ALMADEN OFFICE COMPLEX
ALMADEN BOULEVARD / WOZ WAY
SAN JOSE, CALIFORNIA

GEOTECHNICAL EXPLORATION

PREPARED FOR
Boston Properties, Inc.
Four Embarcadero Center, Suite 2600
San Francisco, CA 95111

PREPARED BY
ENGEO Incorporated

January 31, 2019
Revised April 10, 2020

PROJECT NO.
15540.000.000

ENGEO
Expect Excellence

Project No.
15540.000.000

January 31, 2019
Revised April 10, 2020

Ms. Christina Bernardin
Project Manager
Boston Properties, Inc.
Four Embarcadero Center, Suite 2600
San Francisco, CA 94111

Subject: Almaden Office Complex
Almaden Boulevard / Woz Way
San Jose, California

GEOTECHNICAL EXPLORATION

Dear Ms. Bernardin:

As requested, we completed this geotechnical exploration for the proposed Almaden Office Complex Project in San Jose, California. The accompanying report presents our field exploration and laboratory testing with our conclusions and recommendations regarding the proposed project.

It is our opinion from a geotechnical standpoint that the site is suitable for the proposed development, provided the recommendations and guidelines in this report are implemented during project planning, design, and construction. The main geologic/geotechnical concerns at the site include settlement of moderately compressible layers due to building loads, strong ground motions, presence of groundwater and its effect on below-grade structures, necessity of shoring and dewatering systems during construction, flooding potential, and corrosive soils. Our recommendations to address these concerns are presented in the accompanying report.

We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical exploration report, as described in our revised proposal dated September 5, 2018, is to provide design-level geotechnical services for the proposed Almaden Office Complex project in San Jose, California.

Our scope was developed to include field exploration services, laboratory testing, analysis, and reporting to assist the design team. Each service is outlined in greater detail in the following sections.

1.1.1 Field Exploration and Lab Testing Program

Our field exploration included exploring the site through the following means:

- Four cone penetration tests (CPTs).
- One mud-rotary boring to collect subsurface soil samples.
- Geophysical testing, consisting of surface wave measurements.
- Installation of one vibrating-wire piezometer (VWP) to provide site-specific groundwater data.

Upon completion of field exploration, soil samples were routed to our in-house laboratory for various geotechnical tests to further characterize the site.

1.1.2 Data/Document Review, Engineering Analysis, and Reporting

Utilizing the site-specific data from this study in conjunction with exploration data previously obtained by others, we completed literature and document review/research and engineering analyses, as follows:

- Review of historic aerial photographs.
- Review of various geologic maps for the San Jose area, including assessment of nearby faults and potential earthquake ground motions.
- Groundwater evaluation based on our experience in the area, records of historic high groundwater levels, and site-specific VWP information.
- Analysis of seismic hazards, including liquefaction, cyclic softening, and site-specific seismic hazards.
- Compilation of current California Building Code (CBC) seismic design parameters.
- Three-dimensional analyses to determine the effect of site constraints, including adjacent existing developments and the Guadalupe River, on the proposed development.
- Analyses of settlement due to liquefaction, static loading, and cyclic loading.
- Development of design and construction recommendations based on findings and engineering analyses.

Additional scope items, including a soil-structure interaction (SSI) analysis, have not yet been completed and will be conducted as the project design continues.

Our findings and recommendations outlined in the aforementioned scope were compiled into this report. Our recommendations are based on the following plans and documents provided to us for the proposed Almaden Office Complex project:

- Topographic & Utility Survey of Almaden Boulevard and Woz Way, Kier & Wright, November 2018.
- Architectural Plans, South Almaden Offices, Kohn Pedersen Fox Associates PC, Sheets A-100.1 through A-118, and Scheme A Stacking Chart, January 8, 2019.
- Preliminary Foundation Loads, South Almaden Offices, Magnusson Klemencic Associates, Kohn Pedersen Fox Associates PC, January 16, 2019.

We prepared this report exclusively for Boston Properties, Inc. and its design team consultants. ENGEO should review any changes made in the character, design, or layout of the development to modify the conclusions and recommendations contained in this report, as necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

1.2 SITE LOCATION AND DESCRIPTION

The approximately 3.6-acre parcel is located at the northwest corner of the intersection of Woz Way and Almaden Boulevard in San Jose, California. Generally, the site is located within the downtown area of San Jose, near the Highway 87 and Interstate 280 interchange. The site is bordered by the Guadalupe River to the west, Woz Way and existing single-family homes to the south, existing office buildings and the San Jose Convention Center beyond Almaden Boulevard to the east, and existing office buildings to the north.

EXHIBIT 1.2-1: Site Location

The site is located within the Santa Clara Valley, located in the southern portion of the San Francisco Bay Area. The site is relatively level and existing site elevations (based on datum NAVD88) range from approximately 88½ feet on the northern side of the site to roughly 93 feet within the southern portion of the site.

Currently, the property is being used as general surface parking, which appears to be predominantly used for nearby downtown San Jose destinations. The site is currently paved and includes other appurtenant parking facilities, such as street lighting, pay station kiosks, and perimeter walls. A review of the survey performed by Kier & Wright (dated November 2018) indicates underground utilities are also located within site bounds, including storm drains, street lights, and telephone lines. A roughly 60-foot-wide public storm drain easement extends across the central portion of the site, leading from Almaden Boulevard to the Guadalupe River.

The Guadalupe River runs along the length of the western site boundary and is located at roughly 15 to 20 feet below the site ground-surface elevation. The slopes extending down to the river are range from ½:1 (horizontal:vertical) to 3:1 and include a paved pedestrian pathway at the crest of the slope. In addition, a pedestrian bridge crosses the river at the northwest corner of the site.

1.3 PROPOSED DEVELOPMENT

Based on our review of the provided documents, we understand the proposed project will consist of an office complex comprising one structure with two towers. Preliminary architectural exhibits show 3 below-grade levels and 15 above-grade levels, for a total height of approximately 280 feet above ground level. Current project designs indicate the complex will contain roughly 1.3 million square feet of office space, 70,000 square feet of outdoor terraces, 280,000 square feet of flex office space, and 555,000 square feet of parking. Other amenities include a coffee shop/brewery, restaurant, daycare, library, athletic club, and amphitheater spaces.

A review of the architectural exhibits provided to us indicates the structure height will be 263 to 293 feet, depending on which configuration is chosen. Basement finished floor elevation will be 32 feet below the ground floor level. Based on our experience, we anticipate basement excavations will extend at least 35 to 40 feet below the ground floor elevation. Exhibit 1.3-1 below shows current project renderings prepared by Kohn Pedersen Associates.

EXHIBIT 1.3-1: Proposed Project Rendering Looking Northeast

EXHIBIT 1.3-2: Proposed Project Rendering Looking Southeast

1.4 PREVIOUS GEOTECHNICAL STUDIES

The site was previously investigated by another consultant. Subsurface exploration locations available at the time of this report are shown on Figure 2A. The following discussion represents some of the available reports we reviewed. We incorporated select data from past investigations in our analyses for this study, as deemed appropriate. Fieldwork and lab testing conducted as part of the prior studies are provided as appendices to this report.

Treadwell & Rollo 2000 – Geotechnical Investigation

Treadwell & Roll (T&R) previously prepared a geotechnical report for the subject property. The scope of the study consisted of a 2-task approach: a geotechnical investigation and a seismic study. At the time of the report, the project consisted of a three-tower, 16- to 19-story office building, and a three-level basement extending over the entire building footprint.

The geotechnical investigation included exploring the site by means of eight soil borings and nine CPTs, extending to a maximum depth of approximately 101½ feet below the existing ground surface. In addition, two monitoring wells were installed at the site, at the locations of Borings B-2 and B-3; the wells were identified by T&R as MW-1 and MW-2, respectively. The report provides a geologic and geotechnical site characterization, T&R's findings with respect to geotechnical hazards, and geotechnical design and construction recommendations.

The seismic study task was included as a section within T&R's Geotechnical Investigation report. The scope was intended to provide site-specific recommendations for soil and foundation support elements. T&R performed a probabilistic seismic hazard analysis, design spectra with variable damping levels, and site parameters consistent with the 1997 Uniform Building Code.

Treadwell & Rollo 2005 – Response to Review Comments by City of San Jose

The City of San Jose provided comments to the 2000 geotechnical report on April 26, 2005. Review comments included the following requests, as summarized by T&R:

- Update the 2000 report to address changes in site conditions, project design and concept, standard of practice, or other changes that may affect the recommendations.
- Re-evaluate liquefaction potential at the site, using methods outlined in the California Division of Mines and Geology (now known as the California Geologic Survey) Special Publication 117, and presentation of the results in the geotechnical report.
- Evaluate the potential for lateral spreading along the Guadalupe River and provide mitigation measures.

T&R addressed the comments in its letter, providing additional analyses and recommendations, as necessary.

2.0 FINDINGS

2.1 SITE HISTORY

To characterize and understand site development history and geomorphology, we reviewed historic aerial photographs and topographic maps. We viewed numerous historic aerial photographs flown from 1948 through 2018, available on Google Earth and www.historicaerials.com. We also viewed historic topographic maps published back to 1897 to understand the site history before aerial photographic coverage was available.

Early topographic maps show that the site is located within the downtown portion of San Jose at an elevation of less than 100 feet above sea level. Minor development was evident at the time of map preparation with small buildings located within the bounds of the property. The alignment of nearby city streets resemble their current layout, including Auzerais Avenue extending across the Guadalupe River via a bridge. In the 1948 aerial photo, the site appears to be occupied by residences along the southern boundary and other miscellaneous structures within the central-northern portion of the site. Auzerais Avenue bisects the site into northern and southern halves. By 1987, structures located within the northern portion of the site appear to have been demolished and the area was paved for surface parking, while the southern portion still contains minor structures. By 1993, the entire site has been developed into surface parking. The Auzerais Avenue bridge appears to have been demolished by 1998 and a pedestrian bridge is visible, crossing the Guadalupe River at the northwestern corner of the site. Subsequent photos indicate the site has remained largely unchanged over the last 20 years.

2.2 GEOLOGIC SETTING

San Jose is located within the Coast Ranges geomorphic province of California. The Coast Ranges are characterized by a series of northwest-trending valleys and mountain ranges formed due to the interactions of the San Andreas Fault zone. The bedrock in this region has been folded and faulted in a tectonic setting that is experiencing translational and compressional deformations of the earth’s crust.

More specifically, San Jose is located within the Santa Clara Valley, an alluvium-filled basin that consists of gently sloping topography formed by coalescing alluvial fans. As depicted on Figure 3, regional mapping by Dibblee (2007) indicates the site is situated on younger alluvium (Qya). The alluvial deposits are estimated to be over 500 feet thick in this area of the Santa Clara Valley, and underlain by bedrock that outcrops around the Communications Hill area to the southeast. Based on geophysical testing conducted as part of this study, we estimate the depth to bedrock is roughly between 800 and 1,000 feet below the existing ground surface.

The upper soil profile within the project site consists predominately of alluvial fan deposits and alluvium of Holocene age. These Holocene deposits primarily consist of medium stiff to very stiff silty clays and clayey silts with varying amounts of sand. The Holocene deposits are generally underlain by late-Pleistocene alluvial fan deposits. The Pleistocene deposits are similar to Holocene soils, except that the soils are denser with variable amounts of gravels.

2.3 REGIONAL FAULTING

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone for active faults and no known faults cross the site. As such, fault rupture risk at the site is considered low.

Numerous small earthquakes occur every year in the San Francisco Bay Region and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded within the greater Bay Area Region. The most common nearby active faults within 25 miles of the site and their estimated maximum earthquake magnitudes are provided in the following table based on United States Geologic Survey (USGS) 2008 National Seismic Hazard Maps. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

TABLE 2.3-1: Approximate Fault Distances and Locations Relative to Project Site

FAULT	DISTANCE (Miles)	LOCATION RELATIVE TO SITE	ESTIMATED MAXIMUM MAGNITUDE, M_W
Monte Vista-Shannon	6.9	West	6.5
Calaveras	8.6	East	7.0
Hayward-Rodgers Creek	9.0	East	7.3
San Andreas	12.2	West	8.1
Zayante-Vergeles	17.1	Southwest	7.5
Greenville Connected	22.6	East	7.0

Latitude: 37.327463°N, Longitude: 121.890460°W

In addition, two concealed faults, the Silver Creek Fault and the San Jose Fault, are located in the vicinity of the project site (within 5 miles).

The United States Geologic Survey evaluated Bay Area seismicity through a study by the 2014 Working Group on California Earthquake Probabilities (USGS, 2016). The WGCEP estimated that the probability of a moment magnitude (M_w) 6.7 or greater earthquake occurring before 2043 is 22 percent on the San Andreas Fault, 33 percent on the Hayward Fault, and 26 percent on the Calaveras Fault. The aggregate probability of a similarly sized earthquake in the San Francisco Bay Area was estimated to be 72 percent in the study.

2.4 FIELD EXPLORATION

Our field exploration included advancing four CPTs (1-SCPT1 through 1-SCPT3, and 1-CPT4), drilling one boring (1-B1), installing and monitoring one vibrating-wire piezometer (VWP) (at 1-CPT4/1-B1), and performing geophysical testing. Our field exploration was intended to supplement and confirm the findings from T&R during its previous exploration of the site. Our field explorations were performed between October 22 and October 27, 2018. We continue to monitor the VWP.

The locations of the current explorations, in addition to past exploration locations, are shown on Figures 2A and 2B.

2.4.1 Rotary-Wash Boring

One soil boring was drilled on October 27, 2018, and extended to a maximum depth of approximately 121½ feet below the existing ground surface. Exploration locations were established by visual sighting from existing features. All current locations should be considered only as accurate as the methods used to determine them.

The boring was performed with a truck-mounted rig using 4-inch-diameter mud-rotary drilling methods. An ENGEO geotechnical engineer logged the borehole in the field and collected soil samples using a 2½-inch-inside-diameter Dames and Moore tube, 2½-inch-inside-diameter California-type split-spoon sampler fitted with 6-inch-long stainless steel liners, or a 2-inch-outside-diameter Standard Penetration Test (SPT) split-spoon sampler. The penetration of the samplers into underlying materials was recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments (SPT and California-type samplers), or as the pressure necessary to push the sampler 18 inches (Dames and Moore sampler). The boring logs present blow count results as the actual number of blows required for the last 1 foot of penetration; no conversion factors have been applied. The SPT and California-type samplers were driven with a 140-pound hammer falling a distance of 30 inches. The field logs were then used to develop the report logs, presented in Appendix A. The logs depict subsurface conditions within the boring at the time of drilling; however, subsurface conditions may vary with time.

2.4.2 Cone Penetration Tests

Cone Penetration Tests (CPTs) were conducted on October 22 and 23, 2018. The CPT scope included four test locations at the project site and extended to a maximum depth of approximately 95 feet below the existing ground surface. CPT locations were obtained by taping or pacing from existing features; as a result, the boring locations should be considered as accurate as the methods used to determine them. CPT logs are included in Appendix C.

The CPT equipment has a 30-ton compression-type cone with a 15-square-centimeter (cm^2) base area 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 of 2 cm per second. Cone readings are taken at approximately 2.5-cm intervals. Measurements include the tip resistance to penetration of the cone (Qc), the friction resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson, 2009). The CPT data were provided by California Push Technologies, Inc.

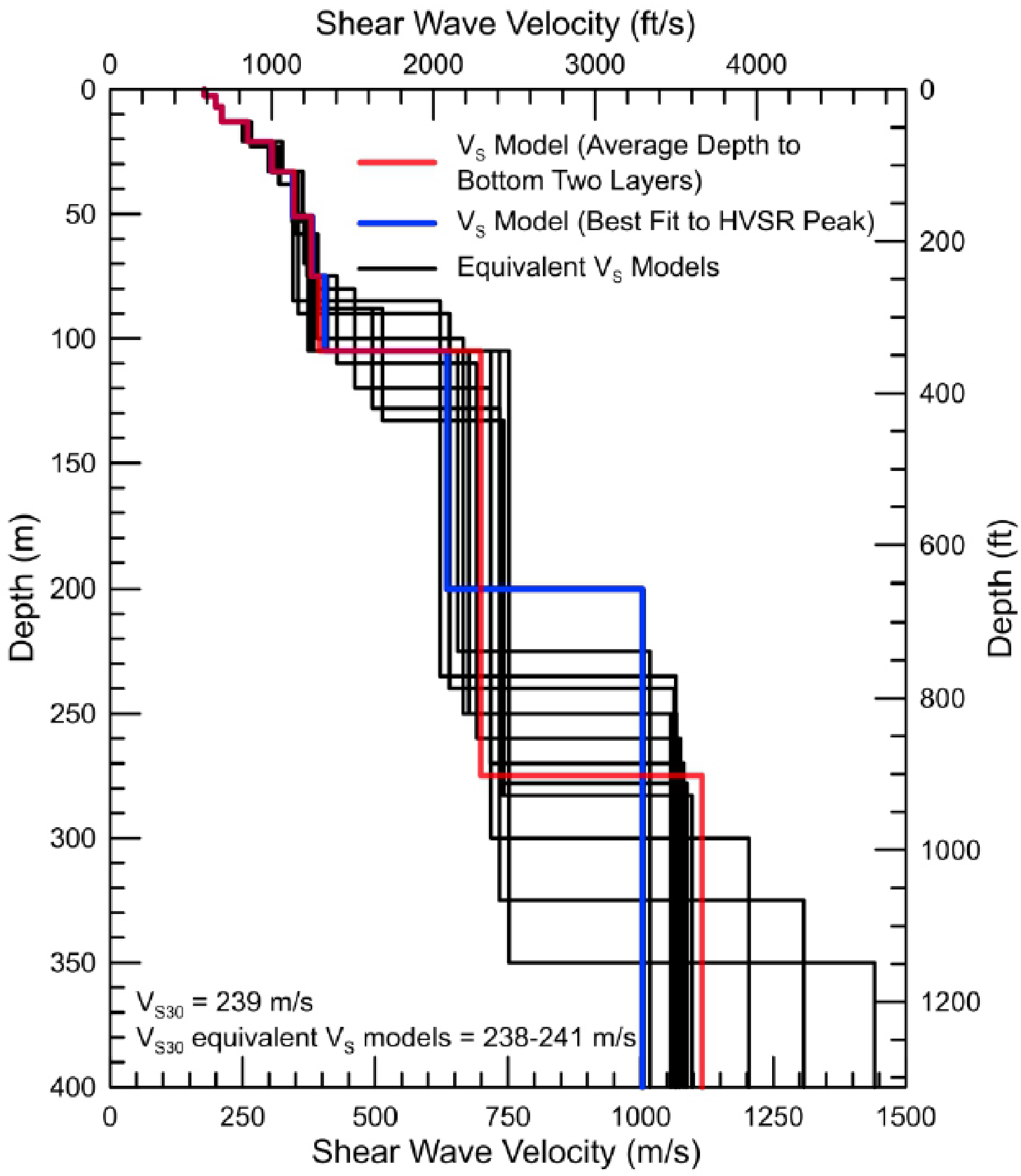
Shear wave velocity (V_S) measurements were performed by the CPT contractor in 1-SCPT01 through 1-SCPT03 using the downhole seismic method specified in ASTM D7400. We present the CPT logs in Appendix C.

2.4.3 Geophysical Survey

The geophysical exploration was performed by GEOVision and consisted of active-source Multi-Channel Analysis of Surface Waves (MASW) and Microtremor Array (MAM) surface wave methods. Additionally, they performed horizontal-to-vertical spectral ratio (HVSR) testing. The purpose of this portion of the field exploration was to obtain shear wave velocities at the site within the upper 300 meters, and estimate the average shear wave velocity in the upper 30 meters (V_{S30}). The geophysical seismic survey was performed at the locations shown on Figure 2B. Details of the GEOVision testing are contained in its report, presented in Appendix D.

The V_S profiles obtained from the geophysical testing are presented in Exhibit 2.4.3-1 for comparison. The time-averaged shear wave velocity over the top 100 feet or 30 meters (V_{S30}) for these V_S profiles ranges from 775 to 780 feet/sec.

EXHIBIT 2.4.3-1: Vs profiles obtained from surface wave testing shown to a depth of approximately 1,300 feet.



2.5 LABORATORY TESTING

We performed the following laboratory tests on select samples recovered during boring operations.

TABLE 2.5-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD
Natural Unit Weight and Moisture Content	ASTM D7263, D2216
Atterberg Limits	ASTM D4318
Particle Size Distribution	ASTM D1140, D6913
Unconsolidated Undrained Triaxial Compression	ASTM D2850
Unconfined Compressive Strength of Soils	ASTM D2166
Incremental Consolidation	ASTM D2435
Cyclic Simple Shear	ASTM D6528 - Modified
Corrosivity Testing (Redox, pH, Resistivity, Chloride, Sulfide, Chloride, Sulfate)	ASTM D1498, D4972, G57, D4658M, D4327

Many of the laboratory test results are shown on the bore logs (Appendix A), with individual test results presented in Appendix B.

2.6 SURFACE AND SUBSURFACE CONDITIONS

As previously mentioned, the surface elevation of the site ranges from roughly 88½ to 93 feet (NAVD88) from north to south. The site is currently paved, with a section of 3 to 6 inches of asphaltic concrete over 6 inches of base rock.

Subsurface conditions at the site include alluvial deposits, consisting of silty and clayey material with variable amounts of sand, extending to the full depth of exploration, to roughly 120½ feet below the existing ground surface. T&R identified artificial fill in Borings B-2, B-3, and B-6. Fill was encountered to a maximum depth of approximately 25 feet in Boring B-2.

In the upper 40 feet, olive brown to gray clayey and silty layers, interbedded with sand layers were encountered. Consistency of the clayey and silty layers range from soft to very stiff and were generally of low plasticity. Sand layers encountered were found to have variable amounts of silt and medium dense to dense. A sand layer, roughly 10 feet thick, was encountered in numerous borings and CPT logs across the site beginning at approximately 15 to 20 feet below the existing ground surface. Beginning between 30 and 40 feet below the existing ground surface, a silty sand layer was found, roughly 2 to 5 feet thick.

Below 40 feet, subsurface material consisted of silt and clay, with increasing sand and gravel content with depth. Fine-grained material was found to be stiff to very stiff, with a few zones of softer material. Below 80 feet, material was predominantly sandy and gravelly, with blow counts generally exceeding 45 blows per foot (dense to very dense consistencies).

We developed two generalized subsurface cross sections, A-A' and B-B' which depict our interpretation of the soil conditions based on past and present field explorations, presented in Figure 8. These interpreted cross sections may assist in the visualization of layering and general subsurface trends in two dimensions across the site.

2.7 GUADALUPE RIVER

The Guadalupe River bounds the western edge of the project site. The natural creek begins in the Santa Cruz mountains and flows north through the Santa Clara Valley, ultimately discharging into the San Francisco Bay. Tributaries include the Los Gatos Creek, Canoas Creek, and Ross Creek.

The creek extends through urban portions of San Jose; numerous crossings and improvements have been constructed near the creek in downtown San Jose. Within the vicinity of the project site, the creek ranges from roughly 30 to 60 feet wide, and varies in depth depending on the season. The creek is measured to be several feet deep, with the bottom of the creek bed ranging in approximate elevations from 74 to 79 feet (NAVD88) along the length of the project site.

The river banks are subject to flooding, especially within the downtown San Jose area. Based on a review of the FEMA flood insurance study, the one-percent annual chance of flood elevations of the Guadalupe River between the northern and southern bounds of the site show maximum flood elevations of 92 and 94 feet (NAVD88), respectively.

2.8 GROUNDWATER

During the current field exploration, we measured the approximate depth to groundwater with pore-pressure dissipation tests at all CPT locations. In addition, a VWP was installed at CPT Location 1-CPT4 to provide continuous depth-to-groundwater measurements. Pore pressure dissipation tests indicated the groundwater table ranges from roughly 17 to 19 feet below the ground surface across the site. This groundwater depth coincides with the approximate elevation of the adjacent Guadalupe River.

In the 13 months following VWP installation, groundwater at the site was observed to fluctuate between depths ranging from approximately 14 feet to 17 feet, generally following seasonal wet-weather trends.

We also reviewed groundwater data provided by T&R during its previous geotechnical investigation. T&R installed two monitoring wells at the site during its exploration activities in 2000. Well MW-1 included a screened casing from 20 to 30 feet and Well MW-2 has a screened casing between depths of 50 and 80 feet below the ground surface. At the time of publication of its report, T&R found that groundwater levels in both wells ranged from 15½ to 17 feet below the ground surface.

Plate 1.2 of the Seismic Hazard Zone Report for the San Jose West Quadrangle (2002) maps the highest historical groundwater within the site vicinity to be less than approximately 20 feet below the ground surface. Plate 3 of Special Report 107 (1974) provides the approximate first depth to groundwater in Santa Clara County; this map shows groundwater in the vicinity of the project site to be approximately 15 feet below the ground surface. For purposes of our analyses and recommendations, we considered an appropriate design groundwater depth of 14 feet below the ground surface, which corresponds to an elevation range of 73½ to 78 feet (NAVD88).

3.0 DISCUSSION AND CONCLUSIONS

Based on the exploration and laboratory test results, the project site is feasible for the proposed development provided the recommendations contained in this report are properly incorporated into the design plans and specifications.

The primary geotechnical concerns for the proposed site redevelopment include:

- The settlement of moderately compressible layers due to building loads.
- Strong ground motions.
- The presence of groundwater and its influence on below-grade construction.
- The need for shoring systems to protect the excavation walls, adjacent streets and improvements, and the potential need for dewatering of excavations extending below the groundwater surface.
- The potential for flooding due to the adjacent Guadalupe River.
- Corrosive soils and their effect on buried utilities.

These and other issues are discussed in the following sections.

3.1 STATIC CONSOLIDATION SETTLEMENT

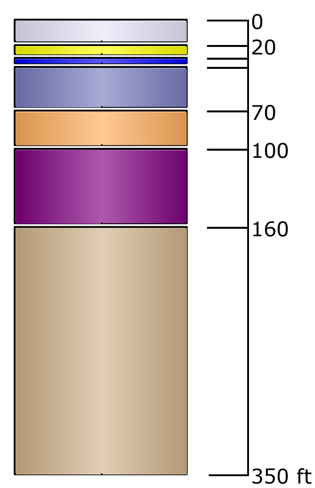
We understand building loads and bearing pressures are still being determined by the structural designer. For our use in preparation of this report, preliminary building loads were provided to us. Based on our exploration and the preliminary building loads, immediate and recompression settlements are anticipated below the base of the foundation. We anticipate that the majority of these settlements will take place during construction as the subgrade material is reloaded. Provided the recommendations in this report are followed during design and construction, post-construction settlement can be appropriately managed.

We evaluated settlement potential at the site with the software program Settle 3D (Version 4). To develop our model, we reviewed available laboratory testing from our current exploration, as well as information from the previous exploration to determine representative, site-specific design parameters. The exhibit below shows the design parameters and soil profile used in our Settle 3D analysis; output is provided in Appendix E.

EXHIBIT 3.1-1: Settle 3D Parameters

Material Name	Color	Sat. Unit Weight (kips/ft3)	Es (ksf)	Cc	Cr	OCR	e0	Cv (ft2/y)	Cvr (ft2/y)
CLAY1_ABOVE		0.125	-	-	-	-	-	-	-
SAND2_ABOVE		0.125	-	-	-	-	-	-	-
CLAY3_ABOVE		0.125	-	-	-	-	-	-	-
CLAY4		0.125	-	0.22	0.026	2	0.6	10	60
SAND/GRAVEL5		0.125	3000	-	-	-	-	-	-
DEEPCLAY6		0.125	-	0.25	0.024	1.5	0.6	10	60
DEEPSAND/GRAVEL7		0.13	3700	-	-	-	-	-	-

Soil Layer Column



Our Settle 3D model includes soil layers identified in current and previous subsurface explorations to the maximum depth explored by means of drilled borings or CPTs. Due to the nature of the proposed basement (i.e. the basement footprint coincides with the approximate site footprint), soils above the bottom of the basement excavation were not assigned settlement parameters. Soil strata encountered at depth in drilled borings and CPTs were found to consist of interbedded layers of gravelly sand and clay, with varying amounts of silt. Although borings and CPTs do not extend deeper than approximately 120 feet below existing ground surface, a review of geologic maps indicates that the alluvium extends below this depth and likely consists of very dense and stiff interbedded sandy and clayey layers. Since precise depths and layer thicknesses are unknown, the anticipated sandy and clayey layers were grouped together. The collective clay layer was placed above the combined sandy gravel layer to model conservatively the building load distribution and its effect on the clay layers.

Shear-wave velocities at the site generally indicate that the interface between rock (site class B) and very dense soils (site class C) is located at approximately 350 feet below the ground surface. Therefore, we set the limit of our Settle 3D soil profile at a depth of 350 feet.

3.1.1 Over-Consolidation Ratio Parameters

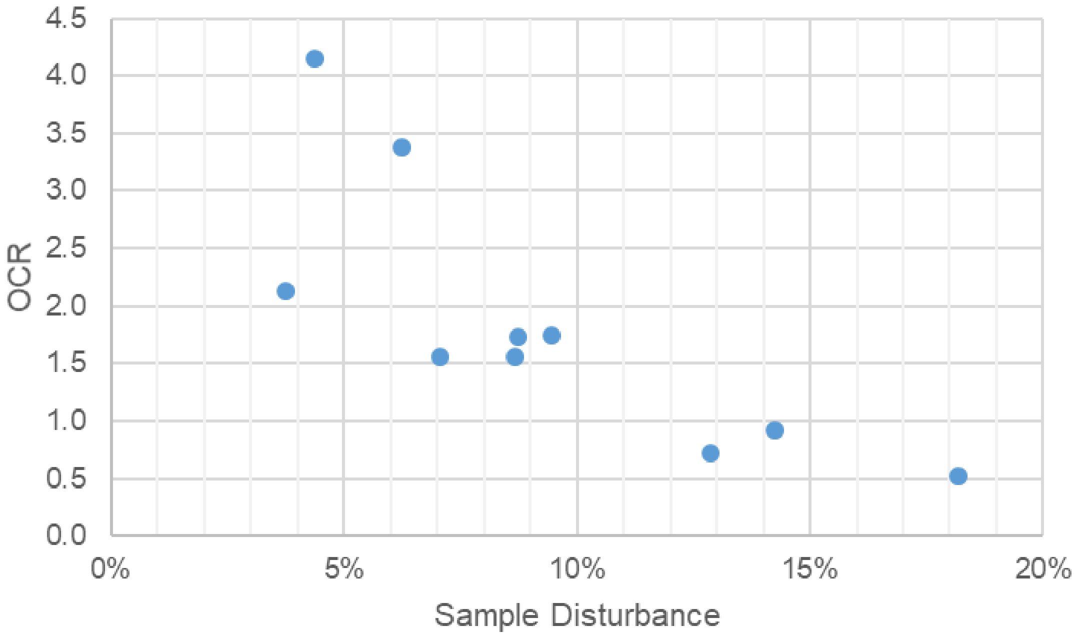
Over-consolidation ratios were determined from consolidation lab testing using methods developed by Casagrande and Pacheco. We also determined specimen quality with methods developed from Lunne et al. (1997) to establish the reliability of the OCR results. Table 3.1.1-1 provides a summary of project over-consolidation ratios.

TABLE 3.1.1-1: OCR Results

SAMPLE LOCATION	DEPTH (feet)	ELEVATION (NAVD88)	$\Delta e/e_0$	OCR (Casagrande)	OCR (Pacheco)	AVERAGE OCR	SAMPLE QUALITY RATING (Lunne et al.)
B-1	55	37½	0.087	1.4	1.7	1.6	Poor
B-3	50	37½	0.063	3.1	3.6	3.4	Poor
B-4	45	42½	0.087	1.7	1.8	1.7	Poor
B-5	30	58	0.037	2.0	2.3	2.1	Good to Fair
B-6	50	42	0.044	4.2	4.2	4.2	Good to Fair
B-7	70	20½	0.095	1.7	1.7	1.7	Poor
B-8	50	40	0.071	1.6	1.6	1.6	Poor
1-B1	48	41.4	0.143	--	--	--	Very Poor
1-B1	51	38.4	0.129	--	--	--	Very Poor
1-B1	121	-31.6	0.182	--	--	--	Very Poor

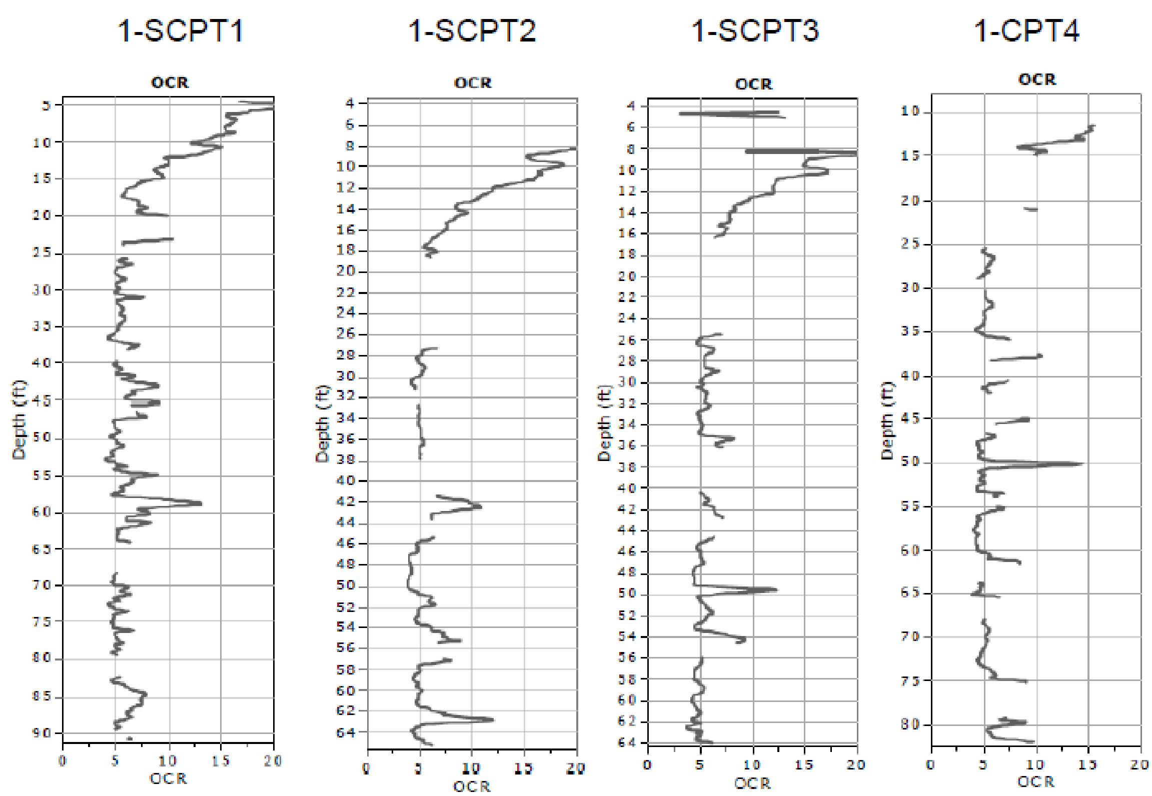
Based on the OCRs and the corresponding sample disturbance, we can conclude that site soils have OCRs higher than what was determined from poorer quality samples. Exhibit 3.1.1-1 shows OCR versus sample disturbance; as sample disturbance (calculated as percent strain) increases, the OCR of the sample decreases.

EXHIBIT 3.1.1-1: Sample Disturbance Effects



Furthermore, we reviewed our CPT results and utilized the program CPeT-IT (Version 2.3.1.6) to generate an additional rough approximation for OCRs at the site. While OCR estimates generated from CPT results are based on empirical correlations, we chose to utilize this information as an upper bound for the site. As shown in Exhibit 3.1.1-2, CPTs indicated OCRs are generally greater than 2 below a depth of 40 feet (assumed bottom of foundation).

EXHIBIT 3.1.1-2: CPeT-IT Output for OCR Estimates



Based on the sample quality and relevant depth, we selected a design OCR value of 2 for the clay layer located directly below the foundation, corresponding to depths of 40 to 70 feet below the existing ground surface. We conservatively selected an OCR value of 1.5 for the deeper clay layer, which extends between depths of 100 and 160 feet below ground surface in our model.

3.1.2 Settle 3D Results

Our model examined long-term settlement conditions, and the following settlement values represent the total amount between the end of construction and 30 years after the end of construction. Our Settle 3D output is included in Appendix E.

TABLE 3.1.2-1: Settle 3D Results Summary

AVERAGE BEARING PRESSURE (psf)	ESTIMATED LONG-TERM STATIC SETTLEMENT (in)
5,000	Less than 1
6,000	2½
6,500	3

Additional foundation recommendations are provided in Section 5.1.

3.2 EXISTING ARTIFICIAL FILL

Artificial fill was identified in Borings B-2, B-3, and B-6 by T&R during its initial study. Based on the site history and location, the fill is likely related to past improvements (both above ground and below ground), which may no longer exist at the site. No documentation of fill placement was provided or discovered during the preparation of this report. Without documentation regarding the manner of placement, type of material used, and degree of compaction, the existing fill should be considered non-engineered.

Based on the proposed design of the development, non-engineered fills will be removed as part of the basement excavation and do not pose a concern to the proposed development.

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 and liquefaction. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lurching, landslides, tsunamis, or seiches is low to negligible at the site.

3.3.1 Ground Rupture

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

3.3.2 Ground Shaking

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the actual 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 site is located within a State of California Seismic Hazard Zone (CDMG, 2002) for areas that may be susceptible to liquefaction (Figure 5).

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, the sand may undergo deformation. If the sand undergoes virtually unlimited deformation without developing significant resistance, it is said to have liquefied, and if the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. In addition to liquefaction of sandy materials, clayey soil can also undergo “cyclic-softening” or strength loss as a result of cyclic loading. Since the site is composed of many thick clay layers, we considered this effect in our analyses.

3.3.3.1 Liquefaction Analysis Overview

We performed a liquefaction assessment based on guidelines provided in Special Publication 117A (2008), as well as methods described herein.

We used the in-situ data (blow counts and soil descriptions), laboratory data (PI, moisture content, fines content, and CSS test), and Bray and Sancio (2006) methodologies to establish a relationship between soil that is potentially liquefiable in the CPTs by comparing them to an adjacent “matched-pair” boring.

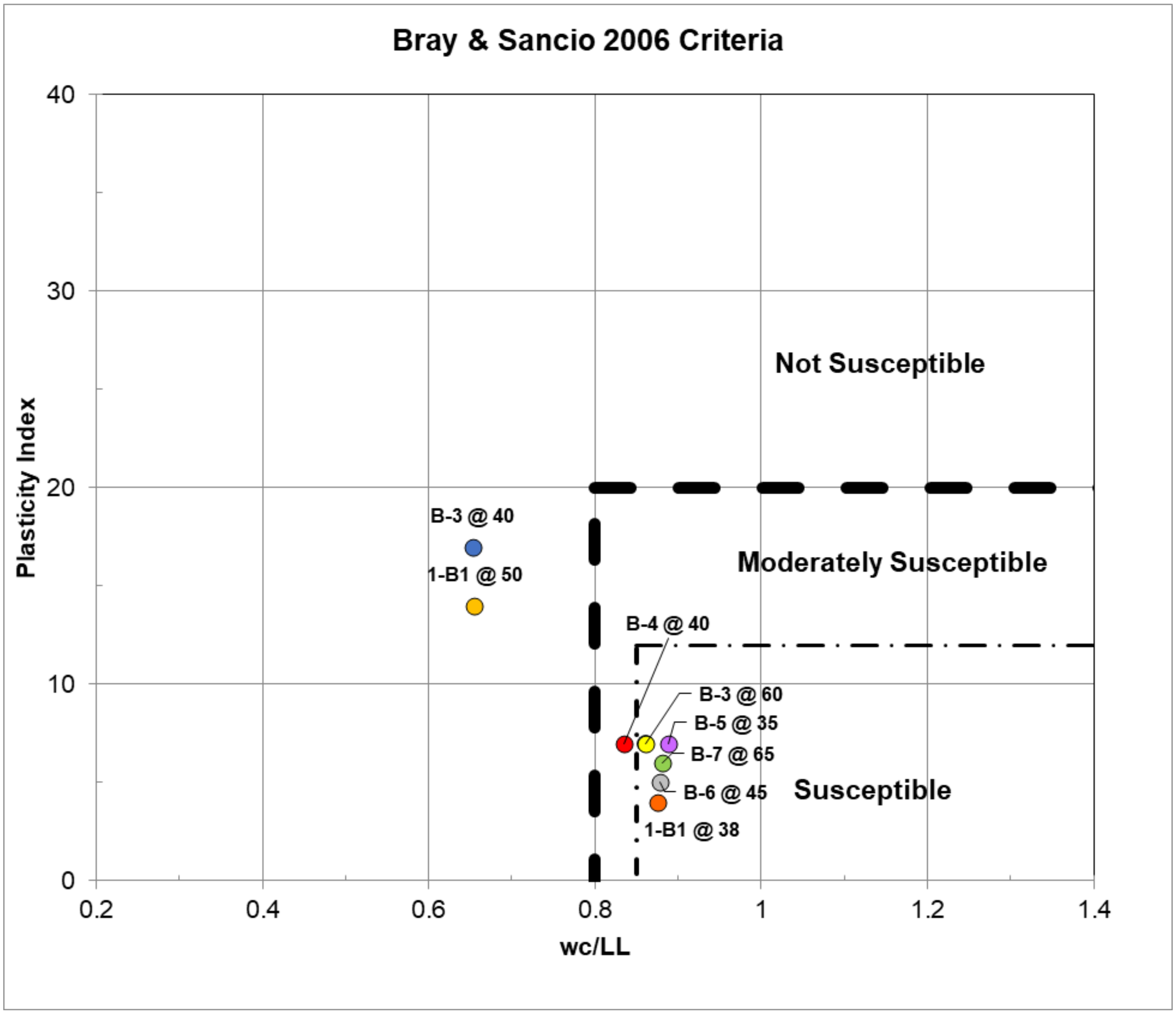
Our assessment began with using the methodologies presented by Bray and Sancio (2006). Section 3.3.3.2 presents the details of screening of soil samples for liquefaction susceptibility. We then performed an analysis of liquefaction potential based on the CPT data using the computer software CLiq, as described in Section 0. Finally, we performed cyclic simple shear (CSS) testing on a select representative sample of the fine-grained deposits to more accurately assess and confirm the cyclic response of the fine-grained soil at the base of the foundation.

3.3.3.2 Liquefaction Susceptibility Screening of Soil Samples

Fine-grained soil samples collected at the assumed depth of the bottom of foundation appeared to be potentially liquefiable. As such, we considered the criteria presented by Bray and Sancio to assess the potential for liquefaction triggering on these soils. Bray and Sancio observed that soils with a plasticity index (PI) less than 12 and a water content (w_c) to liquid limit (LL) ratio of more than 0.85 are susceptible to liquefaction/cyclic-softening. Soils with PI greater than 18 and/or w_c/LL less than 0.8 were deemed to be not susceptible to liquefaction because they are too plastic and/or their water contents are too low.

We considered the Bray and Sancio criteria at this site and plotted w_c/LL versus PI for our available laboratory data. As shown in Exhibit 3.3.3.2-1, some soils appear to be susceptible to liquefaction based on these criteria.

EXHIBIT 3.3.3.2-1: Assessment of the Liquefaction/Cyclic-Softening Potential of Fine-Grained based on the Bray and Sancio (2006) Criteria.



3.3.3.3 Liquefaction Analysis of CPT Data

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. 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 Robertson (2009). We estimated the Cyclic Stress Ratio (CSR) for a Maximum Considered Earthquake (MCE) Peak Ground Acceleration (PGA_M) value of 0.5g as outlined in the current California building code with an earthquake magnitude of 7.8. We used a groundwater depth of 15 feet for this analysis. We also considered the depth of excavation in the CLiq analysis.

Upon conducting the CLiq analysis, the layers in question were found to have a soil behavior Type Index (I_c) greater than 2.6 and yielded a low susceptibility to liquefaction. Appendix F presents the results of the CLiq analyses.

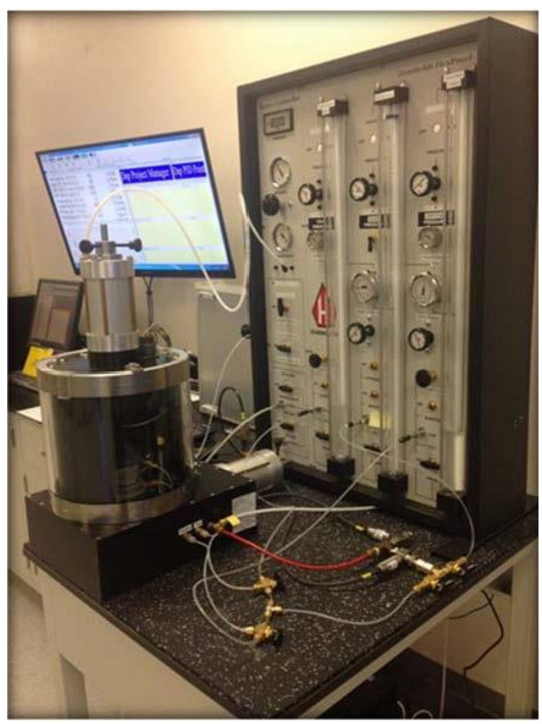
Based on the results of the CLiq analysis, liquefaction-induced settlement for the proposed building is estimated to be less than 1 inch.

3.3.3.4 Cyclic Simple Shear Tests

Since the Bray and Sancio method is considered a screening test of potential for liquefaction susceptibility, we performed cyclic simple shear (CSS) testing on a select sample of fine-grained deposits recovered from our Dames and Moore samplers to more accurately assess the cyclic response of the fine-grained site soil, and confirm our findings from the CLiq analysis.

The CSS undrained loading test consists of a number of cycles of stress-controlled loading at a given load amplitude. All tests are performed in a "constant height" mode, wherein the vertical position of the top cap is rigidly locked immediately prior to the shearing portion of the test, such that specimen cannot change height during shearing. In this situation, materials that are prone to contracting or developing positive pore water pressure are observed to have the vertical deviatoric stress drop during shearing (which can be measured, since the load cell is beneath the clamping point on the load system). Such a decline in vertical stress in essentially a loss of confining stress, which combines with any positive pore pressures generated to reduce the effective stress during a test. It should be noted that the system is designed to allow a sample to be consolidated and then sheared under constant height conditions (simulating undrained shear of a saturated specimen). Exhibit 3.3.3.4-1 shows the ENGEO CSS device.

EXHIBIT 3.3.3.4-1: ENGEO CSS Apparatus



We performed CSS testing on a fine-grained sample recovered at a depth of approximately 41 feet below the existing ground surface. Based on review of the CSS test results (Appendix B), the sample showed cyclic mobility when subjected to a cyclic stress ratio (CSR) of 0.38. As such, laboratory testing confirmed this material is susceptible to cyclic mobility as predicted with CLiq. Settlement due to liquefaction below the proposed structure is estimated to be less than 1 inch.

3.3.4 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. The Guadalupe River is located approximately 30 to 45 feet west of the project site. The eastern river bank slopes are up to approximately 15 to 20 feet high and as steep as ½:1 (horizontal:vertical) in some areas. As shown on Cross Sections A-A' and B-B', a sandy layer is located at approximately 15 to 25 feet below the ground surface (Figure 8) and daylights at the face of the river bank (shown on Cross Section B-B'). Based on our liquefaction analysis, this layer is potentially liquefiable and the eastern Guadalupe River bank in this area is subject to failure during a seismic event.

We evaluated the potential for lateral movement of the slope at the proposed building limit using slope stability methods recommended by the California Geological Survey's Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California." The subject sand layer varies in CPT tip resistance from 100 to 350 tons per square foot (tsf) across the site. 1-SCPT2 was conservatively selected to further assess potential lateral spreading at the site due to the layer's relative thickness at this location and lower tip resistances encountered, thereby producing a higher potential for liquefaction and corresponding lateral movement.

For conservative analysis, we evaluated the stability of the potentially liquefiable soil between the basement and slope face. Undrained shear strengths of fine-grained clayey soils were estimated from laboratory and field testing information. To evaluate the residual shear strength of the potentially liquefiable sand, we used the methods presented in "Engineering Evaluation of Post-Liquefaction Strength" by Weber (2015). The estimates of residual strength are based on calculations of vertical effective stress and normalized blow counts.

TABLE 3.3.4-1: Soil Properties Used in Slope Stability Analysis

LAYER NO.	MATERIAL	UNIT WEIGHT γ_{sat} (pcf)	FRICTION ANGLE (ϕ)	UNDRAINED SHEAR STRENGTH (psf)
1	CL	120	-	1000
2	SP-SM (liquefied)	70	30°	-
3	CL	120	-	1250
4	SP-SM	120	34°	-

Based on the above strengths, we estimated a yield coefficient of 0.29g (pseudo-static coefficient to achieve a FS of at least 1.0). Comparing this yield coefficient with the Bray and Travasarou methodology (2014), which considers the period of the sliding mass to calculate displacements, we estimate seismic slope displacements to be less than 6 inches during the MCE event. Based on SP117A, these displacements are unlikely to correspond to serious movement or damage.

3.3.5 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area region, but based on the site location, it is our opinion that the offset is expected to be minor. We provide recommendations for foundation and pavement design in this report that are intended to reduce the potential for adverse impacts from lurch cracking.

3.3.6 Flooding

Federal Emergency Management Agency (FEMA) Flood Insurance Maps (Figure 7) indicate that the site is within a special flood hazard area subject to inundation by 1- and 0.2-percent annual chances of flood. This area of San Jose has been subject to flooding in the past due to heavy rainfall. The Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project.

3.4 SHALLOW GROUNDWATER AND EXCAVATION CONSIDERATIONS

Based on our findings described in Section 2.8 of this report and the proposed development, groundwater may impact basement design and construction at the site. Shallow groundwater conditions may result in the following impacts:

1. Require construction dewatering.
2. Result in unstable conditions at the base of excavation requiring stabilization prior to foundation construction.
3. Cause moisture damage to sensitive floor coverings.
4. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.
5. Require waterproofing for the proposed basement structures.

As discussed previously, an excavation up to approximately 35 to 40 feet deep will be necessary for the construction of the proposed basement. During excavation of the basements, the sides of the excavation will need to be shored. The primary considerations related to the selection of the shoring systems are:

1. Distance of the excavation from improvements sensitive to movement that will remain after building construction.
2. Potential presence of groundwater during construction, and the need to keep the dewatering to a minimum due to environmental concerns.

3.5 CBC SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions encountered, CPT shear wave velocity testing, and geophysical testing, we classified the site as Class D in accordance with the 2016 CBC. We provide the 2016 CBC seismic design parameters for a Site Class D in Table 3.5-1 below, which includes design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters. We will provide site-specific MCE_R spectrum under a separate report. We also provide values utilizing ASCE 7-16 in Table 3.5-1.

TABLE 3.5-1: 2016 CBC Seismic Design Parameters, Latitude: 37.327463° **Longitude:** -121.890460°

PARAMETER	VALUE (ASCE 7-10)	VALUE (ASCE 7-16)
Site Class	D	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.50	1.50
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.600	0.600
Site Coefficient, F_A	1.00	1.00
Site Coefficient, F_V	1.50	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.50	1.50
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.900	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.00	1.00
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.600	Null*
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.500	0.538
Site Coefficient, F_{PGA}	1.00	1.1
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.500	0.592
Long-period transition-period, T_L	12 sec	12 sec

*These values require a site-specific seismic hazard analysis, currently in progress

3.6 SOIL CORROSION POTENTIAL

As part of this study, we collected three soil samples and submitted them to a California State certified analytical lab for determination of redox potential, pH, resistivity, sulfide, sulfate, and chloride. In addition, we reviewed the corrosivity test results, from samples previously tested by T&R. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes. The results from both explorations are included in Appendix G and Appendix I, and are summarized in the table below.

TABLE 3.6-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (feet)	REDOX (mV)	pH	RESISTIVITY (OHMS-CM)	SULFIDE (mg/kg)	CHLORIDE* (mg/kg)	SULFATE* (mg/kg)
1-B1	26-26.5	23	7.65	1,400	N.D.	16	27
1-B1	44.5-45	250	8.00	2,100	N.D.	N.D.	20
B-3	5	370	6.9	950	-	57	130
B-4	20.5	350	7.6	4,000	-	25	41

*ASTM D4327

The 2016 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Chapter 19, Sections 19.3.1.1 for structural concrete requirements. Based on the test results and ACI criteria, the tested soil would classify as 'Not Applicable' for sulfate exposure; there is no requirement for cement type or water-cement ratio for this category; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

Soil with a pH less than 6.0 is considered to be corrosive to buried metal piping and reinforced concrete structures. The samples had a pH of above 6.9, which does not present corrosion concerns for buried iron, steel, mortar-coated steel, and reinforced concrete structures.

Based on resistivity measurements, the samples from 1-B1 at the depth of 26 to 26.5 feet and from B-3 at the depth of 5 feet are classified as “corrosive” to buried metal piping. The samples from 1-B1 at the depth of 44.5 to 45 feet and B-4 are classified as “moderately corrosive” to buried metal piping.

If it is desired to investigate this further, we recommend a corrosion consultant be retained to evaluate whether specific corrosion recommendations are advised for the project.

4.0 EARTHWORK RECOMMENDATIONS

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

As used in this report, the term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define “structural areas” as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

4.1 DEMOLITION AND STRIPPING

Grading operations should be observed and tested by our qualified field representative. We should be notified a minimum of three days prior to grading and excavation operations in order to coordinate our schedule with the contractor.

Site development should commence with the removal of existing pavement and minor parking-related structures as well as buried structures such as utilities (unless they are to remain). All excavations from demolition should be cleaned to a firm undisturbed native soil surface determined by our representative in the field. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. All backfill materials should be placed and compacted as engineered fill according to the recommendations in Sections 4.4 and 4.5.

Materials and debris should be removed from the project site. With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), the upper 10 feet of subsurface material is suitable for reuse as engineered fill.

4.2 EXISTING FILL REMOVAL

As described in Section 3.2, artificial fill may be present onsite within the bounds of the basement. Based on the borings performed by T&R, we anticipate all artificial fill material will be excavated during basement construction.

If unexpected existing fill is encountered below proposed improvements during construction, we recommend removal of the fill to competent native soil, as evaluated by our field representative. If in a fill area, the base of the subexcavations should be processed, moisture conditioned (as needed), and compacted in accordance with the recommendations for engineered fill.

If existing fill is left in place in portions of the site that are being developed with walkways or other improvements that are not sensitive to settlement, on-going maintenance should be anticipated.

4.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, during or following periods of rain, within areas below the groundwater table, or beyond the extent of the dewatering program. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

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

We should evaluate Options 3 and 4 prior to implementation.

4.4 ACCEPTABLE FILL

4.4.1 Soil

Most onsite soil material is suitable as fill material provided it has a Plasticity Index (PI) less than 20 and it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index less than 12 and at least 20 percent passing the No. 200 sieve. It is important that we sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

4.4.2 Reuse of Onsite Recycled Materials

If desired, the existing asphalt, aggregate, and concrete can be considered for use as recycled aggregate to replace some of the import aggregate base for pavements, as well as for structural fill. The material will need to be broken down, but not pulverized, to have a maximum particle size less than 6 inches if used for fill and should conform to the gradations of aggregate base if used to substitute for roadway base.

4.5 FILL COMPACTION

4.5.1 Grading in Structural Areas

After removing the loose soil, the contractor should scarify to a depth of at least 8 inches then moisture condition and compact the subgrade in accordance with the table below. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

TABLE 4.5.1-1: Fill Placement Requirements

MATERIALS		FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM)
Low-Expansive	PI < 20	General Fill	90	3
		Upper 6 inches in Pavement Areas	95	1

The contractor should compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557), at a moisture content above the optimum.

4.5.2 Landscape Fill

In landscaping areas, the contractor should process, place, and compact fill in accordance with Section 4.5.1, but to at least 85 percent relative compaction.

4.5.3 Underground Utility Backfill

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

Utility trench backfill should conform to the recommendations in Section 4.5.1 and requirements by the appropriate jurisdiction, when applicable. Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath or into the building. The plug should be constructed using a sand-cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Jetting of backfill is not an acceptable means of compaction. Thicker loose lift thicknesses may be allowed based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

4.5.4 Controlled Low-Strength Material

Controlled low-strength material (CLSM) should consist of a fluid, workable mixture of aggregate, cement, and water. Aggregate should generally consist of sand, free of deleterious and organic material. The CLSM should have a maximum compressive strength of 50 psi. Prior to placement of CLSM, the base of the excavation should be cleared of loose material and standing water should be evacuated and controlled.

4.6 SITE DRAINAGE

The project Civil Engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, finish grades should be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations.

Landscaped areas are planned at finished grade elevations, as well as on top of structures. Proper subsurface drainage is required to prevent ponding on covered roofs or along walls. The roofs and drainage systems should be designed with appropriate slope to expediently transfer moisture across and off the roofs.

4.7 STORMWATER BIORETENTION AREAS

A clay layer was generally observed directly beneath the aggregate base. Thus, the existing site soil is not expected to have adequate permeability for stormwater infiltration, unless subdrains are installed. We recommend assuming little stormwater infiltration will occur through the existing site soil.

If bioretention areas are planned, we recommend that, when practical, they be placed a minimum of 5 feet away from property lines and 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 (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, 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 that will experience lateral loads (such as from impact or traffic), additional design considerations may be required. 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 that 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.

It should be noted that 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 minimize the exposure time such that the improvements are not detrimentally impacted.

5.0 FOUNDATION RECOMMENDATIONS

The main consideration in foundation design for this project is the potential for statically and seismically induced settlement. We developed foundation recommendations using data obtained from our exploration and engineering analyses.

5.1 STRUCTURAL MAT FOUNDATION

A combination of a structural mat foundation and waterproofing is a common system for structures founded below the groundwater table. This option avoids the need for permanent dewatering. Based on the depth of the excavation and groundwater depths, the mat foundation may have to be designed to resist hydrostatic uplift forces.

The thickness of the structural mat will be driven by the structural design. Similar buildings with similar constraints typically have mat foundations that are 3 to 4 feet or thicker. The structural mat should be designed to impose an average allowable bearing pressure corresponding to the acceptable settlement, as presented in Table 5.1-1, below. The provided bearing pressures and corresponding settlements are intended to be net average values acting over the entire footprint of the mat foundation and are applicable for long-term loading (allowable dead plus live loads). In addition, as discussed in Section 3.3.3, the total estimated liquefaction-induced settlement is estimated to be less than 1 inch.

TABLE 5.1-1: Structural Mat Foundation Allowable Bearing Capacities

AVERAGE ALLOWABLE BEARING CAPACITY	TOTAL STATIC SETTLEMENT	TOTAL DIFFERENTIAL SETTLEMENT
5,000 psf	Less than 1 inch	Less than ½ inch over 40 feet
6,000 psf	2½ inches	1¼ inches over 40 feet
6,500 psf	3 inches	1½ inches over 40 feet

The pressure can be locally increased under areas of high loads. In addition, the bearing capacities may be increased for temporary loading conditions; we will assess reported short-term loads provided by the structural engineer with further iterative analyses. At this time, the provided bearing capacities may be increased one-third for short-term loading conditions (wind and seismic).

If a spring constant is needed for design, the preliminary moduli of subgrade reaction (k_s) presented in Table 5.1-1 may be used. The following moduli are intended to serve as the initial step of an iterative process to refine the final moduli for the project.

TABLE 5.1-2: Moduli of Subgrade Reaction Based on Average Bearing Pressure

AVERAGE ALLOWABLE BEARING CAPACITY	MODULUS OF SUBGRADE REACTION (psi/in)
5,000 psf	35
6,000 psf	17
6,500 psf	15

These preliminary spring constants are provided based on the preliminary settlement analyses presented above. The structural designer should provide ENGEO with mat pressures and deflections based on these recommendations to optimize the design of the mat.

Resistance to short-duration (earthquake-induced) lateral loads may be provided by frictional resistance between the base of the foundation and the bearing soils and by passive earth pressure acting against the side of the foundation.

A coefficient of friction of 0.30 can be used between concrete and the subgrade. Where the bottom of the mat will be underlain by a waterproofing membrane, the coefficient of friction should be reduced further depending on membrane properties.

There have been several published results of shear tests with geomembranes (typically HDPE, PPE, or PVC) in contact with different soils or with other geosynthetics. The U.S. Bureau of Reclamation (USBR) Design Standards for Embankment Dams (DS-13, 2014) provides a summary of typical interface strength values for geomembranes against various materials that were compiled in a database collected by Koerner and Narejo (2005). For smooth HDPE material against granular soil, DS-13 provides a typical peak interface friction angle ($\phi_{if,p}$) of 21 degrees and a residual friction angle ($\phi_{if,r}$) of 17 degrees, which correspond to ultimate friction coefficients of about 0.4 and 0.3, respectively. Shear displacement plots indicate that peak friction angle is reached at very small displacements, on the order of 1 to 2 millimeters, whereas residual friction remains relatively constant over larger displacements (e.g. 1 inch). Based on this, we recommend an allowable coefficient of friction of 0.15. This coefficient can be increased by one-third for use in dynamic analyses.

The passive pressure is based on an equivalent fluid weight in pounds per cubic foot (pcf). Due to the site proximity to the Guadalupe River bank, less soil cover is available to provide full passive pressure along the western side of the building. As such, we have provided specific passive pressure values for various conditions at the site in Table 5.1-3.

TABLE 5.1-3: Allowable Passive Pressures

SCENARIO	ALLOWABLE PASSIVE PRESSURE
West Side	230 pcf
All Other Sides	260 pcf

We recommend neglecting the uppermost 12 inches of embedment at the ground surface of the passive pressures provided above. Passive lateral pressure should not be used for foundations on or above slopes.

5.2 UPLIFT FORCES

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

6.0 BASEMENT WALLS AND NON-BUILDING WALLS

6.1 SOIL PRESSURES

The basement walls will act as retaining walls. Basement walls should be designed for at-rest lateral loading conditions. Should cantilever retaining walls at the site be required, they can be designed for active lateral loading conditions. The recommended lateral equivalent fluid pressures (static case) are presented below.

TABLE 6.1-1: Lateral Earth Pressures

LOADING CONDITION	EQUIVALENT FLUID PRESSURES (PCF)	
	WITHOUT HYDROSTATIC PRESSURES (PCF)	WITH HYDROSTATIC PRESSURES (PCF)
Cantilevered (Active)	45	85
Restrained (At-Rest)	65	105

The above lateral earth pressures assume level backfill conditions. The design groundwater level should be assumed to be located at 15 feet below the existing ground surface. Permanent dewatering is not recommended below the design groundwater level, and basement walls should be designed to resist hydrostatic pressures. We recommend placing a drain behind all walls above the design groundwater level to reduce hydrostatic pressure; if a drain is not feasible, the basement walls should be designed with hydrostatic pressure. Recommendations for wall drainage follow in the next section.

Where surcharge loads from vehicles or other loads are expected within a horizontal distance equal to the height of the walls, the walls should be designed for an additional uniform lateral pressure of 125 psf to be applied over the entire height of the wall or the uppermost 10 feet, whichever is less. Passive pressures acting on retaining walls may be assumed as 300 pounds per cubic foot (pcf), provided that the area in front of the retaining walls is level for a distance of at least 10 feet or three times the depth of foundation, whichever is greater.

6.2 RETAINING WALL DRAINAGE

Unless the full height of the basement walls is designed for hydrostatic pressures, these walls should be provided with drainage facilities. Wall drainage may be provided using a

4-inch-diameter perforated pipe embedded in Class-2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about 1 foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper 1 foot of wall backfill should consist of clayey soils. Drainage should be collected by perforated pipes and discharged by gravity or directed to a sump(s).

All backfill should be placed in accordance with recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to minimize possible overstressing of the walls.

The foundation details and structural calculations for retaining walls should be submitted for our review.

6.3 SEISMIC DESIGN CONSIDERATIONS

Seismic conditions need to be considered in the design of the basement retaining walls. Under seismic conditions, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures, and can be calculated as follows.

$$\Delta P = 12 \times H^2$$

H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at 0.3 x H from the base of the wall. Since seismic loading requires soil movement, evaluation of the seismic case should include adding the seismic increment to the active soil pressure for all wall types. The above force has an equivalent triangular fluid pressure distribution of 24H.

7.0 TEMPORARY EXCAVATION SUPPORT AND DEWATERING

Excavation, dewatering, and shoring are temporary works that are typically the responsibility of the contractor to design, install, maintain and monitor. An experienced shoring and dewatering system designer should be retained to select and design these systems. The following sections provide some general considerations that should be incorporated into shoring and dewatering system design. Geotechnical shoring design recommendations are dependent on performance criteria, the type of system selected, and construction sequencing.

Where possible, temporary construction slopes may be used above the groundwater level. The soils at the site are considered to be “Type C” soils according to OSHA criteria. The contractor should establish appropriate setback distances from the tops of the slopes for vehicles, equipment and spoil piles, and should establish appropriate protective measures for exposed slope faces.

7.1 TEMPORARY SHORING

Temporary shoring will likely be required to facilitate site construction. Shoring design pressures and construction sequences should be selected to limit horizontal and vertical ground deformations due to shoring deflection.

Given the proposed excavation depth, it may be necessary to restrain the shoring by using a single-level or multi-level system of tie-back anchors or to provide internal bracing. Tie-back anchors should be installed to avoid adjacent underground utilities. The tiebacks may be installed through the selected shoring system with 15- to 20-degree inclinations. For cost estimating purposes, an ultimate grout-to-soil side friction of 1,000 psf along the "bonded zone" can be considered for post-grout tie backs. The recommended apparent lateral earth pressures to be used for temporary support of excavation are presented on Figure 9. Based on preliminary analyses, we anticipate shoring embedment will extend to at least 25 feet below the bottom of the excavation to provide excavation stability.

The water level should be maintained at least 3 feet below the bottom of the deepest excavation during construction. The selection of equipment and actual depth and spacing of the wells should be determined by the dewatering designer/contractor. We recommend selecting a dewatering system which has a minimal impact on the groundwater level surrounding the proposed excavation, such as an internal dewatering system.

7.1.1 Recommended Shoring Types

To reduce potential effects on the adjacent properties, we recommend the perimeter shoring system consist of a watertight system in which the design considers resistance to water pressures in addition to earth pressures such as an impervious soil-cement slurry cutoff wall system. Furthermore, the shoring system should extend adequately below the bottom of the excavation such that groundwater can be controlled from within the excavation and impacts to adjacent developments and the Guadalupe River can be minimized. Ultimately, the selection and design of the dewatering system should be the responsibility of the contractor.

7.1.1.1 Secant Pile Walls

Reinforced concrete secant piles are considered to be a watertight rigid shoring system which has the ability to limit the lateral deflection and resulting surface settlement around the excavation. The configuration of the secant piles can add stability to the excavation. A secant pile shoring system for the assumed excavation depth will likely require internal bracing and struts or tie-backs.

7.1.1.2 CDSM Cut-Off Walls

Cement deep soil mixing (CDSM) cut-off walls are an increasingly common shoring method around the San Francisco Bay Area. This method integrates soldier piles or king piles into the shoring system with CDSM being used as the watertight lagging. CDSM cut-off wall systems use a combined approach between soldier pile and wood lagging and slurry diaphragm walls because of the similar soldier pile configuration and the general type of equipment to be used.

7.2 PRE-CONSTRUCTION SURVEY AND CONSTRUCTION MONITORING

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

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

8.0 PAVEMENT DESIGN

We prepared pavement design recommendations based on assumed Traffic Index and subgrade resistance values (R-value). The Traffic Index should be determined by the Civil Engineer or appropriate public agency. The sections provided below should be reviewed and revised, if applicable, based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading.

8.1 FLEXIBLE PAVEMENTS

We developed the following pavement sections for parking areas and access streets using Traffic Indices of 5 to 9, based on an assumed R-value of 5 and Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety).

TABLE 8.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (AC) (INCHES)	CLASS 2 AGGREGATE BASE (AB) (INCHES)
5	4	7½
6	4	11½
7	4	15½
8	4½	18½
9	5	21½

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

We recommend that representative bulk samples of subgrade soil be obtained during street grading operations to allow confirmation R-value testing for the design R-value assumed above.

8.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections and reinforcement should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 Aggregate Base.
- Provide concrete with a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

8.3 PAVEMENT SUBGRADE PREPARATION

Pavement subgrade preparation should comply with the following minimum requirements:

- All pavement subgrades should be scarified to a depth of 10 inches below finished subgrade elevation and compacted in accordance with Section 4.5.1. Pavement subgrades should also be prepared in accordance with City of San Jose requirements if they are located in public streets.
- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.
- Adequate provisions must be made such that the subgrade soils and aggregate baserock materials are not allowed to become saturated.
- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted in accordance with Section 4.5.1. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.

8.4 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain towards pavement. If it is desired to install pavement cutoff barriers, they should be placed 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 6 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, the cutoff barrier may be eliminated.

9.0 SECONDARY SLABS-ON-GRADE

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor plazas exposed to foot traffic only. Concrete flatwork should have a minimum thickness of 4 inches and include control and construction joints in accordance with current Portland Cement Association guidelines.

Exterior slabs should slope away from the buildings to prevent water from flowing toward the foundations. Site soil should be moistened just prior to concrete placement.

We recommend that flatwork leading to a building entrance area be structurally independent of the building foundation to allow for differential movement between the flatwork and the building. Where smooth transition to provide access is necessary (ADA ramps), a hinge-slab should be designed to accommodate movements of approximately ½ inch. Flatwork should be reinforced to allow for the appropriate span in the event of settlement. Maintenance or replacement of entry slabs should also be expected following a seismic event as the ground settles at the perimeter of buildings.

10.0 CONSTRUCTION MONITORING

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

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to identify certain changes, which may have occurred in the nature, design, or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

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

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

11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the Almaden Office Complex project discussed in Section 1.3. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; 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 are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, fill, and groundwater, additional unexpected costs may be incurred in completing the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO should 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, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials should be notified immediately.

This document must not be subject to unauthorized reuse, that is, reuse 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 recommendations. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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

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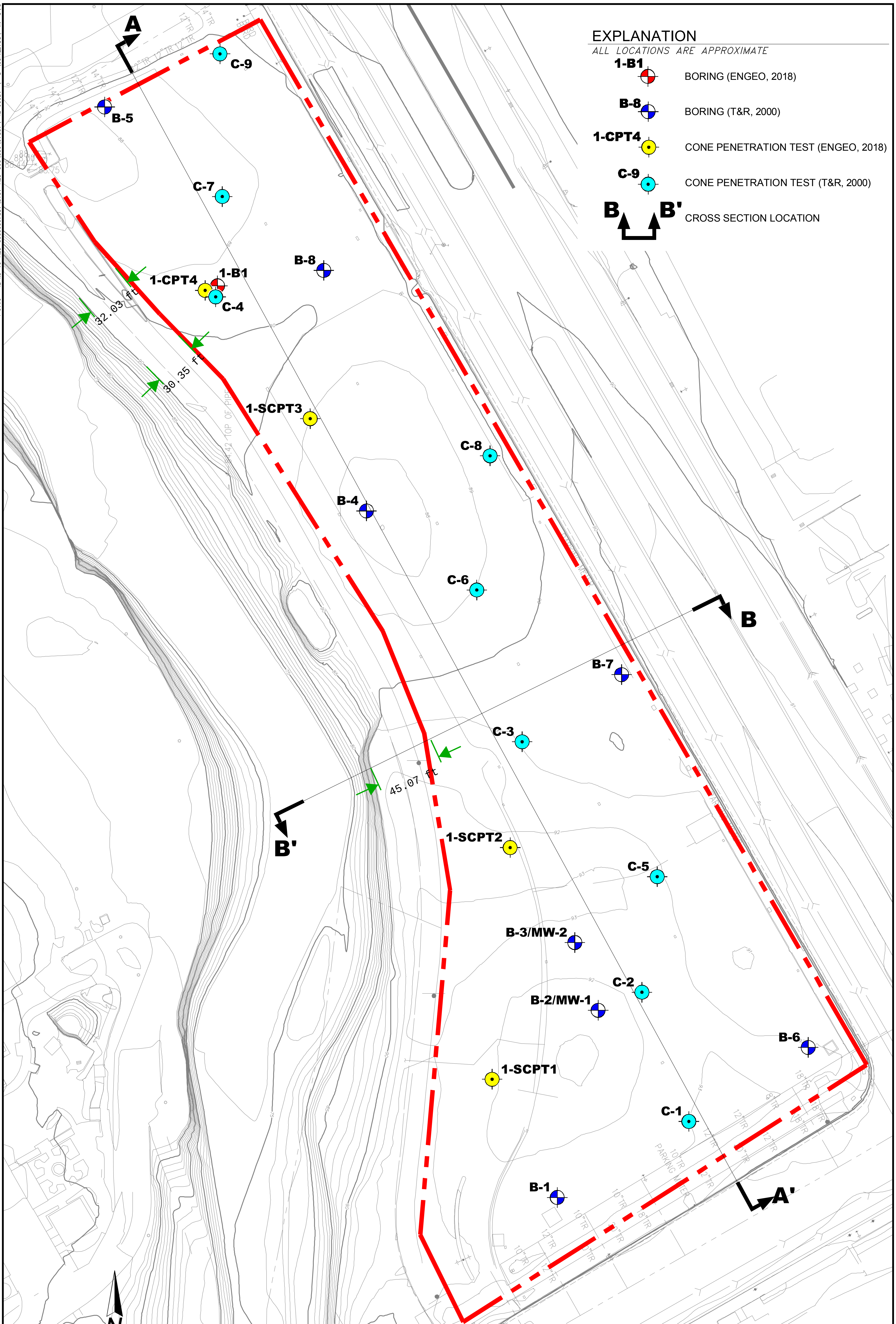
FIGURES

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP ALMADEN OFFICE COMPLEX SAN JOSE, CALIFORNIA	PROJECT NO.: 15540.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: PJE	FIGURE NO. 1



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SITE PLAN
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000		FIGURE NO.
SCALE: AS SHOWN		2A
DRAWN BY: GLJ	CHECKED BY: PJE	

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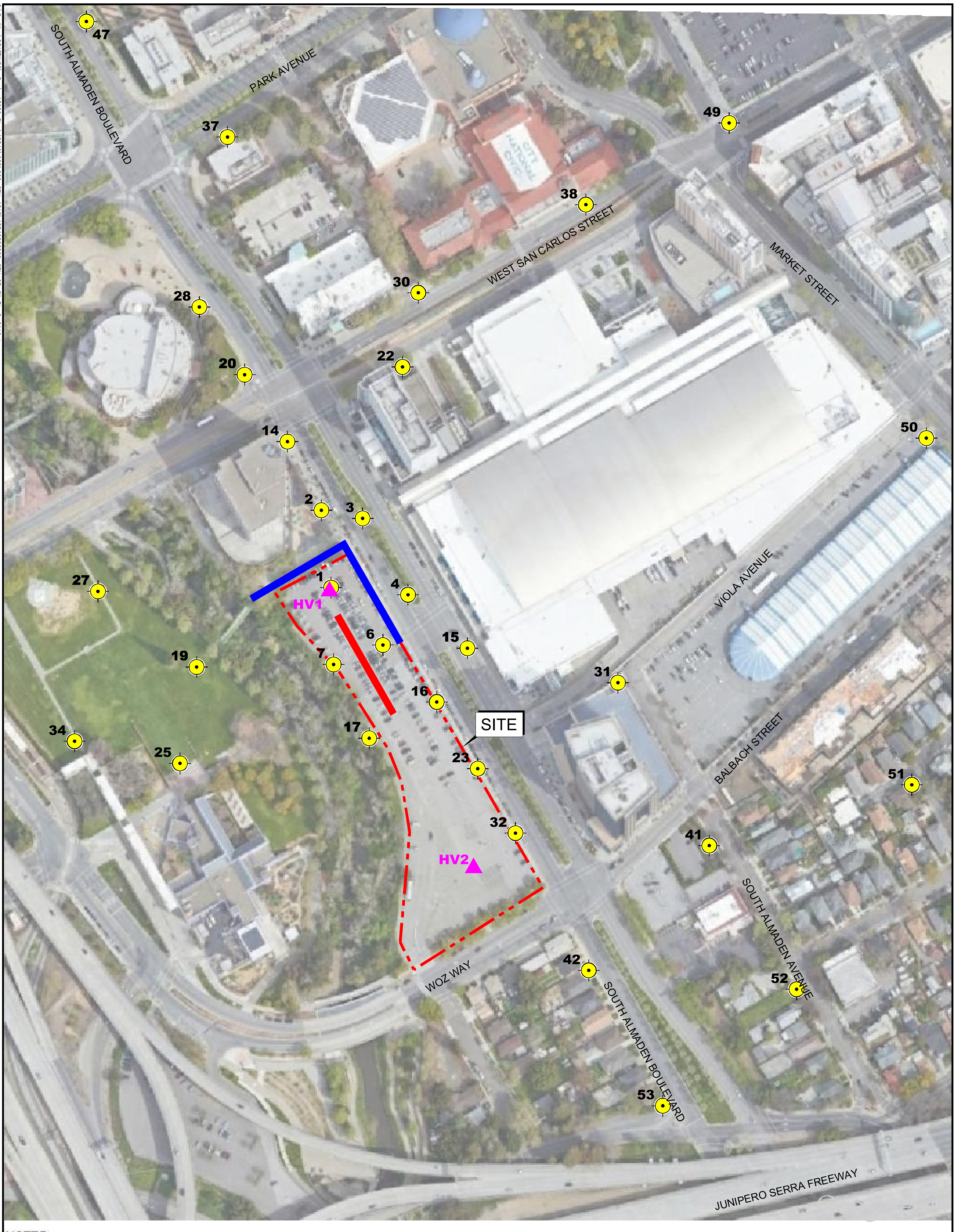

NOTES:
1. Coordinate System: California State Plane NAD83, Zone III (0403), US Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

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EXPLANATION
ALL LOCATIONS ARE APPROXIMATE
Array 1 - Small Aperture Microtremor Array (L-Shaped)
Array 2 - Active Surface Wave Array (MASW)
Array 3 - Large Aperture Mictrotremor Array Sensor Location
H/V Spectral Ratio Measurement Location

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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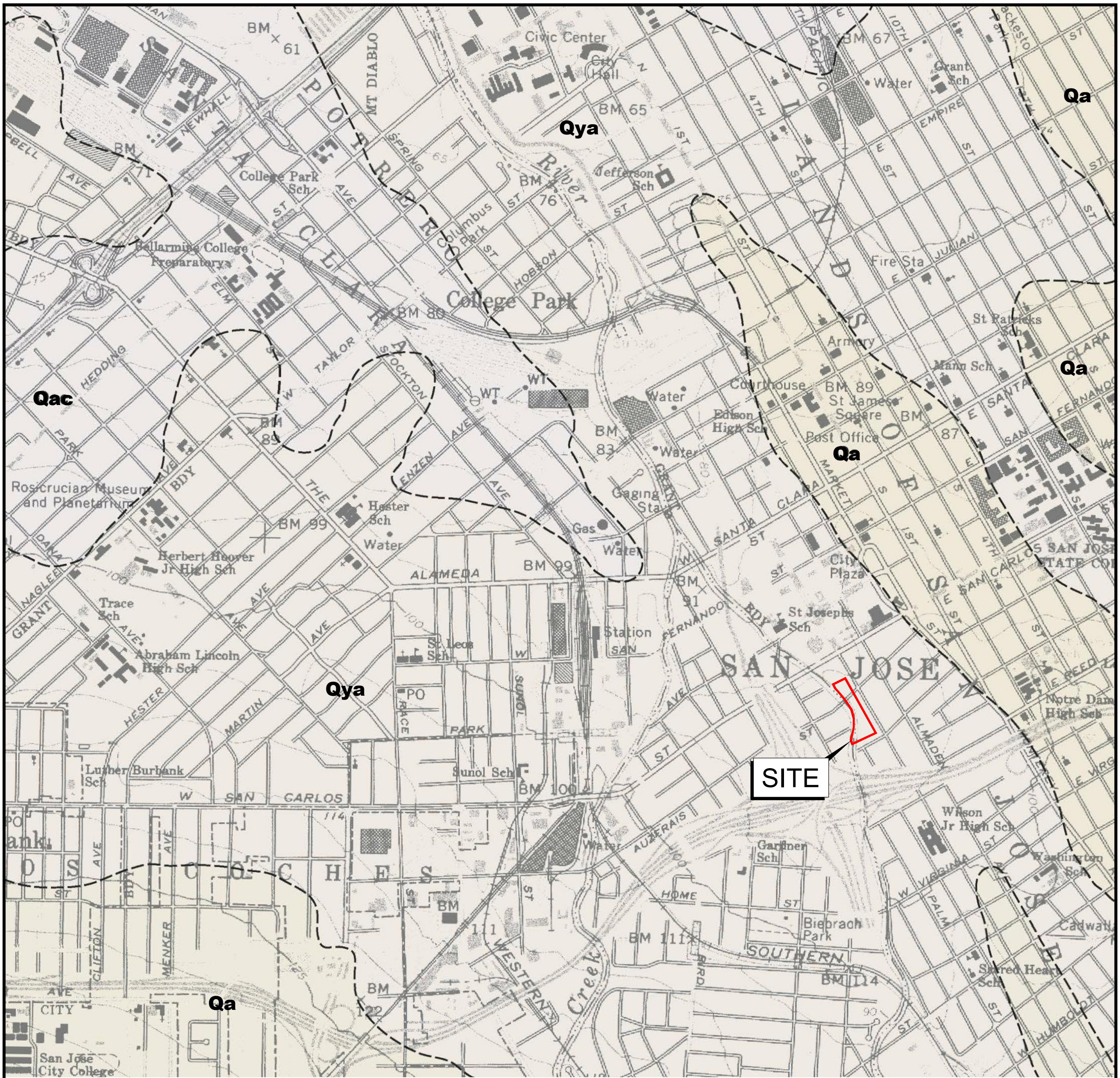
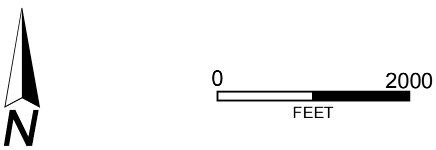
SURFACE WAVE TESTING LOCATIONS
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000
SCALE: AS SHOWN
DRAWN BY: GLJ
CHECKED BY: PJE
FIGURE NO. 2B

ORIGINAL FIGURE PRINTED IN COLOR





EXPLANATION

——— – – – –	GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED
Qa	ALLUVIAL GRAVEL, SAND, SILT, AND CLAY
Qya	ALLUVIAL SAND, FINE-GRAINED SAND, SILT, AND CLAY
Qac	SILTY CLAY AND ORGANIC CLAY

BASE MAP SOURCE: DIBBLEE, 2007

ENGEO Expect Excellence	REGIONAL GEOLOGIC MAP ALMADEN OFFICE COMPLEX SAN JOSE, CALIFORNIA	PROJECT NO.: 15540.000.000 SCALE: AS SHOWN DRAWN BY: GLJ — CHECKED BY: PJE	FIGURE NO. 3

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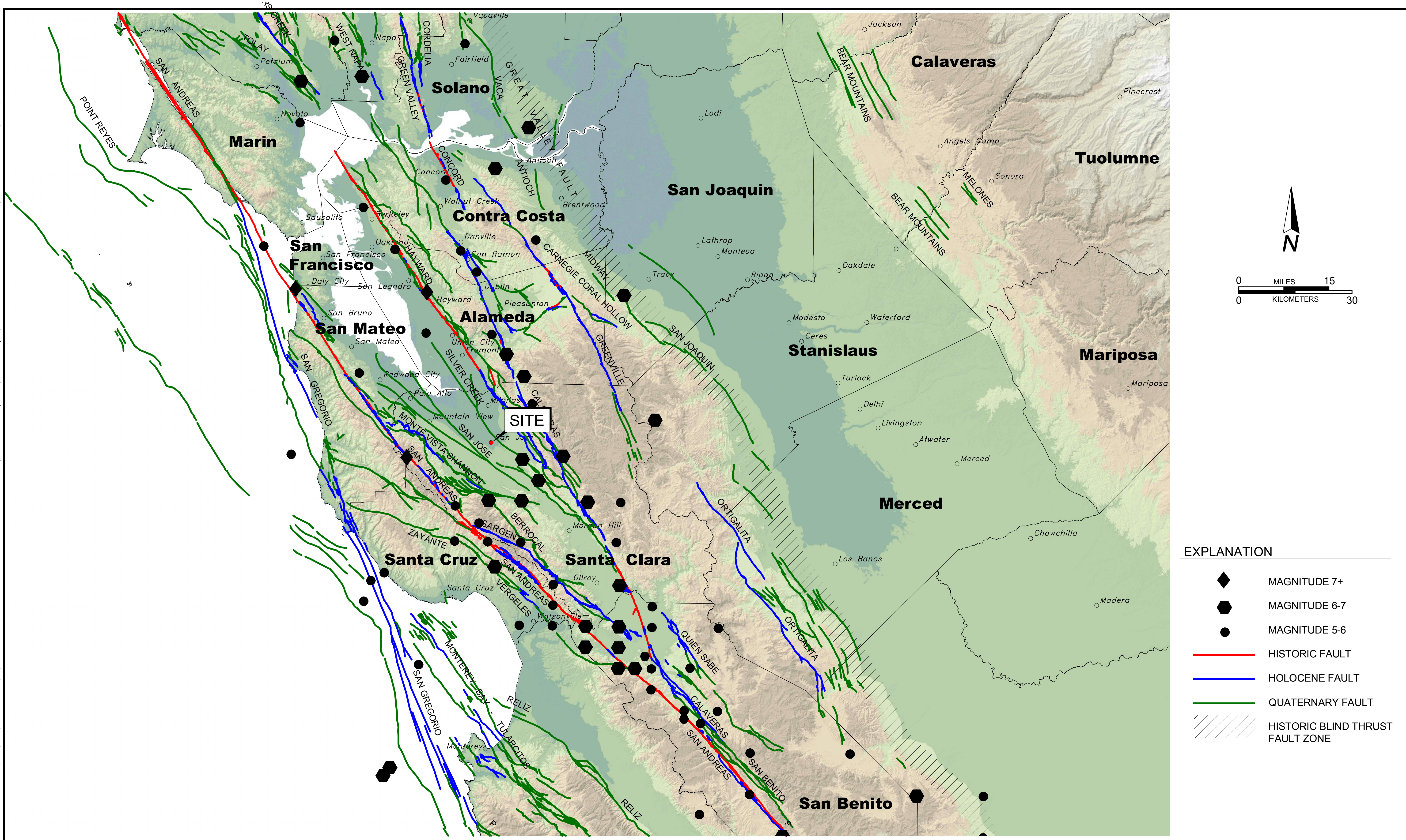

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

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REGIONAL FAULTING AND SEISMICITY
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000	FIGURE NO. 4
SCALE: AS SHOWN	
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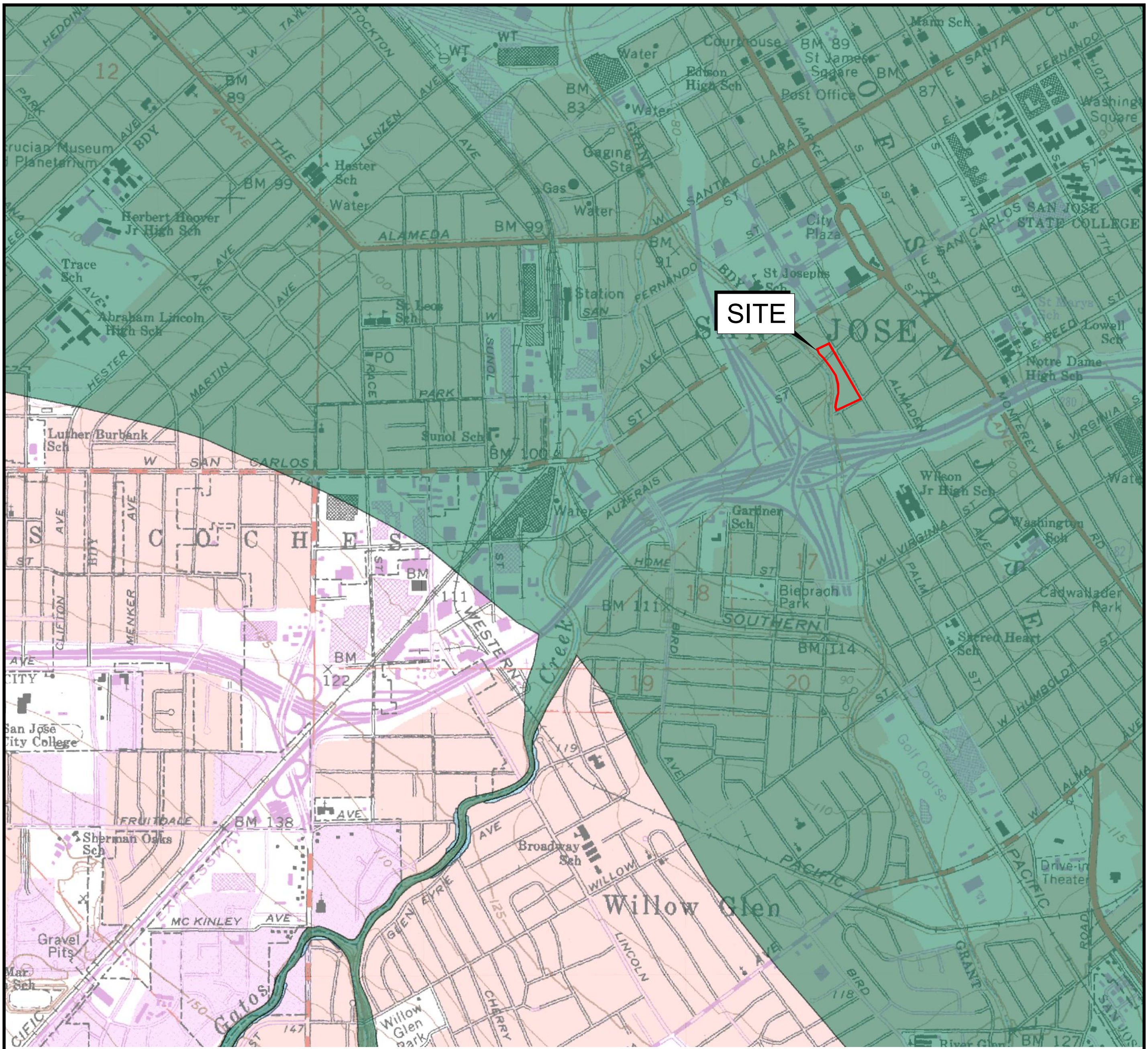

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EXPLANATION

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 ALMADEN OFFICE COMPLEX SAN JOSE, CALIFORNIA	PROJECT NO.: 15540.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: PJE	FIGURE NO. 5

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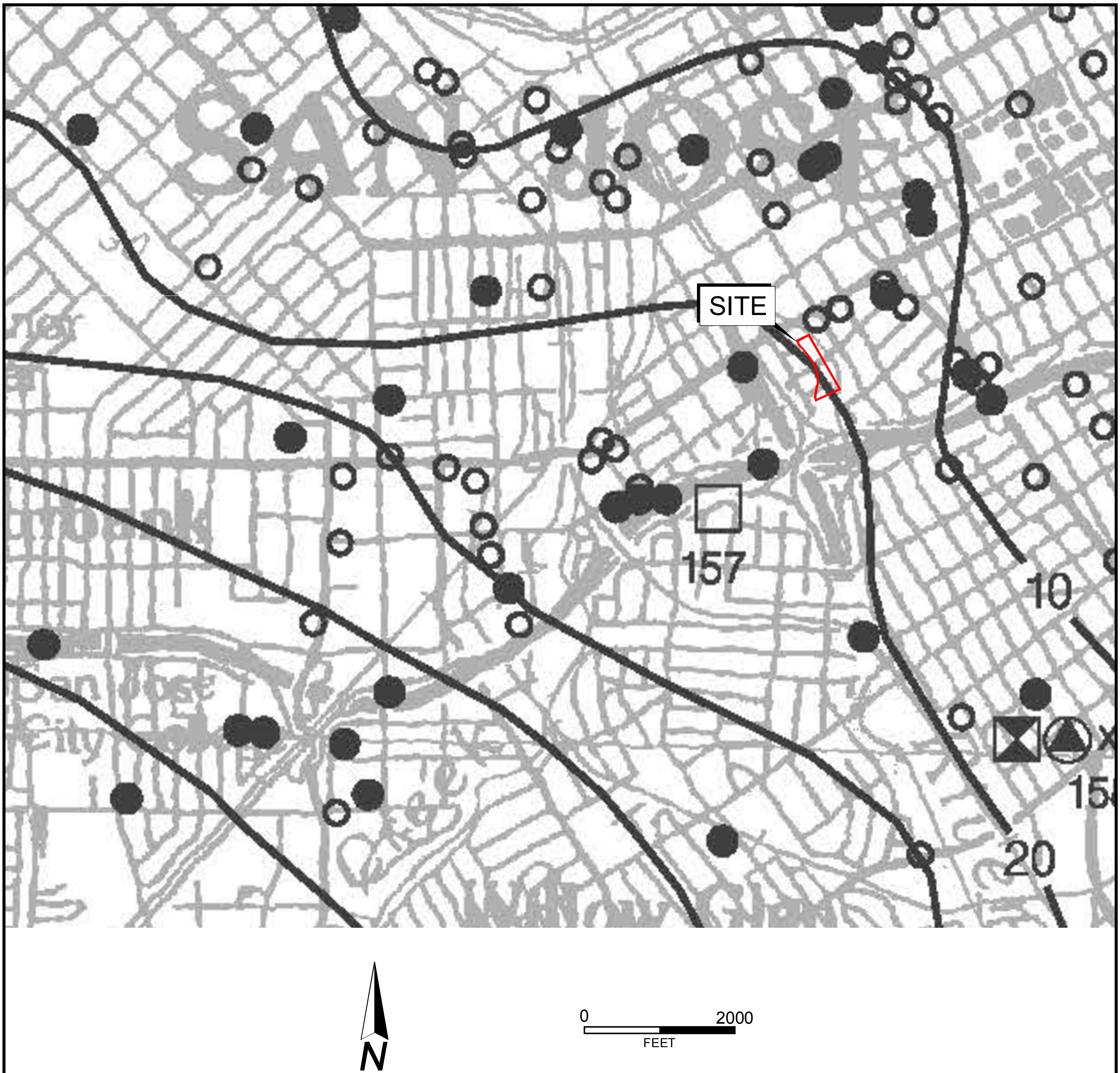
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- x Location of multiple ground effects. (See corresponding symbols)
- Cracks in streets
- Disturbed well
- Sand boil
- Miscellaneous effects
- Absence of ground failure noted
- Ground settlement
- Reach of Coyote Creek along which multiple failures were recorded. Symbol shows failure type.
- Lateral spread
- 152 Number assigned to ground failure site (adapted from Youd and Hoose, 1978, and Tinsley and others, 1998, by Knudsen and others, 2000).
- Bedrock
- 10 Depth to ground water, in feet
- Geotechnical boreholes used in liquefaction evaluation
- Ground–water level data provided by the Santa Clara Valley Water District

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

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HISTORIC HIGH GROUNDWATER CONTOURS MAP
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000		FIGURE NO. 6
SCALE: AS SHOWN		
DRAWN BY: GLJ	CHECKED BY: PJE	

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EXPLANATION

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

	1% annual chance floodplain boundary
	0.2% annual chance floodplain boundary
	Floodway boundary
	Zone D boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
513	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

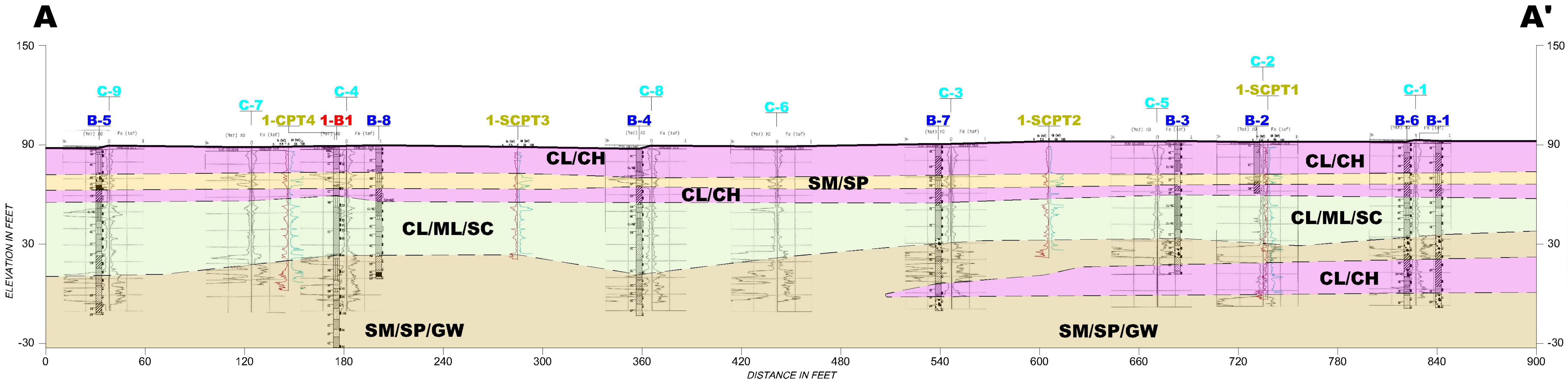
A — A	Cross section line
23 - - - 23	Transect line
87°07'45", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
4276000mN	1000-meter Universal Transverse Mercator grid values, zone 10N
600000 FT	5000-foot grid ticks: California State Plane coordinate system, zone III (FIPSZONE 0403), Lambert Conformal Conic projection
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
● M1.5	River Mile

BASE MAP SOURCE: FEMA FIRM MAP

ENGEO Expect Excellence	FEMA FLOOD INSURANCE MAP ALMADEN OFFICE COMPLEX SAN JOSE, CALIFORNIA	PROJECT NO.: 15540.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: PJE	FIGURE NO. 7

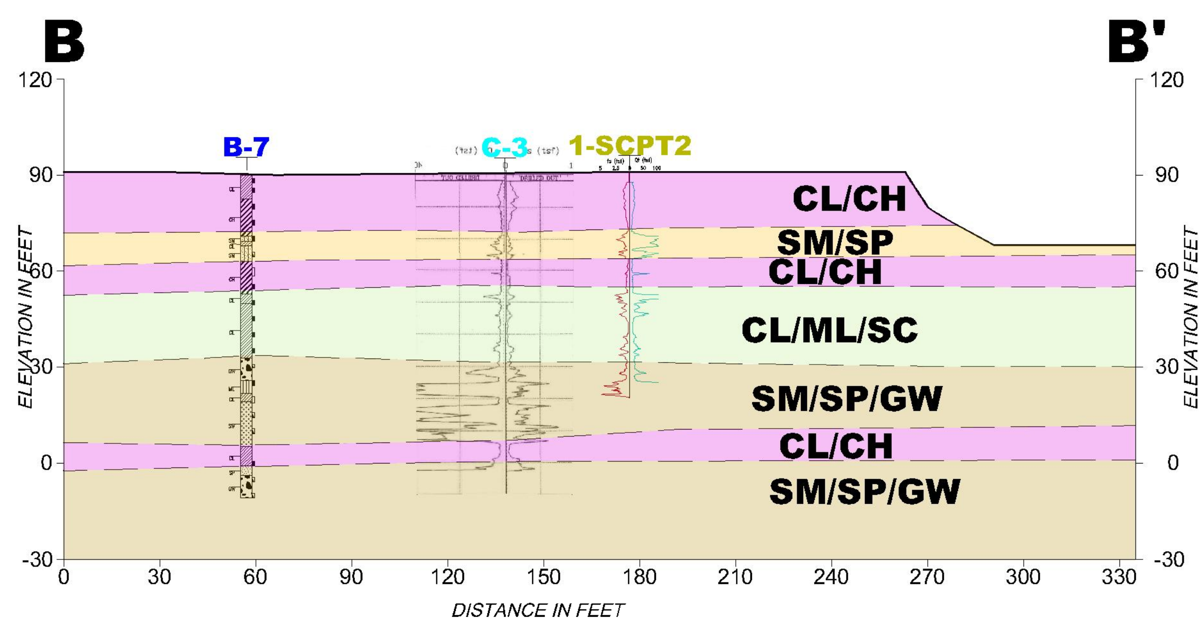
ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg_13000 Plus\15540\000\000-GEX-1218\15540000000-GEX-7-FIRM-0119.dwg



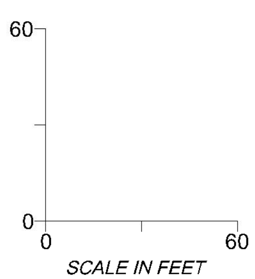
SECTION A-A'

1"=60'



SECTION B-B'

1"=60'



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B1	BORING (ENGEO, 2018)
B-8	BORING (T&R, 2018)
1-SCPT4	CONE PENETRATION TEST (ENGEO, 2018)
C-9	CONE PENETRATION TEST (T&R, 2018)
SM/SP	SILTY SAND/POORLY GRADED SAND
CL/ML/SC	LEAN CLAY/SILT/CLAYEY SAND
CL/CH	LEAN CLAY/FAT CLAY
SM/SP/GW	SILTY SAND/POORLY GRADED SAND/WELL GRADED GRAVEL

ENGEO — Expect Excellence —

CROSS SECTIONS A-A' AND B-B'

ALMADEN OFFICE COMPLEX

SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000

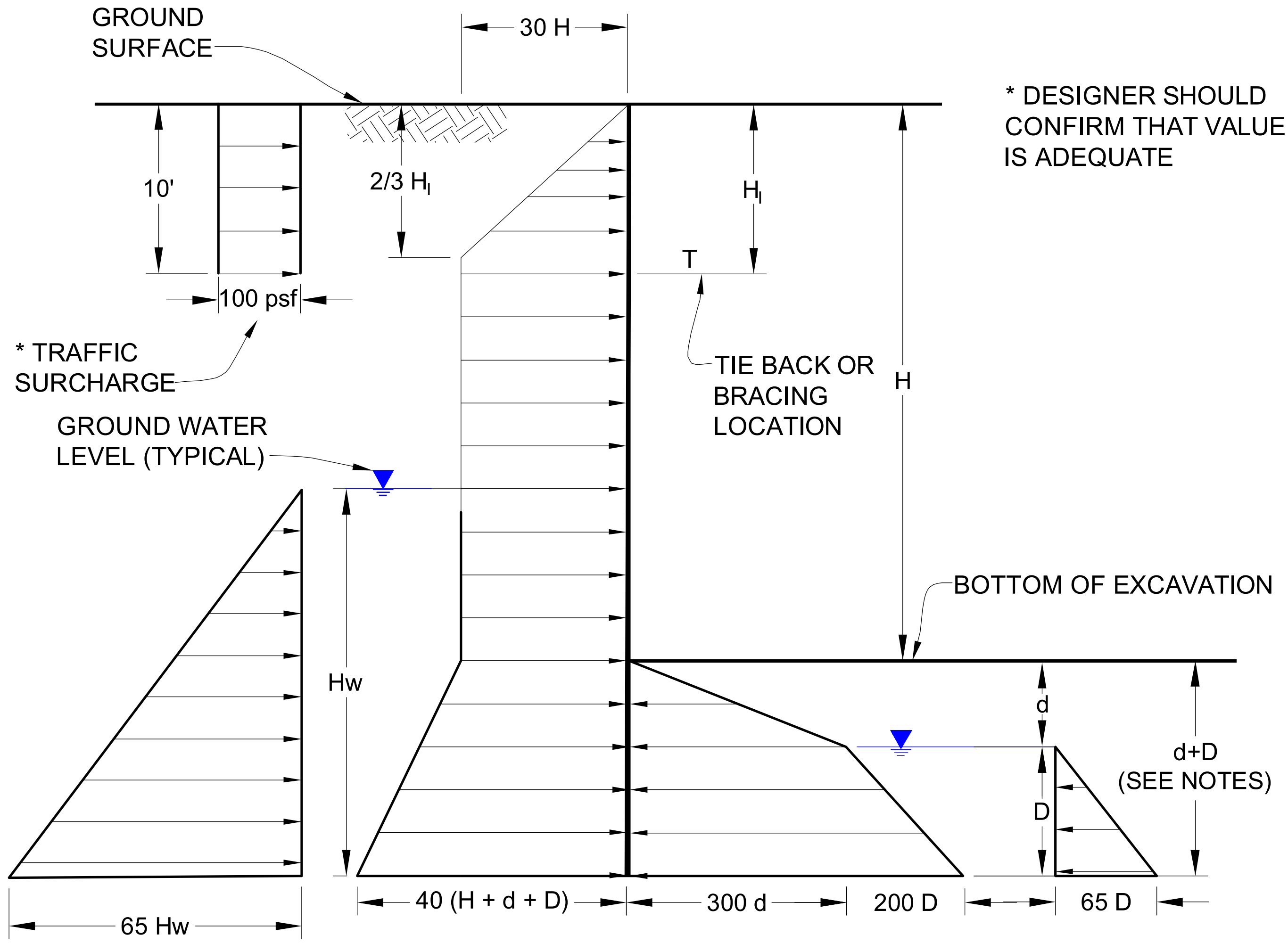
SCALE: AS SHOWN

DRAWN BY: GLJ

CHECKED BY: PJE

FIGURE NO. 8

G:\Drafting\DRAFTING2_Dwg_13000 Plus\15540\000\000-GEX-1218\15540000000-GEX-8-CrossSection-0119.dwg



NOTES:

1) WATER TABLE IS ASSUMED TO BE A MINIMUM OF 3 FEET BELOW BOTTOM OF EXCAVATION

2) D, d, H, H AND Hw ARE TO BE IN FEET. PRESSURES ARE IN PSF

3) SAFETY FACTOR TO BE INCLUDED BY DESIGNER

4) ASSUMES LEVEL GROUND SURFACE AT TOP OF SHORING

5) ASSUMES INTERNALLY BRACED SYSTEMS

6) D+d MUST BE ADEQUATE TO PROVIDE STABILITY TO THE BOTTOM OF THE EXCAVATION AND TO PREVENT PIPING OF WATER

7) FOR CANTILEVER CONDITIONS, ACTIVE EARTH PRESSURES SHOULD BE AS SHOWN IN TABLE 7.1-1 OF THIS REPORT

ENGEO Expect Excellence	TEMPORARY SHORING PRESSURE DIAGRAM ALMADEN OFFICE COMPLEX SAN JOSE, CALIFORNIA	PROJECT NO.: 15540.000.000 SCALE: NO SCALE DRAWN BY: GLJ CHECKED BY: PJE	FIGURE NO. 9

G:\Drafting\DRAFTING2_Dwg_13000 Plus\15540\000\000-GEX-1218\15540000000-GEX-9-TempShoreDia-0119.dwg

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

KEY TO BORING LOGS
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 — CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4 "	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

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

CONSISTENCY

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

SAMPLER SYMBOLS

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

MOISTURE CONDITION

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

LINE TYPES

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

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

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

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

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LOG OF BORING 1-B1

LATITUDE: 37.327997 LONGITUDE: -121.890984

Geotechnical Exploration
Almaden Office Complex
San Jose, CA
15540.000.000

DATE DRILLED: 10/27/2018
HOLE DEPTH: Approx. 121½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (NAVD88): Approx. 89½ ft.

LOGGED / REVIEWED BY: I. McCreery / UE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: 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)	Unconfined Strength (tsf) *field approx
			3½" AC over 6" AB										
5	85		SILTY CLAY (CL), dark olive brown, very stiff, slightly moist, 5 to 10% sand										
			POORLY GRADED SAND WITH SILT (SP), dark olive brown, loose, slightly moist			7	41	23	18				2.5*
			SILTY CLAY WITH SAND (CL), dark olive brown, very stiff, slightly moist										
10	80		SILTY CLAY (CL), gray mottled with olive brown, very stiff, slightly moist, low plasticity, iron oxide staining			22	33	19	14				2.75*
15	75		SANDY SILT (ML), olive brown to gray, moist, fine-grained sand			30				56			
			POORLY GRADED SAND (SP), olive brown to gray, dense, moist, 5 to 10% gravel										
20	70												
25	65												

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LOG OF BORING 1-B1

LATITUDE: 37.327997 LONGITUDE: -121.890984

Geotechnical Exploration
Almaden Office Complex
San Jose, CA
15540.000.000

DATE DRILLED: 10/27/2018
HOLE DEPTH: Approx. 121½ ft.
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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)	Unconfined Strength (tsf) *field approx
			LEAN CLAY (CL), dark brown to gray, soft, medium plasticity			5							.25*
	60												
30													
	55												
35													
			SILTY SAND (SM), gray, medium dense, wet			23				39			.75*
			SANDY SILT (ML), gray, medium stiff, wet, low plasticity Triax UU = 1495 psf				28	24	4		24.5	99.9	0.75
	50												
40													
			SANDY SILT (ML), olive brown, wet, non plastic plasticity				NP	NP	NP		21.6	105.7	
			SILTY CLAY (CL), olive brown mottled with orange brown, stiff to very stiff, wet, low plasticity			25					27.5	98.5	1.09
	45												
45													
			LEAN CLAY (CL), gray, soft to medium stiff, wet, medium plasticity, <5% silt			10					26.3	98.7	.25*
	40												
50													

ENGEO
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LOG OF BORING 1-B1

LATITUDE: 37.327997 LONGITUDE: -121.890984

Geotechnical Exploration
Almaden Office Complex
San Jose, CA
15540.000.000

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DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: 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)	Unconfined Strength (tsf) *field approx
			LEAN CLAY (CL), gray, medium stiff to stiff, wet, medium plasticity, <5% silt			25	29	15	14		19	110.6	1*
55	35		Stiff			15					21.6	106.8	1.25* 1.06
60	30												
65	25		Yellowish brown, medium stiff to very stiff, moist			56					18.1	114.4	0.47
			POORLY GRADED SAND (SP), dark yellowish brown, dense to very dense, wet, 5 to 10% gravel										
70	20												
75	15												

ENGEO
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LOG OF BORING 1-B1

LATITUDE: 37.327997 LONGITUDE: -121.890984

Geotechnical Exploration
Almaden Office Complex
San Jose, CA
15540.000.000

DATE DRILLED: 10/27/2018
HOLE DEPTH: Approx. 121½ ft.
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LOGGED / REVIEWED BY: I. McCreery / UE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: 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)	Unconfined Strength (tsf) *field approx
			SILTY SAND (SM), gray, dense, wet			47				70			2.5*
	10												
80													
	5												
85													
			POORLY GRADED SAND (SP), olive brown, dense, wet			45					11.2		
			SILTY SAND (SM), olive brown to orange brown, dense, wet										
	0												
90													
			POORLY GRADED SAND (SP), olive brown, very dense, wet			53					18.9	113.1	
	-5												
95													
			LEAN CLAY (CL), gray, medium stiff, wet			2					24		.75*
	-10												
100													

LOG OF BORING 1-B1

LATITUDE: 37.327997 LONGITUDE: -121.890984

ENGEO Expect Excellence

Geotechnical Exploration
Almaden Office Complex
San Jose, CA
15540.000.000

DATE DRILLED: 10/27/2018
HOLE DEPTH: Approx. 121½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (NAVD88): Approx. 89½ ft.

LOGGED / REVIEWED BY: I. McCreery / UE
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

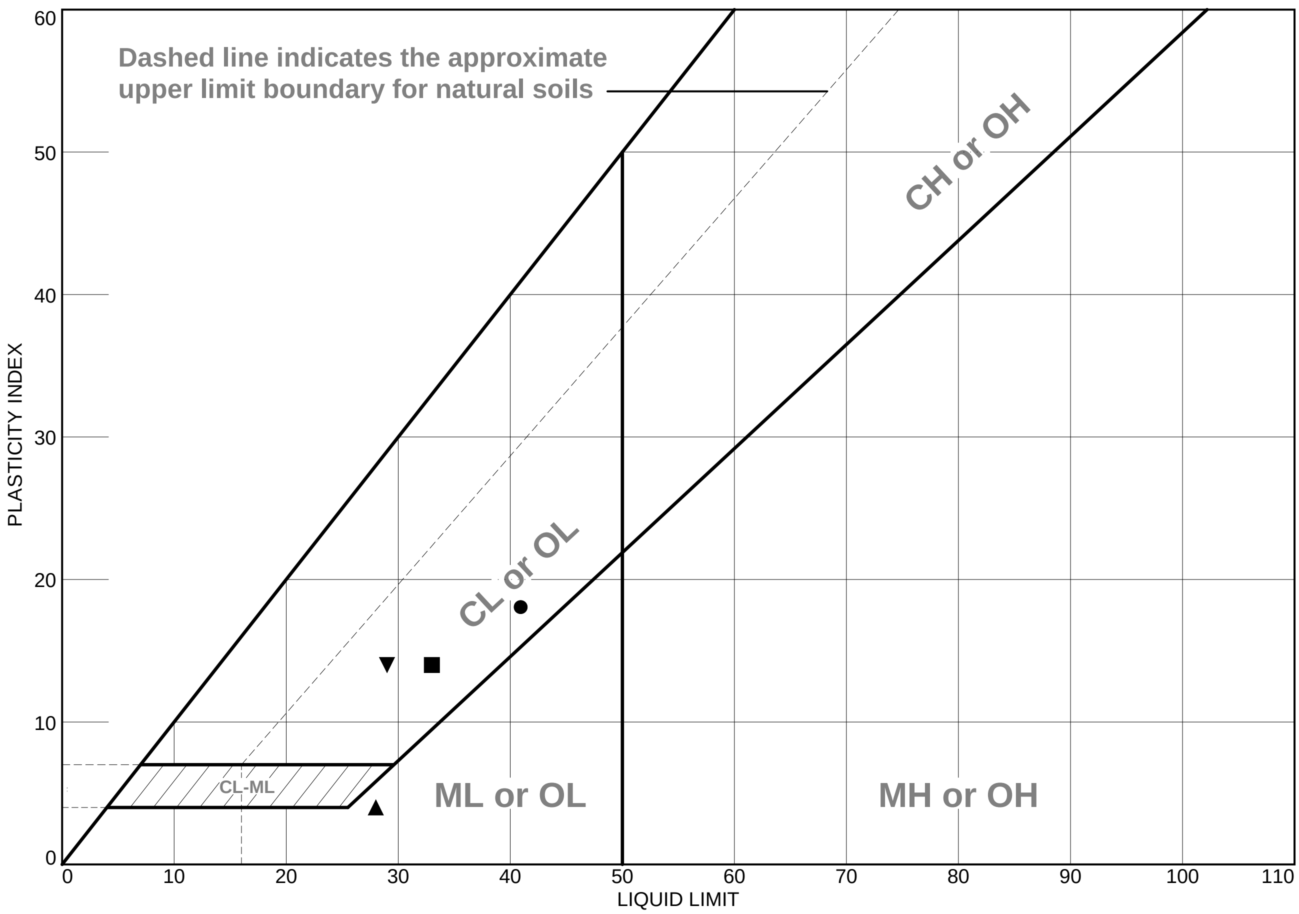
Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			POORLY GRADED SAND (SP), dark olive brown, very dense, wet, 5 to 10% gravel			>50					11		
105	-15												
110	-20		SILTY CLAY (CL), olive brown, hard, wet										
			POORLY GRADED SAND (SP), olive brown, very dense, wet, 5 to 10% silt, 5 to 10% gravel			66							4.5*
115	-25												
120	-30		LEAN CLAY (CL), dark olive brown to gray, very stiff, wet			19					24.5	101.3	

End of boring at 121½ feet below ground surface.
Groundwater not encountered due to to drilling method.

APPENDIX B

LABORATORY TEST RESULTS

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	41	23	18			
■	See exploration logs	33	19	14			
▲	See exploration logs	28	24	4			
◆	See exploration logs	NP	NP	NP			
▼	See exploration logs	29	15	14			

Project No. 15540.000.000 **Client:** Boston Properties, Inc.

Project: Almaden Office Complex

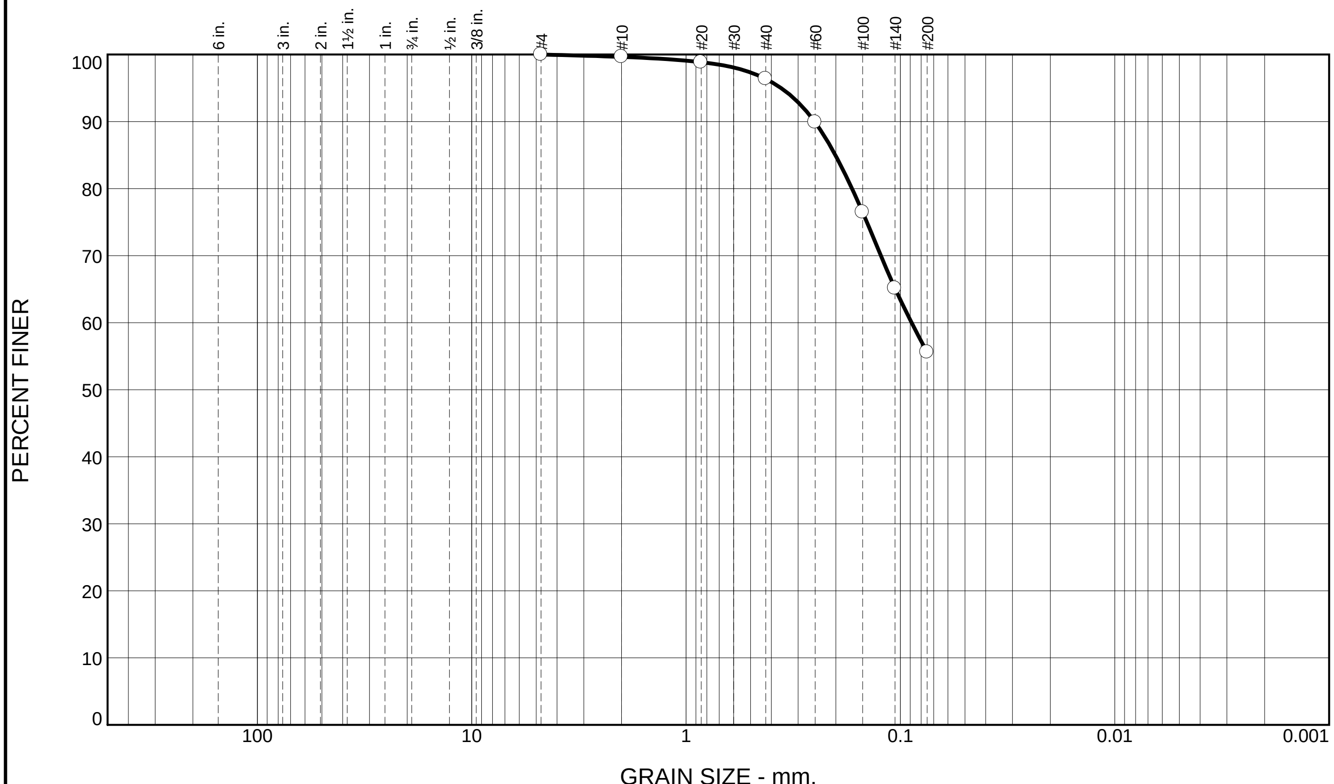
●**Depth:** 5.0 feet **Sample Number:** 1-B1 @ 5
■**Depth:** 11.0 feet **Sample Number:** 1-B1 @ 11
▲**Depth:** 38.0 feet **Sample Number:** 1-B1 @ 38
◆**Depth:** 41.0 feet **Sample Number:** 1-B1 @ 41
▼**Depth:** 51.0-51.5 feet **Sample Number:** 1-B1 @ 51-51.5

Remarks:
●ASTM D4318, Wet method
■ASTM D4318, Wet method
▲ASTM D4318, Wet method
◆ASTM D4318, Wet method
▼ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: ○ M. Bromfield □ M. Bromfield △ M. Bromfield ◇ M. Bromfield ▽ M. Quasem **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.3	3.3	40.8	55.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.7		
#20	98.9		
#40	96.4		
#60	89.9		
#100	76.5		
#140	65.1		
#200	55.6		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2516 D_{85}= 0.2012 D_{60}= 0.0887

D_{50}= D_{30}= D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913, Method A

Sample Number: 1-B1 @ 15

Date: 11/26/2018

ENGEO INCORPORATED

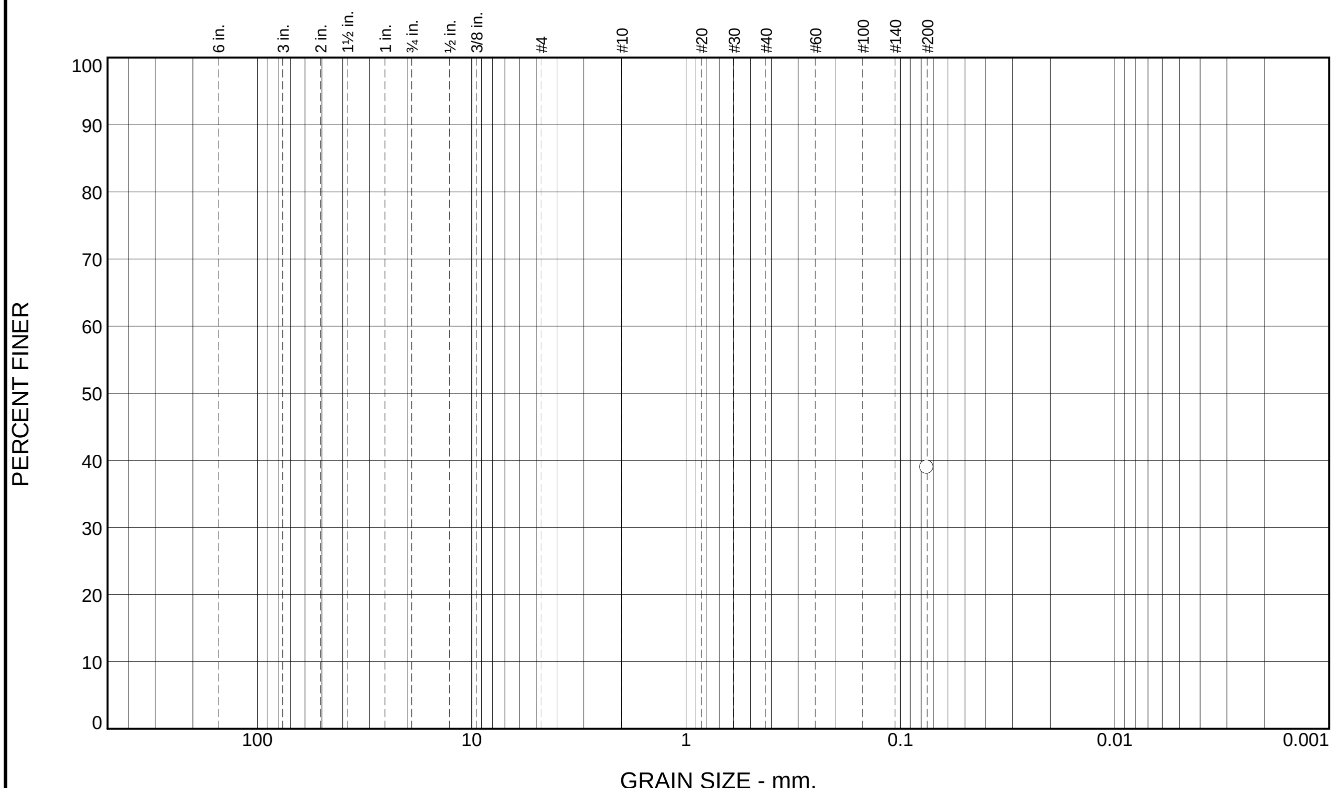
Client: Boston Properties, Inc.

Project: Almaden Office Complex

Project No: 15540.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +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		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 189.94; Soak Time = 4 hrs

Sample Number: 1-B1 @ 36

Date: 11/21/2018

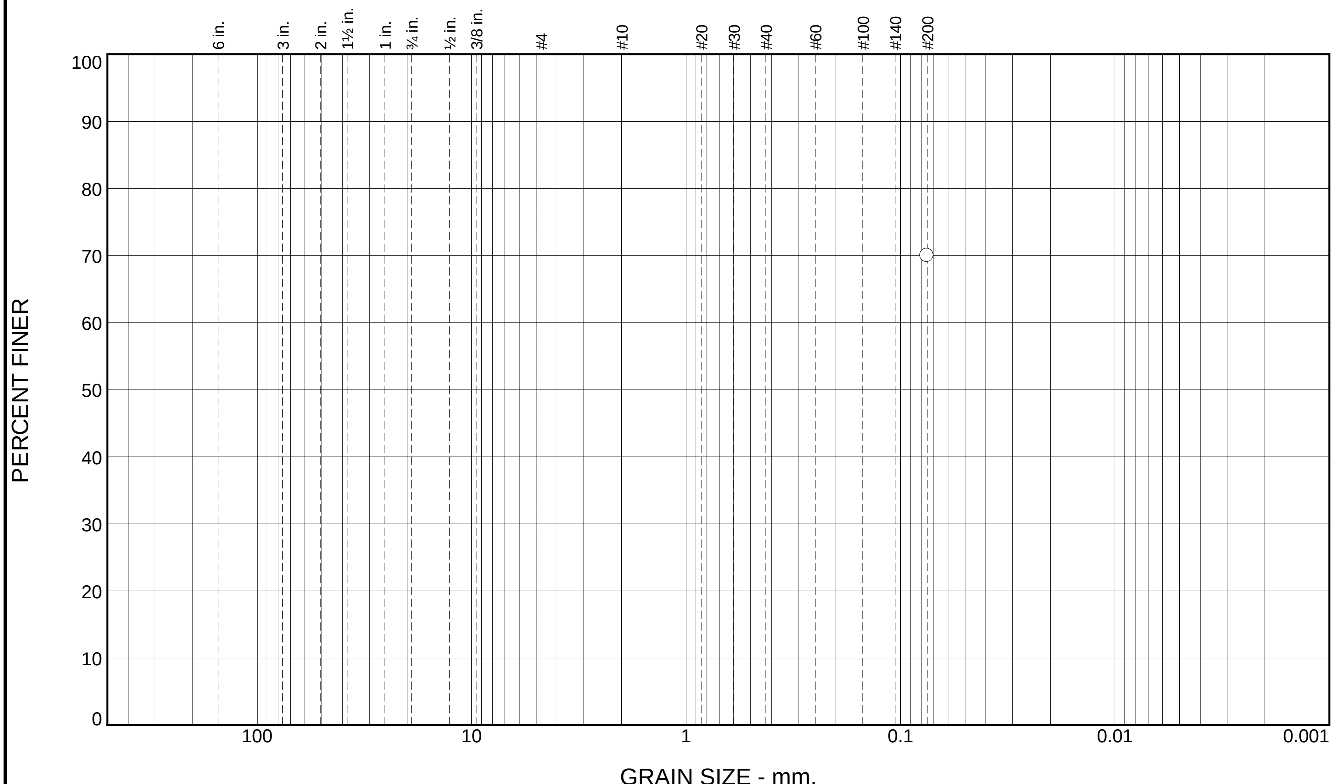
ENGEO INCORPORATED

Client: Boston Properties, Inc.
Project: Almaden Office Complex

Project No: 15540.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						70.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	70.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 199.06; Soak Time = 4 hrs

Sample Number: 1-B1 @ 76

Date: 11/21/2018

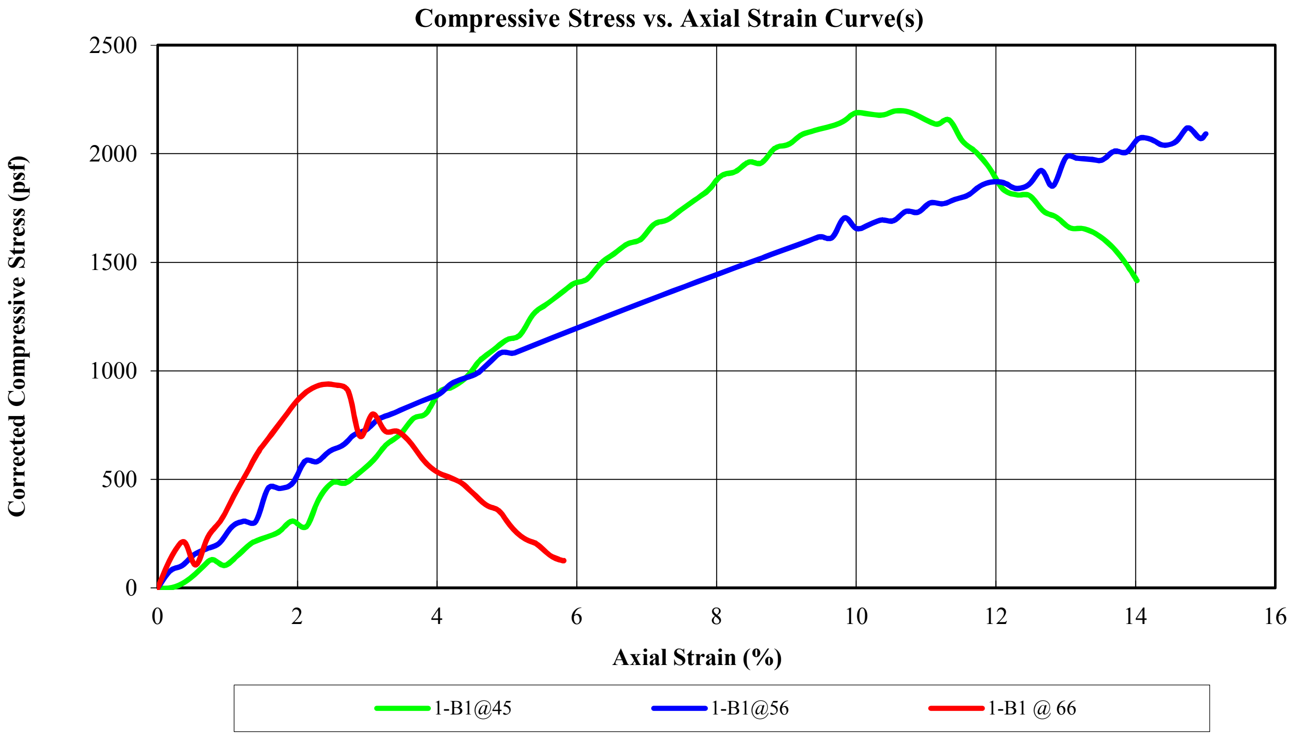
ENGEO INCORPORATED

Client: Boston Properties, Inc.
Project: Almaden Office Complex

Project No: 15540.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



	SPECIMEN		
BEFORE TEST	**1-B1@45**	**1-B1@56**	**1-B1 @ 66**
Moisture Content (%)	27.5	21.6	18.1
Dry Density (pcf)	98.5	106.8	114.4
Saturation (%)	99.5	100.0	100.0
Void Ratio	0.77	0.58	0.50
Diameter (in)	2.405	2.416	2.382
Height (in)	5.35	5.85	5.68
Height-To-Diameter Ratio	2.22	2.42	2.38
TEST DATA			
Unconfined Compressive Strength (psf)	2197	2119	936
Undrained Shear Strength (psf)	1098	1060	468
Strain Rate (in./min.)	0.05	0.05	0.05
Specific Gravity (Assumed)	2.800	2.800	2.800
Strain at Failure (%)	10.56	14.75	2.35
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Test Remarks			

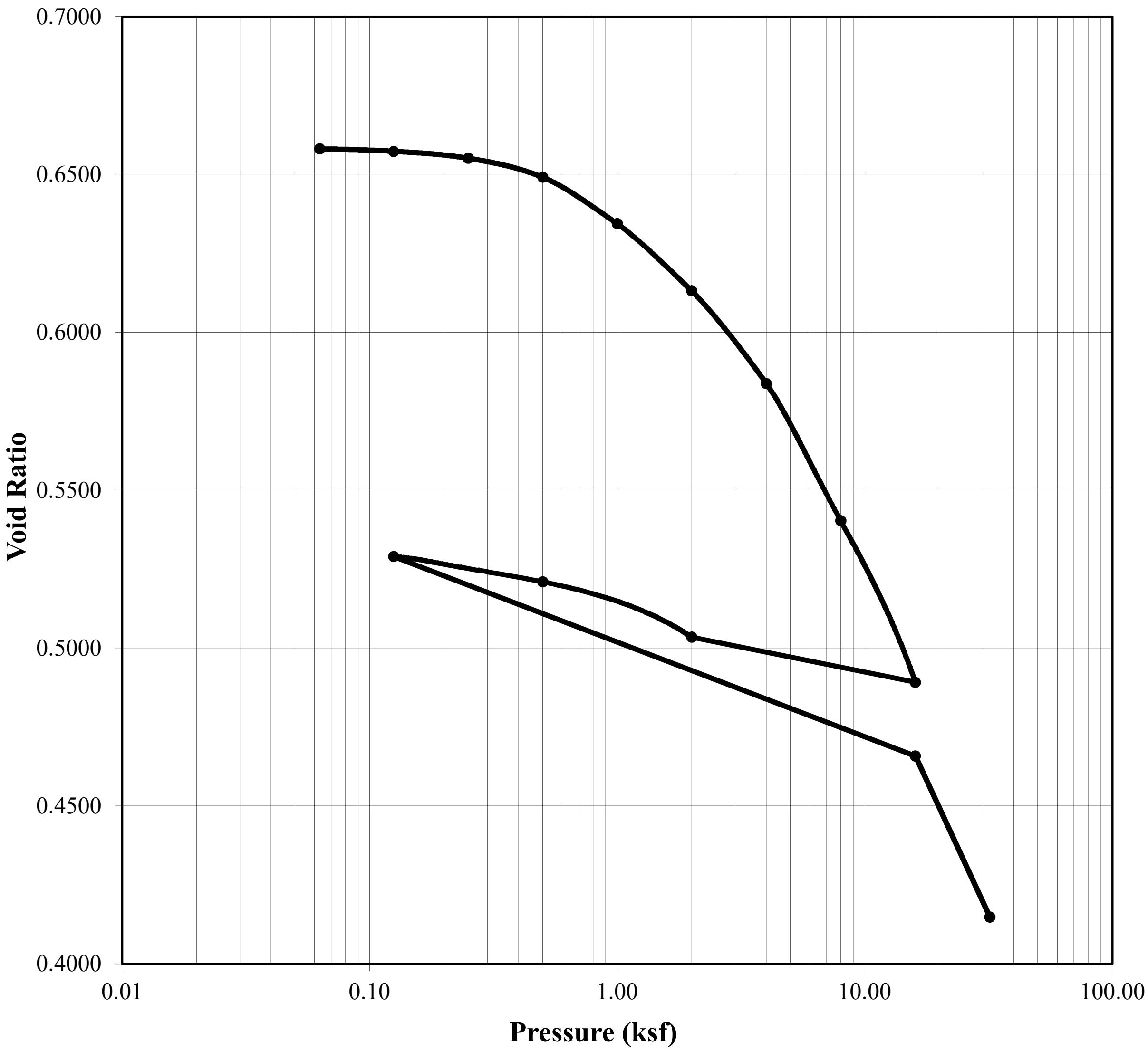
SPECIMEN	DESCRIPTION
1-B1@45	See exploration logs
1-B1@56	See exploration logs
1-B1 @ 66	See exploration logs

PROJECT NAME: Almaden Office Complex
PROJECT NO: 15540.000.000
CLIENT: Boston Properties, Inc.
LOCATION: San Jose, CA
PHASE NO: 001

Test Date: 11/20/18
Tested By: M. Bromfield
Reviewed By: M. Quasem

ENGEO Expect Excellence

Incremental Consolidation
ASTM D2435 - Method B



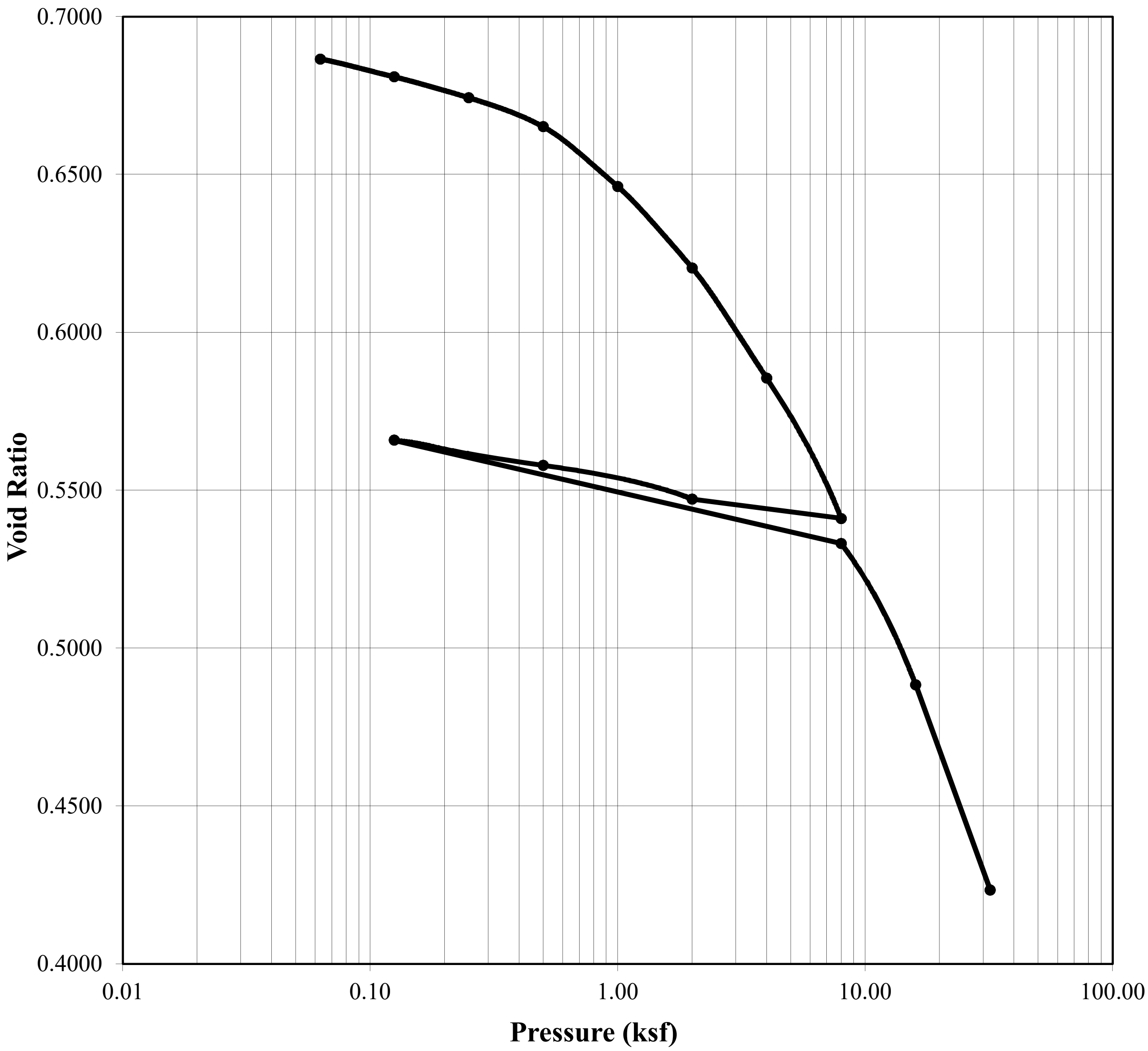
	Before	After	ASTM D4318 - Wet Method		Test Date: 11/29/18
Moisture (%):	24.45	15.50	**Liquid Limit:**	n/a	
Dry Density (pcf):	101.31	118.70	**Plastic Limit:**	n/a	
Saturation (%):	99.69	100.00	ASTM D854 - Measured		
Void Ratio:	0.6624	0.4184	**Specific Gravity:**	2.696	
Soil Description:	See exploration logs		**Remarks:**		

Project Number: 15540.000.000
Sample Number: 1-B1@121-121.5
Project Name: Almaden Office
Client: Boston Properties, Inc.
Location: San Jose, California
Tested By: G. Criste **Checked By:** K. Lecce

Depth: 121-121.5 feet
Boring #: 1-B1

ENGEO Expect Excellence

Incremental Consolidation
ASTM D2435 - Method B



	Before	After	ASTM D4318 - Wet Method		Test Date:	11/30/18
Moisture (%):	26.29	15.92	Liquid Limit:	n/a		
Dry Density (pcf):	98.66	117.00	Plastic Limit:	n/a		
Saturation (%):	100.00	100.01	ASTM D854 - Measured			
Void Ratio:	0.6881	0.4243	Specific Gravity:	2.671		
Soil Description:	See exploration logs		Remarks:			

Project Number: 15540.000.000
Sample Number: 1-B1@48-48.5
Project Name: Almaden Office
Client: Boston Properties, Inc.
Location: San Jose, California
Tested By: G. Criste **Checked By:** K. Lecce

Depth: 48.0-48.5 fee
Boring #: 1-B1

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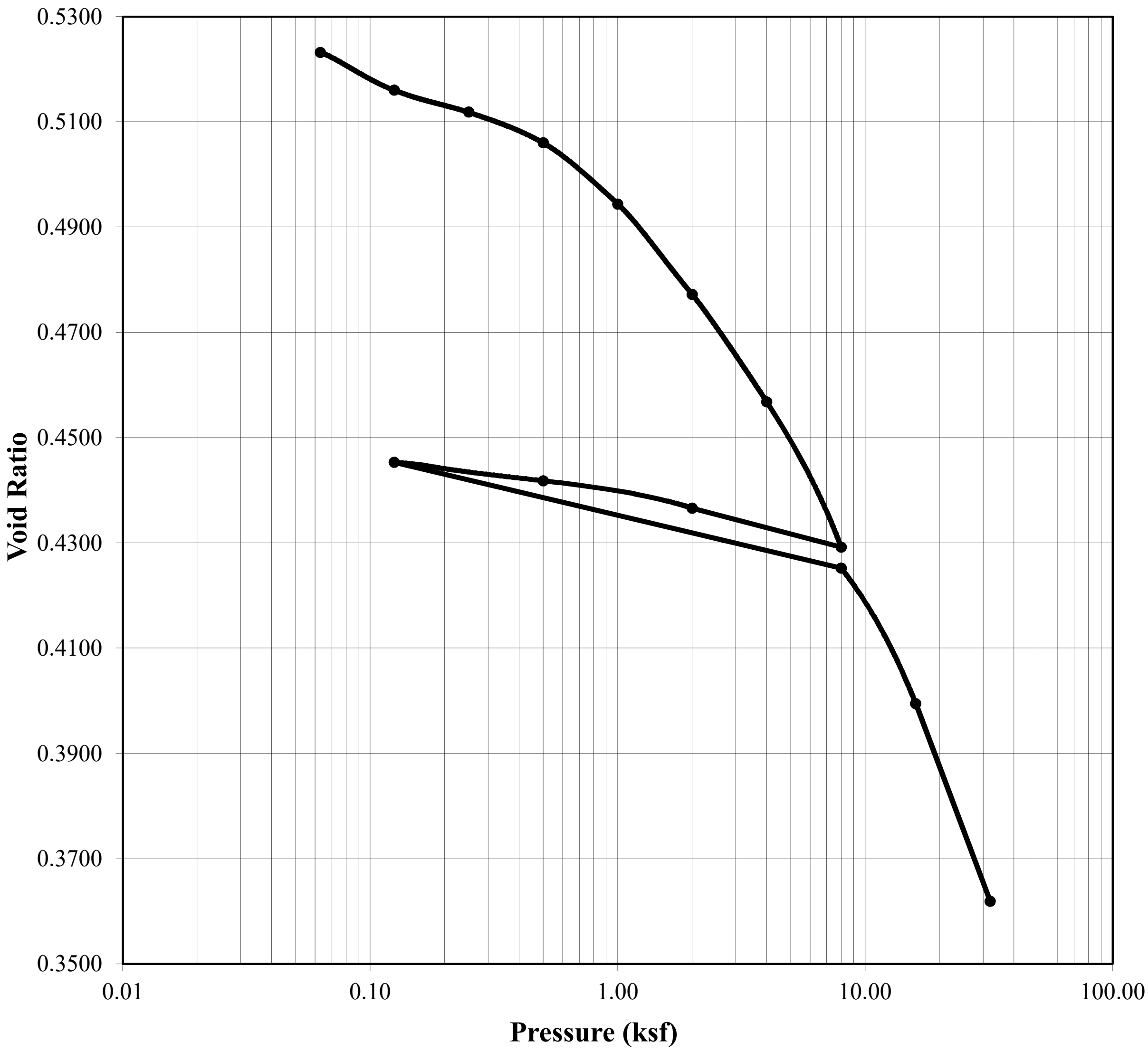
Incremental Consolidation
ASTM D2435 - Method B



	Before	After	ASTM D4318 - Wet Method		Test Date:	11/30/18
Moisture (%):	18.95	13.46	Liquid Limit:	n/a		
Dry Density (pcf):	110.62	123.94	Plastic Limit:	n/a		
Saturation (%):	97.08	100.02	ASTM D854 - Measured			
Void Ratio:	0.5283	0.3639	Specific Gravity:	2.709		
Soil Description:	See exploration logs		Remarks:			

Project Number:	15540.000.000	**Depth:**	51.0 feet
Sample Number:	1-B1@51	**Boring #:**	1-B1
Project Name:	Almaden Office		
Client:	Boston Properties, Inc.		
Location:	San Jose, California		
Tested By: G. Criste	**Checked By:** K. Lecce		

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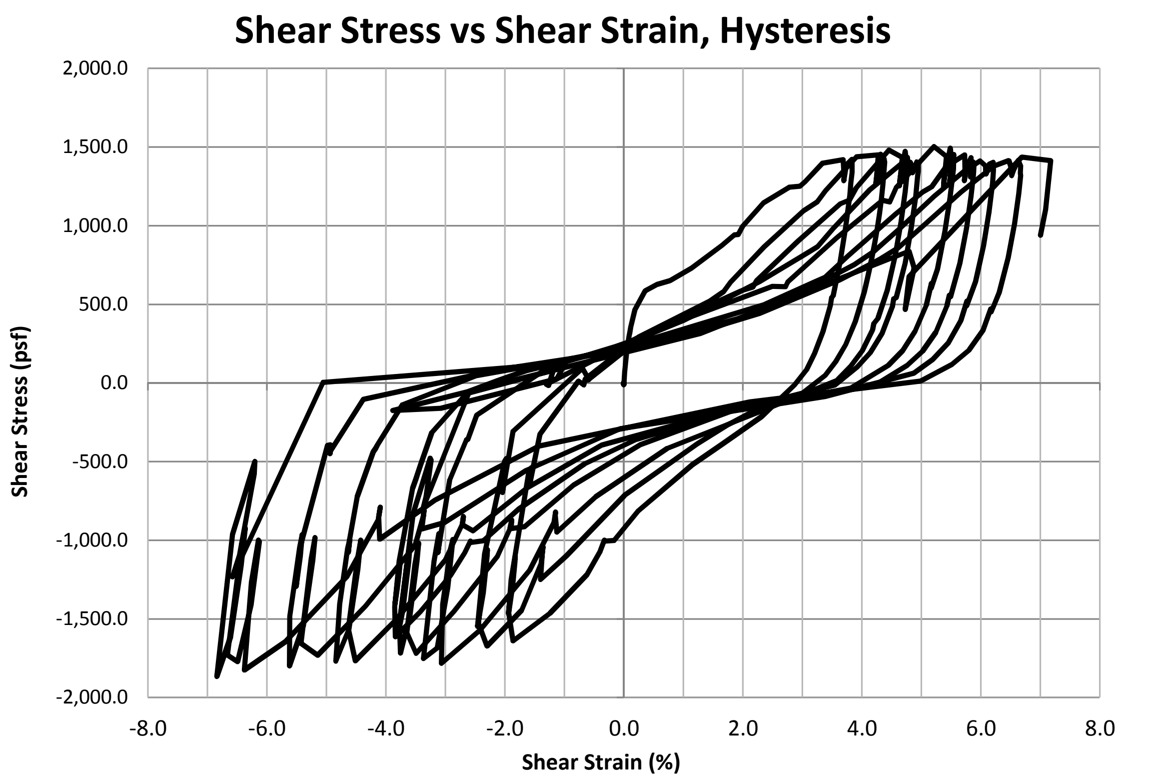
Cyclic Simple Shear

ASTM D6528 - Modified



Test Conditions		Results		Maximum	Minimum
Confining Condition	Plain Membrane w/Chamber Pressure	Vertical Effective Stress (psi)	Deviatoric	9.1	-3.7
Applied Lateral Effective Stress (psi)	18.1		Isotropic	18.1	18.1
Resulting K_o after applied, σ_{vc}'	0.665	Shear Strain (%)		7.8	-6.8
Applied Loading Frequency (Hz)	0.500	Post Cyclic Vertical Strain (%)		0.01	
Cycle Limit, N	9				
Single Amp. Strain (+/-)	7.3	Cyclic Preshear Specimen Condition			
Applied τ_{cyc} (psf)	1503	Preshear Moisture Content (%)		21.62%	
Applied σ_{vc}' (psf)	3928	Preshear Void Ratio		0.5759	
Applied CSR	0.383	Preshear Saturation (%)		100.28%	
		Preshear Dry Density (pcf)		105.70	

ASTM D2974 - Method A (OD Mass) **Test Date:** 12/5/2018

	Initial	Final
Moisture (%):	24.43%	19.92%
Dry Density (pcf):	102.97	105.92
Saturation (%):	100.00%	100.00%
Void Ratio:	0.5870	0.5715

ASTM D4318 - Wet Method

Liquid Limit:	
Plastic Limit:	

ASTM D854 - Measured

Specific Gravity:	2.671
Soil Description:	See exploration logs

Project Number: 15540.000.000
Sample Number: 1-B1 @ 41-42.5
Project Name: Almaden Office Complex
Client: Boston Properties, Inc.
Location: San Jose, California
Tested By: D. Seibold

Depth: 42-42.5 ft
Boring #: 1-B1

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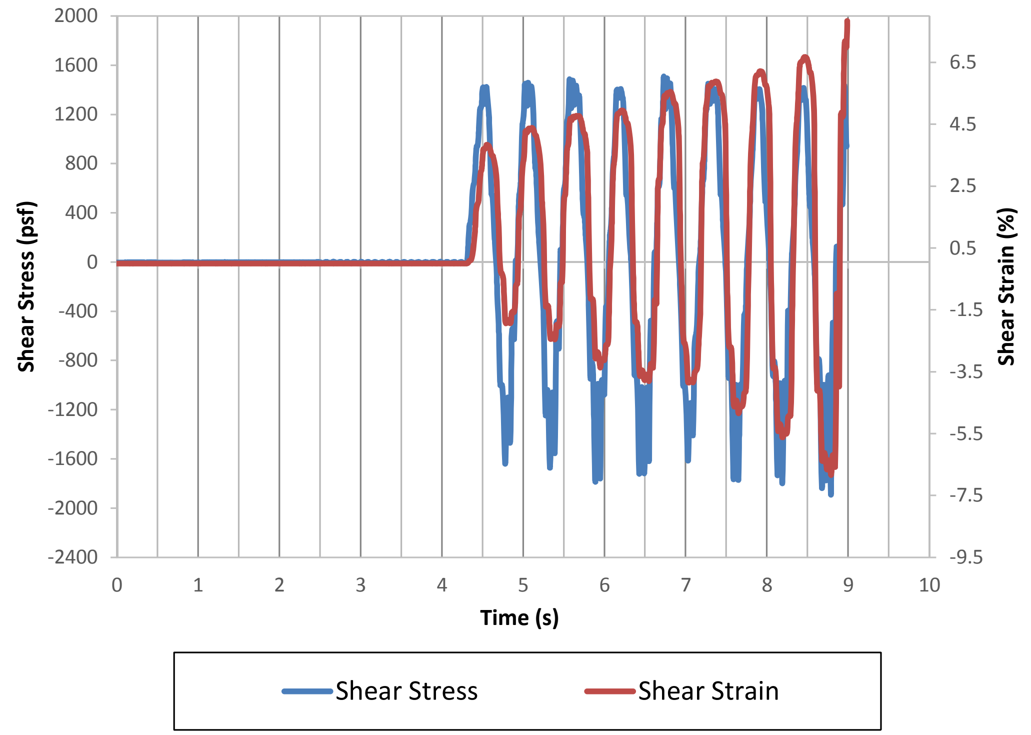
Reviewed By: I. McCreery

Remarks: The test specimen contained 45.6% passing the #200 sieve

Cyclic Simple Shear

ASTM D6528 - Modified

Shear Stress & Shear Strain vs Time



Test Conditions		Test Results		Maximum	Minimum
Confining Condition	Plain Membrane w/Chamber Pressure	Vertical Effective Stress (psi)	Deviatoric	9.1	-3.7
Applied Lateral Effective Stress (psi)	18.1		Isotropic	18.1	18.1
Resulting K_o after applied, σ_{vc}'	0.665	Shear Strain (%)		7.8	-6.8
Applied Loading Frequency (Hz)	0.500	Post Cyclic Vertical Strain (%)		0.01	
Cycle Limit, N	9				
Single Amp. Strain (+/-)	7.3	**Cyclic Preshear Specimen Condition**			
Applied τ_{cyc} (psf)	1503	Preshear Moisture Content (%)		21.62%	
Applied σ_{vc}' (psf)	3928	Preshear Void Ratio		0.5759	
Applied CSR	0.383	Preshear Saturation (%)		100.28%	
		Preshear Dry Density (pcf)		105.70	

ASTM D2974 - 2974 Method A (OD mass) — **Test Date:** 12/5/2018

	Initial	Final
Moisture (%):	24.43%	19.92%
Dry Density (pcf):	102.97	105.92
Saturation (%):	100.00%	100.00%
Void Ratio:	0.5870	0.5715

ASTM D4318 - Wet Method

Liquid Limit:	
Plastic Limit:	

ASTM D854 - Measured

Specific Gravity:	2.671
Soil Description:	See exploration logs

Project Number:	15540.000.000	**Depth:**	42-42.5 ft
Sample Number:	1-B1 @ 41-42.5	**Boring #:**	1-B1
Project Name:	Almaden Office Complex		
Client:	Boston Properties, Inc.		
Location:	San Jose, California		
Tested By:	D. Seibold	**Reviewed By:**	I. McCreery

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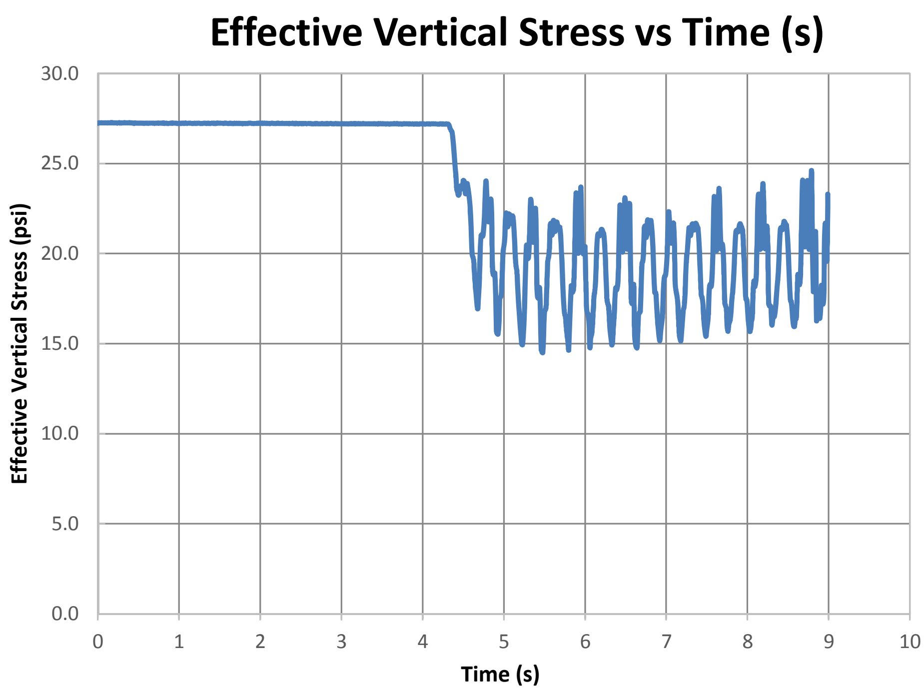
Remarks:

Cyclic Simple Shear
ASTM D6528 - Modified



Test Conditions			Results		Maximum	Minimum
Confining Condition	Plain Membrane w/Chamber Pressure		Vertical Effective Stress, (psi)	Deviatoric	9.1	-3.7
Applied Lateral Effective Stress, (psi)	18.1			Isotropic	18.1	18.1
Resulting K_o after applied σ_{vc}'	0.665		Shear Strain, (%)		7.8	-6.8
Applied Loading Frequency, (Hz)	0.500		Post Cyclic Vertical Strain, (%)		0.01	
Cycle Limit, N	9					
Single Amp. Strain, (+/-)	7.3		Cyclic Preshear Specimen Condition			
Applied τ_{cyc}, (psf)	1503		Preshear Moisture Content (%)		21.62%	
Applied σ_{vc}', (psf)	3928		Preshear Void Ratio		0.5759	
Applied CSR	0.383		Preshear Saturation (%)		100.28%	
			Preshear Dry Density (pcf)		105.70	

ASTM D2974 - 2974 Method A (OD mass) — **Test Date:** 12/5/2018

	Initial	Final
Moisture (%):	24.43%	19.92%
Dry Density (pcf):	102.97	105.92
Saturation (%):	100.00%	100.00%
Void Ratio:	0.5870	0.5715

ASTM D4318 - Wet Method

Liquid Limit:	
Plastic Limit:	

ASTM D854 - Measured

Specific Gravity:	2.671
Soil Description:	See exploration logs

Project Number:	15540.000.000	**Depth:**	42-42.5 ft
Sample Number:	1-B1 @ 41-42.5	**Boring #:**	1-B1
Project Name:	Almaden Office Complex		
Client:	Boston Properties, Inc.		
Location:	San Jose, California		
Tested By:	D. Seibold	**Reviewed By:**	I. McCreery
Remarks:			

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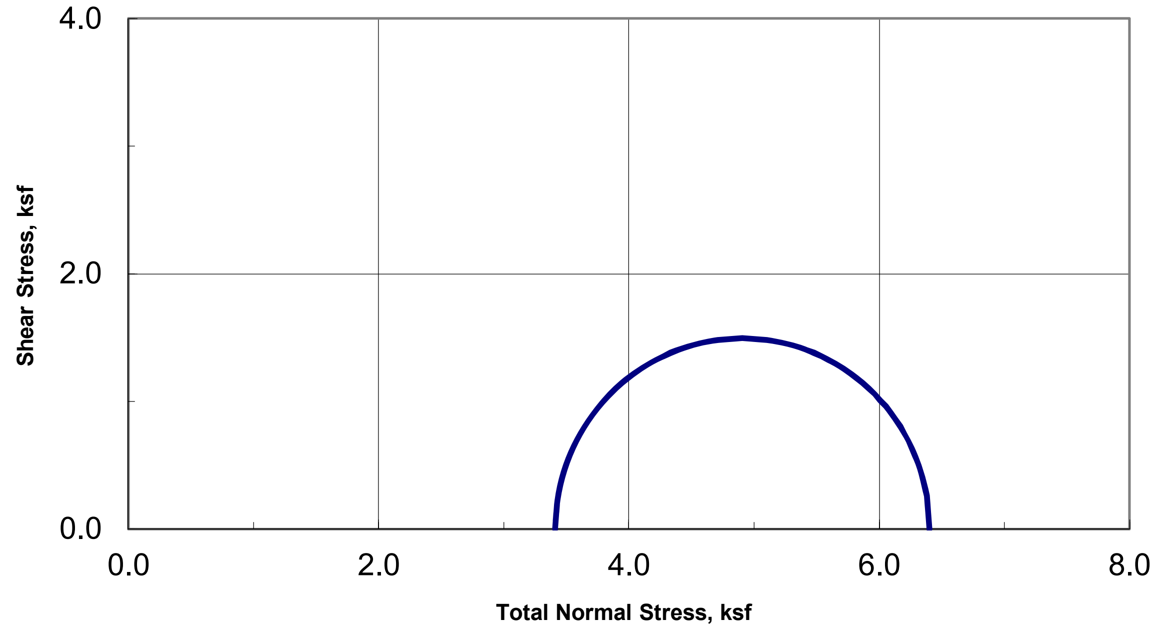
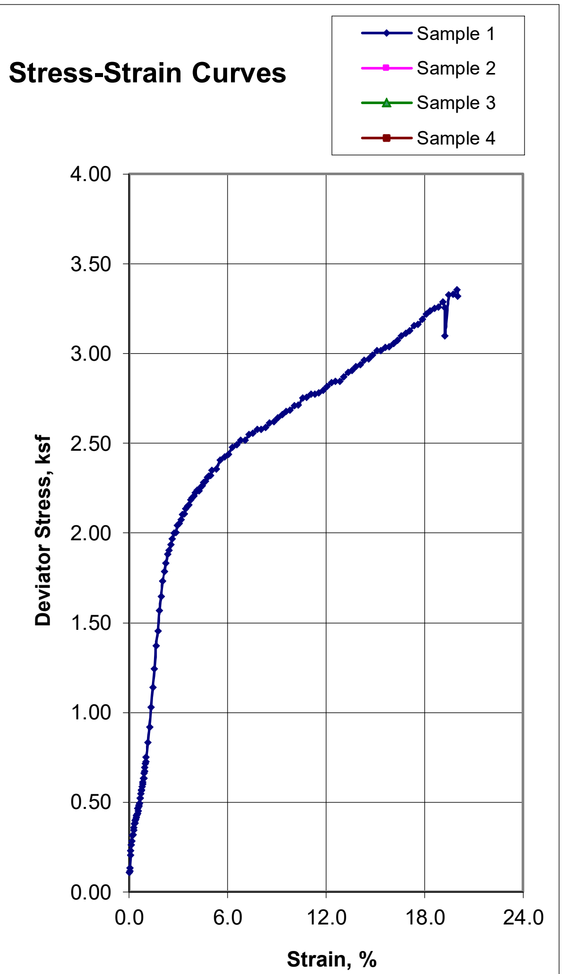
Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	24.5			
Dry Den,pcf	99.9			
Void Ratio	0.688			
Saturation %	96.2			
Height in	5.18			
Diameter in	2.40			
Cell psi	23.7			
Strain %	15.00			
Deviator, ksf	2.990			
Rate %/min	1.00			
in/min	0.052			
Job No.:	**414-115**			
Client:	**ENGEO Incorporated**			
Project:	**15540.000.000 P:001**			
Boring:	1-B1			
Sample:				
Depth ft:	38-39.5			

Visual Soil Description	
Sample #	
1	Gray Sandy CLAY
2	
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

APPENDIX C

CONE PENETRATION TEST REPORT
(California Push Technologies, Inc.)

PRESENTATION OF SITE INVESTIGATION RESULTS

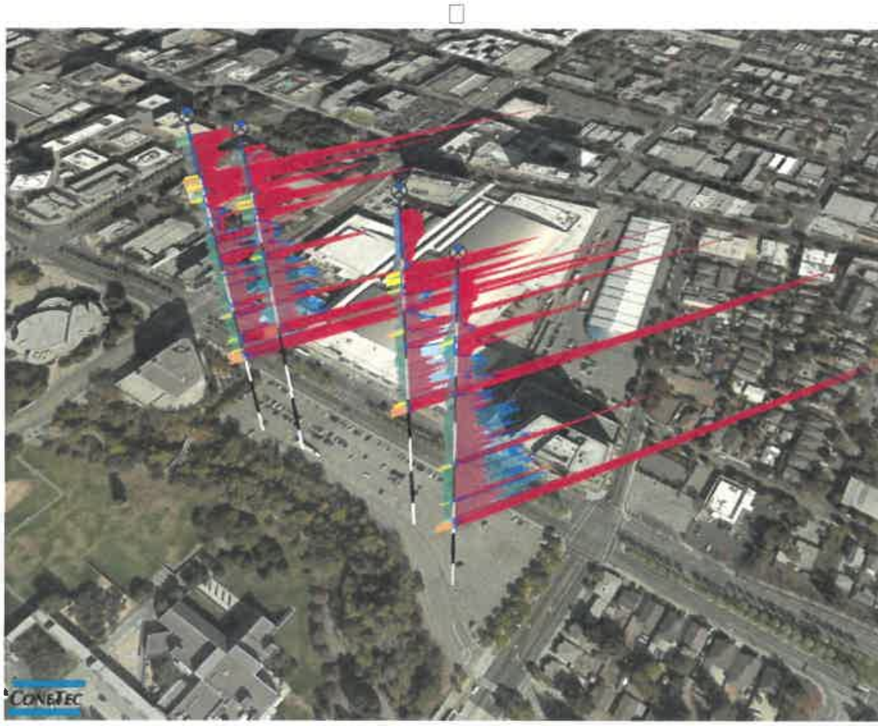
Almaden Office Complex

Prepared for:

ENGEO Inc.

CPT Inc. Job No: 18-56175

Project Start Date: 22-Oct-2018
Project End Date: 23-Oct-2018
Report Date: 24-Oct-2018



Prepared by:

California Push Technologies Inc.
820 Aladdin Avenue
San Leandro, CA 94577

Tel: (510) 357-3677

Email: cpt@cptinc.com
www.cptinc.com

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED   CONETEC

Introduction

The enclosed report presents the results of the site investigation program conducted by CPT Inc. for ENGEO Inc. at South Almaden Blvd. and Woz Way, San Jose, CA. The program consisted of one cone penetration test (CPT), three seismic cone penetration tests (SCPT), and one Geokon piezometer installation.

Project Information

Project	
Client	ENGEO Inc.
Project	Almaden Office Complex
CPT Inc. project number	18-56175

A map from Google Earth including the CPT and SCPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30 ton rig cylinder	CPT, SCPT, installation

Coordinates		
Test Type	Collection Method	EPSG Reference
CPT, SCPT, installation	Consumer Grade GPS	32610

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Standard plots, standard plots with expanded scales, advanced plots, soil behavior type (SBT) scatter plots, and seismic plots have been included in the data release package.

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)
448:T1500F15U500	448	15	225	1500	15	500
483:T1500F15U500	483	15	225	1500	15	500
The CPT summary indicates which cone was used for each sounding.						

CPT Calculated Parameters	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Qtn) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT 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 behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behavior Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Geokon Piezometer Installation	
Depth reference	Depths are referenced to the existing surface at the time of each installation.
Additional information	Geokon piezometer calibration records are provided in the data release folder.

Limitations

This report has been prepared for the exclusive use of ENGEO Inc. (Client) for the project titled "Almaden Office Complex". The report's contents may not be relied upon by any other party without the express written permission of CPT Inc. CPT Inc. 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 CPT Inc. 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 both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

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

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

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

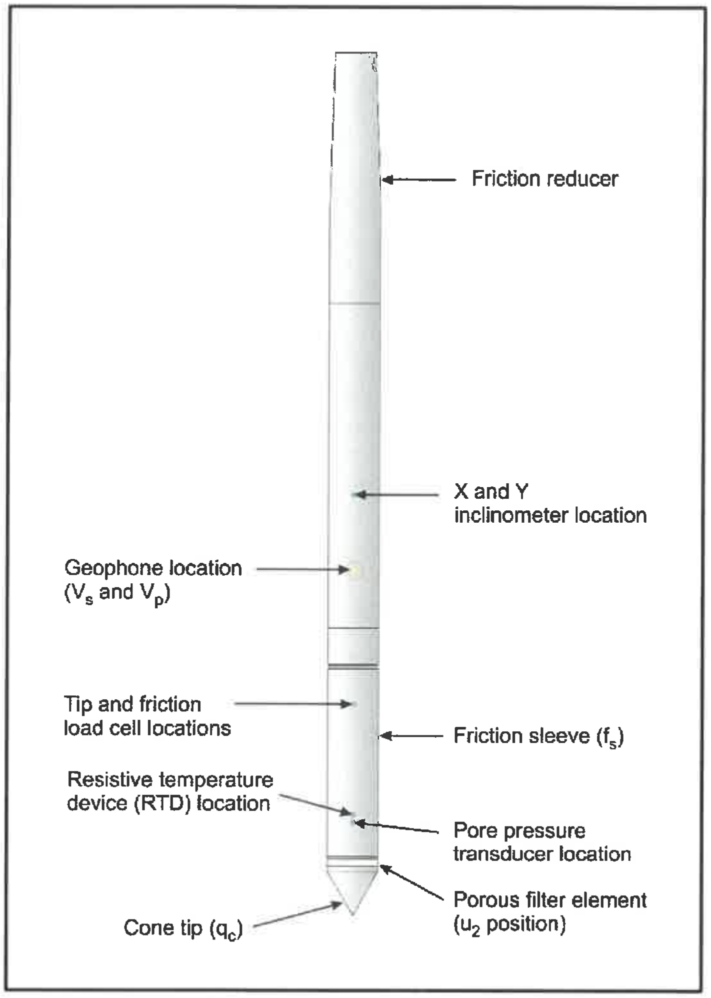


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods.

The typical recording interval is 2.5 cm; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

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

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

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

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

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

- Each filter is saturated in silicone oil 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 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 maybe 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 up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

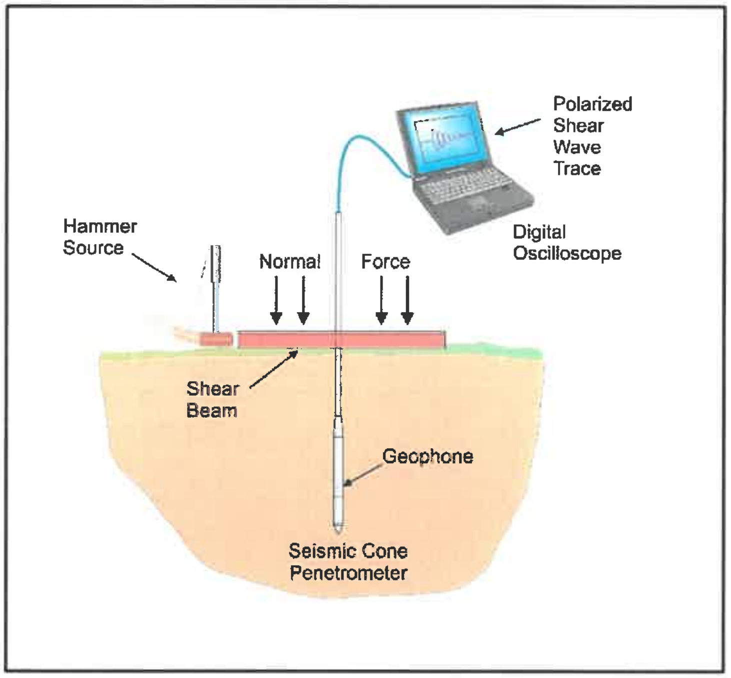


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM 5778 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 purposes and uncertainty analysis. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

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

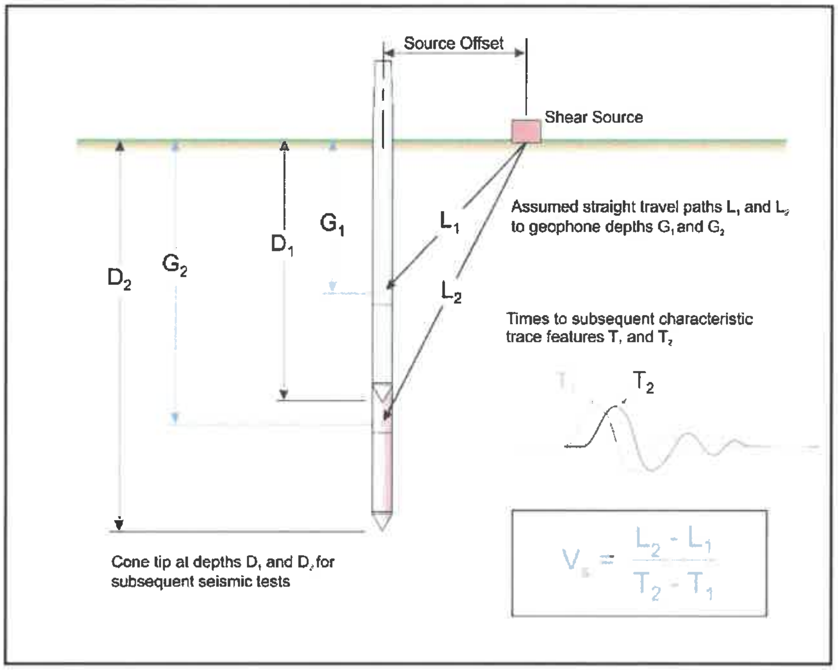


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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

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

$\sum_{i=1}^{n} d_i$ = 100 ft (30 m)

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

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

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

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

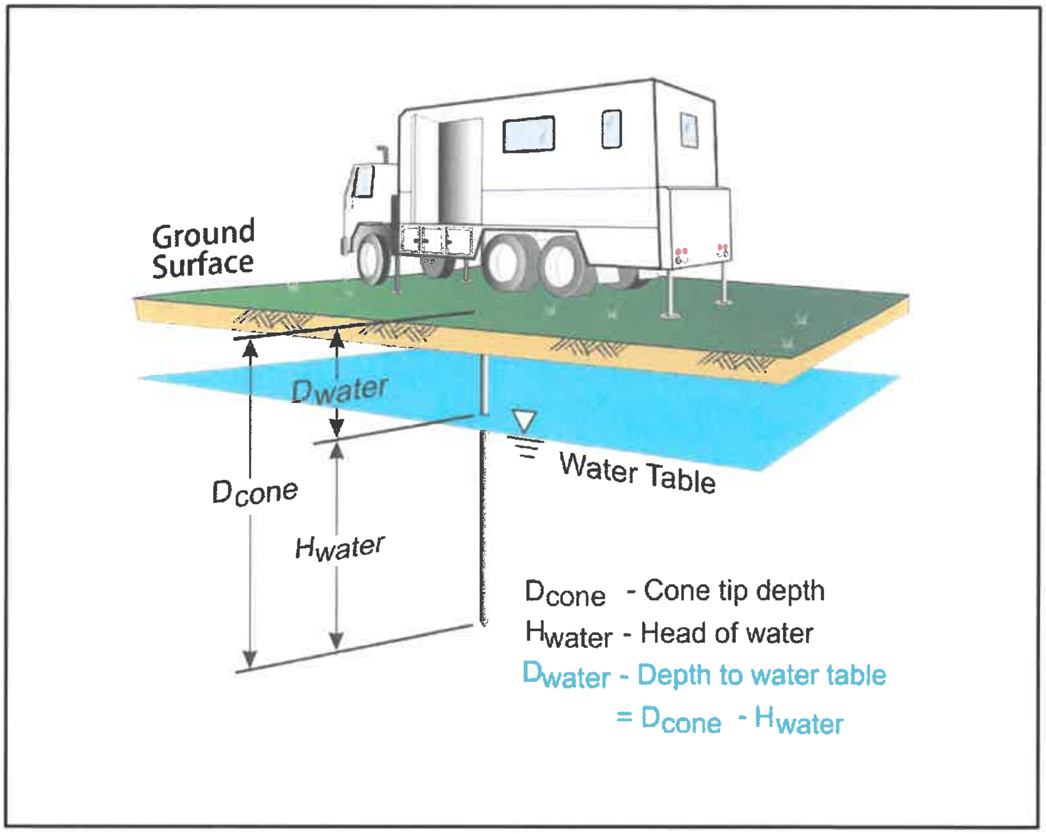


Figure PPD-1. Pore pressure dissipation test setup

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

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

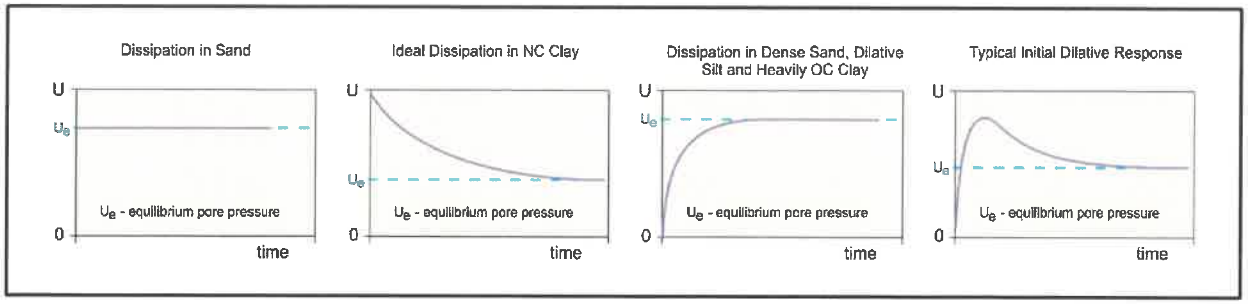


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve 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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ASTM D7400-14, 2014, "Standard Test Methods for Downhole Seismic Testing", ASTM, West Conshohocken, US.

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Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

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

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

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

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

Vibrating wire piezometers manufactured by Geokon, Inc., measure in situ water pressure and temperature. The pressure is determined by measuring the resonant frequency at which the internal tensioned wire vibrates. Calibration constants relate the recorded frequency to the applied pressure. Temperature is measured using a built-in thermistor.

Prior to deployment the piezometers are saturated as per the manufacturer's guidelines and the piezometer serial number and baselines are recorded.

The piezometers are pushed into the ground from ground surface with a CPT rig or drill rig and the installation depths are referenced to the existing ground surface at the time of installation.

An installation summary is provided in the relevant appendix.

For more details about Geokon vibrating wire piezometers, refer to the manufacturer's website. http://www.geokon.com/Piezometers

The appendices listed below are included in the report:

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

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

CONETEC

ConeTec

Job No: 18-56175
Client: ENGEO Inc.
Project: Almaden Office Complex
Start Date: 22-Oct-2018
End Date: 22-Oct-2018

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting (m)	Refer to Notation Number
1-SCPT01	18-56175_SP01	22-Oct-2018	448:T1500F15U500	17.4	95.143	4131680	598315	
1-SCPT02	18-56175_SP02	22-Oct-2018	483:T1500F15U500	17.0	70.455	4131728	598310	
1-SCPT03	18-56175_SP03	22-Oct-2018	483:T1500F15U500	17.2	67.995	4131816	598263	
1-CPT04	18-56175_CP04	22-Oct-2018	483:T1500F15U500	19.4	87.351	4131844	598242	

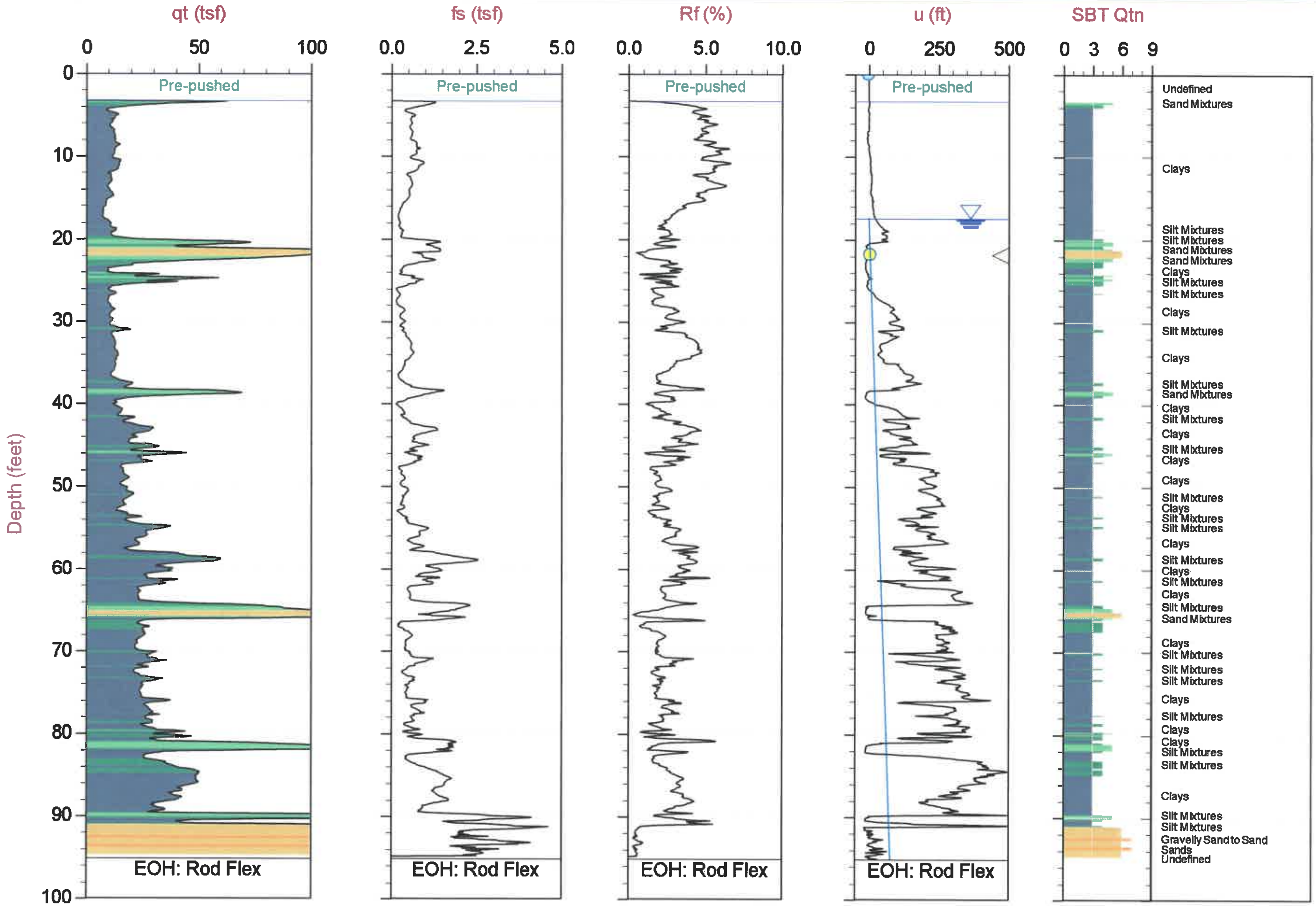
1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with consumer grade GPS device with datum WGS84 / UTM 10.

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 08:17
Site: Almaden Office Complex

Sounding: 1-SCPT01
Cone: 448:T1500F15U500



Max Depth: 29.000 m / 95.14 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131680m E: 598315m
Sheet No: 1 of 1

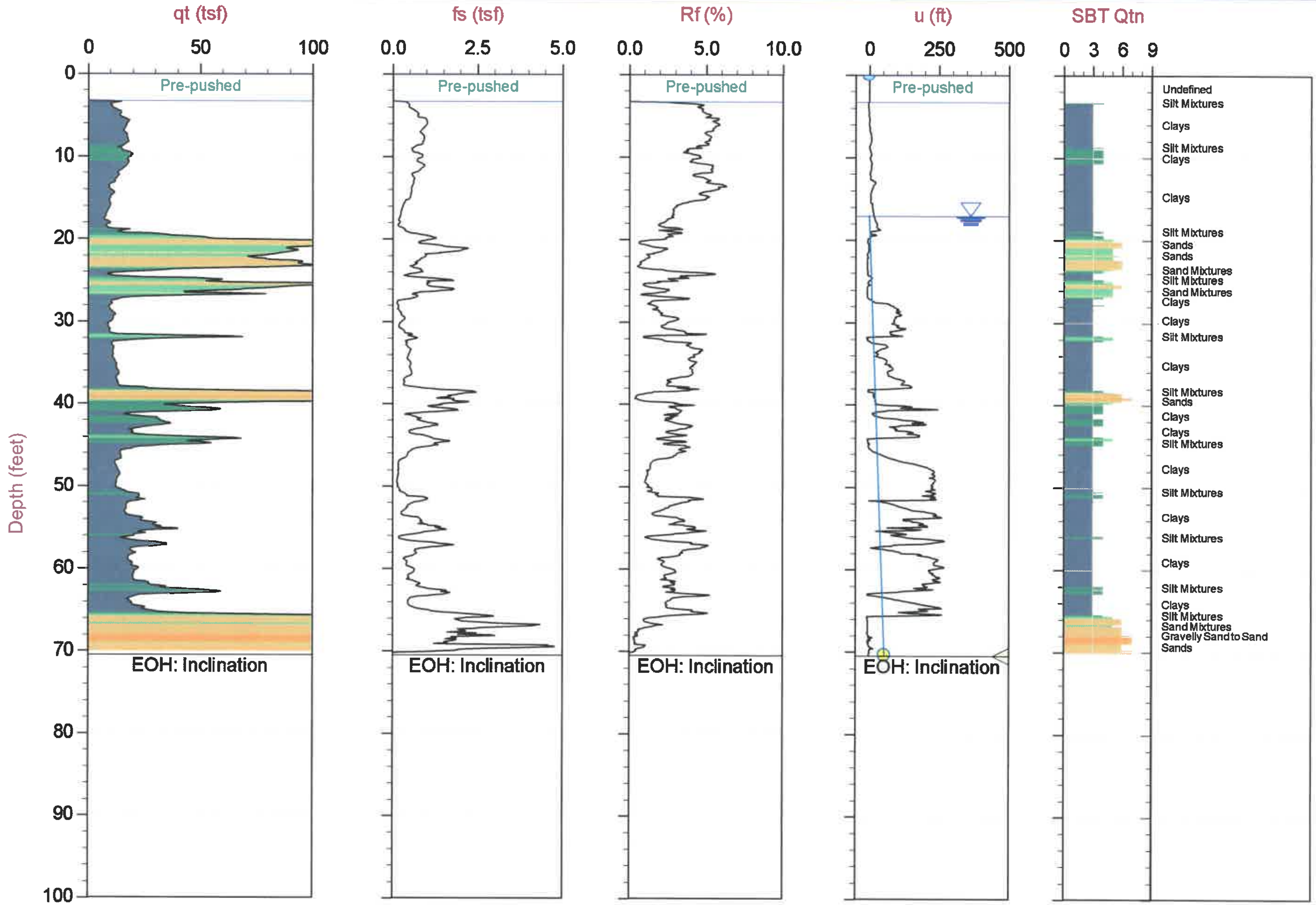
Overplot Item: Ueq   Assumed Ueq   ◁ Dissipation, Ueq achieved   ◁ Dissipation, Ueq not achieved   Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 13:16
Site: Almaden Office Complex

Sounding: 1-SCPT02
Cone: 483:T1500F15U500



Max Depth: 21.475 m / 70.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131728m E: 598310m
Sheet No: 1 of 1

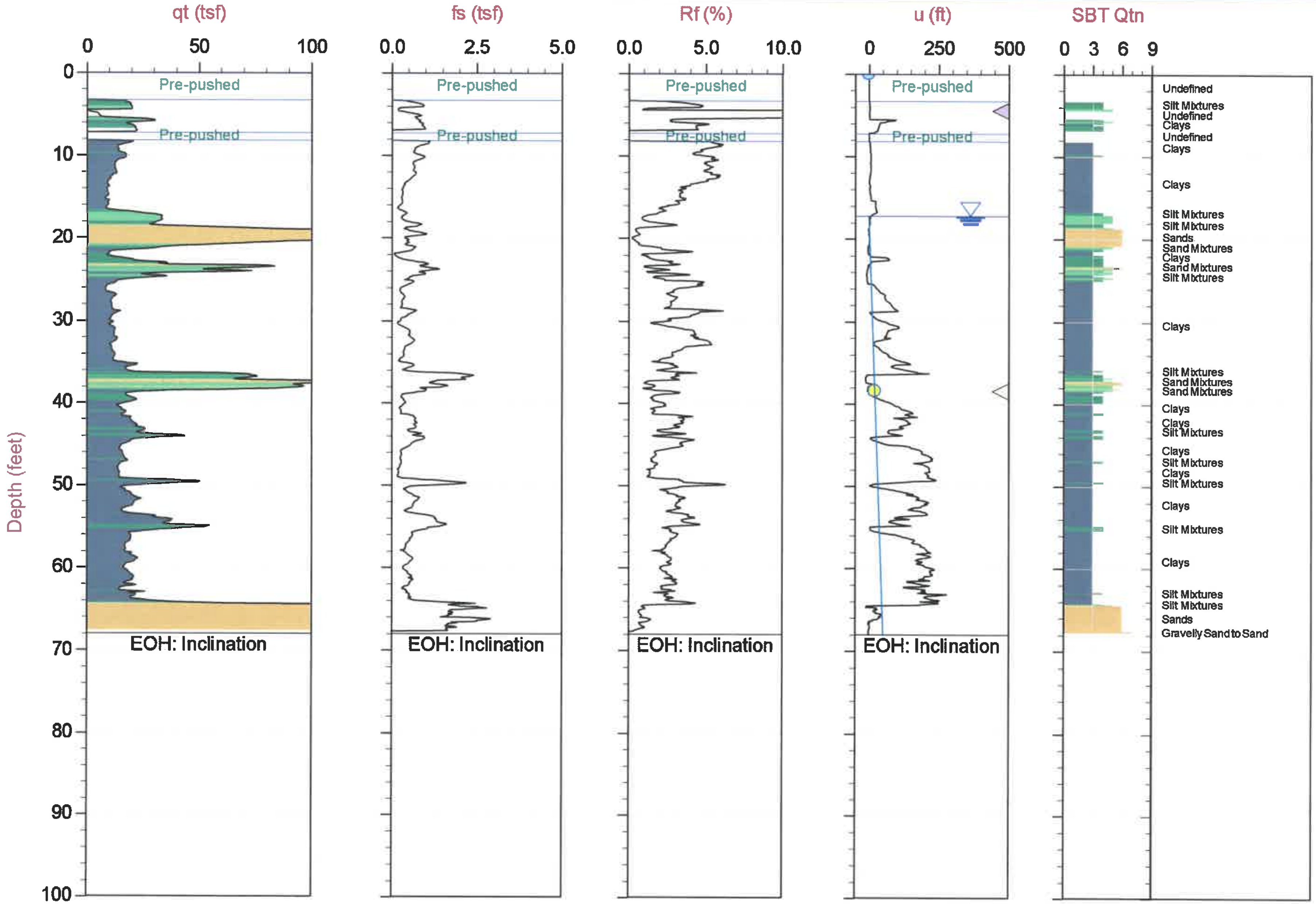
Overplot Item: Ueq Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 10:17
Site: Almaden Office Complex

Sounding: 1-SCPT03
Cone: 483:T1500F15U500



Max Depth: 20.725 m / 67.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131816m E: 598263m
Sheet No: 1 of 1

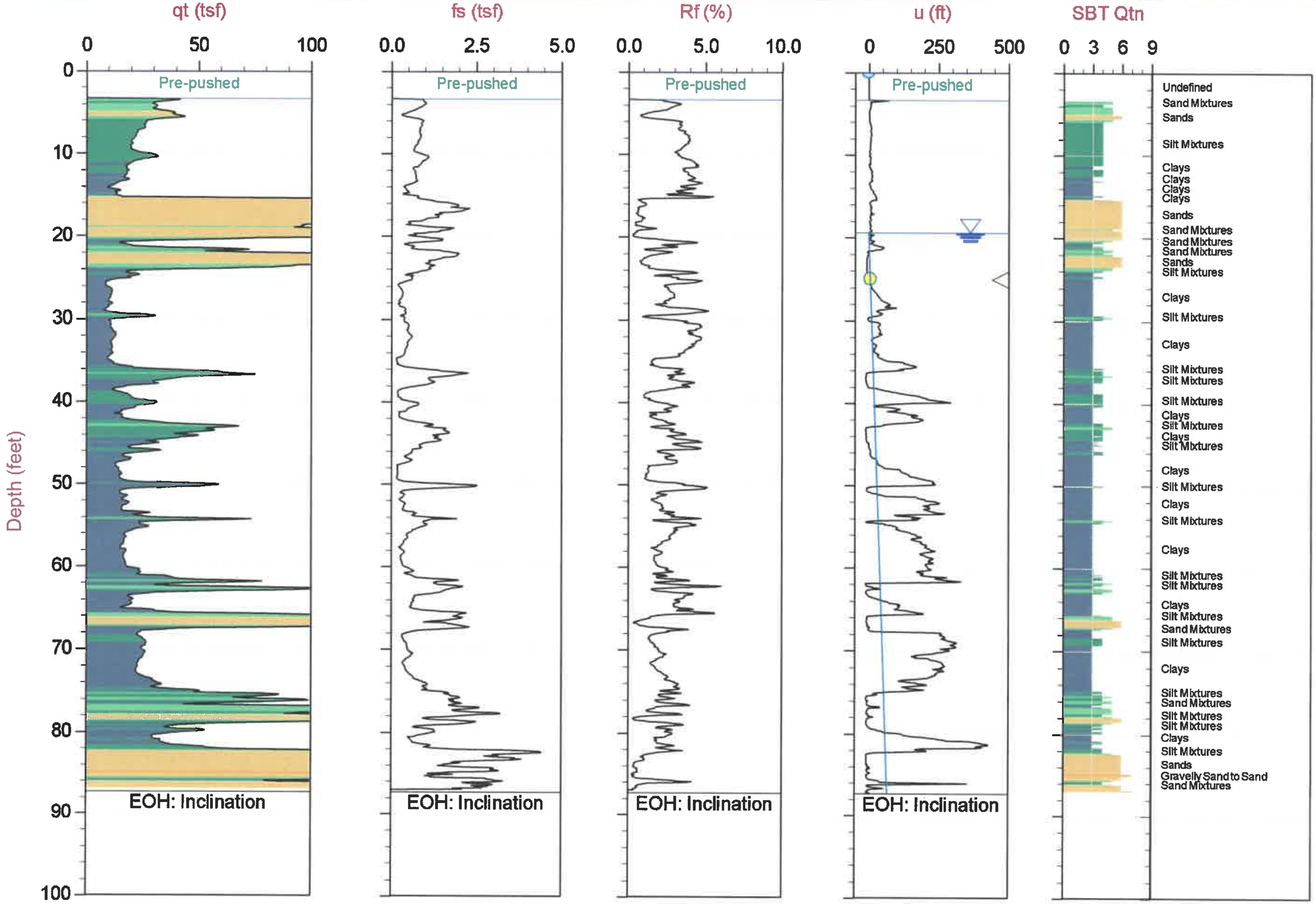
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

ConeTec

ENGEO

Job No: 18-56175
Date: 2018-10-22 12:13
Site: Almaden Office Complex

Sounding: 1-CPT04
Cone: 483:T1500F15U500



Max Depth: 26.625 m / 87.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131844m E: 598242m
Sheet No: 1 of 1

Overplot Item: Ueq · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Hydrostatic Line

Standard Cone Penetration Test Plots with Expanded Scales

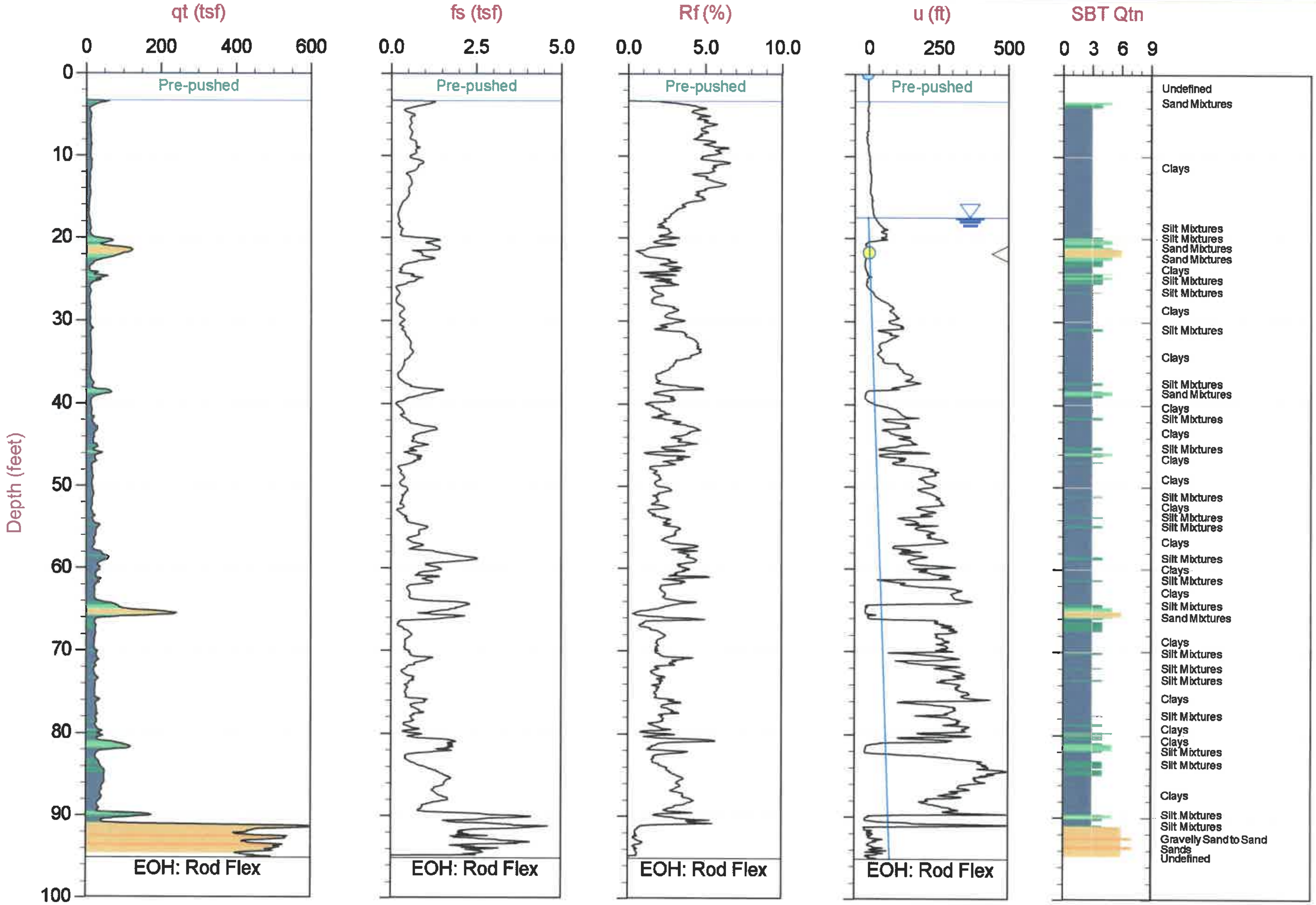
ConeTec

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 08:17
Site: Almaden Office Complex

Sounding: 1-SCPT01
Cone: 448:T1500F15U500

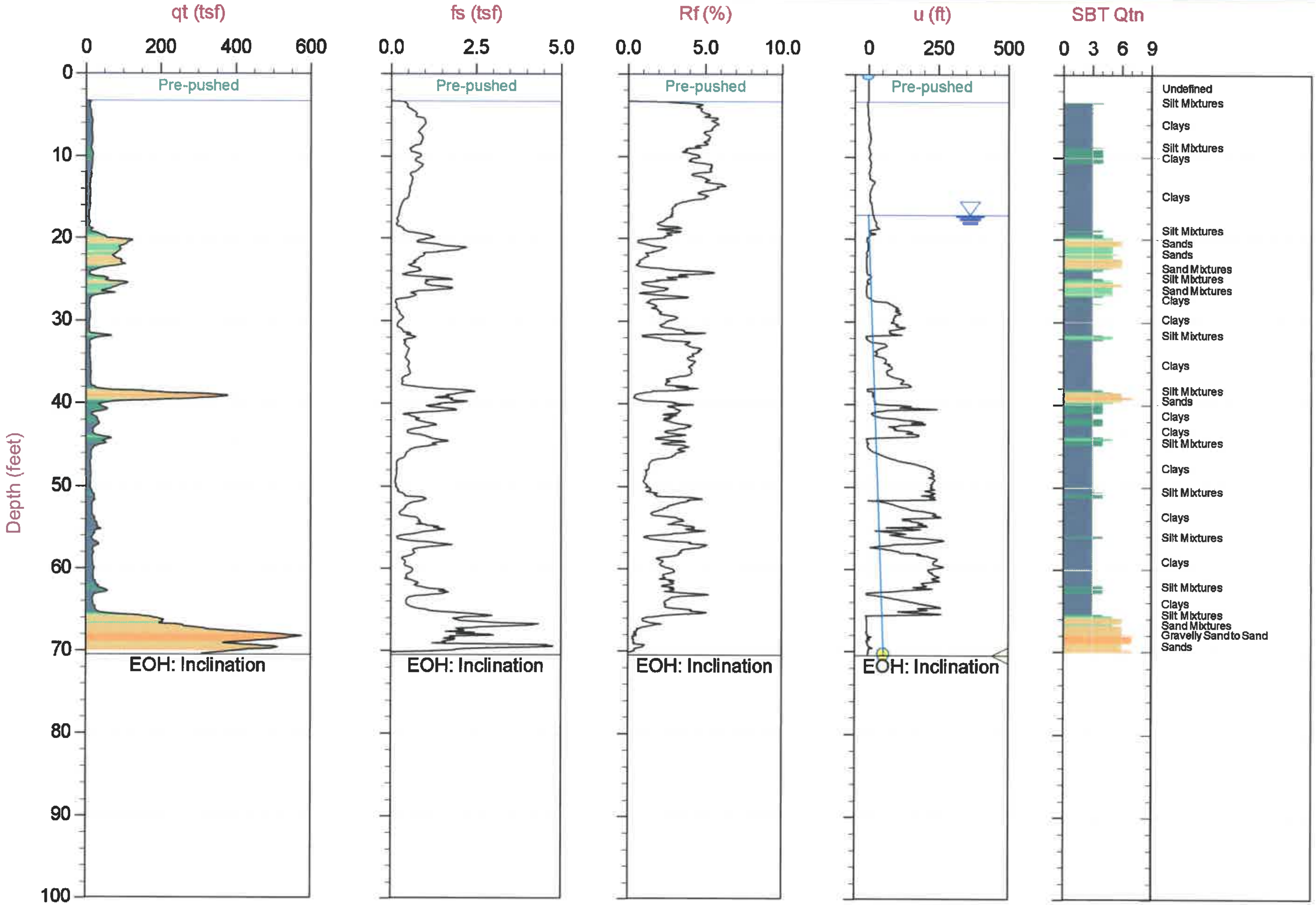


Max Depth: 29.000 m / 95.14 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131680m E: 598315m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line



Max Depth: 21.475 m / 70.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 18-56175_SP02.COR
Unit Wt: SBTQtn (PKR2009)
SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131728m E: 598310m
Sheet No: 1 of 1

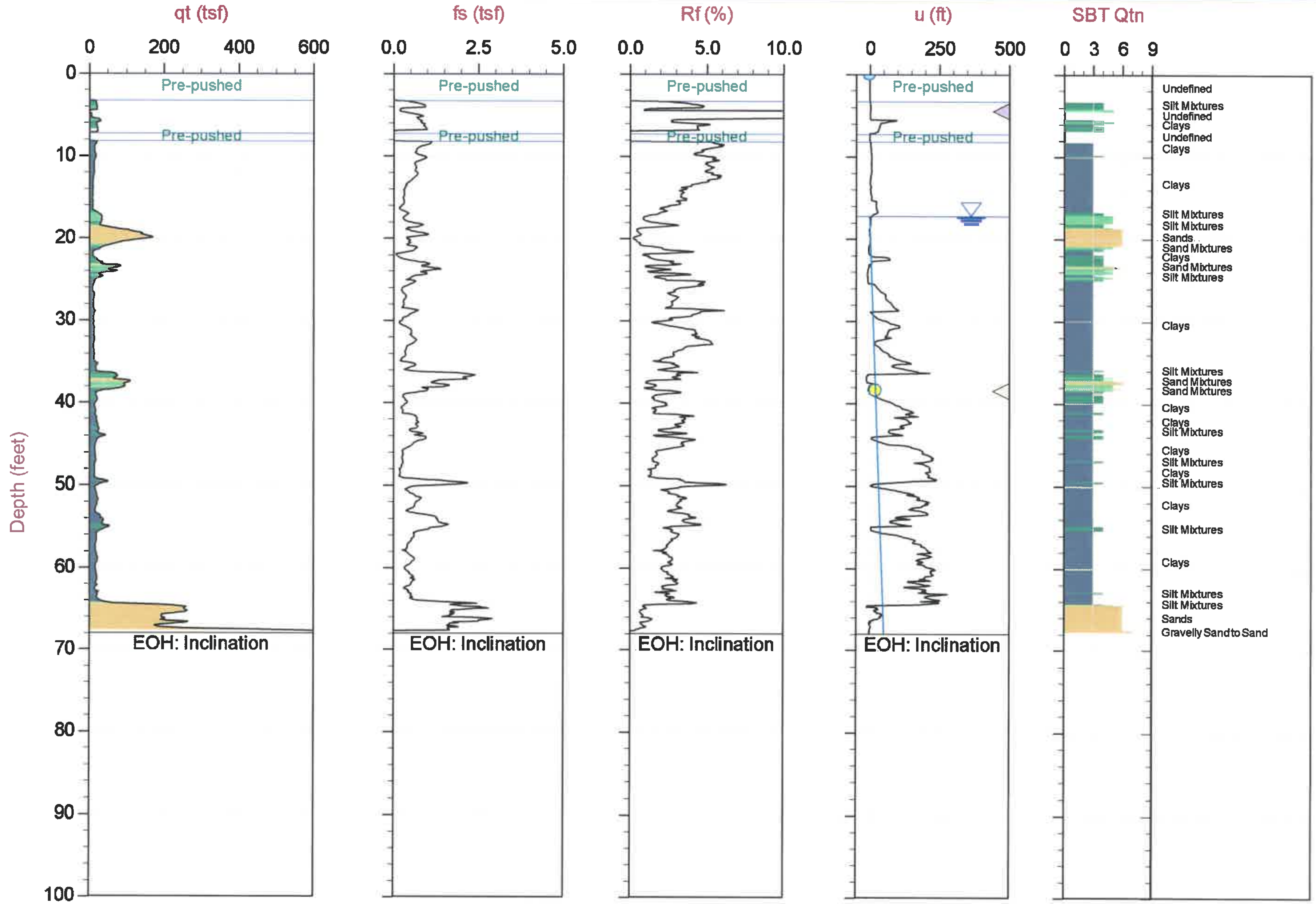
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 10:17
Site: Almaden Office Complex

Sounding: 1-SCPT03
Cone: 483:T1500F15U500



Max Depth: 20.725 m / 67.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131816m E: 598263m
Sheet No: 1 of 1

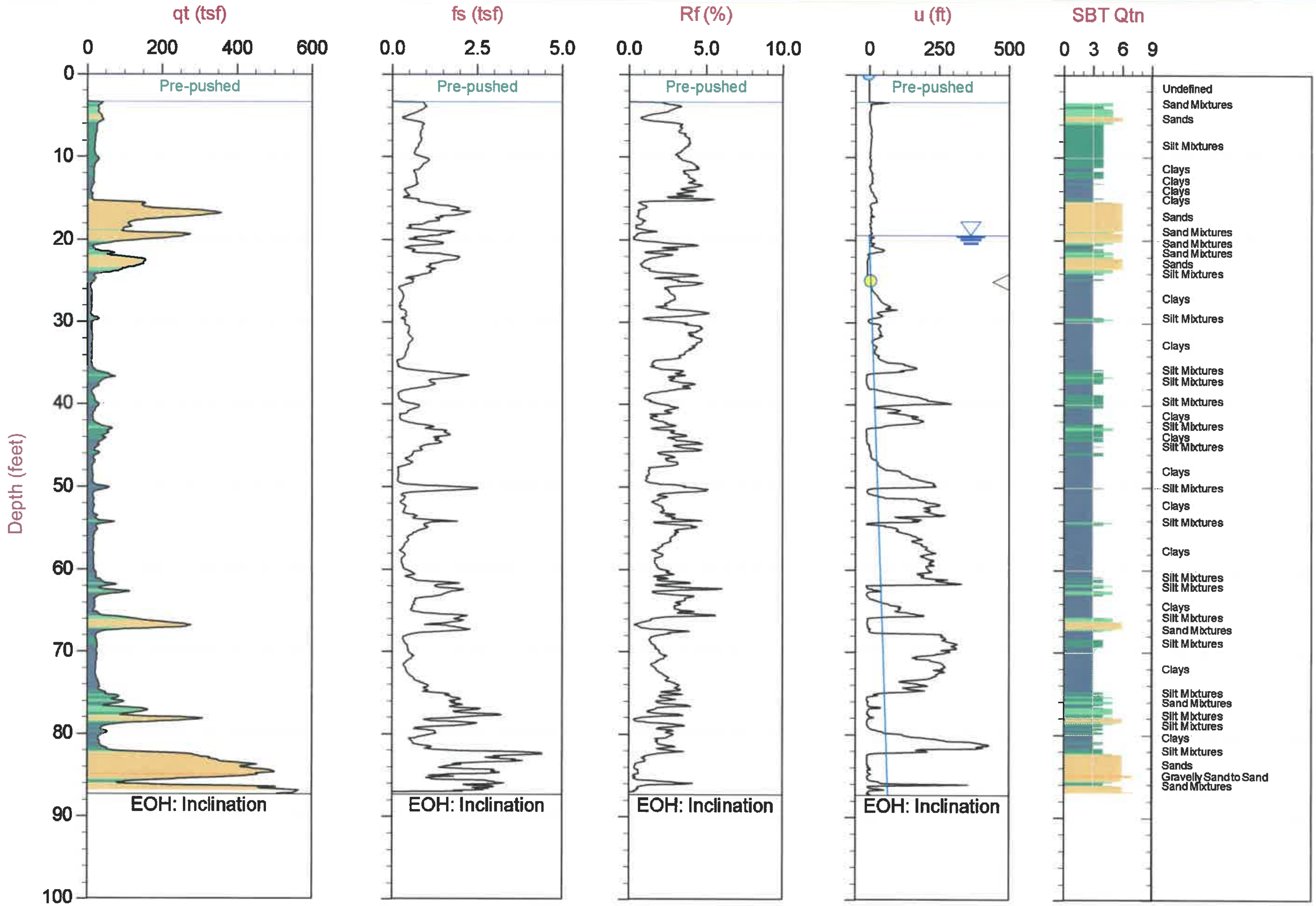
Overplot Item: Ueq Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 12:13
Site: Almaden Office Complex

Sounding: 1-CPT04
Cone: 483:T1500F15U500



Max Depth: 26.625 m / 87.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131844m E: 598242m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

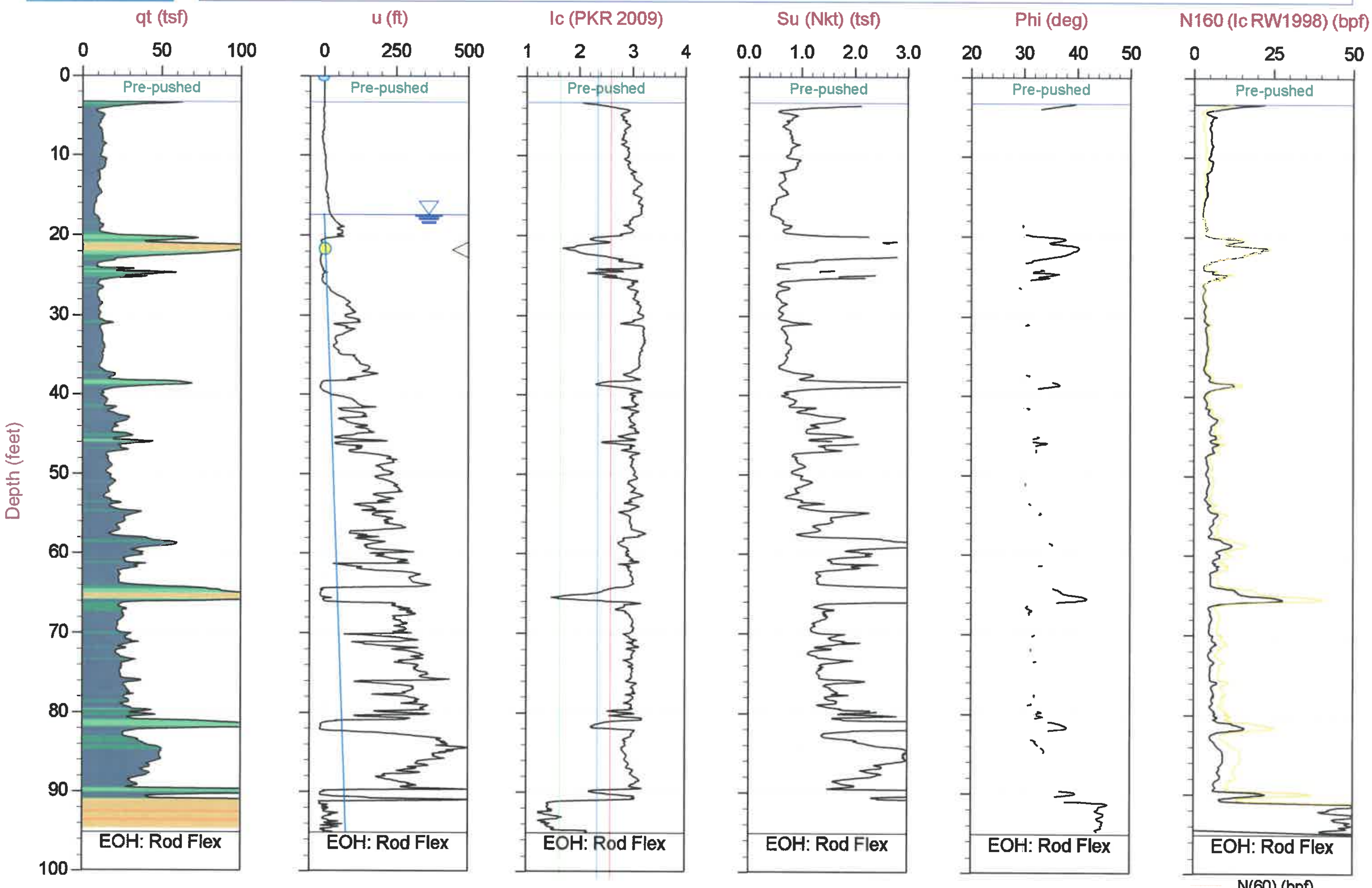
ConeTec

ConeTec

ENGEO

Job No: 18-56175
Date: 2018-10-22 08:17
Site: Almaden Office Complex

Sounding: 1-SCPT01
Cone: 448:T1500F15U500



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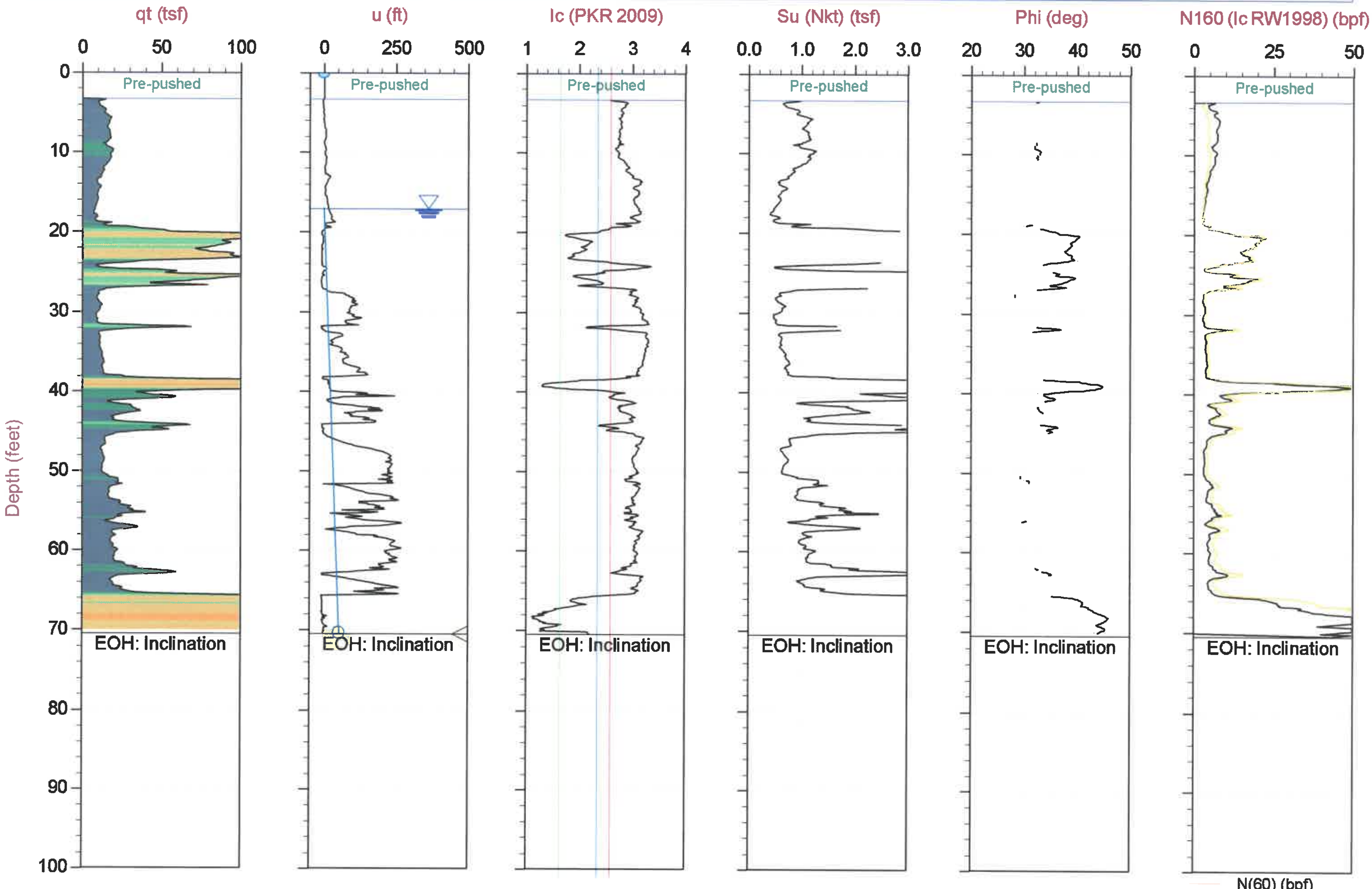
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 18-56175_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0
SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131680m E: 598315m
Sheet No: 1 of 1
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 13:16
Site: Almaden Office Complex

Sounding: 1-SCPT02
Cone: 483:T1500F15U500



Max Depth: 21.475 m / 70.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131728m E: 598310m
Sheet No: 1 of 1

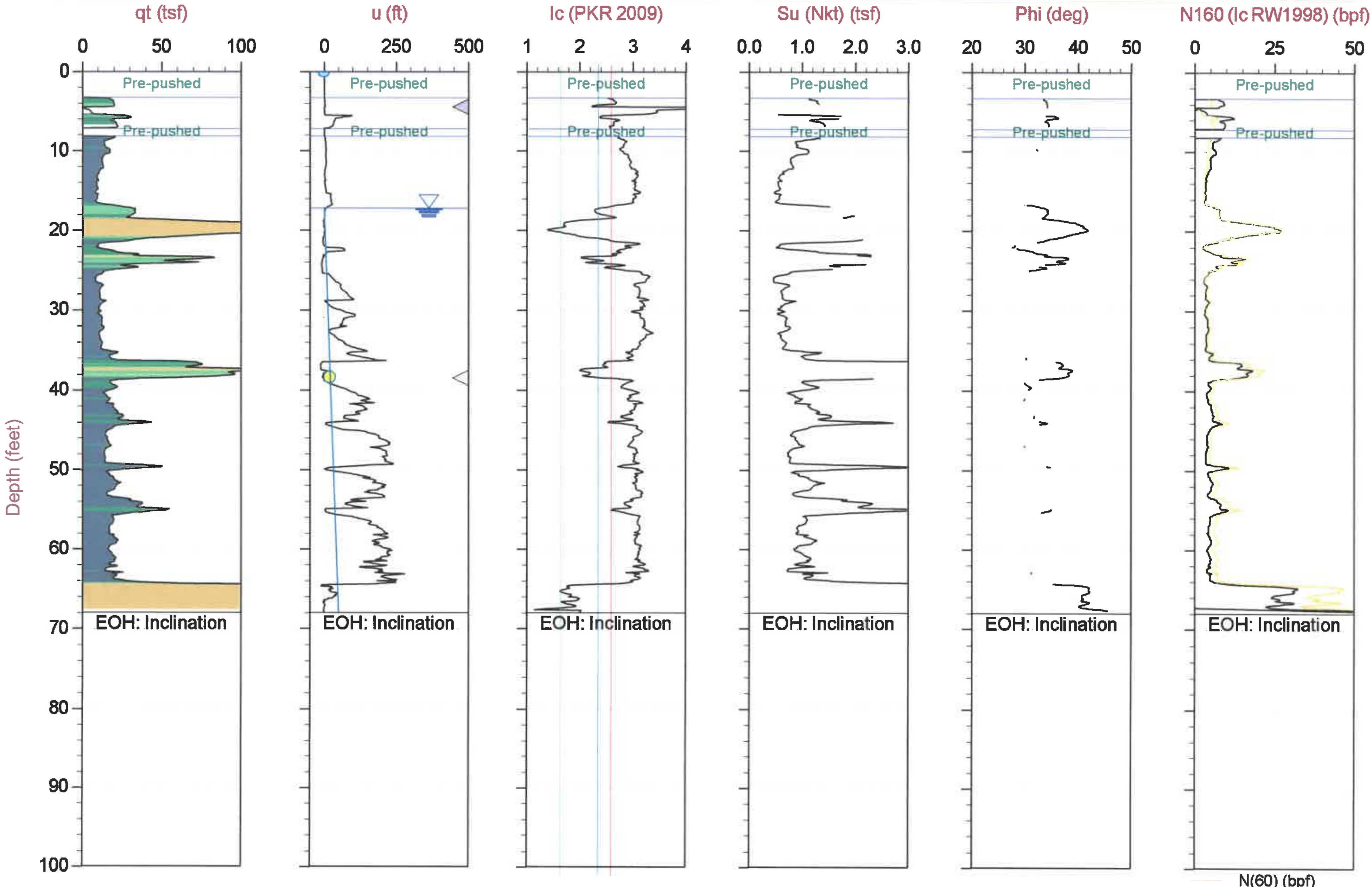
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 10:17
Site: Almaden Office Complex

Sounding: 1-SCPT03
Cone: 483:T1500F15U500

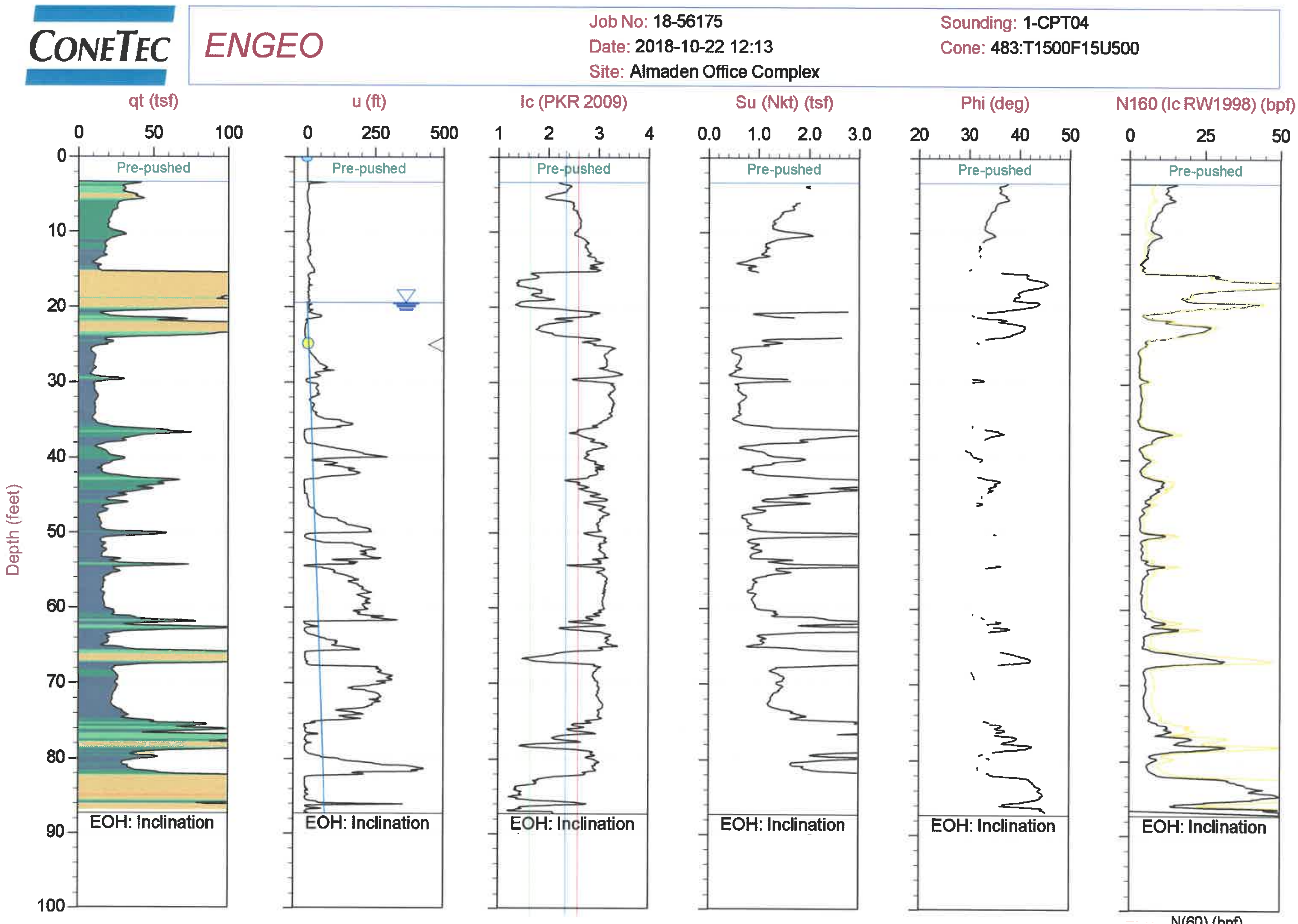


Max Depth: 20.725 m / 67.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131816m E: 598263m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line



Max Depth: 26.625 m / 87.35 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

File: 18-56175_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131844m E: 598242m
Sheet No: 1 of 1

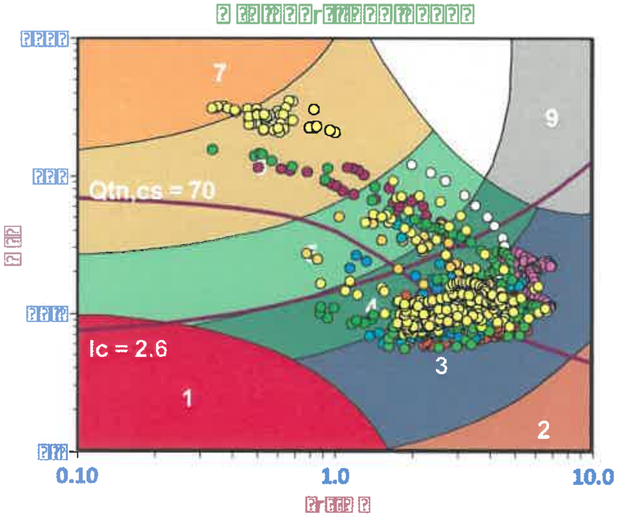
Soil Behavior Type (SBT) Scatter Plots

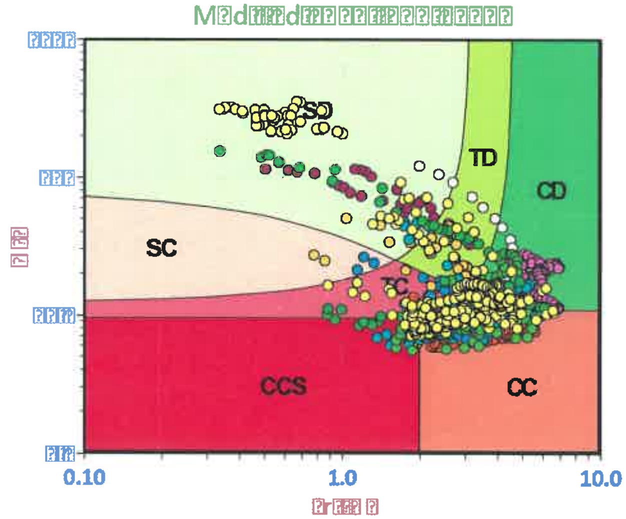
CONETEC

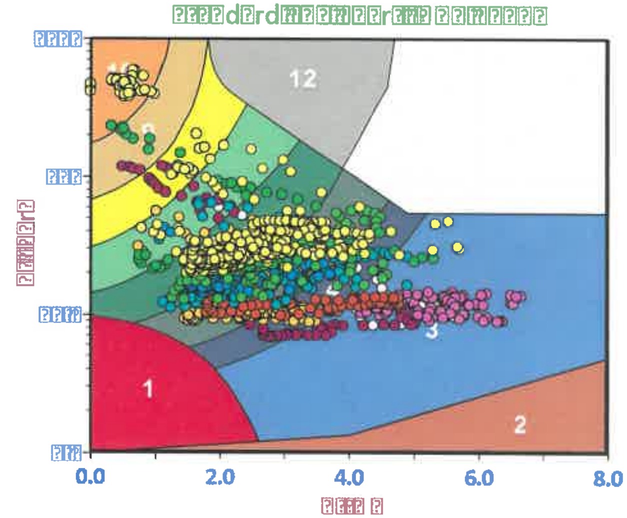
ENGEO

Job No: 18-56175
Date: 2018-10-22 08:17
Site: Almaden Office Complex

Sounding: 1-SCPT01
Cone: 448:T1500F15U500







Depth Ranges

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

Legend

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

Legend

- 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)

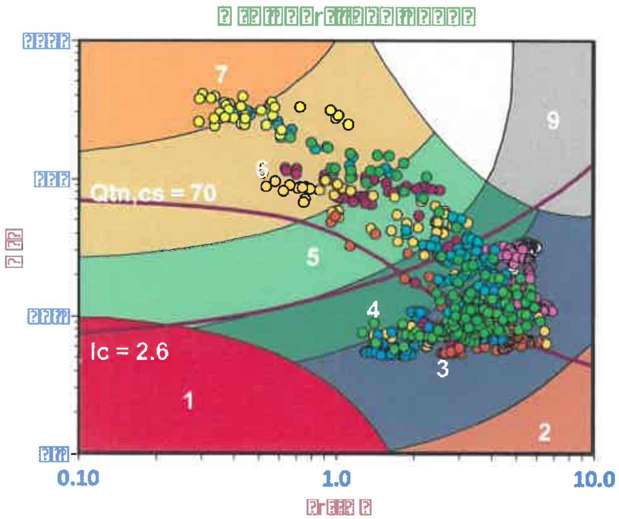
Legend

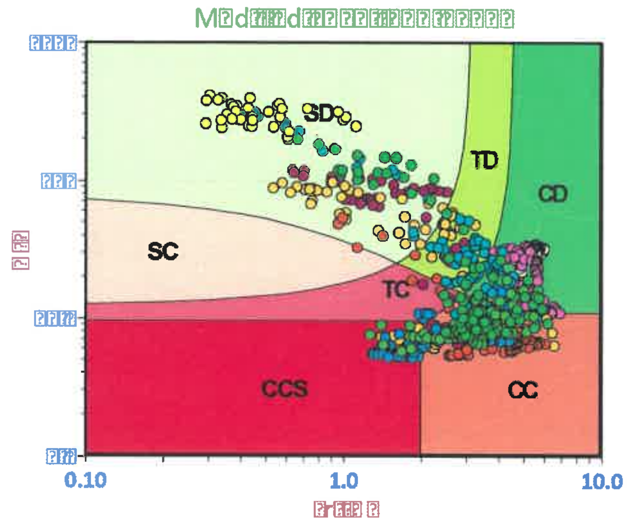
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

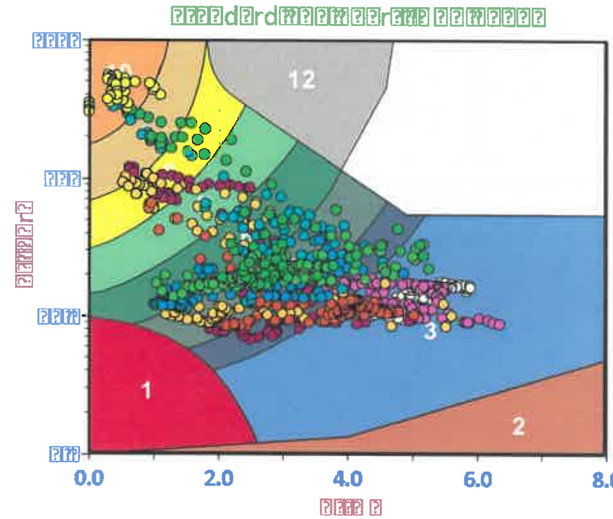
ENGEO

Job No: 18-56175
Date: 2018-10-22 13:16
Site: Almaden Office Complex

Sounding: 1-SCPT02
Cone: 483:T1500F15U500







Depth Ranges

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

Legend

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

Legend

- 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)

Legend

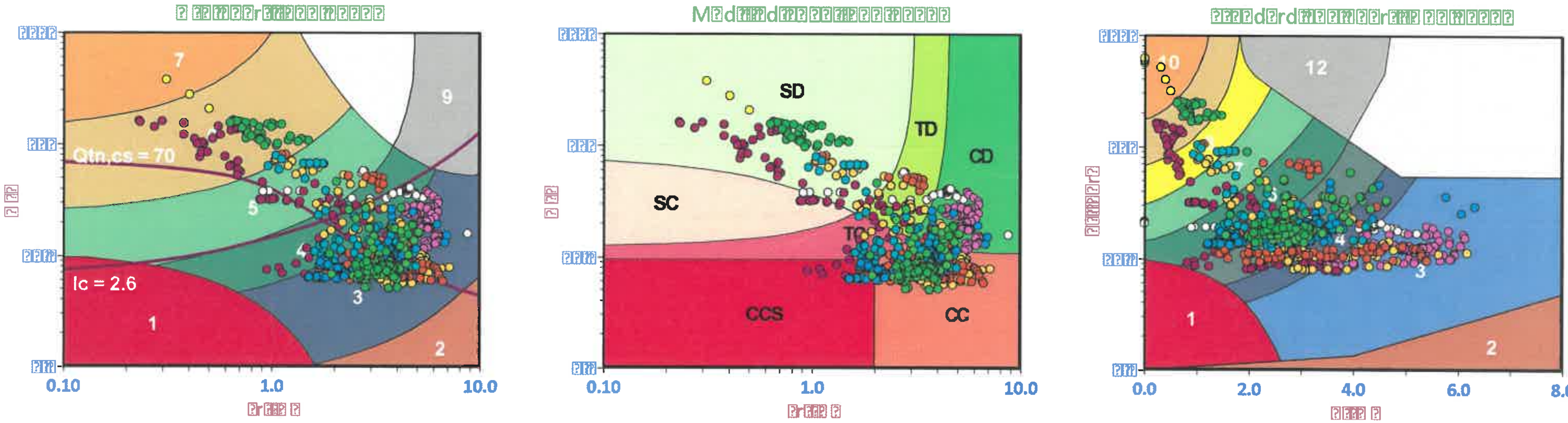
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 10:17
Site: Almaden Office Complex

Sounding: 1-SCPT03
Cone: 483:T1500F15U500



Depth Ranges
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- >30.0 to 37.5 ft
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- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

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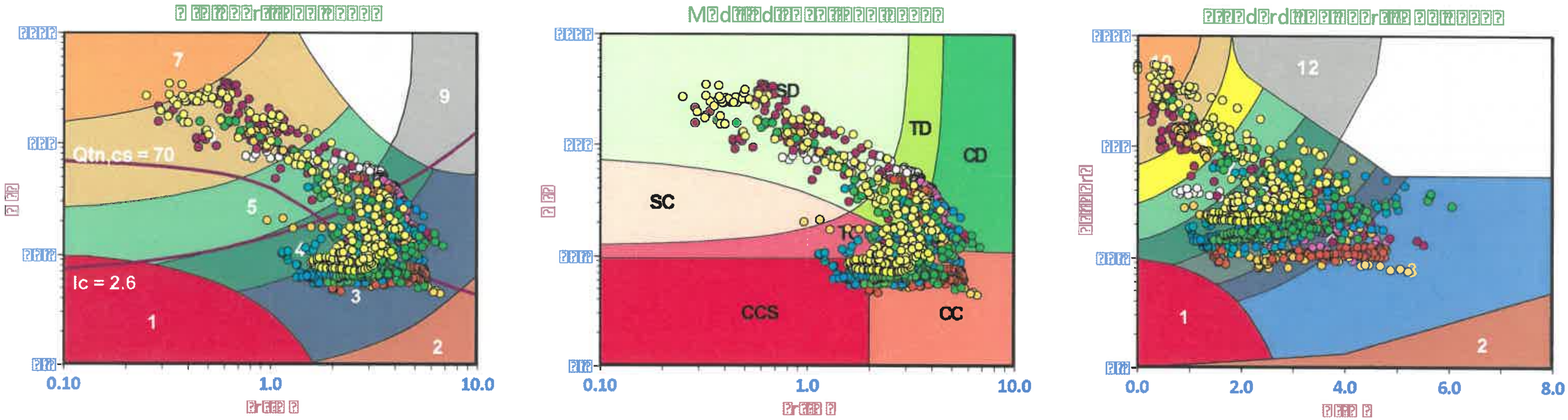
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

ConeTec

ENGEO

Job No: 18-56175
Date: 2018-10-22 12:13
Site: Almaden Office Complex

Sounding: 1-CPT04
Cone: 483:T1500F15U500



Depth Ranges

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

Legend

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

Legend

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

Legend

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

Seismic Cone Penetration Test Plots

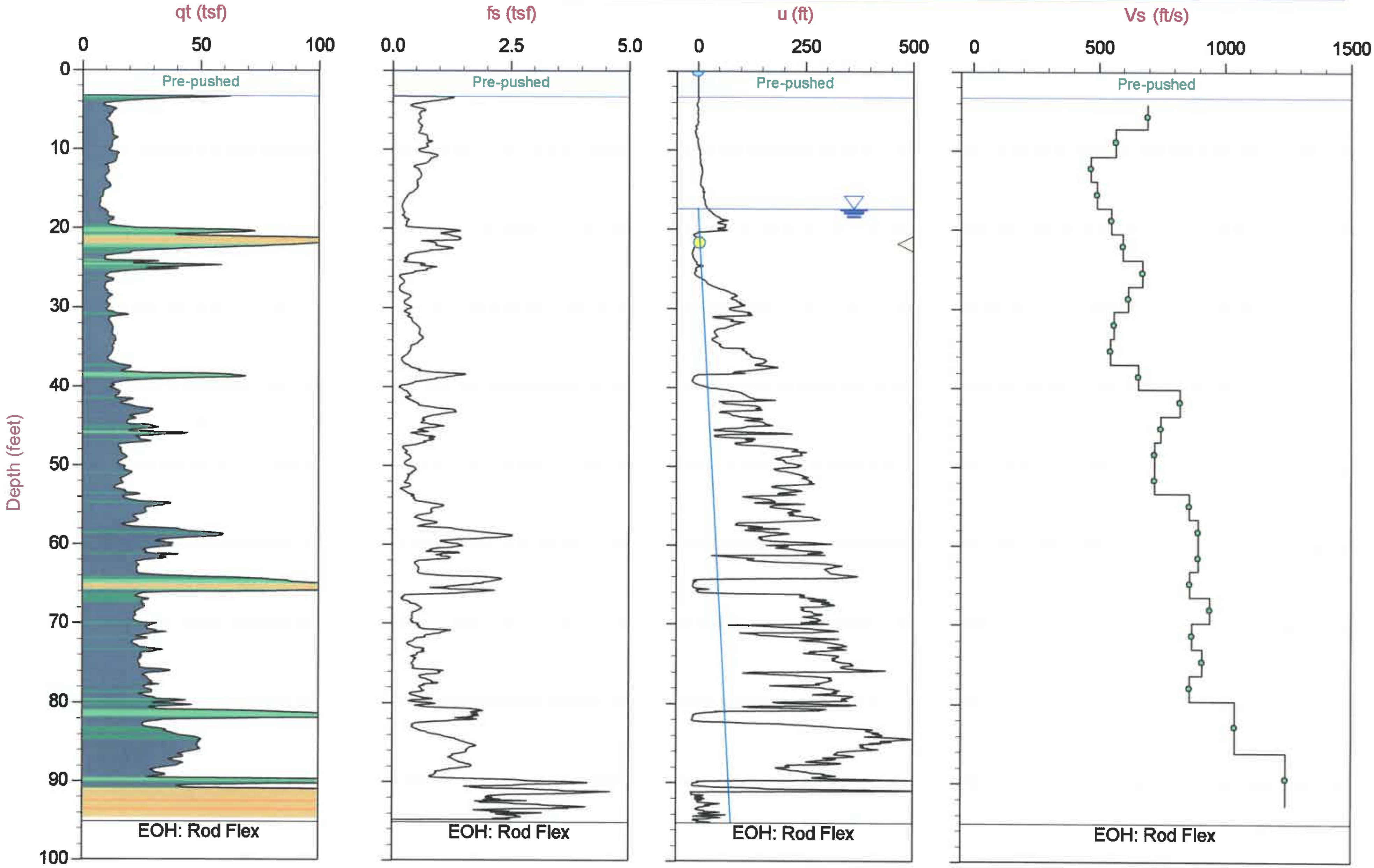
ConeTec

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 08:17
Site: Almaden Office Complex

Sounding: 1-SCPT01
Cone: 448:T1500F15U500



Max Depth: 29.000 m / 95.14 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131680m E: 598315m
Sheet No: 1 of 1

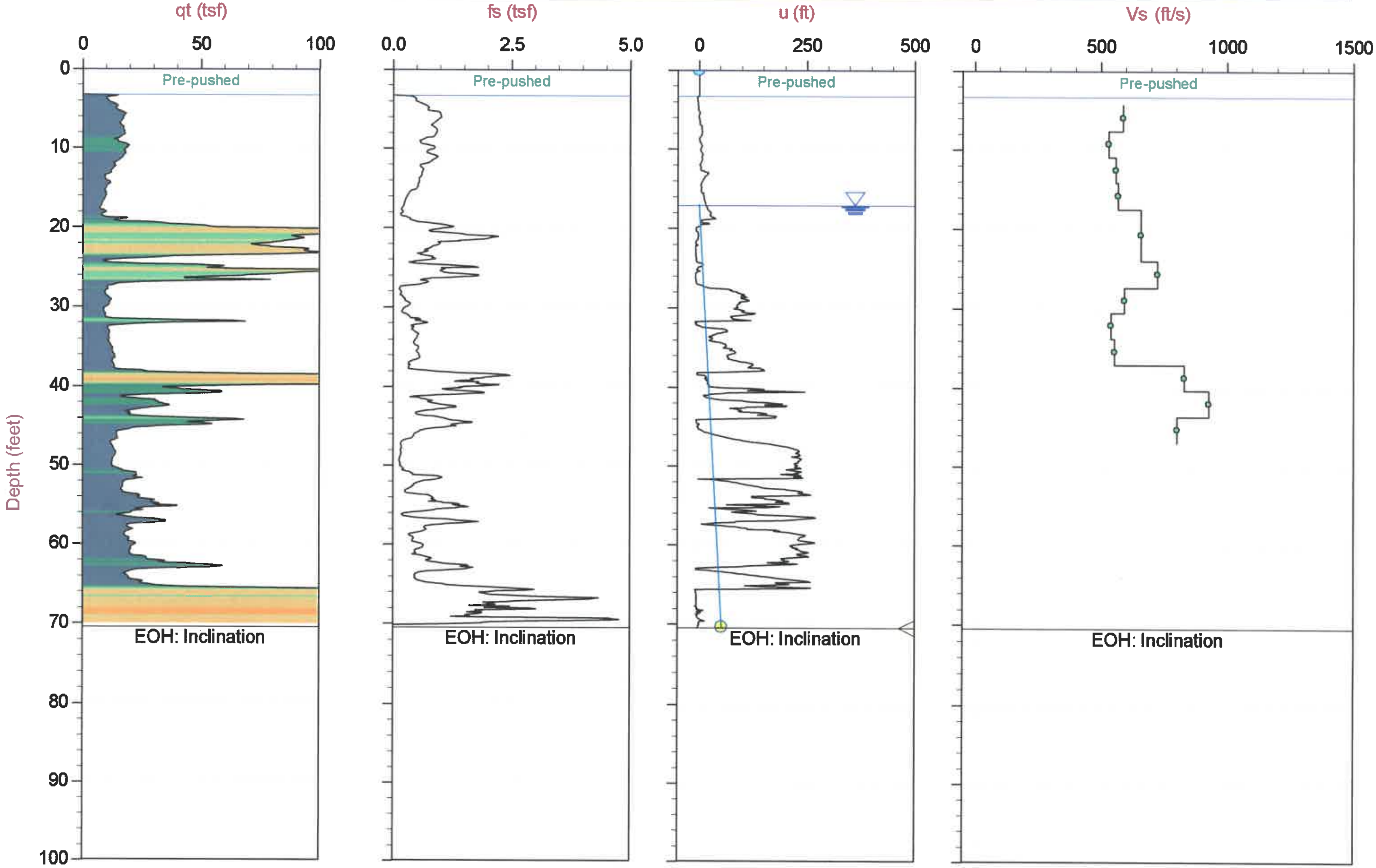
Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 13:16
Site: Almaden Office Complex

Sounding: 1-SCPT02
Cone: 483:T1500F15U500



Max Depth: 21.475 m / 70.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131728m E: 598310m
Sheet No: 1 of 1

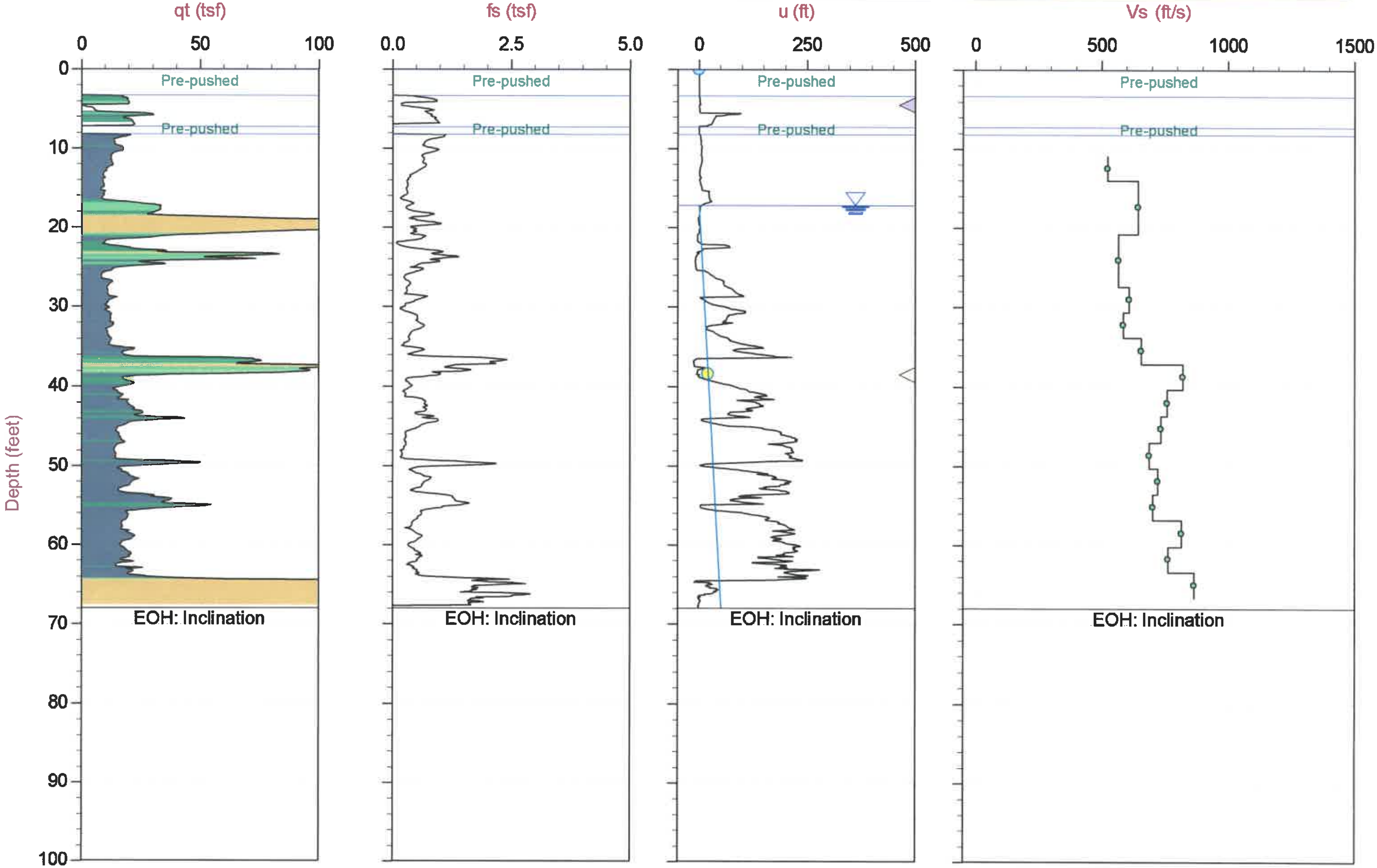
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

CONETEC

ENGEO

Job No: 18-56175
Date: 2018-10-22 10:17
Site: Almaden Office Complex

Sounding: 1-SCPT03
Cone: 483:T1500F15U500



Max Depth: 20.725 m / 67.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56175_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4131816m E: 598263m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

Seismic Cone Penetration Test Tabular Results

ConeTec

Job No:	18-56175
Client:	ENGEO Inc.
Project:	Almaden Office Complex
Sounding ID:	1-SCPT01
Date:	22-Oct-2018

Seismic Source:	Beam
Source Offset (ft):	2.07
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)
4.86	4.20	4.68			
7.97	7.32	7.60	2.92	4.21	693
11.38	10.73	10.93	3.32	5.87	566
14.60	13.94	14.10	3.17	6.79	467
17.95	17.29	17.41	3.32	6.75	491
21.16	20.50	20.61	3.20	5.82	549
24.51	23.85	23.94	3.33	5.60	595
27.82	27.16	27.24	3.30	4.91	673
31.00	30.35	30.42	3.17	5.15	616
34.38	33.73	33.79	3.37	6.02	560
37.66	37.01	37.07	3.28	6.01	545
40.85	40.19	40.24	3.18	4.84	656
44.23	43.57	43.62	3.38	4.11	821
47.51	46.85	46.90	3.28	4.40	745
50.79	50.13	50.17	3.28	4.55	721
54.07	53.41	53.45	3.28	4.55	721
57.35	56.69	56.73	3.28	3.82	859
60.63	59.97	60.01	3.28	3.67	894
63.91	63.25	63.29	3.28	3.67	894
67.19	66.54	66.57	3.28	3.82	860
70.47	69.82	69.85	3.28	3.49	941
73.75	73.10	73.13	3.28	3.77	869
77.03	76.38	76.41	3.28	3.60	911
80.31	79.66	79.69	3.28	3.82	860
86.88	86.22	86.24	6.56	6.31	1040
93.44	92.78	92.80	6.56	5.28	1242

Job No: 18-56175
Client: ENGEO Inc.
Project: Almaden Office Complex
Sounding ID: 1-SCPT02
Date: 22-Oct-2018

Seismic Source: Beam
Source Offset (ft): 2.07
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)
4.92	4.27	4.74			
8.30	7.64	7.92	3.18	5.42	587
11.58	10.92	11.12	3.20	6.04	530
14.83	14.17	14.32	3.20	5.73	559
18.11	17.45	17.58	3.25	5.73	568
24.67	24.02	24.10	6.53	9.91	659
28.05	27.39	27.47	3.37	4.64	725
31.23	30.58	30.65	3.17	5.36	592
34.45	33.79	33.86	3.21	5.94	540
37.80	37.14	37.20	3.34	6.04	553
41.01	40.35	40.41	3.21	3.87	830
44.36	43.70	43.75	3.34	3.60	928
47.64	46.98	47.03	3.28	4.09	802

	Job No:	18-56175
	Client:	ENGEO Inc.
	Project:	Almaden Office Complex
	Sounding ID:	1-SCPT03
	Date:	22-Oct-2018

Seismic Source:	Beam
Source Offset (ft):	2.07
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

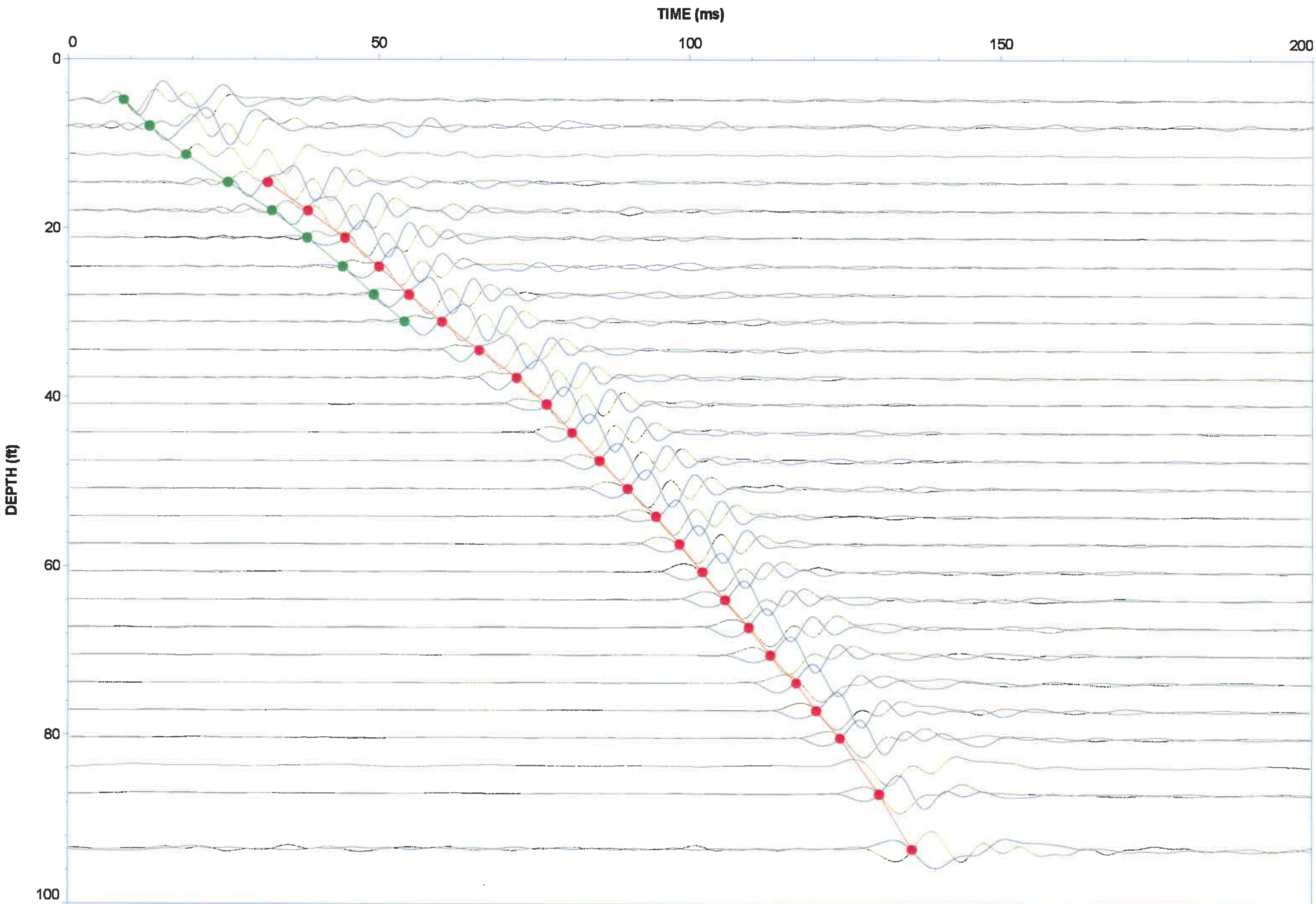
SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
11.58	10.92	11.12			
14.70	14.04	14.19	3.07	5.88	523
21.49	20.83	20.94	6.74	10.47	644
28.15	27.49	27.57	6.64	11.71	567
31.33	30.68	30.75	3.17	5.21	609
34.51	33.86	33.92	3.18	5.42	586
37.80	37.14	37.20	3.28	4.99	657
41.01	40.35	40.41	3.21	3.91	822
44.36	43.70	43.75	3.34	4.40	760
47.64	46.98	47.03	3.28	4.46	735
50.92	50.26	50.31	3.28	4.76	688
54.23	53.58	53.62	3.31	4.58	723
57.51	56.86	56.89	3.28	4.67	702
60.86	60.20	60.24	3.34	4.09	817
64.07	63.42	63.45	3.21	4.21	763
67.36	66.70	66.73	3.28	3.79	866

Seismic Cone Penetration Test Shear Wave (Vs) Traces

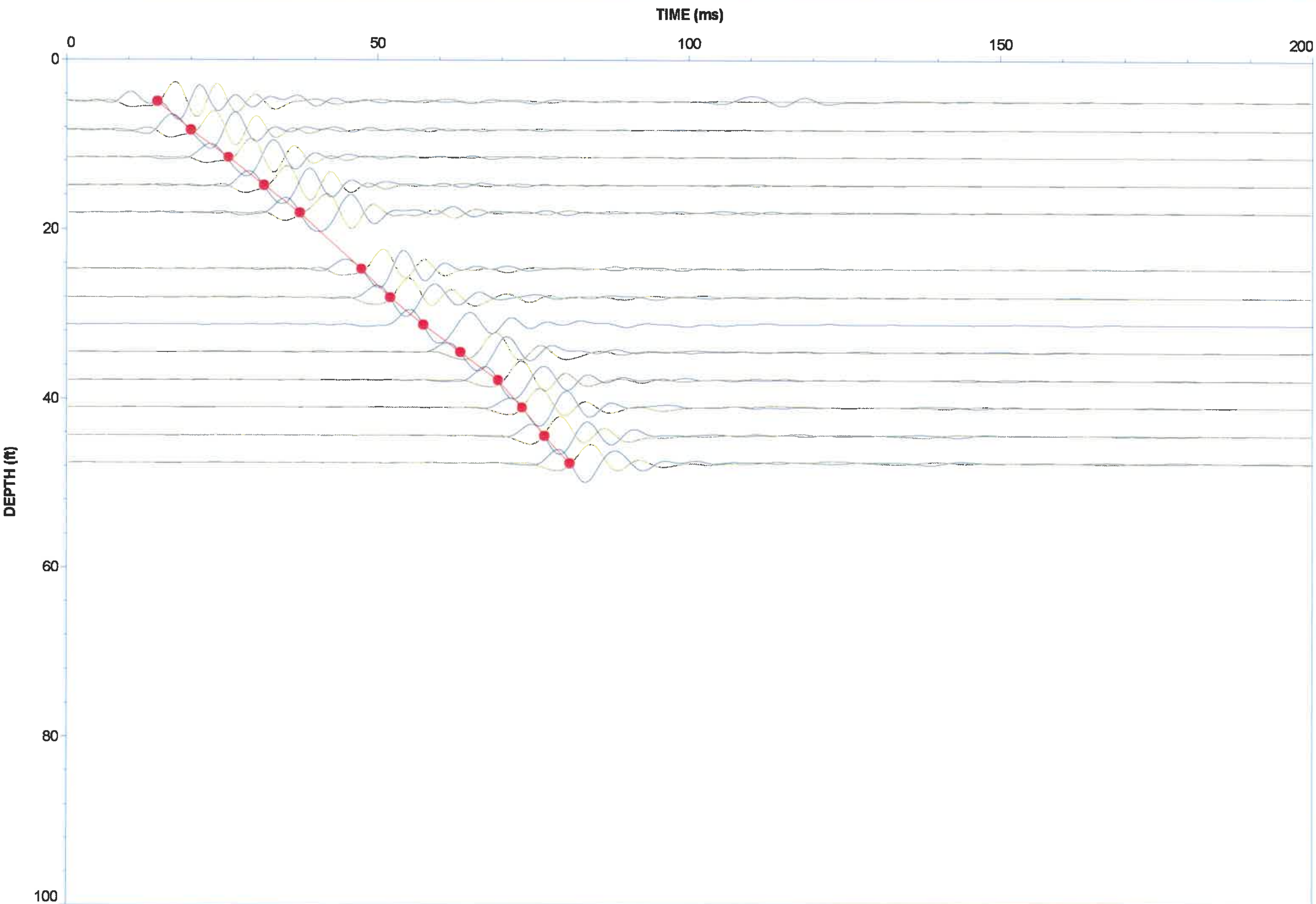
ConeTec

Job No: 18-56175 | Client: ENGEO | Project Title: Almaden Office Complex | Filter: 0 -200 Hz | Sounding ID: 1-SCPT-01 | Date: 22-Oct-2018

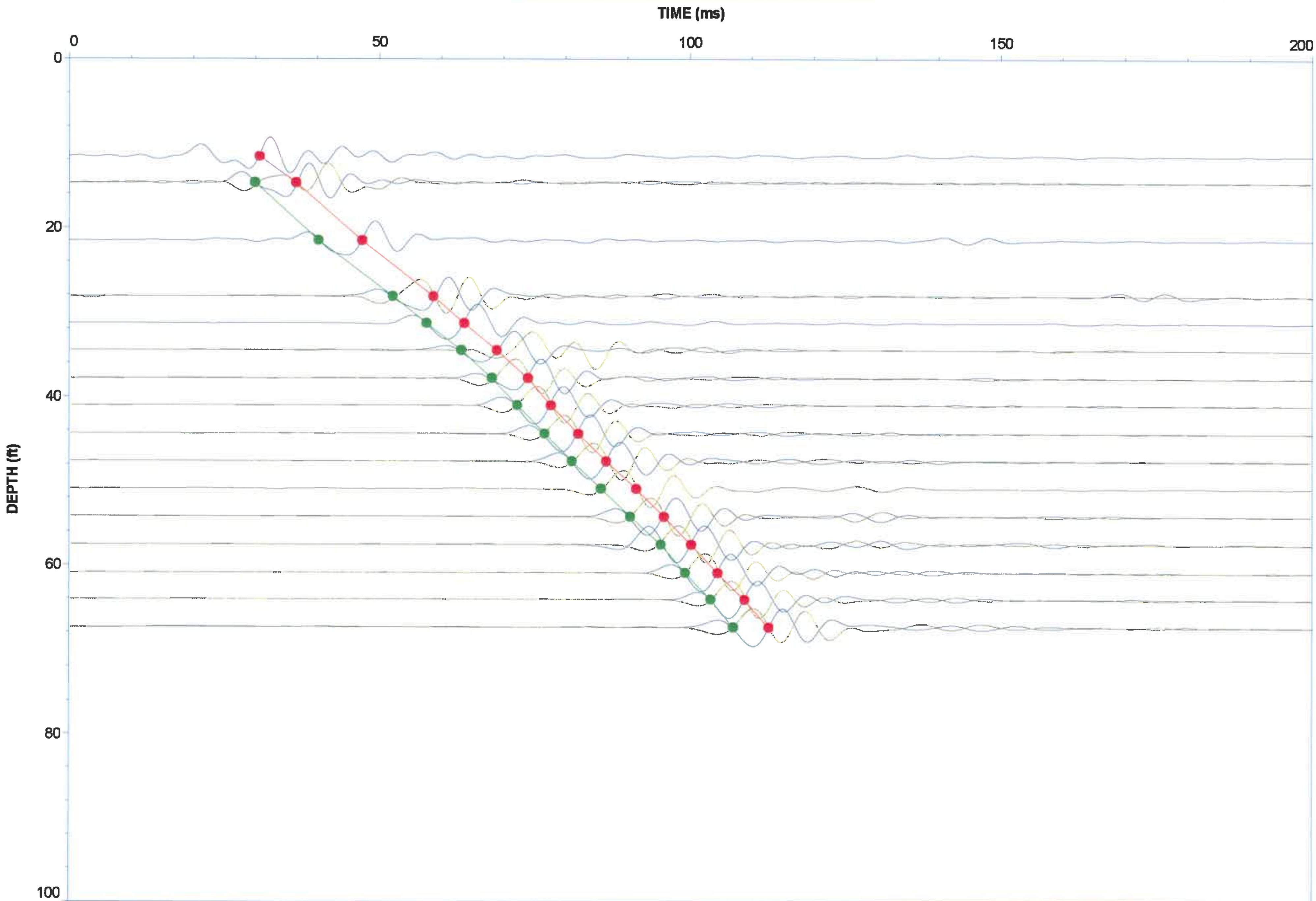


ConeTec

Job No: 18-56175 | Client: ENGEO | Project Title: Almaden Office Complex | Filter: 0 -200 Hz | Sounding ID: 1-SCPT-02 | Date: 22-Oct-2018



Job No: 18-56175	Client: ENGEO	Project Title: Almaden Office Complex	Filter: 0 -200 Hz	Sounding ID: 1-SCPT-03	Date: 22-Oct-2018



Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

ConeTec

Job No: 18-56175
Client: ENGEO Inc.
Project: Almaden Office Complex
Start Date: 22-Oct-2018
End Date: 22-Oct-2018

CPTu PORE PRESSURE DISSIPATION SUMMARY

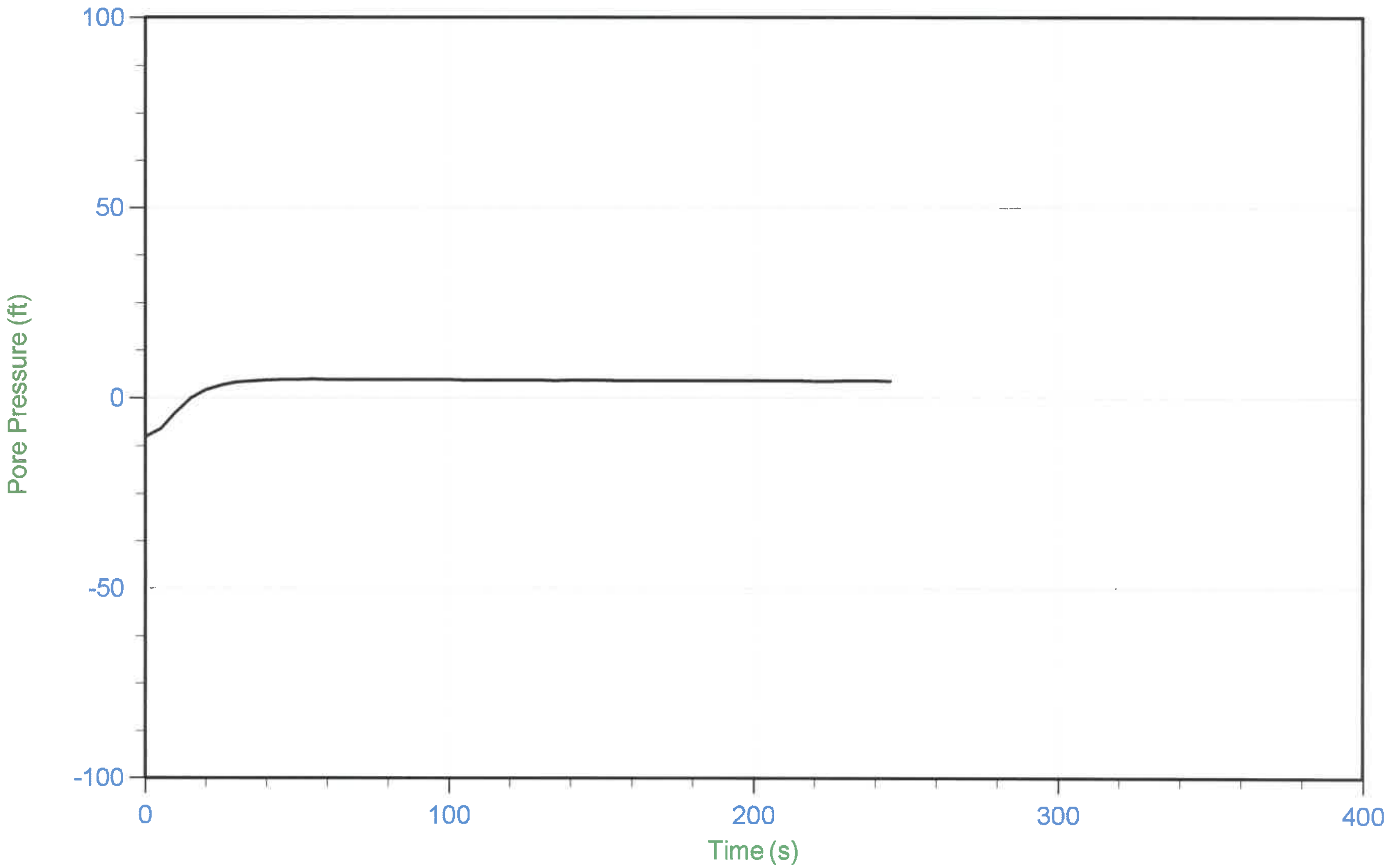
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
1-SCPT01	18-56175_SP01	15	245	21.817	4.4	17.4
1-SCPT02	18-56175_SP02	15	265	70.455	53.4	17.0
1-SCPT03	18-56175_SP03	15	205	4.429	Not Achieved	
1-SCPT03	18-56175_SP03	15	330	38.467	21.3	17.2
1-CPT04	18-56175_CP04	15	355	25.016	5.6	19.4

CONETEC

ENGEO

Job No: 18-56175
Date: 10/22/2018 08:17
Site: Almaden Office Complex

Sounding: 1-SCPT01
Cone: 448:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-56175_SP01.PPF
Depth: 6.650 m / 21.817 ft
Duration: 245.0 s

U Min: -10.2 ft
U Max: 4.9 ft

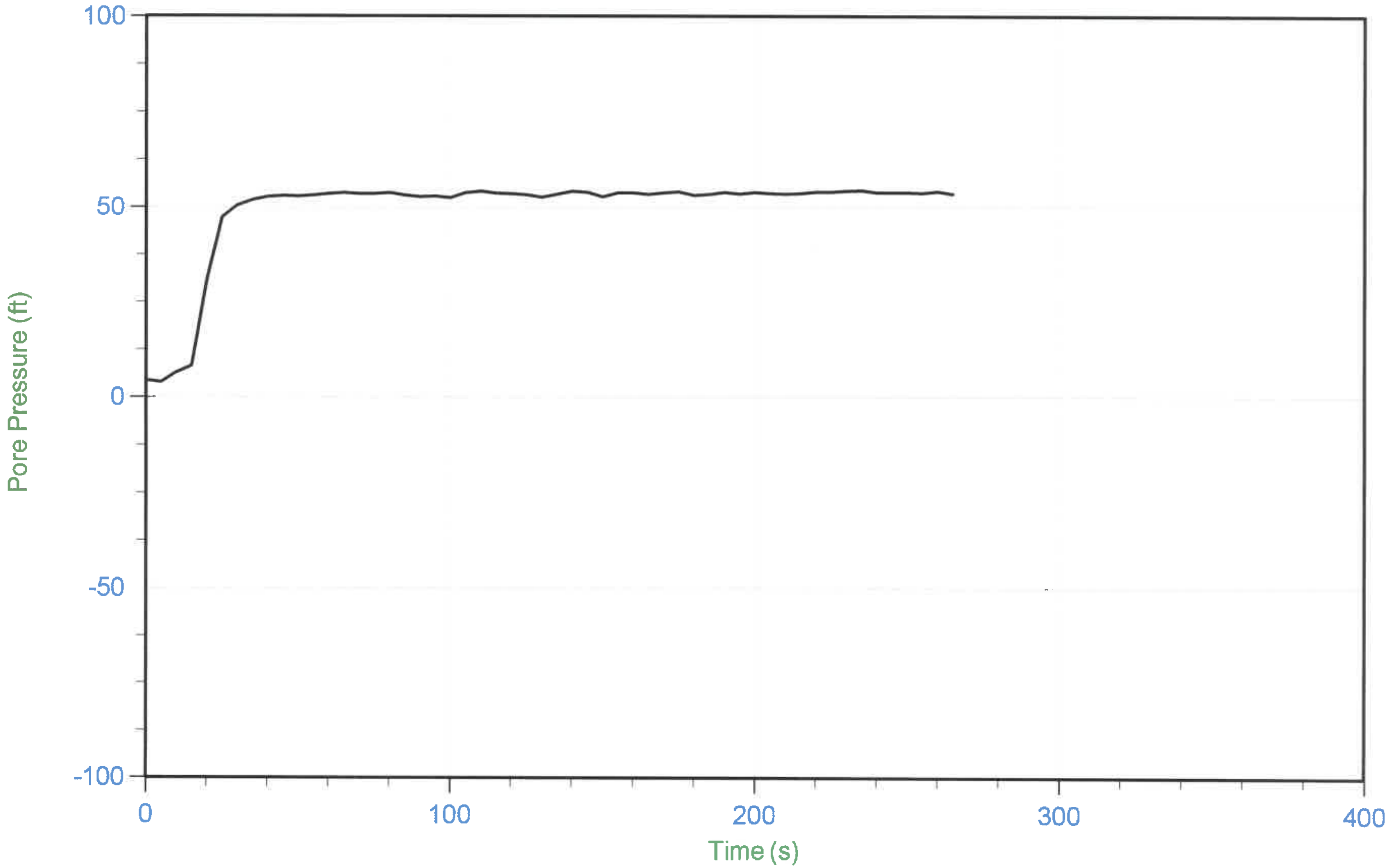
WT: 5.303 m / 17.400 ft
Ueq: 4.4 ft

CONETEC

ENGEO

Job No: 18-56175
Date: 10/22/2018 13:16
Site: Almaden Office Complex

Sounding: 1-SCPT02
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-56175_SP02.PPF
Depth: 21.475 m / 70.455 ft
Duration: 265.0 s

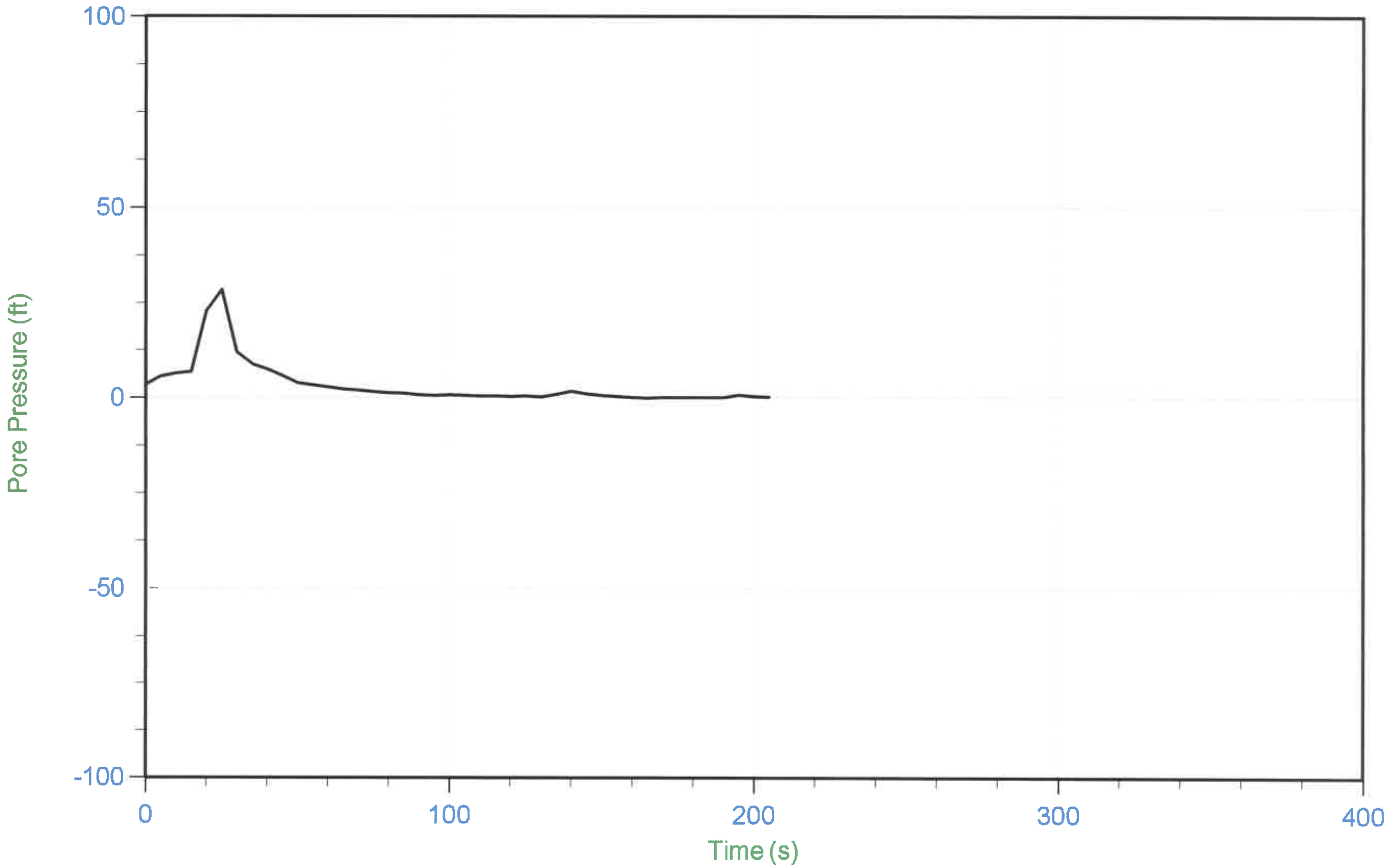
U Min: 3.9 ft
U Max: 54.4 ft

WT: 5.195 m / 17.042 ft
Ueq: 53.4 ft

ENGEO

Job No: 18-56175
Date: 10/22/2018 10:17
Site: Almaden Office Complex

Sounding: 1-SCPT03
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

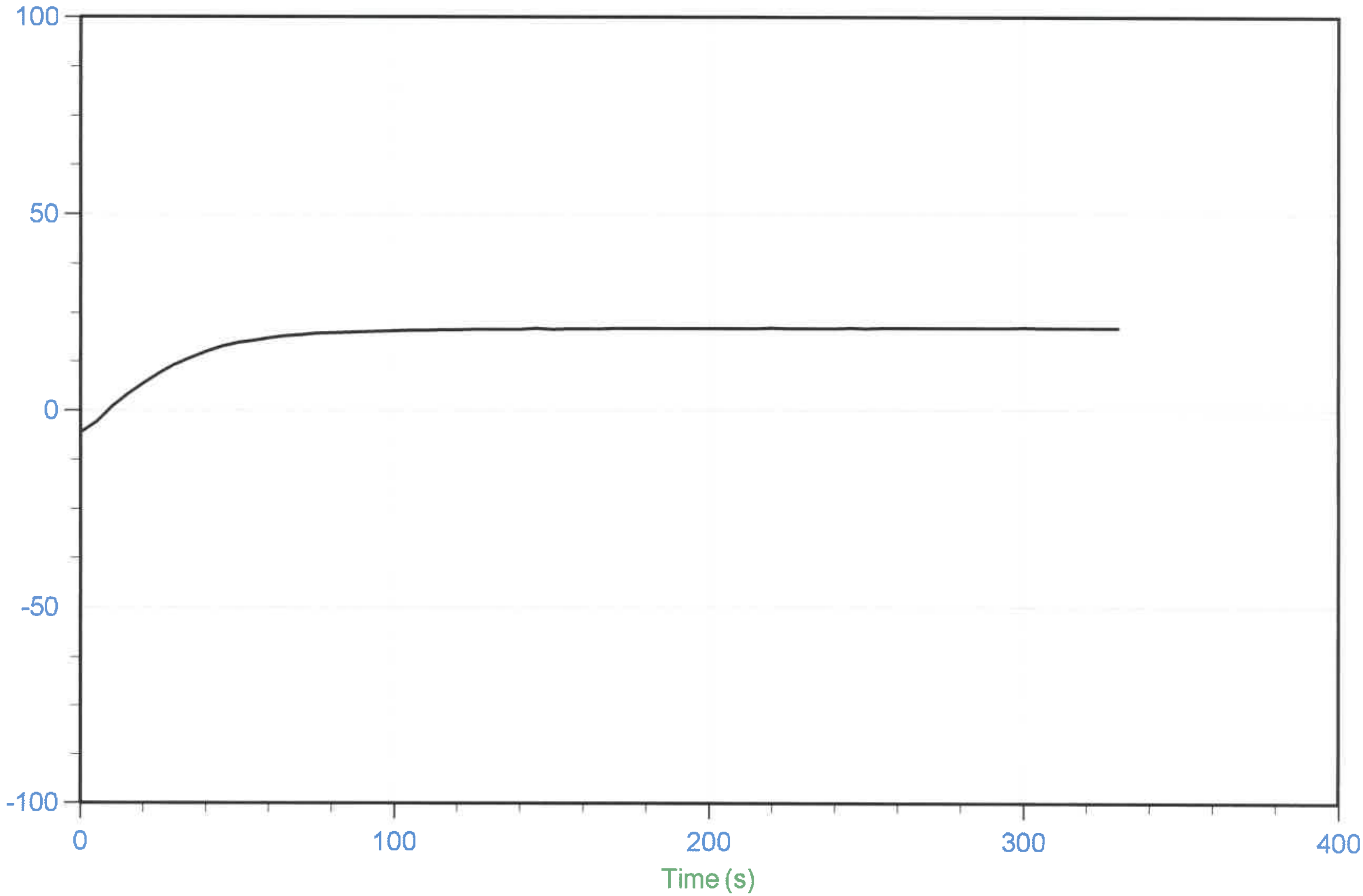
Filename: 18-56175_SP03.PPF
Depth: 1.350 m / 4.429 ft
Duration: 205.0 s

U Min: -0.0 ft
U Max: 28.4 ft

ENGEO

Job No: 18-56175
Date: 10/22/2018 10:17
Site: Almaden Office Complex

Sounding: 1-SCPT03
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-56175_SP03.PPF
Depth: 11.725 m / 38.467 ft
Duration: 330.0 s

U Min: -5.7 ft
U Max: 21.2 ft

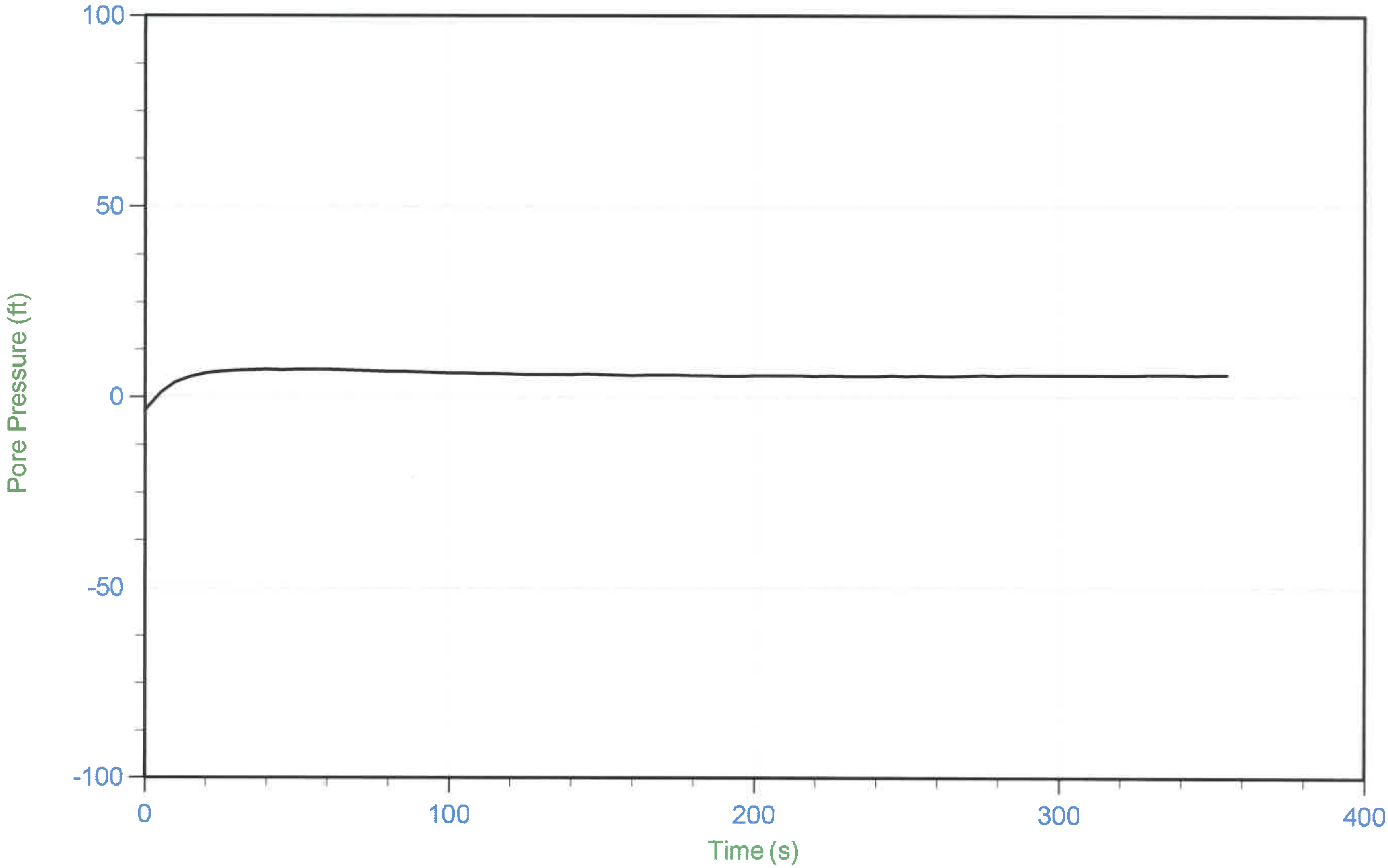
WT: 5.237 m / 17.182 ft
Ueq: 21.3 ft

CONETEC

ENGEO

Job No: 18-56175
Date: 10/22/2018 12:13
Site: Almaden Office Complex

Sounding: 1-CPT04
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-56175_CP04.PPF
Depth: 7.625 m / 25.016 ft
Duration: 355.0 s

U Min: -3.5 ft
U Max: 7.3 ft

WT: 5.911 m / 19.394 ft
Ueq: 5.6 ft

Geokon Piezometer Installation Summary

ConeTec

CONETEC

Job No: 18-56175
Client: ENGEO Inc.
Project: Almaden Office Complex
Start Date: 23-Oct-2018
End Date: 23-Oct-2018

GEOKON VIBRATING WIRE PIEZOMETER INSTALLATION SUMMARY

Location ID	Adjacent CPT	Logger Type	Data Logger Serial Number	Installation Depth (ft)	Deployment Date	Deployment Time (hh:mm)	Piezometer Serial Number[1]	Piezometer Model Number	Piezometer Baseline Prior to Deployment (digit)	Thermistor Baseline Prior to Deployment (°C)	Piezometer Reading after Deployment (digit)	Thermistor Reading after Deployment (°C)	Northing[2] (m)	Easting (m)	Refer to Notation Number
1-VWP1	1-CPT04	LC-2	1837572	45.0	23-Oct-2018	09:15	1808098	4500DPCT-350 kPa	8715.0	14.7	7844.1	20.5	4131844	598242	

1. Geokon calibration sheets for the piezometers are provided in the data release folder.
2. Coordinates were collected with consumer grade GPS device with datum WGS84 / UTM 10.

Geokon Piezometer Calibration Records

GEOKON 48 Spencer St. Lebanon, NH 03766 USA

Vibrating Wire Pressure Transducer Calibration Report

Model Number: 4500DPCT-350 kPa

Date of Calibration: March 13, 2018

This calibration has been verified/validated as of 03/27/2018

Serial Number: 1808098

Temperature: 21.30 °C

Calibration Instruction: VW Pressure Transducers

Barometric Pressure: 984.5 mbar

Cable Length: 65 feet

Technician:

Applied Pressure (kPa)	Gage Reading 1st Cycle	Gage Reading 2nd Cycle	Average Gage Reading	Calculated Pressure (Linear)	Error Linear (%FS)	Calculated Pressure (Polynomial)	Error Polynomial (%FS)
0.0	8735	8736	8736	-1.066	-0.30	0.077	0.02
70.0	8034	8035	8035	70.09	0.03	69.86	-0.04
140.0	7336	7337	7337	141.0	0.27	140.0	0.01
210.0	6647	6648	6648	210.9	0.27	210.0	0.01
280.0	5965	5965	5965	280.2	0.06	280.0	0.01
350.0	5288	5289	5289	348.8	-0.33	350.0	-0.01

(kPa) Linear Gage Factor (G): -0.1015 (kPa/ digit)

Polynomial Gage factors: A: 7.151E-07 B: -0.1115 C: ______

Thermal Factor (K): 0.03026 (kPa/ °C)

Calculate C by setting P=0 and R_1 = initial field zero reading into the polynomial equation

(psi) Linear Gage Factor (G): -0.01472 (psi/ digit)

Polynomial Gage Factors: A: 1.037E-07 B: -0.01618 C: ______

Thermal Factor (K): 0.004389 (psi/ °C)

Calculate C by setting P=0 and R_1 = initial field zero reading into the polynomial equation

Calculated Pressures:

Linear, $P = G(R_1 - R_0) + K(T_1 - T_0) - (S_1 - S_0)$*

Polynomial, $P = AR_1^2 + BR_1 + C + K(T_1 - T_0) - (S_1 - S_0)$*

*Barometric pressures expressed in kPa or psi. Barometric compensation is not required with vented transducers.

Factory Zero Reading: 8705 Temperature: 21.4 °C Barometer: 1010.1 mbar

The above instrument was found to be in tolerance in all operating ranges.
The above named instrument has been calibrated by comparison with standards traceable to the NIST, in compliance with ANSI Z540-1.

APPENDIX D

SURFACE WAVE MEASUREMENTS REPORT
(GEOVision Geophysical Services)

REPORT

SURFACE WAVE MEASUREMENTS

ALMADEN OFFICE COMPLEX PROJECT
NORTHWEST CORNER OF SOUTH ALMADEN BLVD AND BALBACH STREET
SAN JOSE, CALIFORNIA

GEO*Vision* Project No. 18450

Prepared for

ENGEO, Inc.
2010 Crow Canyon Place, Suite 250
San Ramon, California 94583-4634
(925) 866-9000

Prepared by

GEO*Vision* Geophysical Services, Inc.
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

Report 18450-01

November 21, 2018

TABLE OF CONTENTS

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1 INTRODUCTION

In-situ seismic measurements using active- and passive-source surface wave techniques were performed at the Almaden office complex project site located at the northwest corner of South Almaden Blvd. and Balbach St. in San Jose, California on October 24-25 and November 17 and 18, 2018. The purpose of this investigation was to provide a shear (S) wave velocity profile to a depth of over 300 m and estimate the average S-wave velocity of the upper 30 m (V_{S30}). The active-source surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The passive-source surface wave techniques consisted of the horizontal over vertical spectral ratio (HVSR) and array microtremor methods. The locations of the active- and passive-source surface wave arrays are shown on Figure 1. HVSR measurements were made at two locations on site (Figure 1). MASW measurements were made at a single location at the site (Array 2) as shown in Figure 1. Array microtremor measurements were made using both small and large aperture arrays (Arrays 1 and 3) as shown in Figure 1.

V_{S30} is used in the NEHRP provisions and the Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design (BSSC, 2003). The average shear wave velocity of the upper 100 ft (V_{S100ft}) is used in the International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – $V_{S30} > 1500$ m/s (UBC) or $V_{S100ft} > 5{,}000$ ft/s (IBC)
Class B – rock – $760 < V_{S30} \leq 1500$ m/s (UBC) or $2{,}500 < V_{S100ft} \leq 5{,}000$ ft/s (IBC)
Class C – very dense soil and soft rock – $360 < V_{S30} \leq 760$ m/s (UBC)
or $1{,}200 < V_{S100ft} \leq 2{,}500$ ft/s (IBC)
Class D – stiff soil – $180 < V_{S30} \leq 360$ m/s (UBC) or $600 < V_{S100ft} \leq 1{,}200$ ft/s (IBC)
Class E – soft soil – $V_{S30} < 180$ m/s (UBC) or $V_{S100ft} < 600$ ft/s (IBC)
Class F – soils requiring site-specific evaluation

At many sites, active surface wave techniques (MASW) with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain a 30 m (100 ft) S-wave velocity sounding. At sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to 30 m and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive surface wave techniques, such as the array microtremor technique can be used to extend the depth of investigation at sites that have adequate ambient noise conditions. It should be noted that two-dimensional passive-source surface wave arrays (e.g. triangular, circular, or L-shaped arrays) are expected to perform better than linear arrays.

This report contains the results of the active and passive surface wave measurements conducted at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Data modeling is presented in Section 5 and interpretation and results are presented in Section 6. References and our professional certification are presented in Sections 7 and 8, respectively.

2 OVERVIEW OF SURFACE WAVE TECHNIQUES

2.1 Introduction

Active- and passive-source (ambient vibration) surface wave techniques are routinely utilized for site characterization. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the horizontal over vertical spectral ratio (HVSR) technique and the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. Surface waves of different wavelengths (λ) or frequencies (f) sample different depth. As a result of the variance in the shear stiffness of the distinct layers, waves with different wavelengths propagate at different phase velocities; hence, dispersion. A surface wave dispersion curve is the variation of V_R or V_L with λ or f. The Rayleigh wave phase velocity (V_R) depends primarily on the material properties (V_S, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. The Love wave phase velocity (V_L) depends primarily on V_S and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor (Q). Rayleigh wave techniques are utilized to measure vertically polarized S-waves (S_V-wave); whereas, Love wave techniques are utilized to measure horizontally polarized S-waves (S_H-wave).

2.2 Surface Wave Techniques

The MASW, array microtremor, and HVSR techniques were utilized during this investigation and are discussed below. The MASW and array microtremor surveys were designed to measure Rayleigh wave propagation.

2.2.1 MASW Technique

A description of the MASW method is given by Park, 1999a and 1999b and Foti, 2000. Ground motions are typically recorded by 24, or more, geophones typically spaced 1 to 3 m apart along a linear array and connected to a seismograph. Energy sources for shallow investigations include various sized hammers and vehicle mounted weight drops. When applying the MASW technique to develop a one-dimensional (1-D) V_S model, the surface-wave data, preferably, are acquired using multiple-source offsets at both ends of the array. The most commonly applied MASW technique is the Rayleigh-wave based MASW method, which we refer to as MAS_RW to distinguish from Love-wave based MASW (MAS_LW). MAS_RW and MAS_LW acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively. MAS_RW data are generally recorded using a vertical source and vertical geophone, but may also be recorded using a horizontal geophone with radial (in-line) orientation. MAS_LW data are recorded using transversely orientated horizontal source and transverse horizontal geophone.

A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to frequency-wavenumber (f-k) space in which the fundamental or higher surface-wave modes can be easily identified as energy maxima and picked. Frequency and/or wavenumber can easily be mapped to phase velocity, slowness, or wavelength using the following properties: $k = 2\pi/\lambda$, $\lambda = v/f$. Common wave-field transforms include: the f-k transform (a 2D fast Fourier transform), slant-stack transform (also referred to as intercept-slowness or τ-p transform and equivalent to linear Radon transform), frequency domain beamformer, and phase-shift transform. The minimum wavelength that can be recovered from an MASW data set without spatial aliasing is equal to the minimum receiver spacing. Occasionally, SASW analysis procedures are used to extract surface wave dispersion data, from fixed receiver pairs, at smaller wavelengths than can be recovered by wavefield transformation. Construction of a dispersion curve over the wide frequency/wavelength range necessary to develop a robust V_S model while also limiting the maximum wavelength based on an established near-field criterion (e.g. Yoon and Rix, 2009; Li and Rosenblad, 2011), generally requires multiple source offsets.

Although the clear majority of MASW surveys record Rayleigh waves, it has been shown that Love wave techniques can be more effective in some environments, particularly shallow rock sites and sites with a highly attenuative, low velocity surface layer (Xia, et al., 2012; **GEO***Vision*, 2012; Yong, et al., 2013; Martin, et al., 2014). Rayleigh wave techniques, however, are generally more effective at sites where velocity gradually increases with depth because larger energy sources are readily available for generation of Rayleigh waves. Rayleigh wave techniques are also more applicable to sites with high velocity layers and/or velocity inversions because the presence of such structures is more apparent in the Rayleigh wave dispersion curves than in Love wave dispersion curves. Rayleigh wave techniques are preferable at sites with a high velocity surface layer because Love waves do not theoretically exist in such environments. Occasionally, the horizontal radial component of a Rayleigh wave may yield higher quality dispersion data than the vertical component because different modes of propagation may have more energy in one component than the other. Recording both the vertical and horizontal components of the Rayleigh wave is particularly useful at sites with complex modes of propagation or when attempting to recover multiple Rayleigh wave modes for multi-mode modeling as demonstrated in Dal Moro, et al, 2015. Joint inversion of Rayleigh and Love wave data may yield more accurate V_S models and also offer a means to investigate anisotropy, where S_V- and S_H-wave velocity are not equal, as shown in Dal Moro and Ferigo, 2011.

2.2.2 Array Microtremor Technique

A detailed discussion of the array microtremor method can be found in Okada, 2003. Unlike active source techniques which use an active energy source (i.e. hammer), the array microtremor technique (also referred to as passive surface wave or array ambient vibration method) records background noise (ambient vibrations) emanating from ocean wave activity, wind noise, traffic, industrial activity, construction, etc. The technique uses 4, or more, receivers aligned in a 2-dimensional array. Triangle, circle, semi-circle, and "L" shaped arrays are commonly used, although any 2-dimensional arrangement of receivers can be used. For investigation of the upper 100 m, receivers typically consist of 1 to 4.5 Hz geophones. For deeper investigations, 5 to 120 s seismometers are generally utilized. The nested triangle array, which consists of several embedded equilateral triangles, is popular as it provides accurate dispersion curves with a relatively small number of geophones. The "L" array is useful at sites located at the corner of intersecting streets. The maximum receiver separation in an array should be at a minimum equal to the desired depth of investigation. Typically, 15 to 60 minutes of ambient vibration data is recorded depending on the size of the array, desired depth of investigation, and noise conditions. Investigations to depths on the order of 1 km may require that ambient vibrations are recorded for a much longer duration. The surface wave dispersion curve is typically estimated from array microtremor data using various f-k methods such as beam-forming (Lacoss, et al., 1969), and maximum-likelihood (Capon, 1969), and the spatial-autocorrelation (SPAC) method. The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods. The SPAC method was originally based on work by Aki, 1957 and has since been extended and modified (Ling and Okada, 1993 and Ohori *et al.*, 2002) to permit the use of noncircular arrays, and is now collectively referred to as extended spatial autocorrelation (ESPAC or ESAC). Further modifications to the SPAC method permit the use of irregular or random arrays (Bettig *et al.*, 2001). Although it is common to apply SPAC methods to obtain a surface wave dispersion curve for modeling, other approaches involve direct modeling of the coherency data, also referred to as SPAC coefficients (Asten, 2006 and Asten, *et al.*, 2015).

FK and HRFK methods are generally expected to perform better when ambient vibration sources are not azimuthally well-distributed (e.g. rural area where primary noise source is a large industrial facility). SPAC methods are expected to perform better when noise sources are azimuthally well-distributed (e.g. in a large urbanized area).

The minimum and maximum wavelength surface wave that can be extracted from an array microtremor dataset acquired utilizing a symmetric array is typically set equal to the minimum and twice the maximum receiver spacings, respectively.

2.2.3 H/V Spectral Ratio Technique

The horizontal-to-vertical spectral ratio (HVSR) technique was first introduced by Nogoshi and Igarashi (1971) and popularized by Nakamura (1989). This technique utilizes single-station recordings of ambient vibrations (also referred to as microtremors and ambient noise) made with a three-component seismometer. In this method, the ratio of the Fourier amplitude spectra of the horizontal and vertical components is calculated to determine the frequency of the maximum HVSR response (HVSR peak frequency), commonly accepted as an approximation of the fundamental frequency (f_0) of the sediment column overlying bedrock. The HVSR peak frequency associated with bedrock is a function of the bedrock depth and S-wave velocity of the sediments overlying bedrock. The theoretical HVSR response can be calculated for an S-wave velocity model using modeling schemes based on surface wave ellipticity, vertically propagating body waves, or diffuse wavefields containing body and surface waves. The HVSR frequency peak can also be estimated using the quarter-wavelength approximation:

$$f_0 = \frac{\overline{V}_S}{4z}$$

where f_0 is the site fundamental frequency and $\overline{V}_S$ is the average shear-wave velocity of the soil column overlying bedrock at depth z.

2.3 Surface Wave Dispersion Curve Modeling

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. The theoretical model used to interpret the dispersion curve assumes horizontally layered, laterally invariant, homogeneousisotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good "global" estimate of the material properties along the array. The results may be more representative of the site than a borehole "point" estimate.

The surface wave forward problem is typically solved using the Thomson-Haskell transfer-matrix (Thomson, 1950; Haskell, 1953) later modified by Dunkin (1965) and Knopoff (1964), dynamic stiffness matrix (Kausel and Roësset, 1981), or reflection and transmission coefficient (Kennett, 1974) methods. All of these methods can determine fundamental- and higher-mode phase velocities, which correspond to plane waves in 2-D space. The transfer-matrix method is often used in MASW and passive surface-wave software packages, whereas the dynamic stiffness matrix is utilized in many SASW software packages. MAS_RW and/or passive surface-wave modeling may involve modeling of the fundamental mode, some form of effective mode, or multiple individual modes (multi-mode). As outlined in Roësset et al. (1991), several options exist for forward modeling of Rayleigh wave SASW data. One formulation takes into account only fundamental mode plane Rayleigh-wave motion (called the 2-D solution), whereas another includes all stress waves (e.g. body, fundamental, and higher mode surface waves) and incorporates a generalized receiver geometry (3-D global solution) or actual receiver geometry (3-D array solution).

The fundamental mode assumption is generally applicable to modeling Rayleigh-wave dispersion data collected at normally dispersive sites, providing there are not abrupt increases in velocity or steep velocity gradients. Effective-mode or multi-mode approaches are often required for irregularly dispersive sites and sites with steep velocity gradients at shallow depth. If active and passive surface wave data are combined or MAS_RW data are combined from multiple seismic records with different source offsets and receiver gathers, then effective-mode computations are limited to algorithms that assume far-field plane Rayleigh wave propagation. Local search (e.g. linearized matrix inversion methods) or global search methods (e.g., Monte Carlo approaches such as simulated annealing, generic algorithms and neighborhood algorithm) are typically used to solve the inverse problem.

The maximum wavelength (λ_{max}) recovered from a surface wave data set is typically used to estimate depth of investigation although a sensitivity analysis of the V_S models would be a more robust means to estimate depth of investigation. For normally dispersive velocity profiles with a gradual increase in V_S with depth, maximum depth of investigation is on the order of $\lambda_{max}/2$ for both Rayleigh and Love wave dispersion data. Velocity profiles with an abrupt increase in V_S at depth, maximum depth of investigation is on the order of $\lambda_{max}/3$ for Rayleigh wave dispersion data but less than $\lambda_{max}/3$ for Love wave dispersion data. Depth of investigation can be highly variable for sites with complex velocity structure (e.g. high velocity layers).

As with all surface geophysical methods, inversion of surface wave dispersion data does not yield a unique V_S model and there are multiple possible solutions that may equally well fit the experimental data. Based on our experience at other sites, the shear wave velocity models (V_S and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods (Brown, 1998). The average velocity of the upper 30 m or 100 ft, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model. V_{S30} does not appear to suffer from the non-uniqueness inherent in V_S models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011). Therefore, V_{S30} is more accurately estimated from inversion of surface wave dispersion data than the resulting V_S models.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength (V_{R40}) in which case V_{S30} can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on statistical analysis of a large number of surface wave data sets from sites with control by velocities measured in nearby boreholes and has been further evaluated by Martin and Diehl, 2004, and Albarello and Gargani, 2010. Further investigation of this approach has revealed that V_{S30} is generally between V_{R40} and V_{R45} with V_{R40} often being most appropriate for shallow groundwater sites and V_{R45} for deep ground water sites. A detailed study of such an approach for Love wave dispersion data has not been conducted; however, preliminary analysis demonstrates that V_{S30} is generally between V_{L50} and V_{L55}. Although we do not recommend that these empirical V_{S30} estimates replace modeling of surface wave dispersion data, they do offer a means of cost effectively evaluating V_{S30} over a large area. V_{R40} or V_{L55} can also be used to quantify error in V_{S30} by evaluating the scatter in the dispersion data at these wavelengths.

3 FIELD PROCEDURES

The active- and passive-source surface wave sounding locations were established by **GEO***Vision* personnel and are shown in Figure 1 with surveyed locations presented in Table 1. Four types of surface wave data were acquired at the site: an active-source surface wave survey to characterize near-surface velocity structure, a small aperture microtremor array to characterize intermediate depth velocity structure, a large aperture microtremor array to characterize deep velocity structure, and HVSR measurements to estimate the fundamental site period.

Active surface wave data were acquired along Array 2 using the MASW technique. Passive surface wave data were acquired on two (2) arrays, a small aperture L-shaped array and large aperture circular array, using the array microtremor method. The small aperture L-shaped microtremor array (Array 1) consisted of a 48, 4.5 Hz geophones spaced 3 m apart and aligned along two orthogonal linear arrays as shown on Figure 1. The large aperture microtremor array (Array 3) consisted of three (3), eleven channel double-circle arrays with diameters of about 100 and 200, 300 and 400, and 600 and 800 m, respectively (Figure 1). HVSR measurements were made near the center of Array 3 (HVSR measurement location HV1) and at the southeast end of the parking lot (HVSR measurement locations HV2) as shown on Figure 1.

MASW equipment used during this investigation consisted of two Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, seismic cable, a 4 lb hammer, and 10 and 20 lb sledgehammers. A 240 lb accelerated weight drop (AWD) was also available but not utilized to minimize noise. MASW data were acquired along a linear array of 36 geophones spaced 2 m apart on October 24, 2018. Shot points were located between 2 and 30 m from the end geophone locations and at 12 m intervals in the interior of the array. The 4 lb hammer and/or 10 lb sledgehammer were used for the 2 m offset source location and interior source locations. The 20 lb sledgehammer was used for all off-end source locations. Data from the transient impacts (hammers) were typically averaged 10 times to improve the signal-to-noise ratio. All field data were saved to hard disk and documented on field data acquisition forms.

The small aperture microtremor array equipment consisted of two Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, and seismic cables. Array microtremor data were acquired along L-shaped Array 1 on October 24, 2018. The L-shaped array consisted of 48, 4.5 Hz geophones spaced 3 m apart with the linear legs of the array being 69 and 72 m long, respectively. Ambient noise measurements were made along this array for one hour at a 2 ms sample rate (120, 30 second records). All passive surface wave data were stored on a laptop computer for later processing. The field geometry and associated files names were documented in field data acquisition forms.

The large aperture microtremor array data were collected on October 25, 2018 along three, 11-sensor double circle arrays, as shown on Figure 1. These arrays consisted of 11, 1 Hz vertical geophones connected to Geometrics Atom wireless seismographs with a sensor at the center of the array and four to six sensors distributed around each of two circular arrays. The three double circular arrays had approximate diameters of 100 and 200, 300 and 400, and 600 and 800 m, respectively. Passive surface wave measurements were made for between 1 and 1.5 hrs on each array. Each sensor location was surveyed using a decimeter-accuracy GPS prior to data acquisition. Seismic data stored on the Atom seismographs were downloaded to a laptop computer at the end of the survey.

HVSR data were acquired at a two (2) locations (Figure 1) October 25, 2018 and November 17 to 18, 2018 using a Nanometrics Trillium Compact 120 second seismometer coupled to a Nanometrics Centaur data acquisition unit (referred to herein as Trillium). Over 1.5 hours of ambient vibration data were acquired at each measurement location at a 100 Hz sample rate. Microtremor data were stored in the Centaur data acquisition system and downloaded as miniseed format files at the end of data acquisition. The HVSR measurements were initially made at location HV1 on October 25, 2018. The HVSR data did not show a distinct peak and after modeling surface wave dispersion data, it became apparent that an HVSR peak predicted by the V_S models was not observed. Therefore, HVSR measurements were also made in the mornings of November 17 and 18, 2018 at locations HV1 and HV2.

4 DATA REDUCTION

The MASW data were reduced using the software Seismic Pro Surface V9.0 developed by Geogiga and multiple in-house scripts for various data extraction and formatting tasks, with all data reduction documented in a Microsoft Excel spreadsheet.

The following steps were used for data reduction:

- Input seismic records to be used for analysis into software package.
- Check and correct source and receiver geometry as necessary.
- Select offset range used for analysis (multiple offset ranges utilized for each seismic record as discussed below) and document in spreadsheet.
- Apply phase shift transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify, pick, save, and document dispersion curve.
- Change the receiver offset range and repeat process.
- Repeat process for all seismic records.
- Use in-house script to apply near-field criteria with maximum wavelength set equal to lesser of 40 m (source frequency limitation) or 1 times the source to midpoint of receiver array distance.
- Use in-house script to merge multiple dispersion curves extracted from the MASW data collected along each seismic line for a specific source type (different source locations, different receiver offset ranges, etc.).
- Edit dispersion data, as necessary (e.g. delete poor quality curves and outliers).
- Calculate a representative dispersion curve at equal log-frequency or log-wavelength spacing for the MASW dispersion data using a moving average, polynomial curve fitting routine.

This unique data reduction strategy, which can involve combination of over 100 dispersion curves for a 1D sounding, is designed for characterizing sites with complex velocity structure that do not yield surface wave dispersion data over a wide frequency range from a single source type or source location. The data reduction strategy ensures that the dispersion curve selected for modeling is representative of average conditions beneath the array and spans as broad a frequency/wavelength range as possible while considering near field effects.

The array microtremor data were reduced using the Seisimager software package developed by Oyo Corporation/Geometrics, Inc. and the following steps:

The processing sequence for implementation of the ESAC method in the SeisImager software package is as follows:

- Input all seismic records for a dataset into software.
- Load receiver geometry (x and y positions) for each channel in seismic record.
- Apply time-segmentation routine, as necessary, to break data file into multiple seismic records. Time segmentation not necessary for smaller arrays where data acquired as 30 s records. For the large array, data was divided into multiple approximate 80 s time windows for analysis.
- Calculate the SPAC coefficients for each seismic record and average.

- Optionally, combine SPAC coefficients from different arrays (e.g. multiple double circle arrays from large array).
- For each frequency calculate the RMS error between the SPAC coefficients and a Bessel function of the first kind and order zero over a user defined phase velocity range and velocity step.
- Plot an image of RMS error as a function for frequency (f) and phase velocity (v).
- Identify and pick the dispersion curve as the continuous trend on the f-v image with the lowest RMS error.
- Repeat process for all arrays and time blocks.
- Use in-house script to convert dispersion curves to appropriate format for editing.
- Edit dispersion data, as necessary, and use in-house script to combine all dispersion data after setting maximum wavelength to about 2 to 2.5 times the maximum receiver spacing (2 times maximum receiver spacing approximately equivalent to $k_{min}/2$ for a symmetrical array).
- Calculate a representative dispersion curve for the passive dispersion data from each array using a moving average polynomial curve fitting routine.

The representative dispersion curves from the active and passive surface wave data were combined and the moving average polynomial curve fitting routine in WinSASW V3 was used to generate a composite representative dispersion curve for modeling. During this process the active surface wave data and the small and large array passive surface wave data were given equal weights. An equal logarithm wavelength sample rate was used for the representative dispersion curve to reflect the gradual loss in model resolution with depth. For the application of global inversion routines, it is necessary to add uncertainty bounds to the dispersion curves; however, there is no standardized approach to quantify uncertainty for the wide range of data reduction strategies utilized. With the data reduction approach used during this investigation, the scatter in the dispersion data naturally reflects a combination of measurement error and the effects of lateral velocity variability beneath the array. To develop the uncertainty bounds at each frequency on the representative dispersion curve, an in-house script was used that calculates the square root of the mean of the difference between the representative and observed dispersion data over a frequency or wavelength bin defined as a percentage of the difference between adjacent points on the representative dispersion curve. For this investigation, we used a wavelength bin with width of 50% of the wavelength difference between adjacent points on the dispersion curve and doubled the resulting root-mean-square of the differences, which seemed to define the scatter in the dispersion data in an acceptable manner.

HVSR data were reduced using Geopsy Version 2.9.1 (http://www.geopsy.org) developed by Marc Wathelet, ISTerre, Grenoble, France with the help of many other researchers. Microtremor data recorded by the Trillium were exported to miniseed format. Data files were then loaded into the Geopsy software package, where data file columns containing the vertical and horizontal (north and east) components and the sample rate were specified. After applying a demean and 0.1 Hz high-pass filter, the H/V spectral ratio was calculated over the 0.1 to 15 Hz frequency range using a time window length of 200 s. Fourier amplitude spectra were calculated after applying a 5% cosine taper and smoothed by the Konno and Ohmachi filter with a smoothing coefficient value of 30. The vertical amplitude spectra were divided by the root-mean-square (RMS) of the horizontal amplitude spectra to calculate the HVSR for each time window and the average HVSR. Time windows containing clear transients (high amplitude near-field signals caused by nearby foot or vehicular traffic, etc.) or yielding poor quality results were then deleted and the computations repeated. The average HVSR peak frequency and its standard deviation from all time windows used for analysis is computed and presented along with the standard deviation of the HVSR amplitudes for all time windows.

5 DATA MODELING

Two surface wave modeling packages were used for data analysis including WinSASW V3 and Seisimager. Preliminary V_S models were first developed using the truncated fundamental mode assumption (2D analysis routine) in WinSASW. The theoretical HVSR peak for these models, calculated using the diffuse field assumption, did not match the observed HVSR peak and further inspection of the V_S models in Seisimager indicated that the first higher mode is expected to be dominant at low frequencies. Therefore, data modeling was completed using the effective mode modeling routine in Seisimager.

The final composite representative dispersion curve was loaded into the inverse modeling software package and data modeled using both the fundamental mode solution in WinSASW and the effective mode solution in Seisimager. During this process an initial velocity model was generated based on general characteristics of the dispersion curve and the inverse modeling routine utilized to adjust the layer V_S until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted, and the inversion process repeated until a V_S model was developed with low RMS error between the observed and calculated dispersion curves. Once an acceptable V_S model was developed, layer thicknesses were again adjusted and the inversion process repeated to develop an ensemble of V_S models with similar RMS error to quantify non-uniqueness. The assessment of non-uniqueness focused on the deeper two layers in the velocity models. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e. less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.78 to 2.20 g/cm^3 (111 to 137 lb/ft^3) were used in the profile for subsurface soils/rock depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible (±2%) affect on the estimated V_S from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity, V_P, for unsaturated sediments was estimated using a Poisson's ratio, ν, of 0.3 and the relationship:

$$V_P = V_S\,[(2(1-\nu))/(1-2\nu)]^{0.5}$$

Poisson's ratio has a larger affect than density on the estimated V_S from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity (V_R), V_S and ν:

$$V_R = V_S\,[(0.862 + 1.14\,\nu)/(1 + \nu)]$$

Using this relationship, it can be shown that V_S derived from V_R only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$V_S = 1.16 V_R \text{ for } \nu = 0$$
$$V_S = 1.05 V_R \text{ for } \nu = 0.5$$

The realistic range of the Poisson's ratio for typical unsaturated sediments is about 0.25 to 0.35. Over this range, V_S derived from modeling of Rayleigh wave dispersion data will vary by about 5%. An intermediate Poisson's ratio of 0.3 was selected for modeling to minimize any error associated with the assumed Poisson's ratio.

To reduce errors associated with expected high Poisson's ratio of saturated sediments, the saturated zone was anchored at a depth of 7 m (23 ft), based on inspection of seismic refraction first arrival data. V_P of the saturated zone was set to a minimum velocity of 1,450 m/s (4,757 ft/s) and allowed to gradually increase with depth with increase in V_S.

Theoretical HVSR response, based on the diffuse field assumption, was computed for the ensemble of V_S models developed during inversion of surface wave dispersion data using the open source software package *HV-Inv* Release 2.3., which is summarized in García-Jerez, et al., 2016. Computations were made assuming that the microtremor wavefield consists of both Rayleigh and Love waves.

6 INTERPRETATION AND RESULTS

The observed HVSR data for measurement stations HV1 and HV2, collected on November 17 and 18, 2018, are presented as Figure 2. HVSR station HV1 was located near the center of the large aperture microtremor array (Array 3) and HV2 was located in the southeastern portion of the parking lot. The dominant feature in the HVSR plots is a high amplitude peak at a frequency between 0.73 and 0.75 Hz. The shape of the HVSR plots are very similar with all plots having a low amplitude trough at about 1.5 Hz. The amplitude of the HVSR peaks are similar at both measurement locations. HVSR data collected at location HV1 on October 25 was similar at frequencies less than 0.6 Hz and greater than 1 Hz; however, the 0.75 Hz peak was not well defined, possibly due to meteorological conditions at the time of the measurement or an external noise source.

V_S models were developed from the surface wave dispersion data derived from a MASW array (Array 2), 48 channel (4.5 Hz geophone) L-shaped array (Array 1), and three (3) large 11 channel double circle arrays (Array 3). Vs models were developed using both the fundamental and effective mode forward solutions.

The fit of the calculated fundamental and effective mode dispersion curves to the experimental data collected at the site and the associated modeled V_S profiles for the surface wave sounding are presented as Figures 3 and 4, respectively. Multiple "equivalent" V_S models were developed to characterize non-uniqueness, particularly at depth. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The Rayleigh wave phase velocities from the various active and passive surface wave arrays are in excellent agreement in the regions of overlapping wavelength. The estimated depth of investigation for the combined active and passive surface wave sounding is about 350 m (between one-half and one-third the maximum Rayleigh wave wavelength).

HVSR response was calculated from the V_S models developed using both the fundamental and effective mode assumption using the diffuse wave assumption and is presented in Figure 5. All V_S models developed using the effective mode assumption have a similar HVSR peak frequency, which is in good agreement with that of the observed HVSR response. However, the peak frequency in the calculated HVSR response for the fundamental mode V_S models is more variable. The V_S model having the shallowest depth to the half-space has a calculated HVSR peak frequency that is in best agreement with that from the observed HVSR response. As the modeled half-space depth becomes greater the calculated HVSR peak frequency decreases quite significantly.

The V_S profile developed using the fundamental mode assumption and intermediate depth to the bottom two layers is provided in tabular form in both metric and Imperial units as Tables 2 and 3, respectively. The V_S profile developed using the effective mode assumption and intermediate depth to the bottom two layers is provided in tabular form in both metric and Imperial units as Tables 4 and 5, respectively. The V_S profile developed using the fundamental mode assumption and shallowest depth to the half space is provided in tabular form in both metric and Imperial units as Tables 6 and 7, respectively. The V_S profile developed using the effective mode assumption and shallowest depth to the half space is provided in tabular form in both metric and Imperial units as Tables 8 and 9, respectively. Other equivalent V_S models are provided in digital form. The primary difference between V_S models developed using the fundamental and effective mode assumptions is that the half space velocity is much higher in the fundamental mode V_S models.

We suggest that the V_S models developed using the effective mode assumption are most accurate and use the V_S model with intermediate depth to the lower two layers (Figure 4 and Tables 4 and 5) for purpose of site characterization. In this model, V_S gradually increases with depth from about 177 m/s (581 ft/s) near the surface to 421 m/s (1,380 ft/s) at a depth of about 75 m (246 ft). There is a sharp increase in modeled V_S to about 643 m/s (2,109 ft/s) at a depth of 105 m (345 ft) and again to 891 m/s (2,923 ft/s) at a depth of 200 m (656 ft).

The average shear wave velocity to a depth of 30 m (V_{S30}) is 237 m/s for the sample V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 780 ft/s. V_{S30} is between 236 and 238 m/s for the equivalent V_S models, although it should be noted that the evaluation of non-uniqueness focused on velocity structure at depths greater than 30 m. According to the NEHRP provisions of the Uniform Building Code, the site is classified as Site Class D, stiff soil.

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8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Prepared by

11/21/2018

Antony J. Martin
California Professional Geophysicist, P. Gp. 989
GEO*Vision* Geophysical Services

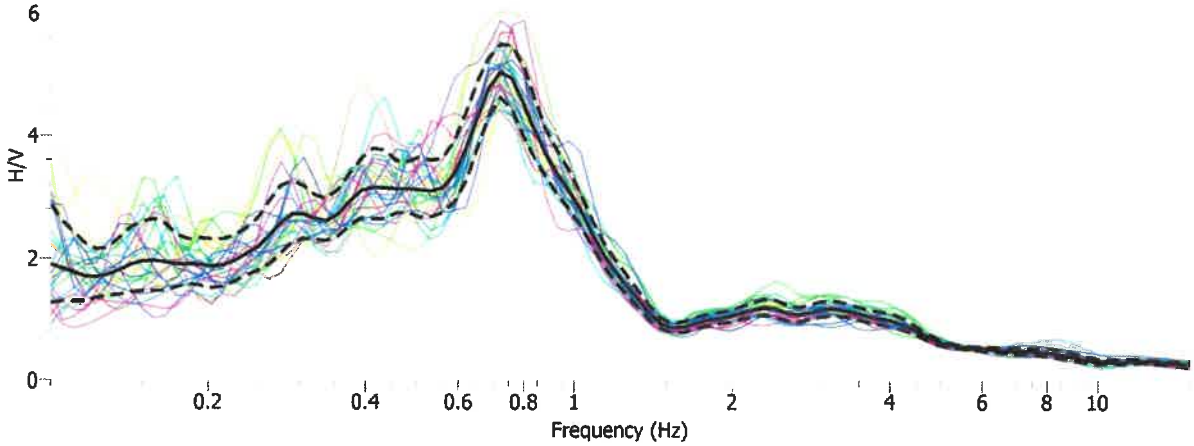
Date

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

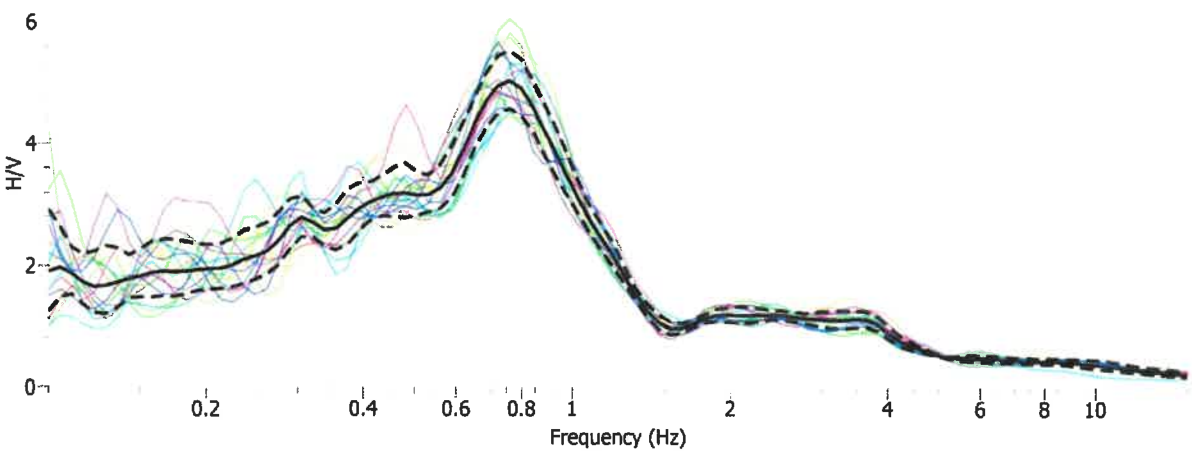
A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

FIGURES

47
S Almaden Blvd
37
49
38
Park Ave
30
28
Market St
22
20
Array 3
14
50
2
3
Array 1
W San Carlos St
27
HV1
1
4
Vioia Ave
Array 2
19
15
6
7
31
34
17
16
25
23
Balbach St
51
32
41
HV2
42
Woz Wy
S Almaden Ave
52
S Almaden Blvd
53
Junipero Serra Fwy



HVSR Station HV1



HVSR Station HV2

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Date: Nov 18, 2018

Drawn By: A MARTIN

Approved By: Antony J Martin

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FIGURE 2
OBSERVED H/V SPECTRAL RATIO
HVSR STATIONS HV1 AND HV2

NW CORNER OF SOUTH ALMADEN BLVD
AND BALBACH STREET
SAN JOSE, CALIFORNIA

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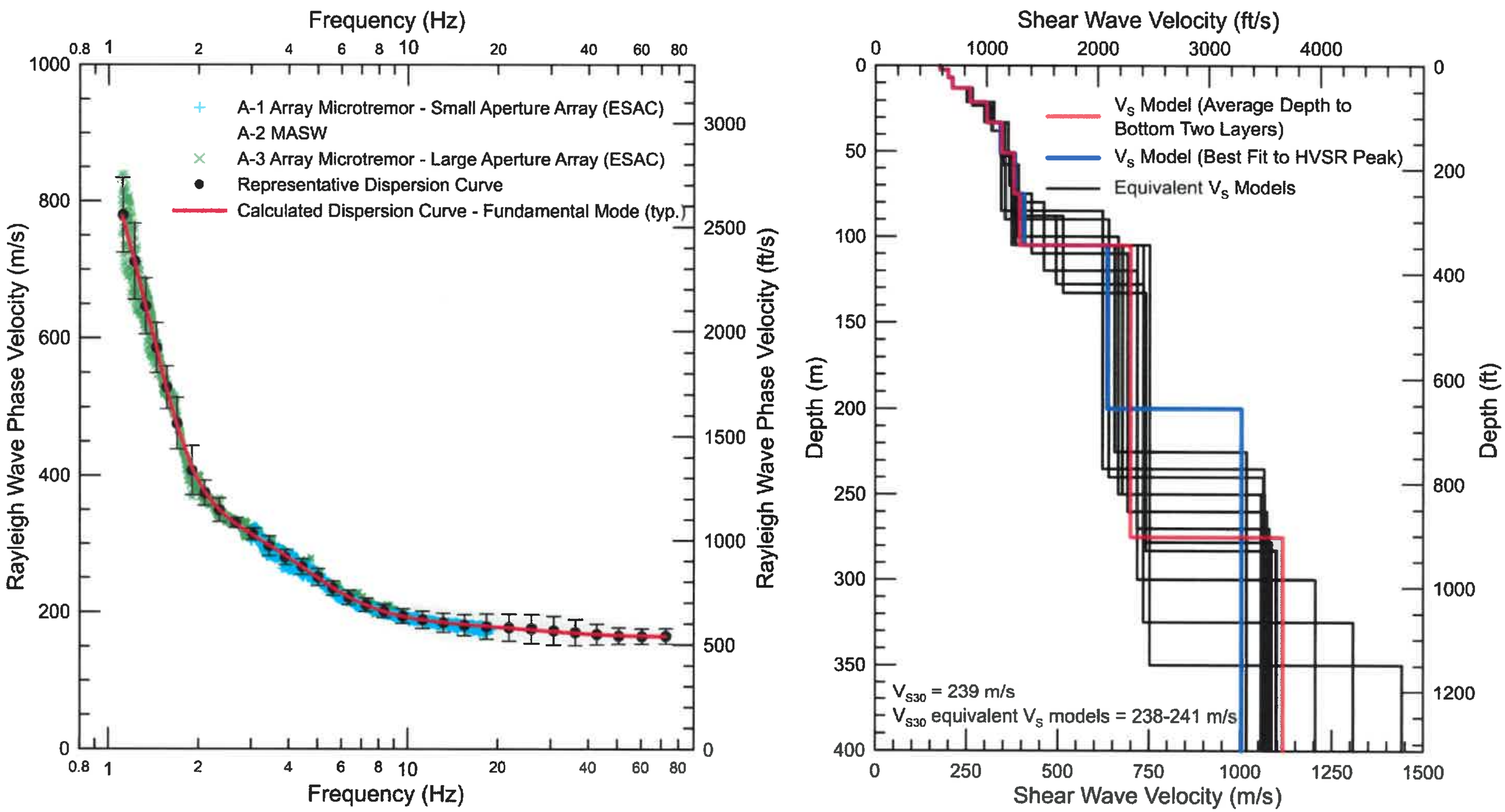

GEOVision geophysical services

Project No: 18450
Date: NOV 9, 2018
Drawn By: A MARTIN
Approved By:

FIGURE 3
SURFACE WAVE MODEL
FUNDAMENTAL MODE ASSUMPTION

NW CORNER OF SOUTH ALMADEN BLVD
AND BALBACH STREET
SAN JOSE, CALIFORNIA

PREPARED FOR
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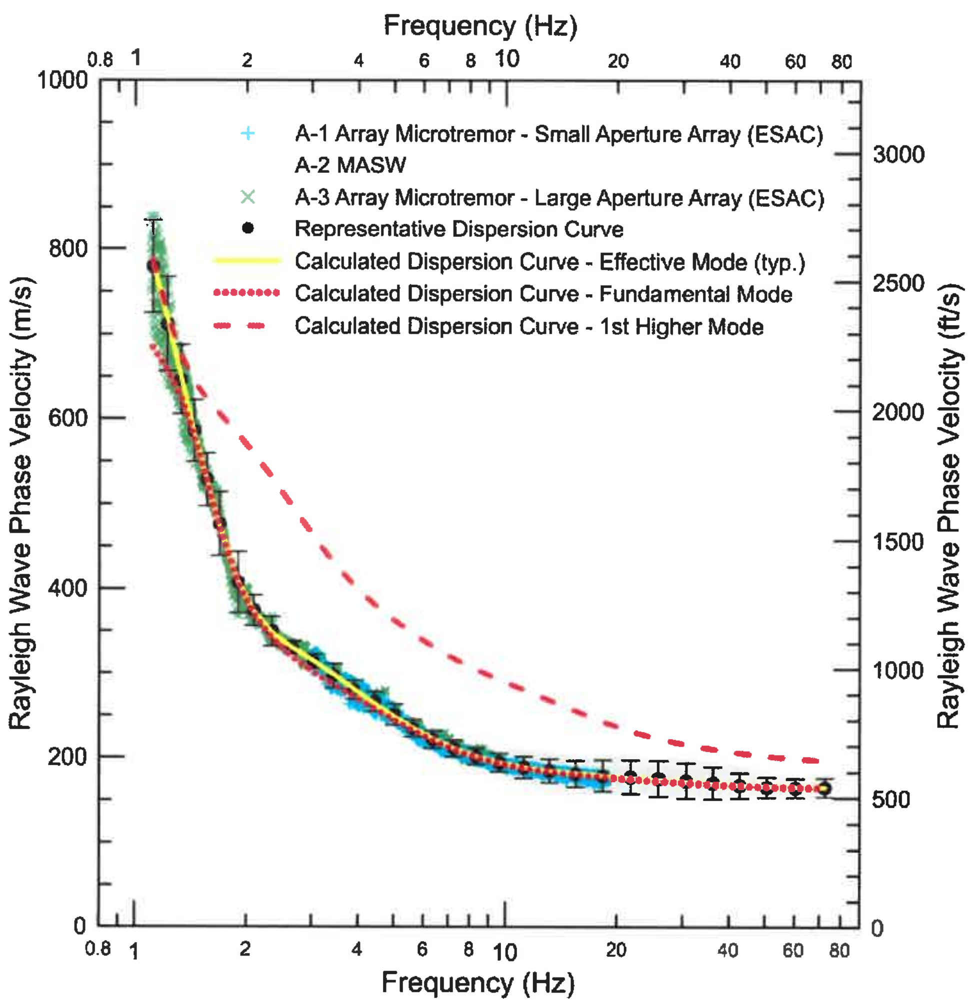

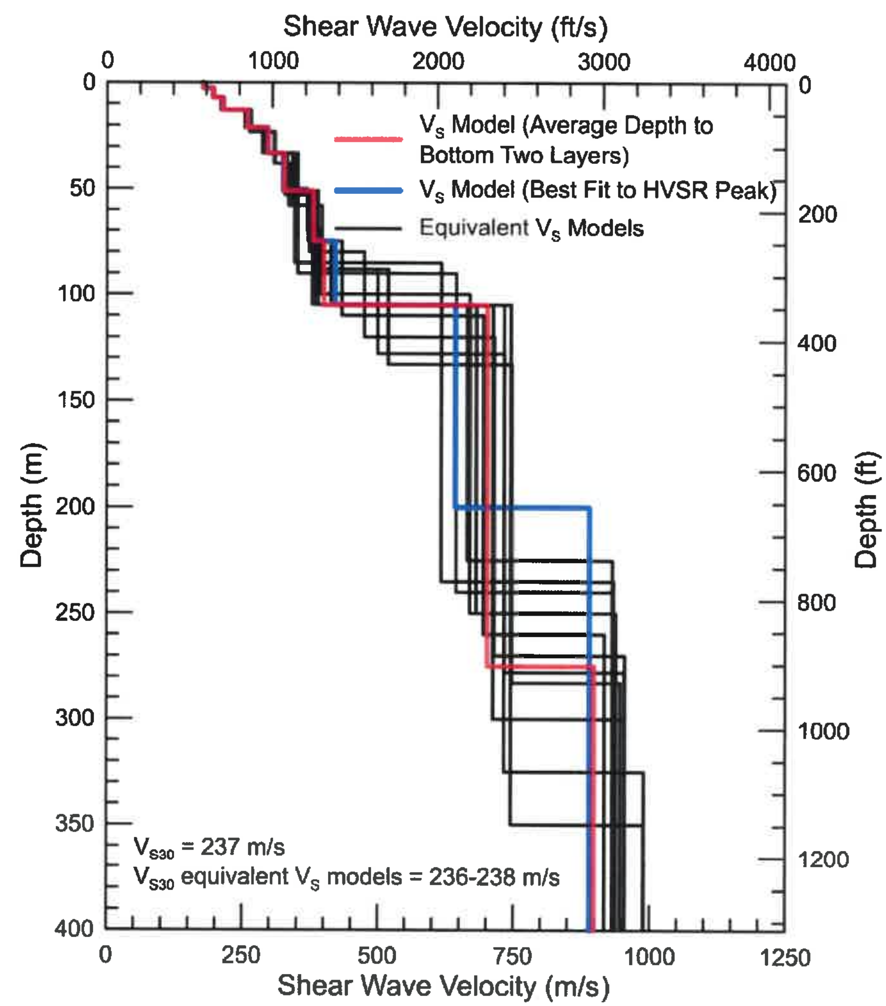

GEOVision geophysical services

Project No: 18450

Date: NOV 9, 2018

Drawn By: A MARTIN

Approved By: [signature]

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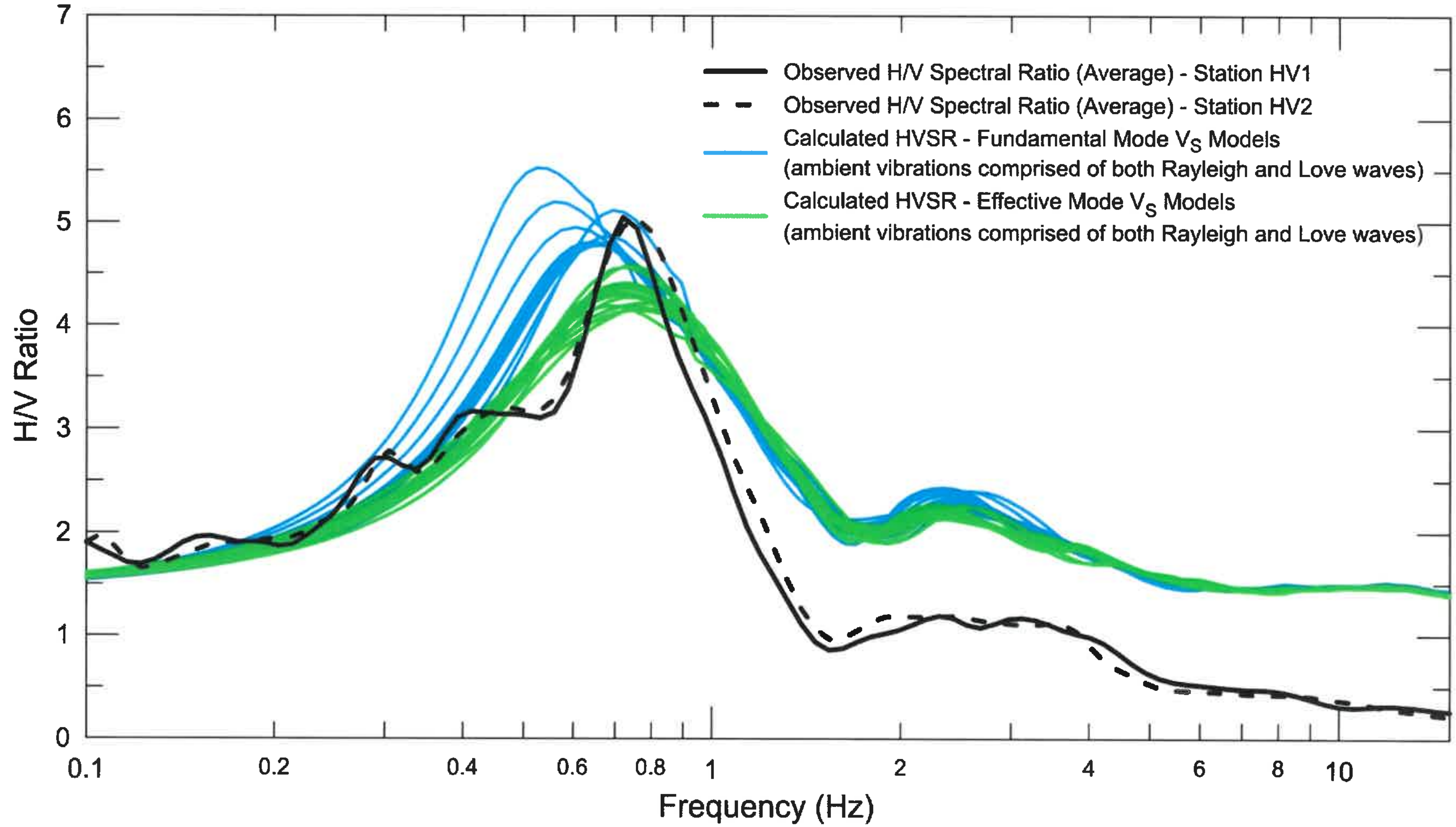
FIGURE 4
SURFACE WAVE MODEL
EFFECTIVE MODE ASSUMPTION

NW CORNER OF SOUTH ALMADEN BLVD
AND BALBACH STREET
SAN JOSE, CALIFORNIA

PREPARED FOR
ENGEO, INC.



GEOVision geophysical services

Project No: 18450

Date: Nov 18, 2018

Drawn By: A MARTIN

Approved By:

FIGURE 5
CALCULATED HVSR RESPONSE

NW CORNER OF SOUTH ALMADEN BLVD
AND BALBACH STREET
SAN JOSE, CALIFORNIA

PREPARED FOR
ENGEO, INC.

TABLES

Table 1 Location of Surface Wave Arrays

Description	Northing (US ft)	Easting (US ft)	Elevation (ft)
Passive L-Shaped Array 1 - SE End	1944926.5	6157363.7	88.9
Passive L-Shaped Array 1 - Corner	1945130.9	6157245.9	89.1
Passive L-Shaped Array 1 - NW End	1945017.3	6157053.1	95.8
MASW Array 2 - SE End	1944775.9	6157349.6	84.7
MASW Array 2 - Center	1944874.8	6157292.0	87.3
MASW Array 2 - NE End	1944974.1	6157234.0	85.9
HVSR Location 1	1945038.1	6157216.3	86.5
HVSR Location 2	1944446.4	6157521.5	89.8
Large Aperture Microtremor Array-1	1945038.1	6157216.3	86.5
Large Aperture Microtremor Array-2	1945202.9	6157196.1	88.3
Large Aperture Microtremor Array-3	1945187.8	6157285.3	88.6
Large Aperture Microtremor Array-4	1945022.5	6157379.5	87.5
Large Aperture Microtremor Array-6	1944918.2	6157328.3	87.3
Large Aperture Microtremor Array-7	1944875.1	6157220.9	87.9
Large Aperture Microtremor Array-14	1945351.0	6157124.5	87.4
Large Aperture Microtremor Array-15	1944909.0	6157515.8	88.5
Large Aperture Microtremor Array-16	1944798.3	6157444.6	87.8
Large Aperture Microtremor Array-17	1944721.6	6157302.2	87.1
Large Aperture Microtremor Array-19	1944870.4	6156934.0	85.9
Large Aperture Microtremor Array-20	1945495.2	6157039.0	81.1
Large Aperture Microtremor Array-22	1945510.4	6157373.9	86.9
Large Aperture Microtremor Array-23	1944653.6	6157528.0	88.1
Large Aperture Microtremor Array-25	1944664.4	6156895.7	91.9
Large Aperture Microtremor Array-27	1945029.2	6156724.2	87.9
Large Aperture Microtremor Array-28	1945636.7	6156938.4	84.9
Large Aperture Microtremor Array-30	1945666.9	6157404.4	93.6
Large Aperture Microtremor Array-31	1944835.4	6157828.0	84.3
Large Aperture Microtremor Array-32	1944512.2	6157610.3	88.4
Large Aperture Microtremor Array-34	1944711.8	6156670.3	88.9
Large Aperture Microtremor Array-37	1945999.4	6157000.5	85.5
Large Aperture Microtremor Array-38	1945854.3	6157765.8	89.1
Large Aperture Microtremor Array-41	1944489.5	6158024.9	86.5
Large Aperture Microtremor Array-42	1944223.3	6157768.1	88.6
Large Aperture Microtremor Array-47	1946244.8	6156701.3	83.2
Large Aperture Microtremor Array-49	1946029.0	6158068.0	88.7
Large Aperture Microtremor Array-50	1945357.1	6158485.6	91.1
Large Aperture Microtremor Array-51	1944617.9	6158455.8	90.7
Large Aperture Microtremor Array-52	1944183.3	6158209.2	90.2
Large Aperture Microtremor Array-53	1943933.9	6157924.1	90.8

Notes. 1. Survey data acquired with Spectra Precision SP60 with Centerpoint RTX.
2. California State Plane Zone 3 (0403), NAD83 (Conus), US Survey feet.

Table 2 Vs Model, Intermediate Depth to Bottom Two Layers, Fundamental Mode Solution (metric units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	2.5	177	332	0.300	1.78
2.5	4.5	199	373	0.300	1.82
7	6	211	1450	0.489	1.85
13	8	260	1500	0.485	1.90
21	12	306	1550	0.480	1.93
33	18	347	1600	0.475	1.95
51	24	379	1650	0.472	1.96
75	30	395	1700	0.472	1.98
105	170	700	2050	0.434	2.10
275	>75	1116	2323	0.350	2.20

Table 3 Vs Model, Intermediate Depth to Bottom Two Layers, Fundamental Mode Solution (Imperial units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	8.2	581	1088	0.300	111
8.2	14.8	654	1224	0.300	114
23.0	19.7	692	4757	0.489	115
42.7	26.2	852	4921	0.485	119
68.9	39.4	1002	5085	0.480	120
108.3	59.1	1137	5249	0.475	122
167.3	78.7	1243	5413	0.472	122
246.1	98.4	1295	5577	0.472	124
344.5	557.7	2295	6726	0.434	131
902.2	>246.1	3661	7622	0.350	137

Table 4 Vs Model, Intermediate Depth to Bottom Two Layers, Effective Mode Solution (metric units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	2.5	177	332	0.300	1.78
2.5	4.5	196	365	0.299	1.82
7	6	213	1467	0.489	1.85
13	8	259	1524	0.485	1.90
21	12	297	1571	0.481	1.93
33	18	327	1608	0.478	1.95
51	24	380	1675	0.473	1.96
75	30	400	1700	0.471	1.98
105	170	702	2078	0.436	2.10
275	>75	899	2325	0.412	2.20

Table 5 Vs Model, Intermediate Depth to Bottom Two Layers, Effective Mode Solution (Imperial units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	8.2	582	1088	0.300	111
8.2	14.8	641	1198	0.299	114
23.0	19.7	699	4812	0.489	115
42.7	26.2	850	4999	0.485	119
68.9	39.4	974	5154	0.481	120
108.3	59.1	1072	5277	0.478	122
167.3	78.7	1248	5496	0.473	122
246.1	98.4	1312	5577	0.471	124
344.5	557.7	2303	6816	0.436	131
902.2	>246.1	2951	7628	0.412	137

Table 6 Vs Model, Shallowest Depth to Half Space, Fundamental Mode Solution (metric units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	2.5	177	332	0.300	1.78
2.5	4.5	199	373	0.300	1.82
7	6	211	1450	0.489	1.85
13	8	260	1500	0.485	1.90
21	12	305	1550	0.480	1.93
33	18	343	1600	0.476	1.95
51	24	381	1650	0.472	1.96
75	30	405	1700	0.470	1.98
105	95	636	2050	0.447	2.10
200	>150	1004	2090	0.350	2.20

Table 7 Vs Model, Shallowest Depth to Half Space, Fundamental Mode Solution (Imperial units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	8.2	581	1088	0.300	111
8.2	14.8	654	1224	0.300	114
23.0	19.7	692	4757	0.489	115
42.7	26.2	852	4921	0.485	119
68.9	39.4	1001	5085	0.480	120
108.3	59.1	1126	5249	0.476	122
167.3	78.7	1250	5413	0.472	122
246.1	98.4	1330	5577	0.470	124
344.5	311.7	2086	6726	0.447	131
656.2	>492.1	3293	6856	0.350	137

Table 8 Vs Model, Shallowest Depth to Half Space, Effective Mode Solution (metric units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	2.5	177	331	0.300	1.78
2.5	4.5	195	366	0.301	1.82
7	6	212	1466	0.489	1.85
13	8	260	1525	0.485	1.90
21	12	297	1571	0.482	1.93
33	18	325	1606	0.479	1.95
51	24	379	1673	0.473	1.96
75	30	421	1726	0.468	1.98
105	95	643	2003	0.443	2.10
200	>150	891	2313	0.413	2.20

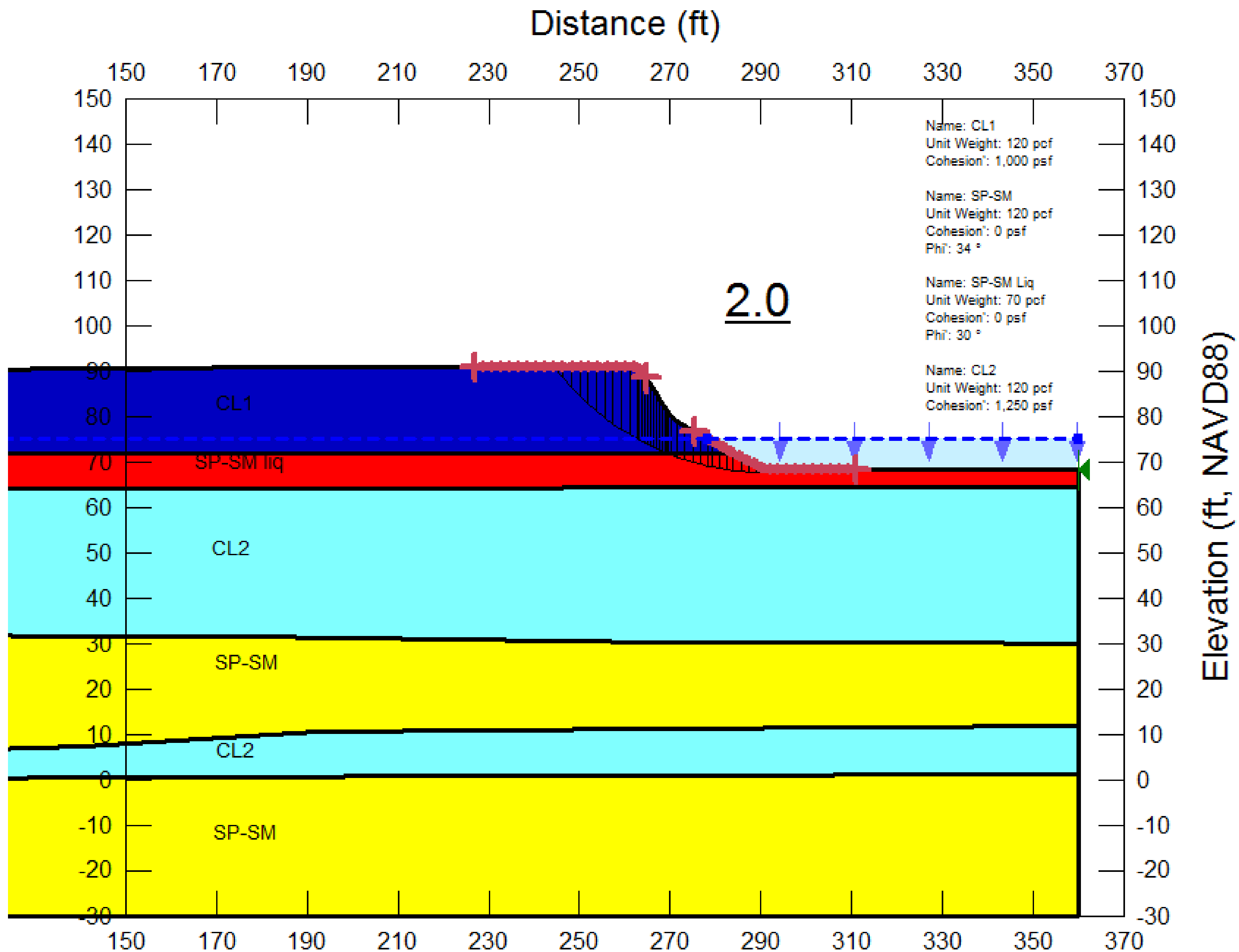
Table 9 Vs Model, Shallowest Depth to Half Space, Effective Mode Solution (Imperial units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	8.2	581	1086	0.300	111
8.2	14.8	640	1199	0.301	114
23.0	19.7	696	4808	0.489	115
42.7	26.2	854	5003	0.485	119
68.9	39.4	973	5153	0.482	120
108.3	59.1	1065	5269	0.479	122
167.3	78.7	1243	5490	0.473	122
246.1	98.4	1380	5663	0.468	124
344.5	311.7	2109	6572	0.443	131
656.2	>492.1	2923	7589	0.413	137

APPENDIX E

STATIC CONSOLIDATION SETTLEMENT ANALYSIS

POST EARTHQUAKE (CIRCULAR)



ENGEO Expect Excellence

SEISMIC SLOPE STABILITY ANALYSIS
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000		FIGURE NO.
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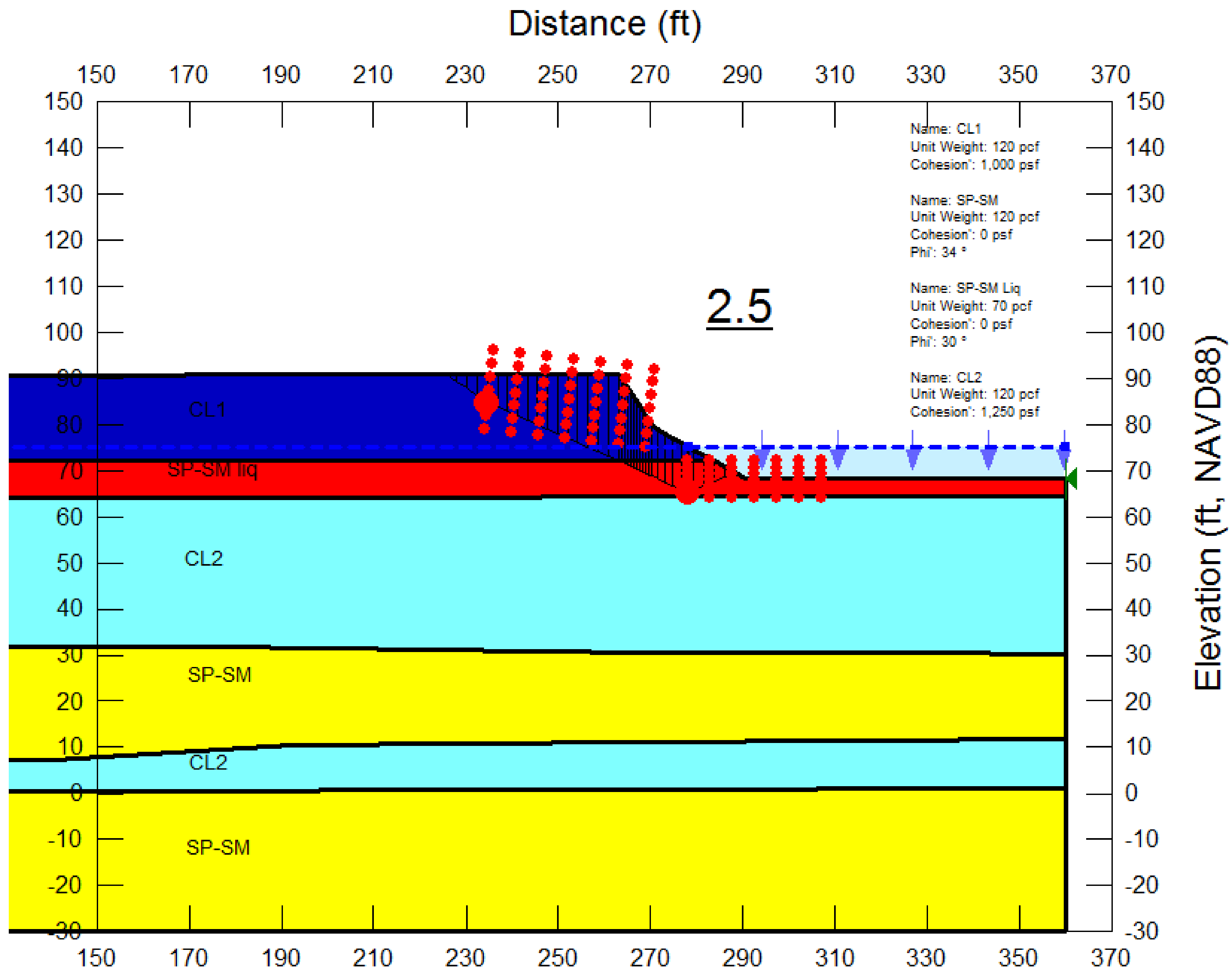
POST EARTHQUAKE (NON-CIRCULAR)



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SEISMIC SLOPE STABILITY ANALYSIS
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000		FIGURE NO.
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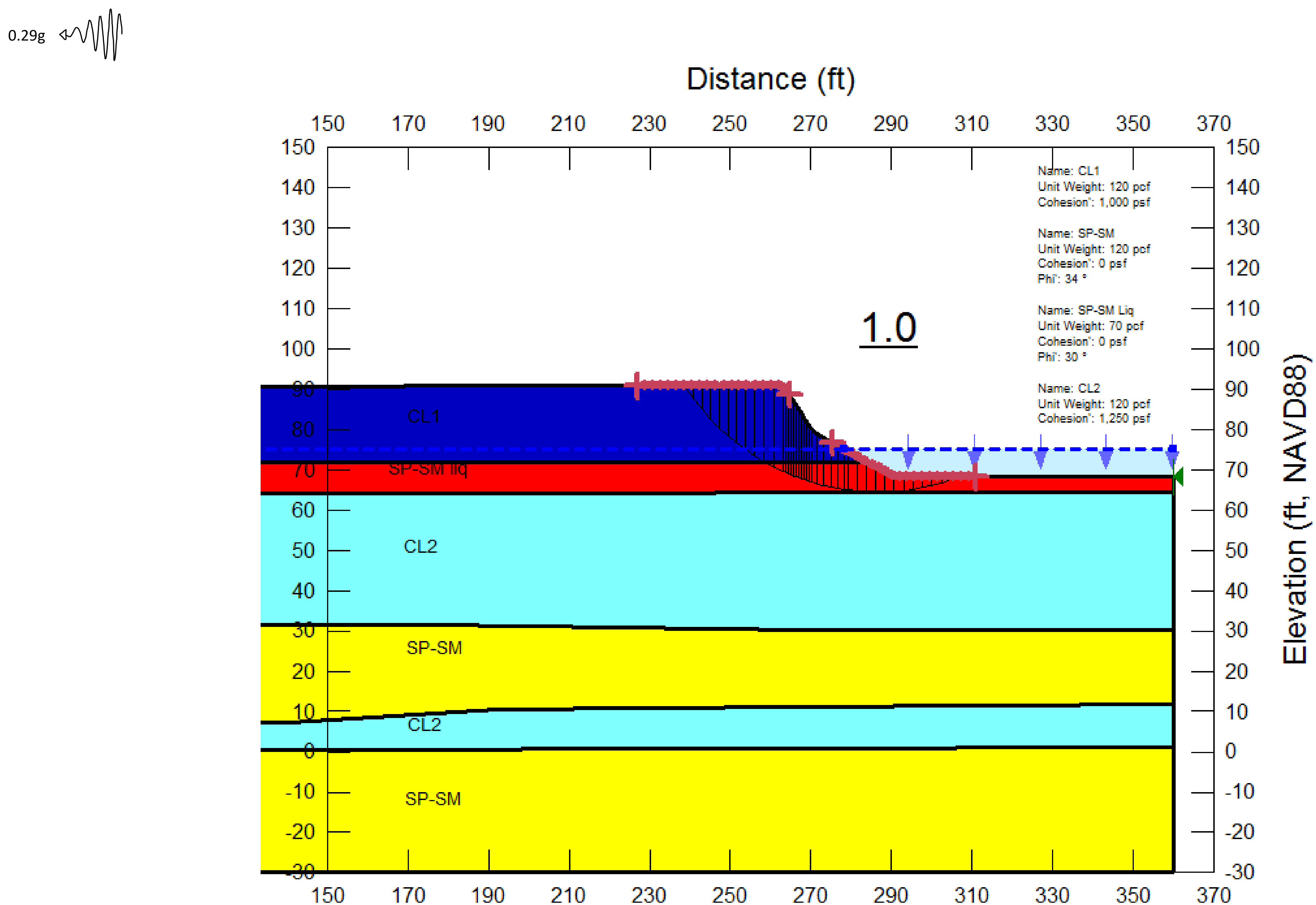

ENGEO Expect Excellence

SEISMIC SLOPE STABILITY ANALYSIS
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000		FIGURE NO.
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DRAWN BY: TS/AL	CHECKED BY: CC	

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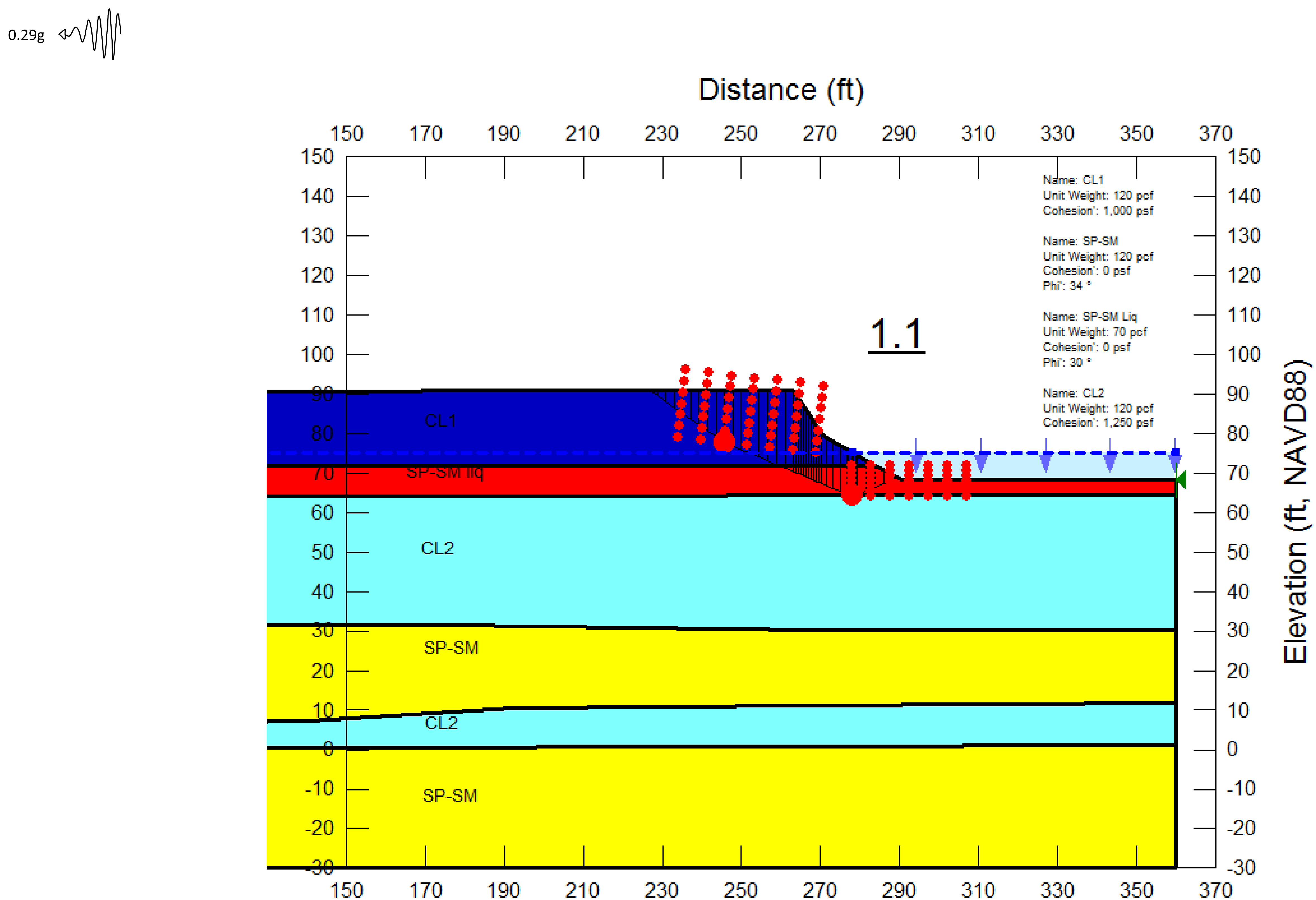
PSUEDO-STATIC (NON-CIRCULAR)



ENGEO Expect Excellence	SEISMIC SLOPE STABILITY ANALYSIS ALMADEN OFFICE COMPLEX SAN JOSE, CALIFORNIA	PROJECT NO.: 15540.000.000	FIGURE NO.
		SCALE: NO SCALE	APP-E
		DRAWN BY: TS AL / CHECKED BY: CC	

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APPENDIX F

LIQUEFACTION AND LATERAL SPREADING ANALYSIS

ENGEO Incorporated
6399 San Ignacio Ave, Suite 150
San Jose, CA 95119

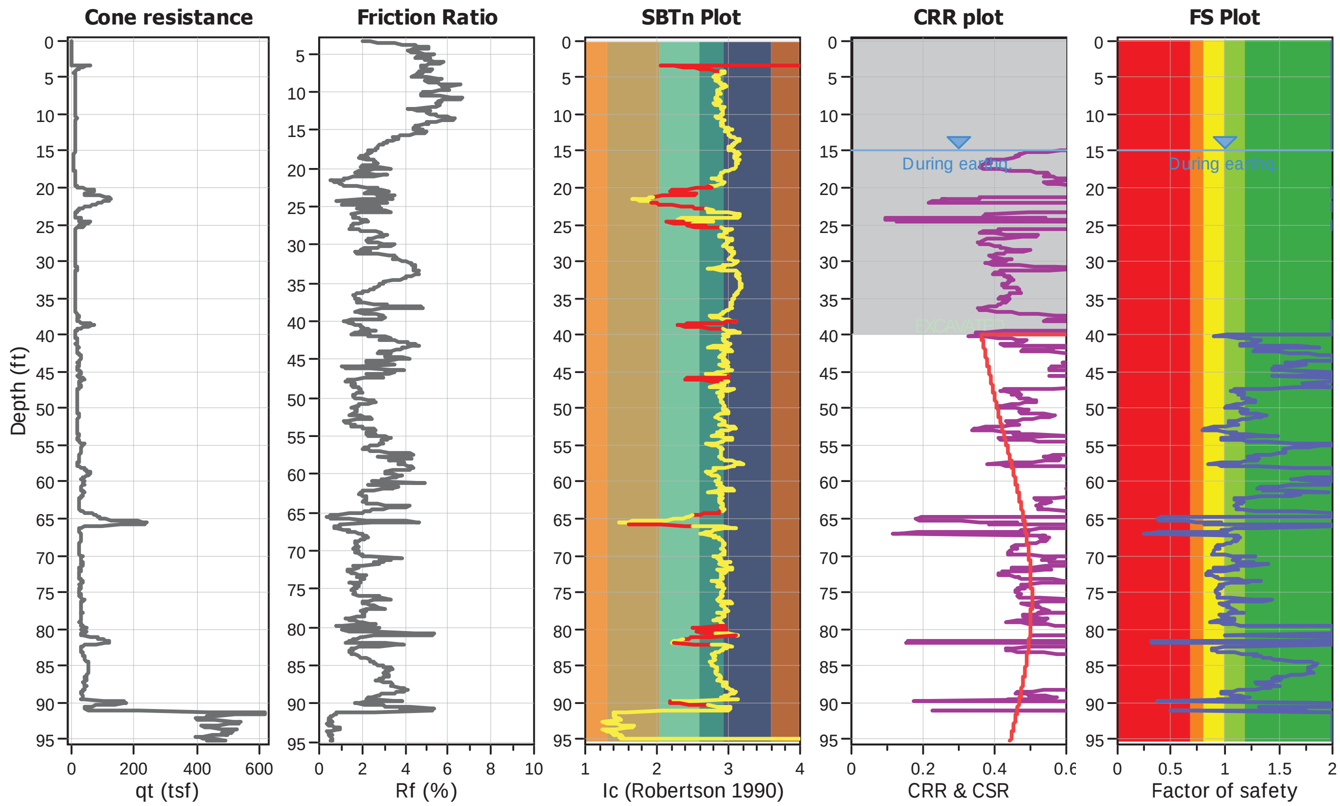
LIQUEFACTION ANALYSIS REPORT

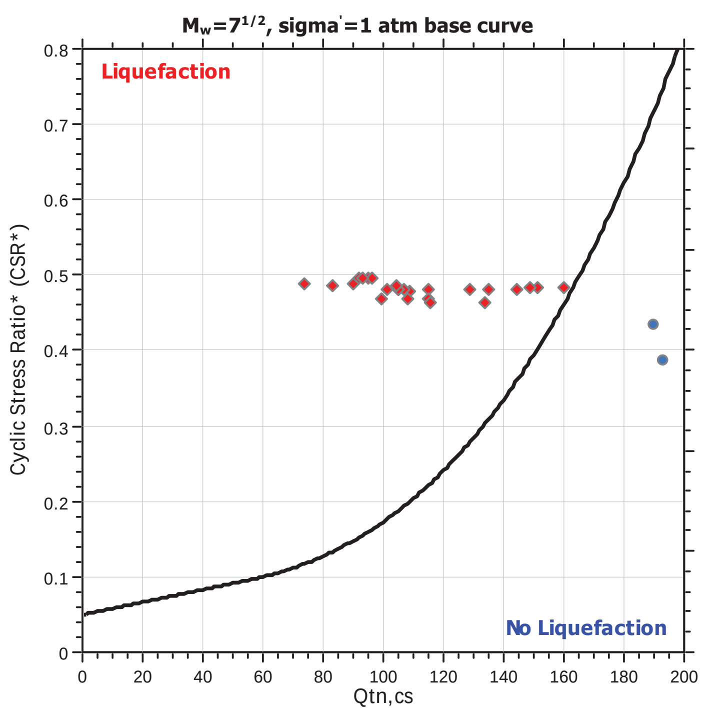
Project title : Almaden Office Complex **Location : San Jose, CA**

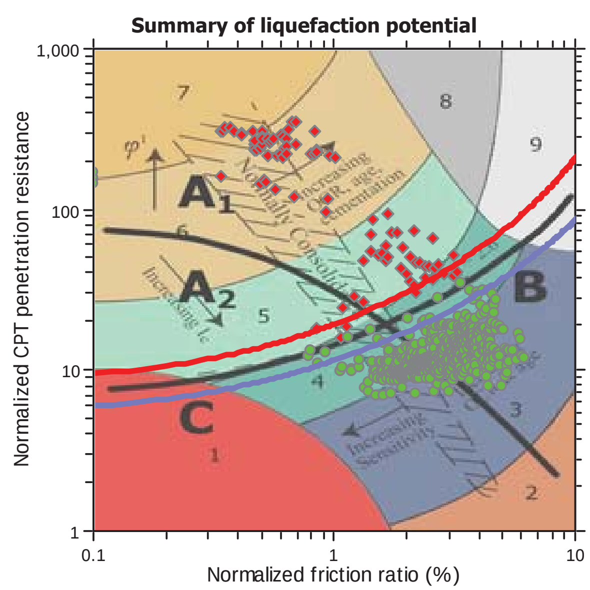
CPT file : 1-SCPT1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	40.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Footing load:	2.00 tsf	Limit depth:	N/A
Earthquake magnitude M_w:	7.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		

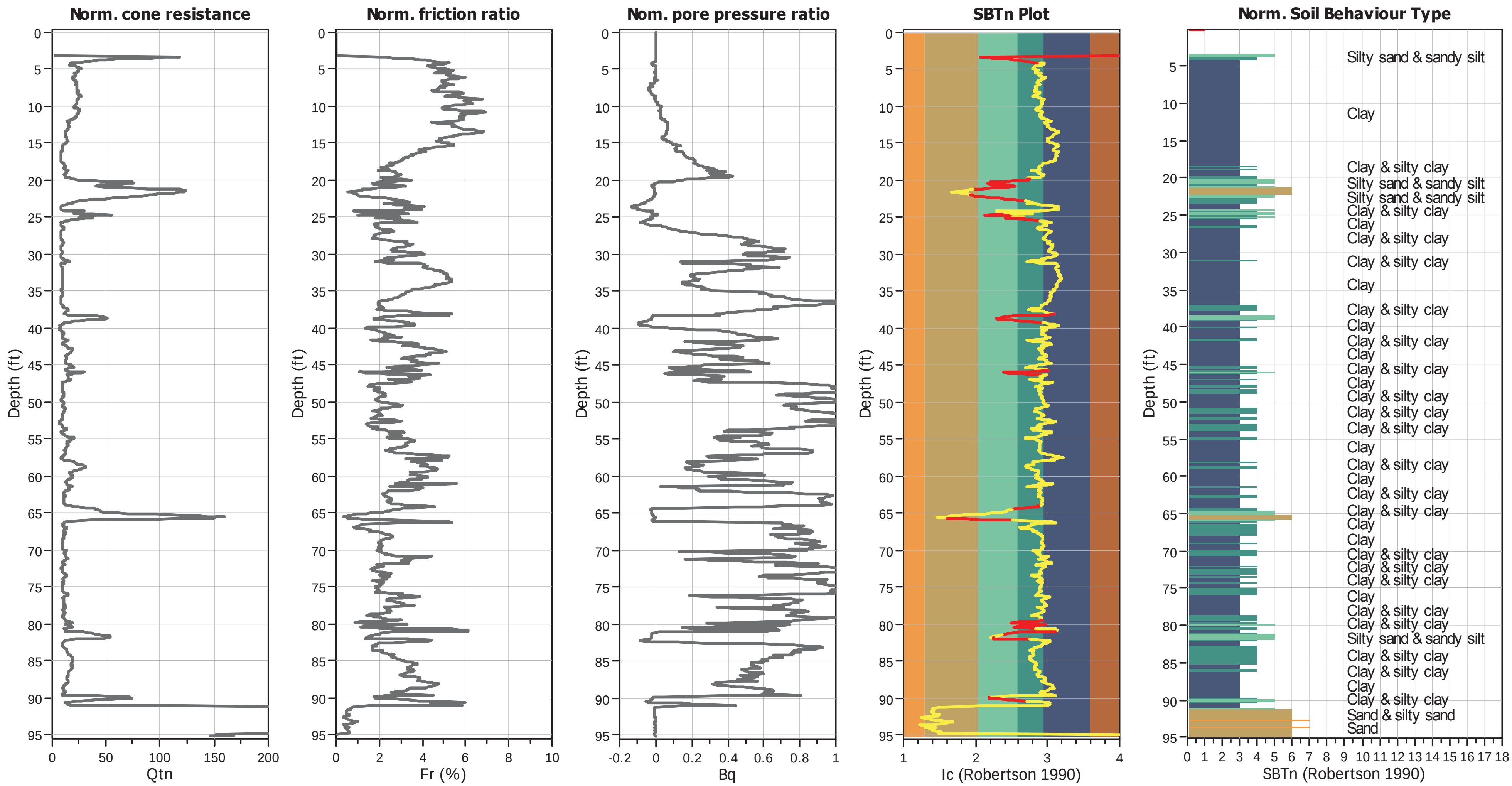






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



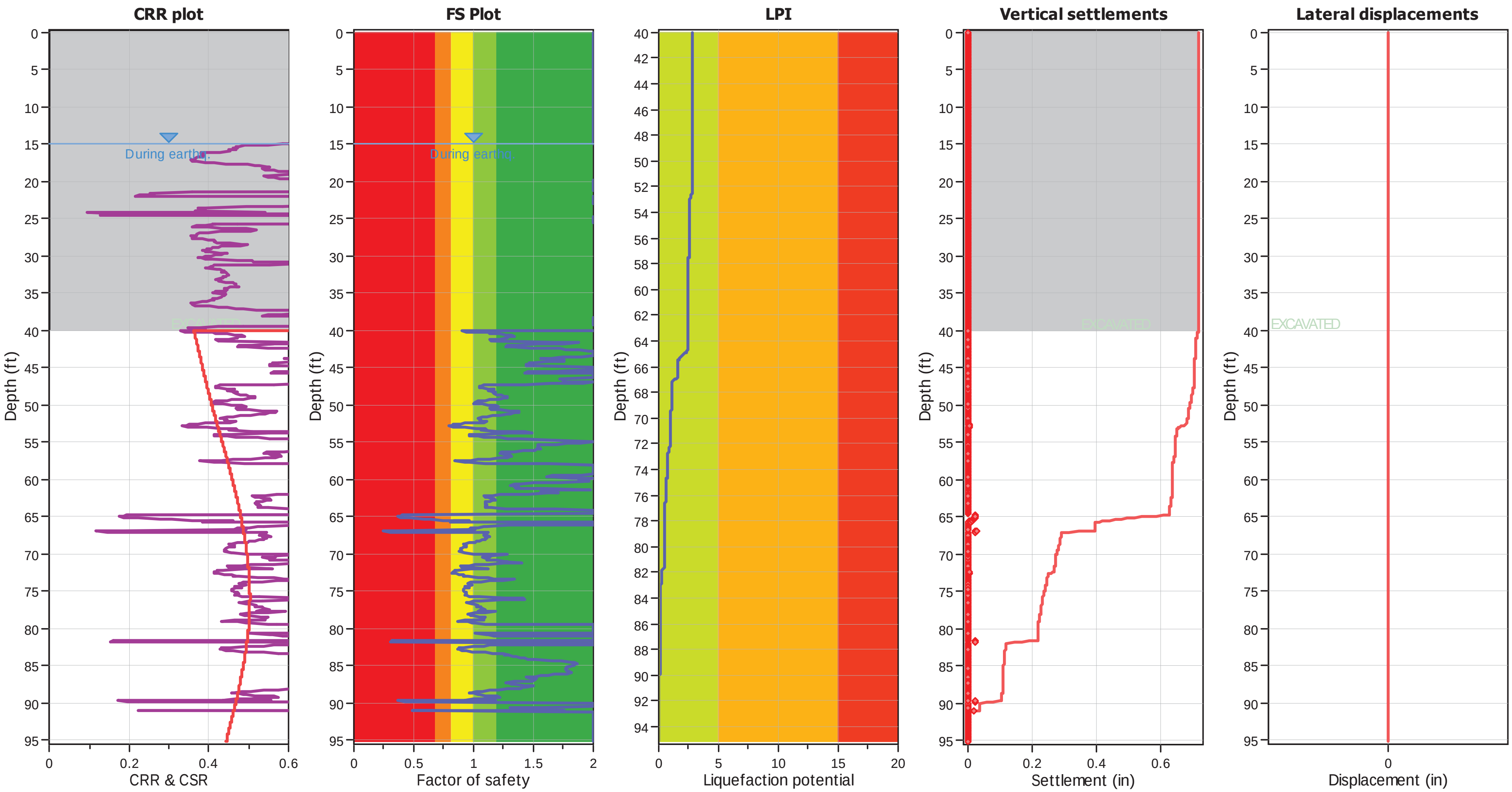
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Ave, Suite 150
San Jose, CA 95119

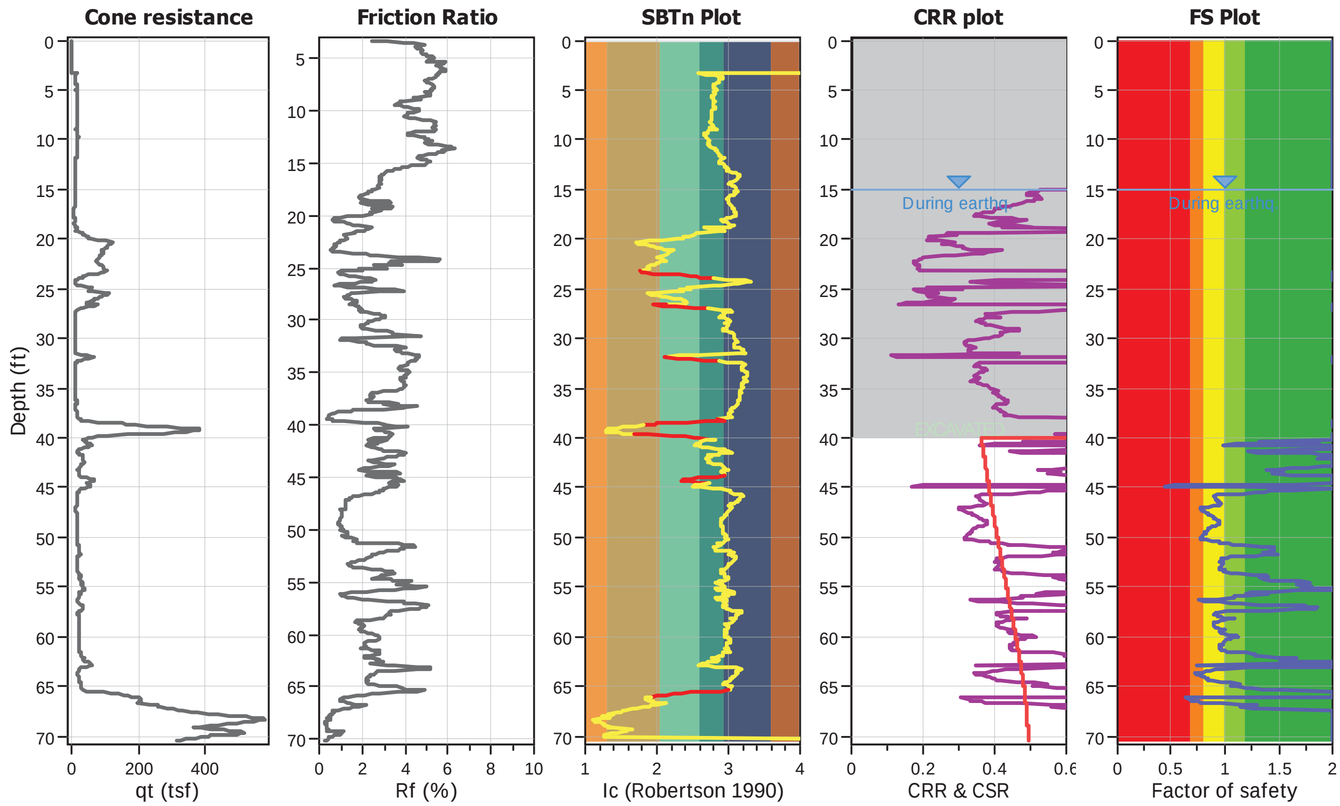
LIQUEFACTION ANALYSIS REPORT

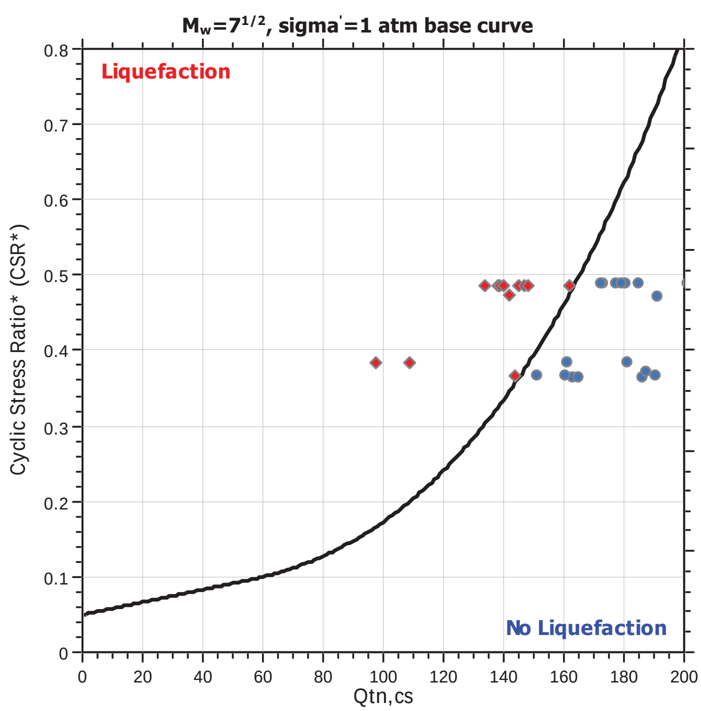
Project title : Almaden Office Complex **Location : San Jose, CA**

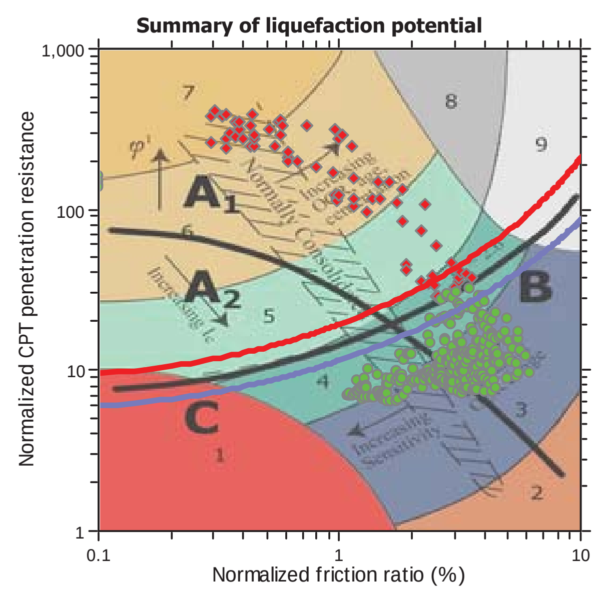
CPT file : 1-SCPT2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	40.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Footing load:	2.00 tsf	Limit depth:	N/A
Earthquake magnitude M_w:	7.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		

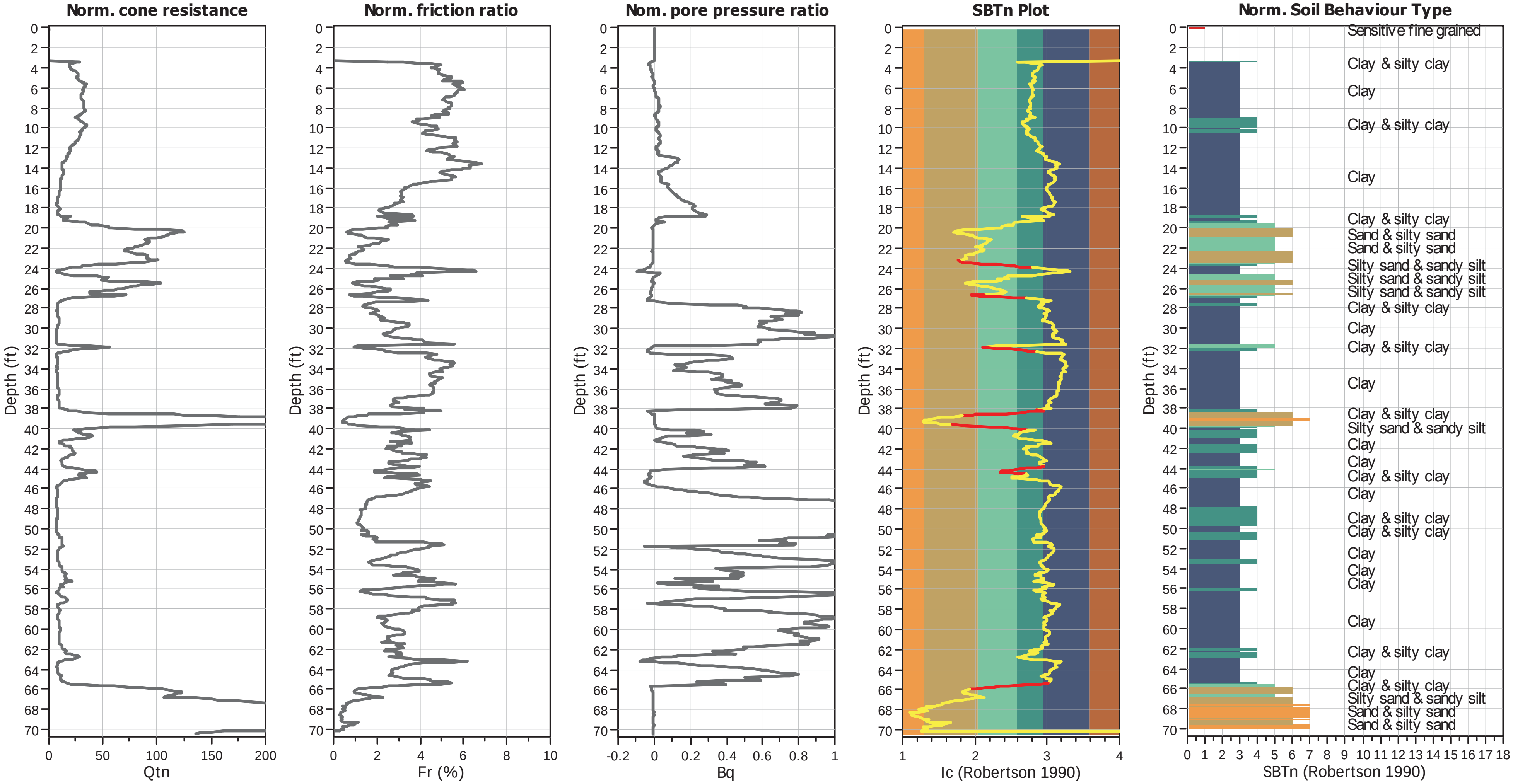






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



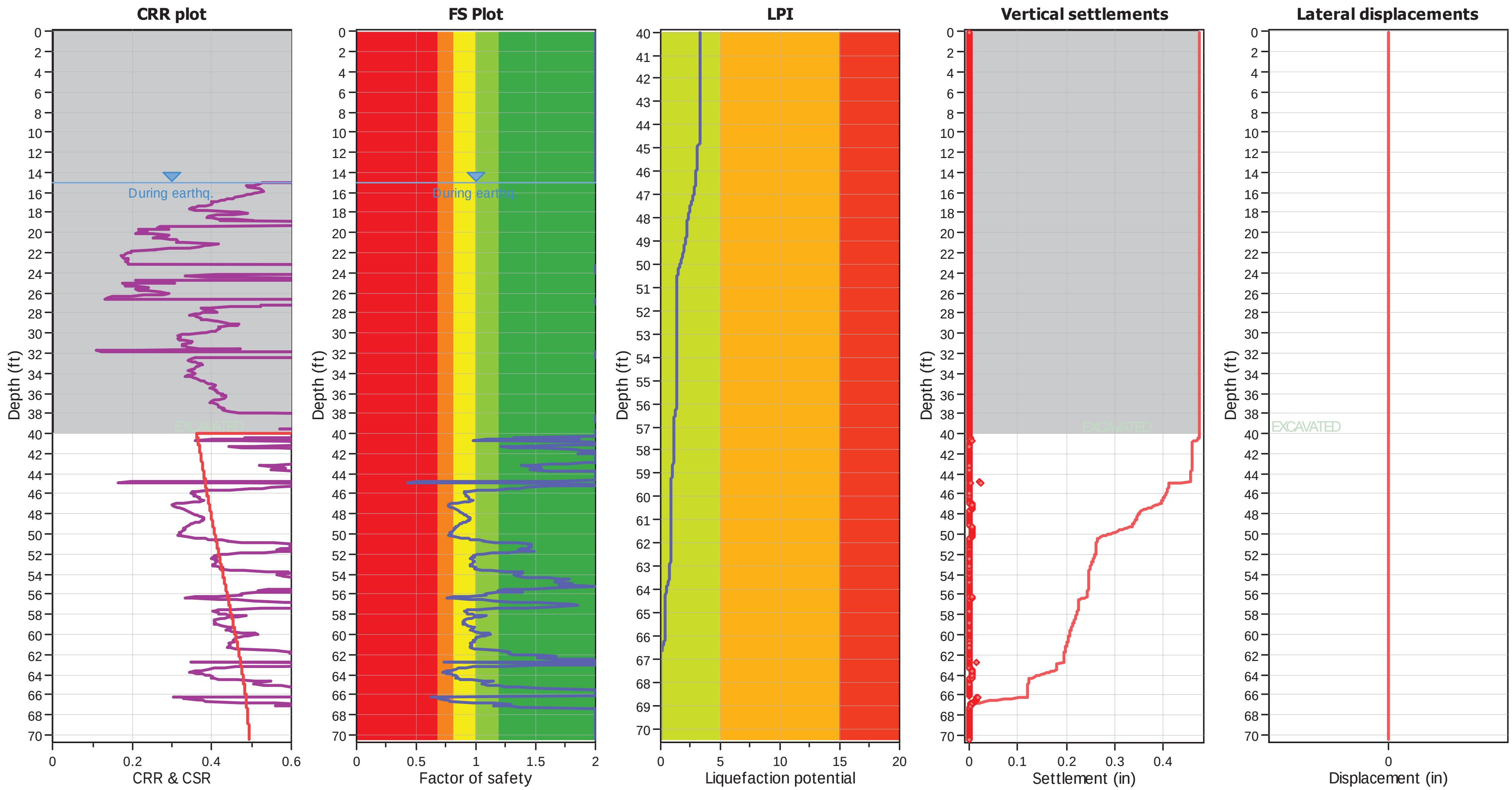
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Ave, Suite 150
San Jose, CA 95119

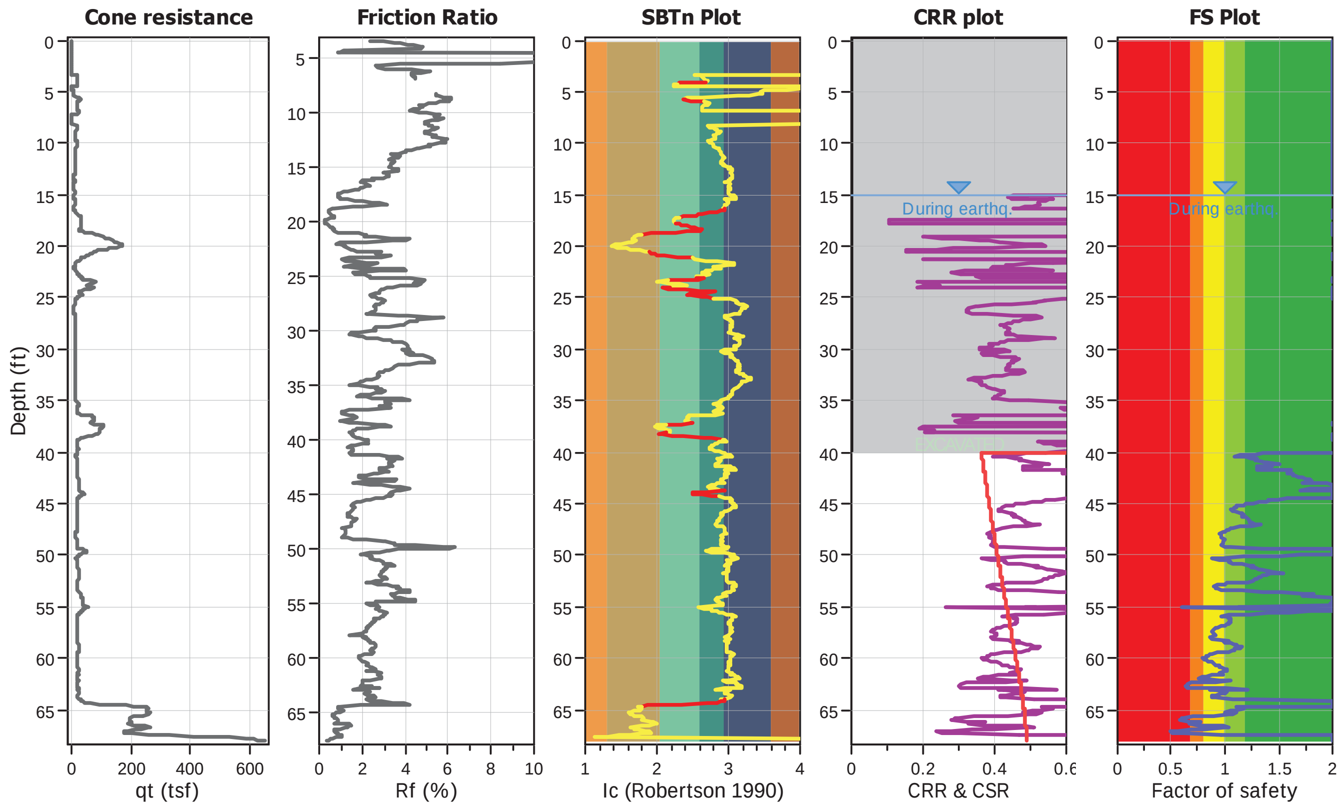
LIQUEFACTION ANALYSIS REPORT

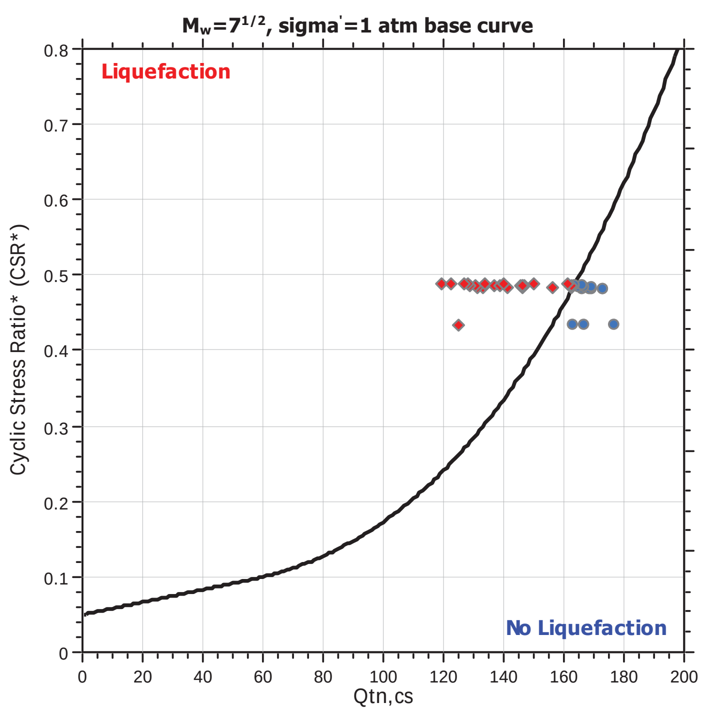
Project title : Almaden Office Complex **Location : San Jose, CA**

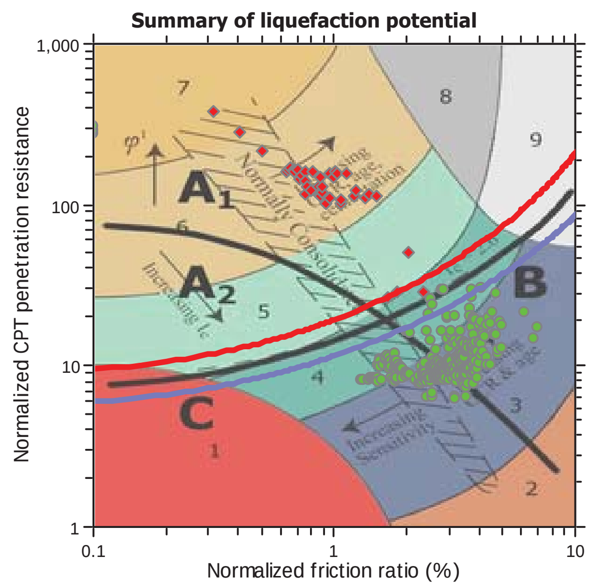
CPT file : 1-SCPT3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	40.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Footing load:	2.00 tsf	Limit depth:	N/A
Earthquake magnitude M_w:	7.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		

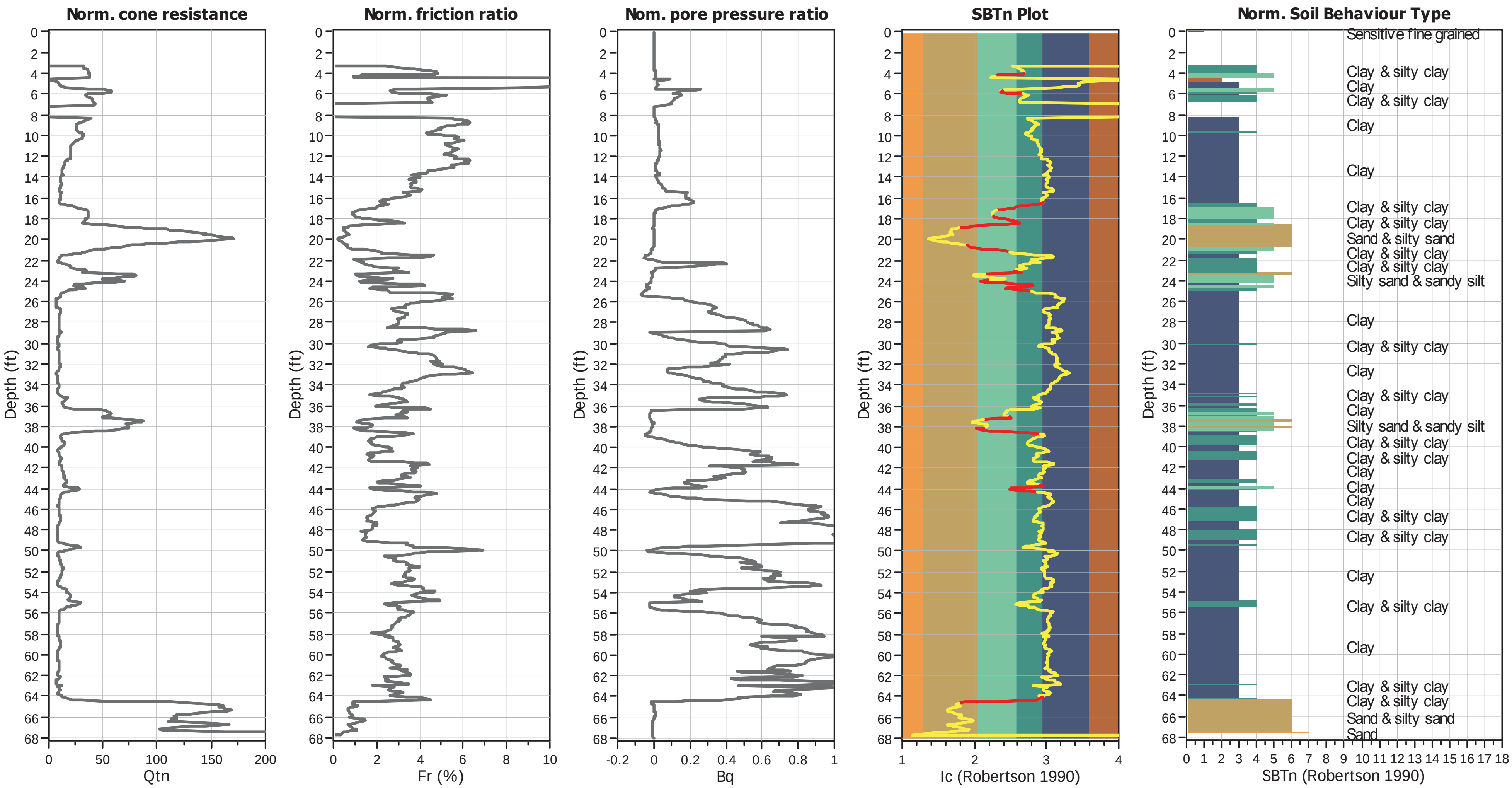






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



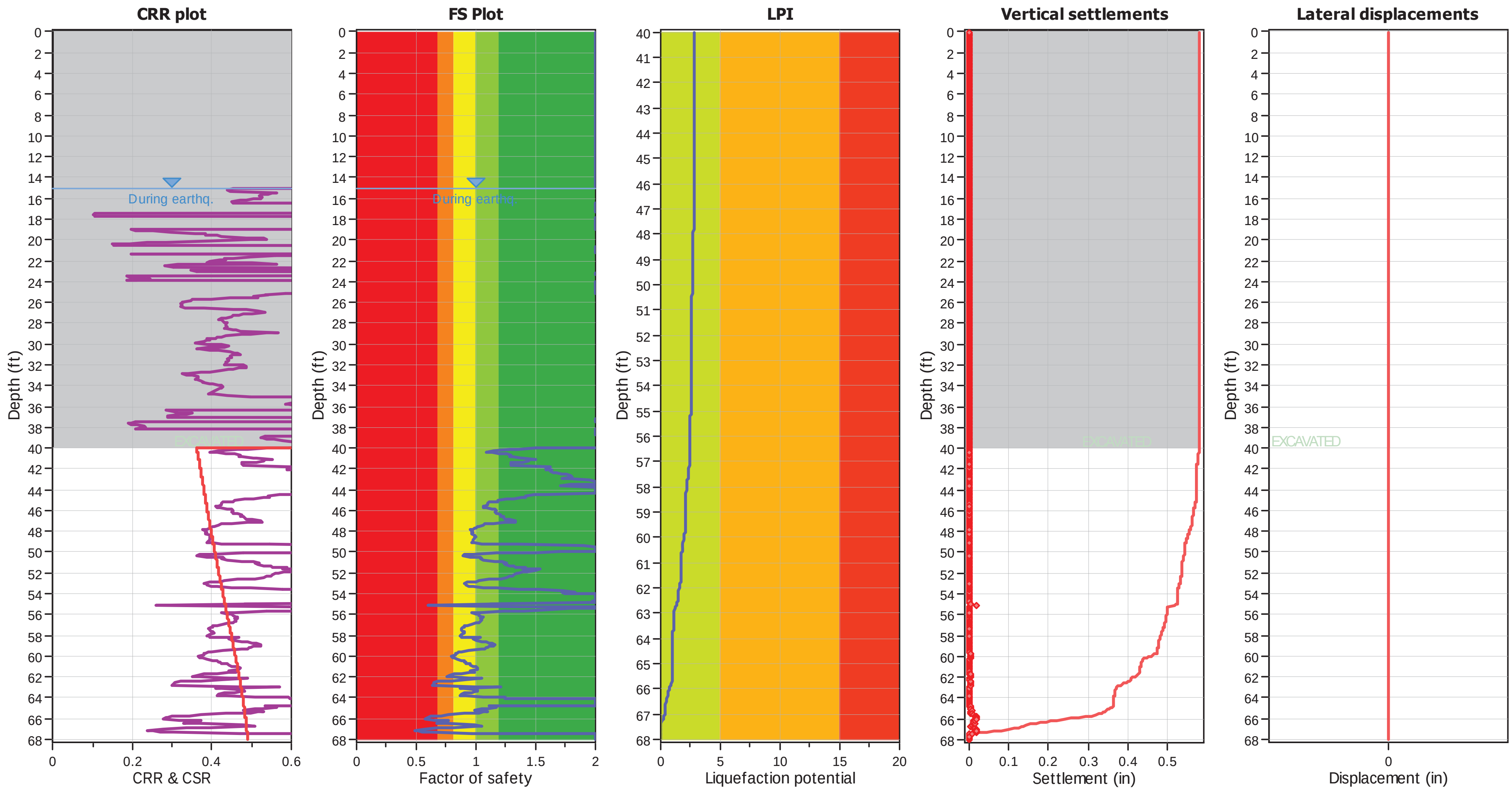
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Ave, Suite 150
San Jose, CA 95119

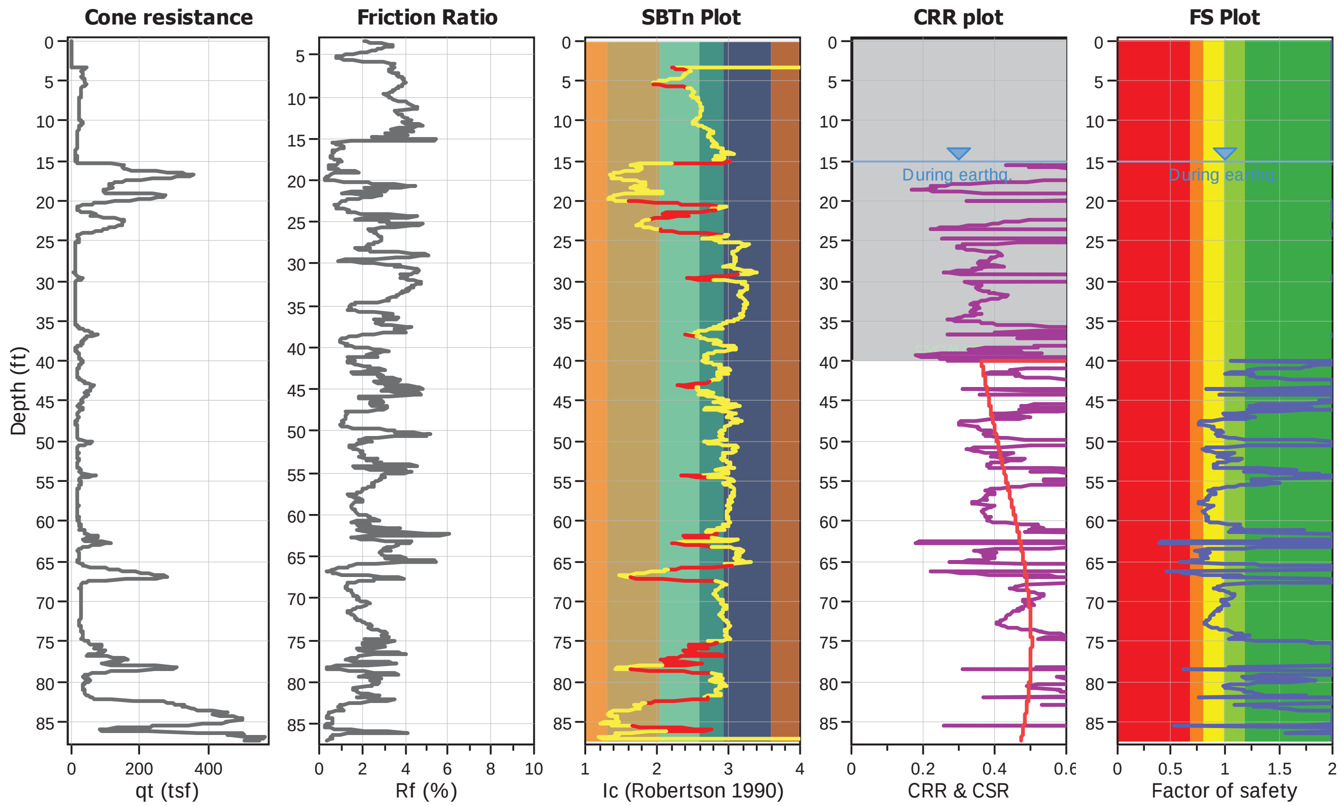
LIQUEFACTION ANALYSIS REPORT

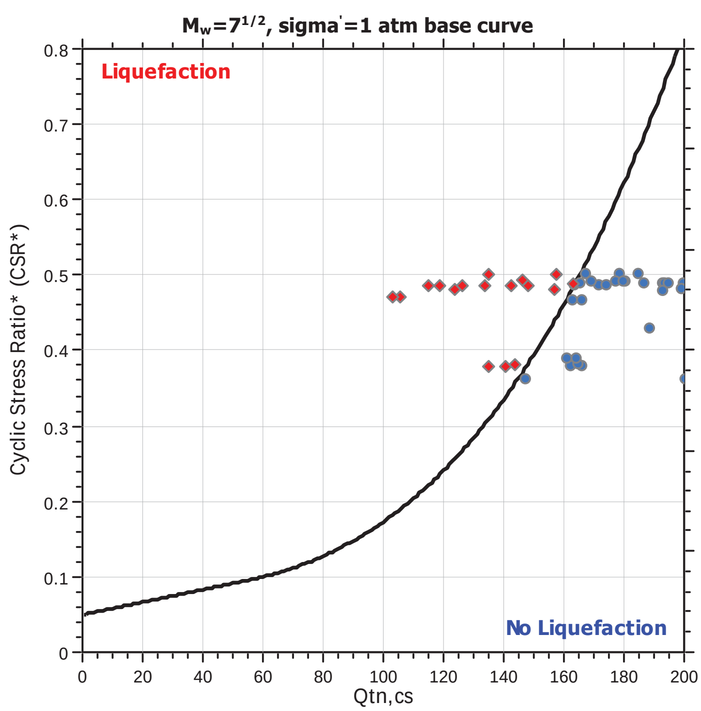
Project title : Almaden Office Complex **Location : San Jose, CA**

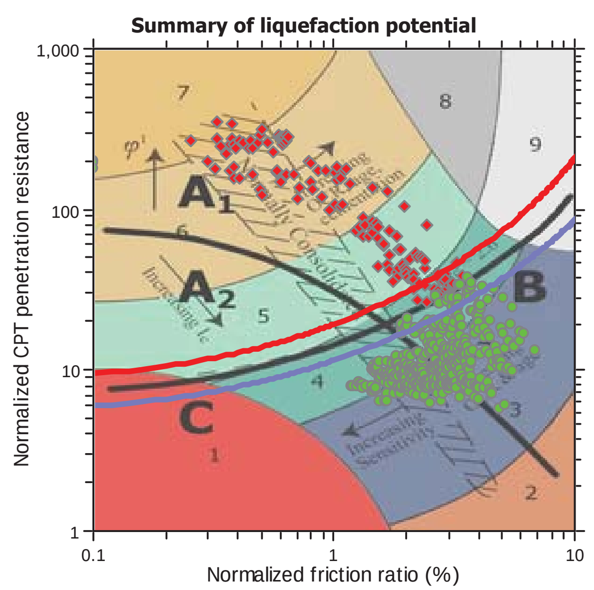
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	15.00 ft	Excavation:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Excavation depth:	40.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Footing load:	2.00 tsf	Limit depth:	N/A
Earthquake magnitude M_w:	7.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	No		

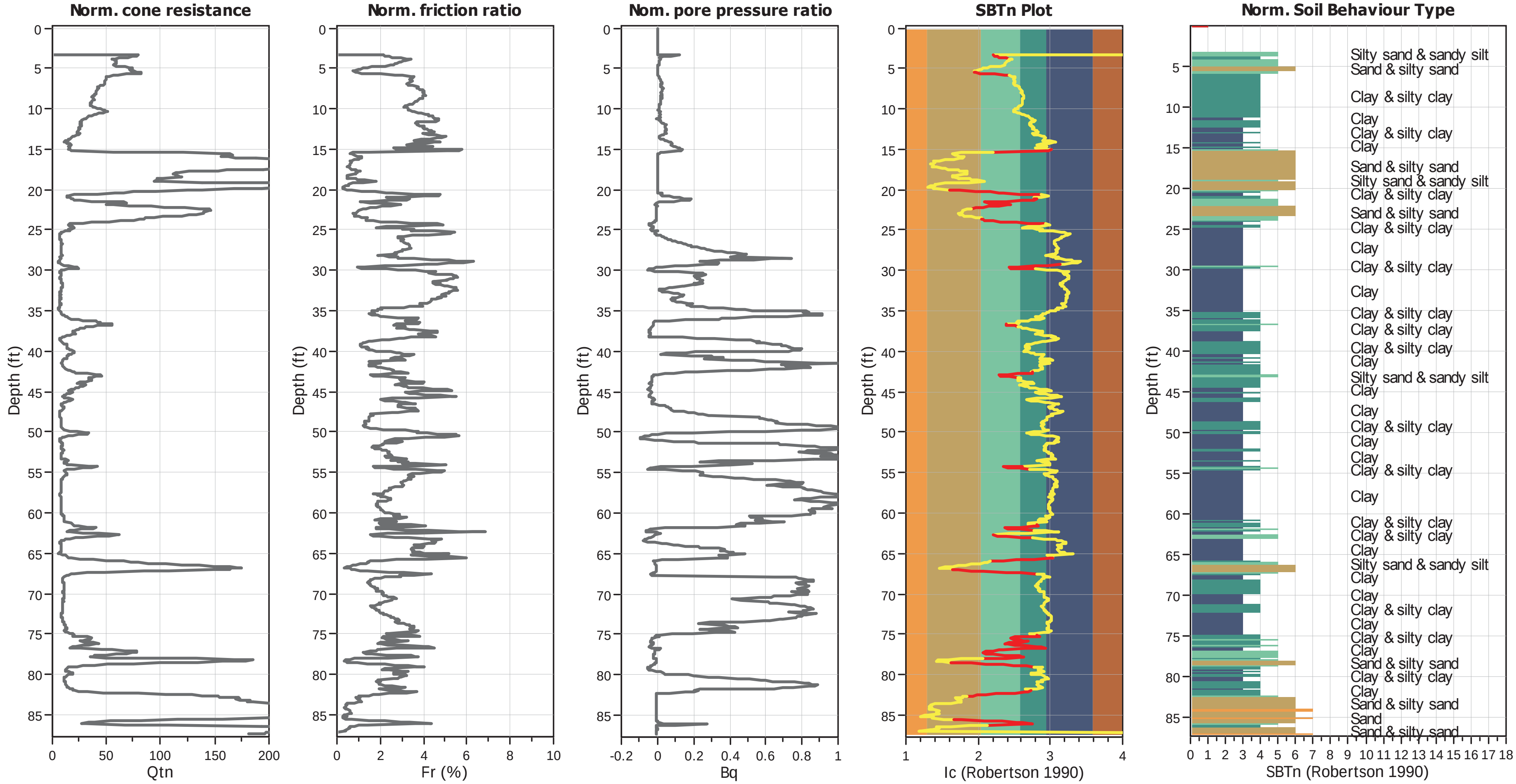






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



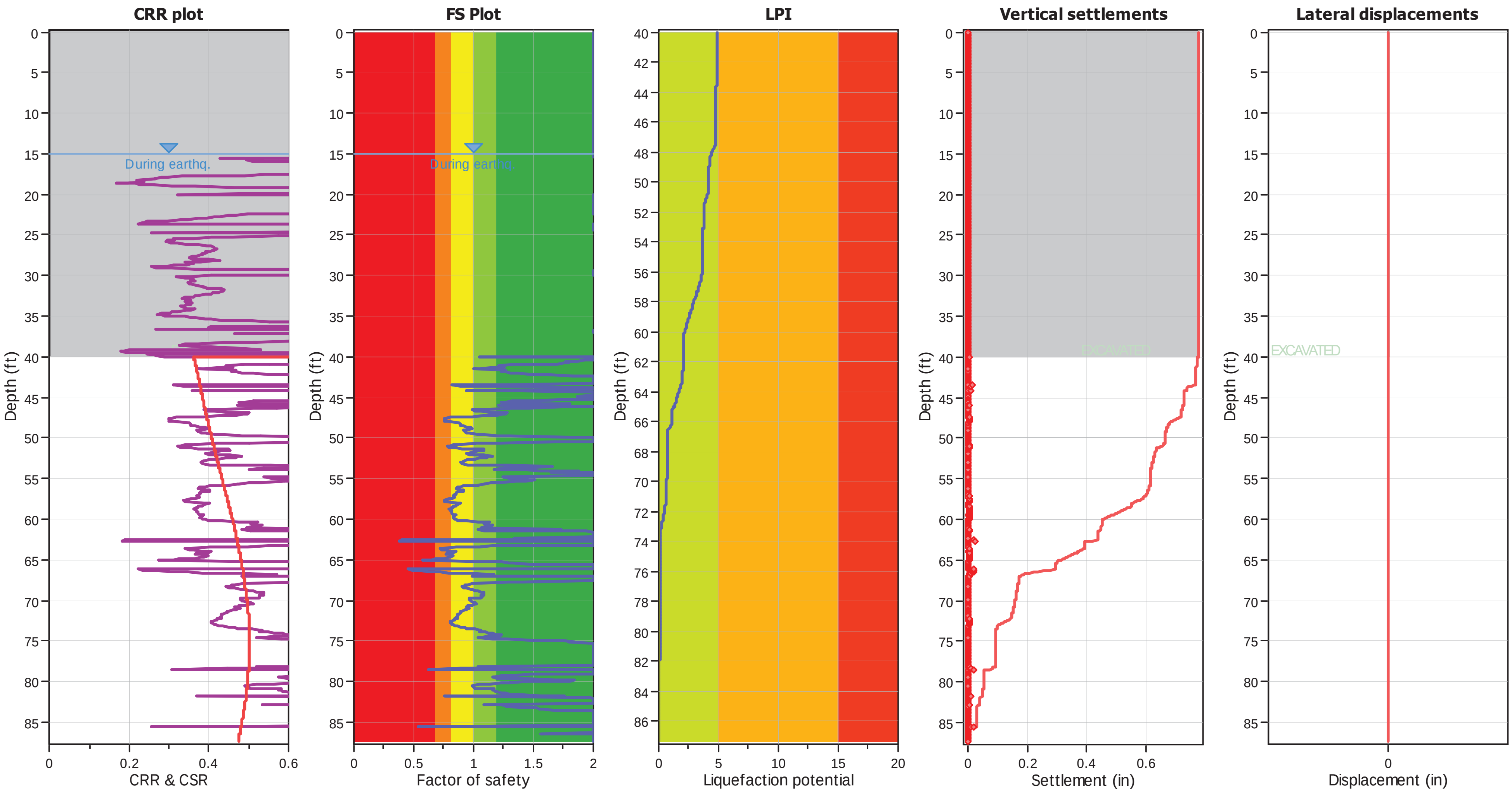
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.50	Excavation:	Yes	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Excavation depth:	40.00 ft	Limit depth:	N/A

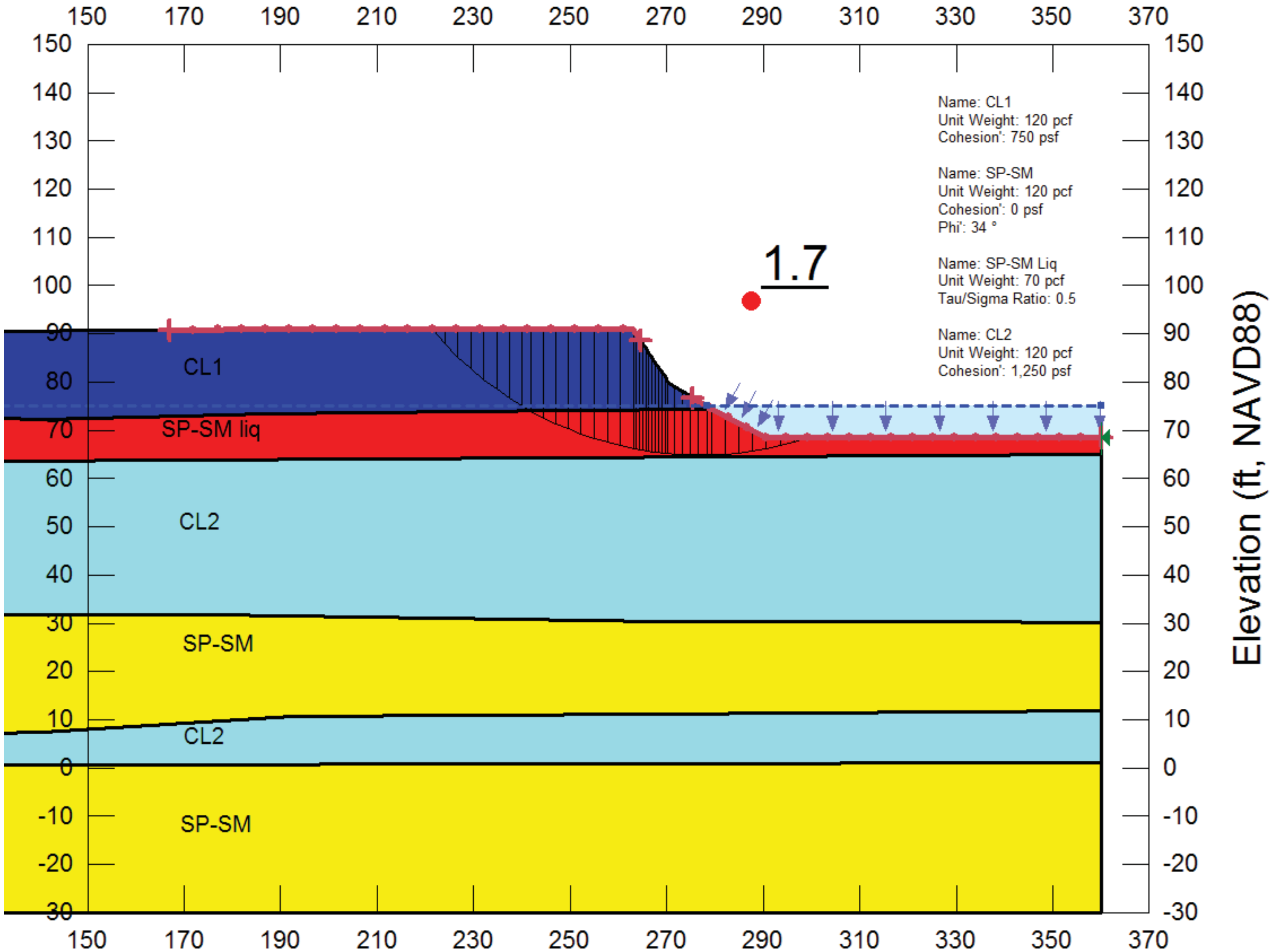
F.S. color scheme

- 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

POST EARTHQUAKE (CIRCULAR)



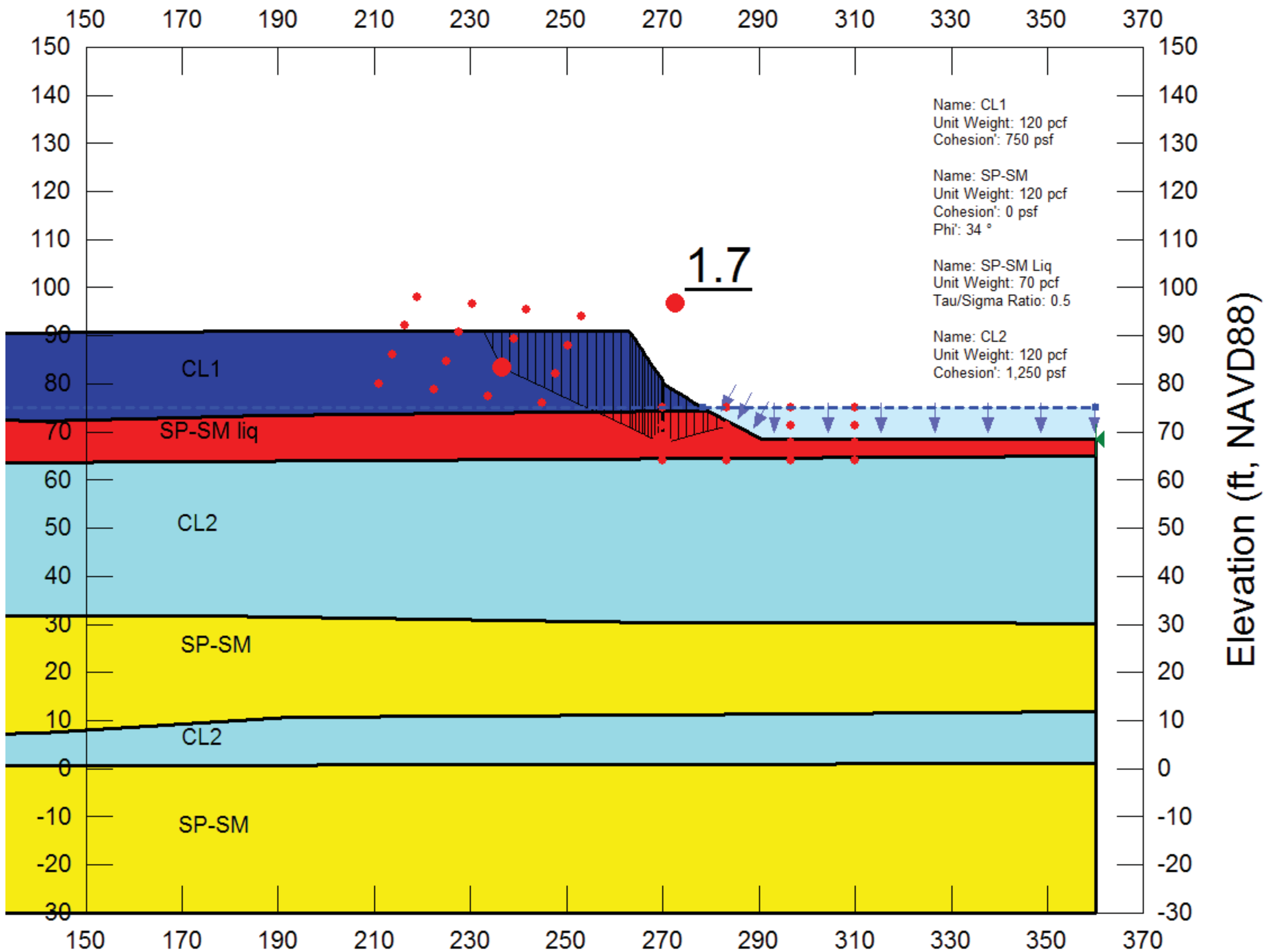
ENGEO Expect Excellence

SEISMIC SLOPE STABILITY ANALYSIS
ALMADEN OFFICE COMPLEX
SAN JOSE, CALIFORNIA

PROJECT NO.: 15540.000.000		FIGURE NO.
SCALE: NO SCALE		APP-E
DRAWN BY: TS	CHECKED BY: im	

1/25/2019 10:36 AM

POST EARTHQUAKE (NON-CIRCULAR)



ENGEO Expect Excellence	SEISMIC SLOPE STABILITY ANALYSIS ALMADEN OFFICE COMPLEX SAN JOSE, CALIFORNIA	PROJECT NO.: 15540.000.000 SCALE: NO SCALE DRAWN BY: TS AL / CHECKED BY: im CC	FIGURE NO. APP-E

1/25/2019 10:36 AM

APPENDIX G

CORROSIVITY TEST RESULTS
(CERCO Analytical)

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

30 November, 2018

Job No. 1811102
Cust. No. 10169

Mr. Ian McCreery
ENGEO Inc.
2010 Crow Canyon Place, Suite 250
San Ramon, CA 94583

Subject: Project No15540.000.000
Project Name: Almadenn Office Complex
Corrosivity Analysis – ASTM Test Methods

Dear Mr. McCreery:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on November 14, 2018. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.002 is classified as "corrosive" and Sample No.002 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected & 16 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations are 20 mg/kg & 27 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The sulfide ion concentrations reflect none detected with a detection limit of 50 mg/kg.

The pH of the soils are 7.65 & 8.00, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 23-mV & 250-mV. Sample No.002 is indicative of potentially "severely corrosive" soils and Sample No.001 is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO Incorporated
Client's Project No.: 15540.000.000
Client's Project Name: Almaden Office Complex
Date Sampled: 27-Oct-18
Date Received: 14-Nov-18
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 30-Nov-2018

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1811102-001	1-B1 @ 44.5-45'	250	8.00	-	2,100	N.D.	N.D.	20
1811102-002	1-B @ 26-26.5'	23	7.65	-	1,400	N.D.	16	27

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
	27-Nov-2018	27-Nov-2018	-	30-Nov-2018	16-Nov-2018	27-Nov-2018	27-Nov-2018

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

1811102 cct# 10169

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 15540.000.000

PROJECT NAME: Almaden Office Complex

SAMPLED BY: (SIGNATURE/PRINT) Ian McCreery

PROJECT MANAGER: Ian McCreery

ROUTING: E-MAIL imccreery@engeo.com Hard Copy --

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	pH	Resistivity	Sulfate	Chloride	Redox	Sulfide	REMARKS REQUIRED DETECTION LIMITS
001	1-B1 @ 44.5-45	27-Oct-18	10:00am	Soil	1	liner	None	x	x	x	x	x	x	TUBES
002	1-B2 @ 26-26.5	27-Oct-18	9:15 AM	Soil	1	LINER	NONE	x	x	x	x	x	x	

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
[signature]	2018-11-13 16:45	[signature]	[signature]	11/14/18 10:18	[signature]
RELINQUISHED BY: (SIGNATURE) [signature]	11/14/18 1435	RECEIVED BY: (SIGNATURE) [signature]	RELINQUISHED BY: (SIGNATURE) [signature]	11/14/18 1525	RECEIVED BY: (SIGNATURE) [signature]
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED FOR LABORATORY BY: (SIGNATURE)	DATE/TIME		

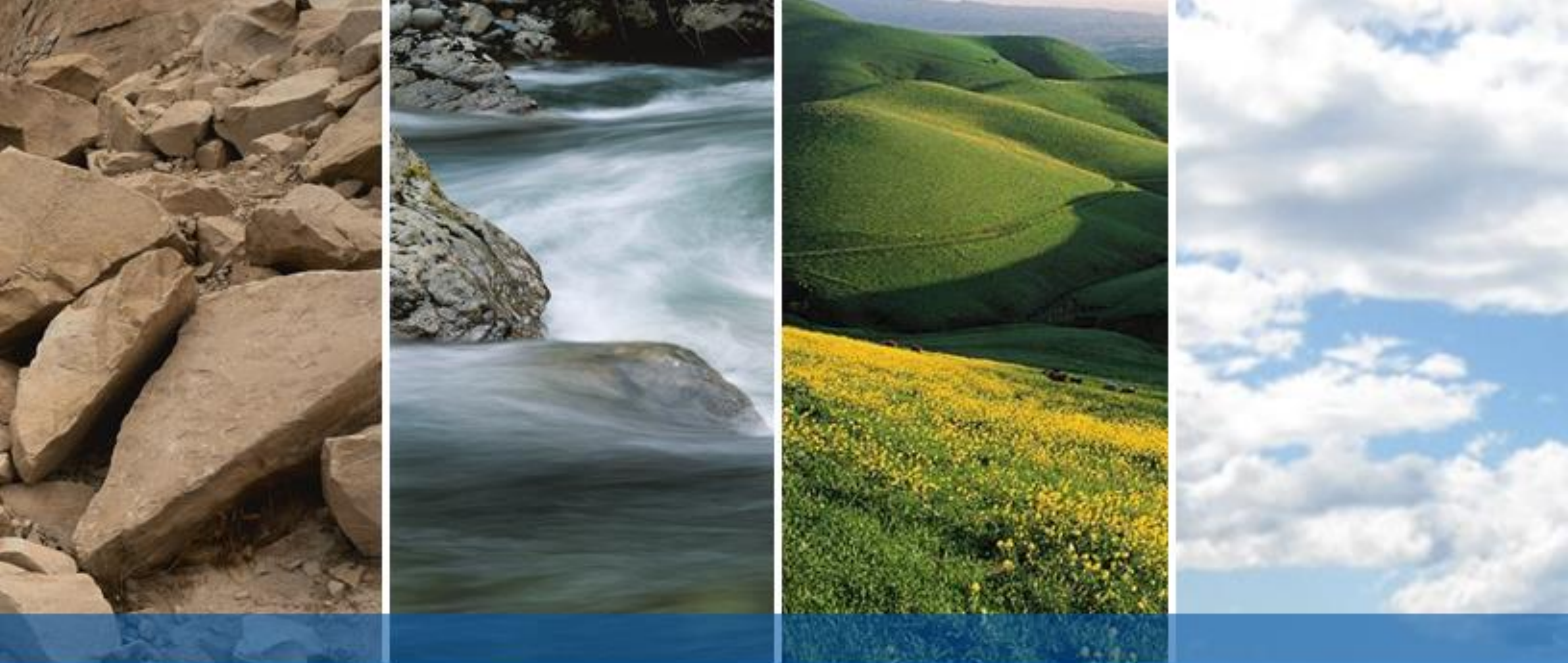
REMARKS

PLEASE INCLUDE BRIEF EVAL

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

ENGEO INCORPORATED

6399 SAN IGNACIO AVENUE, SUITE 150
SAN JOSE, CALIFORNIA 95119
(408) 574-4900 FAX (888) 279-2698
WWW.ENGEO.COM



APPENDIX H

PREVIOUS BORING LOGS BY OTHERS
(Treadwell & Rollo)

PROJECT:	PLAZA AT ALAMADEN San Jose, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2

Logged by: E. Banaag

Date started: 7/6/00 | Date finished: 7/6/00

Drilling method: Rotary Wash

Hammer weight/drop: 140lbs./30-inches | Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Surface Elevation: 90.0 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1					Asphalt concrete, 3 inches Aggregate base, 6 inches						
1–2				CL	CLAY (CL) brown, medium stiff, moist						
3	S&H		7								
6	S&H		8			TxUU	600	1,300		30.4	82
9				CH	CLAY (CH) olive-gray, medium stiff to stiff, wet LL = 77, PI = 45						
11	S&H		9								
14					grading with sand, soft						
16	S&H		4							42.4	79
19.5				SM	SILTY SAND (SM) gray, medium dense, wet						
21	SPT		13						18		
26	SPT		15								
27.5				CH	CLAY (CH) grey with olive-brown mottling, stiff, wet						
30											

Treadwell&Rollo

Project No.: 2869.01

Figure: A-1a

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-1

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30–34	ST		200 to 400 psi	CH	CLAY (CH) (continued)						
34–41	S&H (35–36.5)		10	CL	CLAY (CL) gray with olive-brown mottling, stiff, wet, with trace organics	TxUU	3,600	1,100		26.9	98
					grading with sand and gravel						
	S&H (40–41.5)		11								
41–46	SPT (45–46.5)		13	SP	SAND (SP) olive-gray, medium dense, wet						
46–58.5	S&H (50–51.5)		9	CL	CLAY (CL) gray, stiff, with trace sand						
	ST (55–57.5)		200 to 390 psi		Consolidation Test						
58.5–60				GW	GRAVEL with SAND (GW) gray-brown, very dense, wet						

LABORATORY TEST DATA

Treadwell&Rollo

Project No.: 2869.01

Figure: A-1b

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-1

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	SPT		63	GW	GRAVEL with SAND (GW) (continued)						
62											
63											
64											
65											
66	SPT		37		dense						
67											
68											
69				CH	CLAY (CH) brown, stiff to very stiff, wet						
70											
71	S&H		16								
72					grades to gray						
73											
74					sand layer from 73.0 to 73.5						
75											
76	S&H		17			TxUU	7,500	2,300		22.7	105
77											
78											
79											
80											
81											
82											
83											
84											
85											
86	S&H		19							23.1	106
87											
88											
89				CL	CLAY (CL) gray-brown, stiff, wet						
90											

LABORATORY TEST DATA

Treadwell&Rollo

Project No.: 2869.01

Figure: A-1c

TEST GEOTECH LOG 286901.GPJ T&R.GDT 8/15/00

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-1

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		13	CL	CLAY (CL) (continued) grading with sand						
92											
93											
94				GW	GRAVEL with SAND (GW) gray-brown, very dense, wet						
95	SPT		50/6"								
96											
97											
98											
99											
100	SPT		70								
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 101.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not measured.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

Treadwell&Rollo

Project No.: 2869.01 | Figure: A-1d

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

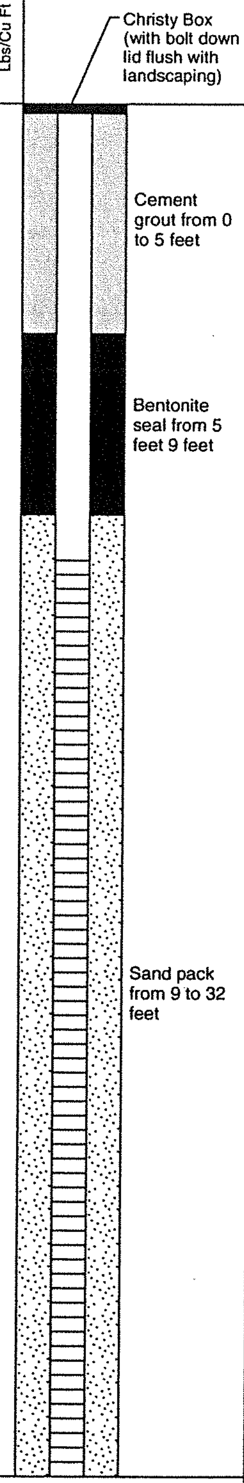
Log of Boring B-2/MW-1

Boring location: See Site Plan, Figure 2	Logged by: E. Banaag
Date started: 7/3/00	Date finished: 7/5/00
Drilling method: Rotary Wash	
Hammer weight/drop: 140lbs./30-inches	Hammer type: Safety
Sampler:	

DEPTH (feet)	SAMPLES: Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					Ground Surface Elevation: 89.6 feet[2]						
0					Asphalt concrete, 4 inches						
					Aggregate base, 6 inches						
1–3				CL	CLAY (CL) brown, stiff to very stiff, dry						
4–25				CL	GRAVELLY CLAY (CL) dark brown, medium stiff, wet, with wood chips and brick fragments						
15					▼ (7/11/00)						
25–30				CH	CLAY (CH) olive gray, stiff, wet						

FILL (0 to 25 feet)

WELL COMPLETION INFORMATION



Treadwell&Rollo

Project No.: 2869.01 | Figure: A-2a

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-2/MW-1 PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft	WELL COMPLETION INFORMATION
31				CH	CLAY (CH) (continued)							
32												
33												
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 32 feet.
Groundwater encountered at a depth of 15.5 feet.
Boring converted to a monitoring well.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

Treadwell&Rollo

Project No.: 2869.01 | Figure: A-2b

GEOTECH WELL LOG 286901.GPJ T&R.GDT 8/15/00

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-3/MW-2

Boring location: See Site Plan, Figure 2

Logged by: E. Banaag

Date started: 7/3/00 Date finished: 7/5/00

Drilling method: Rotary Wash

Hammer weight/drop: 140lbs./30-inches Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube

Ground Surface Elevation: 89.6 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				Asphalt concrete, 6 inches						
0.5				Aggregate base, 6 inches						
1			CL	SANDY CLAY (CL) dark brown, very stiff, moist, with trace gravel, brick fragments (FILL)						
3	S&H	17								
5.5	S&H	8								
6			ML	SILT with SAND (ML) gray, stiff, wet (FILL)						
9			CL	CLAY (CL) dark brown, medium stiff, wet						
10.5	S&H	7							29.2	95
13			CL-ML	SILTY CLAY with SAND (CL-ML) olive-brown, medium stiff, wet, with trace organics						
15				(7/11/00)						
16	ST	200 to 360 psi								
19.5			SP	SAND (SP) gray with olive mottling, medium dense, wet, trace fines						
21	S&H	16							21.7	108
25.5	SPT	24								
28.5			CH	CLAY (CH) gray with olive mottling, stiff, wet						

WELL COMPLETION INFORMATION

Christy Box (with bolt down lid flush with landscaping)

Cement grout from 0 to 45 feet

Treadwell&Rollo

Project No.: 2869.01

Figure: A-3a

GEOTECH WELL LOG 286901.GPJ T&R.GDT 8/15/00

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-3/MW-2

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft	WELL COMPLETION INFORMATION
31	S&H		12	CH	CLAY (CH) continued							
32				SP	SAND (SP) olive-gray, medium dense, wet, trace fines							
33–34					SILTY CLAY (CL-ML) gray with olive-brown mottling, stiff, wet							
35–36	S&H		9			TxUU	3,600	1,350		26.1	100	
37–39				CL-ML								Cement grout from 0 to 45 feet
40–41	S&H		8		LL = 40, PI =17							
42–44												
45–46	S&H		20		very stiff	TxUU	4,600	3,750		18.0	115	
46–49					SILTY CLAY with SAND (CL-ML) gray, stiff, wet							Bentonite seal from 45 to 50 feet
50–52	ST		200 to 400 psi	CL-ML	Consolidation Test LL = 28, PI = 7							
53–54												
55–56	S&H		11		CLAY (CL) gray with brown mottling, stiff, wet	TxUU	5,500	1,400		24.1	103	Sand pack from 50 to 81.5 feet
57				CL	grades to brown							
58–60												

GEOTECH WELL LOG 286901.GPJ T&R.GDT 8/2/00

Treadwell&Rollo

Project No.:	Figure:
2869.01	A-3b

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-3/MW-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft	WELL COMPLETION INFORMATION
	SPT		80		CLAYEY GRAVEL with SAND (GC) olive brown, very dense, wet LL = 28, PI = 7							
61												
62				GC								
63												
64					SAND (SP) brown, dense, wet							
65				SP								
66	S&H		41		GRAVEL with SAND (GW) gray-brown, dense, wet							
67												
68				GW								
69					clay stratum from about 69.5 to 71.0 feet							
70												Sand pack from 50 to 81.5 feet
71	S&H		45		SAND with GRAVEL (SW) olive-brown, dense to very dense, wet	TxUU	7,100	1,700		20.6	108	
72												
73												
74												
75				SW								
76	SPT		51									
77												
78												
79												
80					SILT with SAND (ML) gray, very stiff, wet							
81	S&H		18	ML		TxUU	8,100	2,050		21.3	109	
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 81.5 feet.
Groundwater encountered at a depth of 15.5 feet.
Boring converted to a monitoring well.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

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Project No.: 2869.01
Figure: A-3c

GEOTECH WELL LOG 286901.GPJ T&R.GDT 8/15/00

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-4

Boring location: See Site Plan, Figure 2	Logged by: E. Banaag
Date started: 6/30/00	Date finished: 6/30/00
Drilling method: Rotary Wash	
Hammer weight/drop: 140lbs./30-inches	Hammer type: Safety
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)	

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Surface Elevation: 85.0 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					Asphalt concrete, 6 inches						
0.5					Aggregate base, 6 inches						
1				CL	CLAY (CL) dark brown, very stiff, moist						
3	S&H		16								
4					grades to olive brown						
6	S&H		8		medium stiff, with red-brown mottling						
9				CL-ML	SILTY CLAY (CL-ML) olive-gray with red-brown mottling, medium stiff, wet (6/30/00)						
11	S&H		6								
15				CL	CLAY with SAND (CL) olive-gray, medium stiff, wet						
16	ST		200 to 280 psi								
20				SP	SAND (SP) olive-brown, medium dense, wet, with trace fines						
21	SPT		16								
23.5				CH	CLAY (CH) olive-gray, medium stiff, wet						
27	S&H		6							25.6	101

Treadwell&Rollo

Project No.: 2869.01

Figure: A-4a

TEST GEOTECH LOG 286901.GPJ T&R.GDT 8/7/00

PROJECT:	**PLAZA AT ALAMADEN** San Jose, California	**Log of Boring B-4**

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		10	CH	CLAY (CH) (conitinued) grades to gray, stiff	TxUU	3,100	1,500		33.4	90
32–34											
35–36	SPT		15	SM	SILTY SAND with GRAVEL (SM) gray, medium dense, wet						
37–39				CL	SANDY CLAY (CL) gray, stiff, wet						
40–42	S&H		9	ML	SILT (ML) olive-brown, stiff, wet LL = 32, PI = 7	TxUU	4,000	1,550	84	26.7	99
43–48	ST		200 to 300 psi	CL-ML	SILTY CLAY with SAND (CL-ML) olive-gray, stiff, wet Consolidation Test						
49–52	S&H		13	SM	SILTY SAND (SM) gray, medium dense, wet						
53–60	S&H		15	SC	CLAYEY SAND (SC) olive-gray with olive-brown mottling, stiff to very stiff, wet grades to brown	TxUU	5,600	1,650	44	29.8	95

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Project No.: 2869.01

Figure: A-4b

TEST GEOTECH LOG 286901.GPJ T&R.GDT 8/7/00

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-4 PAGE 3 OF 4

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	SPT		44	SC	CLAYEY SAND with GRAVEL (SC) olive-brown, dense, wet						
62											
63											
64											
65					grading with decreased fines						
66	SPT		50		very dense						
67											
68				CL	CLAY (CL) olive-gray with olive mottling, very stiff, wet, with sand						
69											
70											
71	S&H		16							21.9	108
72											
73											
74											
75											
76	S&H		20			TxUU	7,600	1,550		24.8	103
77					grades to olive-gray with yellow-brown mottling						
78											
79											
80				SC	CLAYEY SAND with GRAVEL (SC) olive-brown, very dense, wet						
81	SPT		100								
82											
83											
84											
85											
86	SPT		21	CL	CLAY (CL) olive-gray with red-brown mottling, very stiff, wet						
87											
88											
89											
90											

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Project No.: 2869.01

Figure: A-4c

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-4

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	SPT		50	SC	CLAYEY SAND with GRAVEL (SC) dark gray with olive-brown mottling, very dense, wet						
92											
93											
94											
95				SP	SAND with GRAVEL (SP) olive-brown, very dense, wet						
96	SPT		58								
97											
98											
99											
100											
101	SPT		77								
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 101.5 feet.
Groundwater encountered at a depth of 10 feet below ground surface.
Boring backfilled with cement grout.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

Treadwell&Rollo

Project No.: 2869.01

Figure: A-4d

TEST GEOTECH LOG 286901.GPJ T&R.GDT 8/7/00

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-5

Boring location:	See Site Plan, Figure 2
Date started:	6/28/00
Date finished:	6/28/00
Drilling method:	Rotary Wash
Hammer weight/drop:	140lbs./30-inches
Hammer type:	Safety
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)
Logged by:	E. Banaag

Surface Elevation: 85.5 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				Asphalt concrete, 6 inches						
0.5				Aggregate base, 6 inches						
1			CL	CLAY (CL) dark brown, stiff, moist						
3	S&H	11								
3.5			SM	SILTY SAND (SM) brown, loose, moist						
5			ML	SILT with SAND (ML) olive-brown, medium stiff, moist						
6	S&H	8							22.0	104
11	ST	200 to 400 psi								
12				color change to gray with black mottling, wet						
14.5				grading with increased sand						
15				▼ (6/29/00)						
16	S&H	28								
16			GW	GRAVEL with SAND (GW) gray, medium dense, wet, with trace fines						
21	S&H	7								
21			CL	SANDY CLAY (CL) gray-brown, medium stiff, wet						
23.5			GW	GRAVEL with CLAY (GW) gray, loose, wet						
26	S&H	5								
26			CH	CLAY (CH) gray, medium stiff, wet						
30										

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Project No.: 2869.01

Figure: A-5a

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-5

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	ST			CH	CLAY (CH) (continued) Consolidation Test						
32											
33				CL-ML	SILTY CLAY with SAND (CL-ML) gray-brown, very stiff, wet						
34											
35					LL = 27, PI = 7						
36	S&H		15								
37					gravel layer from 37 to 38 feet						
38											
39											
40											
41	S&H		6								
42											
43											
44					gravel layer from 44 to 45 feet						
45				CL	CLAY with SAND (CL) olive-gray with brown mottling, very stiff, wet						
46	S&H		16							20.9	109
47											
48											
49				ML	SILT with SAND (ML) olive-gray, medium stiff, wet, with clay						
50											
51	S&H		8			TxUU	5,100	1,550		21.6	107
52											
53				CL	CLAY (CL) gray, stiff, wet						
54											
55											
56	S&H		12								
57											
58											
59											
60											

TEST GEOTECH LOG 286901.GPJ T&R.GDT 8/2/00

Treadwell&Rollo

Project No.: 2869.01

Figure: A-5b

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-5

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H		16	CL	CLAY (CL) (continued) grading with sand from 60 to 61 feet	TxUU	6,000	1,200		26.4	100
62											
63											
64				SP	SAND (SP) gray, medium dense, wet						
65											
66	SPT		18								
67				CH	CLAY (CH) gray, very stiff, wet						
68											
69											
70											
71	S&H		20			TxUU	7,100	1,600		23.9	103
72											
73											
74				CL	CLAY (CL) gray, stiff, wet						
75											
76	S&H		14							22.6	106
77											
78											
79											
80											
81	S&H		20		very stiff	TxUU	8,000	1,850		24.3	104
82					color change to yellow-brown						
83											
84											
85				SW	SAND with GRAVEL (SW) brown, dense, wet, with trace fines						
86	SPT		44								
87											
88											
89											
90											

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Project No.: 2869.01 | Figure: A-5c

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-5
		PAGE 4 OF 4

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	SPT		38	SW	SAND with GRAVEL (SW) (continued) grading with increased fines						
92											
93					clay layer from 93.0 to 93.5 feet						
94											
95				CH	CLAY (CH) brown, stiff, wet						
96	SPT		12								
97											
98											
99											
100				SW	SAND with GRAVEL (SW) brown, very dense, wet						
101	SPT		50								
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 101.5 feet.
Groundwater encountered at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Elevations based on San Jose City Datum.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

Treadwell&Rollo

Project No.:	Figure:
2869.01	A-5d

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-6

Boring location: See Site Plan, Figure 2

Logged by: E. Banaag

Date started: 7/7/00 | Date finished: 7/7/00

Drilling method: Rotary Wash

Hammer weight/drop: 140lbs./30-inches | Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Surface Elevation: 90.0 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.25				Asphalt concrete, 3 inches (FILL)						
0.25–0.75				Aggregate base, 6 inches (FILL)						
0.75–7.5			SM	SILTY SAND (SM) dark brown, loose, moist, with brick and glass fragments (FILL)						
3	S&H	7								
6	S&H	2		very loose					19.3	76
7.5–11			CL	CLAY (CL) dark brown, medium stiff, moist						
11	S&H	6								
11–19.5			CH	CLAY (CH) olive-gray, medium stiff, moist						
14				▼ 7/10/00						
16	S&H	7								
19.5–26			SP	SAND (SP) gray, medium dense, wet						
21	S&H	11								
26	SPT	10		silt layer from 25.5 to 26.0 feet						
26–30			CH	CLAY (CH) dark gray, stiff, wet						

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Project No.: 2869.01

Figure: A-6a

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-6

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31–32	ST		200 to 400 psi	CH	CLAY (CH) (continued)						
33											
34–37	S&H		9	CL	CLAY (CL) gray with olive-brown mottling, stiff, wet, with trace organics						
38–40				CL	SANDY CLAY (CL) olive-brown, stiff, wet						
41	S&H		8								
41–45				ML	SANDY SILT (ML) olive-gray with olive-brown mottling, stiff, wet						
45					LL = 28, PI =5						
46	S&H		10			TxUU	4,600	1,450	66	24.6	102
46–48				ML	SILT (ML) dark gray, stiff, wet						
48–50				CL	CLAY (CL) gray-brown, stiff, wet						
51–52	ST		200 to 380 psi		Consolidation Test						
55					grading with sand						
56	S&H		17			TxUU	5,600	1,750		26.7	99
59–60				GW	GRAVEL with SAND (GW)						

Treadwell&Rollo

Project No.: 2869.01

Figure: A-6b

PROJECT:	**PLAZA AT ALAMADEN** San Jose, California	**Log of Boring B-6**

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60	S&H		36/6"	GW	GRAVEL with SAND (GW) (continued) gray-brown, very dense, wet						
66	SPT		51								
69				CH	CLAY (CH) olive-gray, stiff, wet						
71	S&H		13			TxUU	7,100	1,700		25.5	101
75					grading with less plasticity, very stiff						
76	S&H		19							23.1	106
78				GW-GM	GRAVEL with SILT and SAND (GW - GM) gray-brown, medium dense, wet						
81	SPT		20						9		
82.5				CL	CLAY (CL) gray, stiff, wet						
86	S&H		13			TxUU	8,600	2,750		21.3	109
87.5				CL	SANDY CLAY (CL) olive-gray, very stiff, wet						
90											

Treadwell&Rollo

Project No.:	Figure:
2869.01	A-6c

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-6

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					SANDY CLAY (CL) (continued)						
91	S&H		22							20.5	110
92				CL							
93											
94					SAND with GRAVEL (SP) dive-brown, very dense, wet						
95											
96	SPT		70								
97											
98				SP							
99											
100											
101	SPT		57								
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 101.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 14 feet below ground surface.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

Treadwell&Rollo

Project No.: 2869.01 | Figure: A-6d

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-7 PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Logged by: E. Banaag

Date started: 7/8/00 — Date finished: 7/10/00

Drilling method: Rotary Wash

Hammer weight/drop: 140lbs./30-inches — Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Surface Elevation: 87.9 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				Asphalt concrete, 3 inches						
0.5				Aggregate base, 6 inches						
1			CL	CLAY (CL) dark brown, stiff, moist						
3	S&H	10								
6	S&H	18		very stiff						
9			CH	CLAY (CH) gray-brown with olive mottling, stiff, moist						
11	S&H	15							36.9	85
15.5				medium stiff, wet						
16	S&H	2			TxUU	1,600	650		27.2	99
20			SM	SILTY SAND (SM) dark gray, loose, wet						
21	S&H	7								
21.5			CL	CLAY (CL) dark gray, medium stiff, wet						
23.5			SM	SILTY SAND (SM) olive gray, medium dense, wet						
26	SPT	14								
28.5			CH	CLAY (CH) dark gray, stiff, wet						
30										

Treadwell&Rollo

Project No.: 2869.01 — Figure: A-7a

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30					CLAY (CH) (continued)						
31	ST		200 to 360 psi								
32											
33											
34				CH							
35											
36	S&H		11		color change to yellow-gray with yellow-brown mottling, stiff, wet					27.2	99
37											
38					SANDY CLAY (CL) gray-brown, medium stiff, wet						
39				CL							
40											
41	S&H		7		CLAY (CL) olive-brown, medium stiff, wet						
42											
43											
44											
45											
46	S&H		13		stiff						
47											
48											
49				CL							
50					grading with sand						
51	S&H		13			TxUU	5,100	1,200		23.9	103
52											
53											
54											
55					very stiff						
56	S&H		17							24.2	102
57											
58					GRAVEL with SAND (GW) gray-brown, very dense, wet, with trace fines						
59				GW							
60											

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60–61	SPT		50/ 6"	GW	GRAVEL with SAND (GW) (continued)						
65–66.5	S&H		4	ML	SANDY SILT (ML) yellow-brown, medium stiff, wet LL = 31, PI = 6	TxUU	6,600	2,100		27.3	98
69				CL	CLAY (CL) olive-gray, medium stiff, wet						
70–72	ST		200 to 600 psi	CL	Consolidation Test						
71.5				SW	SAND with GRAVEL (SW) dark brown, very dense, wet						
75–76.5	SPT		80	SW	interbedded silt lenses from 75 to 80 feet						
80–81.5	SPT		49	SW							
85.5				CL	SANDY CLAY (CL) yellow-brown, very stiff, wet						

Depth scale: 61, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-7

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		28	CL	SANDY CLAY (CL) (continued)	TxUU	9,100	5,500		18.4	112
92					SAND (SP) olive-brown, very dense, wet						
93				SP							
94											
95					SAND with GRAVEL (GW) olive-brown, very dense, wet						
96	SPT		54								
97											
98				GW							
99											
100											
101	SPT		80								
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 101.5 feet.
Boring backfilled with cement grout.
Groundwater not measured.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

Treadwell&Rollo

Project No.: 2869.01
Figure: A-7d

TEST GEOTECH LOG 286901.GPJ T&R.GDT 8/7/00

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-8 PAGE 1 OF 3

Boring location: See Site Plan, Figure 2

Logged by: E. Banaag

Date started: 6/29/00 — Date finished: 6/29/00

Drilling method: Rotary Wash

Hammer weight/drop: 140lbs./30-inches — Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Surface Elevation: 87.0 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				Asphalt concrete, 6 inches						
0.5–1				Aggregate base, 6 inches						
1				CLAY (CL) dark brown, very stiff, moist						
3	S&H	20	CL	LL = 58, PI = 28						
5				CLAY (CL) olive-brown, stiff, moist						
6	S&H	14								
10	S&H	11	CL	dark brown						
13.5				SILTY CLAY with SAND (CL-ML) gray with olive-brown mottling, soft to medium stiff, wet						
16	S&H	4	CL-ML						21.9	107
18				SILTY SAND with GRAVEL (SM) olive-gray, medium dense, wet						
21	SPT	11	SM							
23				SILT (ML) olive-gray, medium stiff, wet, trace very fine-grained sand						
26	SPT	5	ML							
27.5			CH	CLAY (CH) gray with olive-brown mottling, stiff, wet						
30										

Treadwell&Rollo

Project No.: 2869.01

Figure: A-8a

PROJECT:	PLAZA AT ALAMADEN San Jose, California	Log of Boring B-8

DEPTH (feet)	Sampler Type	Sample	SPT N-Value¹	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31–32	ST		200 to 400 psi	CH	CLAY (CH) (continued)	TxUU	3,000	1,300		39.8	82
34–44				CL-ML	SILTY CLAY with SAND (CL-ML) gray with olive-brown mottling, medium stiff, wet						
36	S&H		8								
41	S&H		13		grading stiff, with trace sand	TxUU	4,100	3,850		21.4	109
44–45				SM	SILTY SAND (SM) olive-brown, loose to medium dense, wet						
46	S&H		12	ML	SILT with SAND (ML) olive-gray with olive-brown mottling, stiff, wet						
51	ST		200 to 260 psi		grading with increased sand Consolidation Test						
53–56				ML	SANDY SILT (ML) gray, stiff, wet						
56	S&H		18			TxUU	5,600	2,500		20.7	109
56–60				CL	CLAY (CL) gray, very stiff, wet, with trace organics						

Treadwell&Rollo

Project No.: 2869.01

Figure: A-8b

TEST GEOTECH LOG 286901.GPJ T&R.GDT 8/2/00

PROJECT: **PLAZA AT ALAMADEN**
San Jose, California

Log of Boring B-8

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H		15	ML	SILT (ML) gray, stiff, wet, with trace organics, with very fine-grained sand					26.3	100
66	S&H		27			TxUU	6,600	4,450		20.1	109
66–68				SM	SILTY SAND with GRAVEL (SM) yellow-brown, dense, wet						
68–76				SW	SAND with GRAVEL (SW) olive-brown, very dense, wet, trace fines						
71	SPT		60								
76	SPT		24								
76–77.5				ML	SILT with SAND (ML) yellow-brown, very stiff, wet						
77.5–80				GW	GRAVEL with SAND (GW) yellow-brown, very dense, wet						
80–81.5				SW	GRAVELLY SAND (SW) red-brown, very dense, wet						
81	SPT		55								

Boring terminated at a depth of 81.5 feet.
Groundwater not measured.
Boring backfilled with cement grout.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevations based on San Jose City Datum.

Treadwell&Rollo

Project No.: 2869.01

Figure: A-8c

UNIFIED SOIL CLASSIFICATION SYSTEM			
Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART		
Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with split-barrel sampler other than Standard Penetration Test sampler. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Groundwater level at the time and date indicated

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

PLAZA AT ALMADEN
San Jose, California

Treadwell&Rollo

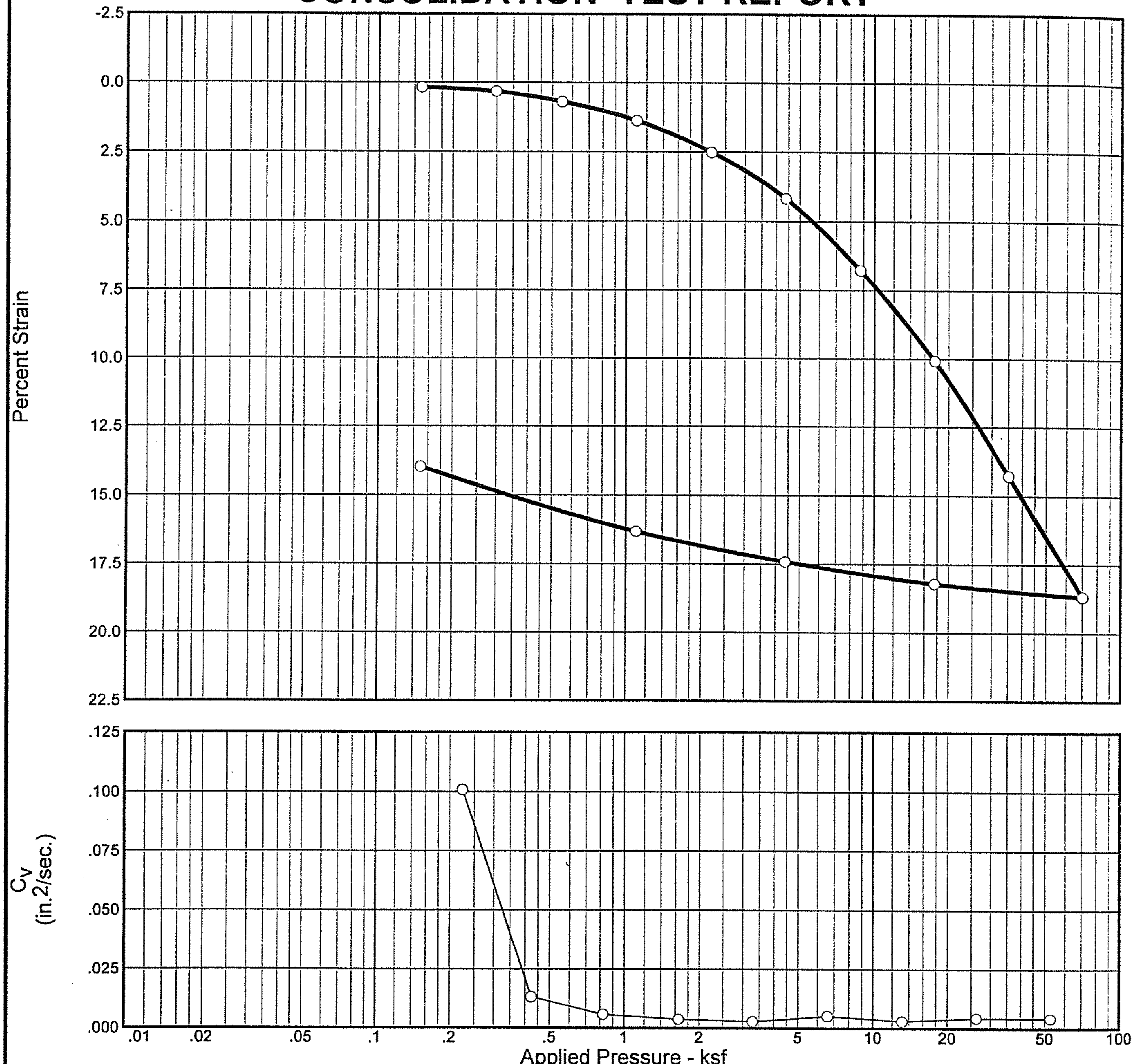
CLASSIFICATION CHART

Date 08/07/00 | Project No. 2869.01 | Figure A-9

APPENDIX I

PREVIOUS LABORATORY TESTING BY OTHERS (Treadwell & Rollo)

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
97.1 %	27.5 %	95.6			2.7			0.764

MATERIAL DESCRIPTION

gray clayey SAND

Project No. 010-483 **Client:** Treadwell & Rollo

Project: 2869.01

Source: B-1 **Sample No.:** 14 **Elev./Depth:** 55'

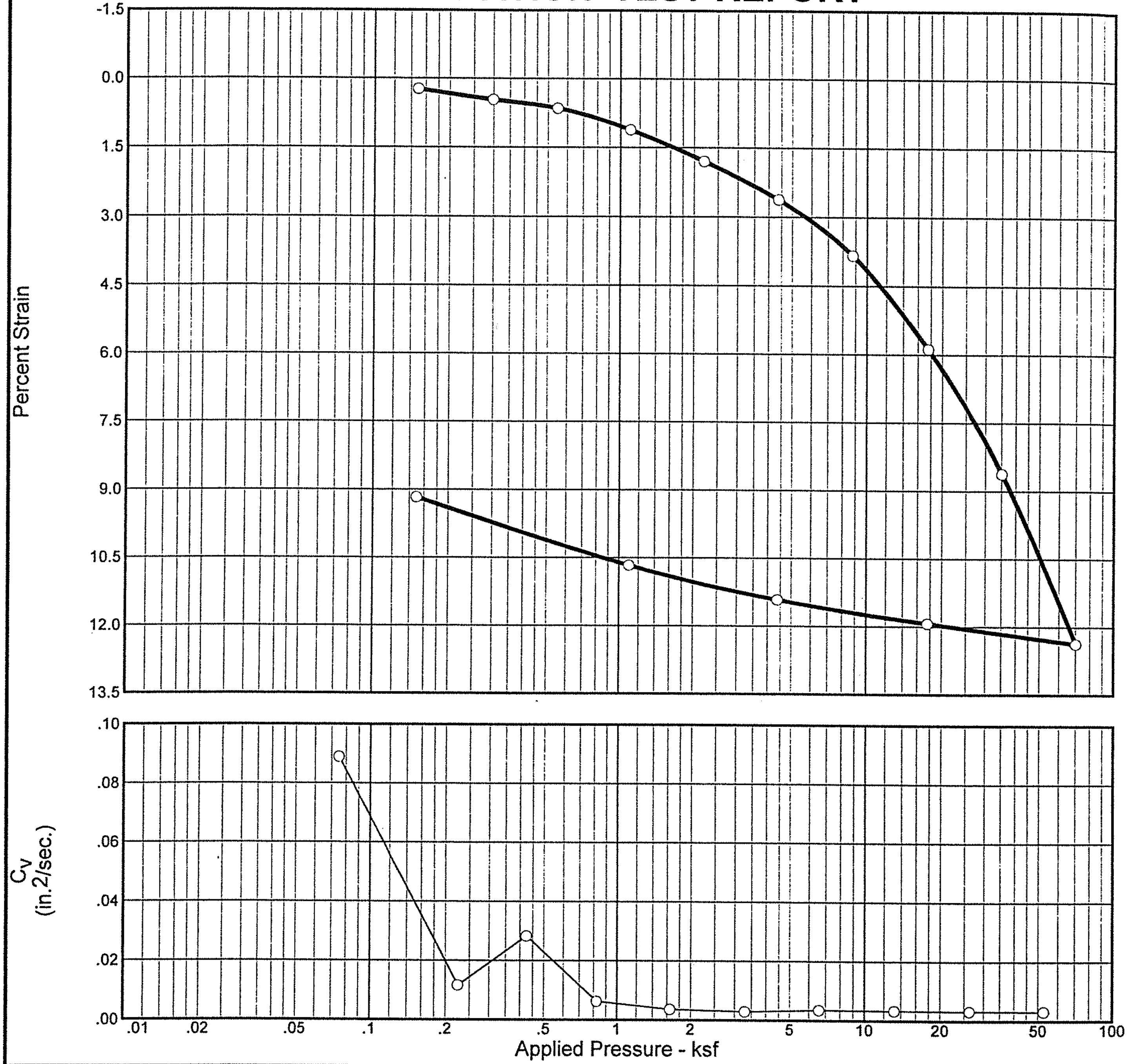
Remarks:

CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

Plate

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
99.0 %	23.3 %	103.0			2.7			0.636

MATERIAL DESCRIPTION

gray sandy CLAY, (silty) near clayey SAND

Project No. 010-483 **Client:** Treadwell & Rollo

Project: 2869.01

Source: B-3 **Sample No.:** 12 **Elev./Depth:** 50'

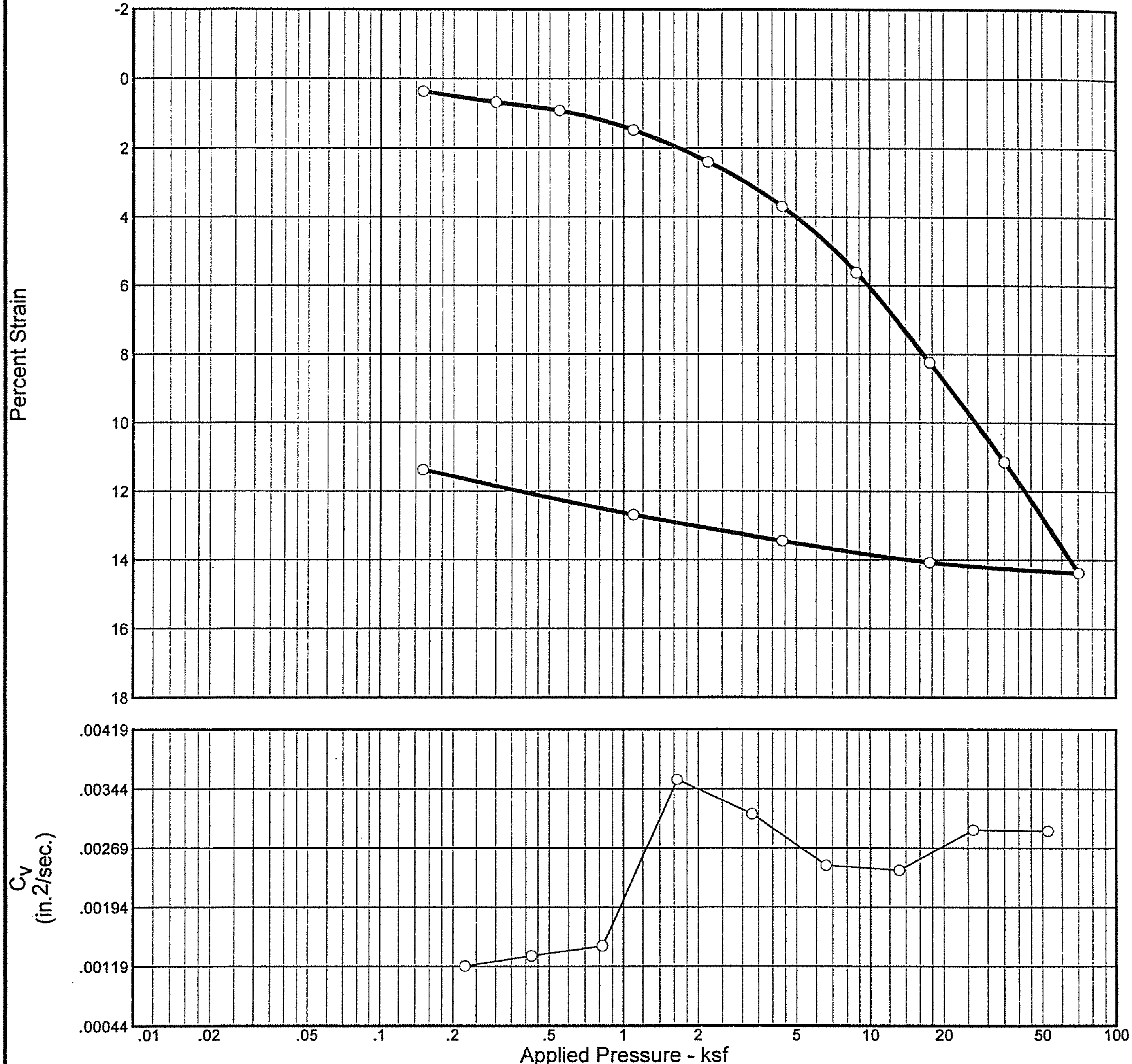
CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

Remarks:

Plate

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
96.3 %	21.1 %	105.8			2.7			0.592

MATERIAL DESCRIPTION

gray clayey SAND, (silty), w/nodules

Project No. 010-483 **Client:** Treadwell & Rollo

Project: 2869.01

Source: B-4 **Sample No.:** 10 **Elev./Depth:** 45'

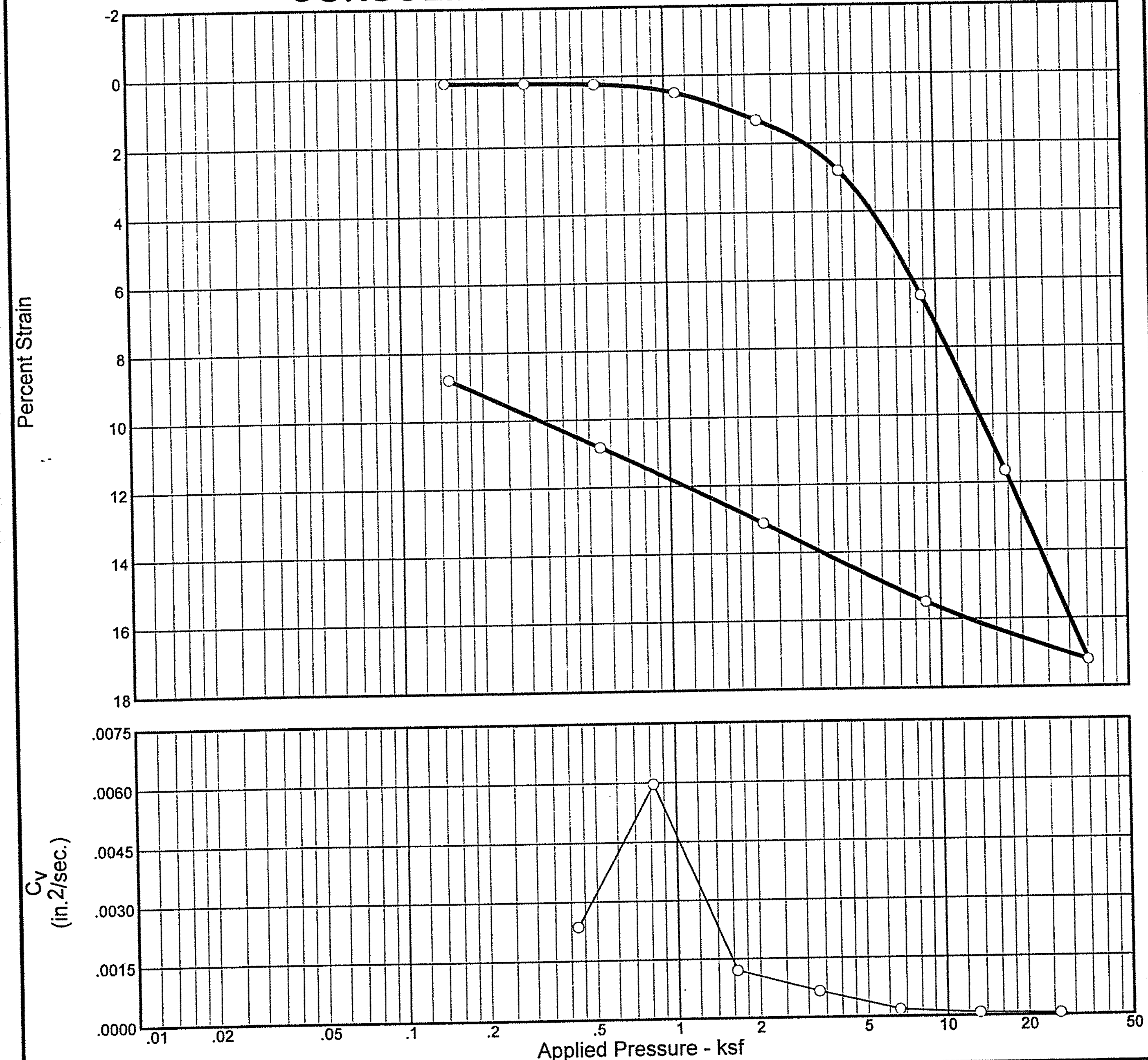
CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

Remarks:

Plate

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
98.0 %	33.6 %	87.5			2.7			0.925

MATERIAL DESCRIPTION

dark gray CLAY

Project No. 010-483 **Client:** Treadwell & Rollo

Project: 2869.01

Source: B-5 **Sample No.:** 9 **Elev./Depth:** 30'

CONSOLIDATION TEST REPORT

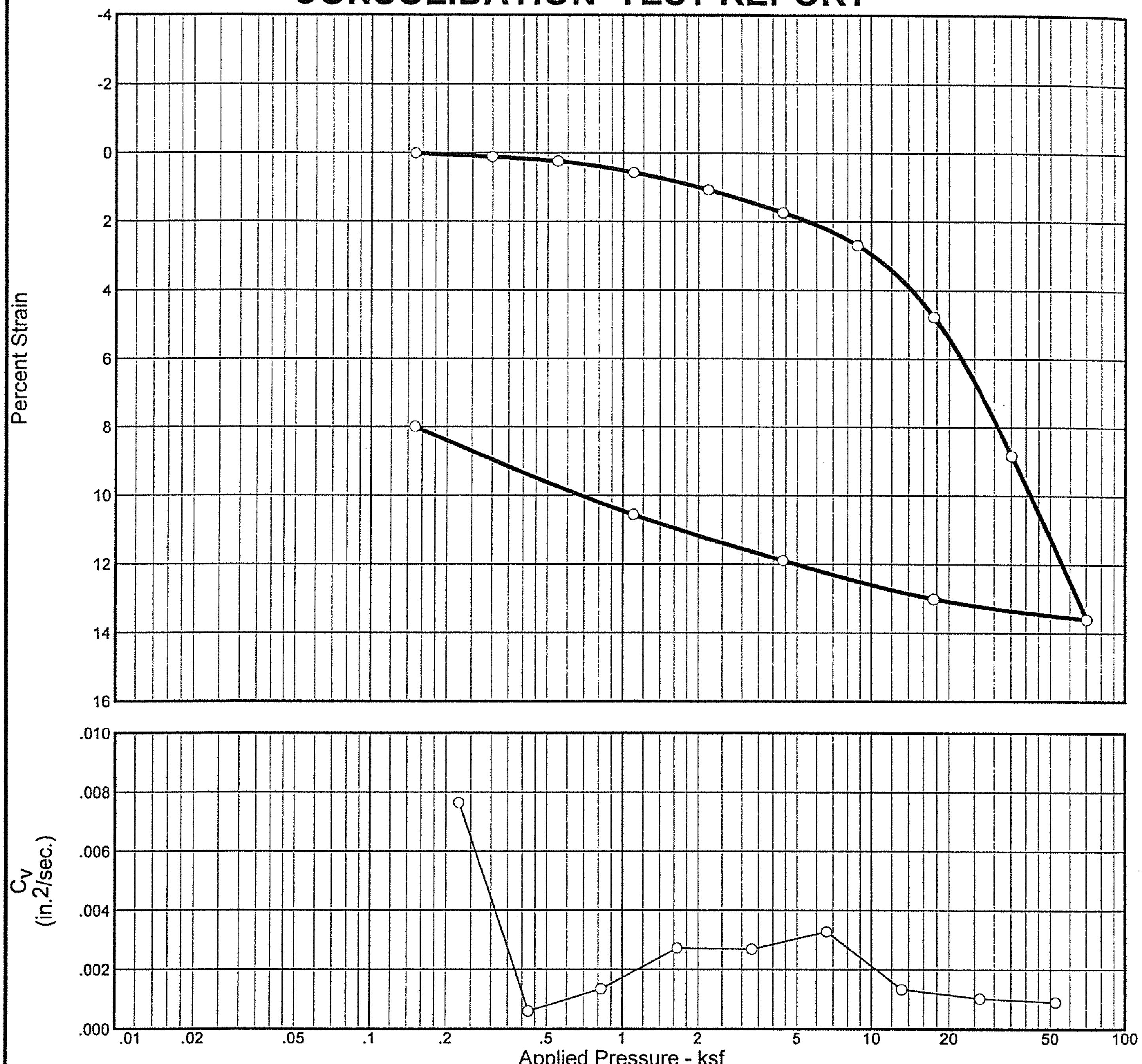
COOPER TESTING LABORATORY

Remarks:

Curve plotted at D100 of each load.
D100 = dial reading at 100% consolidation.

Plate

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
98.7 %	24.9 %	101.3			2.75			0.695

MATERIAL DESCRIPTION

olive brown mottled orange sandy CLAY

Project No. 010-483 **Client:** Treadwell & Rollo

Project: 2869.01

Source: B-6 **Sample No.:** 13 **Elev./Depth:** 50'

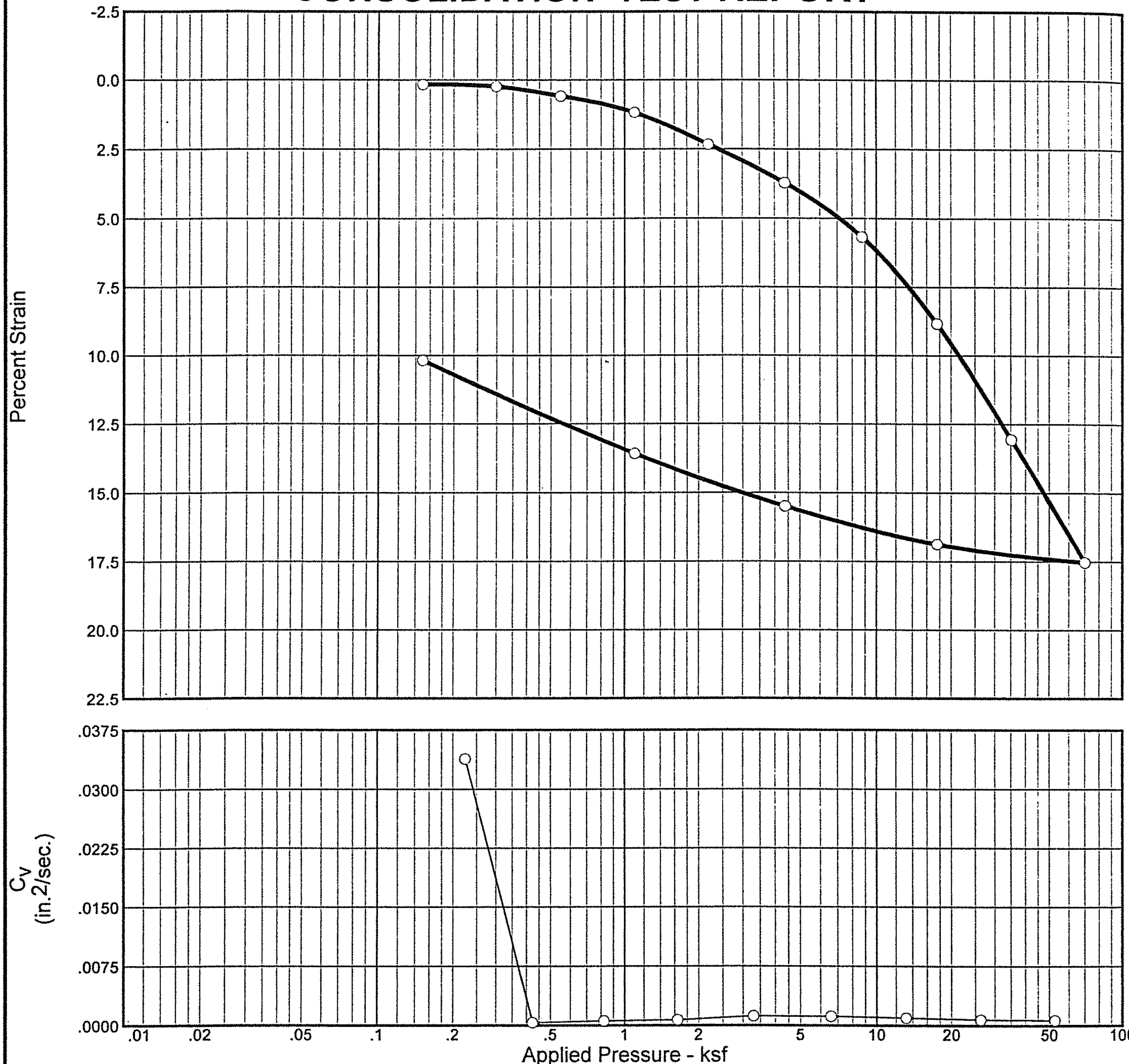
Remarks:

CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

Plate

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
99.6 %	27.0 %	97.4			2.7			0.731

MATERIAL DESCRIPTION

brown mottled orange sandy CLAY

Project No. 010-483 **Client:** Treadwell & Rollo

Project: 2869.01

Source: B-7 **Sample No.:** 15 **Elev./Depth:** 70'

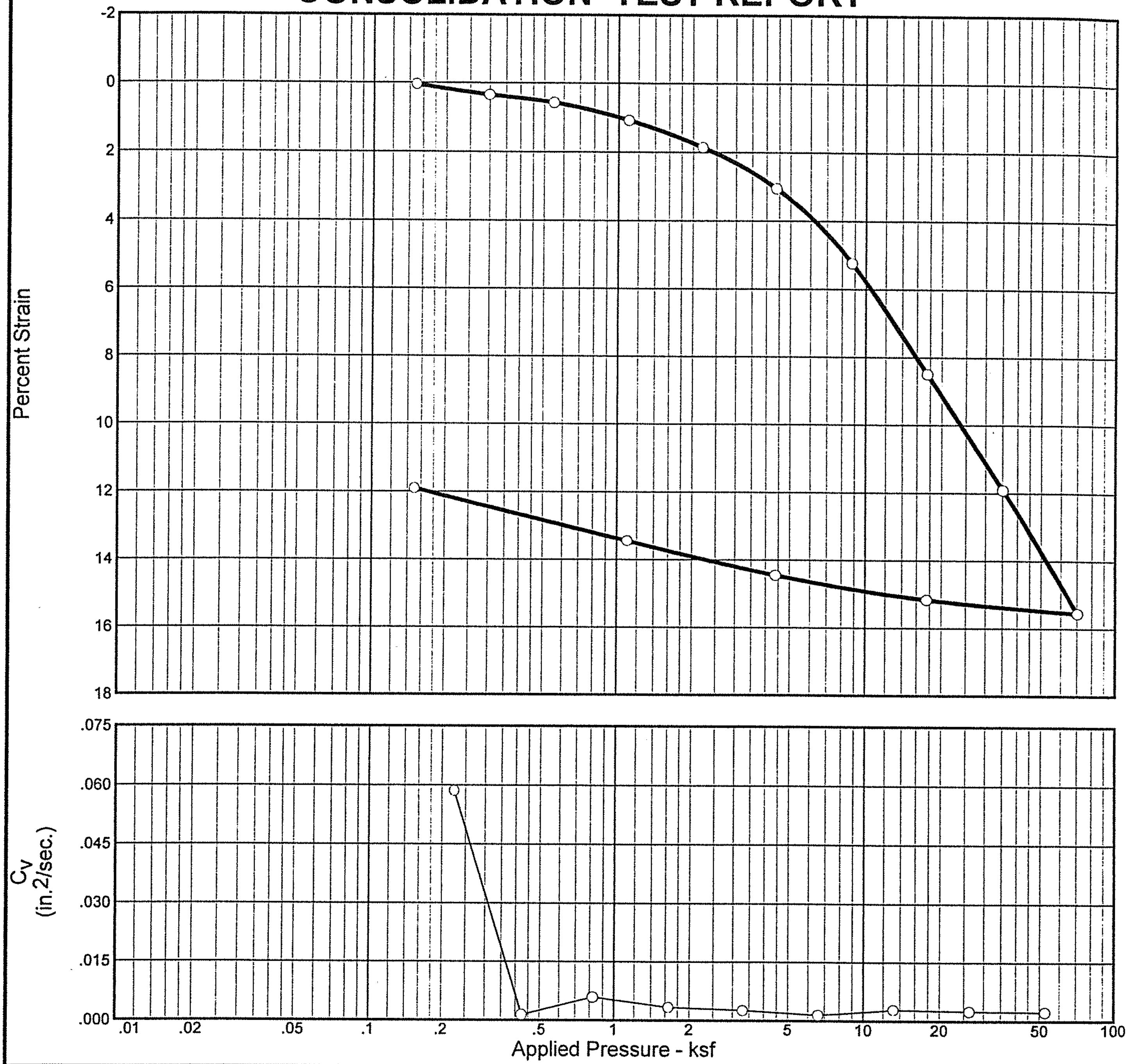
CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

Remarks:

Plate

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
97.2 %	21.6 %	105.3			2.7			0.601

MATERIAL DESCRIPTION

olive gray clayey SAND w/gravel & nodules

Project No. 010-483 **Client:** Treadwell & Rollo

Project: 2869.01

Source: B-8 **Sample No.:** 11 **Elev./Depth:** 50'

Remarks:

CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

Plate

California State Certified Laboratory No.2153
26 July, 2000

CERCO
analytical, inc.

Job No.0007063
Cust. No.10727

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel: 925.462.2771
Fax: 925.462.2775

Ms. Cary Ronan
Treadwell & Rollo
555 Montgomery Street, Suite 1300
San Francisco, CA 94111

Subject: Project No.2869.01
Project Name: Almaden Plaza
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Ronan:

Pursuant to your request, the two soil samples furnished by your office were analyzed in accordance with ASTM Test Methods. The data and brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified as "corrosive" and Sample No.002 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from 25 to 57 mg/kg. Because the chloride ion concentrations are less than 300 ppm, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations range from 41 to 130 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 6.9 to 7.6 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 350 to 370-mV. The redox potentials for both samples are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call JDH Corrosion Consultants, Inc. at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you required further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard

J. Darby Howard, Jr., P.E.
President

JDH/jdl

CERCO Analytical, Inc.

3942-A Valley Avenue, Pleasanton, CA 94566-4715 (925) 462-2771 Fax (925) 462-2775

FINAL RESULTS

Client:	Treadwell & Rollo	Date Sampled:	Not Indicated
Client's Project No.:	2869.01	Date Received:	10-Jul-2000
Client's Project Name:	Almaden Plaza	Date of Report:	26-Jul-2000
Authorization:	Transmittal dated 07-July-2000	Matrix:	Soil

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0007063-001	#2 B-3 @ 5'	370	6.9	-	950	-	57	130
0007063-002	#5 B-4 @ 20.5'	350	7.6	-	4,000	-	25	41

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	21-Jul-2000	21-Jul-2000	-	25-Jul-2000	-	21-Jul-2000	21-Jul-2000

* Results Reported on "As Received" Basis

Cheryl McMillen

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX J

PREVIOUS CONE PENETRATION TEST REPORT BY OTHERS
(Gregg Drilling and Testing)

GREGG

CONE PENETRTATION TESTING (CPT) SUMMARY

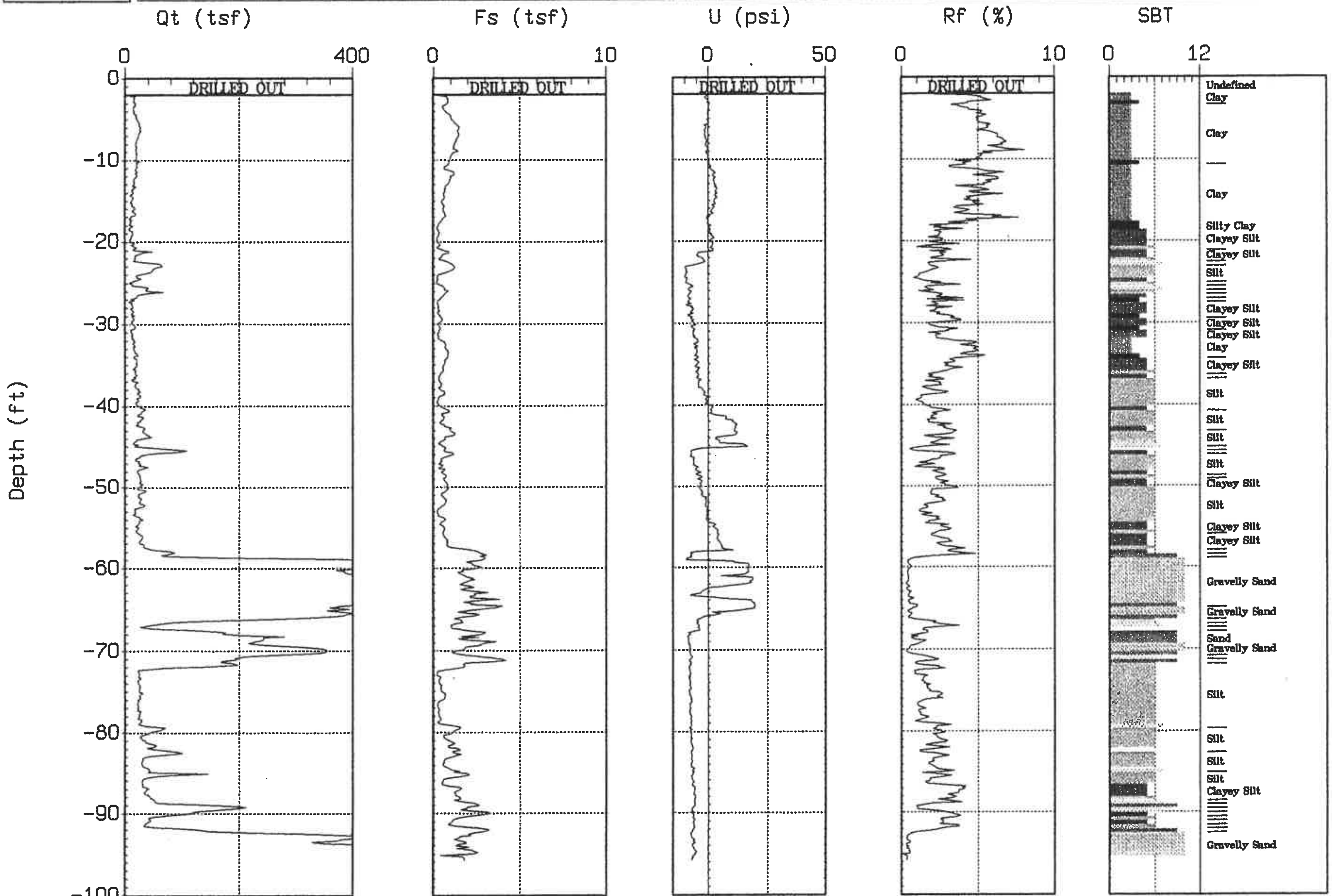
CLIENT: TREADWELL AND ROLLO
PROJECT: ALMADEN PLAZA SITE, SAN JOSE, CA
DATE: JULY 5-6, 2000

FILE NAME	HOLE LOCATION	DATE	CONE ID	DEPTH (ft)	COMMENTS
110C01A.COR	CPT-1	07.06.00	078	95.8	
110C02.COR	CPT-2	07.05.00	078	100.1	
110C03.COR	CPT-3	07.05.00	078	92.7	
110C04.COR	CPT-4	07.06.00	078	100.2	
110C05.COR	CPT-5	07.05.00	078	80.7	
110C06.COR	CPT-6	07.06.00	078	94.2	
110C07.COR	CPT-7	07.05.00	078	83.3	
110C08.COR	CPT-8	07.06.00	078	95.1	
110C09.COR	CPT-9	07.05.00	078	100.1	

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-1

Engineer: E. Banaag
Date : 07:06:00 11:19



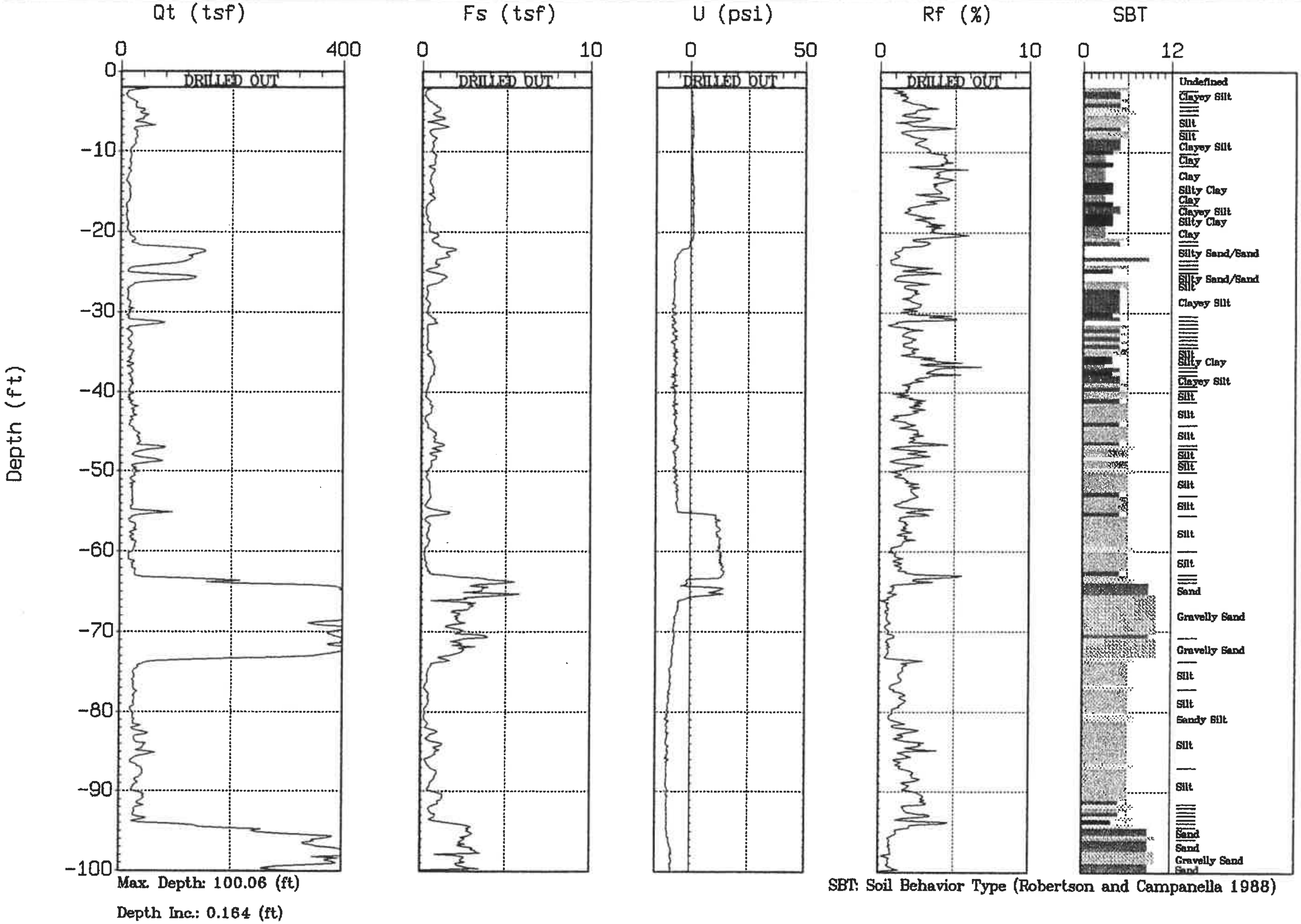
Max. Depth: 95.80 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-2

Engineer: E. Banaag
Date : 07:05:00 13:17

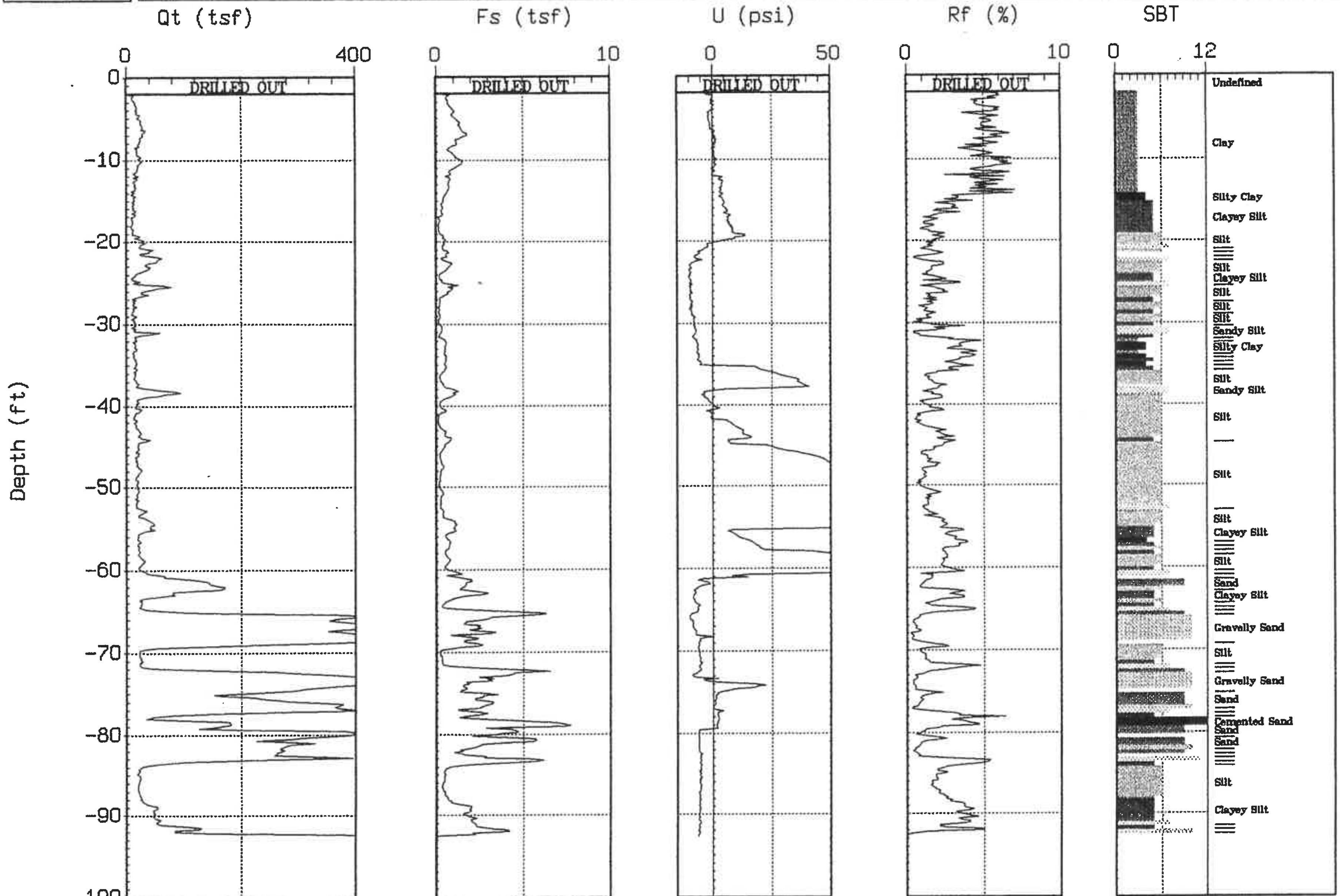


GREGG

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-3

Engineer: E. Banaag
Date : 07:05:00 11:18



Max. Depth: 92.68 (ft)
Depth Inc.: 0.164 (ft)

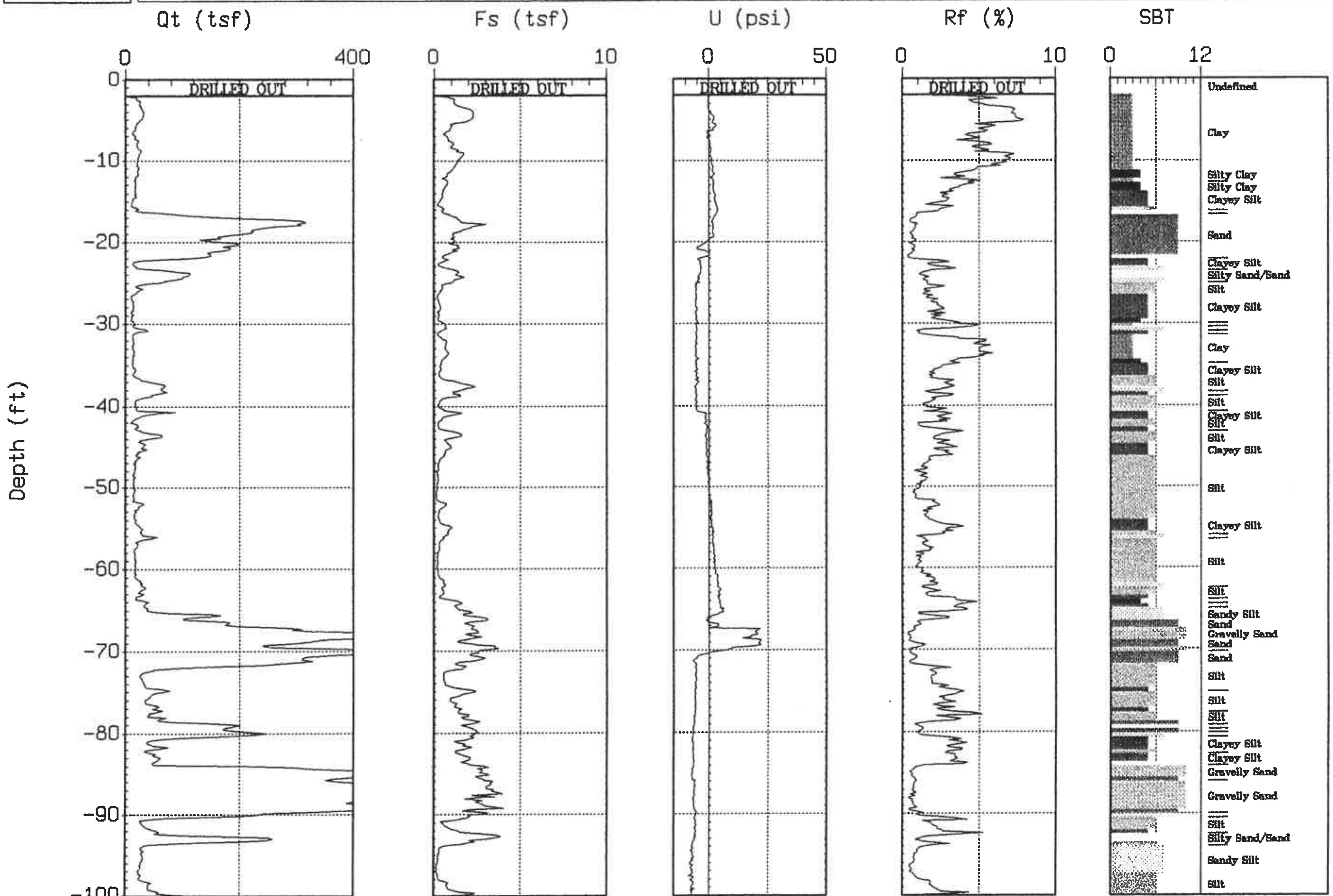
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-4

Engineer: E. Banaag
Date : 07:06:00 12:33



Max. Depth: 100.23 (ft)
Depth Inc.: 0.164 (ft)

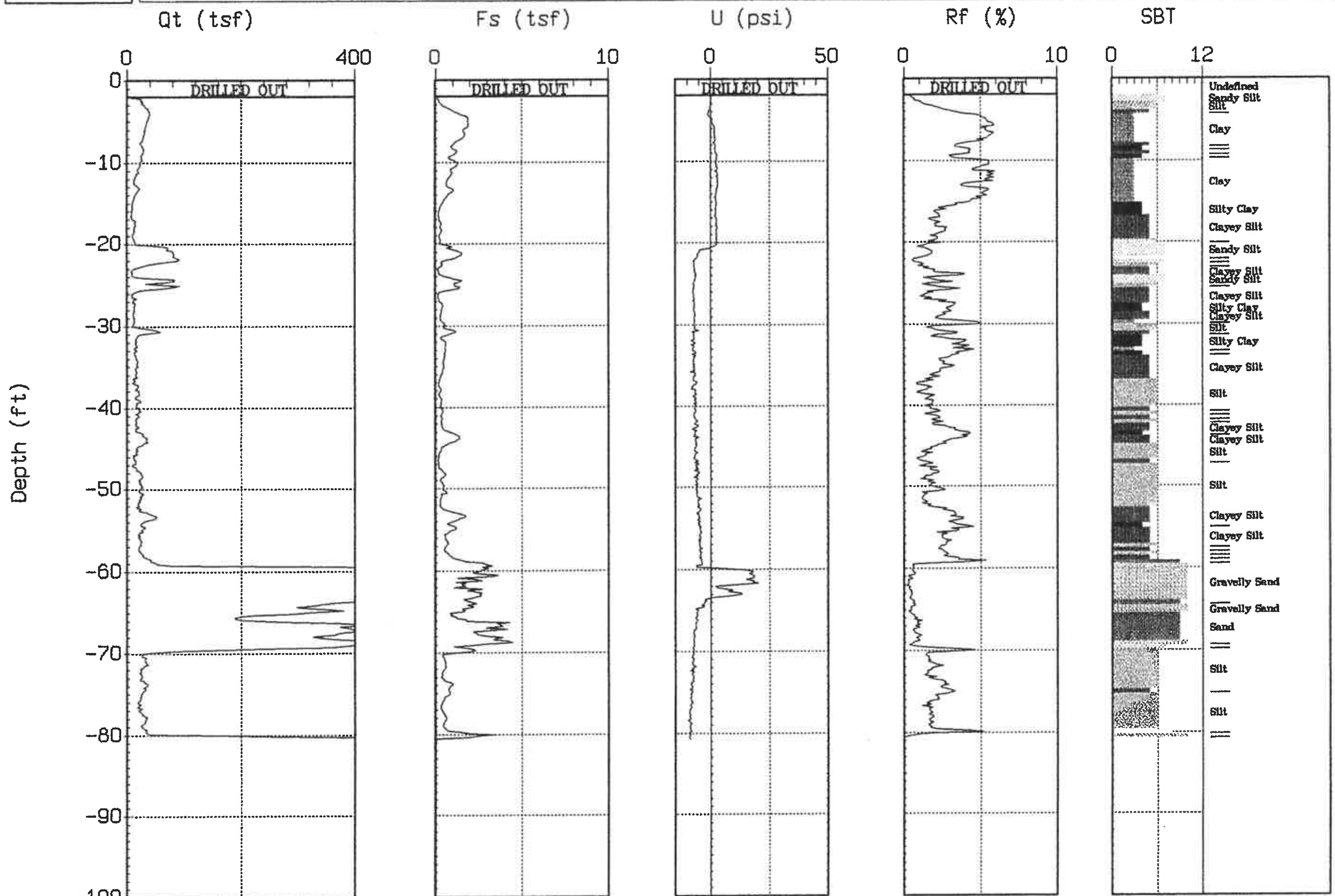
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-5

Engineer: E. Banaag
Date : 07:05:00 14:47



Max. Depth: 80.71 (ft)
Depth Inc.: 0.164 (ft)

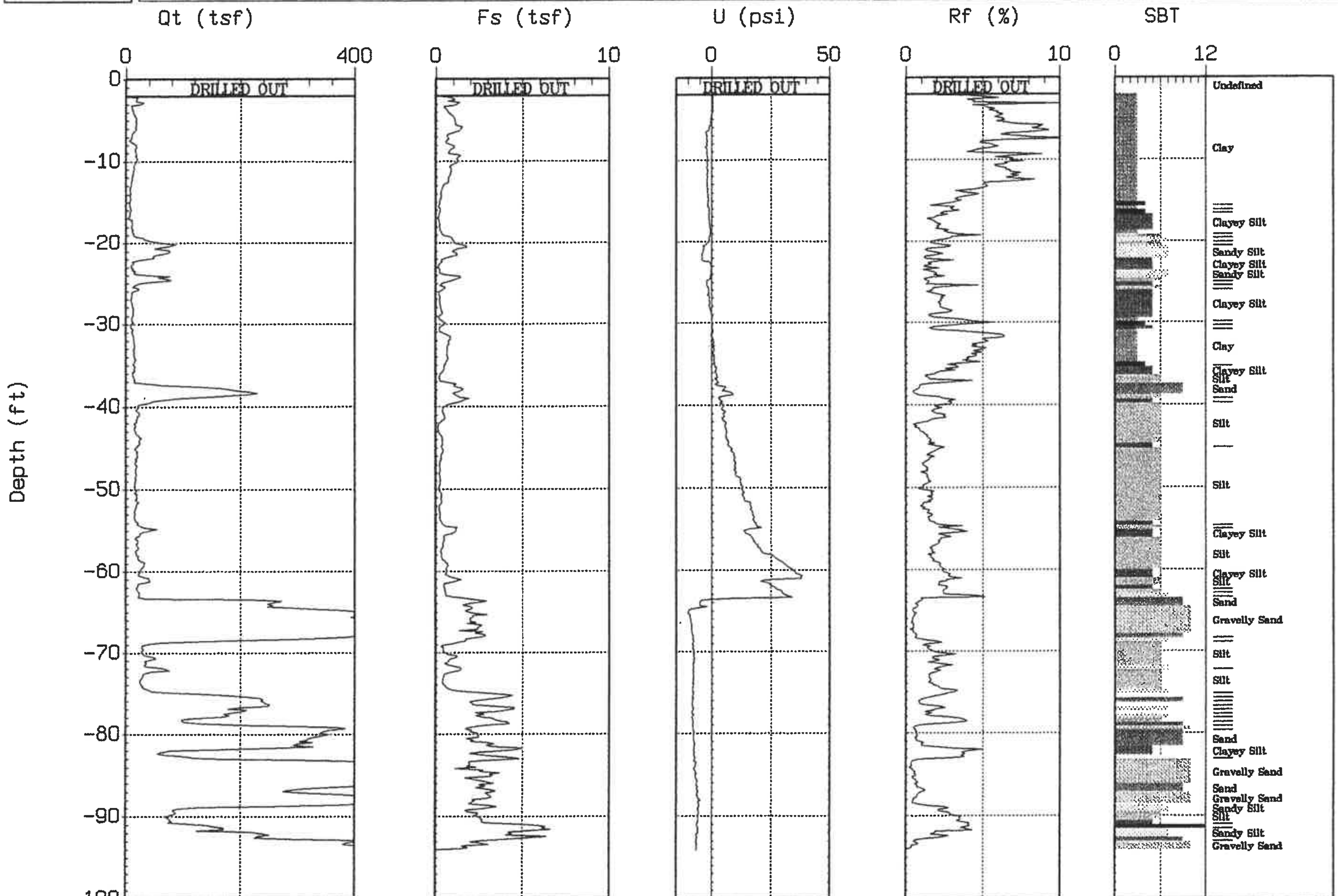
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-6

Engineer: E. Banaag
Date : 07:06:00 12:29



Max. Depth: 94.16 (ft)
Depth Inc.: 0.164 (ft)

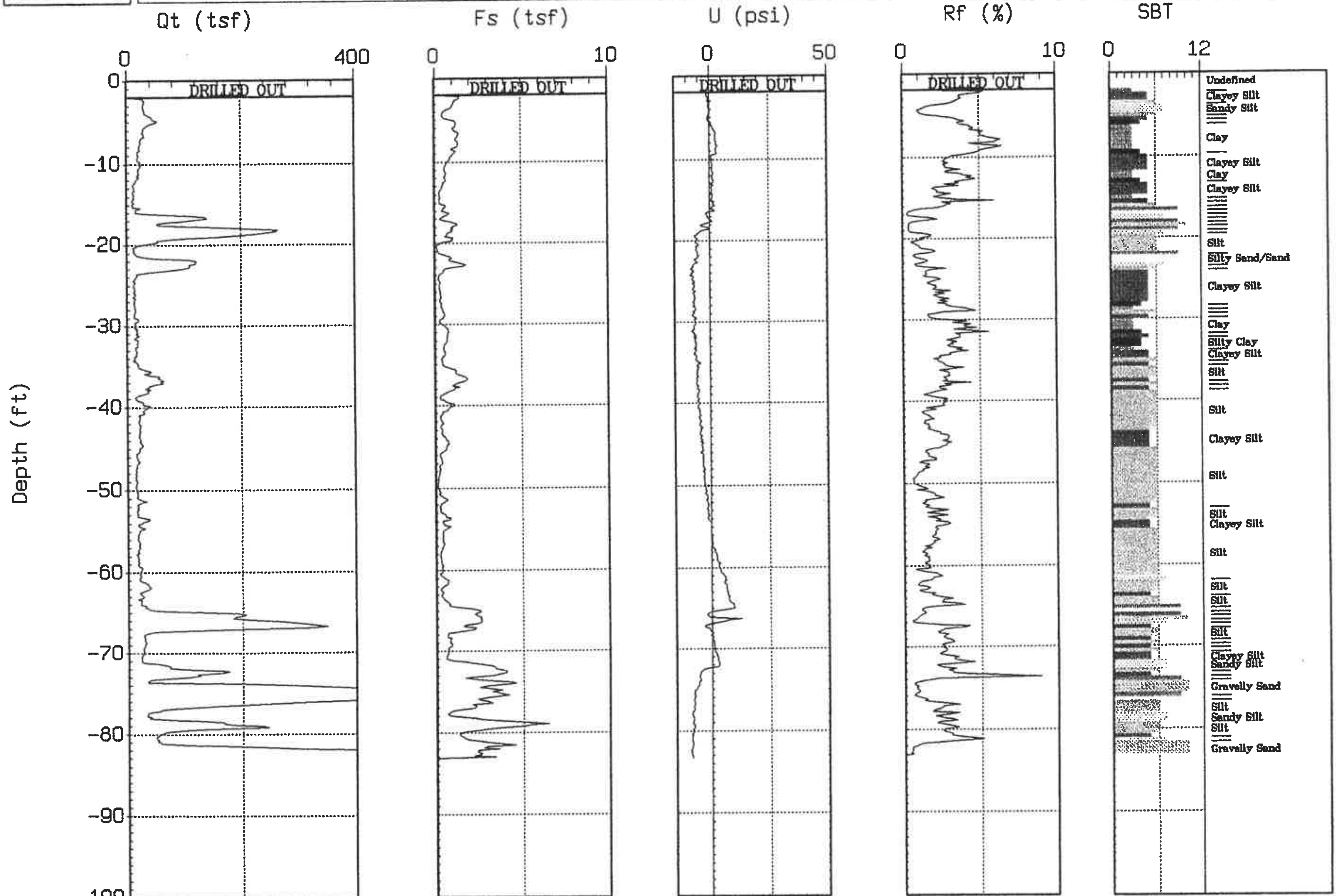
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-7

Engineer: E. Banaag
Date : 07:05:00 17:58



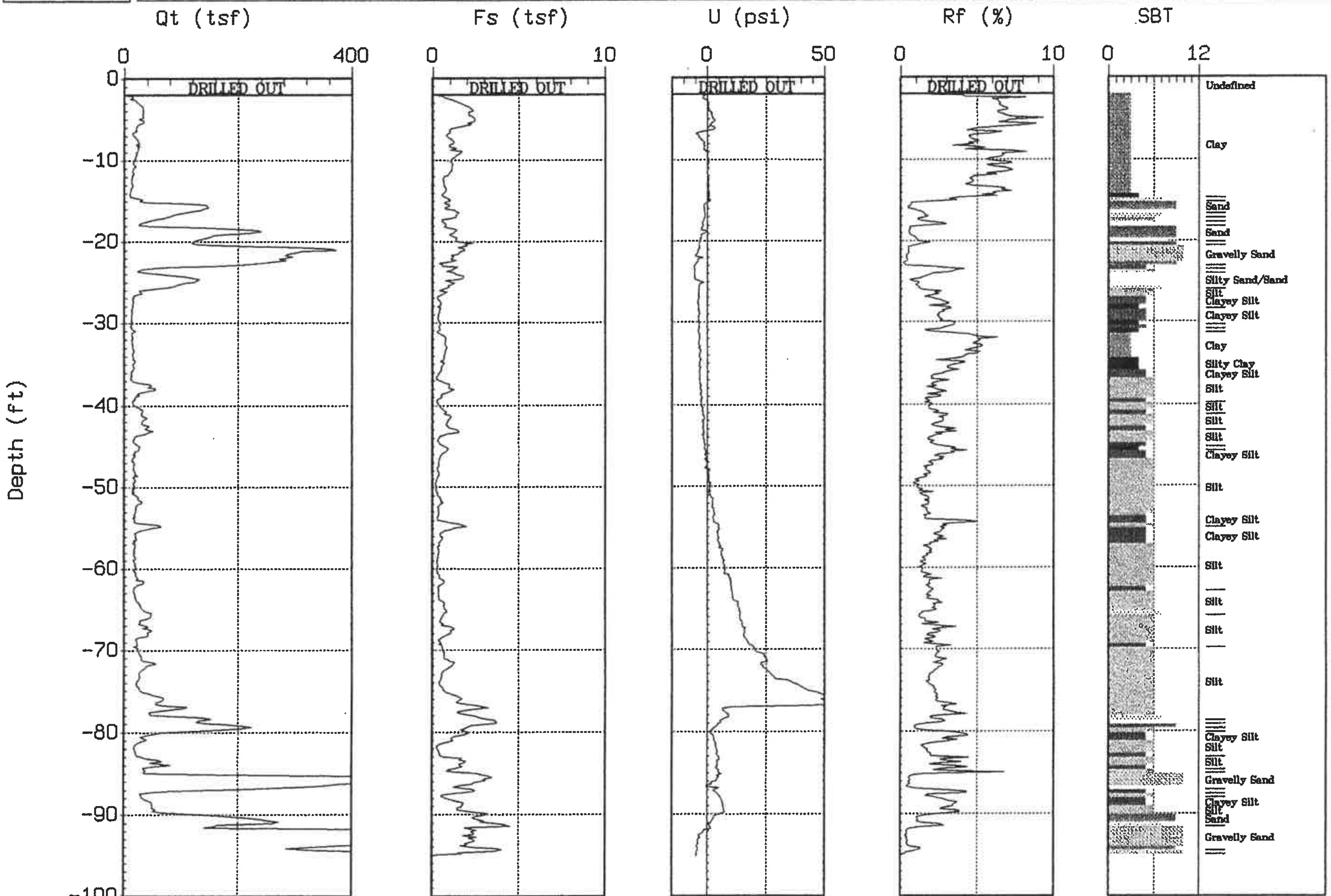
Max. Depth: 83.33 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-8

Engineer: E. Banaag
Date : 07:06:00 13:36



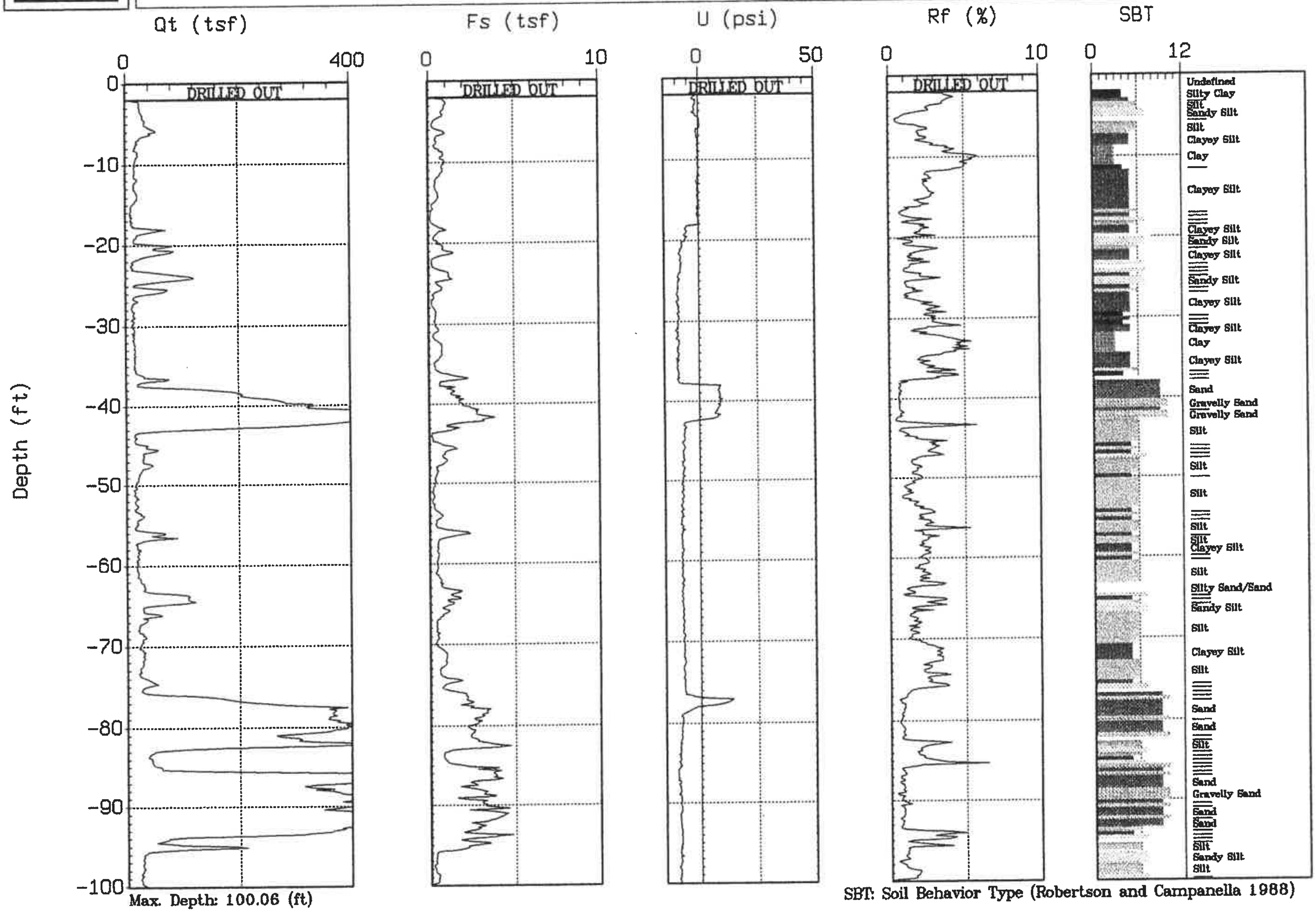
Max. Depth: 95.14 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-9

Engineer: E. Banaag
Date : 07:05:00 16:57



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

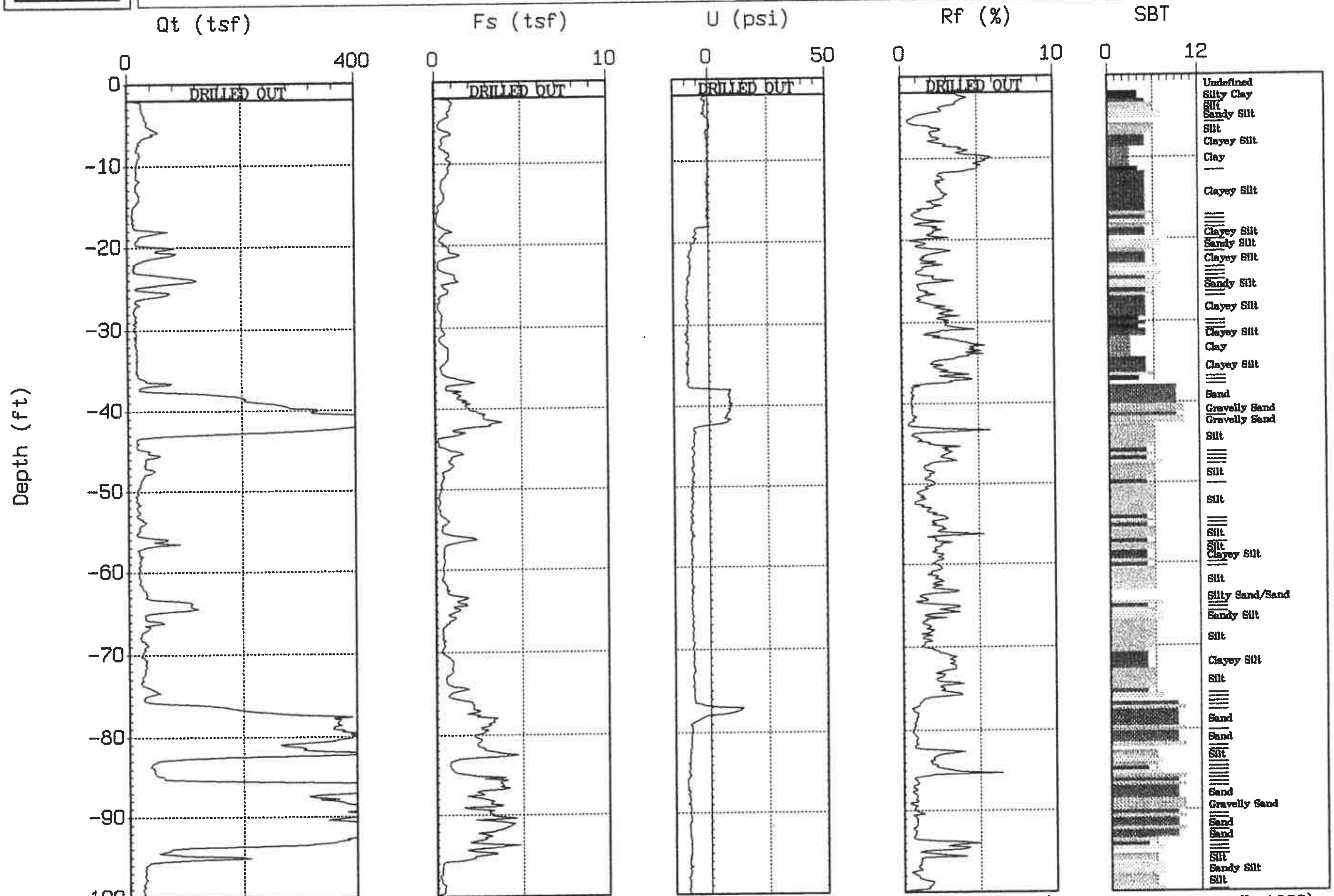
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

TREADWELL & ROLLO

Site: ALMADEN PLAZA
Location : CPT-9

Engineer: E. Banaag
Date : 07:05:00 16:57



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

ConeTec

Geotechnical and Environmental Site Investigation Contractors

ConeTec CPT Interpretations as of January 7, 1999 (Release 1.00.19)

ConeTec's interpretation routine should be considered a calculator of current published CPT correlations and is subject to change to reflect the current state of practice. The interpreted values are not considered valid for all soil types. The interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design. Reference to current literature is strongly recommended.

The CPT interpretations are based on values of tip, sleeve friction and pore pressure averaged over a user specified interval (typically 0.25m). Note that Qt is the recorded tip value, Qc, corrected for pore pressure effects. Since all ConeTec cones have equal end area friction sleeves, pore pressure corrections to sleeve friction, Fs, are not required.

The tip correction is: $Qt = Qc + (1\text{-}a) \bullet Ud$

where: *Qt* is the corrected tip load
Qc is the recorded tip load
Ud is the recorded dynamic pore pressure
a is the Net Area Ratio for the cone (typically 0.85 for ConeTec cones)

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (this can be obtained from CPT dissipation tests). The stress calculations use unit weights assigned to the Soil Behaviour Type zones or from a user defined unit weight profile.

Details regarding the interpretation methods for all of the interpreted parameters is given in table 1. The appropriate references referred to in table 1 are listed in table 2.

The estimated Soil Behaviour Type is based on the charts developed by Robertson and Campanella shown in figure 1.

Table 1 CPT Interpretation Methods

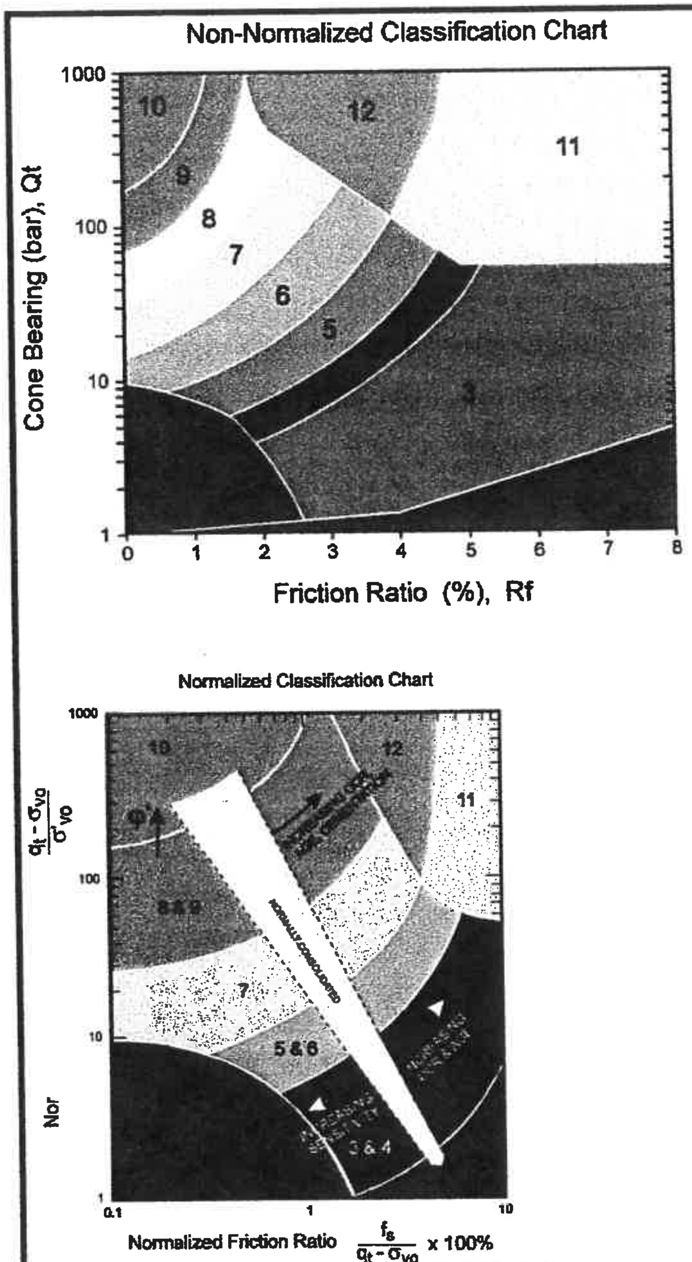
Interpreted Parameter	Description	Equation	Ref
Depth	mid layer depth		
AvgQt	Averaged corrected tip (Qt)	$AvgQt = \frac{1}{n}\sum_{i=1}^{n} Qt_i$	
AvgFs	Averaged sleeve friction (Fs)	$AvgFs = \frac{1}{n}\sum_{i=1}^{n} Fs_i$	
AvgRf	Averaged friction ratio (Rf)	$AvgRf = 100\% \bullet \frac{AvgFs}{AvgQt}$	
AvgUd	Averaged dynamic pore pressure (Ud)	$AvgUd = \frac{1}{n}\sum_{i=1}^{n} Ud_i$	
SBT	Soil Behavior Type as defined by Robertson and Campanella		1

CPT Interpretations

U.Wt.	Unit Weight of soil determined from: 1) uniform value or 2) value assigned to each SBT zone 3) user supplied unit weight profile		
TStress	Total vertical overburden stress at mid layer depth	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ where γ_i is layer unit weight h_i is layer thickness	
EStress	Effective vertical overburden stress at mid layer depth	$EStress = TStress - Ueq$	
Ueq	Equilibrium pore pressure determined from: 1) hydrostatic from water table depth 2) user supplied profile		
Cn	SPT N_{60} overburden correction factor	$Cn = (\sigma_v')^{-0.5}$ where σ_v' is in tsf $0.5 < C_n < 2.0$	
N_{60}	SPT N value at 60% energy calculated from Qt/N ratios assigned to each SBT zone		3
$(N1)_{60}$	SPT N_{60} value corrected for overburden pressure	$N1_{60} = Cn \bullet N_{60}$	3
$\Delta(N1)_{60}$	Equivalent Clean Sand Correction to $(N1)_{60}$	$\Delta(N1)_{60} = \frac{K_{SPT}}{1 - K_{SPT}} \bullet (N1)_{60}$ Where: K_{SPT} is defined as: 0.0 for FC < 5% 0.0167 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	7
$(N1)_{60cs}$	Equivalent Clean Sand $(N1)_{60}$	$(N1)_{60cs} = (N1)_{60} + \Delta(N1)_{60}$	7
Su	Undrained shear strength - Nkt is use selectable	$Su = \frac{Qt - \sigma_v}{N_{kt}}$	2
k	Coefficient of permeability (assigned to each SBT zone)		6
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{Qt - \sigma_v}$	2
Qtn	Normalized Qt for Soil Behavior Type classification as defined by Robertson, 1990	$Qtn = \frac{Qt - \sigma_v}{\sigma'_v}$	4
Rfn	Normalized Rf for Soil Behavior Type classification as defined by Robertson, 1990	$Rfn = 100\% \bullet \frac{f_s}{Qt - \sigma_v}$	4
SBTn	Normalized Soil Behavior Type (slightly modified from that published by Robertson, 1990. This version includes all the soil zones of the original non-normalized SBT chart - see figure 1)		4
Qc1	Normalized Qt for seismic analysis	$qc1 = qc \bullet (Pa/\sigma_v')^{0.5}$ where: Pa = atm. pressure	5
Qc1N	Dimensionless Normalized Qt1	qc1N = qc1 / Pa where: Pa = atm. pressure	

CPT Interpretations

ΔQc1N1	Equivalent clean sand correction	$\Delta qc1N = \frac{K_{CPT}}{1 - K_{CPT}} \bullet qc1N$ Where: K_{CPT} is defined as: 0.0 for FC < 5% 0.0267 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	5
Qc1Ncs	Clean Sand equivalent Qc1N	$qc1Ncs = qc1N + \Delta qc1N$	5
Ic	Soil index for estimating grain characteristics	$Ic = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$	5
FC	Fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $Ic > 3.5$ $FC=0$ for $Ic < 1.26$ $FC = 5\%$ if $1.64 < Ic < 2.6$ AND $Rfn<0.5$	8
PHI	Friction Angle	Campanella and Robertson Durunoglu and Mitchel Janbu	1
Dr	Relative Density	Ticino Sand Hokksund Sand Schmertmann 1976 Jamiolkowski - All Sands	1
OCR	Over Consolidation Ratio		1
State Parameter			9
CRR	Cyclic Resistance Ratio		7



Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	
8	4	and to silty sand
9	6	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

Figure 1 Non-Normalized and Normalized Soil Behaviour Type Classification Charts

Table 2 References

No.	Reference
1	Robertson, P.K. and Campanella, R.G., 1986, "Guidelines for Use, Interpretation and Application of the CPT and CPTU", UBC, Soil Mechanics Series No. 105, Civil Eng. Dept., Vancouver, B.C., Canada
2	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
3	Robertson, P.K. and Campanella, R.G., 1989, "Guidelines for Geotechnical Design Using CPT and CPTU", UBC, Soil Mechanics Series No. 120, Civil Eng. Dept., Vancouver, B.C., Canada
4	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27.
5	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of Sands and its Evaluation", Keynote Lecture, First International Conference on Earthquake Geotechnical Enginnering, Tokyo, Japan.
6	ConeTec Internal Report
7	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997
8	Wride, C.E. and Robertson, P.K., 1997, "Phase II Data Review Report (Massey and Kidd Sites, Fraser River Delta)", Volume 1 - Data Report (June 1997), University of Alberta.
9	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.

APPENDIX K

ENGEO PROJECT EXPERIENCE AND RESUMES

DIRIDON STATION – SAN JOSE GOOGLE VILLAGE (CONFIDENTIAL)
SAN JOSE, CALIFORNIA

A transit oriented district situated in the heart of Silicon Valley, the San Jose GOOGLE Village project includes approximately 50 acres of land with proposed development of approximately 10 million square feet of retail and commercial space along with residential units. The project includes office towers, a residential tower, and sizeable retail spaces that include shops and restaurants.

ENGEO is working with Google and Trammel Crow Company during due diligence, site acquisition and preliminary site planning phases of this project. We performed numerous Phase 1 Environmental Site Assessments, Phase 2 environmental soil, vapor and groundwater sampling, a geotechnical exploration and a site-specific seismic exploration. The massive amount of data collected for this project were presented in a Geographical Information System (GIS) portal developed by ENGEO. The GIS portal allows stakeholders and the project team to review and access site data in one location. The GIS portal also allows overlaying of different site data to help the team draw conclusions and make preliminary development plans.

Our LEED and hydrology staff provided valuable input on this project to achieve a high level of sustainability during design. Our ground source heat pump specialists provide consultation on energy saving systems and worked closely with the project architect and MEP to develop a feasible plan to utilize geothermal systems on the project.

CLIENT
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Senior Vice President
Trammel Crow Company
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KEY PERSONNEL
Ollie Van Rooyen
Janet Kan, GE, CEG, LEED AP
Jeff Fippin, GE
Jeff Adams, PhD, PE
Divya Bhargava, PE
Hue Williams
Uri Eliahu, GE

DURATION
On-going

ENGEO FEE
Confidential

SIZE
50+ acres
10 million sf of development space

DISCIPLINES
Geotechnical
Environmental
Ground Source Heat Pump

3607 KIFER ROAD
SANTA CLARA, CALIFORNIA

Located near the heart of Silicon Valley, this project encompasses approximately 1.3 acres and offers roughly 170,000 square feet of flexible office space. The prominent location provides access to nearby transit stations, including the Lawrence Caltrain Station. Situated at the intersection of Kifer Road and Lawrence Expressway, the site is also a part of the Lawrence Station Area Plan, a new mixed-use urban node of Caltrain.

ENGEO has been involved in the project since 2013, when the site was in the early stages of project conception. The property is located within an area previously owned and operated by Texas Instruments. Former nearby facility operations adversely effected the property and it is now identified as part of SUPERFUND site. Ongoing remediation efforts conducted by Texas Instruments has helped remediate the area, however current and future developments still face challenges. Further, the property is located in an area with a high groundwater table and liquefiable soils.

Our environmental staff has assisted the project by providing additional site characterization and management plans for construction practices. ENGEO has worked closely with the State Water Resources Control Board as well as local jurisdictions to ensure all necessary remediation steps were taken into account during design and construction phases.

Throughout the various development concepts, ENGEO has offered unique solutions to assist the development of this challenging parcel of land. The current office building utilizes a mat foundation combined with interconnected isolated footings. To accommodate relatively shallow liquefiable material, the foundation subgrade preparation includes a 2-foot overexcavation and placement of geogrid prior to backfilling with crushed rock.

CLIENT
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Bayview Development Group
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San Jose, CA 95113
(415) 536-0280
Ted McMahon
tedmcmahon@bayviewdg.com

KEY PERSONNEL
Ian McCreery, PE
Divya Bhargava, PE
Leroy Chan, GE
Ted Bayham, GE, CEG
Shawn Munger, CHG

DURATION
On-going

ENGEO FEE
Confidential

SIZE
1.3 acres
170,000 sf of office space

DISCIPLINES
Geotechnical
Environmental

TREASURE ISLAND / YERBA BUENA ISLAND REDEVELOPMENT
SAN FRANCISCO, CALIFORNIA

ENGEO is the Geotechnical Engineer of Record. The project provides a new, high-density, mixed-use community with a variety of housing types, a retail core, open space and recreation opportunities, on-site infrastructure, and public and community facilities and services.

At Yerba Buena Island, the geotechnical considerations and design features include: slope and foundation design issues associated with existing cut slopes and hillside fills; stability of existing retaining walls; slope stability issues associated with the steep perimeter slopes; slope stability issues associated with the slopes under and adjacent to the Treasure Island Road Viaduct.

In 2014, 2015, and 2016 ENGEO conducted the design-level geotechnical study for the first Major Phase of development, an approximately 171-acre parcel with a new ferry terminal, approximately 3,700 residential units, and 100 acres of parks and open space. The main geotechnical issues for the proposed development include: (1) seismic stabilization of the perimeter shoreline and causeway that connect Treasure Island to Yerba Buena Island, (2) mitigation of long-term static settlements under the development footprint due to the presence of Bay Mud, and (3) mitigation of liquefaction-induced settlement within the development footprint. ENGEO worked closely with the U.S. Navy to coordinate and permit our field exploration and in-situ testing.

CLIENT
Treasure Island Community Development (TICD)
703 Market, Suite 1800
San Francisco, CA 94103
(415) 536-0280

REFERENCE
Dustin Rieger
LENNAR Urban
(415) 995-1770

DURATION
2005 – present

CONSTRUCTION COST
~ $8 Billion

ENGEO FEE
$1.5 Million

SIZE
171 acres
8,000 residential units
240,000 sf commercial/retail
500 hotel rooms
100 acres parks, open space

DISCIPLINES
Geotechnical Engineering
Value Engineering
Environmental Engineering

ST. JAMES PARK
SAN JOSE, CALIFORNIA

The St. James Park project is located in downtown San Jose, and contains landscaping, hardscape, statues, and fountains. The proposed re-development of the City park will enhance this shared community area into a beautiful centralized piece of San Jose. Park improvements include an approximately 35-foot-tall performing arts pavilion, office, café, restrooms, fountain feature, and playground area with play structures.

ENGEO performed the geotechnical exploration for the project, and is continues to act as the Geotechnical Engineer of Record as the project progresses. As part of our scope, ENGEO performed a geotechnical exploration and developed grading, drainage, and foundation recommendations for design and construction Geotechnical considerations included liquefaction-induced settlement, load-induced settlement, corrosive soils, and expansive soils. In addition, testing of collected samples were performed to develop onsite stormwater infiltration opportunities and bioretention areas.

CLIENT
Haley Waterson
CMG Landscape Architecture
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KEY PERSONNEL
Bob Boeche, CEG
Greg Cubbon, GE, CEG
Jeanine Ruffoni, PE
Yanet Zepeda, PE

DURATION
2017
ENGEO FEE
$39,000

SIZE
8.0 acres

DISCIPLINES
Geotechnical
Stormwater

BROOKLYN BASIN
OAKLAND, CALIFORNIA

ENGEO is providing geotechnical engineering and stormwater consultation for this redevelopment of 65 acres of former Port of Oakland land adjacent to the Oakland Estuary and Jack London Square. The project will include an environmentally sustainable, mixed-use urban master plan with 3,100 residential units; 200,000 square feet of retail and commercial space; and 30 acres of parks, public trails and open space, plus new marinas and renewed wetlands. The project will consist of a combination of low-, mid- and high-rise construction and includes reusing a historic wharf structure founded on a combination of wood and concrete piles.

Geotechnical constraints include high seismicity, liquefiable sand, soft Bay deposits, and shoreline stability. The site needs to be raised several feet to address potential future sea-level rise, which will result in consolidation settlement of the Young Bay Mud. The high seismicity could also result in slope deformations along the water's edge due to the low strength of the Young Bay Mud. We have developed innovative approaches to both these effects that best fit the project constraints.

On this project, we worked closely with the marine structural engineer to evaluate the interaction of the structure and the waterside slope to determine if seismically induced slope movement would damage the structure and to develop a cost-effective mitigation for areas where the structure was threatened. We were able to develop an innovative approach through this close collaboration that was peer reviewed and externally reviewed by a panel of experts assembled by BCDC.

CLIENT
ZOHP
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Oakland, CA 94612
Patrick Van Ness
(510) 251-9270

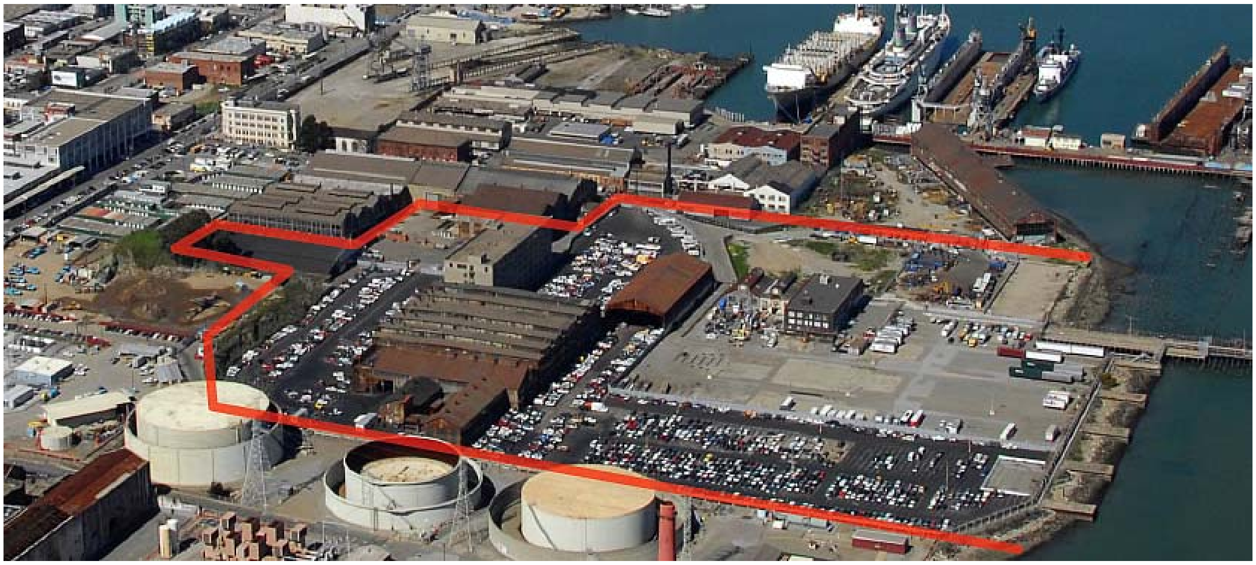
KEY PERSONNEL
Jeff Fippin, GE
Pedro Espinosa, GE
Uri Eliahu, GE

DURATION
2013 – present

ENGEO FEE
$900,000

SERVICES
Geotechnical Engineering
Construction Testing and Observation
Construction Stormwater Consultation and Monitoring

PIER 70
SAN FRANCISCO, CALIFORNIA



The Pier 70 Special Use District consists of an approximately 35-acre area. Two development areas constitute the SUD Site – the 28-Acre Site to be developed by Forest City and the Illinois Parcels to be developed by others. Other sub-districts include The Cove, BAE Ship Repair, and the Historic Core.

ENGEO is the geotechnical engineer for the 28-Acre Site (Site) and its related improvements. The Project Site is generally located between 20th Street, Michigan Street, 22nd Street, and San Francisco Bay, and includes a number of Port-owned parcels. The Project includes offsite roadway improvements for 20th, 21st, and 22nd Streets west of the Project Site up to Illinois Street, as well as offsite facilities such as the combined sewer pump station and potentially other district scale utility facilities located just outside of the Project Site boundary.

The Project will include a mixed-use land development program that includes residential, commercial office, retail, arts, light industrial and open-space uses. It is anticipated that the Project will be developed in three phases.

The major geotechnical constraints at the site is the existing underground fills, soft compressible soils, seismic lateral instability at the shoreline, rock excavations and naturally occurring asbestos (NOA).

CLIENT
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KEY PERSONNEL
Pedro Espinosa, GE
Ollie Van Rooyen, PE
Jeff Fippin, GE
Uri Eliahu, GE

DURATION
On-going

ENGEO FEE
Confidential

SIZE
28 Acres

DISCIPLINES
Geotechnical

OCEANWIDE CENTER
SAN FRANCISCO, CALIFORNIA

The Oceanwide Center project consists of two mixed-use towers – the 605-foot Mission Street Tower accommodating a hotel and residences, and an 850-foot office and residential tower along First Street. Both reflect the existing scale of the area and provide a significant amount of new hotel, office, and residential spaces in this downtown neighbourhood.

ENGEO is working with General Contractor to provide construction support services and help resolve foundation-related challenges. ENGEO's scope involves utilizing finite element modelling software, PLAXIS 3D, to help determine the mechanisms at play during the initial construction phases and excavation operations.

Given ENGEO's expertise with high-rise design and construction forensics, we were brought on board to help determine the cause of adjacent structure settlement, and provide recommendations for possible solutions.

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KEY PERSONNEL
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Jeff Fippin, GE, QSD
Todd Bradford, PE
James Allen, PG
Maggie Parks, PhD, EIT
Bahareh Heidarzadeh, PhD

DURATION
2018 – present

ENGEO FEE
$180,000

SERVICES
Geotechnical
Construction Forensics

STOCKTON COURTHOUSE
STOCKTON, CALIFORNIA

ENGEO prepared the geotechnical report and observed the foundation construction for the State of California courthouse in Downtown Stockton. This 14-story building is currently the tallest in Stockton. The steel-framed building has a 2-level basement with 12 stories above grade. Working closely with the structural engineer, Thornton and Tomisetti, ENGEO observed and analyzed six pile load tests for potential use of cost-saving steel drilled displacement piles and subsequently developed the final pile recommendations. ENGEO documented and verified the pile foundation installation on behalf of the Administrative Office of the Courts. Following foundation construction, ENGEO performed geotechnical testing and observation services during the remainder of the site work. ENGEO also developed site-specific response spectra in accordance with the California Building Code and ASCE 7 and provided geotechnical recommendations for retaining walls, subdrains, seismic lateral earth pressures, rigid and flexible pavement sections, and crane lateral earth pressures on basement walls.

CLIENT
James Tully
NBBJ
223 Yale Avenue North
Seattle, WA 98109

KEY PERSONNEL
Jeff Fippin, GE
Pedro Espinosa, GE
Uri Eliahu, GE

DURATION
2009 – present

ENGEO FEE
$325,000

SERVICES
Geotechnical Engineering
Value Engineering
Construction Testing and Observation

POTRERO POWER PLANT
SAN FRANCISCO, CALIFORNIA

The Potrero Power Plant site comprises 21 acres of former industrial property located along San Francisco's southern waterfront. The site was first developed in the 1870s as a Manufactured Gas Plant. Other industrial uses at the site included sugar refining, barrel manufacturing, fuel oil storage and industrial shipping associated with the since-removed wharf along the eastern shoreline.

About half of the site is east of the original shoreline. This land was reclaimed by excavation into the adjacent hillside and placing the rocky fill into the Bay. Past environmental investigations have identified chemicals of potential environmental concern (COPECs) associated with fill placement and past industrial operations at the site. We understand that in-situ stabilization/solidification has been proposed along portions of the eastern shoreline as part of future remediation activities.

Development of the site is planned to include a mix of residential, retail, industrial and office use with a mix of new construction and retrofit of some of the existing historic structures at the site. Geotechnical constraints include liquefiable fill, shallow hard rock and soft and compressible Young Bay Mud.

We performed a preliminary geotechnical exploration of the site and identified an additional hazard of potential shoreline slope instability. We have preliminarily identified Cement Deep Soil Mixing as a cost-effective measure to mitigate this risk. We also provided preliminary foundation recommendations appropriate for this site underlain by variable soil conditions and mitigation measures for long-term settlement of compressible soil due to planned fill to address sea-level rise.

CLIENT
NRG Potrero Development, LLC
410 China Basin Street
San Francisco, CA 94158Mr. Seth Hamalian
(415) 355-6600

DURATION
2015-present

ENGEO FEE
$100,000

SIZE
2,000 housing units
1.8 million square feet of commercial space
400,000 square feet of retail and manufacturing space
9 acres of parks

DISCIPLINES
Geotechnical Engineering

HUNTERS POINT SHIPYARD PHASE 2/ CANDLESTICK POINT REDEVELOPMENT
SAN FRANCISCO, CALIFORNIA

Together, the Hunters Point Shipyard Phase 2 and Candlestick Point areas comprise over 700 acres of waterfront land along San Francisco's southeastern shore. The integrated development project is designed to provide over 12,000 high-density residential units, over 300 acres of new waterfront parks, including a new "Crissy Field of the South," approximately 885,000 square feet of neighborhood and destination retail and entertainment space and 2.5 million square feet of commercial space oriented around a "green" science and technology campus targeting emerging technologies. Investigations for the site included drilling borings over water, in contaminated subsurface conditions, drilling inside Candlestick Park, drilling in an active housing development, and coordination with the Navy and the City of San Francisco.

Geotechnical constraints include shoreline stability, liquefiable sands, high ground shaking, compressible Young Bay Mud deposits, and existing improvements and utilities. The structures that we are designing at this site need to be designed with foundations that address both the compressible Young Bay Mud and liquefiable fill. The site is also being raised to address potential sea-level rise, which results in consolidation of the underlying Young Bay Mud; we have developed a surcharge program to cost-effectively reduce long-term settlement in the streets and other areas of improvement.

The projects also are underlain by shallow bedrock within the portions of the site landward of the historic shoreline. This bedrock is highly variable in rock quality. Construction of the CP Retail Center requires excavation of up to 50 feet into the rock and construction of a soil nail wall in the bedrock. Our geologists have mapped the rock conditions and assisted in design of the retaining wall. Development of the first phase of Hunters Point included hillside grading within the bedrock formation.

CLIENT
CP Development Co.
c/o FivePoint
One Sansome Street
Suite 3200
San Francisco, CA 94111

Mark Luckhardt
(415) 920-3482

KEY PERSONNEL
Jeff Fippin, GE
Leroy Chan, GE
Uri Eliahu, GE
Brian Flaherty, CEG

DURATION
2008 – present

CONSTRUCTION COST
$9 Billion

SIZE
300 acres
12,000 residential units
2.5 million sf commercial
885,000 sf retail &
entertainment

SERVICES
Geotechnical Engineering
Construction Testing and
Observation
Construction Stormwater
Construction Dust Monitoring

SACRAMENTO COMMONS
SACRAMENTO, CALIFORNIA

ENGEO reviewed nearby subsurface data obtained from public records search with the City of Sacramento and nearby ENGEO projects and prepared a feasibility level geotechnical report. As part of the feasibility level report, ENGEO developed preliminary foundation recommendations for the project, which included deep foundation alternatives for the high-rise structures and mat foundations with ground improvement for the mid-rise and parking structures. The approximately 11.17-acre site will consist of Parcels 1, 2A, 2B, 3, 4A and 4B. The proposed improvements will likely consist of various high-rise residential, mid-rise residential, condominium, hotel, parking and retail structures including the construction of three 7-story mid-rise structure, one 22-story high-rise structure and one 24-story high-rise structure. Long- and short-span parking structures up to five stories are also considered. The Tentative Subdivision Map indicates the proposed land use will result in approximately 1,100 to 1,400 apartment homes, up to 300 condominiums, 200 to 400 hotel rooms, 35,000 to 63,000 square feet of retail space, and 37,000 to 59,000 square feet of live/work space. One structure may have an elevator shaft that would extend one level below the ground surface, with the other structures at grade. Improvements will also include paved streets, parking, drive lanes, flatwork, and underground utilities. The site is currently occupied primarily by multi-family apartments that are planned to be demolished and the 15-story Capital Towers building that is to remain.

CLIENT
Dave Eadie
KW CapTowers, LLC
18401 Von Karman, Suite 350
Irvine, CA 92612
(949) 640-0050
deadie@kennedywilson.com

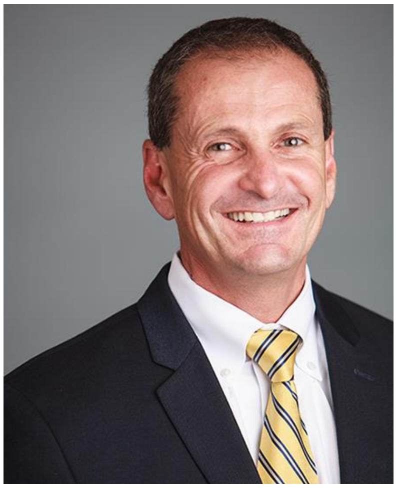
KEY PERSONNEL
Mark Gilbert, GE, QSD
Jonathan Boland, GE, QSD
Nick Broussard, GE
Abram Magel, PE

DURATION
2013 – 2016

ENGEO FEE
$145,000

SIZE
11.17-acres
3 7-story mid-rise structures
22-story high-rise
24-story high-rise
5-story parking structure

DISCIPLINES
Geotechnical



EDUCATION

BS Civil Engineering University of California, Berkeley 1981

EXPERIENCE

Years with ENGEO: 31
Years with Other Firms: 5

REGISTRATIONS & CERTIFICATIONS

Professional Engineer, CA 39522
Professional Engineer, NV 12441
Geotechnical Engineer, CA 2166

SPECIALIZATIONS

- Compressible Soils
- Construction Observation
- Creek Stabilization/Restoration
- Earth Dam Design and Safety Evaluation
- Earth Retaining Structures
- Excavation and Shoring
- Foundation Design
- Geologic Hazard Abatement Districts (GHADs)
- Grading Project Management
- Hillside Grading
- Landslide Investigations and Repairs
- Levee Analyses
- Slope Stability
- Subgrade Stabilization
- Water Quality Studies
- Water Resources

AFFILIATIONS

ASCE – American Society of Civil Engineers

URI ELIAHU, GE

President

As President of ENGEO, Uri promotes technical excellence and extraordinary client service throughout the firm. Under his leadership, ENGEO has become California's consultant of choice for master-planned, mixed-use development, large-scale earthwork, transportation, urban infill and redevelopment of Brownfields, industrial sites and military bases. Uri is a Civil Engineering graduate from the University of California at Berkeley, and is a Registered Geotechnical Engineer in California and a Registered Civil Engineer in California and Nevada. He is a Founding Director of the California Association of Geologic Hazard Abatement Districts (GHADs) and its current President.

In 2009, Uri was selected Civil Engineer of the Year by the American Society of Civil Engineers and in 2008, he was voted Businessman of the Year by the San Ramon Chamber of Commerce.

Uri has evolved into a leading expert of entitlement and regulatory permitting processes. During his career, Uri has lent his expertise to a wide range of complex projects in a number of settings. He has developed and fostered close relationships with a number of decision-making officials in many local, state, and federal jurisdictional agencies. Although some of these past projects have included a range of contentious or controversial issues, he has consistently been able to deftly navigate potentially prohibitive technical and political constraints, resulting in timely, cost-effective delivery of project entitlements. Uri is a trusted advisor to a vast group of public and private clients and colleagues.

Select Project Experience

Pier 70—San Francisco, CA

Group Leader. Uri provided Principal oversight. ENGEO is the geotechnical engineer for the redevelopment of this industrial site. Pier 70 is located on the east side of Illinois Street between 20th and 22nd streets. The site includes a mix of vacant land, deteriorating buildings and storage and staging areas that restrict public access to the waterfront. For more than a century, the site was dedicated to the shipbuilding and manufacturing trades. Considered the center of heavy industry in the Western U.S. for decades, the site began industrial operations in the 1800s, with ships built there as far back as the Gold Rush. The redevelopment project, led by developer Forest City, proposes to build nearly 2,000 new homes, including 600 for middle- and low-income residents, as well as light manufacturing, retail space and nine acres of waterfront parks on the historic site. The Pier 70 development marks the first time San Francisco voters were asked to approve a height-limit increase along the waterfront.

Treasure Island—San Francisco, CA
Group Leader. Uri provides Principal oversight. Development plans for Treasure Island include 8,000 residential unit, 235,000 square feet of retail space, approximately 400 to 500 new hotel rooms, a marina, adaptive reuse of historic structures, and the creation of a major outdoor space. Approximately 85 percent of the development footprint on Treasure Island will be occupied by low-rise structures up to 5-stories in height; the balance will comprise mid- and high-rise buildings that will be supported on deep foundations. Ferry dock and breakwater will be constructed facing the San Francisco waterfront.

Hunters Point Shipyard Redevelopment, 'Parcel A'—San Francisco, CA
Group Leader. Uri provided Principal oversight. The 70-acre project includes 1,800 residential units, approximately 25 acres of parks and open space, limited retail, and supporting infrastructure and roadways. Site preparation included construction of terraced soil nail walls and mechanically stabilized earth walls, geotechnical remediation of 13 landslides totaling over 500,000 cubic yards of soil, and project grading totaling nearly 1.5 million cubic yards.

Lucas Museum of Narrative Art—Los Angeles, CA
Group Leader. Uri provided Principal oversight. ENGEO is the geotechnical and environmental engineer of record for the Lucas Museum of Narrative Art that will house the private art and memorabilia collections of famed filmmaker, George Lucas and his wife, Mellody Hobson. The Museum will be constructed on two state-owned parking lots on Vermont Avenue south of Exposition Boulevard. Construction of the Museum is expected to take approximately three years, beginning in early 2018 and finishing in 2021. The Museum of Narrative Art will be a five-story, 115-foot-tall building. Nearly one-third of the proposed building’s 290,000 sf will be dedicated to gallery space, with other program elements including a movie theater, a lecture hall, a library, a restaurant and several digital classrooms. A publicly accessible green roof terrace will cap the building, while a 2,425-space parking complex will be located underneath. The new museum and surrounding 11-acre public space is set to revitalize Los Angeles’ Exposition Park.

Landings Google Campus—Mountain View, CA
Group Leader. Uri provided Principal oversight of the geotechnical exploration, data analysis and provided geotechnical recommendations. The approximately 19-acre site is part of the main Google Mountain View Campus and is occupied by several existing buildings, asphalt-concrete paved parking areas, trees, and associated landscaping. The site will be redeveloped to include one five-story, approximately 800,000-square-foot office structure with one to two levels of below-grade parking, landscaping and landscape structures, planned fill of up to 11 feet, utilities and other infrastructure improvements, paved streets, parking, and drive lanes, geothermal systems, and widening and slope reconfiguration of the Permanente Creek.

301 Mission High Rise, Causation and Structural Retrofit—San Francisco, CA
Group Leader. Uri provided Principal oversight. The Millennium Tower is located at 301 Mission Street in San Francisco, California. Construction on the tower began in 2005 and was completed in 2009. The building consists of two towers: one 58-story structure and one 12-story tower connected via an atrium. The Millennium Tower is founded on piles that are approximately 60 feet long, go through the fill and soft sediments and derive resistance within the dense Colma sands.

It has been the experience in the San Francisco Bay Area that buildings as tall as 40 stories founded on piles on the dense Colma sands perform adequately. This is because the underlying older bay deposits have been subjected to similar loads in the past. However, the old bay clay deposits did not perform as expected under the loads of a near-60-story, reinforced-concrete building, the heaviest in the western US.

ENGEO assisted in evaluating the causes of the settlement and tilt. We reviewed design documentation for the existing foundation and for excavation and dewatering of surrounding projects built after the tower construction. We constructed a 3-dimensional subsurface model and performed settlement and deformation analyses.

Brooklyn Basin—Oakland, CA
Group Leader. Uri provided Principal oversight. ENGEO is providing geotechnical engineering and stormwater consultation for this redevelopment of 65 acres of former Port of Oakland land adjacent to the Oakland Estuary and Jack London Square. The project will include an environmentally sustainable, mixed-use urban master plan with 3,100 residential units; 200,000 square feet of retail and commercial space; and 30 acres of parks, public trails and open space, plus new marinas and renewed wetlands.

The project will consist of a combination of low-, mid- and high-rise construction and includes reusing a historic wharf structure founded on a combination of wood and concrete piles. Geotechnical constraints include high seismicity, liquefiable sand, soft Bay deposits, and shoreline stability. The site needs to be raised several feet to address potential future sea-level rise, which will result in consolidation settlement of the Young Bay Mud. The high seismicity could also result in slope deformations along the water's edge due to the low strength of the Young Bay Mud. We have developed innovative approaches to both these effects that best fit the project constraints.

On this project, we worked closely with the marine structural engineer to evaluate the interaction of the structure and the waterside slope to determine if seismically induced slope movement would damage the structure and to develop a cost-effective mitigation for areas where the structure was threatened. We were able to develop an innovative approach through this close collaboration that was peer reviewed and externally reviewed by a panel of experts assembled by BCDC.

Oak Knoll Naval Hospital—Oakland, CA
Group Leader. Uri provided Principal oversight. ENGEO's services have included preparing a design-level geotechnical report for the entire Oak Knoll site, providing plan review services, and developing earthwork and construction cost estimates. ENGEO has also consulted on establishing a Geologic Hazard Abatement District (GHAD) for the project. The project consists of a 192-acre hillside development including housing, retail, children's services, and a Native American cultural center.

The project includes the use of various shallow and deep foundation systems, pile-supported bridges, earth retaining structures, liquefaction potential mitigation, and corrective grading. The plan is to demolish the 225-bed hospital and create a self-sufficient community, with 800 residential, office, and retail spaces. There will also be 37 acres of open space and recreation facilities for children and adults.

EDUCATION

MS Civil and Environmental Engineering University of California, Berkeley 2006

BS Civil and Environmental Engineering University of California, Berkeley 2004

EXPERIENCE

Years with ENGEO: 11
Years with Other Firms: 3

REGISTRATIONS & CERTIFICATIONS

Professional Engineer, CA 71540

Geotechnical Engineer, CA 2954

SPECIALIZATIONS

- Compressible Soils
- Deep Foundations
- Earth Retaining Structures
- Excavation and Shoring
- Foundation Design
- Geosynthetic Materials
- Geotechnical/Geologic Instrumentation
- High-Rise Structures
- Levee Analyses
- Liquefaction Analyses
- Port and Harbor Facilities
- Seepage Evaluation
- Seismic Retrofit
- Seismic Spectra Development
- Slope Stability
- Soil Structure Interaction
- Subgrade Stabilization
- Transportation Design
- Tunneling

PEDRO J. ESPINOSA, GE

Principal

Pedro is an experienced engineer who has worked on many high-profile projects throughout California. He specializes in complex geotechnical explorations, seismic design, earthquake engineering, foundation design, ground improvement, elevated structures, transportation projects, waterfront projects and deep foundations. Pedro is the lead geotechnical engineer for ENGEO's work at Treasure Island in San Francisco, the new Firestone Blvd. bridge over the San Gabriel River in Norwalk, and the Lucas Museum in Los Angeles, among many other projects.

SELECT PROJECT EXPERIENCE

Lucas Museum of Narrative Art at Exposition Park—Los Angeles, CA
Associate Engineer. Pedro performed ground motion studies including ground motion selection, modification, and scaling, site response, and seismic analyses. The Lucas Museum of Narrative Art will be constructed on two state-owned parking lots on Vermont Avenue south of Exposition Boulevard. Construction of the Museum is expected to take approximately three years, beginning in January 2018 and finishing in 2021. ENGEO is the geotechnical and environmental engineer of record for this project that will house the private art and memorabilia collections of famed filmmaker, George Lucas and his wife, Mellody Hobson. The Museum of Narrative Art will be a five-story, 115-foot-tall building. Nearly one-third of the proposed building's 290,000 sf will be dedicated to gallery space, with other program elements including a movie theater, a lecture hall, a library, a restaurant and several digital classrooms. A publicly accessible green roof terrace will cap the building, while a 2,425-space parking complex will be located underneath. The new museum and surrounding 11-acre public space is set to revitalize Los Angeles' Exposition Park.

Caribbean 100 and 200 Tech Campus—Sunnyvale, CA
Seismic Analysis Senior Reviewer. Pedro reviewed the site-specific seismic analysis and provided recommendations regarding non-ergodic seismic site response. The Caribbean 100 and 200 campus will be developed with two five-story office buildings, a parking garage and a central utility plant. The office buildings are architecturally outstanding, both with continuous green roofs and designed to receive abundant natural light.

AFFILIATIONS
ASCE American Society of Civil Engineers

PUBLICATIONS
Conference Papers
Espinosa, P.J., Heidarzadeh, B., Pestana, J., Bray, J., Vahdani, S., "Seismic Deformation Analyses of the Existing Shoreline at Treasure Island," 3rd International Conference on Performance-based Design in Earthquake Geotechnical Engineering (PBD-III), Vancouver, BC, July 2017.

Encinal Terminals—Alameda, CA
Associate Engineer. Pedro provided technical review of the seismic analysis and shoreline stability evaluation. The Encinal Terminal site lies along the Oakland Estuary on the northern side of Alameda. The proposed site development consists of a combination of podium-type and townhouse-type residential buildings. The site was marshland that was reclaimed in the 1920s for use as a ship terminal; more recently the site was used for storing shipping containers. An approximately 1,500-foot-long wharf forms the western shoreline of the approximately 25-acre site. The wharf wraps around the site on the northern boundary and extends another 500 feet along the northern shoreline. The wharf was constructed in phases between the 1920s and 1960s and consists of concrete and timber decks supported by concrete and timber piles. The site is underlain by non-engineered fill and soft, compressible Young Bay Mud. These geotechnical conditions result in potential shoreline instability during an earthquake and settlement from new fill and building loads. To assess the shoreline stability, we performed a combination of analyses including limit equilibrium, and 1-dimensional and 2-dimensional time-history combined with Newmark-type analyses. Our findings indicated that the potential displacement during seismic loading is excessive, and we developed ground improvement solutions, including buttressing the shorelines of the project with deep soil mixing.

Treasure Island Sub- Phase 1A Geotechnical Services—San Francisco, CA
Associate Engineer. Pedro provided seismic analysis, ground mitigation alternatives, and preliminary foundation concepts for the project. The project provides a new, high-density, mixed-use community with a variety of housing types, a retail core, open space and recreation opportunities, on-site infrastructure, and public and community facilities and services. In all, there will be up to approximately 8,000 residential units; up to approximately 140,000 square feet (sq. ft.) of new commercial and retail space; approximately 100,000 sq. ft. of new office space; up to 500 hotel rooms; approximately 300 acres of parks and open space; bicycle, transit, and pedestrian facilities; a ferry terminal and intermodal transit hub; and new and/or upgraded public services and utilities, including a new or upgraded wastewater treatment plant and a new recycled water plant. In 2014, ENGEO conducted the design level geotechnical study for the first Major Phase of development, an approximately 171 acre parcel with approximately 3,700 residential units, and 100 acres of parks and Open Space.

Trestle Glen at Colma—Colma, CA
Project Engineer. Pedro provided field observation during installation of impact piers. Trestle Glen is a transit-oriented, mixed-use, urban redevelopment on an approximately 1.7-acre site adjacent to the Colma Bay Area Rapid Transit (BART) Station. The development includes a five-story structure that consists of four stories of wood-frame construction over a reinforced concrete podium which house 119 units of affordable housing and a child care facility. The development is surrounded by city streets and future townhouse development within a mixed residential, commercial, and light industrial area of Colma, California. The project included soil improvement for mitigation of liquefaction potential and the foundation consists of spread footings with a slab-on-grade.

EDUCATION

BS Civil Engineering San Francisco State University 2013

MS Geotechnical Engineering University of California, Berkeley 2016

EXPERIENCE

Years with ENGEO: 7

REGISTRATIONS & CERTIFICATIONS

Nuclear Gauge Operator, CA 16854 PNT

Professional Engineer, CA 87513

SPECIALIZATIONS

- Compressible Soils
- Deep Foundations
- Earth Retaining Structures
- Foundation Design
- Geographic Information System (GIS)
- Geotechnical/Geologic Instrumentation
- Laboratory Testing
- Levee Analyses
- Liquefaction Analyses
- Pavement Evaluation and Design
- Seepage Evaluation
- Slope Stability

TAYLOR J. STRACK, PE

Project Engineer

Taylor coordinates and performs geotechnical explorations and analysis. Taylor's expertise include levee analysis including seepage and slope stability. Settlement analysis on compressible deposits, such as Bay Mud and alluvial deposits and liquefaction determination and mitigation. In addition, he is experienced in foundation design (shallow and deep foundations), retaining wall, and pavement design. He is knowledgeable with codes and regulations including CBC 2016, ASCE 7-10, ULDC, FEMA and U.S. Army Corp of Engineers. He is proficient with engineering software such as SLOPE/W, SEEP/W, SLIDE, SETTLE3D, gINT, CPET-IT, C-LIQ, ArcGIS, PLAXIS 2D/3D, L-Pile and Unipile and Settle 3D.

SELECT PROJECT EXPERIENCE

Landings Google Campus—Mountain View, CA

Project Engineer. The approximately 19-acre site will be redeveloped to include one five-story, approximately 800,000-square-foot office structure with one to level of below-grade parking, landscaping and landscape structures, planned fill of up to 20 feet.

Taylor reviewed subsurface data and estimated consolidation parameters for use in settlement analysis. Modeling of the proposed improvements used Plaxis 3D and Settle 3D for soil structure interaction of adjacent engineered fills and proposed building loads.

Lucas Museum of Narrative Art at Exposition Park—Los Angeles, CA

Project Engineer. The Lucas Museum of Narrative Art will be constructed on two state-owned parking lots on Vermont Avenue south of Exposition Boulevard. The Museum of Narrative Art will be a five-story, 115-foot-tall building. Nearly one-third of the proposed building's 290,000 sf will be dedicated to gallery space, with other program elements including a movie theater, a lecture hall, a library, a restaurant and several digital classrooms. A publicly accessible green roof terrace will cap the building, while a 2,425-space parking complex will be located underneath.

Taylor reviewed subsurface data and pressuremeter testing to estimate elastic properties for settlement analysis. In addition, Taylor assisted with bearing capacity and lateral earth pressure recommendations for use in design of the subject structure.

Treasure Island Sub- Phase 1A Geotechnical Services—San Francisco, CA
Project Engineer. ENGEO is the geotechnical Engineer of Record of the Treasure Island Development Project. The project provides a new, high-density, mixed-use community with a variety of housing types including several high-rise structures, a retail core, open space and recreation opportunities, on-site infrastructure, and public and community facilities and services. Specifically, Taylor assisted with geotechnical analysis and reporting for Block C2 of the development which consists of high-rise and mid-rise structures.

The proposed structures include one to two levels of basement and will be supported by either deep foundations or a mat foundation. Taylor reviewed boring, Cone Penetration Tests and laboratory testing to estimate soil properties for use in settlement analysis. To understand structure performance for mat foundations and deep foundations- Plaxis 3D and Settle 3D was used. Taylor provided recommendations for foundation design for each block of the subject project.

301 Mission High Rise, Causation and Structural Retrofit—San Francisco, CA
Project Engineer. The Millennium Tower is located at 301 Mission Street in San Francisco, California. Construction on the tower began in 2005 and was completed in 2009. The building consists of two towers: one 58-story structure and one 12-story tower connected via an atrium. The Millennium Tower is founded on piles that are approximately 60 feet long, go through the fill and soft sediments and derive resistance within the dense Colma sands.

It has been the experience in the San Francisco Bay Area that buildings as tall as 40 stories founded on piles on the dense Colma sands perform adequately. This is because the underlying older bay deposits have been subjected to similar loads in the past. However, the old bay clay deposits did not perform as expected under the loads of a near-60-story, reinforced-concrete building, the heaviest in the western US.

ENGEO assisted in evaluating the causes of the settlement and tilt. We reviewed design documentation for the existing foundation and for excavation and dewatering of surrounding projects built after the tower construction. We constructed a 3-dimensional subsurface model and performed settlement analysis to understand the behavior of the subsurface and mitigation options. We also performed pile deformation analysis using L-Pile for use in design of mitigation alternatives.

EDUCATION

BS Civil and Environmental Engineering University of Michigan 2013

MS Civil Engineering University of Michigan 2014

EXPERIENCE

Years with ENGEO: 6
Years with Other Firms: 0

REGISTRATIONS & CERTIFICATIONS

Nuclear Gauge Operator, CA PNT 17661

Hazmat Certified as Required by USDOT and IATA, CA

Professional Engineer, CA 86816

24 Hour HAZWOPER Training, CA 1508232138856

8 Hour HAZWOPER Training, CA 1609185138856

8 Hour HAZWOPER Training, CA 1712035138856

SPECIALIZATIONS

- Construction Observation
- Earth Retaining Structures
- Excavation and Shoring
- Foundation Design
- Grading Project Management
- Liquefaction Analyses
- Pavement Evaluation and Design
- SWPPP Implementation

IAN D. MCCREERY, PE

Project Engineer

Ian joined ENGEO in 2014 and serves a variety of projects including commercial and residential developments in the San Francisco Bay Area. He has served several roles at ENGEO including project manager, staff engineer, special inspector, and field representative. His experience includes geotechnical and environmental engineering consultation, and SWPPP and construction project management. His design experience includes earth retaining structures, analysis and mitigation of geologic hazards, and foundations. He is committed to providing ENGEO's clients excellent service and strives to ensure project success.

SELECT PROJECT EXPERIENCE

603 Jefferson Avenue—Redwood City,
Project Manager. Ian served as the lead geotechnical engineer during the site investigation and foundation design for the project. He performed field exploration activities and developed design-level recommendations for deep excavations, post-construction settlement, and foundation design criteria. The geotechnical design level exploration for the eight-story mixed use building involved a detailed evaluation of settlement and liquefaction due to the presence soft deposits. The mixed-use building includes eight levels of commercial retail and condominium space above ground, and three levels of below-grade parking. Construction of the below-grade levels required a 35-foot excavation in medium to soft soils with shallow groundwater. Additional geotechnical design considerations included accommodating existing adjacent structures, undocumented fill in the area of a former creek, and designing for future flood events. ENGEO provided design recommendations and practical solutions for the excavation, shoring and retaining walls, and the foundation. ENGEO also assisted the project by providing guidance to the design team regarding challenging conditions.

Kifer Dev-Lawrence Station Campus—Santa Clara, CA
Project Manager. Ian assumed lead geotechnical engineer duties for the project in early 2017. He prepared geotechnical recommendations for the new building configuration, including allowable bearing capacity values, recommended foundation types, basement retaining wall parameters, and temporary shoring and dewatering recommendations. During construction in 2019, Ian provided consultation to assist challenging basement excavation conditions, shoring installation, groundwater issues. The project consists of a five-story office building with one subterranean parking level

and an accompanying five-level above-grade parking structure. The construction of the office building basement required a 20-foot excavation. The main geotechnical consideration for the site included the presence of loose, potentially liquefiable sand layers located at varying depths and complicated groundwater and hydrology conditions.

Sequoia Station Redevelopment—Redwood City, CA
Project Manager. Ian served as the lead geotechnical engineer during the initial geotechnical site exploration, which included navigating current use buildings and developing a work plan to accomplish the geotechnical exploration scope with minimal impact to the existing 24-hour businesses. During the project design, Ian conducted analyses to develop recommendations for the proposed project considering liquefiable materials and soft bay deposits at depth. Ian also developed recommendations for the large six-block mass excavation and foundation recommendations for the singular six-block development. Located at the heart of downtown Redwood City, this project includes redevelopment of existing single-story commercial buildings into six new city blocks set over an area of 12.1 acres. The project includes a two-level basement parking garage over the 6-block footprint, which will require a massive excavation. The new city blocks will be comprised of a 250-foot tall tower, along with 75- to 135-foot mid-rise buildings for residential, office, retail, and hotel use. City streets and public gathering spaces are planned at the ground level between the proposed structures.

Hecker Pass - East Cluster—Gilroy, CA
Project Manager. Ian was the project manager for ENGEO during the construction of the project. He provided oversight and consultation services through multiple phases of the development, including SWPPP implementation, grading activities, construction of improvements, paving operations, and assisted with special inspections. The overall Hecker Pass development consists of over 300 single-family homes within an area of approximately 130 acres. The development included site grading operations for individual pads and public roadways, underground utility installation, retaining wall construction, and environmental mitigation. ENGEO provided services including construction quality control, SWPPP management, special inspections during vertical construction, and ongoing geotechnical consultation to assist construction activities.

The Preserve—San Ramon, CA
Project Engineer. Ian provided engineering design support for the project. He was responsible for the structural design of site retaining walls, including concrete masonry unit walls, dry stack masonry walls, and cast-in-place concrete walls. He has also provided support during wall construction by responding to plan reviews and assisted the contractor with field adjustments. The 456 acre Preserve Project (formerly Faria Preserve) in San Ramon, California, includes 618 residential units, educational facilities, park sites, two East Bay Municipal Utility District (EBMUD) water storage tanks, roadways, utilities and a detention basin. The Preserve project is located in hilly terrain, and the geotechnical challenges at the site included many existing landslides, compressible soils and steep slopes. The project construction involved approximately 4 million cubic yards of civil design earthwork and approximately 3 million cubic yards of corrective and stabilization grading. ENGEO provided geotechnical characterization and design services during project planning, and consultation services during public agency permitting and design of wetland impacts mitigation. ENGEO also performed regional storm water impact modeling and mitigation studies during project approval. Project construction began in late 2015, and construction of final street and utility improvements was completed in 2018. During construction, ENGEO provided observation and testing, of grading, engineering geology oversight, supplemental engineering design and SWPPP monitoring.

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