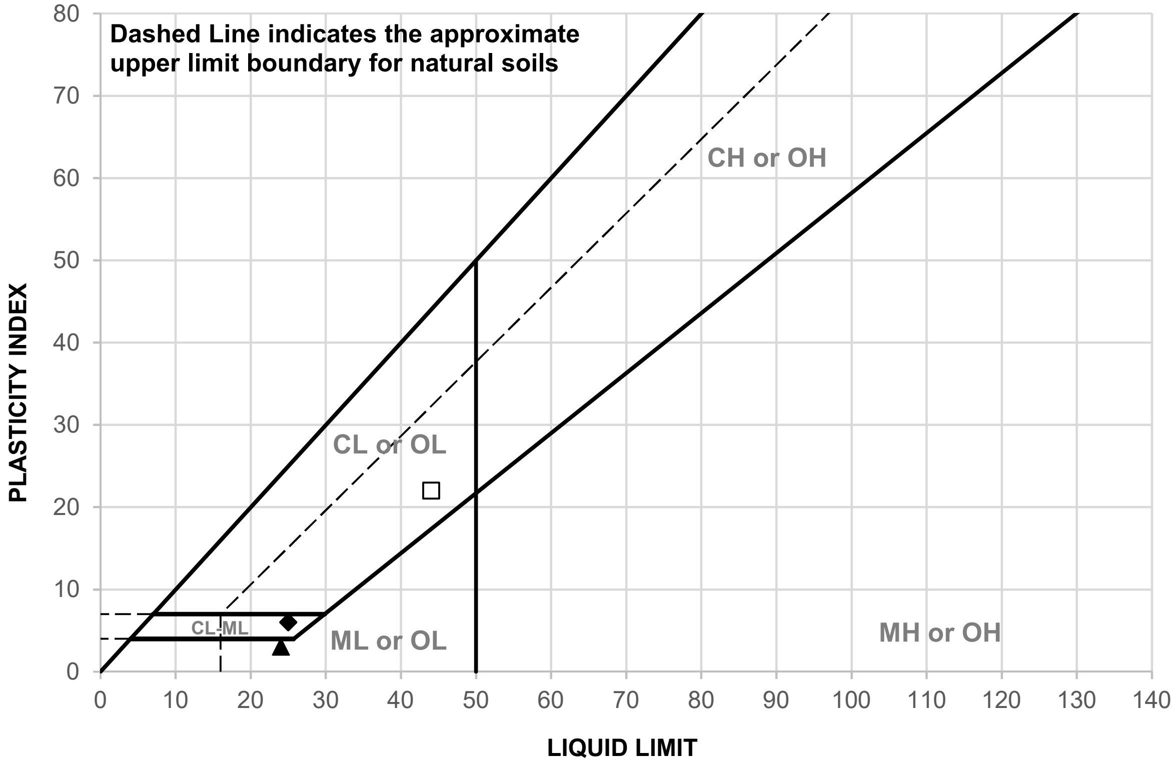
CLIENT: Hanover R.S. Limited Partnership
PROJECT NAME: Hanover North San Jose - Phase I and II
PROJECT NO: 18233.000.002 PH001
PROJECT LOCATION: San Jose, CA
REPORT DATE: 7/20/2021
TESTED BY: V.Navarro
REVIEWED BY: K. Lecce

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B3@3.5'	3.5 feet	See exploration logs	24	21	3
◆	1-B4@3.5'	3.5 feet	See exploration logs	25	19	6
□	1-B4@11'	11 feet	See exploration logs	44	22	22

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B3@3.5'	PI: ASTM D4318, Wet Method	
◆	1-B4@3.5'	PI: ASTM D4318, Wet Method	
□	1-B4@11'	PI: ASTM D4318, Wet Method	

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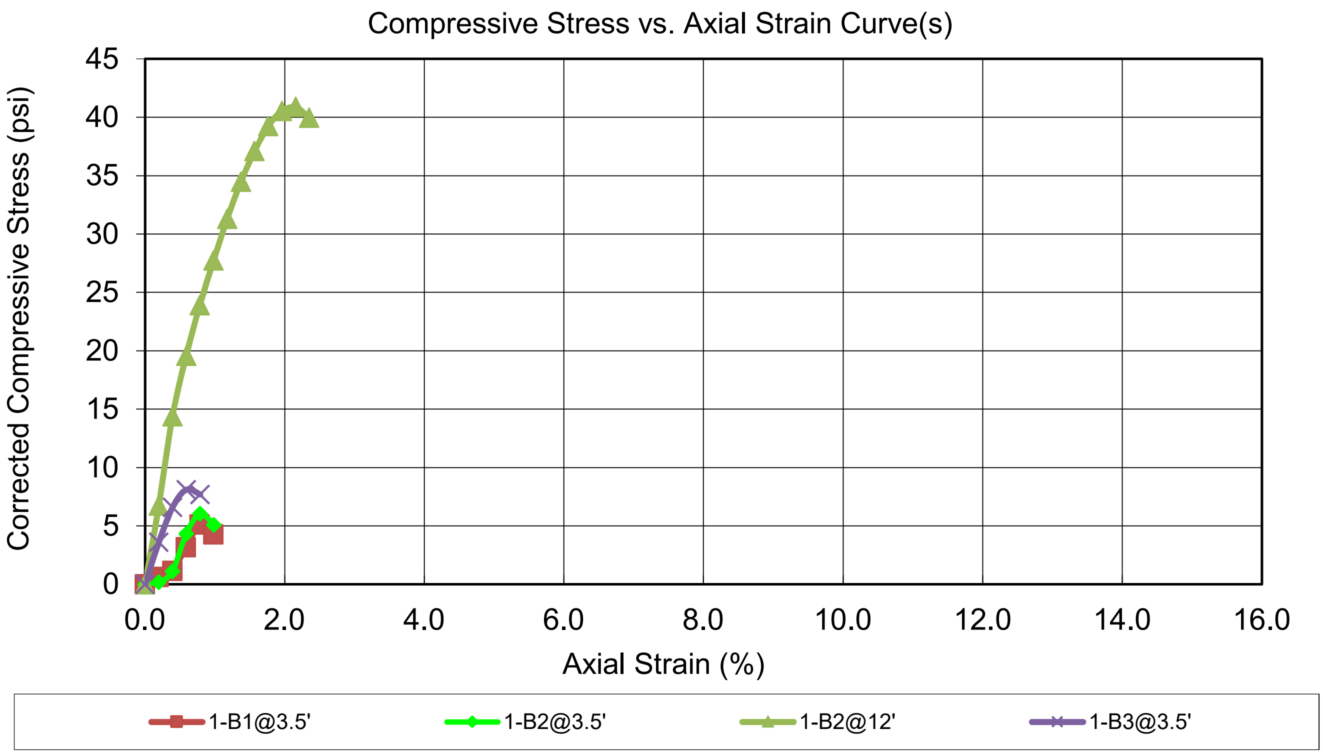
CLIENT: Hanover R.S. Limited Partnership
PROJECT NAME: Hanover North San Jose - Phase I and II
PROJECT NO: 18233.000.002 PH001
PROJECT LOCATION: San Jose, CA
REPORT DATE: 7/20/2021
TESTED BY: V.Navarro
REVIEWED BY: K. Lecce

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B1@3.5'	SPECIMEN 1-B2@3.5'	SPECIMEN 1-B2@12'	SPECIMEN 1-B3@3.5'
Test Moisture Content (%)	3.71	6.63	16.89	7.96
Dry Density (pcf)	96.3	88.0	100.1	88.5
Saturation (%)	13.2	19.4	66.0	23.6
Void Ratio	0.76	0.93	0.70	0.92
Diameter (in)	2.402	2.383	2.401	2.390
Height (in)	5.112	5.092	5.103	5.069
Height-To-Diameter Ratio	2.13	2.14	2.13	2.12
TEST DATA				
Unconfined Compressive Strength (psi)	5	6	41	8
Undrained Shear Strength (psi)	2.56	3.02	20.43	4.05
Strain Rate (in/min)	0.050	0.050	0.050	0.050
Specific Gravity (ASSUMED)	2.720	2.720	2.720	2.720
Strain at Failure(%)	0.78	0.79	2.16	0.59
Test Remarks				

SPECIMEN	DESCRIPTION
1-B1@3.5'	See exploration logs
1-B2@3.5'	See exploration logs
1-B2@12'	See exploration logs
1-B3@3.5'	See exploration logs

PROJECT NAME: Hanover North San Jose - Phase I and II
PROJECT NO: 18233.000.002 PH001 **Test Date:** 7/20/2021
CLIENT: Hanover R.S. Limited Partnership **Tested By:** V. Navarro
LOCATION: San Jose, CA **Reviewed By:** K. Lecce

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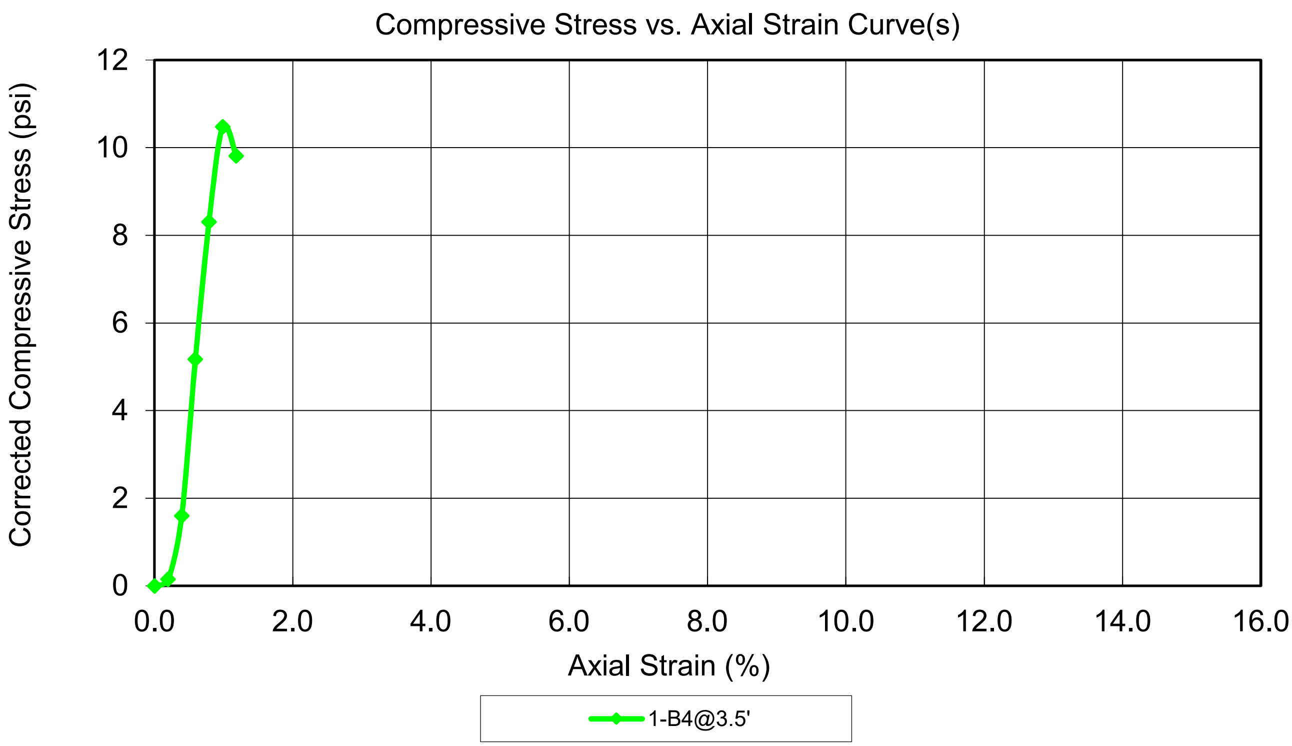
UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B4@3.5'
Test Moisture Content (%)	8.33
Dry Density (pcf)	89.4
Saturation (%)	25.4
Void Ratio	0.88
Diameter (in)	2.392
Height (in)	5.076
Height-To-Diameter Ratio	2.12
TEST DATA	
Unconfined Compressive Strength (psi)	10
Undrained Shear Strength (psi)	5.24
Strain Rate (in/min)	0.050
Specific Gravity (ASSUMED)	2.700
Strain at Failure(%)	0.99
Test Remarks	

SPECIMEN	DESCRIPTION
1-B4@3.5'	See exploration logs

PROJECT NAME: Hanover North San Jose - Phase I and II

PROJECT NO: 18233.000.002 PH001 **Report Date:** 7/20/2021

CLIENT: Hanover R.S. Limited Partnership **Tested By:** V. Navarro

LOCATION: San Jose, CA **Reviewed By:** K. Lecce

ENGEO
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Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

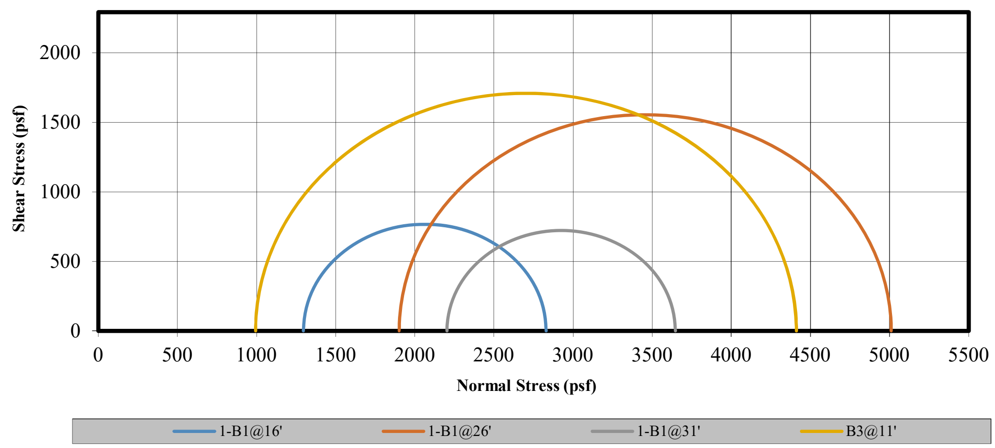
Date: 07/20/21

Checked By: K. Lecce

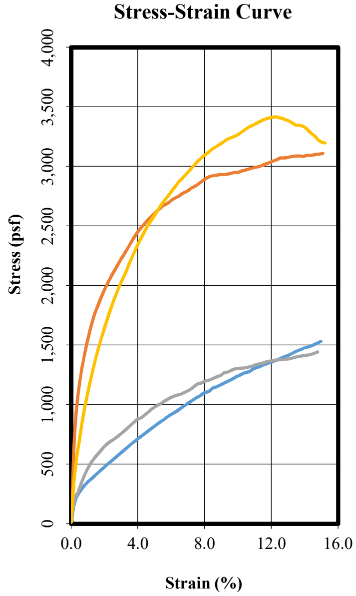
Date: 7/20/2021

Tested By: V. Navarro

Mohr Circles



Stress-Strain Curve



Before Test	Specimen 1-B1@16'	1-B1@26'	1-B1@31'	B3@11'
Water Content (%)	22.09	22.01	26.19	21.76
Dry Density (pcf)	108.00	108.10	100.80	102.60
Saturation (%)	99.99	99.89	99.91	90.42
Void Ratio	0.62	0.62	0.73	0.65
Diameter (in)	2.393	2.384	2.390	2.399
Height (in)	4.945	5.107	5.010	5.130
Height-to-Diameter Ratio	2.066	2.142	2.096	2.138
ASTM D4318 - Wet Method				
Liquid Limit	NV			
Plastic Limit	NP			
ASTM D854 - Measured				
Specific Gravity	2.800	2.800	2.800	2.720
After Test	**1-B1@16'**	**1-B1@26'**	**1-B1@31'**	**B3@11'**
Water Content (%)	22.09	22.01	26.19	21.76
Saturation (%)	99.99	99.89	99.91	90.42
Strain Rate (in/min)	0.05	0.05	0.05	0.05
Peak Deviator Stress (psf)	1533.2	3109.5	1442.9	3417.2
Axial Strain @ Failure (%)	14.965	15.078	14.771	12.281
	Cell Pressure			
Cell (psf)	1296.0	1900.8	2203.2	993.6
Back (psf)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psf)	2829.2	5010.3	3646.1	4410.8
σ3 (psf)	1296.0	1900.8	2203.2	993.6
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		766.6	1554.7	721.4	1708.6
Friction Angle Ø	n/a		n/a	n/a	n/a	n/a

Project Information	
Project Name:	Hanover North San Jose - Phase I and II
Project Number:	18233.000.002 PH001
Project Location:	San Jose, CA
Client:	Hanover R.S. Limited Partnership
Description:	See exploration logs
Test Remarks:	

ENGEO

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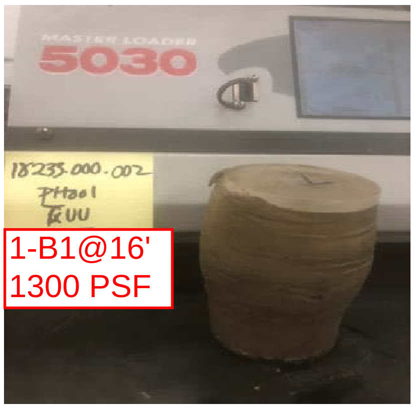
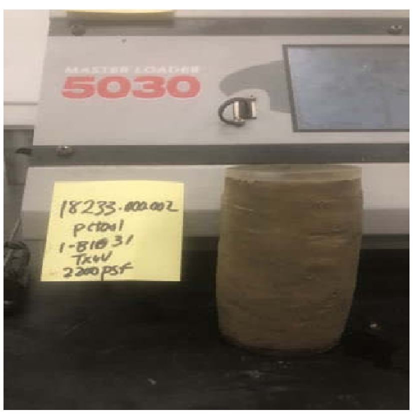
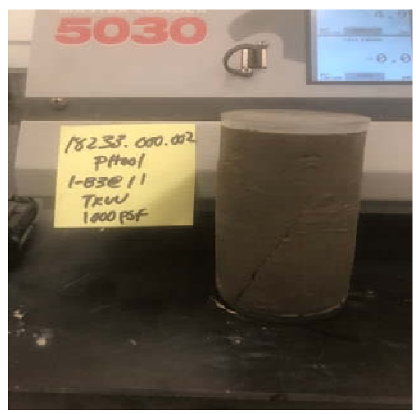
Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

Tested By: V. Navarro | Date: 7/20/2021 | Checked By: K. Lecce | Date: 07/20/21

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B1@16'	SAMPLE NUMBER: 1-B1@26'
	
SAMPLE NUMBER: 1-B1@31'	**SAMPLE NUMBER: B3@11'**
	

Project Information	
Project Name:	Hanover North San Jose - Phase I and II
Project Number:	18233.000.002 PH001
Project Location:	San Jose, CA
Client:	Hanover R.S. Limited Partnership
Description:	See exploration logs
Test Remarks:	

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