GEOTECHNICAL INVESTIGATION
PROJECT FOUNTAIN ALLEY
35 South 2nd Street
San Jose, California

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18 February 2021
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation performed by Langan Engineering and Environmental Services, Inc. (Langan) for the proposed Fountain Alley development in San Jose, California. The site address is 35 South 2nd Street, and it is located on the west side of South 2nd Street between East Santa Clara Street and East San Fernando Street; the approximate location of the site is shown on Figure 1. The site is bound by Fountain Alley to the north, South 2nd Street to the east, and two- to three-story buildings to the south and west, as shown on Figure 2. A Santa Clara Valley Transportation Authority (VTA) light rail line runs along South 2nd Street parallel to the eastern part of the site. The site is rectangular with plan dimensions of about 393 feet north-south by 138 feet east-west and encompasses about 1.25 acres. Currently, the site is occupied by a surface parking lot with existing grades near Elevations 85 feet to 87 feet[1] (Kier & Wright, 2020).

Based on our review of the available 100 Percent Schematic Design drawings prepared by Bjarke Ingles Group (BIG, 2020), the project architect, and Glotman Simpson (Glotman Simpson, 2020), the project structural engineer, we understand the proposed development will consist of a 21-story residential and office building above four levels of below-grade parking; the basement parking area will occupy the entire site footprint. The concrete-framed tower will have a first floor at Elevation 86.2 feet and a total height of about 290 feet above the adjacent street grades. The current building layout features two towers at the street level that join and form a single structure above the 10th floor level; the approximate locations of the street-level tower footprints are shown on Figure 2. The lowest basement level finished floor will be approximately 56 feet below street level, corresponding to Elevation 30.2 feet, and a mat foundation up to 12 feet thick is being considered to support the structure; however, we understand considerable project programming is still underway. An excavation depth of approximately 70 feet below existing grades is being considered to accommodate the planned four-level basement.

Based on our correspondence with Glotman Simpson and preliminary foundation loading information (Glotman Simpson, 2021), dead plus live loads on the basement floor slab are anticipated to be about 6,000 to 10,000 pounds per square foot (psf) beneath the tower footprints

[1] All elevations are approximate and reference the North American Vertical Datum of 1988 (NAVD88).

and approximately 3,900 to 6,000 psf elsewhere. Because the building height will exceed 240 feet, a Performance-Based Seismic Design (PBSD) approach will be implemented by the project team; a PBSD basis of design has not been issued yet.

2.0 SCOPE OF SERVICES

Our scope of services for the geotechnical investigation was outlined in our proposal dated 22 October 2020. The purpose of our geotechnical investigation was to evaluate site-specific subsurface conditions and seismic hazards, assist the design team in selecting appropriate foundation type(s) for the proposed structure, and provide recommendations for the foundations and other geotechnical aspects of the development.

2.1 Geotechnical Investigation

We used the results of available past subsurface explorations and our current field investigation at the site, including borings, cone penetration tests (CPTs), and laboratory testing, to perform our engineering analysis and develop conclusions and recommendations for the following geotechnical aspects of the planned development:

- anticipated subsurface conditions, including estimates of groundwater level(s);
- site seismicity and potential for seismic hazards including liquefaction, lateral spreading, and fault rupture;
- appropriate foundation type(s) including shallow and deep foundations and/or ground improvement, as necessary;
- design parameters for the recommended foundation type(s), including vertical and lateral capacities and associated estimated settlements;
- temporary shoring and underpinning, as appropriate;
- lateral earth pressures for below-grade walls and temporary shoring;
- subgrade preparation for slabs-on-grade, mat foundations (if appropriate), exterior slabs and flatwork, including sidewalks;
- site preparation, grading, and excavation, including criteria for fill quality and compaction;
- soil corrosivity with brief evaluation; and
- construction considerations.

2.2 Seismic Studies

Because the building height will exceed 240 feet, the structural design will be performed in accordance with ASCE 7-16 and PEER Tall Building Initiative (TBI) Version 2.03. Our scope of services includes the development of site-specific ground motions in terms of the response spectra and time series for use in the seismic evaluation and design of the proposed structure.

The PBSD basis of design has not been issued yet. Once the PBSD basis of design is available, we will develop time series, which will be forwarded to the design team.

2.3 Report

The results of our geotechnical investigation and seismic studies are presented herein.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

We began our subsurface investigation by reviewing the results of the geotechnical explorations previously performed at and in the site vicinity (Cornerstone Earth Group, 2020 and Earth Systems Pacific, 2020) as discussed in Section 3.5. To further evaluate subsurface conditions at the site and obtain additional data below the bottom of the proposed excavation, we drilled two borings and advanced three CPTs.

Prior to performing our field investigation at the site, we:

- obtained a drilling permit from the Santa Clara Valley Water District (SCVWD),
- notified Underground Service Alert (USA) and followed up with USA utility companies as required by law, and
- checked the boring and CPT locations for underground utilities using a private utility locator.

Details of our field exploration activities and laboratory testing program, and the previous geotechnical investigations at the site, are described in the remainder of this section.

3.1 Borings

Two borings, designated as LB-1 and LB-2, were drilled for our subsurface investigation at the approximate locations shown on Figure 2. The borings were drilled on 5, 6, and 9 through 11 November 2020 by Pitcher Services, LLC (Pitcher) of East Palo Alto, California, using a truck-mounted drill rig equipped with rotary wash drilling equipment. Borings LB-1 and LB-2 were advanced to depths of approximately 201½ and 181½ feet below the existing ground surface (bgs), respectively. During drilling, our field engineer logged the borings and obtained samples of the material encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 and A-2. The soil encountered in the borings was classified in accordance with the classification chart presented on Figure A-3.

Soil samples were obtained using four different types of samplers: two driven split-barrel samplers and two thin-walled piston samplers. The sampler types are as follows:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.38-inch inside diameter, without liners
- Shelby Tube (ST) sampler with a 3-inch outside diameter and a 2.93-inch inside diameter.
- Pitcher Barrel (PB) sampler, a spring-loaded ST sampler, with the ability for overcoring in stiff to hard soils.

The sampler types were chosen on the basis of the soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to hard cohesive soil and the SPT sampler was used in the sandy soil. The ST sampler was used to obtain less disturbed samples of soft to medium stiff cohesive soil, while the PB sampler was similarly used in stiff to hard cohesive soil.

The SPT and S&H samplers were driven with a 140-pound, automatic hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy, and are shown on the boring logs. The blow counts used for the conversions were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

The ST and PB samplers were pushed hydraulically into the soil; the piston pressure required to advance the samplers is shown on the boring logs, measured in pounds per square inch (psi).

The pressure required to advance the sampler varies between drill rigs and is included for general information only.

Upon retrieval from the borings, the liners for the S&H, ST, and PB samples were sealed at each end; the SPT samples were transferred to plastic bags to retain the field moisture content.

Upon completion, the boreholes were backfilled with cement grout in accordance with the requirements of the SCVWD, and the pavement surfaces were patched to match the adjacent parking lot surface.

The soil cuttings from the borings were collected in 55-gallon drums, which were stored temporarily at the site, tested, and later transported off-site for proper disposal.

3.2 Cone Penetration Tests

Three CPTs, designated as LCPT-1 through LCPT-3, were performed on 9 and 10 November 2020 by Gregg Drilling, LLC. (Gregg) of Martinez, California at the approximate locations shown on Figure 2. The CPTs were advanced to approximately 195 to 200 feet bgs, as summarized below in Table 1.

TABLE 1
CPT Depths and Elevations

CPT Designation	Approximate Ground Surface Elevation (feet[1])	CPT Termination Depth (feet)	Approximate CPT Termination Elevation (feet[1])
LCPT-1	86.2	200.6	-114.4
LCPT-2	86.6	200.6	-114.0
LCPT-3	87.0	195.0	-108.0

Note:
1. Elevations reference NAVD88.

The CPTs were performed by hydraulically pushing a 1.7-inch-diameter, cone-tipped probe, with a projected area of 15 square centimeters, into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges or load cells within the cone continuously measure the cone tip resistance and frictional resistance during the entire depth of each probing. Accumulated data was processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soil encountered. The CPT data report prepared by Gregg, including logs showing tip resistance, side friction and friction ratio by depth, as well as interpreted soil classification, is presented in Appendix B. Soil types were estimated using the classification chart included in the CPT data report.

Multiple pore pressure dissipation tests (PPDTs) were performed at various depths in each of the CPTs. The PPDTs were conducted to measure hydrostatic water pressures and to estimate the approximate depth to groundwater. During a PPDT, the variation of pore pressure with time is measured behind the tip of the cone and recorded. For this investigation, the duration of the tests ranged from about 50 to 780 seconds. The results of the PPDTs are presented in the CPT data report in Appendix B.

Additionally, Gregg performed in-situ shear wave velocity measurements in LCPT-1 and LCPT-3 at depth intervals of every five feet (typical) for the full depths of the CPTs. The in-situ shear wave velocity measurements and plots are also presented in Appendix B.

After completion, the CPT holes were backfilled with cement grout in accordance with SCVWD requirements, and the pavement surfaces were patched.

3.3 Laboratory Testing

The soil samples collected from the field exploration program were re-examined in the office to check the soil classifications and representative samples were selected for laboratory testing. The laboratory testing program was designed to evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, Atterberg limits, gradation, shear strength, and compressibility, as appropriate. Results of the laboratory testing are included on the boring logs and in Appendix C as Figures C-1 through C-17.

3.4 Soil Corrosivity Testing

To evaluate the corrosivity of the near-surface soil, we performed corrosivity tests on a composite sample obtained from a depth of about two to three feet from Boring LB-1. The corrosivity of the soil sample was evaluated by CERCO Analytical, Inc. (CERCO), of Concord, California, using the following ASTM Test Methods:

- Redox – ASTM D1498
- pH – ASTM D4972
- Resistivity (100% Saturation) – ASTM G57

- Chloride – ASTM D4327
- Sulfate – ASTM D4327

The corrosion test results are briefly discussed in Section 6.7. The laboratory corrosion test results and a brief corrosivity evaluation by CERCO are presented in Appendix D.

3.5 Previous Geotechnical Investigations

Prior to performing our current field investigation, we reviewed the following geotechnical investigations previously performed at and in close proximity to the project site:

- Draft report titled "Geotechnical Investigation, Fountain Alley Tower, 35 South Second Street, San Jose, California" by Cornerstone Earth Group (CEG), dated 5 May 2020.
- Report titled "Geotechnical Engineering Study, 6-Story Fountain Alley Development, 26-34 South 1st Street by Earth Systems Pacific (ESP)" dated 28 January 2020.

Details about these previous investigations are included in the following subsections.

3.5.1 Cornerstone Earth Group (2020)

CEG performed a geotechnical exploration at the site, which consisted of drilling three borings and advancing one CPT. The borings, designated EB-1, EB-2, and EB-2A, were drilled by Exploration Geoservices, Inc. of San Jose, California to maximum depths of about 10 to 91 feet bgs using a hollow-stem auger drill rig on 11 and 12 March 2020. The CPT, designated CPT-1, was advanced by Gregg to a depth of about 130.6 feet bgs on 26 February 2020. Shear wave velocity measurements were collected at 5-foot intervals (typical) in the upper about 100 feet of CPT-1. The approximate locations and depths of the borings and CPTs by CEG are shown on Figure 2 and the logs are included in Appendix E.

CEG's draft geotechnical investigation report also includes the results of two CPTs and one boring that were completed at the site by Lowney Associates in 2003. The CPTs, designated CPT-8 and CPT-10, were advanced to depths of about 60.2 and 80.5 feet bgs, respectively. The boring, designated EB-9, was advanced to a depth of about 50 feet bgs using rotary wash drilling equipment and techniques on 15 January 2003. The approximate locations and depths of the borings and CPTs completed by Lowney Associates are shown on Figure 2, and the logs are also included in Appendix E.

3.5.2 Earth Systems Pacific (2020)

ESP performed a geotechnical exploration at and adjacent to the site, which consisted of drilling one boring and advancing two CPTs. The boring, designated B-1, was drilled to a depth of about 80 feet bgs using a hollow-stem auger drill rig on 15 November 2019. The CPTs, designated CPT-1 and CPT-2, were advanced by Middle Earth Geo Testing, Inc. of Hayward, California to depths of about 73.7 and 75.3 feet bgs, respectively, on 25 November 2019. The approximate locations and depths of the boring and CPTs by ESP are shown on Figure 2 and the logs are included in Appendix E.

3.5.3 Laboratory and Soil Corrosivity Testing

We also reviewed the results of the laboratory testing performed during the previous geotechnical investigations. Samples were tested to measure moisture content, dry density, Atterberg Limits, gradation, shear strength, compressibility, and corrosivity. Results of the laboratory testing are included in Appendix F.

Soil corrosivity tests were performed on one "near surface" sample by CERCO as part of ESP's laboratory testing program. The results of the corrosivity testing are discussed in ESP's geotechnical report, however, the test results are not included with the report.

4.0 SITE CONDITIONS

The existing site and subsurface conditions observed and encountered at the site, respectively, are discussed in this section.

4.1 Site Conditions

The project site is located in downtown San Jose and occupies assessor's parcel 467-22-121. It is bound by a pedestrian walkway known as Fountain Alley to the north, South 2nd Street to the east, and two- to three-story buildings to the south and west. Based on our review of historic aerial images (NTER Online, 2021), the site was occupied by multiple buildings between 1948 and 1968. Since 1968, the site has been occupied by a surface parking lot with grades ranging from approximately Elevations 85 feet to 87 feet. The grades are highest along the edges of the parking lot and slope down toward the center of the site. The parking lot consists of paved parking, travel areas and planted landscape areas around the edge and along the center of the site. A ticket booth and trash enclosure are located near the southeast and southwest corners of the site, respectively. An electric vehicle charging station is located in the northern part of the site.

According to the existing conditions drawings (Kier & Wright 2020), several utility lines are located within the site boundary, including over-head electric lines, 6- and 8-inch storm drain lines, and multiple utility and electric boxes. The drawings also show numerous utilities, such as sanitary sewer, storm drain, electric, communication, gas and water lines, outside the site along Fountain Alley to the north and South 2nd Street to the east.

VTA light rail infrastructure including a trackway and at-grade platforms are located adjacent to the site along the western side South 2nd Street. The VTA rail and stations are about 20 and 30 feet, respectively, away from the eastern site boundary.

4.2 Subsurface Conditions

An idealized subsurface profile through the western part of the site, designated A-A', is presented on Figure 3 and an idealized subsurface profile through the eastern part of the site, designated B-B', is presented on Figure 4. The approximate locations of the profiles are shown on Figure 2.

According to the available subsurface data, the site is blanketed by an about 2- to 12-foot-thick layer of fill that consists of medium stiff to hard clay and dense to very dense sand and gravel with brick, concrete, and organic debris. Where tested, the near surface clayey fill (i.e. within the upper five feet) is moderately expansive[2], with plasticity indices (PIs) up to 21.

The surficial fill is underlain by recent Holocene[3] alluvial deposits that generally consist of medium stiff to hard clays with varying amounts of sand and gravel, and interbedded layers of loose to very dense sand and gravel with varying amounts of fines to the maximum depth explored of approximately 201½ feet bgs. The alluvial clays are generally slightly to moderately overconsolidated[4], with typical overconsolidation ratios[5] of about 1.25 to 2 as measured with laboratory testing and correlated from the CPTs. Based on the laboratory testing and CPT correlations, the undrained shear strength of the clay is typically about 800 psf to over 4,000 psf; however, isolated pockets of soft clay with a shear strength of on the order of 500 psf were encountered at depths of about 20 to 30 feet bgs at several of the exploration locations.

[2] Soil with low expansion potential undergoes no volume changes with changes in moisture content.

[3] The Holocene Epoch began about 11,700 years ago and continues through the present day.

[4] An underconsolidated clay has not yet achieved equilibrium under the existing load; an overconsolidated clay has experienced a pressure greater than its current load.

[5] The overconsolidation ratio (OCR) for a soil is defined as the ratio between the maximum sustained pressure the soil has experienced and the present effective vertical pressure.

The granular layers are typically medium dense to very dense sands and gravels, and generally increase in relative density with depth. Below the bottom of the proposed excavation, the sand layers typically have about 10 to 17 percent fines (by weight), with up to 40 percent fines in a silty sand layer from about 146½ to 155½ feet bgs in Boring LB-2. These sand layers are confined and potentially under artesian pressure as discussed later in this section.

Groundwater levels were measured in the borings and through PPDTs in the CPTs during previous investigations at the site by Lowney Associates in 2003 and CEG and ESP in 2020. During the previous explorations, the groundwater level was observed between about 10 to 27 feet bgs, corresponding to about Elevation 77 feet to 57 feet at the time of exploration; these depths may not represent a stabilized groundwater level. For example, and the groundwater level identified in the borings performed by CEG was observed to drop from about 10 to 16 feet bgs (i.e. about Elevation 71 to 77 feet) at the time of drilling to about 30 to 38½ feet bgs (i.e. about Elevation 48½ to 57 feet) by the end of drilling.

During our current investigation, the groundwater level was measured about 16½ to 19 feet bgs in Borings LB-1 and LB-2, corresponding to approximately Elevation 67½ feet to 70 feet. The PPDTs conducted at LCPT-1 through LCPT-3 were performed at depths from approximately 29 to 187 feet bgs, corresponding to about Elevation 57 feet to -101 feet. The potentiometric surface of the groundwater measured in the Langan CPTs was calculated to be approximately 16 to 40 feet bgs, corresponding to approximately Elevation 47 to 70 feet as summarized below in Table 2.

TABLE 2
Groundwater Level Measurements from PPDTs

CPT Designation	Approximate Ground Surface Elevation (feet[2])	PPDT Depth (feet)	Depth to Potentiometric Surface[1] (feet)	Approximate Potentiometric Surface Elevation (feet[2])	Date of PPDT Measurement
LCPT-1	86.2	32.5	19.9	66.3	11/9/2020
		114.7	23.3	62.9	
		174.4	16.2	70.0	
LCPT-2	86.6	29.2	22.9	63.7	11/9/2020
		45.8	23.7	62.9	
		113.7	22.8	63.8	
		188.0	39.6[3]	47.0[3]	
LCPT-3	87.0	73.0	22.7	64.3	11/10/2020
		94.0	26.8	60.2	
		187.2	22.3	64.7	

Notes:
1. Groundwater level measurements obtained during the field exploration may not represent stabilized groundwater levels at the site.
2. Elevations reference NAVD88.
3. PPDT did not fully stabilize and therefore may not be representative of groundwater conditions at the site.

The higher potentiometric reading in the CPTs indicate the groundwater in the lower sand layers are under artesian pressure[6]. The hydrostatic water pressure measured during the PPDTs may not represent static groundwater conditions at the site.

Based on our review of published maps (California Division of Mines and Geology, 2002), the historic high groundwater level in the project vicinity is approximately 10 to 12 feet bgs, corresponding to approximately Elevation 75 feet. Seasonal fluctuations in rainfall influence groundwater levels and may cause several feet of variation in the actual groundwater level.

5.0 SEISMIC CONSIDERATIONS

Regional seismicity and seismic hazards at the site are discussed in the following sections.

6 Artesian pressure is a condition where the water is confined in a sand layer under pressure and when tapped is able to rise above the level at which it was first encountered.

5.1 Regional Seismicity

The major active faults in the area are the Hayward, San Andreas, Monte Vista-Shannon and Calaveras faults. These and other faults of the region are shown on Figure 5. For each of the active faults within 50 kilometers, the distance from the site and estimated mean characteristic Moment magnitude[7] (Mw) using the data presented in the Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165 (2014 Working Group on California Earthquake Probabilities 2015) are summarized in Table 3.

TABLE 3
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Silver Creek	2.0	Northeast	6.7
Hayward (So) extension	9.7	East	6.1
Total Hayward-Rodgers Creek Healdsburg	9.7	East	7.3
Hayward (So)	10	Northeast	6.9
Monte Vista - Shannon	12	Southwest	7.0
Calaveras (Central)	12	East	6.7
Total Calavares	12	East	7.5
Calaveras (No)	15	Northeast	6.8
Mission (connected)	15	Northeast	6.1
San Andreas (Peninsula)	19	Southwest	7.2
San Andreas 1906 event	19	Southwest	8.1
San Andreas (Santa Cruz Mountains)	21	Southwest	7.0
Sargent	23	South	6.8
Butano	23	Southwest	6.7
Pilarcitos	24	West	6.7
Zayante-Vergeles	28	Southwest	7.1
Zayante-Vergeles	29	Southwest	6.9
Las Positas	31	North	6.3
Greenville (So)	35	East	6.5
Greenville (No)	36	East	6.9
San Gregorio (North)	42	West	7.3
Mount Diablo Thrust South	44	North	6.2
Mount Diablo Thrust	44	North	6.6

7 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Calaveras (So)	45	Southeast	6.4
Mount Diablo Thrust North	49	North	6.4
Reliz	49	Southwest	7.3

Figure 5 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 6) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 33 kilometers from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 105 kilometers north of the site, with a M_W of 6.0.

The U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (USGS, 2016). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 4.

TABLE 4
USGS (2016) Estimates of 30-Year Probability (2014 to 2043) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
N. San Andreas	22
Calaveras	26
Green Valley	16
Greenville	16
Mount Diablo Thrust	16
San Gregorio	6

5.2 Seismic Hazards

During a major earthquake, strong to violent ground shaking is expected to occur at the project site. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[8] lateral spreading,[9] cyclic densification,[10] landsliding, or can cause a tsunami. Each of these conditions has been evaluated based on our literature review, field investigation, and analysis, and is discussed in the following subsections.

5.2.1 Mapped Seismic Hazard and Historic Observations

The site is within a zone designated with the potential for liquefaction, as identified by the California Division of Mines and Geology (CDMG), known now as the California Geologic Survey (CGS), in a map titled "State of California Seismic Hazard Zones, San Jose West Quadrangle" prepared by the CDMG dated 7 February 2002 and shown on Figure 7. Specifically, the map shows the site is in an area "where historic occurrence of liquefaction, or local geological,

8 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

9 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

10 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

geotechnical and groundwater conditions indicate a potential for permanent ground displacements."

We reviewed records from the 1906 and Loma Prieta Earthquakes to better understand the nature of the seismic hazard at the site. Following the 1906 Earthquake, no liquefaction or associated ground deformations were observed at the site (Youd and Hoose, 1978). Similarly, no seismically-induced ground damage was noted at the site during the 1989 Loma Prieta Earthquake (Holzer 1998). The closest liquefaction-related ground failure observations reported for the 1906 and 1989 Earthquakes were over a mile away from the site.

5.2.2 Liquefaction

When a saturated, cohesionless soil liquefies during a major earthquake, it experiences a temporary loss of shear strength caused by a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

We used available subsurface data from the borings and CPTs completed at the site during the current and past investigations to evaluate earthquake-induced liquefaction hazards. We exclude blow count data from borings that were drilled with hollow-stem augers, which includes the borings completed by CEG and ESP. Hollow-stem auger borings are not typically relied upon for liquefaction assessment because of the potential for stress relief, disturbance at the bottom of the borehole and flowing sands into the auger when drilling and sampling in granular soils below groundwater. For these reasons, blow count data below the groundwater levels are likely not representative of the relative density of the sand layers and therefore not reliable to evaluate liquefaction potential.

As discussed in Section 1.0, the proposed development will have a four-level basement. We understand the excavation for the basement and foundation system will extend to a maximum depth of about 68 feet, corresponding to approximately Elevation 18 feet. State guidelines by the Southern California Earthquake Center (SCEC, 1999) recommend a minimum depth of 50 feet below lowest proposed bottom of excavation grade for evaluation of liquefaction potential. During our investigation, Borings LB-1 and LB-2 were drilled to depths of about 201½ and 181½ feet bgs, respectively, and LCPT-1 through LCPT-3 were advanced to depths of about 195 feet to 200½ feet bgs, satisfying the guidelines.

Our liquefaction analyses were performed in general accordance with the State of California Special Publication 117A, Guidelines for Evaluation and Mitigation of Seismic Hazards in California. We used the procedures presented in Idriss and Boulanger (2008) to evaluate the liquefaction potential at the site. The Idriss and Boulanger procedures are updates of the simplified procedures developed by Seed et al. (1971) and later by the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss 2001). To estimate volumetric strain and associated liquefaction-induced settlement, we used the procedure developed by Tokimatsu and Seed (1987) for the borings and CPTs.

These methods are used to estimate a factor of safety against liquefaction triggering by taking the ratio of soil strength (resistance of the soil to cyclic shaking) to the seismic demand that can be expected from a design level seismic event. Specifically, two distinct terms are used in the liquefaction triggering analyses.

- Cyclic Resistance Ratio (CRR), which quantifies the soil's resistance to cyclic shaking; a function of soil depth, density, depth of groundwater, earthquake magnitude, and overall soil behavior

- Cyclic Stress Ratio (CSR), which quantifies the stresses that may develop during cyclic shaking

The factor of safety (FS) against liquefaction triggering can be expressed as the ratio of CRR over CSR. For our analyses, if the FS for a soil layer is less than 1.3, it is considered possible that the soil layer may liquefy during a large seismic event. For our calculations of estimated liquefaction-induced settlement, we assumed layers with a FS equal to or greater than 1.3 will not experience liquefaction-induced settlement.

The primary design parameters used in our liquefaction triggering calculations are summarized in Table 5.

TABLE 5
Primary Input Parameters Used in Liquefaction Evaluation

Parameter	Value
Depth to groundwater (historic depth to high groundwater)	About 10 to 12 feet below ground surface (about Elevation 75 feet)
Peak Ground Acceleration (PGA Geomean)*	0.58 g
Predominant Earthquake Moment Magnitude (M_w)	7.3
Factor of Safety for Liquefaction Triggering	1.3
Conversion factor for SPT sampler blow count to SPT N-value (includes hammer efficiency)	1.2
Conversion for S&H sampler blow count to SPT N-values (includes hammer efficiency)	0.7

* Based on site-specific analysis, see Appendix G.

Because the predominant earthquake is a moment magnitude 7.3, the CRR has been scaled to a moment magnitude of 7.5 using magnitude scaling factors developed by Idriss (Youd and Idriss, 2001). The PGA_M is the Geometric Mean PGA calculated from the site-specific seismic studies detailed in Section 7.8. CRR calculations were based on SPT blow counts and/or CPT tip resistance. The CPT tip pressures were normalized/corrected for overburden pressure, fines content, and thin layers, where appropriate. The CPT method also utilizes the soil behavior type index (I_C) and the exponential factor "n" applied to the Normalized Cone Resistance "q" to evaluate the cohesive nature of the soil. All of these are included in our analyses. In our analyses of the CPT data, soil that has significant amount of plastic fines, I_c greater than 2.6, was considered too cohesive to liquefy, and a corrected cone tip resistance q_{c1N} greater than 160 tons per square foot (tsf) was considered too dense to liquefy. In addition, a corrected shear wave velocity (V_{s1}) greater than 200 meters per second (m/s), was considered too dense to liquefy.

The CSR is obtained using the equations presented in the Idriss and Boulanger (2008) paper and is based on the relative density of the soil, the depth to the design groundwater level, the estimated peak horizontal acceleration at the ground surface (PGA_M), and a stress reduction coefficient (r_d).

Layers of loose to medium dense sand with varying amounts of clay and silt, varying in thickness from several inches to about 5½ feet, were encountered below the groundwater level at the borings and CPTs considered in our analyses. On the basis of the results of our analyses, we conclude some of these layers could potentially liquefy during a major earthquake and experience liquefaction-induced settlement. In our assessment, we considered the approach for soil classification and behavior presented in Robertson (2016). In this approach, CPT data is used to determine dilative and contractive behavior. The soil classification and behavior chart uses the normalized CPT tip resistance and friction ratio to separate material into clayey, sandy, and transitional soil types. The chart further uses another parameter, CD, to divide the dilative and contractive behavior of these soil types. A CD value of 70 or higher separates the soil between contractive and dilative tendencies. To capture transitional and borderline material, we used a CD cut-off value of 80. The available CPTs indicate that many of the medium dense sand layers below the groundwater level will likely exhibit dilative behavior and thus not be prone to settlement during earthquake shaking.

A summary of the data regarding liquefaction triggering and associated settlement from the existing ground surface are presented in Tables 6 and 7 for the borings and CPTs, respectively.

TABLE 6
Summary of Liquefaction Potential and Estimate Settlement from Existing Ground Surface from Boring Data

Boring Number	Approx. Depth to Layer (feet)	Elev. of top of layer (feet)	Layer Thickness (feet)	$(N_1)_{60\text{-}CS}$	PGA_M	CSR	$CRR_{7.5}$	Factor of Safety	Corrected Volumetric Strain ε_v (%)	Estimated Vertical Settlement (inches)
LB-1 by Langan, 2020	14.5	71.7	4.5	22	0.58	0.42	0.25	0.60	1.5	0.8
Total Settlement at LB-1 by Langan (2020)										**0.8**
LB-2 by Langan, 2020	19	67.3	4.5	13	0.58	0.45	0.15	0.33	2.0	1.1
	28	58.3	5.5	24	0.58	0.49	0.28	0.58	0.9	0.6
Total Settlement at LB-2 by Langan (2020)										**1.7**

TABLE 7
Summary of Liquefaction Potential and Estimate Settlement from Existing Ground Surface from CPT Data

CPT Number	Approx. Depth (feet)	Layer Thickness (feet)	I_C	$(q_{c1N})_{CS}$ (tsf)	$(N_1)_{60}$	CSR	$CRR_{7.5}$	Factor of Safety	Average Corrected Volumetric Strain ε_V (%)	Estimated Vertical Settlement (inches)
LCPT-1 by Langan, 2020	16.7	0.2	2.41	69	10	0.40	0.10	0.25	3.3	<0.1
	28.4	0.9	2.35	70	9	0.48	0.10	0.21	2.3	0.3
	31.5	0.2	2.32	81	10	0.49	0.11	0.23	1.9	<0.1
Total Estimated Settlement at LCPT-1 by Langan (2020)										**0.4**
LCPT-2 by Langan, 2020	21.0	3.6	2.29	64	9	0.45	0.09	0.21	3.1	1.3
	29.4	1.5	2.02	64	12	0.49	0.10	0.20	2.5	0.4
Total Estimated Settlement at LCPT-2 by Langan (2020)										**1.7**

CPT Number	Approx. Depth (feet)	Layer Thickness (feet)	I_C	$(q_{c1N})_{CS}$ (tsf)	$(N_1)_{60}$	CSR	$CRR_{7.5}$	Factor of Safety	Average Corrected Volumetric Strain ε_V (%)	Estimated Vertical Settlement (inches)
LCPT-3 by Langan, 2020	34.8	0.3	2.35	76	9	0.50	0.11	0.22	1.8	<0.1
Total Estimated Settlement at LCPT-3 by Langan (2020)										**<0.1**
CPT-1 by CEG, 2020	28.8	0.2	2.59	73	9	0.49	0.10	0.21	2.2	<0.1
	33.2	0.2	2.56	73	10	0.50	0.10	0.21	1.9	<0.1
Total Estimated Settlement at CPT-1 by CEG (2020)										**<0.1**
CPT-1 by ESP, 2020	21.1	0.5	2.55	67	10	0.44	0.10	0.22	3.1	0.2
Total Estimated Settlement at CPT-1 by ESP (2020)										**0.2**
CPT-2 by ESP, 2020	31.5	1.0	2.40	72	12	0.50	0.10	0.21	2.0	0.2
	36.2	0.8	2.48	73	12	0.51	0.11	0.21	1.7	0.2
	53.7	2.3	2.13	78	13	0.51	0.11	0.22	0.3	0.1
Total Estimated Settlement at CPT-2 by ESP (2020)										**0.5**

We conclude several medium dense sand layers in the upper about 55 feet bgs could potentially liquefy during a major earthquake on a nearby fault. The excavation for the basement of the proposed development will remove all of these layers, however, in the areas surrounding the project site where no basement excavation is planned, we conclude up to 1¾ inches of liquefaction-induced settlement could occur at the ground surface. In addition, we conclude up to one inch of differential settlement could occur over a horizontal distance of 30 feet outside the basement footprint.

5.2.3 Seismic Densification

Seismic densification can occur during strong ground shaking in loose, clean cohesionless deposits above the water table, resulting in ground surface settlement. We analyzed the potential for seismic densification using the procedure outlined by Tokimatsu and Seed (1987) and the Pradel (1998) method. The CPTs and borings typically indicate that the soil above the groundwater level is cohesive or sufficiently dense, therefore, the potential for significant seismic densification to occur at the site is generally low.

However, several isolated about 1½- to 2½-foot-thick layers of medium dense sand were encountered above the groundwater level at depths of about 5 to 10½ feet bgs at several of the borings and CPTs performed at and adjacent to the site. Using the Pradel (1998) method, we estimate seismic densification settlements up to about ¼ inch could occur in these layers during a major earthquake. The excavation for the planned basement would remove these layers; however, these settlements could occur outside the building footprint.

5.2.4 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open slope cut, or in the direction of a regional slope or gradient. The potential for lateral spreading to occur at a site is typically evaluated using an empirical relationship developed by Youd et al. (2002). This relationship incorporates the thickness, fines content, mean grain-size diameter, and relative density of the liquefiable layer, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions (such as a free face or edge of shoreline), to estimate the horizontal ground movement.

We used the results of the laboratory tests performed on soil samples from the borings, the CPT data and the Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacements (Youd et al. 2002) to evaluate the potential for lateral spreading. These regression equations indicate that sandy soil layers with $(N_1)_{60}$ values greater than 15 blows per foot may be moderately susceptible to soil liquefaction, but are sufficiently dense to resist the potential for lateral spreading (Youd et al 2002). Tables 6 and 7 indicate there are several layers with $(N_1)_{60}$ values less than 15; however, the layers appear to be discontinuous. In addition, the basement should key the building below the zones of potential lateral spreading.

Furthermore, the Guadalupe River is the nearest free face and is located approximately a half mile west of the site. Lateral spreading was not observed along the Guadalupe River during previous earthquakes (Youd and Hoose 1978). However, during the 1906 earthquake, significant lateral spreading was observed around Coyote Creek, which is approximately one mile east of the site. Lastly, the site and surrounding area are generally flat. Considering the site and subsurface conditions, historic observations, and distance to the nearest free face, we judge the potential for lateral spreading at the site to be low.

5.2.5 Sand Boils

We estimated the potential for sand boils using the Ishihara (1985) and Youd and Garris (1995) method using the non-liquefiable soil cover thickness, thickness of potentially liquefiable sand, and maximum ground acceleration at the site. The potentially liquefiable, near-surface layers will be removed from the building footprint as part of the excavation for the basement. Furthermore, we conclude that outside the building footprint the potentially liquefiable soil layers are thin and have sufficient soil cover to reduce the potential for sand boils to develop; therefore, we conclude that the potential for sand boils to manifest at the ground surface is low.

5.2.6 Tsunami

Published maps by the California Emergency Management Agency (CEMA 2009) indicate the site is not within the tsunami inundation zone; therefore, we conclude the potential risk by inundation from tsunami to be low for the site. However, the project civil engineer should evaluate the impact of sea level rise on the potential risk of inundation from a tsunami.

5.2.7 Surface Faulting

We evaluated the risk of surface faulting at the site associated with active or potentially active fault traces. Historically, ground surface displacements closely follow the trace of geologically young faults. Based on our study, we conclude the site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the proposed development is feasible provided the site conditions and geotechnical issues discussed below are properly addressed during the design and construction of the proposed buildings. The primary geotechnical issues for this project include:

- selection of appropriate foundation systems to support the building loads and accommodate anticipated settlements;
- design criteria for building foundations and basement walls;
- shallow groundwater level;
- dewatering and support for proposed excavations and adjacent structures and improvements during construction;
- providing a stable subgrade and adequate working surface at the base of the excavation; and
- soil corrosion potential

These issues and their impact on the geotechnical aspects of the project are discussed in the following subsections.

6.1 Foundations and Settlement

The primary considerations related to the selection of an appropriate foundation system(s) for the proposed structure are the:

- depth of excavation,
- final structural loads, and
- anticipated building static settlements resulting from consolidation of moderately compressible soil.

A basement that will extend four levels below the existing street grades is currently being considered. Because of the depth of the excavation and the presence of shallow groundwater, we conclude a mat foundation would be an economical foundation system provided the estimated total and differential settlements are tolerable. Assuming the mat foundation is on the order of 12 feet thick, we anticipate the bottom of the mat foundation will be approximately 68 feet bgs, corresponding to approximately Elevation 18 feet. The soil at the foundation subgrade elevation is expected to consist predominantly of stiff to very stiff clay.

According to the preliminary loading estimates (Glotman Simpson 2021), the dead plus live loads for the proposed structure will be approximately 6,000 to 10,000 psf beneath the towers and 3,900 to 6,000 psf elsewhere; the anticipated dead plus live loads include the weight of a mat. The proposed excavation of about 68 feet would result in a stress reduction on the order of 4,700 psf. Laboratory test results and CPT data indicate that the clay layers below the proposed excavation are lightly to moderately overconsolidated, with an overconsolidation ratio (OCR) of about 1.25 to 2. Therefore, the static settlement is anticipated to include both recompression and virgin consolidation settlement.

As the proposed basement excavation is made, we expect the removal of soil would create pressure relief and the base of the excavation should rebound (rise), especially near the center of the excavation. We estimate rebound of up to several inches could occur near the center of the site after basement excavation is complete. As the building is constructed and new foundation and building loads and transferred to the underlying soil, the clay layers would compress. Based on the preliminary dead plus live foundation bearing pressures, we estimate total static settlements would be on the order of 1¼ to 6¼ inches; the largest settlements occur below the tower footprints in the clay layers within about 20 feet of the bottom of the planned mat foundation (i.e. above Elevation 0 feet). We anticipate differential static settlements would be on the order of ¾ to 2¾ inches over a distance of about 50 feet. These settlement estimates do not include the rigidity of the mat, which should reduce the total and differential settlements.

If the static settlement and/or differential settlement are not tolerable, the upper soil beneath the foundation subgrade level could be improved to reduce the settlements. The proposed building could then be supported on a stiff, reinforced-concrete mat foundation bearing on improved ground. Using the preliminary dead plus live foundation bearing pressures and general parameters for improved soil from other nearby projects, we estimate total static settlements for a mat bearing on improved soil could be on the order of ¾ to 2¾ inches with the largest settlements occurring below the tower footprints. We anticipate differential settlements could be on the order of ¼ to 1¼ inches over a distance of about 50 feet, with the largest differential settlements occurring at the edge of the tower footprints.

6.2 Ground Improvement

As discussed previously, if the anticipated settlements are unacceptable from a structural standpoint, we conclude the most practical and economical solution is to stiffen the soil below the subgrade level thereby reducing the settlement of the underlying clay. The ground improvement should be designed to reduce the potential for static settlement and increase the rigidity to the soil and transferring vertical building loads to the underlying dense sand and stiff to hard clay. The ground improvement should extend at least 5 feet into the dense to very dense sand layer between about Elevation 10 to -10 feet. Based on the available subsurface information, the sand layer becomes deeper in the southern part of the site.

On the basis of our experience with the different methods of improvement, we judge deep soil mixing (DSM) columns or panels or jet grouted columns would be the most appropriate to improve the soil and transfer the loads to the underlying soil.

DSM is used to treat soil in place with cement grout using mixing techniques consisting of auger cutting heads, discontinuous flight augers, cutter heads, or blades/paddles to create a soil-cement column or panel. DSM elements may be installed in a variety of patterns including cellular blocks, a grid pattern, or isolated columns. Typical soil-cement columns or panels have a minimum diameter or width of three feet, respectively. A significant volume of cuttings are generated during DSM installation that will require handling and disposal, adding to the cost of this option. We anticipate DSM would be performed from within the basement excavation to reduce the amount of waste during installation, however, alternatives for installing the DSM near the street level could also be considered to simplify the installation logistics.

Jet grouting is performed by advancing a narrow steel drill stem and jetting with high-pressure cement grout and air. The high pressure grout and air act to cut/erode the surrounding soil, as well as mix the soil with cement in place, resulting in a column of soil cement. Jet grouting can be difficult in cohesive soils, but may be a desirable alternative for this site because it could be performed from near the existing site grades by advancing the drill stem to the target elevation before starting to jet with the cement grout and air. This would reduce the amount of waste while avoiding the more challenging logistics of installing ground improvement from the bottom of the planned excavation.

Independent of the type of ground improvement techniques employed, the ground improvement pattern should consider the mat foundation and column layout of the lowest basement level. Based on recent literature and our experience with similar projects, we estimate a replacement ratio[11] of about 35 to 60 percent may be required to maintain internal stability of the elements during seismic loading and transfer building loads to stronger and less compressible soil.

These types of ground improvement systems are typically installed under design-build contracts by specialty contractors; therefore, the site conditions, soil improvement methods, and anticipated settlements need to be estimated and refined by the specialty contractors for the selected ground improvement method.

6.3 Groundwater Considerations

As discussed in Section 4.2, groundwater has generally been encountered between Elevation 60 and 70 feet at the site. The historic high groundwater level mapped for the site is about 10 to 12 feet bgs, corresponding to about Elevation 75 feet. Therefore, we conclude a design groundwater level at Elevation 75 feet is appropriate for the design of permanent structures at the site.

Because the proposed building will be constructed with four basement levels, the basement walls and basement slab should be designed to resist hydrostatic pressures (lateral and uplift, respectively) using a design groundwater level at Elevation 75 feet. If the weight of the building and mat foundation is not sufficient to resist uplift and/or span between columns, tiedown anchors can be used to resist the anticipated uplift pressures.

[11] Replacement ratio is the ratio of the improved soil volume to the total soil volume.

The basement walls and floors should be waterproofed and waterstops should be provided across all below grade construction joints.

For temporary construction of the shoring, a lower groundwater level may be used. We recommend a groundwater depth of 15 to 17 feet bgs, corresponding to about Elevation 70 feet, be used for the design of the shoring. However, prior to installing the shoring, piezometers should be installed to confirm this lower, temporary groundwater level.

Higher potentiometric readings in PPDTs performed during our subsurface investigation indicate the groundwater in the lower sand layers and lenses (i.e. between approximately Elevation 15 and -45 feet) are under artesian pressure. The head in these lower sand layers is at about Elevation 60 to Elevation 70 feet. These layers will likely need to be depressurized prior to excavation, as discussed in Section 6.4.

6.4 Dewatering

Because the planned excavation for the basement extends below the groundwater table, the excavation will need to be dewatered. Variables that influence the performance of a dewatering system and the quantity of water produced include the number of wells, the depth and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped. Different combinations of these variables can be used to successfully dewater the site.

During excavation, the groundwater table within the site should be drawn down to at least three feet below the bottom of the excavation and the sand layers between approximately Elevation 15 and -45 feet should be depressurized to the about the same level to reduce the potential for blow-out of the subgrade. The dewatering contractor should check that there is sufficient overburden to resist hydrostatic pressures in the sand layers below the bottom of the proposed excavation. The dewatered level should be maintained at that depth until sufficient building weight is available to resist the hydrostatic uplift pressure of the design groundwater level at Elevation 75 feet. Seepage under the cutoff shoring wall should be controllable if the cutoff wall extends sufficiently into a clay layer to cutoff groundwater. If the groundwater is lowered more than three feet below the bottom of excavation for an extended period of time, we should be notified to check the impacts to the estimated settlements.

Because of the size of the site and anticipated subsurface conditions, a system of perimeter wells (within the excavation) may not sufficiently dewater it. Interior wells may also be needed to adequately dewater the site and reduce disturbance to the subgrade. In addition to the wells, a working pad, if needed (see Sections 7.2.2 and 7.7), can be used as a temporary drainage blanket to assist with the dewatering of the site. Perforated pipes may be placed in the gravel to collect water and conduct it to a sump. The sump and collector pipes should be decommissioned once they are no longer needed. The need for a working pad will depend on the final elevation of the basement and foundation, the type of soil exposed at the subgrade level, the type of soil improvement performed, and the type of equipment and method used to excavate the soil near the subgrade level. The working pad should be evaluated during the excavation of the basement levels. The site dewatering should be designed and implemented by an experienced dewatering contractor. However, we should review the dewatering system proposed by the contractor prior to installation.

In addition, the hydrostatic head in the sand and gravel layers below the excavation may need to be lowered to prevent blow-out of the excavation bottom; the dewatering contractor should check that there is sufficient overburden to resist hydrostatic pressures in the sand layers below the bottom of the proposed excavation. This should be evaluated by the dewatering contractor and included in the dewatering plans. The groundwater could be lowered to deeper depths depending on the design of the shoring system to provide higher passive pressures in granular materials. If this is the case we should be notified to check the impacts to the estimated settlements.

Dewatering the site should remain as localized as possible and should be limited to within the excavation. As discussed in Section 6.5, a continuous cut-off wall should be installed to reduce the impact of dewatering to the surrounding improvements. Widespread dewatering can result in subsidence of the area around the site due to increases in effective stress in the soil. Nearby streets and other improvements should be monitored for vertical movement and groundwater levels outside the excavation should be monitored through wells while dewatering is in progress. Should excessive settlement or groundwater drawdown be measured, the contractor should be prepared to recharge the groundwater outside the excavation through recharge wells. A recharge program should be submitted as part of the dewatering plan.

6.5 Shoring Considerations

During the excavation for the basement, the adjacent properties and improvements should be supported by temporary shoring. There are several key considerations in selecting a suitable shoring system. Those we consider to be primary concerns are:

- protection of surrounding improvements, including roadways, utilities, the VTA light rail infrastructure, and adjacent structures,

- penetration of shoring supports into the clay and sand below the bottom of the excavation,
- penetration of shoring to cutoff groundwater and reduce the amount of dewatering and potential for heave or blowout at the bottom of the excavation,
- proper construction of the shoring system to reduce the potential for ground movement, and
- cost.

We understand an excavation of about 70 feet below the adjacent sidewalk's existing grades is being considered for the construction of the building's basement and foundation system. The shoring design should also take into account the over-excavation of at least 12 inches across the site to create a working pad (if needed) for the mat foundation. During excavation for the proposed basement, shoring will be required to laterally restrain the sides of the excavation and limit the movement of adjacent improvements, such as neighboring buildings, VTA facilities, utilities, and public streets and sidewalks.

6.5.1 Temporary Shoring

Because of the shallow groundwater level, we conclude the shoring system should be able to support the excavation and cut off the groundwater. We judge the most practical shoring system for these conditions would be a continuous soil-cement-mixed wall cut-off system or concrete diaphragm wall to limit the drawdown of groundwater and subsequent settlement in the area surrounding the site. A soil-cement-mixed wall or concrete diaphragm wall is also stiffer than a conventional soldier pile and lagging system and should reduce lateral and vertical movements.

Soil-cement-mixed walls can be installed by advancing hollow-stem augers and pumping cement slurry through the tips of the augers during auger penetration. In another type of soil-cement-mixed walls, the walls are constructed by excavating slots with moving cutter heads. In either method, the soil is mixed with the cement slurry in situ, forming continuous overlapping soil-cement columns or continuous panels. Steel beams are placed in the soil-cement columns or panels at pre-determined spacing to provide rigidity. Soil-cement walls are considered temporary; permanent walls are usually built in front of the walls. To limit the amount of dewatering and reduce the potential for lowering the groundwater table behind the shoring, the soil-cement walls should be designed as cut-off walls. However, due to the presence of significant sand and gravel below the bottom of the excavation, it may not be possible to completely cut-off the flow of groundwater.

Concrete diaphragm walls are reinforced concrete walls that can be constructed by slurry trench method. The walls are constructed in sections, called panels. During excavation of a panel, bentonite slurry is pumped into the trench to prevent the soil from caving. After the excavation reaches the design depth and the reinforcement cage is placed, the slurry is displaced by concrete that is poured through a tremie pipe. Diaphragm walls can be used as both temporary shoring and the permanent walls.

The shoring system for the basement excavation will require either grouted tiebacks or internal bracing, depending on whether encroachment permits can be obtained to drill beneath the adjacent city streets and properties. Internal braces may be required if there are obstructions precluding the use of tiebacks or if extending them beyond property lines is not permitted.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle. The magnitude of shoring movements and resulting settlements of the ground surface behind shoring walls are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Clough and O'Rourke (1990) summarized the measured settlements adjacent to excavations in sand and concluded that the settlements varied from 0.1 to 0.3 percent of the excavation depth. The data also show the settlements at some sites where the excavations were shored with a soldier-pile-and-lagging system were higher than these values. In addition, Figure 5 in Clough and O'Rourke (1990) presents design curves to obtain the lateral wall movement in medium stiff to stiff clays based on the shoring system stiffness and factor of safety against basal heave. Using these relationships for an estimated excavation depth of up to 70 feet, we estimate settlement and lateral movement immediately behind the shoring wall could be on the order of 2 to 3 inches. These settlements and lateral movements assume the quality of construction will meet or exceed that considered standard in the construction industry. The settlement and lateral movement should decrease with distance from the wall, and should be small at a distance twice the excavation depth.

The anticipated settlements and lateral movements of the shoring wall should be evaluated by the shoring designer and reviewed by the design team. The City of San Jose Department of Public Works has criteria for deflections that the shoring designer and contractor should review.

6.5.2 Underpinning

The shoring wall will need to be designed to accommodate the surcharge loads from adjacent buildings within a distance equivalent to depth of excavation, or the buildings will need to be underpinned. The size and elevation of the bottom of the footings to be underpinned should be confirmed by others prior to excavation and underpinning.

Underpinning often consists of hand-excavated piers that extend below the planned bottom of excavation. However, because the planned excavation will extend on the order of 70 feet bgs, hand-dug underpinning piers will not be feasible. For this site, underpinning can consist of slant-drilled soldier piles under the existing foundations, or steel brackets welded to DSM shoring beams adjacent to the existing foundations. The underpinning elements can be designed to resist neighboring building loads and gain capacity from skin friction along the sides of the elements within the soil below the bottom of the excavation.

Load transfer onto the underpinning should be achieved by jacking; this method will also pre-settle the underpinning piles and reduce settlement of the underpinned building.

6.6 Construction Considerations

The soil at the site consists of materials that can be excavated with conventional earthmoving equipment such as loaders and backhoes. Removal of pavements, utilities, remnants of any previous basement and foundations and existing improvements associated with the existing at-grade parking lot could require the use of jackhammers or hoe-rams.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle slightly. Considering the size and depth of the excavation and the presence of nearby buildings and the VTA infrastructure, we judge a monitoring program should be established to evaluate the effects of the construction on adjacent streets and improvements.

During the late 1800's, extensive dewatering was performed for agricultural purposes in the Santa Clara Valley region using wells up to several hundred feet deep. Over the years, these wells have likely been abandoned and some were properly decommissioned. However, not all were likely decommissioned per SCWVD guidelines. If unidentified wells are encountered during excavation, Langan should be notified and a review of SCWVD or the California Department of Water Resources should be performed to determine, if they were properly decommissioned. If it is determined that they were not properly decommissioned, then they should be properly abandoned in place per SCVWD requirements, which may require drilling out the well the entire depth and backfilling with cement grout. Improper abandonment of the wells could lead to water issues during construction.

6.7 Corrosion Potential

Because corrosive soil can adversely affect underground utilities and foundation elements, laboratory testing was performed as part of this investigation to evaluate the corrosivity of the near-surface soil.

CERCO performed tests on a soil sample to evaluate corrosion potential to buried metals and concrete. In addition, CERCO performed corrosivity tests on a soil sample from the previous geotechnical investigation performed by ESP. The results of the tests are presented in Table 8 and Appendix D.

TABLE 8
Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfates (mg/kg)	Resistivity (ohms-cm)	Conductivity (umhos-cm)	Redox (mV)	Chlorides (mg/kg)
LB-1	2 to 3	8.2	21	2,900	N.P.	280	N.D.
ESP (2020)	"near surface"	7.3	68	N.R.	N.R.	N.R.	38

Notes:
1. N.D. = None Detected
2. N.P. = Not performed
3. N.R. = Not Reported

Based upon resistivity measurements, CERCO determined that the soil samples tested are classified as "moderately corrosive" to "corrosive" to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron. 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. For more detailed recommendations regarding the corrosion protection of buried metals and concrete, a licensed corrosion consultant should be retained.

7.0 RECOMMENDATIONS

From a geotechnical standpoint, the site can be developed as planned, provided the recommendations presented in this section of the report are incorporated into the design and contract documents. Recommendations for foundation design, site preparation, shoring design, below-grade walls, earthwork, at-grade improvements, utilities, and seismic design are presented in the following subsections.

7.1 Site Preparation and Clearing

Demolition of areas to be developed should include the removal of existing pavement and underground obstructions, such as shallow foundations of any previous or existing structures at the site. Any vegetation and organic topsoil should be stripped in areas to receive new site improvements. Stripped organic soil can be stockpiled for later use in landscaped areas, if approved by the owner and architect; organic topsoil should not be used as compacted fill.

Demolished asphalt and concrete from the site can be crushed to provide recycled construction materials, including sand, free-draining crushed rock or Class 2 aggregate base (AB), provided their re-use onsite is acceptable from an environmental standpoint. Where crushed rock will be used in applications where free-draining materials are required, the rock should have no greater than six percent of material passing the 3/8-inch sieve. Where recycled Class 2 AB will be used beneath pavements, it should meet requirements of the current Caltrans Standard Specifications. Recycled Class 2 AB that does not meet the Caltrans specifications should not be used beneath City streets, but it is acceptable for use as general fill at the site, if needed.

Existing underground utilities beneath areas to receive new improvements should be removed or abandoned in-place by filling them with grout. The procedure for in-place abandonment of utilities should be evaluated on a case-by-case basis, and will depend on location of utilities relative to new improvements. However, in general, existing utilities within four feet of final grades should be removed, and the resulting excavation should be properly backfilled based on the recommendations presented in this section.

7.2 Mat Foundation

If the settlements presented in Section 6.1 are tolerable to the structure, we recommend that the proposed structure be supported on a reinforced concrete mat foundation. According to our review of the current building pressure diagrams (Glotman Simpson, 2021), we understand dead plus live load bearing pressures will be on the order of about 4,000 to 10,000 psf. Because the mat foundation will be embedded about 68 feet beneath the surrounding grades, the risk of static bearing capacity failure is nil.

7.2.1 Mat Foundation Design

The structural engineer should design the mat foundation to effectively spread the building loads to limit total and differential settlements.

For preliminary design of the mat using the vertical modulus of subgrade reaction method, we recommend using an initial vertical modulus of subgrade reaction ranging from about 21 to 41 kips per cubic foot (kcf) under static loads provided by Glotman Simpson; a preliminary contour map of the modulus of subgrade reaction for the mat foundation was sent to Glotman Simpson in an email dated 12 February 2021. The static values may be increased by one third for total load conditions, including wind or seismic forces. During final design, this value will be used by the structural to evaluate the performance and settlement of the mat foundation based on the anticipated building loads. We should then review the results of the mat analyses (bearing pressure distribution and settlement) and, in turn, perform additional settlement analyses. This process is iterative and should be repeated until there is general agreement between the two evaluation methods.

Because the proposed foundation will extend below the groundwater level, the base of the mat should be waterproofed. The mat should be designed to resist hydrostatic pressure based on a design groundwater level at Elevation 75 feet. A waterproofing consultant should be retained to determine the most appropriate system for this project and to provide input regarding waterproofing details. Installation of waterproofing should be performed in accordance with the manufacturer's requirements. Waterproofing is typically placed directly on the soil subgrade and covered by a mud slab (thin layer of lean concrete). The mud slab will reduce the potential for subgrade disturbance and protect the waterproofing from damage during mat construction. The mud slab should also provide a firm, smooth working surface for placement of reinforcing steel.

Lateral forces can be resisted by a combination of passive resistance against the vertical face of the mat foundation and basement walls and friction along the contact area between the ground improvement and the base of the mat. Because of the potential for soil settlement between the soil improvement elements, frictional resistance in these areas should be ignored. To calculate the passive resistance against the mat, we recommend using an equivalent fluid equivalent weight pressure of 150 pcf; the passive pressure should not exceed 2,000 psf. Friction along the bottom of the foundation will be affected by the waterproofing and should not exceed 0.2 times the dead load. A friction value of 500 psf may be used along the face of the basement walls used to resist lateral forces. These passive resistance and friction values include a factor of safety of about 1.5 and may be used in combination without reduction. The friction values assume a non-bentonite waterproofing system is used below the mat and along the face of the basement walls. If a bentonite waterproofing system is used the base friction factor and wall friction value will likely be lower. The friction values provided should be confirmed by the waterproofing consultant.

The structural engineer should check the structural capacity of the walls and the amount of movement necessary to develop the passive pressure. We can provide passive mobilization curves, if needed to estimate the amount of wall movement for a given passive pressure.

7.2.2 Mat Foundation Preparation

Because the excavation for four basement levels will extend below the present groundwater level, the soil at subgrade level will be near saturation even after dewatering. Even if ground improvement is performed, the soil between improvement elements will still be near saturation and susceptible to disturbance. To protect the subgrade, we recommend heavy construction equipment not be allowed within three feet of the subgrade elevation and that the final excavations be made with excavators or backhoes with smooth buckets. Without an extended period for drying, we judge the unimproved subgrade may not support even light equipment and foot traffic without experiencing excessive disturbance. To help protect the subgrade if it is susceptible to disturbance, we recommend over-excavating the site and backfilling with drain rock on which the mat is constructed. This layer of crushed rock can also be used as part of a dewatering system, as further discussed in Section 7.7. The need for the working pad should be evaluated once the excavation nears the final subgrade elevation.

For the working pad, we anticipate an over-excavation of about 12 inches will suffice if used in conjunction with a woven reinforcing fabric (geotextile), such as Mirafi 500x (or equivalent). After placing the reinforcing fabric on the exposed subgrade, the overexcavation should be backfilled with clean one-inch minus crushed rock or similar material. A 3- to 4-inch-thick mud slab (thin layer of lean concrete) can be placed on the crushed rock and then the waterproofing can be installed and the mat constructed.

Because the proposed basement foundation will be below the groundwater level, waterproofing the base of the foundation and basement walls is recommended. The waterproofing should be placed directly on the crushed rock or on a mud slab and be covered by a second mud slab. The mud slab covering should reduce the potential for damage to the waterproofing and provide a firm, smooth surface on which to place the reinforcing steel for the mat. We recommend the waterproofing be placed in accordance with the manufacturer's specifications. If they differ from our recommendations, the manufacturer's specification should be followed to preserve their warranty.

We should observe mat subgrade prior to placement of reinforcing steel. The excavation for the mat should be smooth and non-yielding, and free of standing water, debris, and disturbed materials prior to placing concrete. In the event soft areas are encountered at final mat subgrade elevations, they should be overexcavated to competent material and backfilled with lean or structural concrete.

7.3 Ground Improvement

As discussed in Sections 6.1 and 6.2, the compressible soil anticipated near the planned foundation level could be improved to support the planned mat foundation. If needed, ground improvement may consist of either DSM columns/panels or jet grouting.

The mat should be designed to span between DSM or jet grouting elements because there is some potential for settlement within soil improvement cells.

7.3.1 Soil Improvement Design

The selection, design, construction, and performance of the ground improvement should be the responsibility of the specialty contractor. The ground improvement should be designed by a licensed civil engineer experienced in the design of ground improvement and installed by a ground improvement specialty contractor specializing in deep soil improvement and mixing. A submittal demonstrating the contractor's qualifications should be provided to Langan for review and approval prior to entering into contract with the Owner; refer to the Federal Highway Administration (FHWA) DSM manual (Bruce et al. 2013) or the Ground Modification Methods Reference Manual (FHWA, 2017) for additional details and examples for this qualifications submittal.

The ground improvement columns/panels should be designed and installed in a pattern that is sufficiently strong to transfer the loads into the underlying dense to very dense sand and stiff to hard clay. Ground improvement should extend at least five feet into the 10- to 20-foot-thick layer of the dense to very dense sand between about Elevation 10 and -10 feet; the sand layer appears to become thinner and deeper in the southern part of the site. For preliminary estimating purposes, the soil improvement should extend to at least Elevation 5 feet in shoring "Zone A" and at least Elevation -5 feet in "Zone B". The approximate extents of the shoring zones are shown on Figure 2. Based on the preliminary foundation information, the soil improvement elements would typically extend about 13 to 23 feet below a 12-foot-thick mat; the actual column lengths will depend on the mat configuration and thickness.

The planned layout of the ground improvement should take into account the column layout of the overlying building by centering ground improvement elements beneath columns. The ground improvement should have a minimum replacement ratio of 35 percent. The replacement ratio and strength of ground improvement elements should be checked to confirm they are adequate to support building loads. If circular DSM or jet grout columns are used, they should have a minimum diameter of three feet. If DSM panels are used, they should be at least three feet wide.

FHWA guidelines for DSM and jet grouting provide calculation methodologies for several potential failure modes that should be checked by the specialty contractor's designer. Soil improvement elements should have sufficient areal coverage and unconfined compressive strength to provide a factor of safety against bearing capacity failure of at least 2 for static loads and 1.5 for the total design loads, including wind and seismic. The design loads should be provided by the structural engineer. Soil improvement elements can be concentrated beneath columns to provide additional vertical support, and more widely spaced in lightly loaded areas. For budgeting purposes, DSM or jet grouted elements should be assumed to have a minimum compressive strength of 400 pounds per square inch (psi) and an average compressive strength of 500 psi after 28 days of cure time; actual values will depend upon column/panel spacing, building loads, and final replacement ratio. This strength is based on a replacement ratio of about 35 percent and a bearing pressure from the overlying mat foundation of up to 10,000 psf.

7.3.2 Soil Improvement Installation and Quality Control

The ground improvement contractor should prepare a detailed specification following the guide specifications outlined in the FHWA manuals and modified as appropriate for the current project. The specification should include sections on: 1) geotechnical background information on the project, 2) submittals, 3) materials and equipment, 4) execution, including test section, 5) quality control procedures, and 6) acceptance criteria. A soil improvement work plan should also be prepared that includes the sequence of construction, proposed mix designs, mixing equipment and procedures, test section details, soil improvement plans and calculations, schedule, sample daily production report, means and methods of the quality program, and names of proposed subcontractors. The soil improvement specification and work plan should be submitted to Langan for review at least 30 days prior to mobilization.

The ground improvement elements should be installed from an elevation above the groundwater table to limit the potential disruption of the elements from the upward flow of groundwater during installation, unless the site is dewatered and the lower sand layers are depressurized to below the bottom of the excavation.

Prior to production, at least one test section should be installed for each proposed set of mixing or jet grouting parameters to demonstrate the proposed equipment, procedures, and mix design can uniformly improve the onsite soils and achieve the design requirements. At least one full-depth core should be obtained in accordance with the quality control program from each element or group of elements installed using the same mixing or jet grouting parameters. Following the completion of test element installation, coring, and strength testing, we will require at least five working days to review and evaluate the test elements results and propose recommendations for production soil improvement installation.

During production soil improvement, a quality control program should include field observation, review of daily production records for consistency with the mixing parameters established by the test section, full-depth coring, unconfined compressive strength testing of the cored specimens, and testing of wet grab samples.

If DSM elements are planned, the quality control program should include collection of wet grab samples and coring of the DSM elements. At least two wet grab samples should be taken with a "bailer type" sampler every work shift. Grab sample locations should alternate between 1/3 and 2/3 of the element depth; we may occasionally request samples at other depths to evaluate soil variations. Once collected, the wet samples should be immediately provided to the materials testing engineer, who should prepare cylinders without additional mixing. A ¾-inch screen may be used to remove oversized material from the test samples. Light tamping of samples to facilitate consolidation and remove air bubbles is permitted. At least 95 percent of all strength tests performed on the ground improvement elements should meet or exceed the minimum compressive strength.

In addition, the quality control plan should include triple-barrel coring equipment through at least five percent of the DSM elements to show that the equipment is appropriately mixing and improving the ground. The cores should be advanced within representative test panels/columns. Coring should be performed at least seven days after the DSM elements are installed within about six inches from the outer edge of the DSM element, and not within overlap zones. Langan should log all cores. For acceptance, the cores should have a recovery greater than about 90 percent for the entire cored element, as well as a high rock quality designation

(RQD). Zones of no recovery should be considered unmixed. Strength testing should also be performed on representative samples of the core. At least 80 percent of the strength tests performed within an element and at least 90 percent of all strength tests performed at the site should meet or exceed the design compressive strength.

Similar to soil mixing, the quality control plan for jet grouting should include triple-barrel QC-sized coring through the jet grout elements during a test program. Of significant interest during the test program is the ability of the jet grouting technique to effectively erode and thoroughly mix the surrounding soil to the desired diameter. Therefore, the cores should be centered at a distance of no more than six inches from the theoretical outer edge of the jet grout columns. Once the results of coring have confirmed the theoretical jet grout column diameter can be achieved, the final jet grout layout can be established and production grouting can commence. Wet grab samples cannot be obtained in jet grouted columns, because the resulting jet grouted material is much stiffer immediately after jetting than the DSM panels. Because wet grab samples cannot be obtained, five percent of production jet grout columns should be cored as described above to confirm minimum diameter, mixing, and strength gain. Coring should be performed at least seven days after the elements are installed and centered about six inches from the outer edge of the element. For acceptance, the cores should have a recovery greater than about 90 percent for the entire cored element as well as a high rock quality designation (RQD). Strength testing should be performed on representative samples of the core.

7.4 Basement Wall Design

We recommend all basement walls be designed to resist lateral pressures imposed by the adjacent soil, improvements, and vehicles. Because the site is in a seismically active area, the design should also be checked for seismic conditions. Under seismic loading conditions, there will be a seismic pressure increment that should be added to active earth pressures. We used the procedures outlined in Sitar et al. (2012) and the site-specific geomean peak ground acceleration for the Design Earthquake ground motion level to compute the seismic pressure increment. Basement walls should be preliminarily designed for the more critical loading condition of at rest or seismic conditions using the equivalent fluid weights and pressures presented in Table 9.

TABLE 9
Preliminary Basement Wall Design Earth Pressures
(Drained Conditions)

	Equivalent Fluid Weights for Static Conditions		**Seismic Conditions[1]**
	Unrestrained Walls (Active)	**Restrained Walls (At-Rest)**	**Total Pressure – Active Plus Seismic Pressure Increment**
Above Groundwater[2]	40 pcf[3]	60 pcf	60 pcf
Below Groundwater	80 pcf	90 pcf	90 pcf

Notes:
1. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked.
2. For the design groundwater elevation of Elevation 75 feet should be used (corresponds to approximately 10 to 12 feet below existing ground surface (street grade).
3. pcf = pounds per cubic foot

Where traffic will pass within 10 feet of basement walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 pounds per square foot (psf) applied in the upper 10 feet of the walls. In addition, the basement wall should be designed to accommodate the surcharge loads from adjacent buildings within a distance equivalent to depth of excavation, unless they are supported on deep foundations. If the neighboring buildings are supported above the bottom of the planned basement excavation, we can provide surcharge loading diagrams when more information about the neighboring buildings, such as the foundation type, size, layout, and contract pressure are known. Lastly, because the VTA light rail line runs parallel and in close proximity to the eastern perimeter of the site, additional rail traffic surcharge pressures should be accounted for. We can also provide surcharge pressures for the VTA light rail, when more information about the rail loading is available. Alternatively, the neighboring buildings can be underpinned as discussed in Section 7.5.2.

If the basement walls are designed to resist lateral forces such as wind or earthquake loading they should be checked using passive pressures. An equivalent fluid weight of 300 pcf and 150 pcf may be used to compute passive resistance above and below the groundwater table, respectively; the passive pressure should not exceed 2,000 psf. This value includes a factor of safety of about 1.5. The structural engineer should check the structural capacity of the walls and the amount of movement necessary to develop the passive pressure. We can provide passive mobilization curves, if needed to estimate the amount of wall movement for a given passive pressure.

The lateral earth pressures given assume the basement walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. If the walls are not drained, they should be designed for an equivalent fluid weight of 90 pounds per cubic foot (pcf) to account for hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend to a perforated PVC collector pipe at the design groundwater level (Elevation 75 feet). The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68) and should be sloped to drain into an appropriate outlet. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

An acceptable alternative is to backdrain the wall with Caltrans Class 2 permeable material at least one foot wide, extending down to the base of the wall. A perforated PVC pipe should be placed at the bottom of the gravel, as described for the first alternative. The pipe in either alternative should be sloped to drain into an appropriate outlet. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

To protect against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the walls.

If backfill is required behind basement walls, the walls should be braced or hand-compaction equipment used to prevent unwanted surcharges on the walls.

7.5 Excavation, Temporary Slopes, and Shoring

Throughout the project site, if temporary slopes are used they should not be steeper than 1.5:1 (horizontal to vertical) for slopes up to 12 feet in height. Slopes higher than 12 feet should be analyzed on a case-by-case basis.

A shoring system should be used to retain the sides of the excavations because there is insufficient space to slope the excavation. Additionally, as the depth of the basement excavation for the building will be on the order of 70 feet, the shoring should be tied back or internally braced.

7.5.1 Soil-Cement Mixed Walls/Concrete Diaphragm

The selection, design, construction, and performance of the temporary shoring system should be the responsibility of the contractor.

As discussed in Section 6.5, we conclude an impervious shoring system such as a soil-cement mixed or concrete diaphragm wall along with tiebacks or internal bracing can be used to retain the basement excavation. A soil-cement mixed wall or concrete diaphragm wall should be designed using the lateral earth pressures presented on Figures 8 and 9 for excavation depth of about 70 feet bgs; if the planned excavation depth changes, we should be notified so we can update our recommendations accordingly. Figure 8 provides the recommended lateral earth pressures in "Zone A," while Figure 9 includes the recommended lateral earth pressures for "Zone B." The approximate limits of where "Zone A" and "Zone B" soil conditions can be used to design the soil-cement mixed wall is shown on Figure 2.

Figures 8 and 9 provide recommended lateral earth pressures, but do not account for surcharge pressures from the adjacent buildings to the south and west, or from the VTA light rail to the east. Buildings within a distance equivalent to depth of excavation should be underpinned and/or the shoring wall be designed to accommodate the surcharge loads from adjacent buildings within a distant equivalent to depth of excavation. Surcharge pressures from the neighboring buildings and VTA light rail should be developed and incorporated into the shoring design, as appropriate, when more information is available. Additional details about underpinning are presented in Section 7.5.2.

If traffic loads are expected within 10 feet of the walls, an additional design load of 100 psf should be applied to the upper 10 feet of the walls. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed after the surcharge loads are known.

Passive resistance below the bottom of the excavation may be computed using Figures 8 and 9. These values include a factor of safety of about 1.5. The groundwater can be lowered within the excavation to increase the passive resistance in the sand layers below the bottom of the excavation.

A design groundwater level of Elevation 70 feet can be used to design the shoring because it is a temporary condition. However, prior to construction the groundwater level should be checked that it is at Elevation 70 feet or lower.

The shoring designer and dewatering designer should evaluate the required penetration depth of the cutoff wall and soldier piles to provide adequate passive resistance below the bottom of excavation and to prevent blowout of the excavation subgrade due to hydrostatic uplift pressures; depressurizing wells in the lower aquifers will likely be required. In addition, the shoring designer should evaluate the required penetration depth of the wall to support the vertical component of the tiebacks and the vertical load acting on the wall, if any. To compute the axial capacity of the wall, we recommend using an allowable friction of 1,000 psf in the soil below the excavation level. To compute the allowable skin friction against the back side of the wall above the excavation level, we recommend an allowable friction coefficient of 0.3 times the horizontal component of the tieback or internal brace force. End bearing on the continuous shoring wall should be neglected.

The shoring system should be designed by a licensed civil engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The shoring engineer should be responsible for the design of temporary shoring with applicable regulatory requirements. The anticipated deflections of the shoring system should be estimated by the shoring engineered to check if they are acceptable. Control of ground movement will depend as much on the timeliness of installation of lateral restrain as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

7.5.2 Underpinning

Where the excavation extends below the bottom of adjacent footings, the footings should be underpinned or the shoring and basement walls should be designed for the surcharge from the adjacent footings. Steel beams installed in slant-drilled shafts can be used to underpin the existing structures.

Underpinning piles should be designed to resist the neighboring building loads. The slant piles, consisting of steel piles installed in slant-drilled shafts, should gain support in the stiff to very stiff clay and medium dense to very dense sand below the bottom of the excavation. Intermediate overlapping shafts can be installed between the slant piles to retain the soil. The shoring designer should evaluate the required penetration depth of underpinning piles. The frictional capacity of the underpinning piles will depend on the pile type and installation method, however, we recommend using an allowable friction of 1,000 psf in the soil around the underpinning piles below the bottom of the excavation for an initial estimate. The friction above the excavation depth should be ignored.

If slant-drilled piles are used, we recommend piles be preloaded (jacked) prior to dry packing/grouting to reduce settlement as the foundation load that is transferred to the piles. To reduce movement and provide adequate foundation support during installation of the underpinning piles, adjacent piles should not be drilled until they have been dry packed or grouted.

We can provide additional recommendations, as needed, if an underpinning system is selected for this project.

7.5.3 Tiebacks

Temporary tiebacks may be used to restrain the shoring. Tieback installation should not interfere with existing underground utilities or other below-grade improvements adjacent to the excavation. The vertical load from the temporary tiebacks should be accounted for in the design of the vertical elements. Design criteria for tiebacks are presented on Figures 8 and 9 for the planned excavation depth of 70 feet bgs.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks with bar and strand tendons should have a minimum unbonded length of 10 and 15 feet, respectively. The unbonded length should be created by placing an oversized rigid smooth plastic casing (i.e. PVC pipe) over the bars or strands; flexible plastic does not provide an adequate bond-break for the unbonded zone. All tiebacks should have a minimum bonded length of 15 feet and be spaced at least six times the grouted diameter of the bonded zone or four feet, whichever is greater. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tiebacks will be installed through medium stiff to very stiff clay with varying amounts of sand and gravel, and loose to very dense sand with varying amounts of silt, clay, and gravel. The tieback allowable capacity will depend upon the drilling method, hole diameter, grout pressure, post grouting, and workmanship. The use of solid-flight augers to install tiebacks in sand and the fill can result in loss of soil and settlement of structures or the ground surface located above the tiebacks. Therefore, solid flight augers should not be used for tieback installation. We recommend a smooth cased tieback installation method (such as a Klemm type rig) be used. For estimating purposes, we recommend using the allowable skin friction values for pressure-grouted tiebacks shown on Figures 8 and 9; these values include a factor of safety of at least 1.5. Higher allowable skin friction values may be used, if verified by load tests.

The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth and water pressures, as well as all surcharge pressures, imposed on the temporary shoring systems. Determination of the tieback length should be based on the contractor's familiarity with their installation method. The computed bond length should be confirmed by a performance- and proof-testing program under our observation. Replacement tiebacks should be installed for tiebacks that fail the load tests, as directed by the shoring designer.

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other temporary tiebacks should be proof-tested to at least 1.25 times the design load. Recommendations for tieback testing are presented in Section 7.5.4. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

7.5.4 Tieback Testing

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the tieback test results and determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between one and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inch movement between six and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that failed to meet the first criterion will be assigned a reduced capacity.

If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the contractor should replace the tiebacks.

7.6 Tiedown Anchors

If the weight of a building is not sufficient to resist the hydrostatic uplift loads or the mat cannot resist the uplift pressure between columns, tiedown anchors should be installed. Tiedowns typically consist of relatively small-diameter, drilled, grout-filled shafts with steel bars or tendons embedded in the grout. The tiedowns develop their uplift resistance from friction between the perimeter of the shaft and the surrounding soil.

Tiedowns should be spaced at least four shaft diameters apart or a minimum center-to-center spacing of four feet, whichever is greater. Because specialty contractors who install tiedowns use different installation procedures, the uplift capacity of the tiedowns will vary with the procedure. For planning purposes, however, we recommend using an allowable friction of 1,500 psf for post-grouted tiedowns installed in the native stiff clays and dense sands; this value includes a factor of safety of 2.0 for permanent uplift loads (i.e. hydrostatic uplift). A factor of safety of 1.5 should be used for seismic loads. Higher values can be obtained depending upon the installation techniques employed by the contractor and the results of pullout tests.

If the tiedowns are installed from the bottom of the basement excavation, they will likely encounter sand layers that have a piezometric head that is higher than the bottom of the subgrade. These sand layers should be depressurized as discussed in Section 6.4. If the water head is sufficient to rise above the bottom of the excavation, this could cause installation issues, especially when grouting the tiedowns. The contractor should be prepared to deal with this condition. An alternative would be to install the tiedowns above the subgrade elevation.

The tiedowns will be installed below the water table; therefore, the contractor should use an auger-cast system or be prepared to case the holes to prevent caving. High strength bars or strands may be used as tensile reinforcement in the anchors. For stressing, the steel bars and strands should have at least 10 and 15 feet of free length, respectively. After testing, tiedowns should be locked-off. The lock-off load and allowable amount of deformation after the tiedown is locked off should be determined by the structural engineer for the tiedowns.

The bond length should be at least 15 feet. The design capacity of the tiedowns should be confirmed by a performance- and proof-test program conducted under our observation. We recommend the first two production tiedowns and two percent of the remaining tiedowns be performance tested to 2.0 times the design load. The remainder should be proof tested to 1.5 times the design load. The test procedure and acceptance criteria described in Section 7.5.4 for tieback testing should also be used for tiedowns. Replacement tiedowns should be provided, as directed by the structural engineer, for tiedowns that fail the test.

Special attention should be given to waterproofing the connections between the tiedowns and the mat. Because the tiedowns will be permanent, we recommend they be double corrosion protected. Typically, double corrosion protection is provided by placing the inner bar within a corrugated HDPE or PVC sleeve and filling the annular space between the bar and sleeve with a non-shrink cement grout.

7.7 Dewatering

As previously discussed, the water table within the site should be drawn down to three feet below the bottom of the excavation and the sand layers between approximately Elevation 15 and -45 feet should be depressurized, as necessary, during construction. If tiedowns will be installed from near the basement subgrade, the lower sand layers should be depressurized to three feet below the bottom of excavation. The wells installed within the excavation should be properly sealed through the mat upon abandonment to reduce the potential for water leakage. Groundwater should not be lowered beyond the site limits because subsidence of the surrounding area will occur due to increases in effective stresses in the soil. Groundwater levels outside the excavation should be monitored while dewatering is in progress. Should groundwater drawdown be measured, the contractor should be prepared to recharge the groundwater outside the excavation through recharge wells.

As discussed in Section 6.4, the crushed rock working pad can be used as part of the dewatering system as a temporary drainage blanket. To drain the crushed rock, four-inch diameter perforated PVC pipe should be placed near the bottom of the rock, spaced every 30 feet, to direct water trapped in the rock to a sump. The sump should be properly abandoned before the completion of construction.

7.8 Seismic Design

The following presents the recommended site-specific response spectra developed per 2019 CBC and ASCE 7-16. We expect this site will experience strong ground shaking during a major earthquake on any of the nearby faults. To estimate ground shaking at the site, we developed site-specific response spectra for the structural evaluation of the proposed building. Because the structure will be evaluated using performance based design, time series will need to be developed; however, these have not yet been developed. Once the time series are developed, we will forward them to the design team, along with pertinent data of the proposed time series including duration, energy content, and pulse characteristics and the methodology used to develop the spectrally compatible time series.

As part of the development of the site-specific spectra, we performed a Probabilistic Seismic Hazard Analysis (PSHA), deterministic analysis and ground response analysis to develop site-specific horizontal spectra at the ground surface for the Risk-targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE) consistent with the ASCE 7-16 and the Serviceability Level Earthquake (SLE) per PEER Tall Building Initiative (TBI) Version 2.03. The MCE_R is defined in the 2019 CBC as the lesser of the Risk-Targeted probabilistic spectrum having 2 percent probability of exceedance in 50 years or the 84th percentile deterministic event on the governing fault both in the maximum direction; the DE is defined as 2/3 of the MCE_R. The SLE defined as having a 50 percent probability of exceedance in 30 years. We developed the site-specific spectra for the MCE_R, DE and SLE levels of shaking.

The probabilistic seismic hazard analysis (PSHA) was performed using the computer code OpenSHA, Version 1.5.2 (OpenSHA 2020). This approach is based on the probabilistic seismic hazard model developed by Cornell (1973) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources and earthquake activities were assigned to the faults based on historical and geologic data.

The recommended MCE_R and DE spectra for 5 percent damping and the SLE for 2.5 percent damping are presented on Figure 10. Digitized values for the recommended spectra are presented and in Table 10, and details of our analysis are presented in Appendix G.

TABLE 10
Recommended MCE_R, DE, and SLE Spectral Acceleration (g's)

Period (seconds)	MCE_R 5% Damping	DE 5% Damping	SLE 2.1% Damping
0.01	0.694	0.463	0.221
0.10	1.098	0.732	0.485
0.20	1.511	1.007	0.691
0.30	1.732	1.155	0.691
0.40	1.767	1.178	0.626
0.50	1.718	1.145	0.558
0.75	1.414	0.943	0.401
1.00	1.202	0.801	0.299
1.50	0.870	0.580	0.190
2.00	0.663	0.442	0.131
3.00	0.465	0.310	0.076
4.00	0.353	0.235	0.049
5.00	0.274	0.183	0.034

Because site-specific procedure was used to determine the recommended MCE_R and DE response spectra, the corresponding values of S_{MS}, and S_{M1} per Section 21.4 of ASCE 7-16 should be used as shown in Table 11.

TABLE 11
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.590[12]
S_{M1}	1.412[13]
S_{DS}	1.060[12]
S_{D1}	0.941[13]

7.9 At-Grade Improvements and Fill Placement

We recommend new sidewalks and concrete flatwork (in non-vehicular traffic area) be underlain by at least four inches of Class 2 aggregate base (AB) material (or the minimum thickness per City of San Jose Standards) that has been compacted to at least 95 percent relative compaction[14]. To further reduce the potential for shrink/swell cracking of the near-surface, moderately expansive soil, exterior slabs should be underlain by 12 inches of select fill; the upper four inches of select fill can consist of the AB. The select fill should extend at least two feet beyond the edge of slabs. Even with 12 inches of select fill, these slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slabs and adding additional reinforcement will control this cracking to some degree. In addition, where slabs provide access to buildings, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

The soil subgrade should be kept moist to prevent desiccation cracks until it is covered by select fill. After an area is exposed by stripping and/or excavation, it should be evaluated for stability. Stable soils, when properly prepared, should be smooth and non-yielding under the weight of typical grading equipment such as a full water truck. If site grading occurs in late summer or in fall, the surface soil may be dry to depths exceeding 12 inches; the actual depth should be confirmed during site grading with moisture content tests on the upper three feet of soil. Surface

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12 S_{DS} is based on the site-specific response spectra and is based on 90 percent of the maximum spectral acceleration within the period range of 0.2 to 5 seconds; it is governed by 90 percent of the spectral acceleration at a period of 0.4 seconds.

13 S_{D1} is based on the site-specific response spectra and is the maximum of the product of period, T, and spectral acceleration, Sa, for periods from 1.0 to 5.0 seconds; it is governed by the product of the period and spectral acceleration at a period of 4.0 seconds.

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

soil that has a moisture content of less than 18 percent (the approximate plastic limit of the soil) should be excavated, moisture-conditioned to at least three to five percent above optimum moisture content, and compacted to between 88 and 93 percent relative compaction to reduce its expansion potential. Based on our experience in the project area, we judge the maximum depth of required excavation for moisture conditioning will be two feet.

Select fill placed beneath improvements should meet the following criteria:

- be free of organic matter
- non-hazardous
- contain no rocks or lumps larger than three inches in greatest dimension
- have a low expansion potential (defined by a liquid limit of less than 40 and plasticity index lower than 12)
- have at least 20 percent fines (by weight)
- have a low corrosion potential[15]
- be approved by the geotechnical engineer.

The intent of the recommendation for select fill to contain at least 20 percent fines (particles passing the No. 200 sieve) is to reduce the potential for surface water to infiltrate beneath slabs. The on-site soils do not meet the requirements of select fill.

Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. The subgrade should be rolled to a firm, non-yielding surface. If the compacted subgrade is disturbed during utility trench or foundation excavations, the subgrade should be re-rolled to provide a smooth, firm surface for concrete slab support.

Where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a plug consisting of native clay or lean concrete, at least five feet in length, should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the pavements. This trapped water can cause softening of subgrade soil beneath pavement areas.

15 Low corrosion potential is defined as a minimum resistivity of 2,000 ohms-cm and maximum sulfate and chloride concentrations of 250 parts per million.

If used, sand containing less than 10 percent fines (particles passing the No. 200 sieve) should be compacted to at least 95 percent relative compaction. Samples of on-site and proposed import fill materials should be submitted to Langan for approval at least three business days prior to use at the site.

7.10 Utilities and Utility Backfill

Seismically-induced settlements of up to 1¾ inches, with differential settlement of about one inch over a horizontal distance of about 30 feet could occur outside the basement footprint. Where utilities enter and exit the building and the anticipated differential settlement is not tolerable, flexible connections should be used to allow for the differential movement.

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on all sides. Where necessary, trench excavations should be shored and braced to prevent cave-ins and/or in accordance with applicable safety regulations. If trenches extend below the groundwater level, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits and backfill.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped to at least 90 percent relative compaction. If fill with less than 10 percent fines is used, the entire depth of the fill should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted.

Special care should be taken when backfilling utility trenches in pavement or hardscape areas. Poor compaction may cause excessive settlements resulting in damage to the pavement or hardscape section. As discussed in Section 7.9, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, an impermeable plug should be placed at the edge of the pavement to reduce the potential for water to become trapped in trenches beneath the building or pavements.

The corrosivity results provided in Appendix D of this report should be reviewed and corrosion protection measures used to protect underground utilities, if needed. We recommend a corrosion consultant be retained when detailed corrosion protection recommendations are needed.

7.11 Construction Monitoring

A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements. The contractor should install surveying points to monitor the movement of shoring and settlement of the adjacent ground surface during below-grade construction activities. The shoring system and adjacent improvements should be monitored for movements throughout the excavation and at least until the street-level slab is cast.

To monitor ground movements, adjacent improvement movements, groundwater levels, and shoring movements during the excavation activities, we recommend installing the instrumentation listed below:

Slope inclinometers: We recommend installing six slope inclinometers adjacent to the proposed shoring system. Two slope inclinometers should be installed behind, if feasible, or within both the east and west shoring walls because of the proximity to the neighboring buildings and VTA infrastructure along South 2nd Street. One slope inclinometer should be installed behind or within both the north and south shoring walls. The slope inclinometers should be installed following the installation of the cutoff wall and prior to excavation.

Piezometers: We recommend two groundwater monitoring wells be installed to monitor the groundwater level outside the excavation. In addition, we recommend at least one groundwater monitoring well be installed near the center of the site and extend into the sand layers below the bottom of excavation to monitor the potential for blowout of the subgrade. The upper portions of the piezometers should be properly sealed with cement-bentonite mix to reduce surface water infiltration. Baseline groundwater level readings should be collected before the start of onsite dewatering.

Survey points: Survey points should be installed on the shoring (at top and mid-height) and on adjacent streets and improvements that are within 150 feet of the proposed excavation. These points should be used to monitor the vertical and horizontal movements of the shoring and the nearby improvements. The survey point locations should be selected with the help of the geotechnical and structural engineers, so they can provide the most value to the project. The survey points should be read regularly and the results should be submitted to us in a timely manner for review. For estimating purposes, assume that the survey points will be read as follows:

- prior to the start of any shoring work at the site

- after installing cutoff wall elements
- weekly during excavation work
- after the excavation reaches the planned subgrade level
- every week until the street-level floor slab is constructed.

We should obtain inclinometer and piezometer readings regularly. Initially, depending upon the speed of excavation, the instrumentation should be read about every week. The frequency of readings may be modified, as appropriate, in the later stage of construction.

In addition, the conditions of existing buildings within 150 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction. A thorough crack survey of the adjacent buildings, especially those surrounding the proposed excavation, should be performed prior to the start of construction and immediately after its completion.

8.0 GEOTECHNICAL SERVICES DURING DESIGN AND CONSTRUCTION

We should be provided with foundation loads and layouts of the neighboring buildings along the south and west sides of the site so we can evaluate surcharge pressures that may be imposed from these footings onto the planned shoring system and permanent basement walls. Additionally, we should be provided with loading information from the adjacent VTA infrastructure along the east side of the site so we can develop similar surcharge pressures. When a soil improvement contractor is engaged to design ground improvement below the mat foundation, we should review the ground improvement and consult with the project team. We should also consult with the soil improvement contractor and project structural engineer to refine the settlement and modulus of subgrade reaction during final design.

During final design we should also be retained to consult with the design team as geotechnical questions arise. Prior to construction, we should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, grading, placement and compaction of fill, installation of building foundations and ground improvement, shoring, and testing of tiebacks and tiedowns, as appropriate. These observations will allow us to compare actual with anticipated soil conditions and to check that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the site inferred from a limited number of exploration points as well as existing conditions information provided by Kier & Wright, architectural information provided by BIG, and structural information provided by Glotman Simpson. Actual subsurface conditions could vary. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others. Any proposed changes in structures, depths of excavation, or their locations should be brought to Langan's attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan's attention for evaluation, as they may affect our recommendations.

This report has been prepared to assist the Owner, architect, and structural engineer in the design process and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities on adjacent properties which are beyond the limits of that which is the specific subject of this report.

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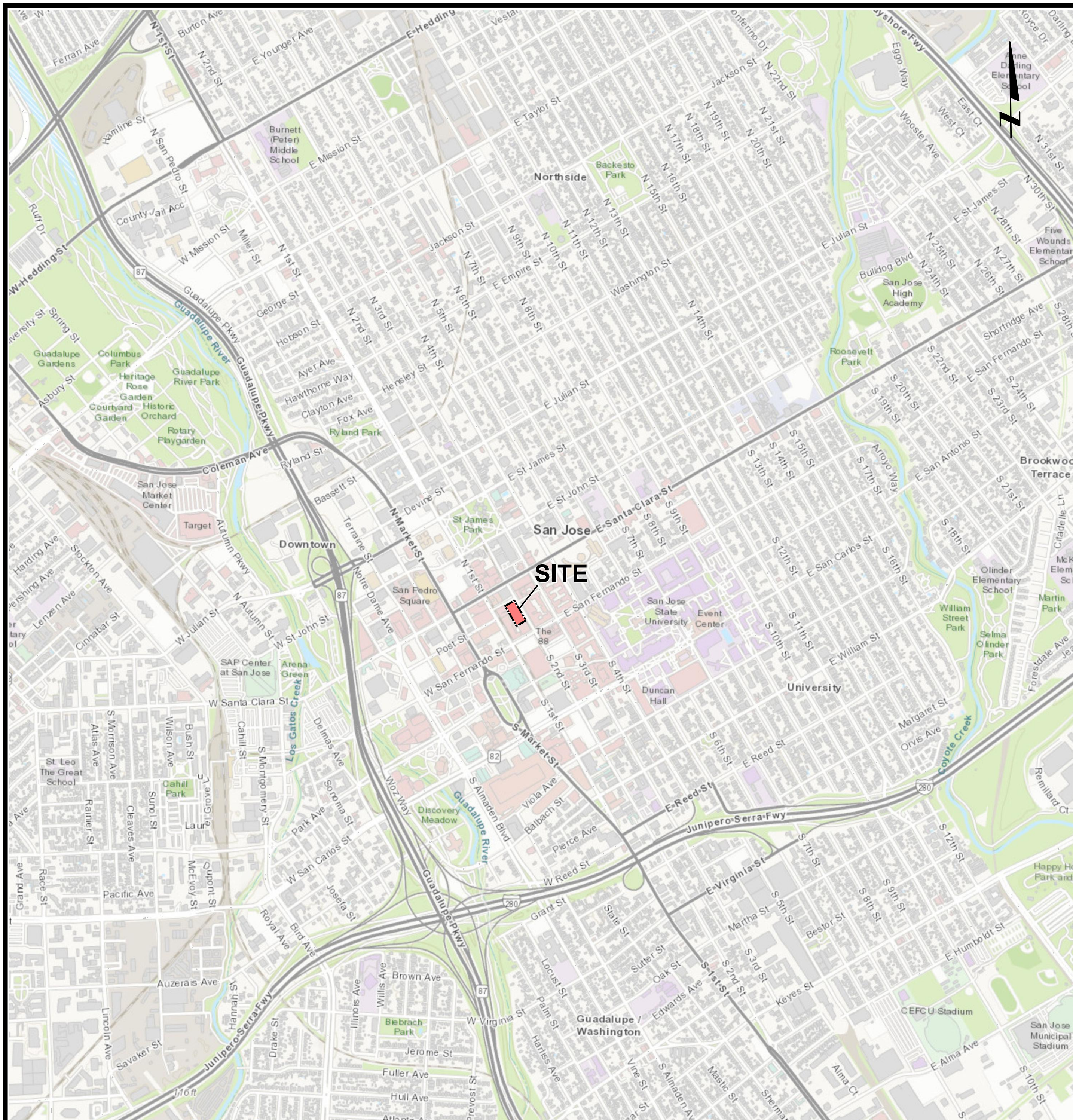
FIGURES



Legend

Approximate site boundary

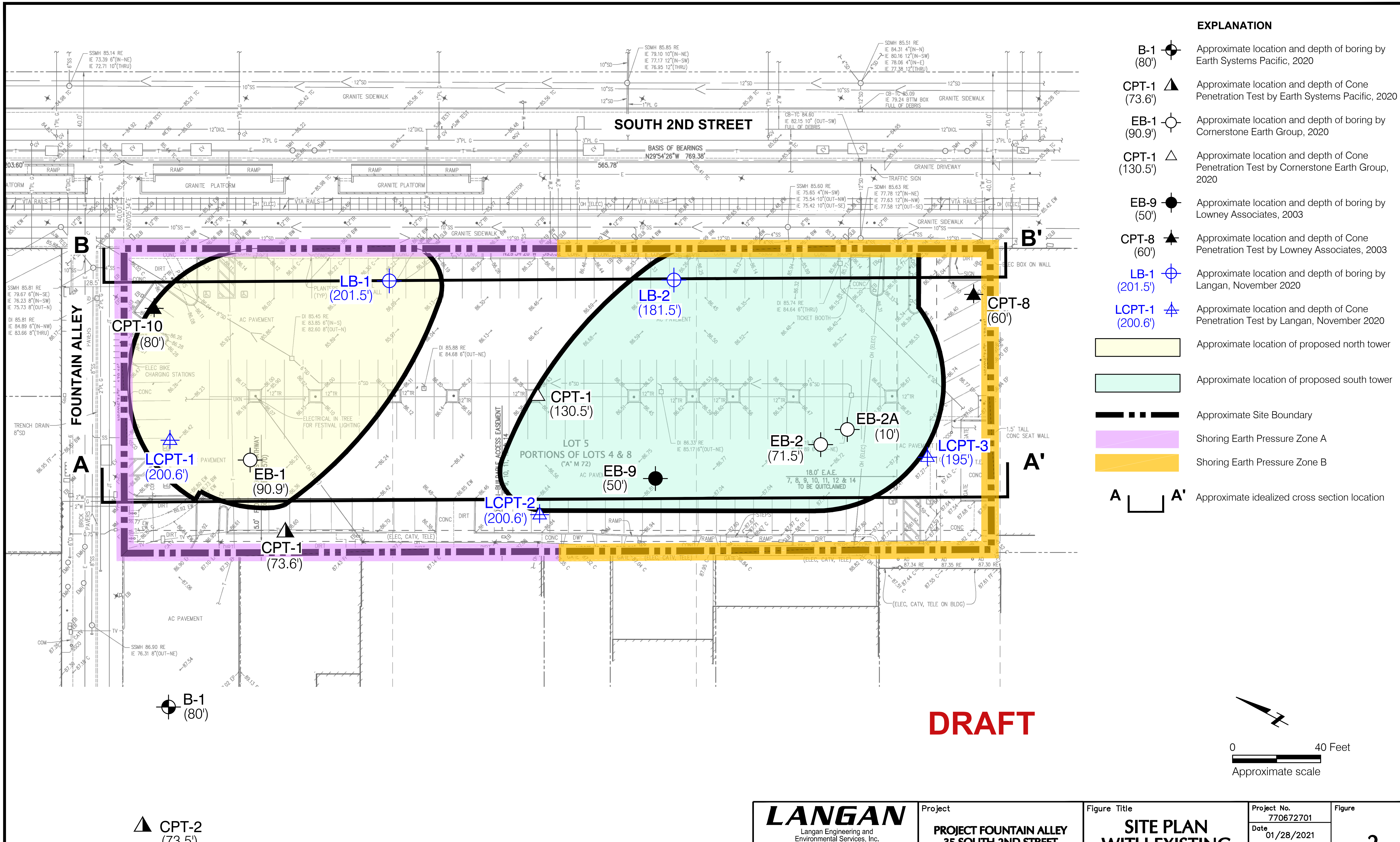
DRAFT

Note:
Topographic base map is provided through Langan's Esri Arc GIS software licensing and Arc GIS online, National Geographic Society, i-cubed.

0 2,000 Feet

Approximate scale

LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	SITE LOCATION MAP	770672701	1
			Date 11/17/2020	
			Drawn By JDF	
			Checked By TL	



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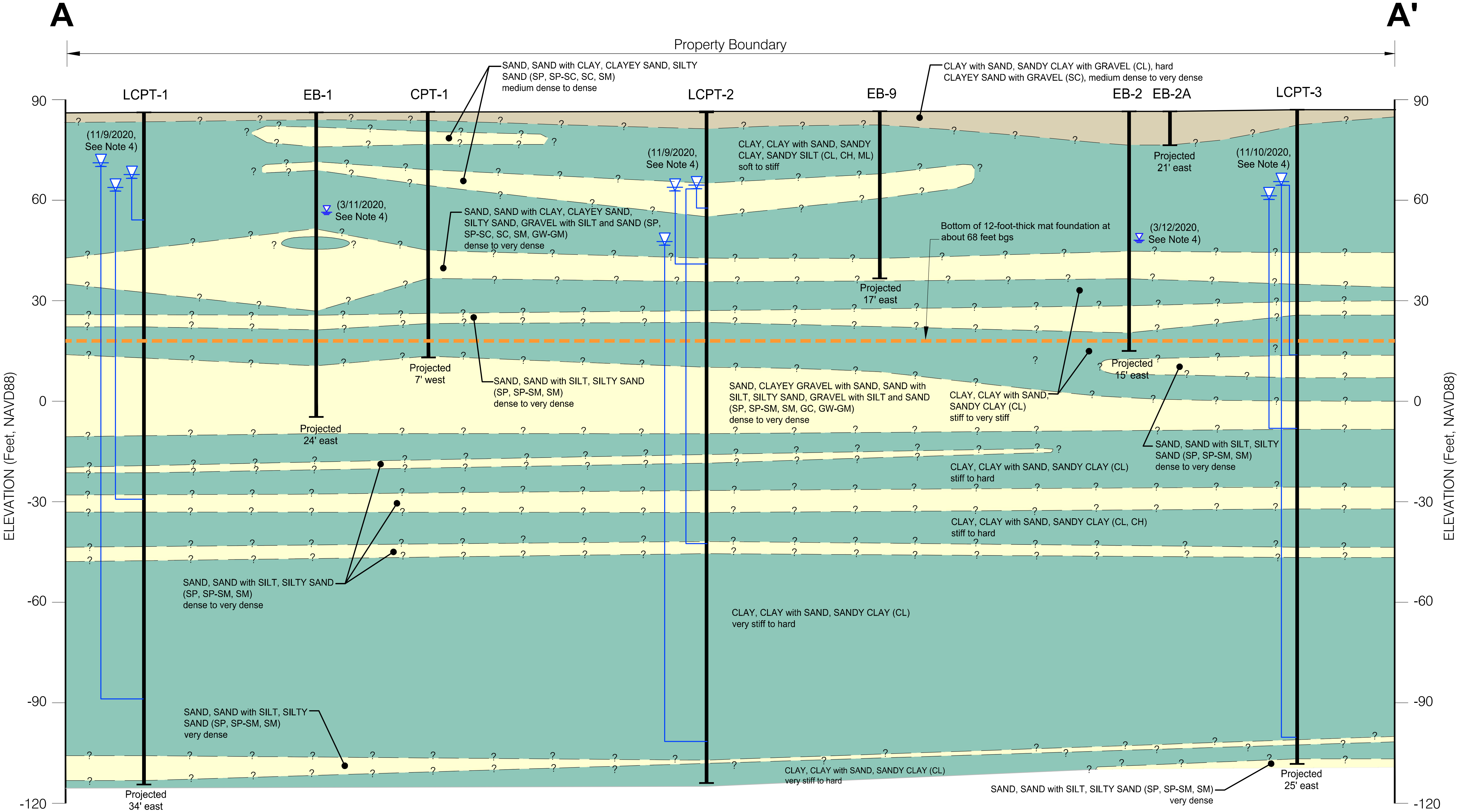
Project	Figure Title		Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	SITE PLAN WITH EXISTING CONDITIONS	Project No. 770672701 Date 01/28/2021 Drawn By JDF Checked By TL	2

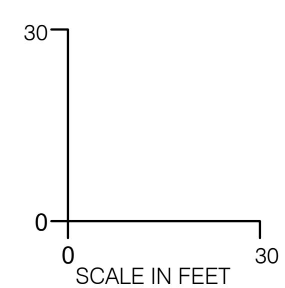
Reference: "Existing Conditions Plan, Sheet C1.01" by Kier & Wright, dated 10/29/2020.

Filename: \\langan.com\data\SJO\data7\770672701\Project Data\CAD\01\2D-DesignFiles\Geotechnical\FG01-770672701-B-GI0101.dwg Date: 2/1/2021 Time: 18:10 User: agekas Style Table: Langan.stb Layout: Fig 2 NEW Site Plan



EXPLANATION

- Fill
- Clay and Silt
- Sand and Gravel
- Piezometric reading
- Water level



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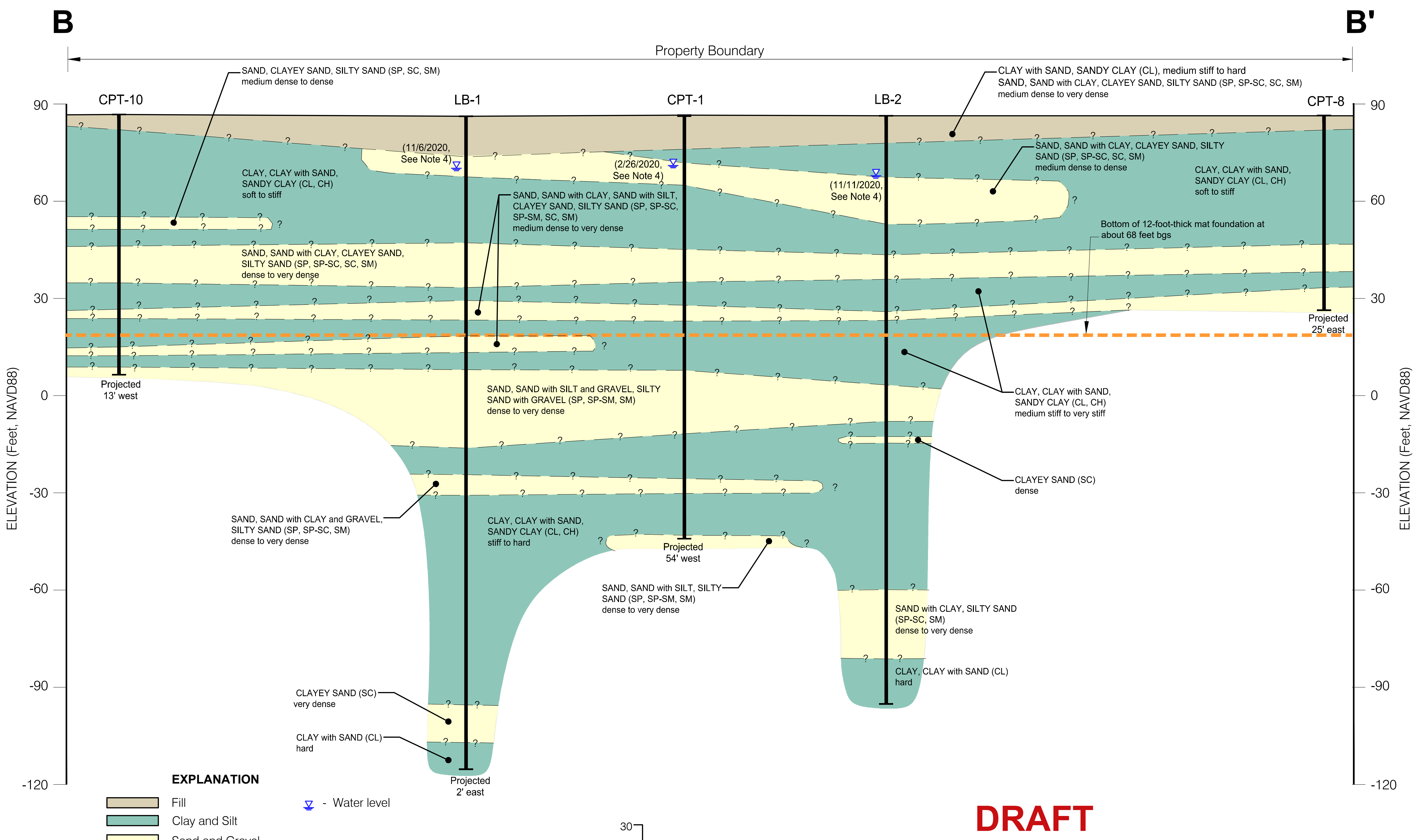
Notes:
1. The above profile represents a generalized subsurface cross section interpreted from widely spaced subsurface exploration points. Soil deposits may vary in type, strength, and other important properties between points of exploration.
2. Existing grades from topographic survey prepared by Kier & Wright titled "Existing Conditions Plan, Sheet C1.01" , dated 10/29/2020.
3. Idealized subsurface profile location shown on Figure 2 (Site Plan).
4. Groundwater levels shown were observed during field exploration and may not be representative of stabilized groundwater levels.
5. Proposed bottom of mat foundation from the structural drawings prepared by Goltman Simpson titled "San Jose Fountain Alley 100% Schematic Design, San Jose, California" dated 30 October 2020.

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project: PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	Figure Title: IDEALIZED SUBSURFACE PROFILE A-A'	Project No. 770672701 Date 02/17/2021 Drawn By AG Checked By TL	Figure 3

Filename: \\langan.com\data\SJO\data7\770672701\Project Data\CAD\01\2D-DesignFiles\Geotechnical\770672701-B-XS0101.dwg Date: 2/17/2021 Time: 13:20 User: jfrank Style Table: Apple LaserWriter 8500.ctb Layout: A-A



DRAFT

Notes:
1. The above profile represents a generalized subsurface cross section interpreted from widely spaced subsurface exploration points. Soil deposits may vary in type, strength, and other important properties between points of exploration.
2. Existing grades from topographic survey prepared by Kier & Wright titled "Existing Conditions Plan, Sheet C1.01" , dated 10/29/2020.
3. Idealized subsurface profile location shown on Figure 2 (Site Plan).
4. Groundwater levels shown were observed during field exploration and may not be representative of stabilized groundwater levels.
5. Proposed bottom of mat foundation from the structural drawings prepared by Goltman Simpson titled "San Jose Fountain Alley 100% Schematic Design, San Jose, California" dated 30 October 2020.

30
0
0 30
SCALE IN FEET

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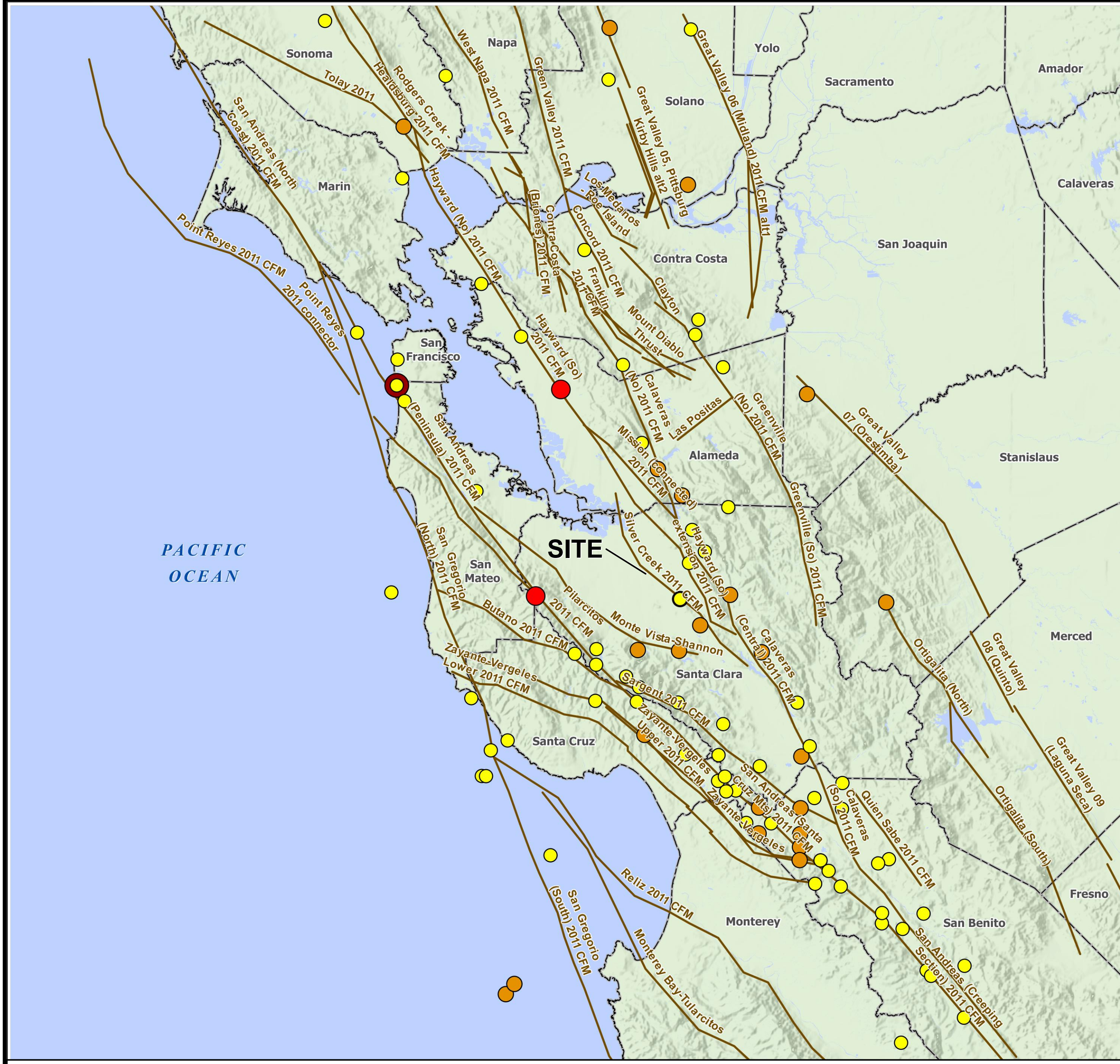
Project	Figure Title	Project No. 770672701	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	IDEALIZED SUBSURFACE PROFILE B-B′	Date 02/17/2021 Drawn By AG Checked By TL	4



Legend

County Boundary

Fault

DRAFT

Notes:

1. Quaternary fault data displayed are based on a generalized version of USGS Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 08/26/2014.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83) , Meters.

Earthquake Epicenter Magnitude

Magnitude 5 to 5.9

Magnitude 6 to 6.9

Magnitude 7 to 7.4

Magnitude 7.5 to 8

0 20 Miles

Approximate scale

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Project	Figure Title	Project No. 770672701	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE	MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA	Date 11/17/2020	5
SANTA CLARA COUNTY CALIFORNIA		Drawn By JDF	
		Checked By TL	

Filename: \\langan.com\data\SJO\data7\770672701\Project Data\CAD\01\2D-DesignFiles\Geotechnical\FG01-770672701-B-GI0101.dwg Date: 1/29/2021 Time: 11:04 User: agekas Style Table: Langan.stb Layout: Fig XX Fault Map

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X Panic is general.
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

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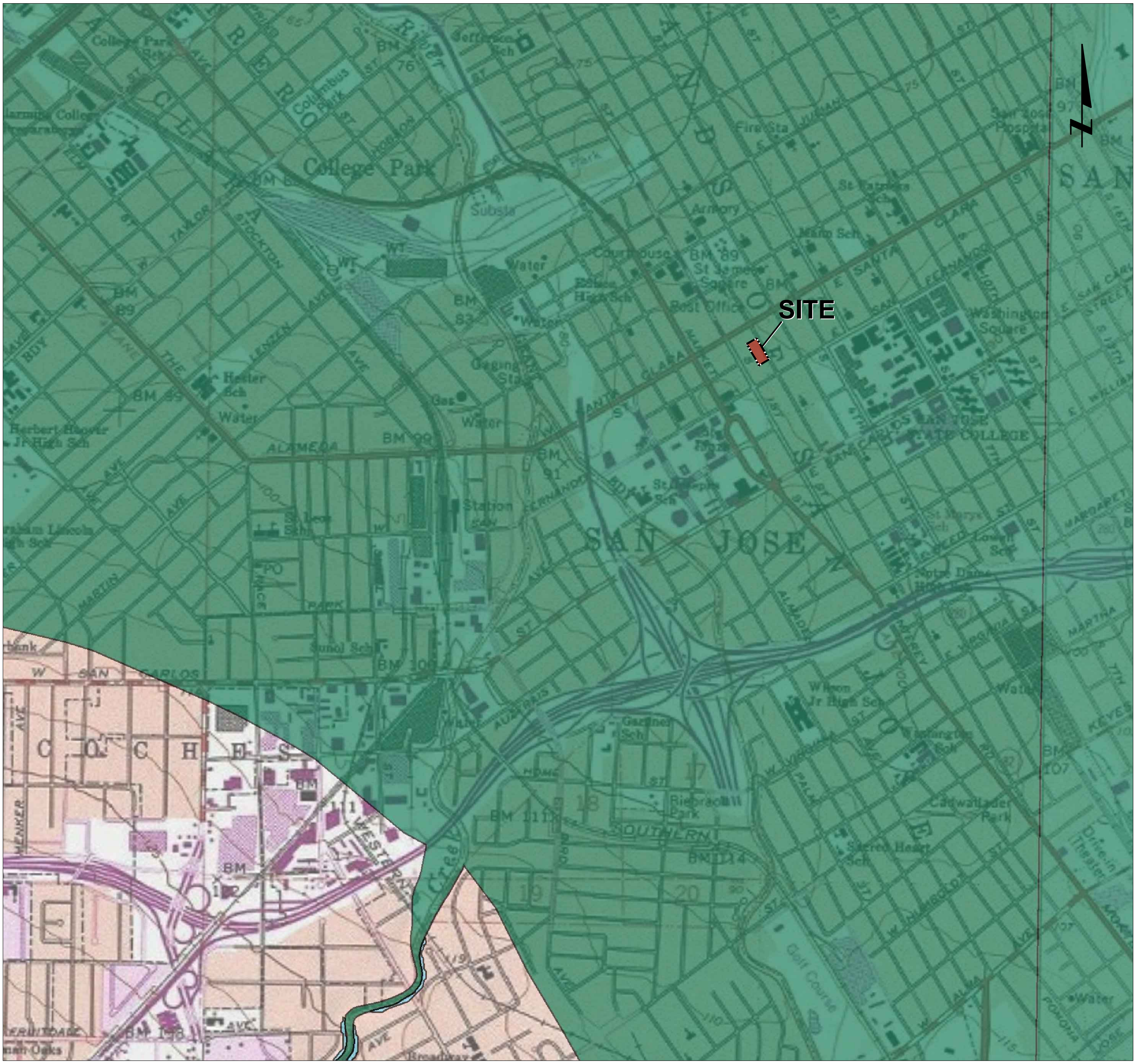
LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	MODIFIED MERCALLI INTENSITY SCALE	770672701 Date 11/17/2020 Drawn By JDF Checked By TL	6



EXPLANATION

Liquefaction; Areas where historic occurrence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

DRAFT

Notes:
1. Topographic basemap is provided through Langan's Esri Arc GIS software licensing and Arc GIS online, National Geographic Society, i-cubed and the USGS.
2. Data provided by the CGS through the GIS Seismic Hazard Zone Map presenting are as where liquefaction and land slides may occur during a strong earthquake.

0 2000 Feet

Approximate scale

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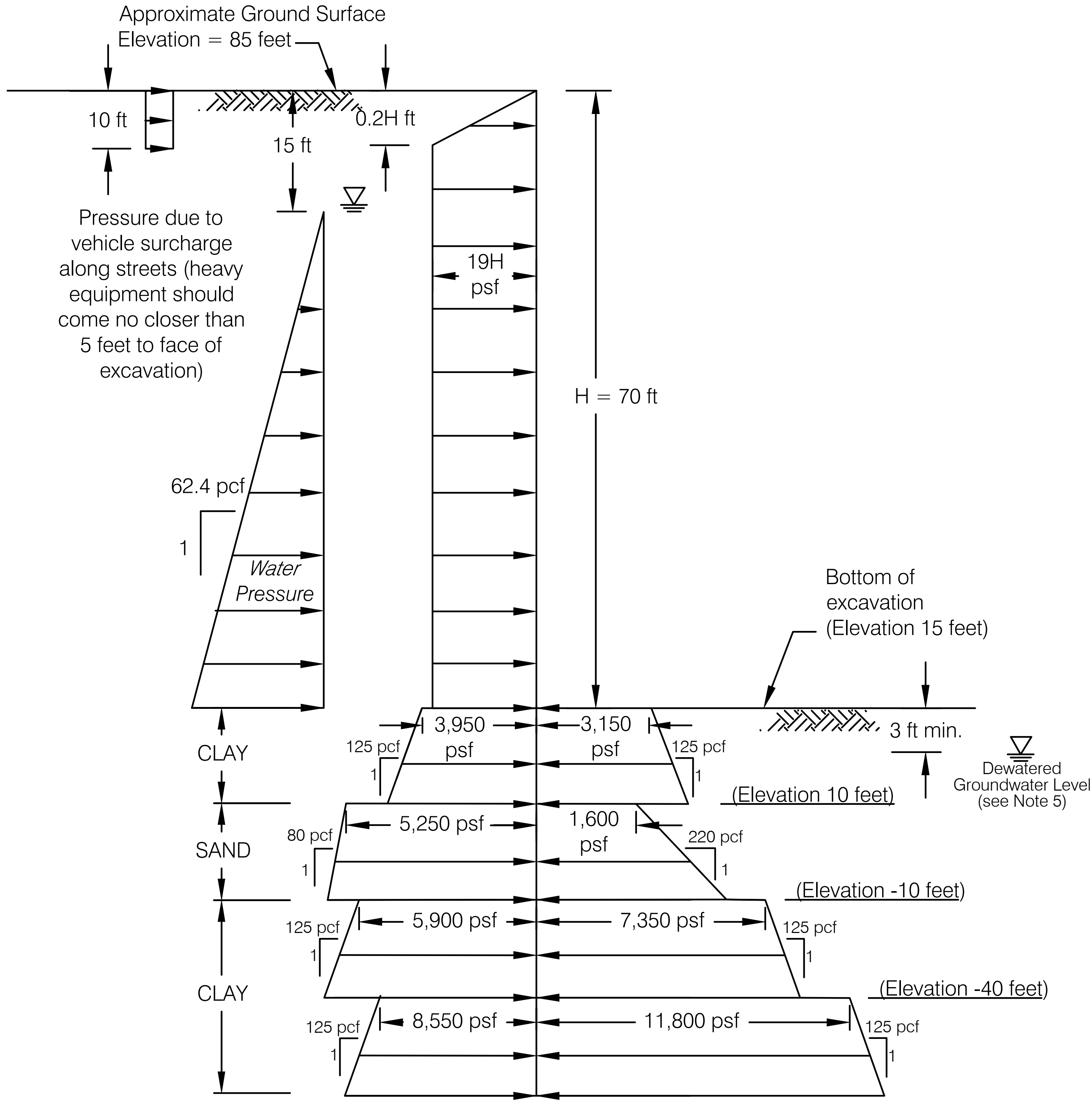
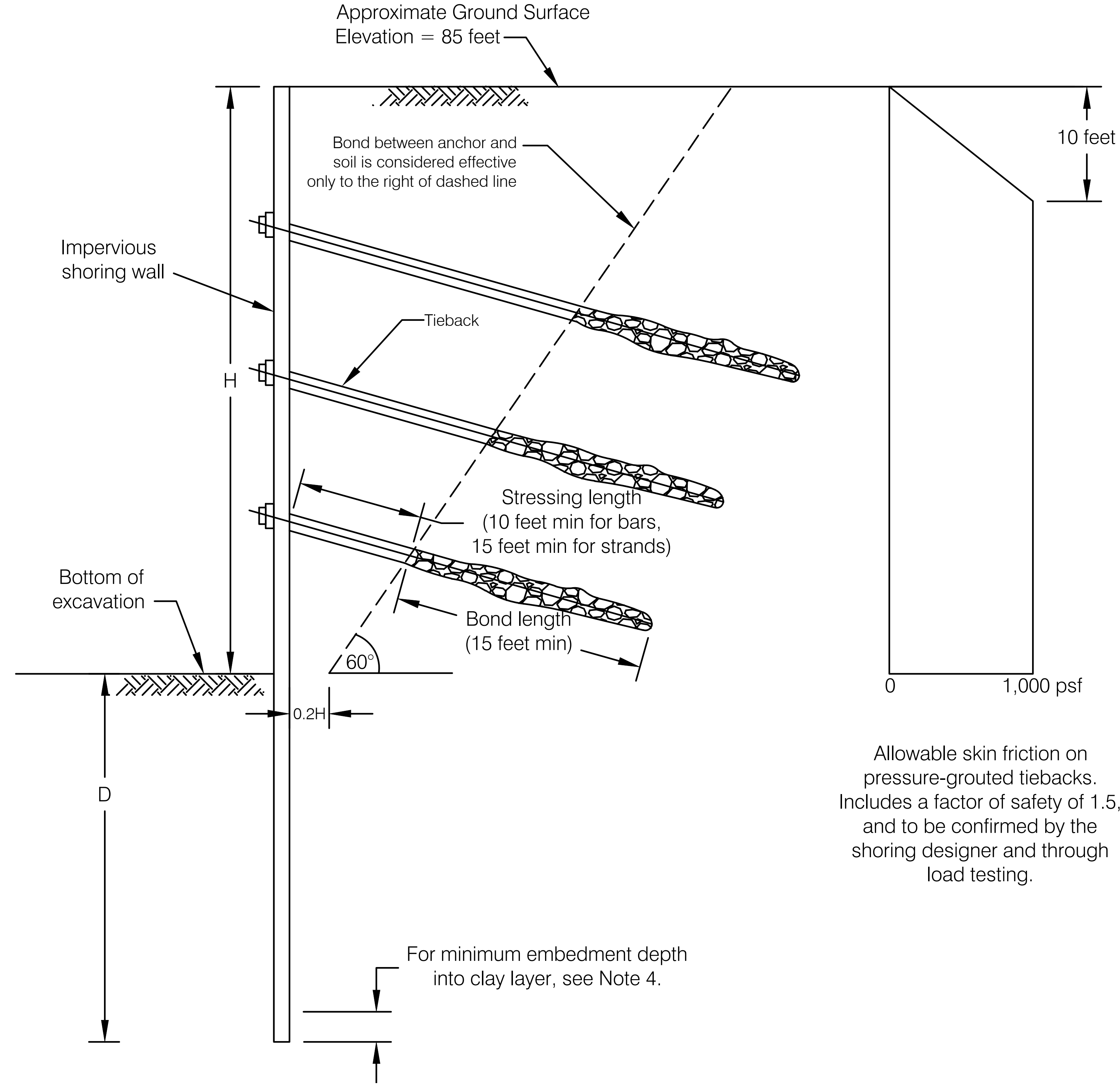
Project	Figure Title	Project No. 770672701	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	REGIONAL SEISMIC HAZARD ZONES MAP	Date 11/17/2020 Drawn By JDF Checked By TL	7

Filename: \\langan.com\data\SJO\data7\770672701\Project Data\CAD\01\2D-DesignFiles\Geotechnical\FG01-770672701-B-GI0101.dwg Date: 2/1/2021 Time: 17:30 User: agekas Style Table: Langan.stb Layout: Fig 7 Haz Map





NOT TO SCALE

DRAFT

Notes:

1. These shoring pressures are applicable to continuous and soil-cement mixed walls with an excavation height of approximately 70 feet.
2. Passive pressure values include a factor of safety of 1.5.
3. psf = pounds per square foot; pcf = pounds per cubic foot; H and D are in feet.
4. The tip of the shoring cutoff panel should extend at least 10 feet into a clay layer and have a minimum tip elevation of at least -60 feet.
5. Groundwater should be drawn to a minimum of three feet below bottom of excavation.
6. Assumes shoring will be braced or tied back; pressures presented assume the internal bracing is preloaded, if used.
7. Surcharge pressure from construction equipment, if any, should be added to the above shoring pressures.
8. The recommended pressures do not include surcharges from adjacent buildings or the Santa Clara Valley Transportation Authority (VTA) light rail along South 2nd Street. Surcharge pressures from adjacent buildings and the VTA light rail should be added to the above shoring pressures.
9. The shoring engineer should check the hydrostatic pressure in the sand layers below the bottom of excavation for bottom of excavation stability. The sand layers will likely have to be depressurized as discussed in Section 6.4 of the report.
10. Embedment depth D, should be determined by the shoring designer, and coordinated with the dewatering designer.
11. The approximate extent of Zone A is shown on Figure 2.

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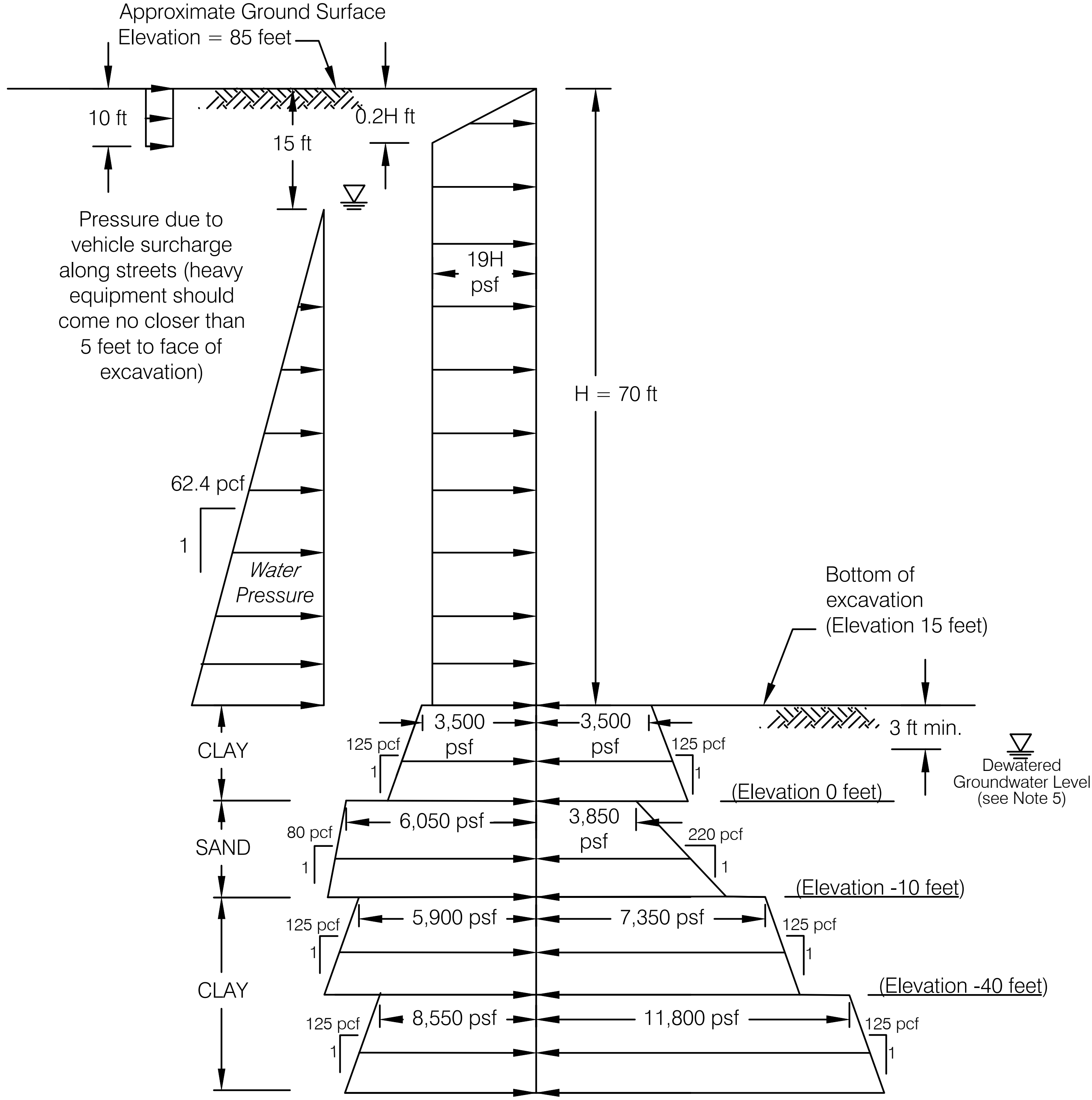
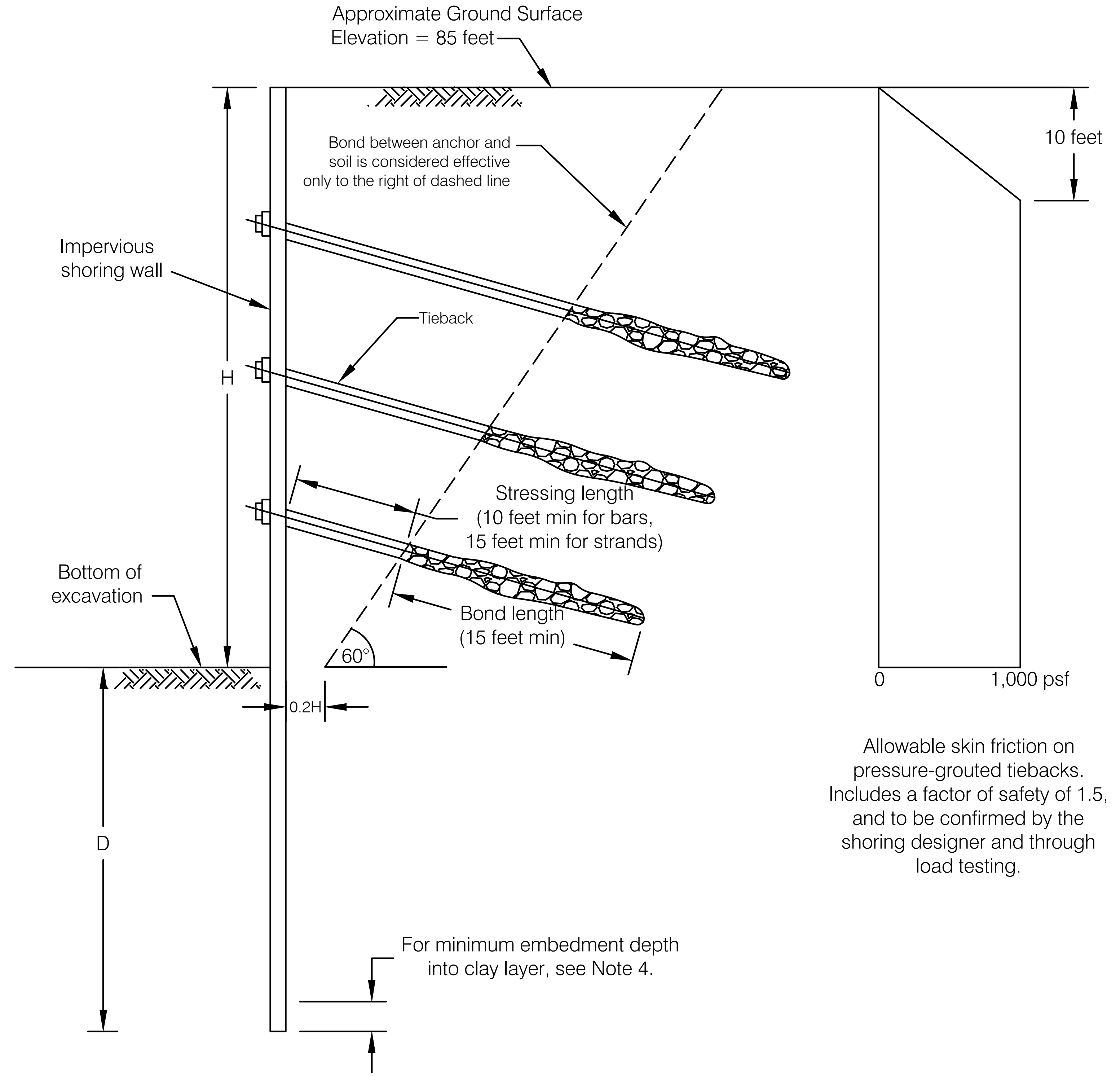
Project	Figure Title	Project No.	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	DESIGN PARAMETERS FOR TEMPORARY IMPERVIOUS SHORING WALL IN ZONE A	770672701 Date 02/17/2021 Drawn By AG Checked By TL	8

Filename: \\langan.com\data\SJO\data7\770672701\Project Data\CAD\01\2D-DesignFiles\Geotechnical\770672701-B-RW0104.dwg Date: 2/17/2021 Time: 13:52 User: jfrank Style Table: Apple LaserWriter 8500.ctb Layout: NEW_Figure 8





Notes:

1. These shoring pressures are applicable to continuous and soil-cement mixed walls with an excavation height of approximately 70 feet.
2. Passive pressure values include a factor of safety of 1.5.
3. psf = pounds per square foot; pcf = pounds per cubic foot; H and D are in feet.
4. The tip of the shoring cutoff panel should extend at least 10 feet into a clay layer and have a minimum tip elevation of at least -60 feet.
5. Groundwater should be drawn to a minimum of three feet below bottom of excavation.
6. Assumes shoring will be braced or tied back; pressures presented assume the internal bracing is preloaded, if used.
7. Surcharge pressure from construction equipment, if any, should be added to the above shoring pressures.
8. The recommended pressures do not include surcharges from adjacent buildings or the Santa Clara Valley Transportation Authority (VTA) light rail along South 2nd Street. Surcharge pressures from adjacent buildings and the VTA light rail should be added to the above shoring pressures.
9. The shoring engineer should check the hydrostatic pressure in the sand layers below the bottom of excavation for bottom of excavation stability. The sand layers will likely have to be depressurized as discussed in Section 6.4 of the report.
10. Embedment depth D, should be determined by the shoring designer, and coordinated with the dewatering designer.
11. The approximate extent of Zone B is shown on Figure 2.

NOT TO SCALE

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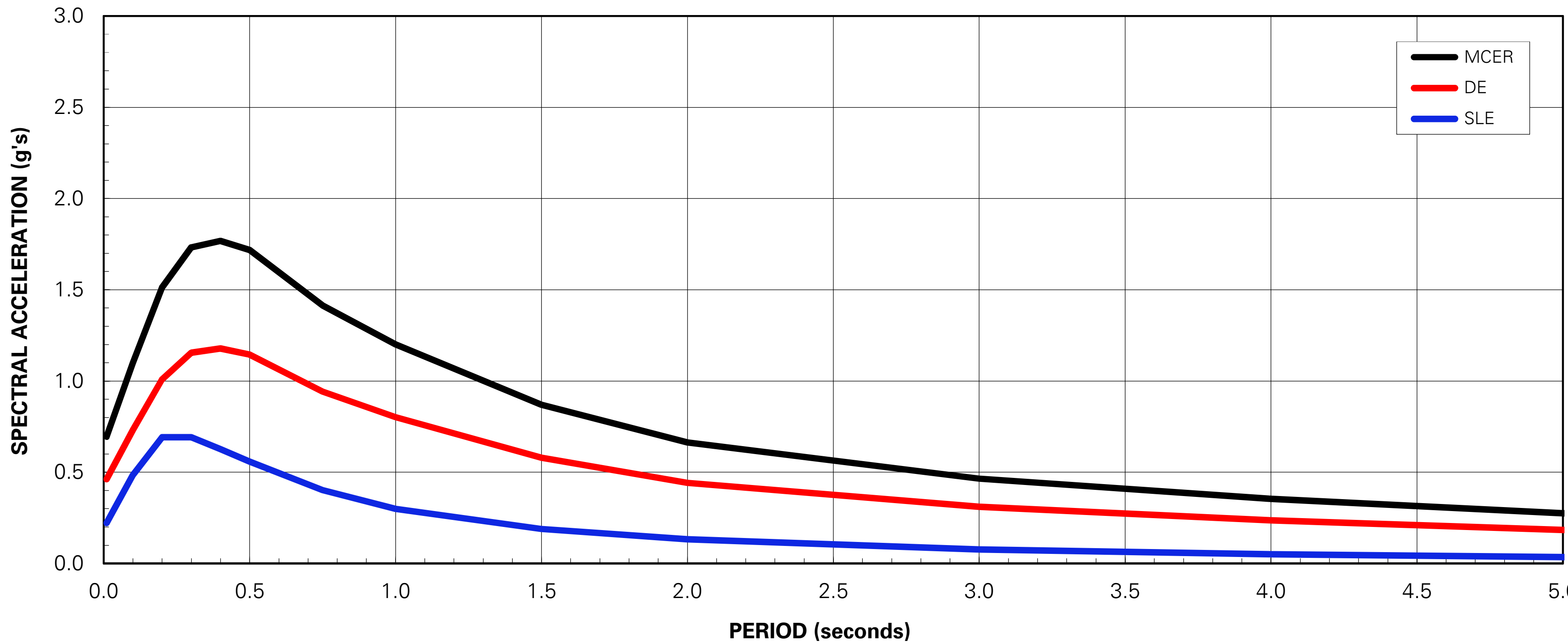
Filename: \\langan.com\data\SJO\data7\770672701\Project Data\CAD\01\2D-DesignFiles\Geotechnical\770672701-B-RW0104.dwg Date: 2/17/2021 Time: 14:00 User: jfrank Style Table: Apple LaserWriter 8500.ctb Layout: NEW_Figure 9



Notes

1. Damping Ratio = 5 percent for the MCE_R and DE, and 2.1 percent for the SLE
2. Estimated V_{S30} = 280 m/s

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LANGAN ENGINEERING AND ENVIRONMENTAL SERVICES, INC. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	**PROJECT FOUNTAIN ALLEY** **35 SOUTH 2ND STREET** CITY OF SAN JOSE SANTA CLARA COUNTY CALIFORNIA	**RECOMMENDED SPECTRA**	770672701 Date 12/29/2020 Scale AS SHOWN Prepared By: TJL	**10**

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APPENDIX A

LOGS OF BORINGS BY LANGAN (2020)

PROJECT:	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET San Jose, California	Log of Boring LB-1

Boring location:	See Site Plan, Figure 2	Logged by:	C. Aytekin
Date started: 11/5/20	Date finished: 11/9/20	Drilled By:	Pitcher Services, LLC
Drilling method:	Rotary Wash		
Hammer weight/drop: 140 lbs. / 30"	Hammer type: Automatic Safety		
Samplers:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Pitcher Barrel (PB)		

Ground Surface Elevation: 86.2 feet [2]

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	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						4 inches Asphalt Concrete (AC)						
0.5						6.5 inches Aggregate Base (AB)						
1–3	HA				CL	CLAY (CL) yellow-brown to brown, moist, trace fine sand [FILL] LL = 39, PI = 21, see Figure C-1						
3.5–5					SP-SC	SAND with CLAY (SP-SC) yellow-brown, moist, fine-grained, with fine to coarse subangular gravel [FILL]						
5–6	SPT		5 10 34	53	SC	CLAYEY SAND (SC) yellow-brown to brown, very dense, moist, fine-grained, with brick debris, trace fine to coarse subangular gravel, trace silt [FILL]						
6–12					SP	SAND (SP) red-brown to red, very dense, moist, fine-grained, with brick debris [FILL]						
9						grades with increase brick debris						
10	SPT		50/ 4"	60/ 4"		fine- to medium-grained, trace silt						
10.5	SPT		50/ 5"	60/ 5"								
12–14.5					CL	CLAY (CL) in drilling cuttings gray, moist						
14.5–19					SC	CLAYEY SAND (SC) brown to yellow-brown, medium dense, moist, fine-grained, trace silt						
16	SPT		4 7 7	17		▽ (11/6/20, 6:43AM) wet				19.3	21.5	
19–28.5					CL	CLAY with SAND (CL) gray to gray-brown, soft, wet, fine sand, trace fine subangular gravel						
21	S&H		0 0 4	3		gray, trace silt	PP		500			
25.5	S&H		3 4 8	8		gray to gray-brown with orange mottling, stiff, trace wood debris	PP		1,000		23.6	104
28.5–30					CL	SANDY CLAY (CL) gray-brown with orange mottling, medium stiff, wet, fine sand, trace silt						

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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		4 5 4	6	CL	SANDY CLAY (CL) (continued) LL = 32, PI = 12, see Figure C-1 Triaxial Test, see Figure C-8	PP TxUU	 3,050	1,000 940		 25.0	 107
32												
33						grades with increase sand content						
34												
35												
36	SPT		5 6 4	12		gray-brown to yellow-brown, stiff, wet, fine sand, trace silt						
37												
38												
39					SP-SC	SAND with CLAY (SP-SC) yellow to yellow-brown, very dense, wet, fine- to medium-grained, trace fine to coarse angular to subangular gravel, trace silt						
40												
41	SPT		7 27 34	73								
42												
43						grades with increase gravel content						
44												
45						with fine to coarse subangular gravel						
46	SPT		14 23 27	60								
47												
48												
49												
50						increase clay content						
51	SPT		7 25 18	52								
52												
53					CL	CLAY with SAND (CL) gray-brown with orange mottling, stiff, wet, fine sand						
54												
55												
56	S&H		10 7 7	10			PP		1,000		25.5	102
57												
58					SC	CLAYEY SAND (SC) gray-brown, medium dense to dense, wet, fine-grained, trace fine subangular gravel, trace silt						
59												
60												

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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
60–62	SPT		9, 11, 14	30	SC	CLAYEY SAND (SC) (continued)				34.8	16.9	
63.7					CL	CLAY with SAND (CL) gray-brown, medium stiff, wet, fine sand						
65–66.5	SPT		2, 1, 5	7								
68					SC	CLAYEY SAND (SC) gray-brown, medium dense, wet, fine-grained						
70–71.5	S&H		7, 7, 9	11								
73					CL	SANDY CLAY (CL) gray-brown, stiff, wet, fine sand, trace organics						
75–76.5	S&H		6, 9, 11	14			PP		1,750			
78					SP-SM	SAND with SILT and GRAVEL (SP-SM) brown to yellow-brown, dense, wet, fine- to medium-grained, trace fine to coarse subangular gravel						
80–81.5	SPT		23, 23, 18	49								
85–86.5	SPT		23, 34, 37	85		very dense, with fine to coarse subangular gravel						
90												

DEPTH (feet)	Sampler Type	Blows/ 6"	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
90–91	SPT	9, 23, 23	55	SP-SM	SAND with SILT and GRAVEL (SP-SM) (continued) Hydrometer Test, see Figure C-14				12		
95–96	SPT	14, 21, 30	61	SP-SM	gray-brown						
100–101	SPT	10, 28, 25	64	SP-SM	increase gravel content						
102–103				CL	SANDY CLAY (CL) gray to gray-brown, very stiff, wet, fine sand, trace wood debris						
105–106	S&H	14, 14, 21	25	CL							
110–111	PB		95 psi	CL	LL = 33, PI = 15, see Figure C-1 Triaxial Test, see Figure C-9 Consolidation Test, see Figure C-3	TxUU	7,600	2,050		15.3 18.6	119 105
111–112				SP-SC	SAND with CLAY and GRAVEL (SP-SC) yellow-brown to gray-brown, very dense, wet, fine- to coarse-grained, fine to coarse subangular gravel						
112–113	S&H	17, 39, 38	54	SP-SC	Particle Size Analysis, see Figure C-15				9.8	10.4	
113.5–114.5	SPT	29, 42, 37	95	SP-SC	fine- to medium-grained						
117–120				CL	CLAY with SAND (CL) gray-brown, very stiff, wet, fine sand						

TEST GEOTECH LOG 770672701 GEOTECH BORING LOGS.GPJ TEMPLATE_CA-MODIFIED.GDT 2/17/21

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	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
121	S&H		10 17 29	32	CL	CLAY with SAND (CL) (continued)	PP		2,500			
122												
123												
124												
125												
126												
127												
128												
129												
130												
131	SPT		5 8 9	20		gray						
132												
133												
134												
135												
136												
137												
138												
139												
140												
141	PB			125 psi		stiff LL = 30, PI = 15, see Figure C-1 Triaxial Test, see Figure C-10 Consolidation Test, see Figure C-4	TxUU	9,300	1,330		19.3 19.8	111 108
142												
143												
144												
145												
146												
147												
148												
149					CL	SANDY CLAY (CL)						
150												

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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
151	S&H		8 14 29	30	CL	SANDY CLAY (CL) (continued) gray-brown with orange mottling, very stiff, wet, fine sand, trace fine to coarse subangular gravel	PP		3,000			
156						grades with increase gravel content						
160–161	SPT		9 8 10	22		very stiff, trace fine subangular gravel						
162–163					CL	CLAY (CL) gray-brown with orange mottling, hard, wet, trace fine subangular gravel, trace fine sand						
170–171	SPT		7 12 19	37								

Depth scale: 151 152 153 154 155 156 157 158 159 160 161 162 163 164 165 166 167 168 169 170 171 172 173 174 175 176 177 178 179 180

PROJECT:	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET San Jose, California	Log of Boring LB-1

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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
181	S&H		8 19 34	37	CL	CLAY (CL) (continued) yellow-brown with orange mottling, very stiff, trace organics Consolidation Test, see Figure C-5 LL = 28, PI = 12, see Figure C-1	PP		3,500		20.2	108
183–193					SC	CLAYEY SAND (SC) yellow-brown, very dense, wet, fine-grained, trace silt						
191	SPT		17 24 37	73		yellow-brown with orange mottling						
193–201.5					CL	CLAY with SAND (CL) gray-brown, hard, wet, fine sand, trace silt						
201	SPT		8 15 20	42								

Boring terminated at a depth of about 201.5 feet below ground surface (bgs).
Boring backfilled with cement grout per Santa Clara Valley Water District (SCVWD) requirements.
Groundwater encountered at about 16.3 feet bgs on 11/6/2020 at 6:43AM.
PP = Pocket penetrometer

[1] S&H and SPT blow counts were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on a topographic survey titled, "Existing Conditions Plan, Sheet C1.01" by Kier & Wright, dated 10/29/2020 and reference the North American Vertical Datum of 1988 (NAVD88).

LANGAN

Project No.: 770672701 | Figure: A-1g

Log of Boring LB-2

PROJECT:	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET San Jose, California
Boring location:	See Site Plan, Figure 2
Date started:	11/9/20
Date finished:	11/11/20
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs. / 30"
Hammer type:	Automatic Safety
Samplers:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tub (ST), Pitcher Barrel (PB)
Logged by:	C. Aytekin
Drilled By:	Pitcher Services, LLC

Ground Surface Elevation: 86.3 feet [2]

DEPTH (feet)	Sampler Type	Blows/ 6"	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					5.5 inches Asphalt Concrete (AC)						
0.5					4 inches Aggregate Base (AB)						
1–5	HA (2–3.5)			CL	SANDY CLAY (CL) yellow-brown to dark brown, moist, fine sand, trace fine to coarse subangular gravel, trace rootlets [FILL]						
5–8.5	S&H (5–6.5)	2, 3, 4	5	CL	CLAY with SAND (CL) yellow brown with orange mottling, medium stiff, moist, trace silt, trace organics [FILL]						
8.5–19.5	S&H (10–11.5)	2, 4, 5	6	CL	grades gray-brown CLAY with SAND (CL) gray-brown with orange mottling to olive-gray with red mottling, medium stiff, moist, trace fine sand, trace organics					27.4	90
13					grades with increase sand content						
15	S&H (15–16.5)	4, 7, 7	10		gray-brown with orange mottling, stiff LL = 31, PI = 11, see Figure C-1						
19					▽ (11/11/20, 6:15AM)						
19.5–30	S&H (20–21.5)	4, 7, 8	11	SM	SILTY SAND (SM) yellow-brown to gray-brown, medium dense, wet, fine- to medium-grained, trace fine angular to subangular gravel, trace clay and wood debris						
25	SPT (25–26.5)	14, 21, 21	50		dense to very dense						

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LANGAN

Project No.: 770672701

Figure: A-2a

Log of Boring LB-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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	SPT		9 9 11	24		SILTY SAND (SM) (continued) yellow-brown, medium dense				12.6	19.1	
32					SM							
33												
34						SANDY CLAY (CH) gray, stiff, wet, fine sand, trace silt						
35												
36	S&H		1 5 8	9	CH		PP		1,250			
37												
38												
39						CLAY with SAND (CH) gray, medium stiff to stiff, wet, fine to medium sand, trace silt						
40												
41	SPT		2 2 5	8	CH							
42				95 to 500 psi								
43	ST					CLAYEY SAND (SC) gray-brown to yellow-brown, dense, wet, fine- to coarse-grained, trace fine to coarse subangular gravel						
44	SPT		11 11 22	40								
45												
46												
47					SC							
48												
49						grades with increase gravel content						
50												
51	SPT		10 8 5	16		CLAY (CL) gray-brown to yellow-brown with orange mottling, very stiff, wet, trace fine to medium sand, trace organics						
52	S&H		5 10 15	18	CL							
53												
54						CLAY with SAND (CL) gray-brown to yellow-brown with orange mottling, stiff, wet, fine sand, trace silt, trace organics						
55												
56	S&H		8 8 10	13			PP		1,250			
57					CL	grades with increase gravel content						
58												
59												
60												

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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		7 35 37	50	CL	CLAY with SAND (CL) (continued) gray, very stiff, trace fine gravel	PP		2,500			
62					SP-SC	SAND with CLAY (SP-SC) gray to yellow-brown, dense to very dense, wet, fine- to medium-grained, trace fine to coarse subangular gravel						
63–67	S&H (65–66)		0 6 9	11	CL	CLAY (CL) gray-brown, stiff, wet, trace fine to medium sand and silt						
68–71	S&H (70–71)		5 7 9	11	CL	SANDY CLAY (CL) gray-brown, very stiff, fine sand, trace silt and organics	PP		2,750		23.2	104
75–76	S&H		9 12 21	23		very stiff, trace coarse subrounded gravel and organics	PP		1,750			
76–78					CL	CLAY with GRAVEL (CL) yellow-brown, very stiff, wet, fine to coarse subangular gravel, trace fine to medium sand and silt						
78–83	S&H (80–81)		4 7 9	11	CL	SANDY CLAY (CL) yellow-brown, stiff, wet, fine sand, trace silt	PP		1,000			
83						grades with increase sand content						
83–90	SPT (85–86)		17 29 34	76	SM	SILTY SAND with GRAVEL (SM) yellow-brown, very dense, wet, fine- to medium-grained, trace fine to coarse subangular gravel						

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PROJECT:	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET San Jose, California	Log of Boring LB-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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		9 6 20	31	SM	SILTY SAND with GRAVEL (SM) (continued) dense, increased fines content Hydrometer Test, see Figure C-16				17		
92												
93												
94												
95						CLAY (CH) gray-brown with orange mottling, very stiff, wet, trace fine- to medium sand and silt						
96	S&H		11 11 15	18	CH	LL = 50, PI = 33, see Figure C-1 Triaxial Test, see Figure C-11	PP TxUU	6,800	3,000 2,350		26.4	101
97												
98												
99												
100					SC	CLAYEY SAND (SC) gray, dense, wet, trace silt						
101	S&H		7 17 30	33		SANDY CLAY (CL) gray-brown with orange mottling, hard, wet, fine sand, trace silt	PP		4,250	57.4	21.6	
102												
103												
104												
105												
106	S&H		7 8 12	14		gray, very stiff	PP		2,250			
107												
108												
109												
110												
111	PB			95 psi	CL	medium stiff LL = 29, PI = 9, see Figure C-1 Triaxial Test, see Figure C-12 Consolidation Test, see Figure C-6	TxUU	7,600	960		21.1 22.6	107 101
112												
113	S&H		8 14 15	20		very stiff						
114							PP		2,000			
115												
116												
117												
118												
119												
120												

LANGAN

Project No.: 770672701

Figure: A-2d

TEST GEOTECH LOG 770672701 GEOTECH BORING LOGS.GPJ TEMPLATE_CA-MODIFIED.GDT 2/17/21

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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
121	S&H		11 14 15	20	CL	SANDY CLAY (CL) (continued) gray-brown to yellow-brown with orange mottling, stiff	PP		1,750			
122												
123												
124						grades with increase sand and gravel content						
125												
126												
127												
128					CL	CLAY (CL) gray, very stiff, wet, trace fine to medium sand and organics						
129												
130												
131	S&H		9 11 17	20		LL = 36, PI = 16, see Figure C-2 Triaxial Test, see Figure C-13	PP TxUU	8,800	2,500 2,220		23.0	106
132–139												
140						gray-brown, hard, trace sand and silt						
141	S&H		14 27 27	38			PP		4,250			
142–146												
147					SM	SILTY SAND (SM) yellow-brown, medium dense to dense, wet, fine- to medium-grained						
148												
149												
150												

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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
151	S&H		18 18 25	30	SM	SILTY SAND (SM) (continued) Hydrometer Test, see Figure C-17				40		
152–155												
156–167					SP-SC	SAND with CLAY (SP-SC) gray-brown, very dense, wet, fine- to coarse-grained, trace fine to coarse gravel and silt						
161	SPT		12 40 50	108								
168–178					CL	CLAY (CL) gray, very stiff to hard, wet, trace coarse subangular gravel, fine to coarse sand and silt						
171	S&H		14 21 22	30		LL = 49, PI = 30, see Figure C-2 Consolidation Test, see Figure C-7	PP		4,250		25.2	98
176						grades with increase sand content						
178–180					CL	CLAY with SAND (CL) gray-brown to yellow-brown, hard, wet, fine to coarse sand, trace silt						

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PROJECT:	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET San Jose, California	Log of Boring LB-2 PAGE 7 OF 7

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	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
181	SPT		7 12 22	41	CL	CLAY with SAND (CL) (continued)						
182												
183												
184												
185												
186												
187												
188												
189												
190												
191												
192												
193												
194												
195												
196												
197												
198												
199												
200												
201												
202												
203												
204												
205												
206												
207												
208												
209												
210												

Boring terminated at a depth of about 181.5 feet below ground surface (bgs).
Boring backfilled with cement grout per Santa Clara Valley Water District (SCVWD) requirements.
Groundwater encountered at about 19 feet bgs on 11/11/2020 at 6:15AM.
PP = Pocket penetrometer.

[1] S&H and SPT blow counts were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on a topographic survey titled, "Existing Conditions Plan, Sheet C1.01" by Kier & Wright, dated 10/29/2020 and reference the North American Vertical Datum of 1988 (NAVD88).

LANGAN

Project No.: 770672701 | Figure: A-2g

UNIFIED SOIL CLASSIFICATION SYSTEM

Majo r Divisions		Symbols	Typica l 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 0.075
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 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. 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

Analytical laboratory sample

Sample taken with Direct Push or Drive sampler

Sonic

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

APPENDIX B

CONE PENETRATION TEST RESULTS BY LANGAN (2020)

GREGG DRILLING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

November 11, 2020

Langan
Attn: Charlie Atekin

Subject: CPT Site Investigation
Fountain Alley
San Jose, California
GREGG Project Number: D2209197

Dear Mr. Atekin:

The following report presents the results of GREGG Drilling Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☒
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☐
7	Vapor Sampling	(VS)	☐
8	Pressuremeter Testing	(PMT)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact me at 949-903-6873.

Sincerely,
Gregg Drilling, LLC.

CPT Reports Team
Gregg Drilling, LLC.

2726 Walnut Ave. • Signal Hill, California 90755 • (562) 427-6899 • FAX (562) 427-3314
950 Howe Road. • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
SCPT-01	11/9/2020	200.62	-	-	32.5, 114.7, 174.4
CPT-02	11/9/2020	200.62	-	-	29.2, 45.8, 113.7, 188.0
SCPT-03	11/10/2020	195.05	-	-	73.0, 93.0, 187.2

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Campanella, R.G. and I. Weemees, “Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies”, Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

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Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

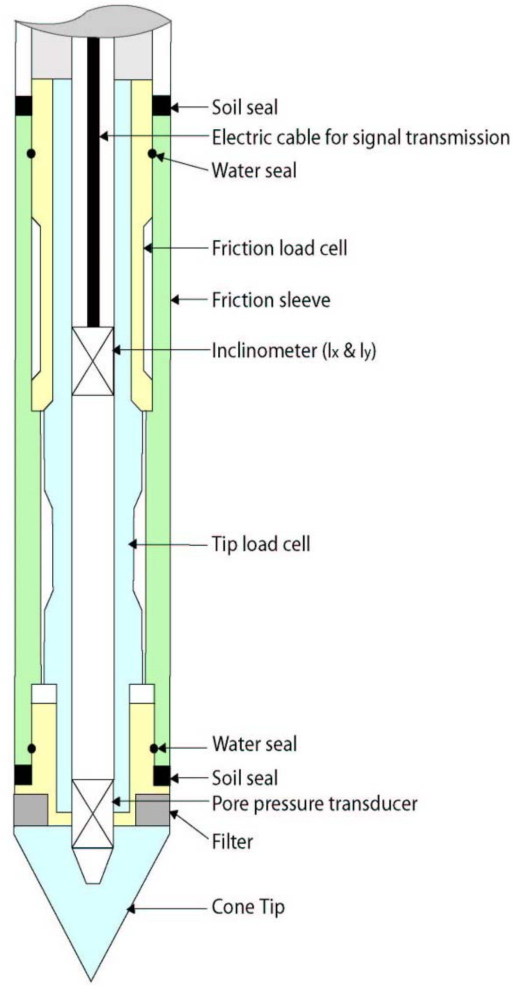


Figure CPT

Gregg 15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

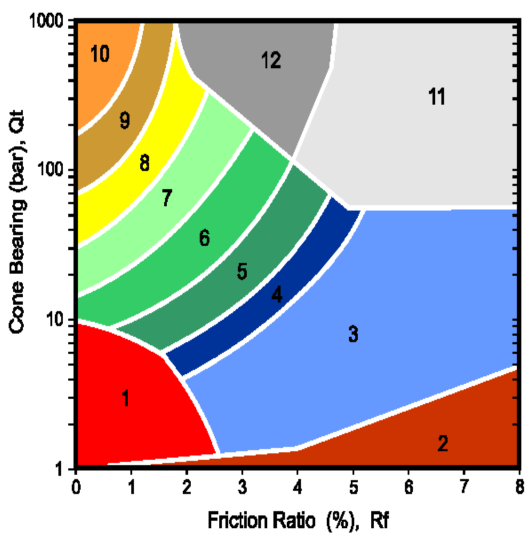
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

*over consolidated or cemented

Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1 Units for display (Imperial or metric) (atm. pressure, p_a = 0.96 tsf or 0.1 MPa)
2 Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3 Elevation of ground surface (ft or m)
4 Depth to water table, z_w (ft or m) – input required
5 Net area ratio for cone, a (default to 0.80)
6 Relative Density constant, C_{Dr} (default to 350)
7 Young's modulus number for sands, α (default to 5)
8 Small strain shear modulus number
    a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
    b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
11 Unit weight of water, (default to γ_w = 62.4 lb/ft^3 or 9.81 kN/m^3)

Column

1 Depth, z, (m) – CPT data is collected in meters
2 Depth (ft)
3 Cone resistance, q_c (tsf or MPa)
4 Sleeve resistance, f_s (tsf or MPa)
5 Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6 Other – any additional data
7 Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u\,(1-a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \sigma\ z$
12	In-situ pore pressure, u_o (tsf)	$u_o = \gamma_w (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u – u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 – \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa)\ (pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, n = 0.5 (clean sand)
When $I_c > 3.30$, n = 1.0 (clays)
When $1.64 < I_c < 3.30$, $n = (I_c – 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} based on Normalized SBT_n (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60}\,C_N$,
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, ϕ', (degrees) $$\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$$

Only SBT_n 5, 6, 7 & 8 *Show'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go
a. $G_o = S_G (q_t\ \sigma'_{vo}\ pa)^{1/3}$ *For SBT_n 5, 6, 7*
b. $G_o = C_G\, q_t$ *For SBT_n 1, 2, 3& 4*
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr}\, Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

The following updated and simplified SBT descriptions have been used in the software:

SBT Zones		**SBT_n Zones**	
1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		

7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u*) with time is measured behind the tip of the cone and recorded.

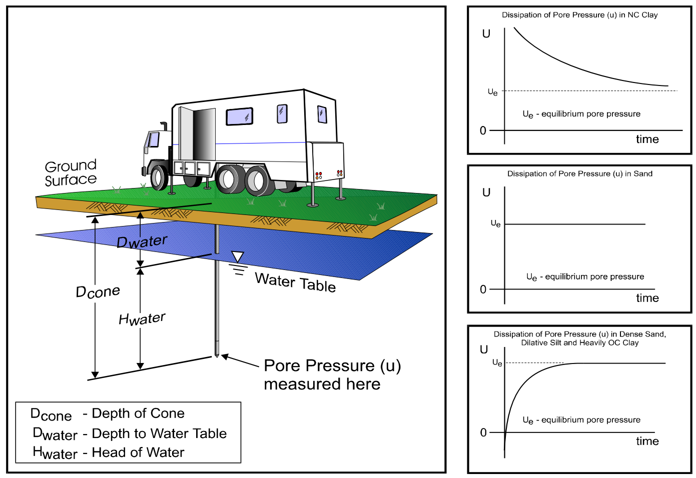
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: $v = \Delta d/\Delta t$

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.

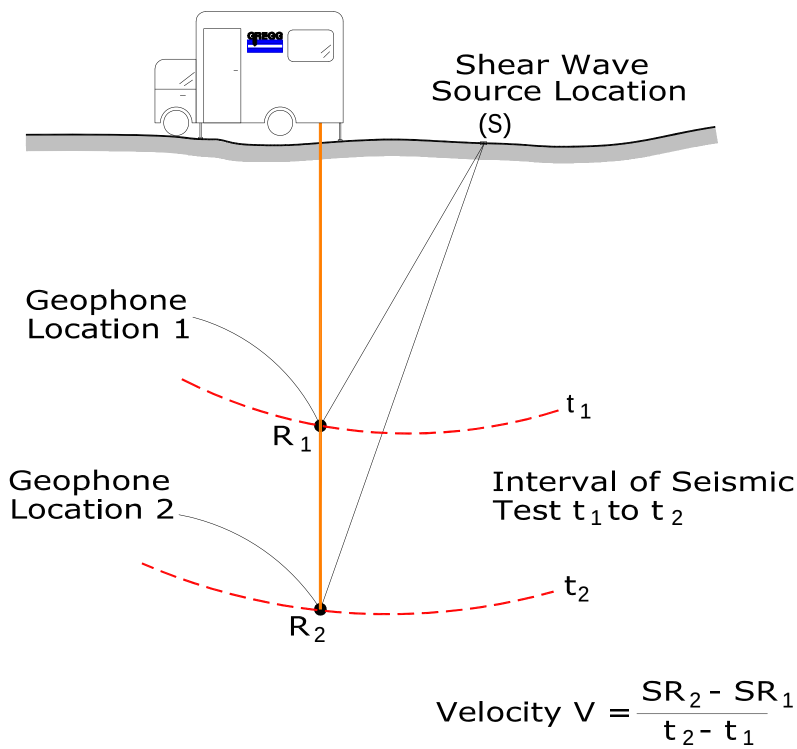


Figure SCPT

Groundwater Sampling

Gregg Drilling & Testing, Inc. conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

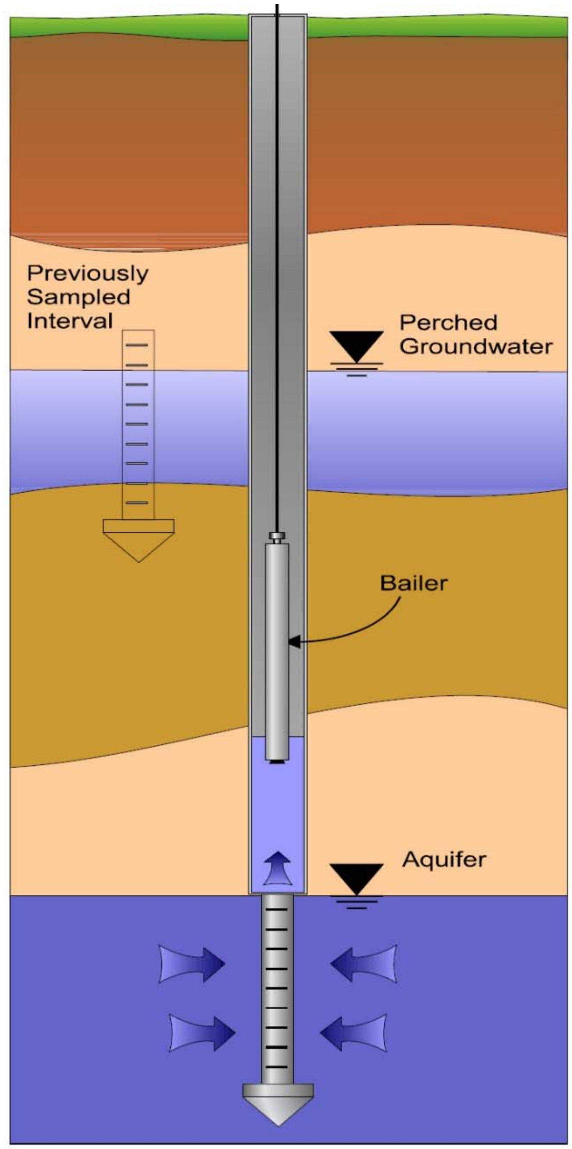


Figure GWS

Soil Sampling

Gregg Drilling & Testing, Inc. uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

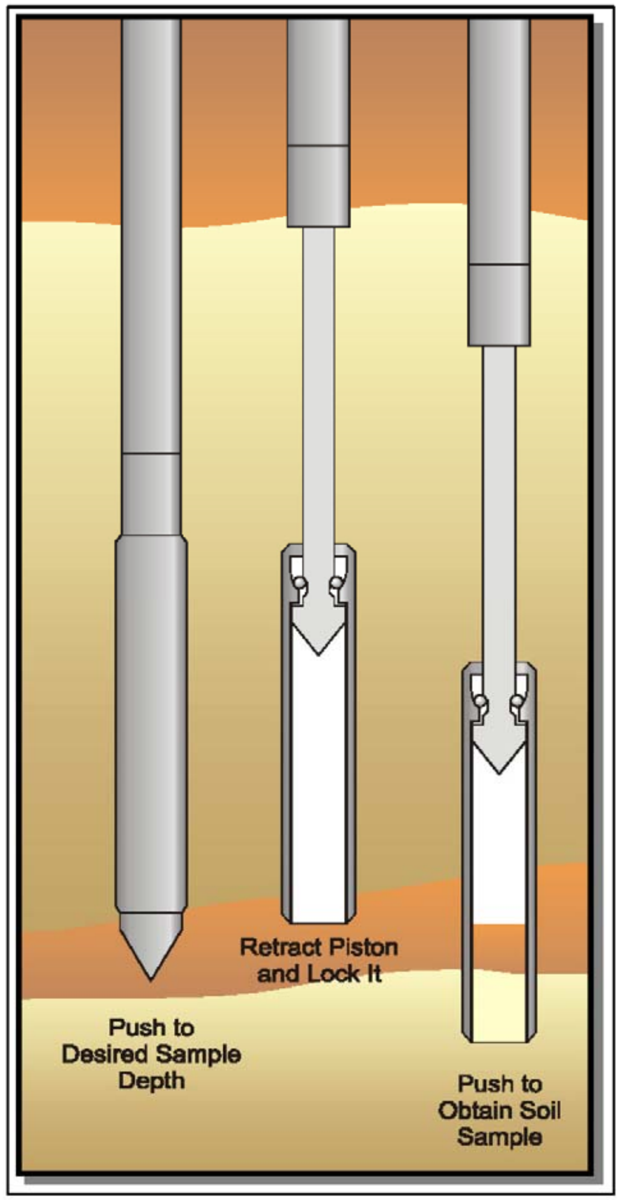


Figure SS

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

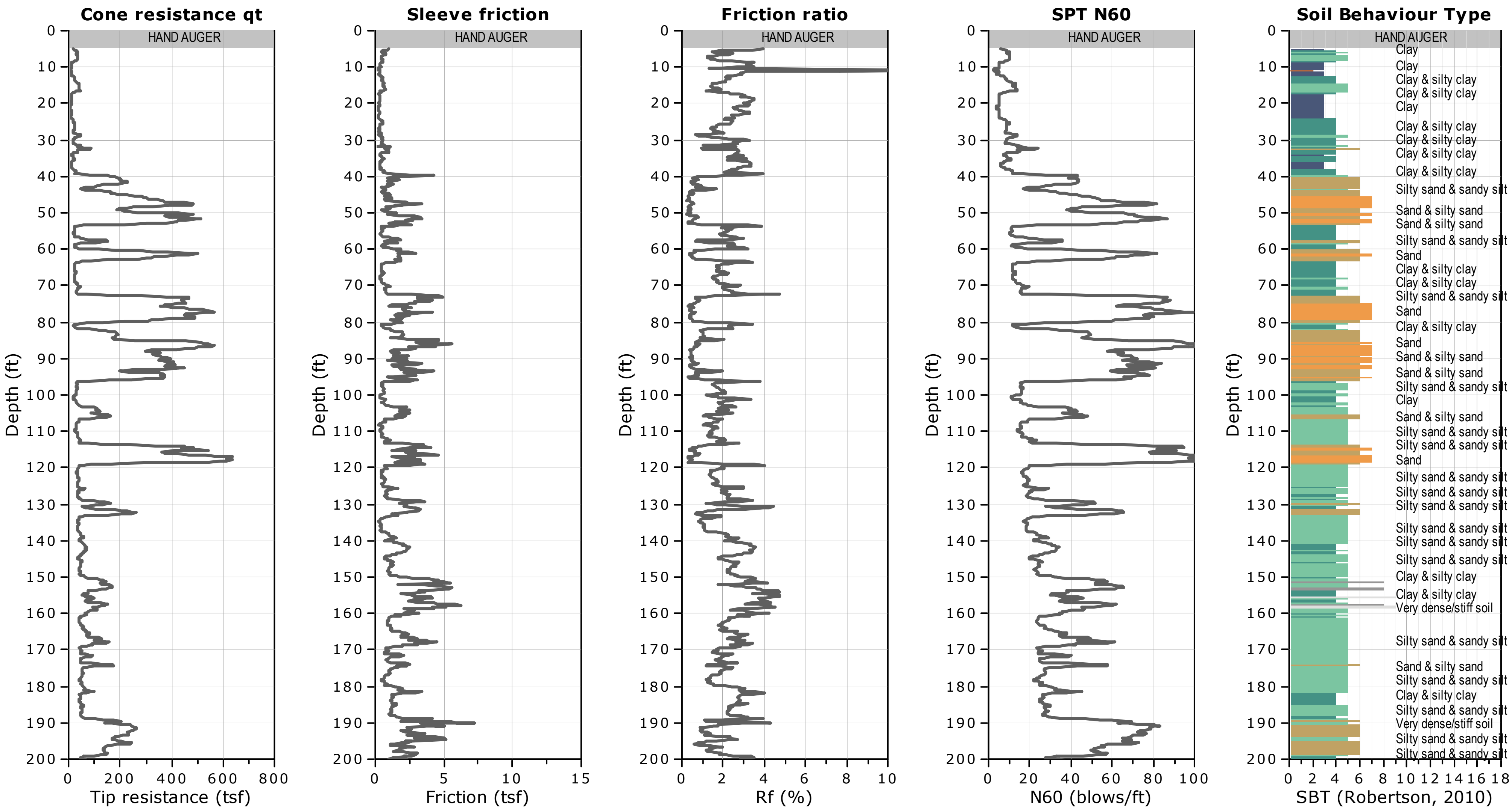
CPT: SCPT-01

CLIENT: LANGAN

SITE: FOUNTAIN ALLEY, SAN JOSE, CA

FIELD REP: CHARLIE AYTEKIN

Total depth: 200.62 ft, Date: 11/9/2020



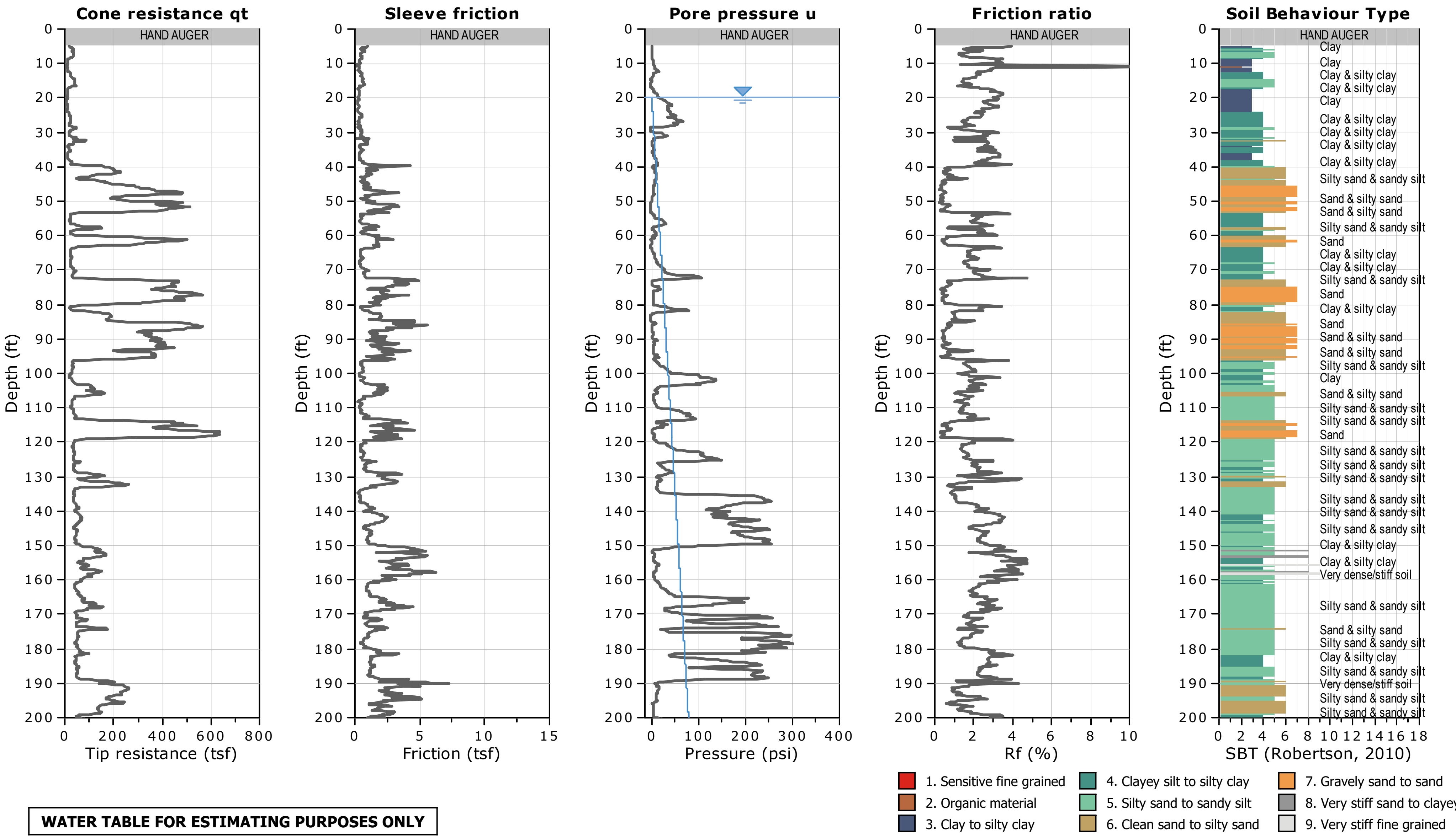
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-01

CLIENT: LANGAN
SITE: FOUNTAIN ALLEY, SAN JOSE, CA

FIELD REP: CHARLIE AYTEKIN
Total depth: 200.62 ft, Date: 11/9/2020



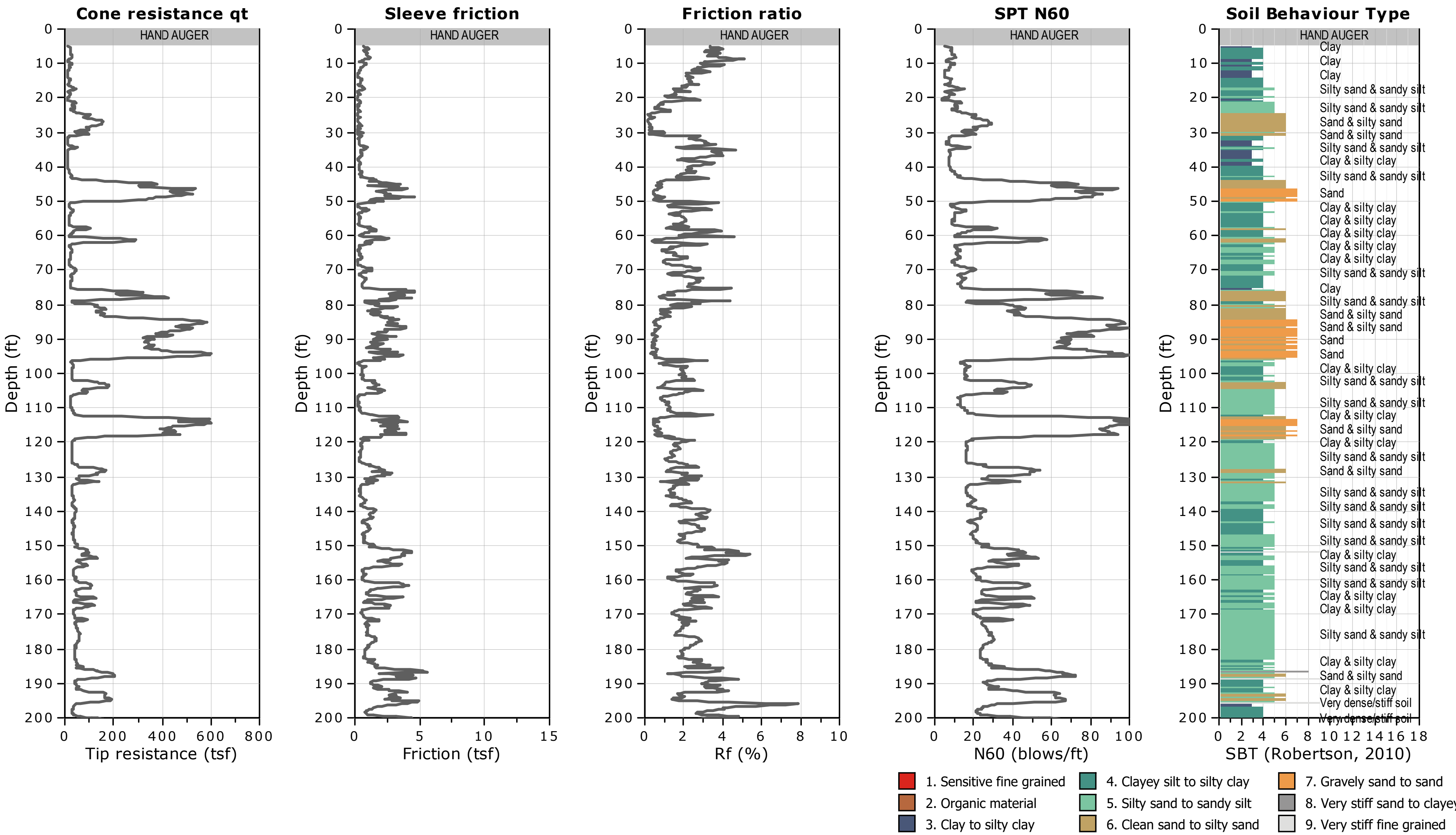
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-02

CLIENT: LANGAN
SITE: FOUNTAIN ALLEY, SAN JOSE, CA

FIELD REP: CHARLIE AYTEKIN
Total depth: 200.62 ft, Date: 11/9/2020



GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

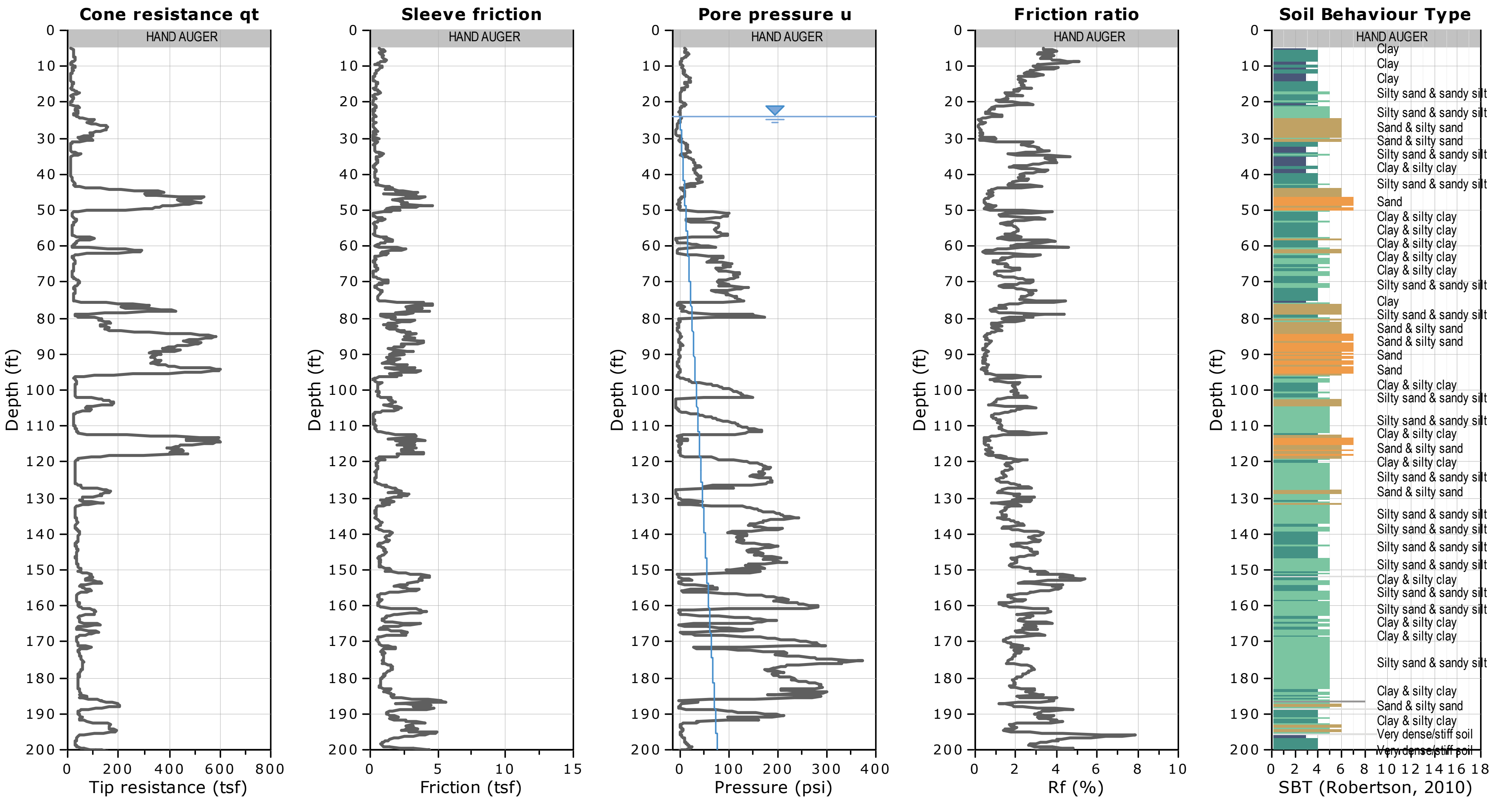
CPT: CPT-02

CLIENT: LANGAN

SITE: FOUNTAIN ALLEY, SAN JOSE, CA

FIELD REP: CHARLIE AYTEKIN

Total depth: 200.62 ft, Date: 11/9/2020



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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

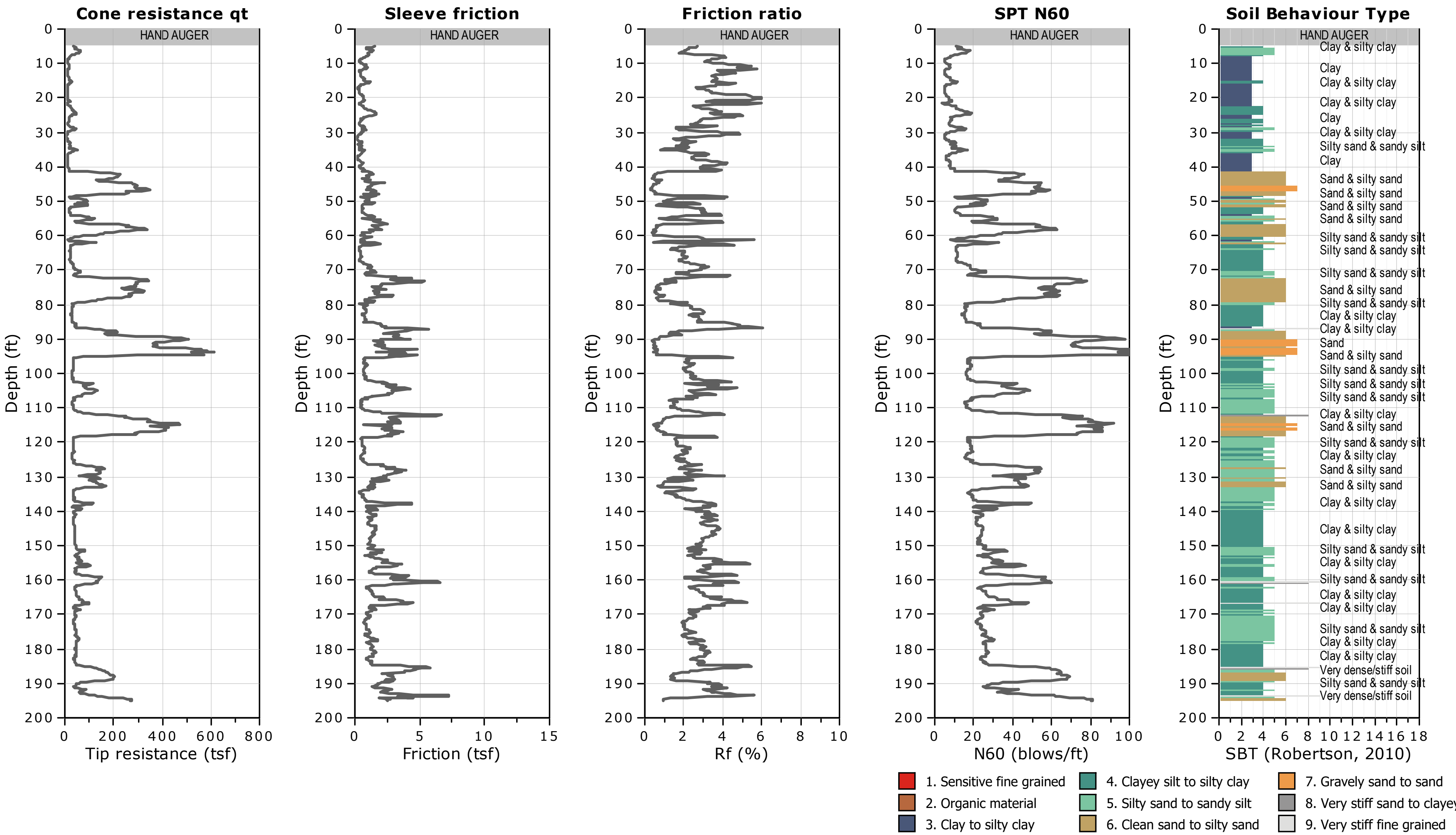
CPT: SCPT-03

CLIENT: LANGAN

SITE: FOUNTAIN ALLEY, SAN JOSE, CA

FIELD REP: CHARLIE AYTEKIN

Total depth: 195.05 ft, Date: 11/10/2020



GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

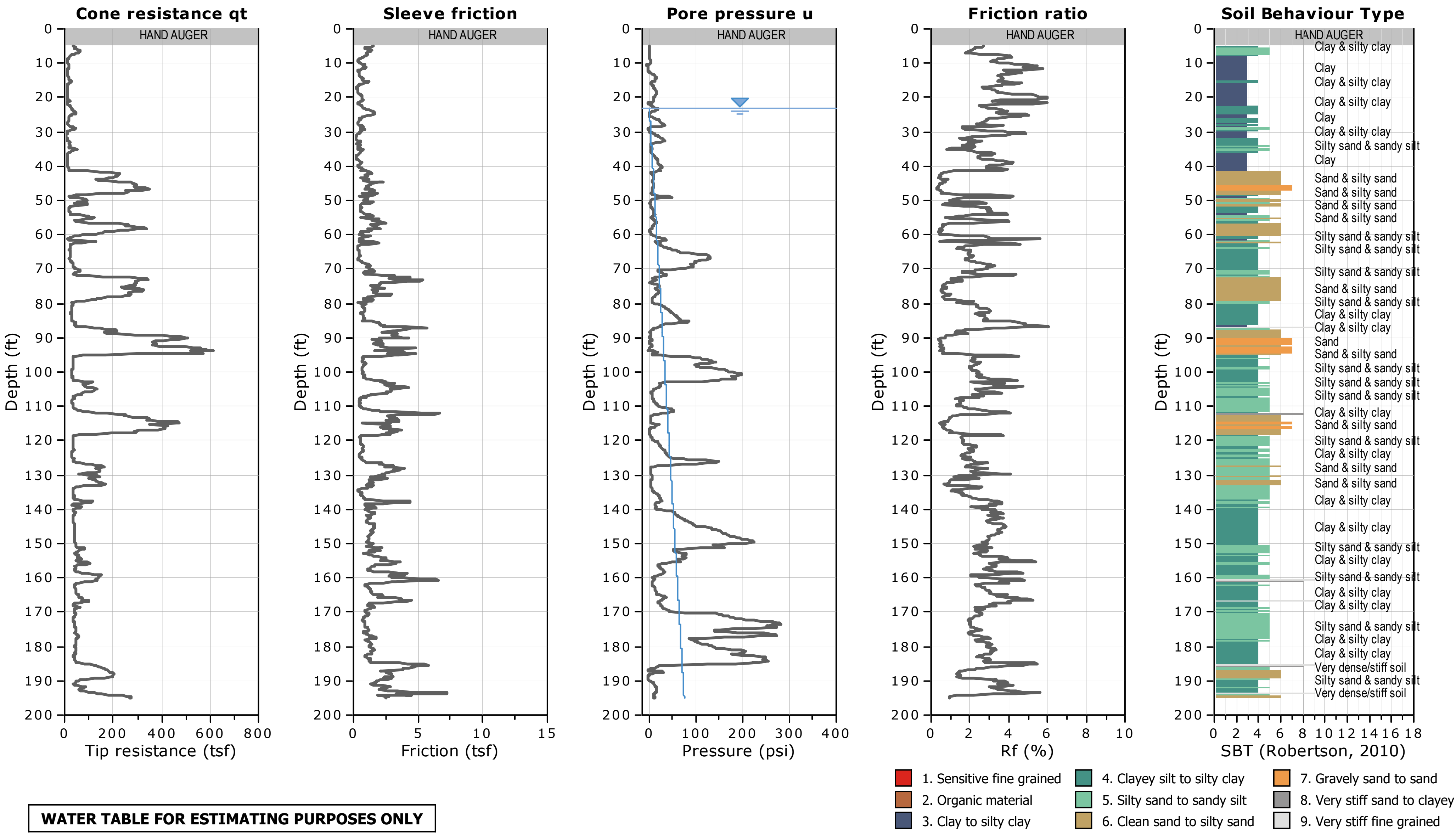
CPT: SCPT-03

CLIENT: LANGAN

SITE: FOUNTAIN ALLEY, SAN JOSE, CA

FIELD REP: CHARLIE AYTEKIN

Total depth: 195.05 ft, Date: 11/10/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
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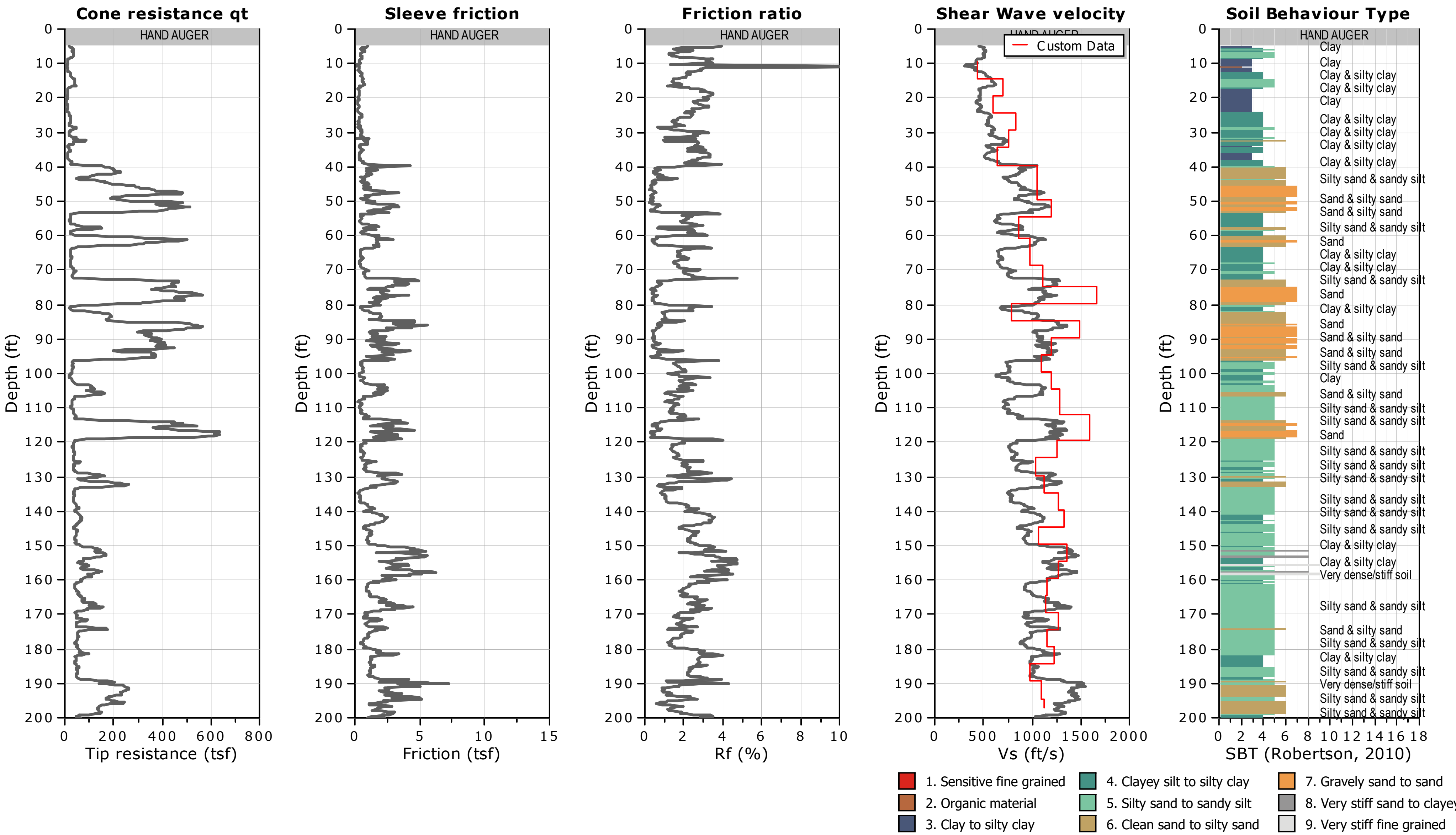
CPT: SCPT-01

CLIENT: LANGAN

SITE: FOUNTAIN ALLEY, SAN JOSE, CA

FIELD REP: CHARLIE AYTEKIN

Total depth: 200.62 ft, Date: 11/9/2020



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CPT: SCPT-03

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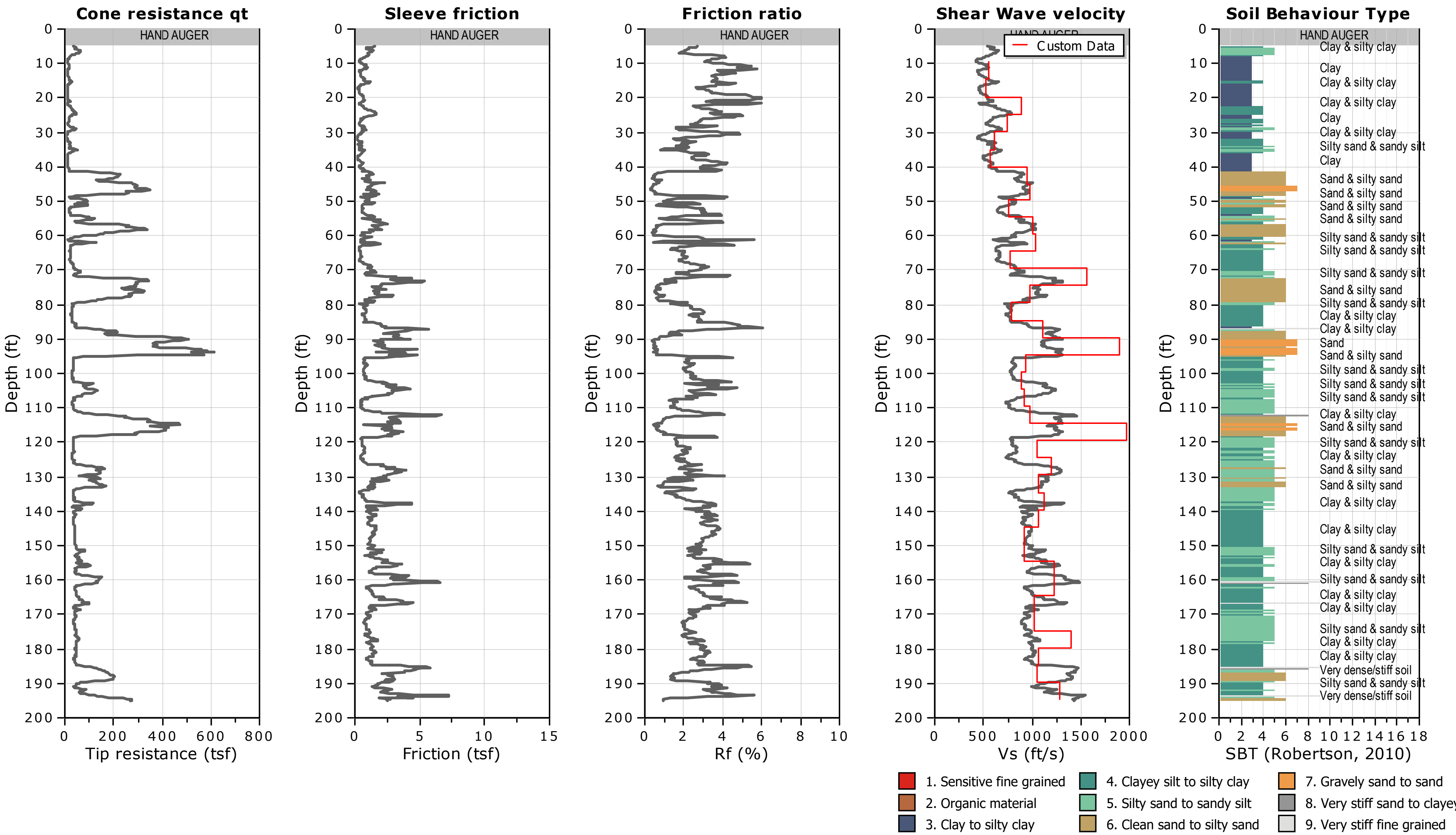
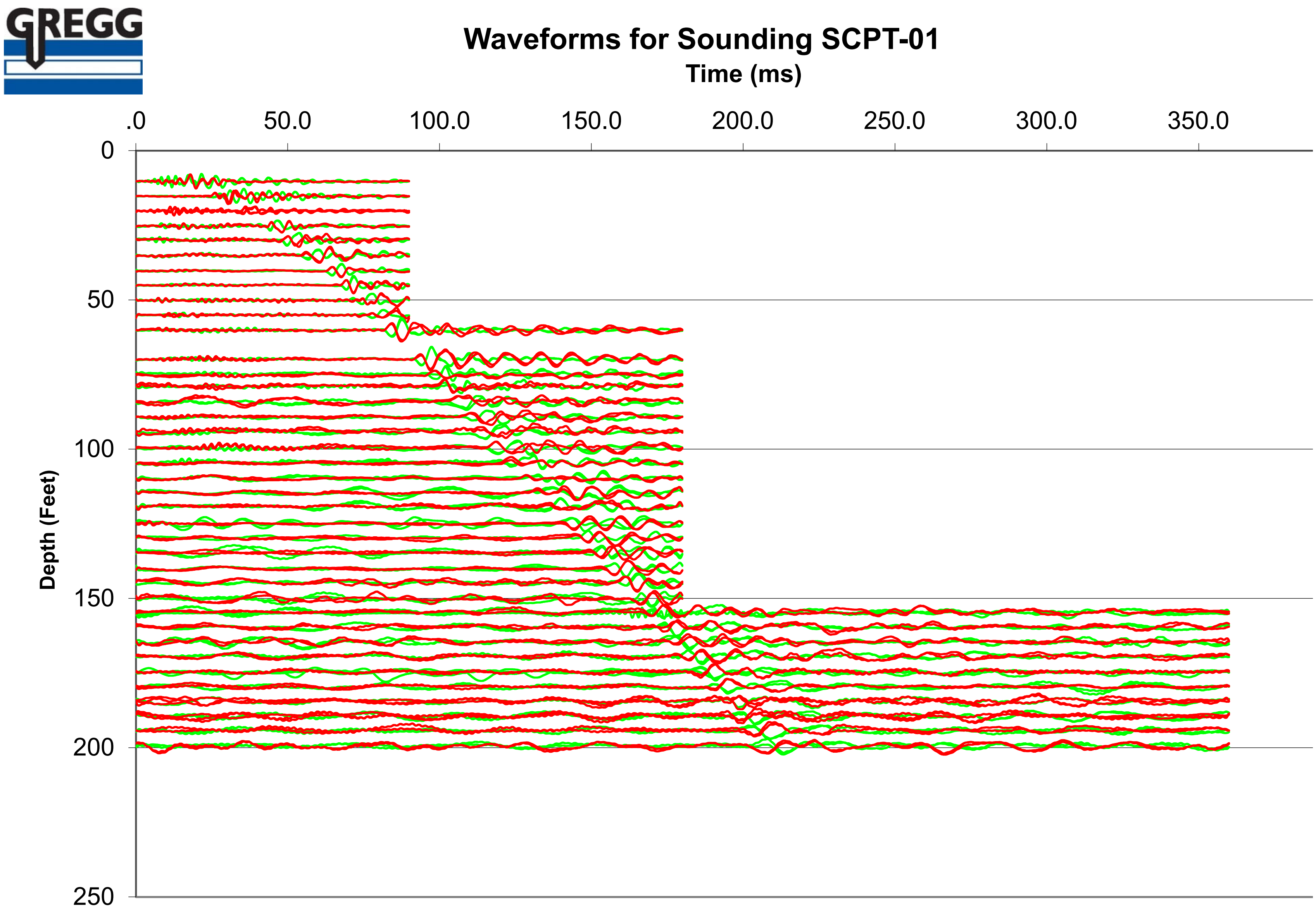
Total depth: 195.05 ft, Date: 11/10/2020




GREGG
Waveforms for Sounding SCPT-01
Time (ms)
.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
Depth (Feet)
0
50
100
150
200
250

Shear Wave Velocity Calculations

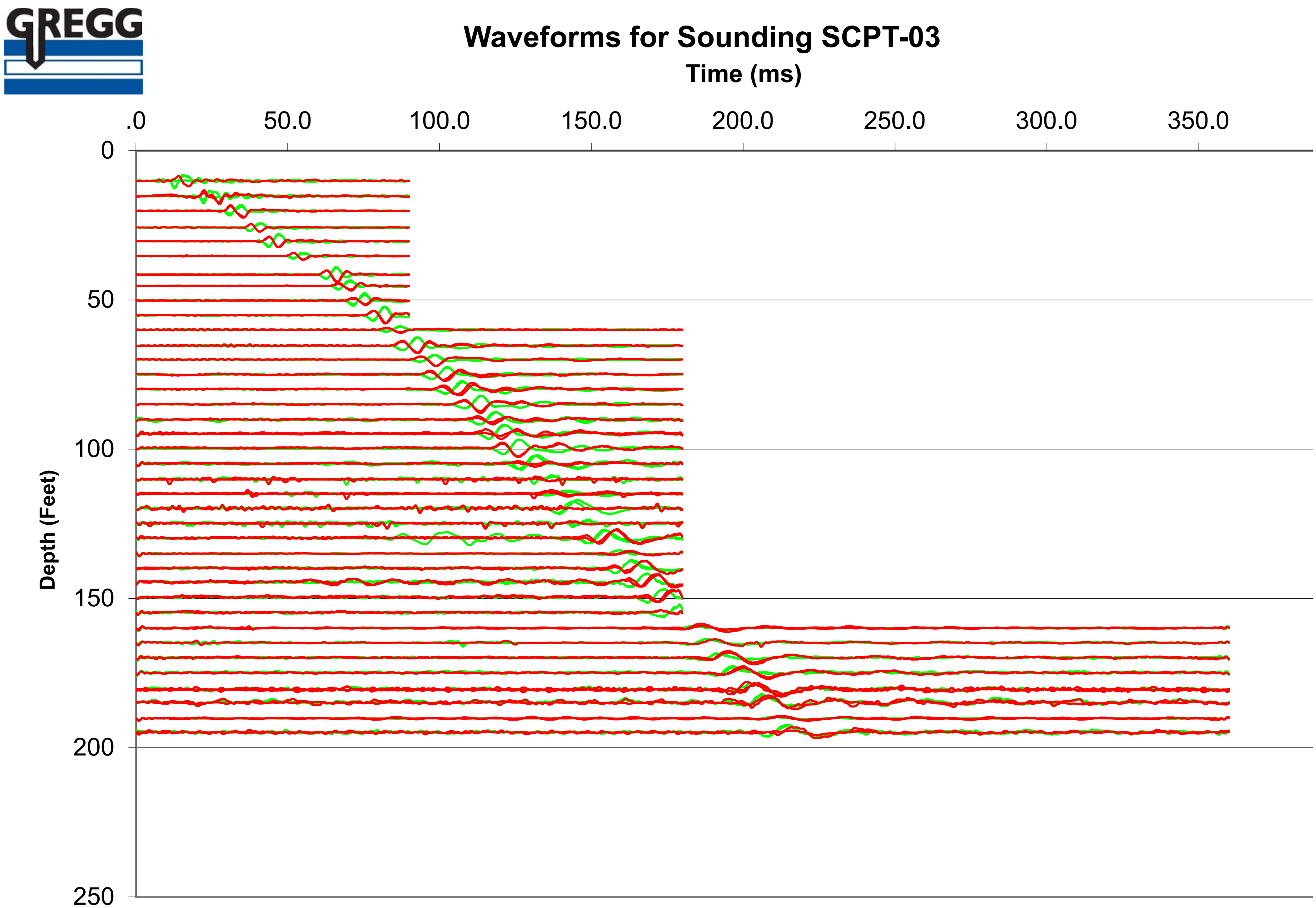
FOUNTAIN ALLEY

SCPT-01

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

11/09/20

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.33	9.67	9.82	9.82	18.6000			
15.26	14.60	14.69	4.87	29.8000	11.2000	435.1	12.14
20.18	19.52	19.59	4.90	36.8000	7.0000	699.6	17.06
25.26	24.60	24.66	5.07	45.2500	8.4500	600.1	22.06
30.02	29.36	29.41	4.75	51.0000	5.7500	825.7	26.98
35.10	34.44	34.49	5.08	57.7500	6.7500	752.3	31.90
40.35	39.69	39.73	5.24	65.9500	8.2000	639.5	37.07
45.11	44.45	44.48	4.75	70.4500	4.5000	1056.3	42.07
50.36	49.70	49.73	5.25	75.4500	5.0000	1049.2	47.08
55.12	54.46	54.48	4.75	79.4000	3.9500	1203.7	52.08
60.53	59.87	59.89	5.41	85.6500	6.2500	865.8	57.16
70.05	69.39	69.41	9.51	95.3500	9.7000	980.5	64.63
75.62	74.96	74.98	5.58	100.3500	5.0000	1115.2	72.17
80.22	79.56	79.57	4.59	103.1000	2.7500	1669.9	77.26
85.30	84.64	84.66	5.08	109.6000	6.5000	782.2	82.10
90.06	89.40	89.41	4.76	112.8000	3.2000	1486.4	87.02
95.14	94.48	94.50	5.08	117.0500	4.2500	1196.3	91.94
100.07	99.41	99.42	4.92	121.5500	4.5000	1093.4	96.94
105.15	104.49	104.50	5.08	125.8000	4.2500	1196.4	101.95
110.24	109.58	109.59	5.08	129.7500	3.9500	1287.3	107.03
115.32	114.66	114.67	5.08	139.0000	9.2500		112.12
120.08	119.42	119.43	4.76	142.0000	3.0000	1585.6	117.04
125.33	124.67	124.68	5.25	146.2000	4.2000	1249.7	122.04
130.25	129.59	129.60	4.92	150.9500	4.7500	1036.0	127.13
135.01	134.35	134.36	4.76	155.2000	4.2500	1119.3	131.97
140.42	139.76	139.77	5.41	159.4500	4.2500	1273.6	137.05
145.01	144.35	144.36	4.59	162.9000	3.4500	1331.3	142.06
150.59	149.93	149.94	5.58	168.1500	5.2500	1062.3	147.14
155.02	154.36	154.37	4.43	171.4000	3.2500	1362.7	152.14
160.10	159.44	159.45	5.08	175.4000	4.0000	1271.2	156.90
165.03	164.37	164.37	4.92	179.6500	4.2500	1157.9	161.91
170.11	169.45	169.46	5.09	184.1000	4.4500	1142.7	166.91
175.20	174.54	174.54	5.09	188.1000	4.0000	1271.3	171.99
180.12	179.46	179.47	4.92	192.3500	4.2500	1157.9	177.00
185.04	184.38	184.39	4.92	196.3500	4.0000	1230.3	181.92
190.12	189.46	189.47	5.09	201.5500	5.2000	977.9	186.92
195.05	194.39	194.39	4.92	206.0500	4.5000	1093.6	191.92
200.13	199.47	199.48	5.09	210.5500	4.5000	1130.0	196.93



GREGG
Waveforms for Sounding SCPT-03
Time (ms)
.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
0
50
100
150
200
250
Depth (Feet)

GREGG

Shear Wave Velocity Calculations

FOUNTAIN ALLEY

SCPT-03

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

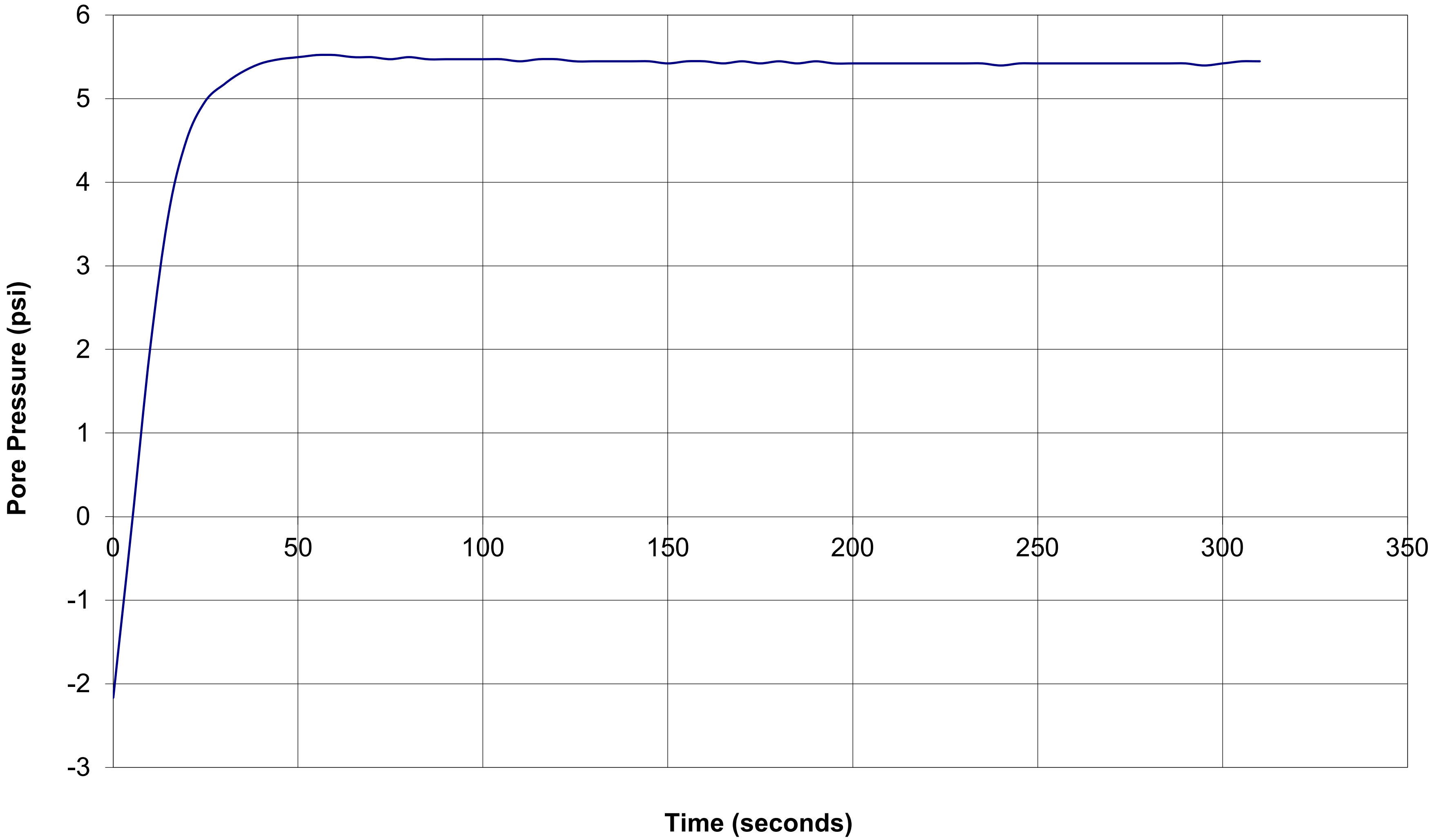
11/10/20

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.17	9.51	9.66	9.66	14.6000			
15.26	14.60	14.69	5.04	23.6000	9.0000	559.4	12.05
20.18	19.52	19.59	4.90	32.8000	9.2000	532.3	17.06
25.75	25.09	25.15	5.56	39.0500	6.2500	889.9	22.31
30.35	29.69	29.73	4.58	45.2500	6.2000	739.4	27.39
35.27	34.61	34.65	4.91	53.2500	8.0000	614.3	32.15
41.50	40.84	40.88	6.23	64.2000	10.9500	568.7	37.73
45.28	44.62	44.65	3.77	68.2000	4.0000	942.5	42.73
50.20	49.54	49.56	4.92	73.2000	5.0000	983.6	47.08
55.28	54.62	54.65	5.08	79.9000	6.7000	758.6	52.08
60.04	59.38	59.40	4.76	84.6500	4.7500	1001.1	57.00
65.45	64.79	64.81	5.41	89.9000	5.2500	1030.7	62.09
70.05	69.39	69.41	4.59	95.8500	5.9500	771.7	67.09
75.13	74.47	74.49	5.08	99.1000	3.2500	1564.3	71.93
80.05	79.39	79.41	4.92	104.1000	5.0000	984.0	76.93
85.14	84.48	84.49	5.08	110.6000	6.5000	782.2	81.93
90.39	89.73	89.74	5.25	115.3000	4.7000	1116.7	87.10
95.14	94.48	94.50	4.76	117.8000	2.5000	1902.6	92.11
100.07	99.41	99.42	4.92	123.0500	5.2500	937.2	96.94
105.15	104.49	104.50	5.08	128.7500	5.7000	892.0	101.95
110.24	109.58	109.59	5.08	134.2500	5.5000	924.5	107.03
115.16	114.50	114.51	4.92	139.2500	5.0000	984.1	112.04
120.08	119.42	119.43	4.92	141.7500	2.5000	1968.3	116.96
125.00	124.34	124.35	4.92	146.4500	4.7000	1047.0	121.88
130.08	129.42	129.44	5.08	150.7000	4.2500	1196.4	126.88
135.17	134.51	134.52	5.08	155.4500	4.7500	1070.5	131.97
140.26	139.60	139.61	5.08	159.9500	4.5000	1130.0	137.05
145.01	144.35	144.36	4.76	164.4000	4.4500	1069.0	141.97
150.10	149.44	149.45	5.08	169.9000	5.5000	924.5	146.90
155.18	154.52	154.53	5.08	175.4000	5.5000	924.5	151.98
160.10	159.44	159.45	4.92	186.1000	10.7000		156.98
165.03	164.37	164.37	4.92	190.1000	4.0000	1230.2	161.91
170.11	169.45	169.46	5.09	195.1000	5.0000	1017.0	166.91
175.20	174.54	174.54	5.09	200.0500	4.9500	1027.3	171.99
180.77	180.11	180.12	5.58	204.0500	4.0000	1394.3	177.33
185.04	184.38	184.39	4.26	208.0500	4.0000	1066.2	182.25
190.29	189.63	189.64	5.25	213.0500	5.0000	1049.8	187.00
195.05	194.39	194.39	4.76	216.7500	3.7000	1285.7	192.01

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

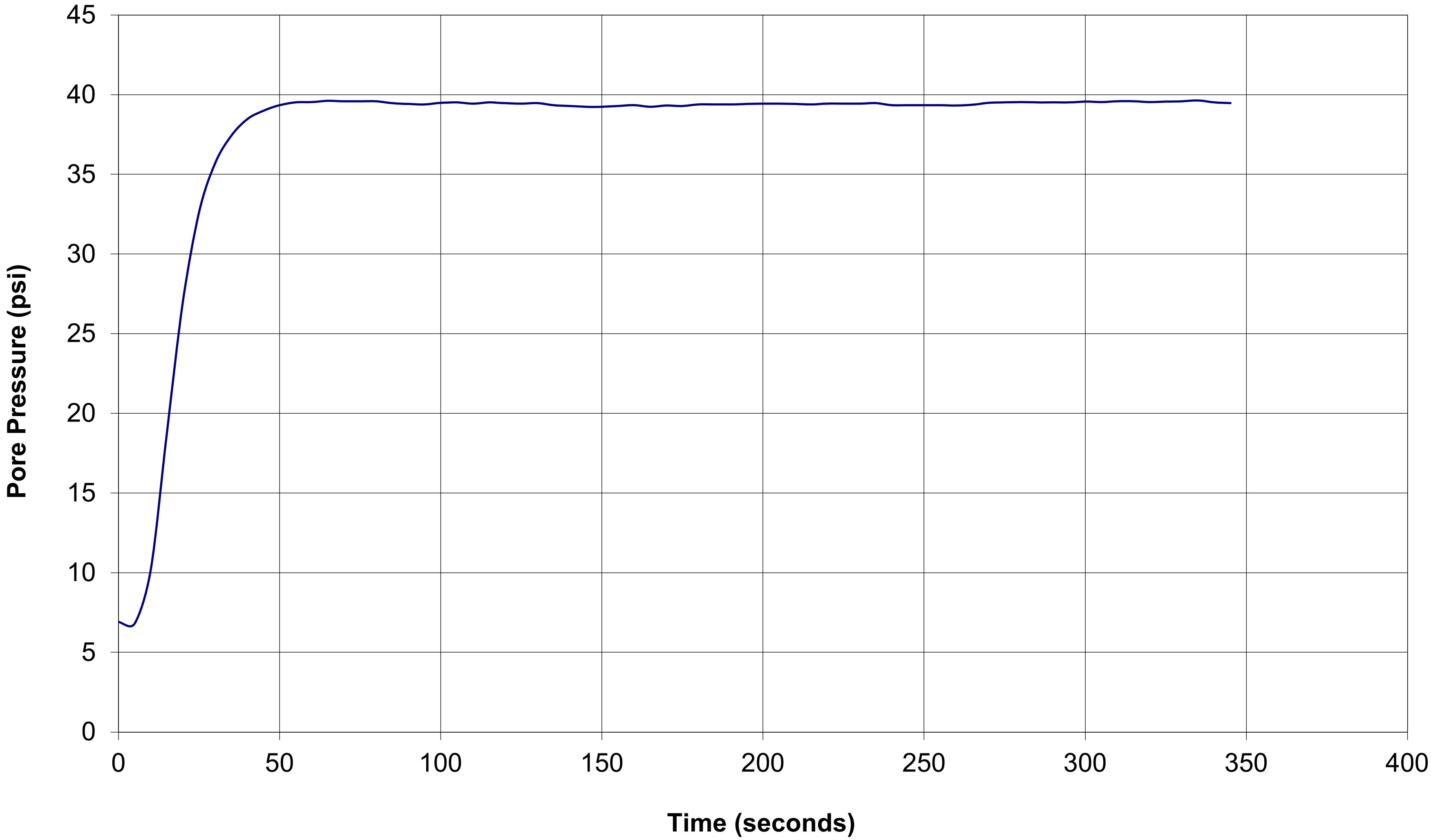
Sounding: SCPT-01
Depth (ft): 32.48
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: SCPT-01
Depth (ft): 114.67
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN

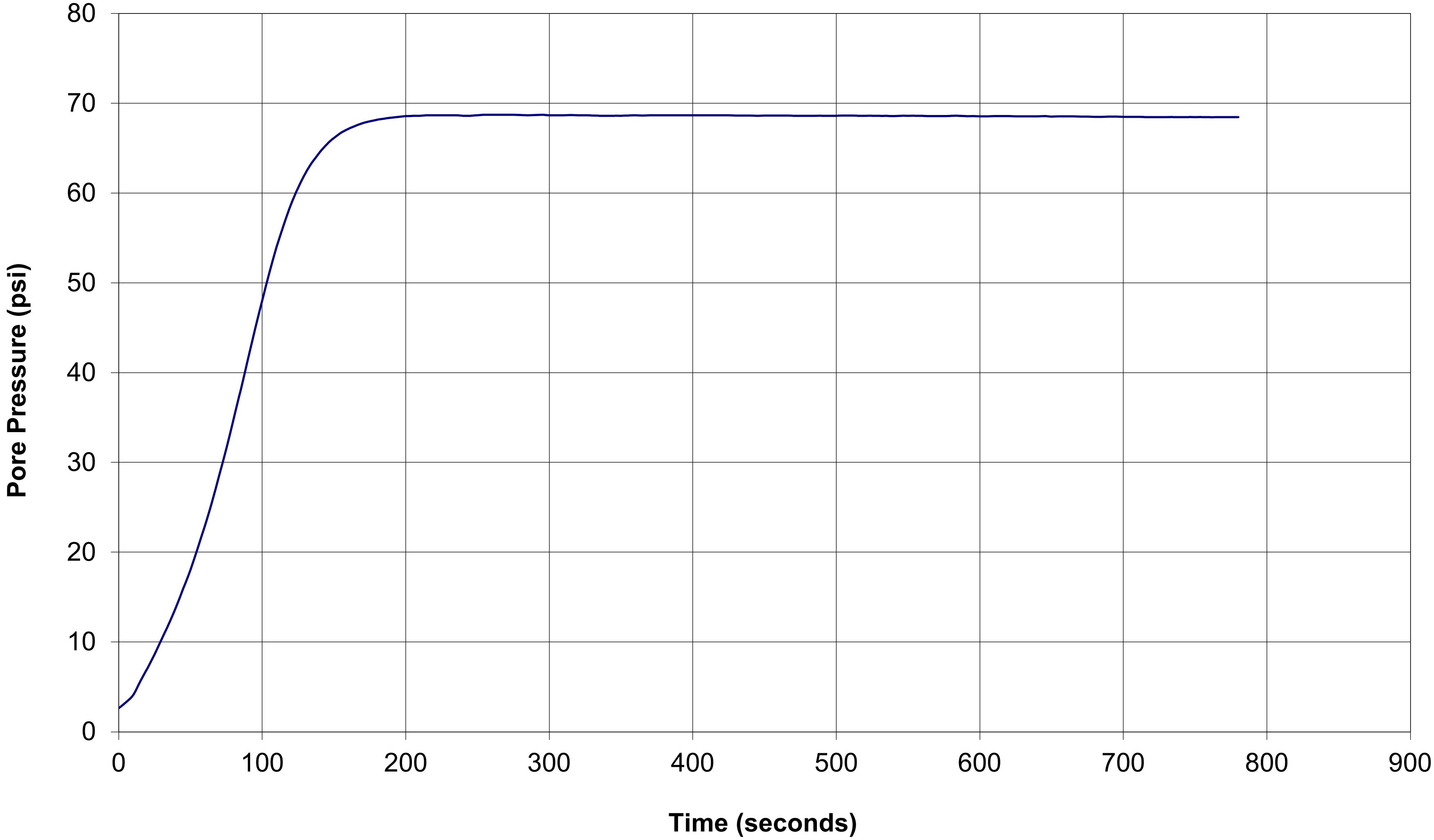


GREGG

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: SCPT-01
Depth (ft): 174.38
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN

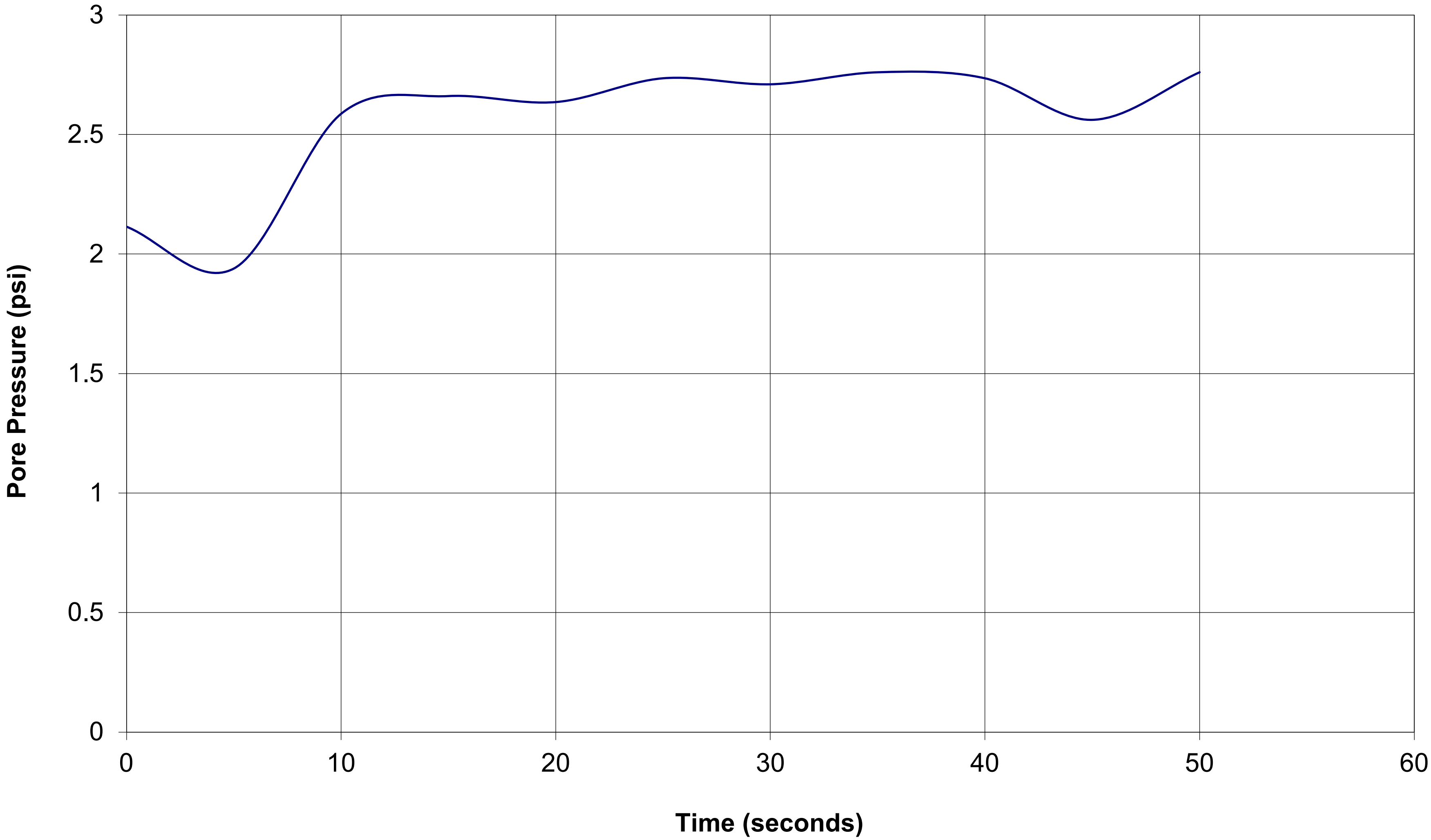


GREGG

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

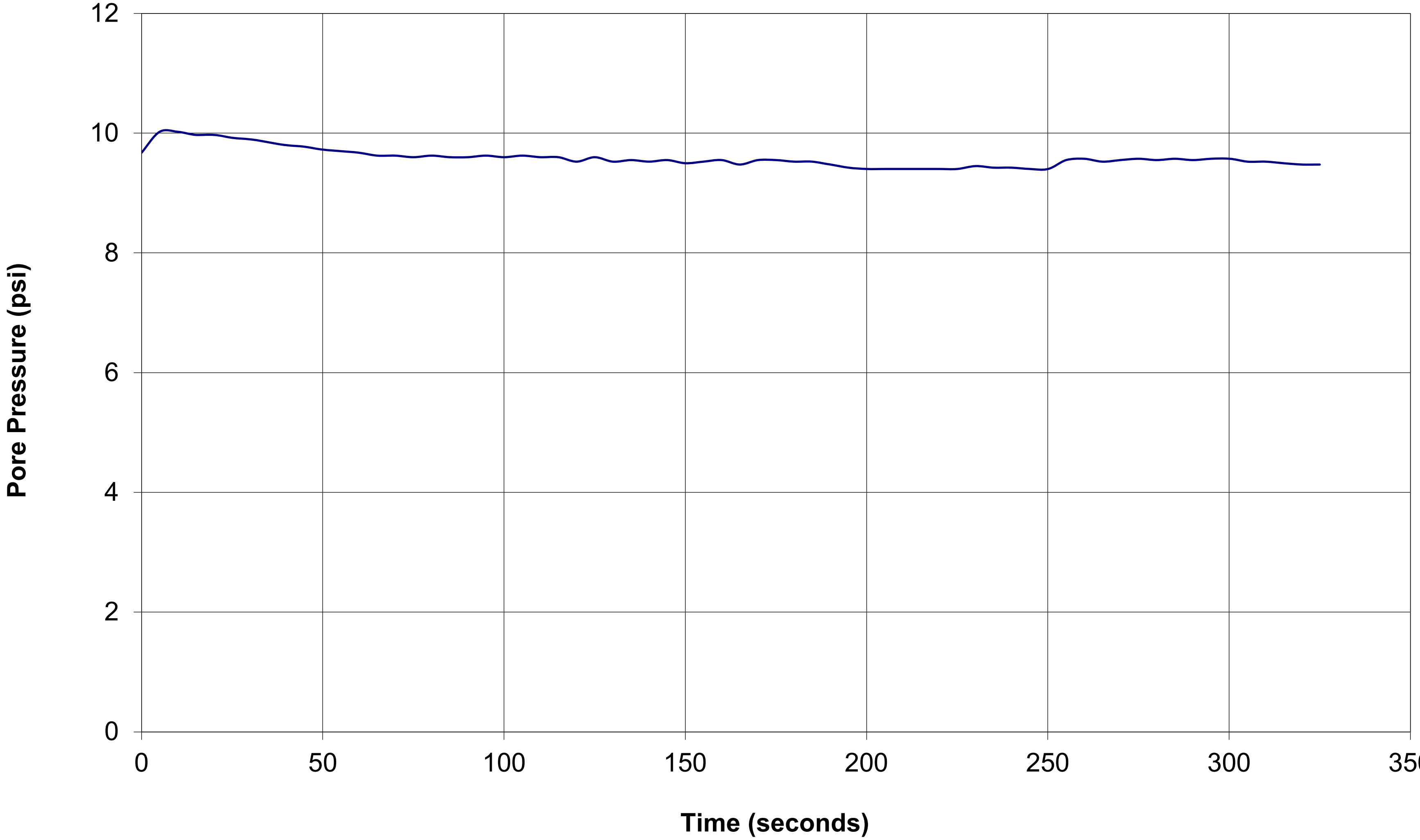
Sounding: SCPT-02
Depth (ft): 29.20
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: SCPT-02
Depth (ft): 45.77
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN

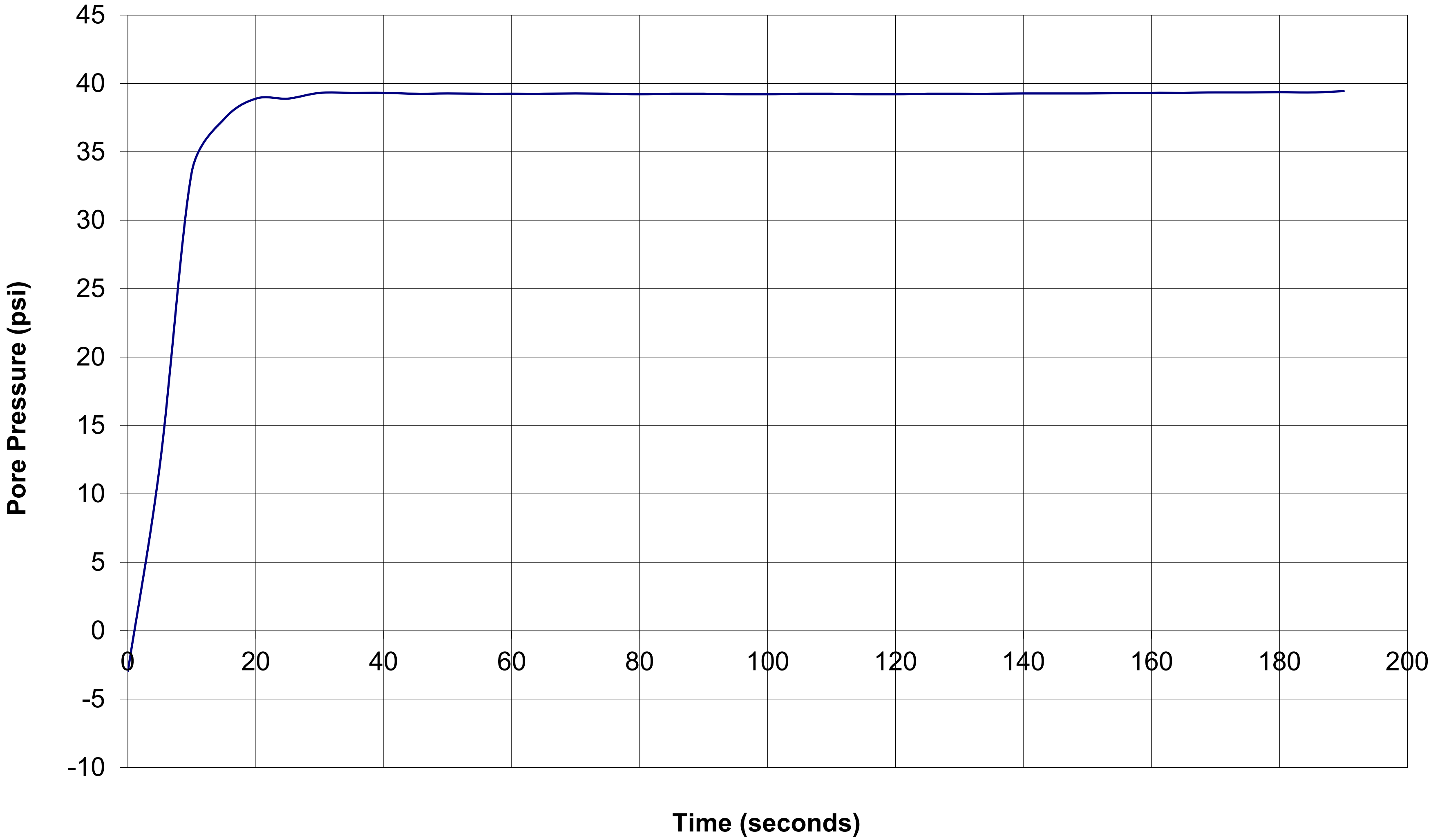


GREGG

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

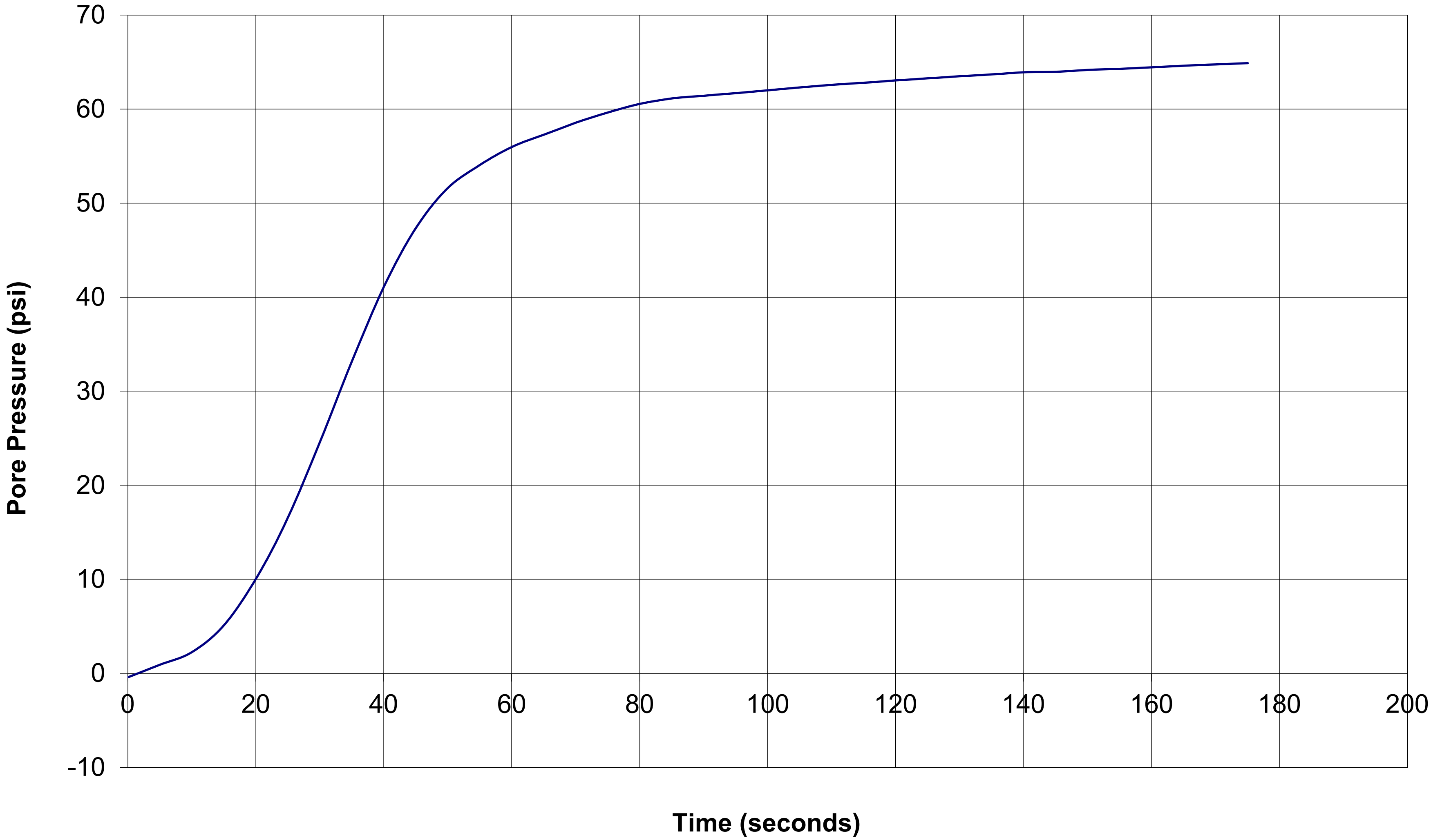
Sounding:	SCPT-02
Depth (ft):	113.68
Site:	FOUNTAIN ALLEY
Engineer:	CHARLIE AYTEKIN



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

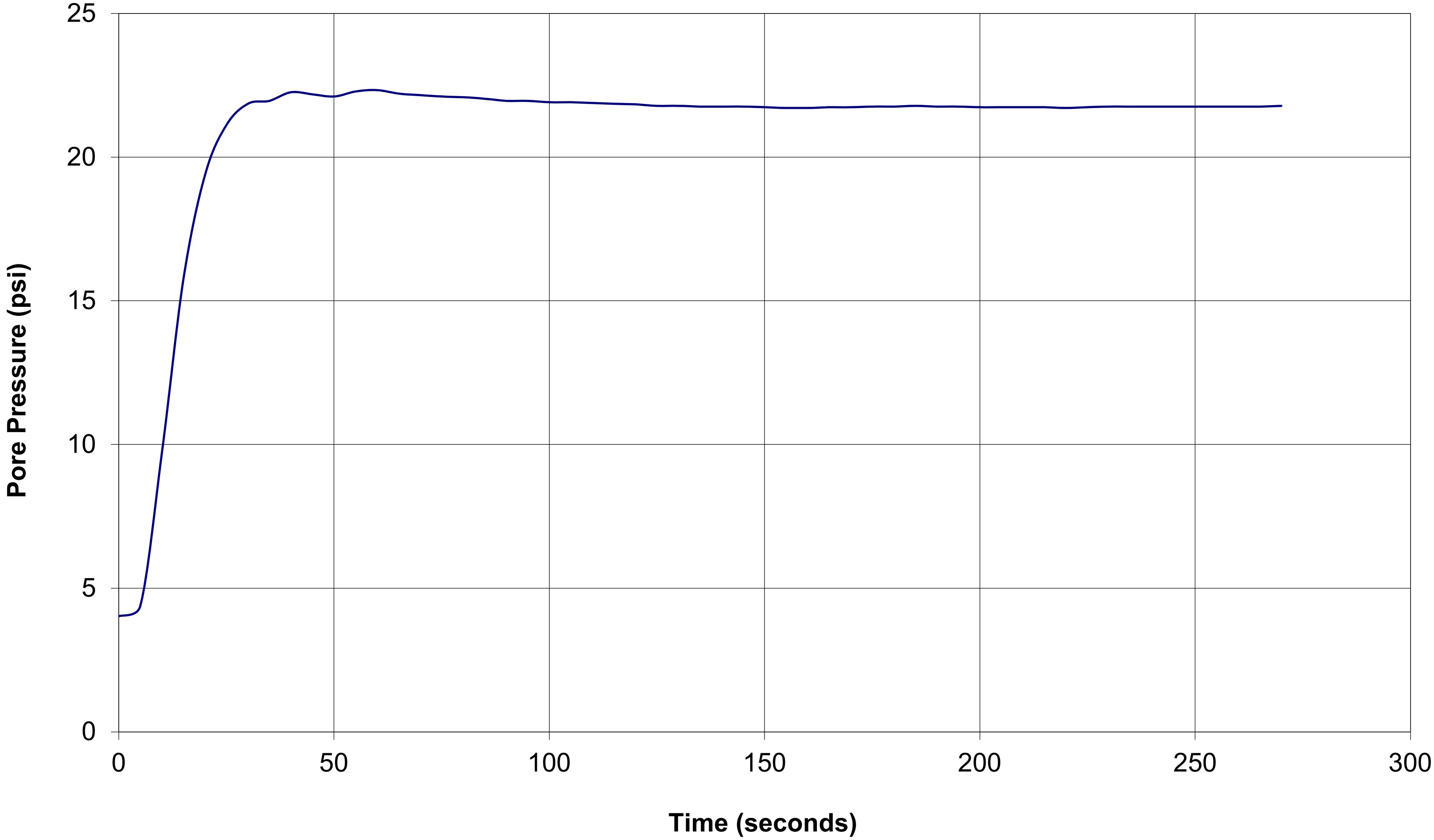
Sounding: SCPT-02
Depth (ft): 187.99
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

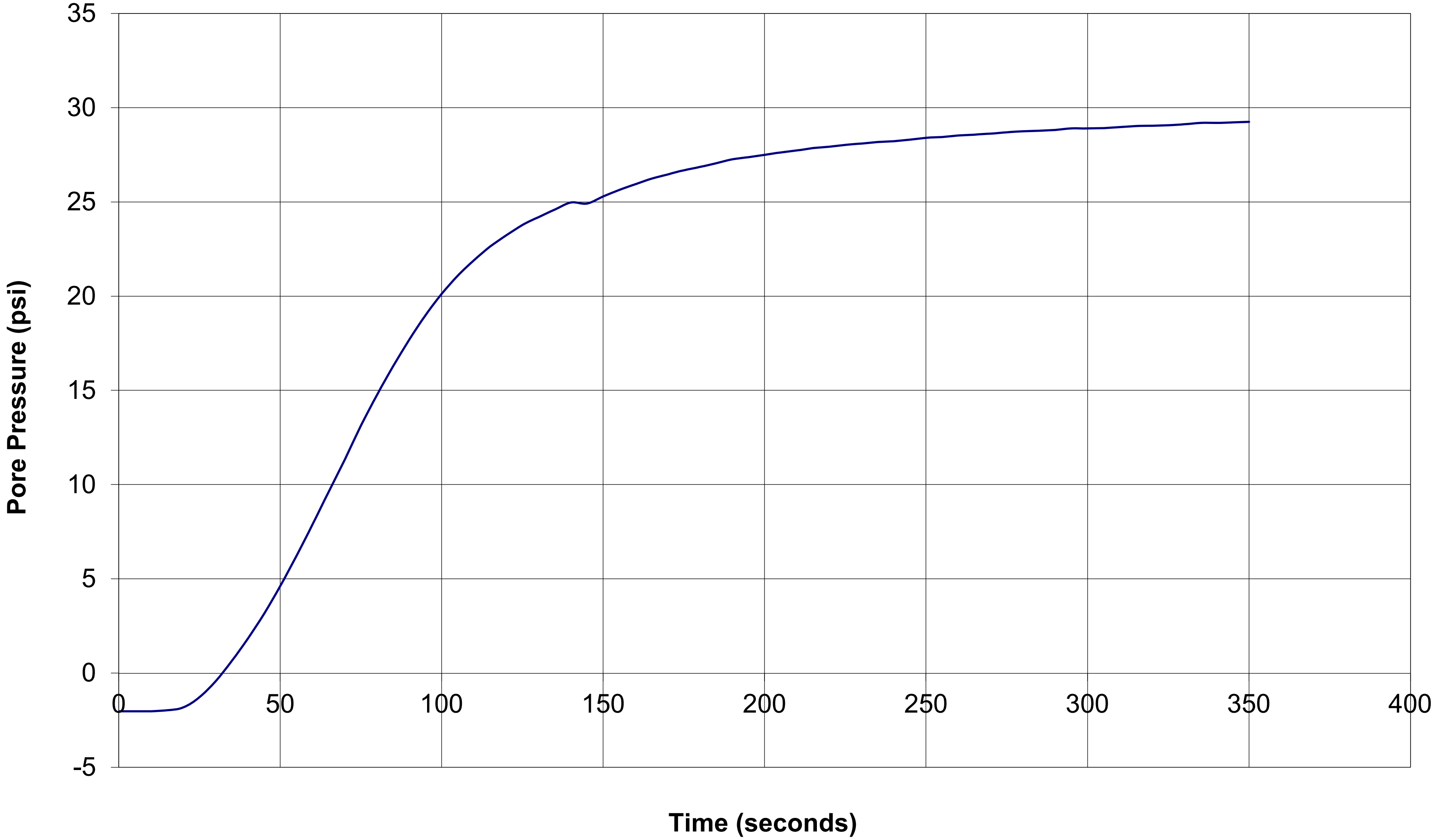
Sounding: SCPT-03
Depth (ft): 73.00
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

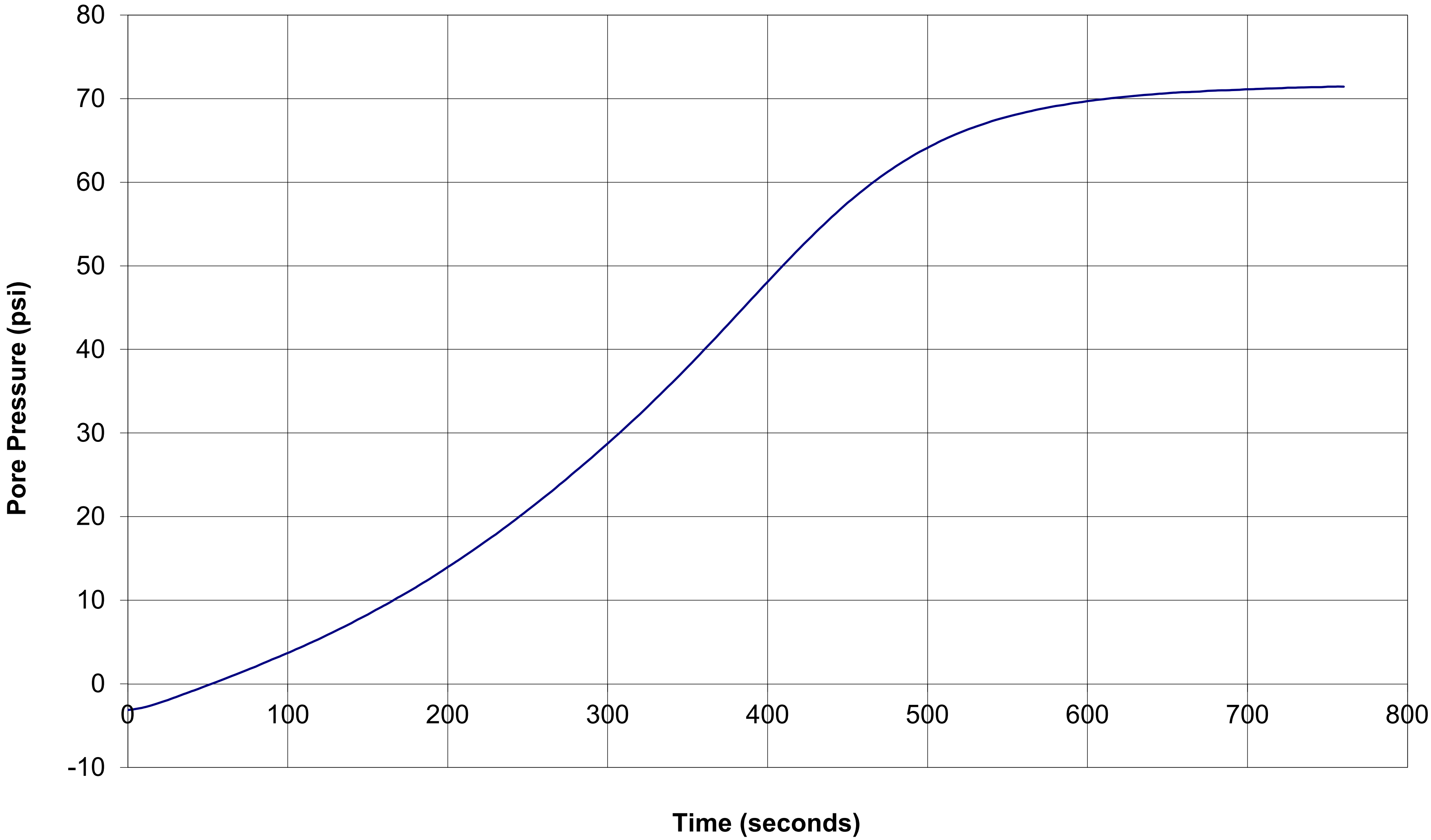
Sounding: SCPT-03
Depth (ft): 94.00
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN



GREGG DRILLING & TESTING

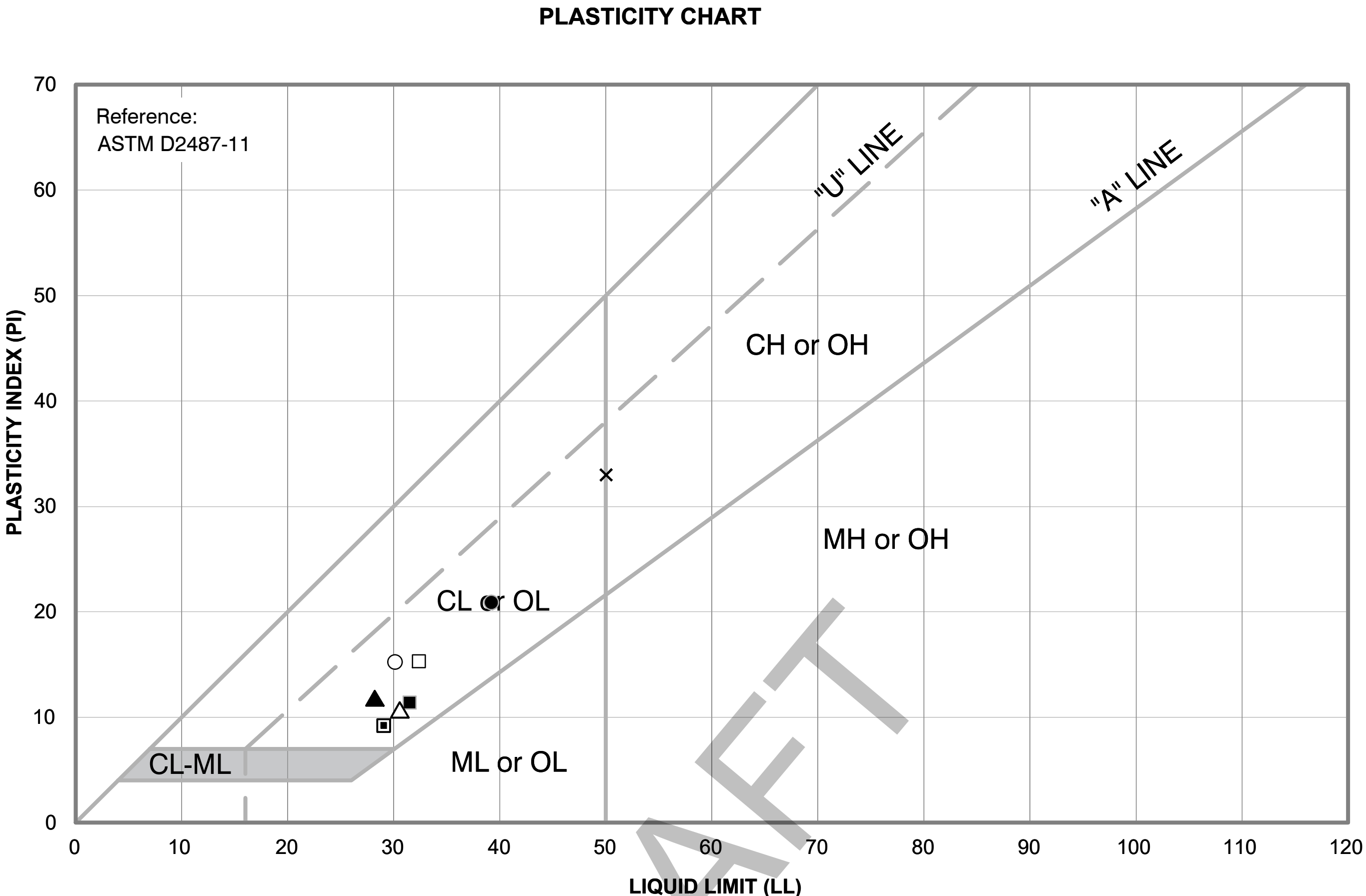
Pore Pressure Dissipation Test

Sounding: SCPT-03
Depth (ft): 187.17
Site: FOUNTAIN ALLEY
Engineer: CHARLIE AYTEKIN



APPENDIX C

LABORATORY DATA BY LANGAN (2020)



Symbol	Source	Description and Classification	Natrual M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	LB-1 at 2 feet	CLAY (CL), yellow-brown to brown	--	39	21	--
■	LB-1 at 31 feet	SANDY CLAY (CL), gray-brown with orange mottling	25.0	32	12	--
□	LB-1 at 110 feet	SANDY CLAY (CL), gray to gray-brown	15.3	33	15	--
○	LB-1 at 140 feet	CLAY with SAND (CL), gray	19.3	30	15	--
▲	LB-1 at 181 feet	CLAY (CL), yellow-brown with orange mottling	20.2	28	12	--
△	LB-2 at 16 feet	CLAY with SAND (CL), gray-brown with orange mottling	--	31	11	--
×	LB-2 at 96 feet	CLAY (CH), gray-brown with orange mottling	26.4	50	33	--
▣	LB-2 at 110 feet	SANDY CLAY (CL), gray	22.6	29	9	--

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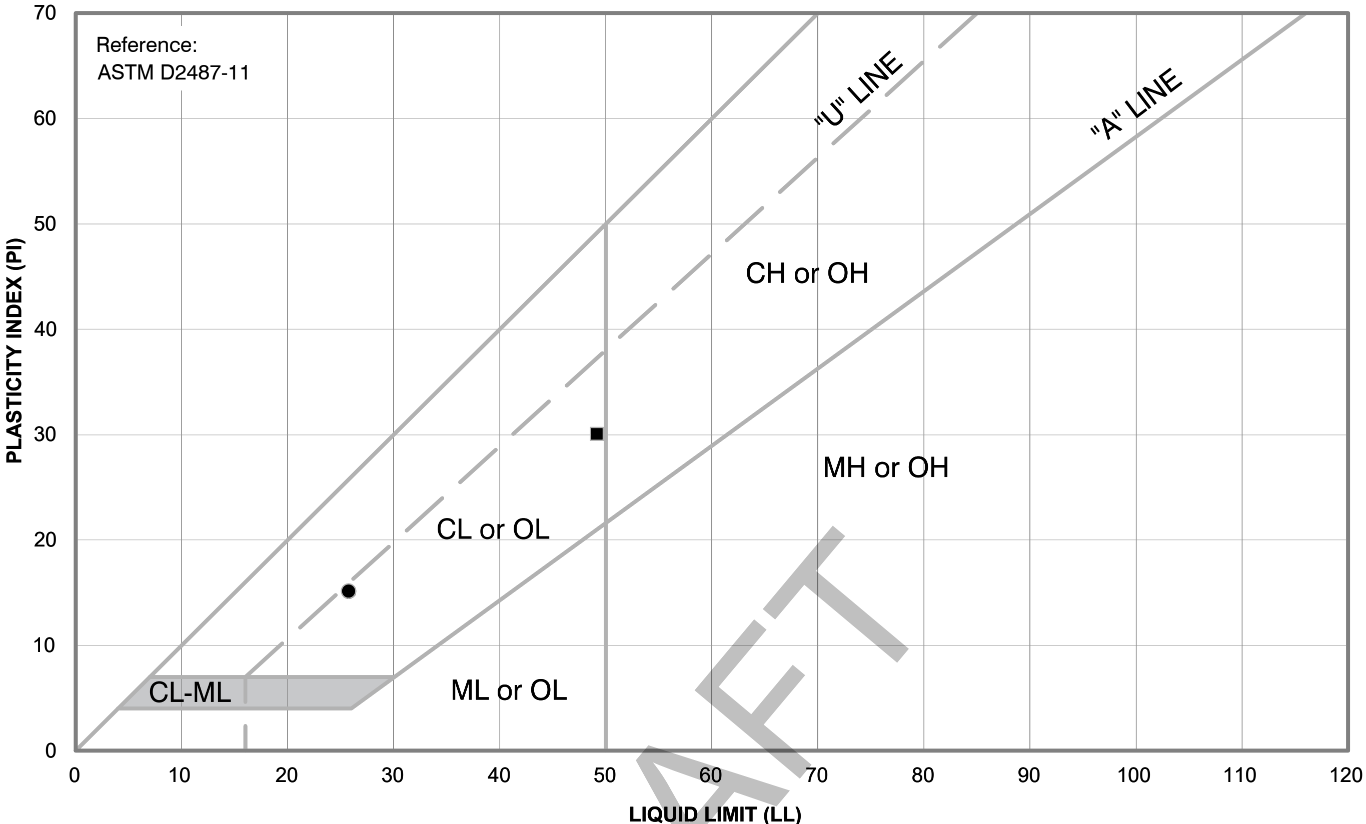
Project	Figure Title	Project No.	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	PLASTICITY CHART	770672701 Date 01/28/2021 Drawn By JDF Checked By TL	C-1

PLASTICITY CHART



DRAFT

Symbol	Source	Description and Classification	Natrual M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-2 at 131 feet	CLAY (CL), gray	23.0	36	16	--
■	B-2 at 171 feet	CLAY (CL), gray	25.2	49	30	--

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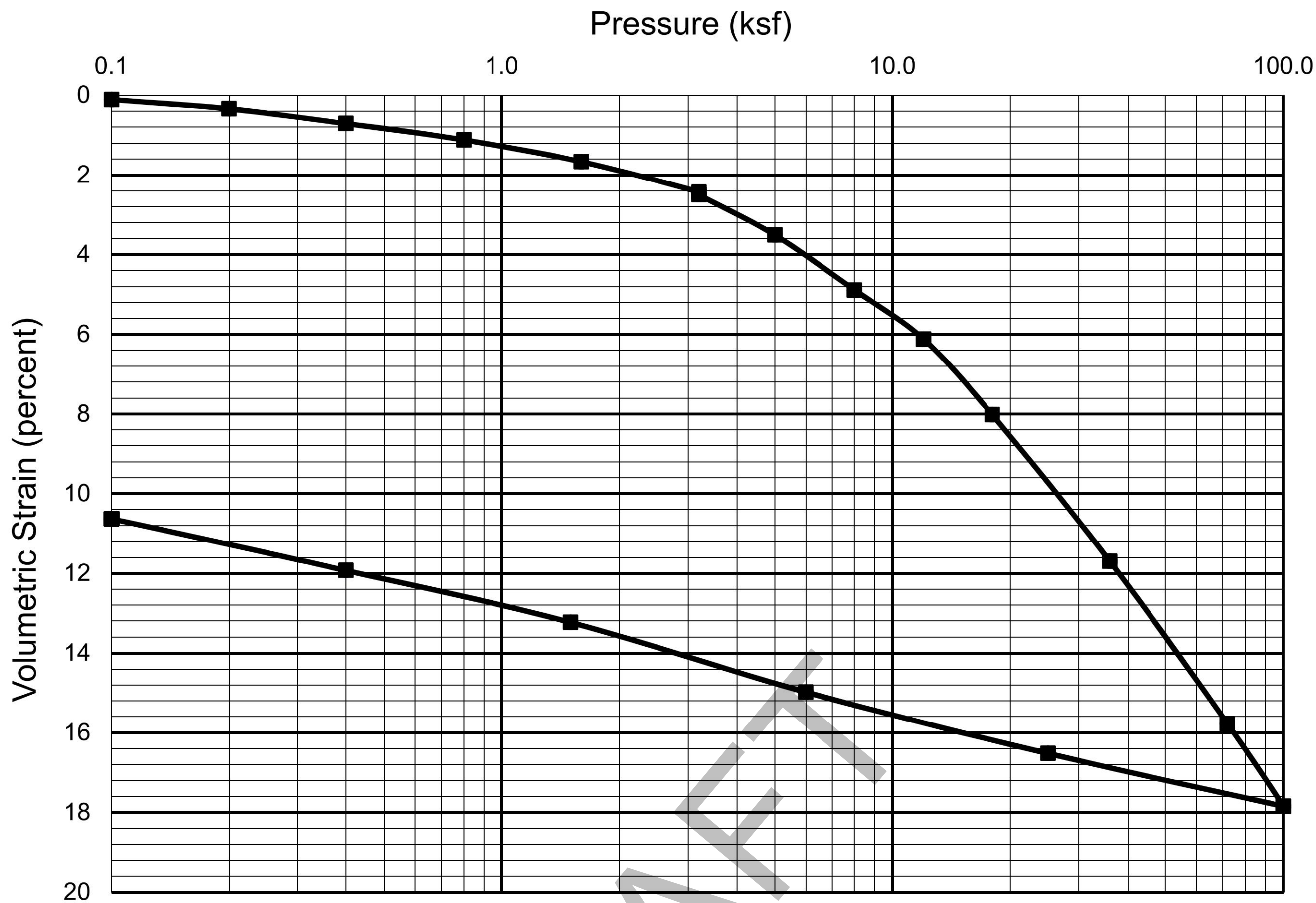
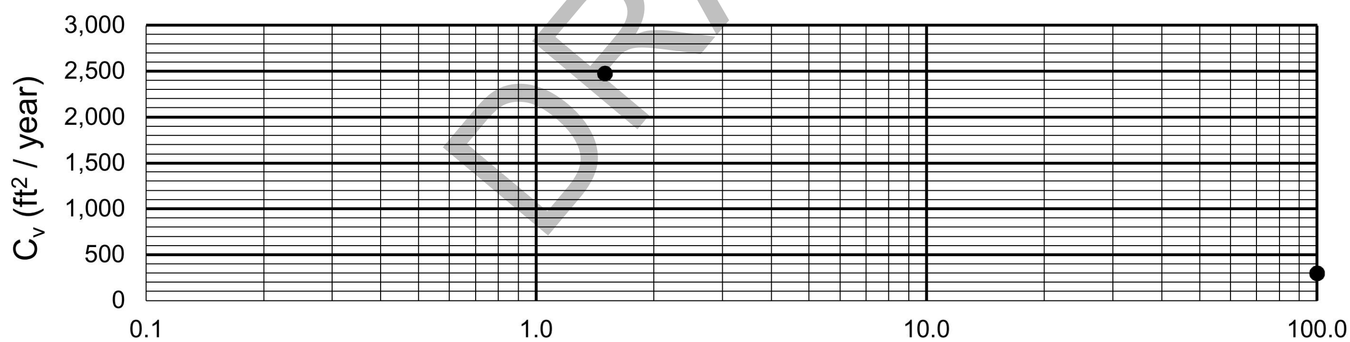
Project	Figure Title	Project No. / Date / Drawn By / Checked By	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	PLASTICITY CHART	Project No. 770672701 Date 01/28/2021 Drawn By JDF Checked By TL	C-2





Sampler Type: Pitcher Barrel				Condition		Before Test			After Test		
Diameter (in)	2.40	Height (in)	1.00	Water Content	w_o	18.6	%	w_f	16.4	%	
Overburden Pressure, p_o	7,350	psf		Void Ratio	e_o	0.61		e_f	0.44		
Preconsol. Pressure, p_c	10,000	psf		Saturation	S_o	82	%	S_f	100	%	
Compression Ratio, $C_{\varepsilon c}$	0.15			Dry Density	γ_d	105	pcf	γ_d	117	pcf	
LL = 33		PL = 18		PI = 15				G_s	2.70	(assumed)	
Classification SANDY CLAY (CL), gray to gray-brown					Source: LB-1 at 110 feet						

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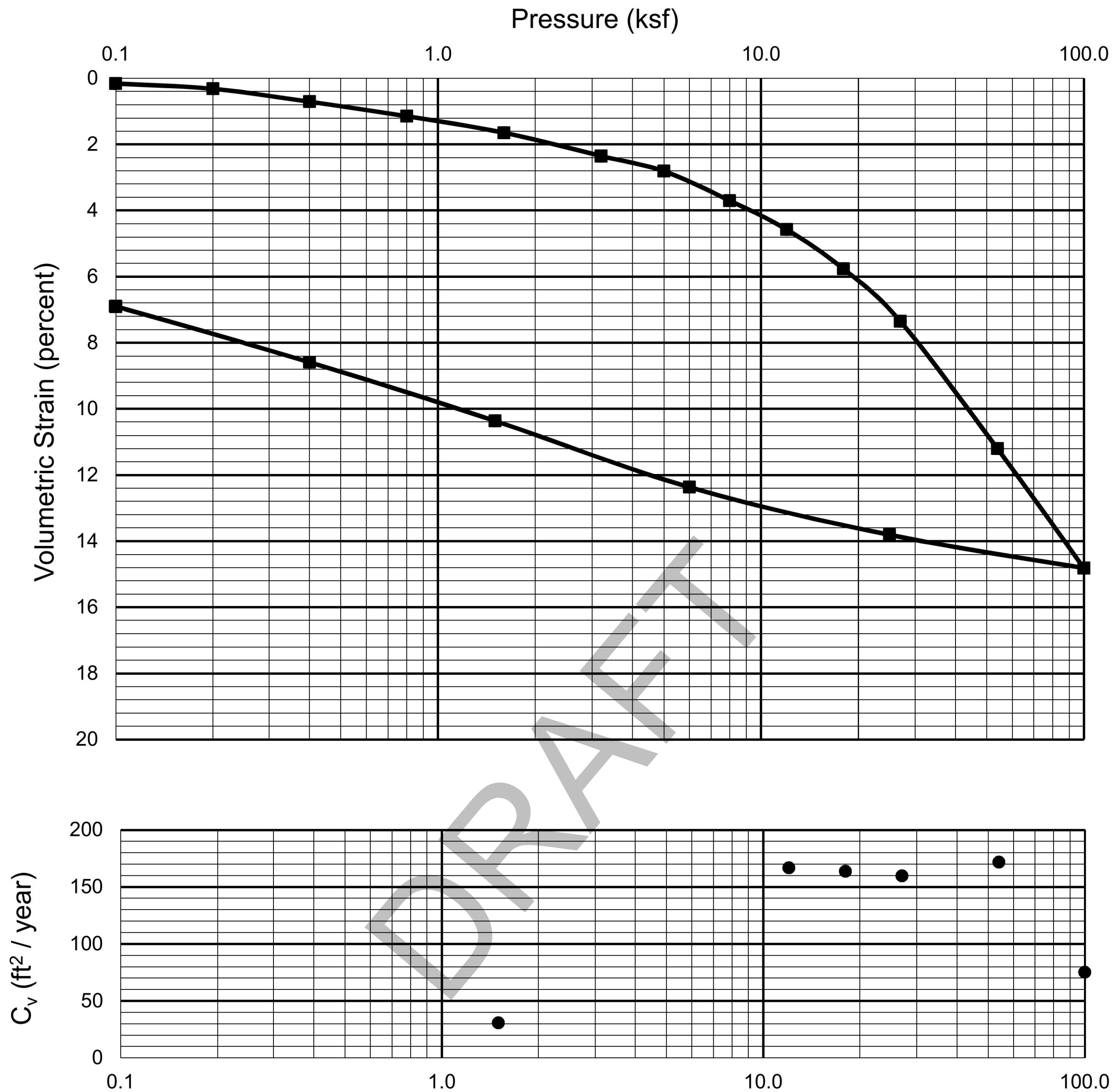
Project
PROJECT FOUNTAIN ALLEY
35 SOUTH 2ND STREET
SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
CONSOLIDATION TEST REPORT

Project No. 770672701
Date 01/28/2021
Drawn By JDF
Checked By TL

Figure
C-3



Sampler Type: Pitcher Barrel				Condition	Before Test			After Test	
Diameter (in) 2.40	Height (in) 1.00			Water Content	w_o	19.8	%	w_f	17.0 %
Overburden Pressure, p_o	9,050	psf		Void Ratio	e_o	0.57		e_f	0.46
Preconsol. Pressure, p_c	18,000	psf		Saturation	S_o	94	%	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.15			Dry Density	γ_d	108	pcf	γ_d	116 pcf
LL = 30	PL = 15			PI = 15		G_s	2.70	(assumed)	
Classification CLAY with SAND (CL), gray				Source	LB-1 at 140.5 feet				

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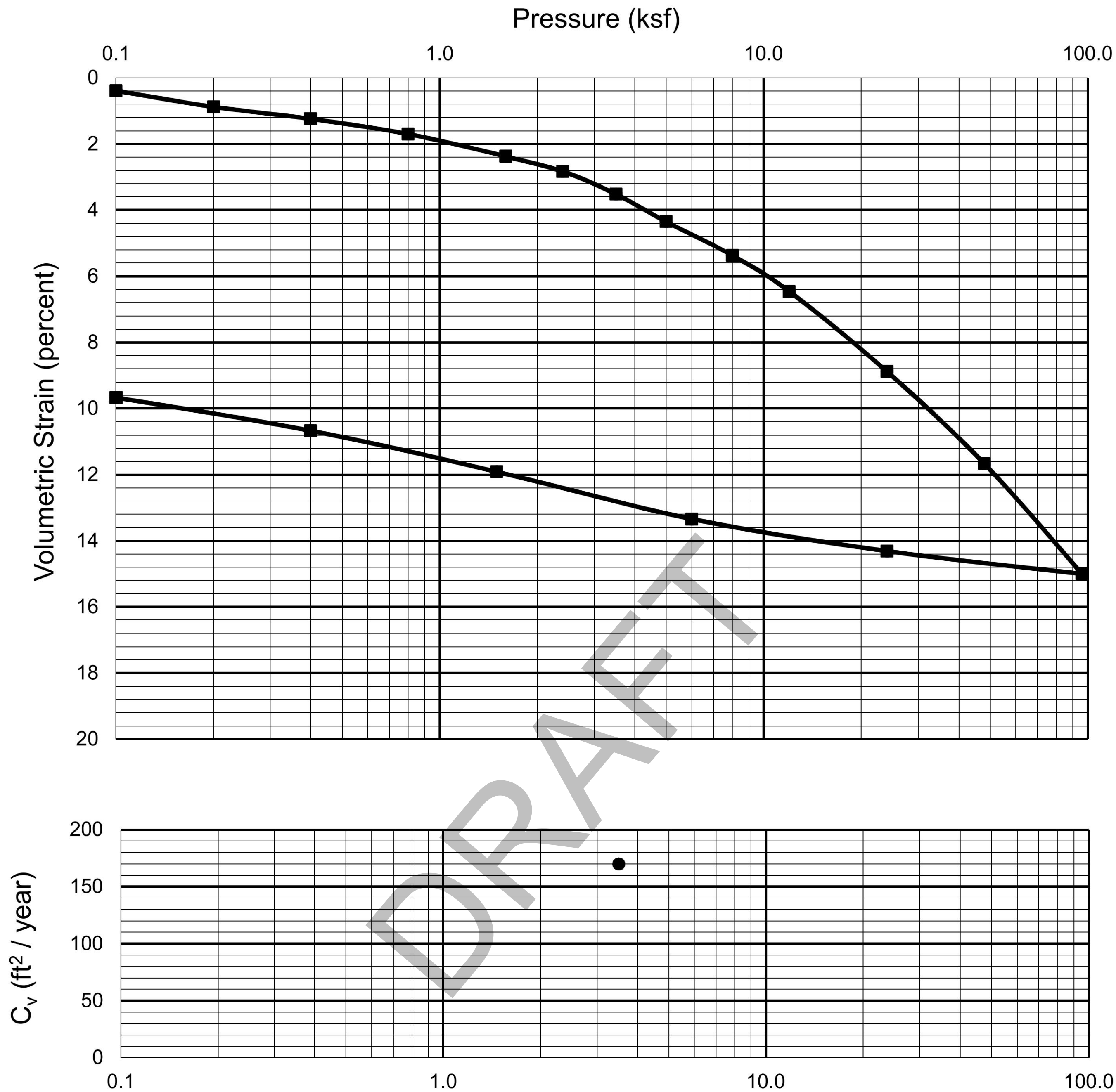

Sampler Type: Sprague & Henwood			Condition	Before Test			After Test	
Diameter (in) 2.40	Height (in) 1.00		Water Content	w_o	20.2 %		w_f	15.4 %
Overburden Pressure, p_o	11,450	psf	Void Ratio	e_o	0.57		e_f	0.41
Preconsol. Pressure, p_c	16,000	psf	Saturation	S_o	97 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.14		Dry Density	γ_d	108 pcf		γ_d	119 pcf
LL = 28	PL = 16		PI = 12		G_s	2.70	(assumed)	
Classification CLAY (CL), yellow-brown with orange mottling				Source	LB-1 at 181 feet			

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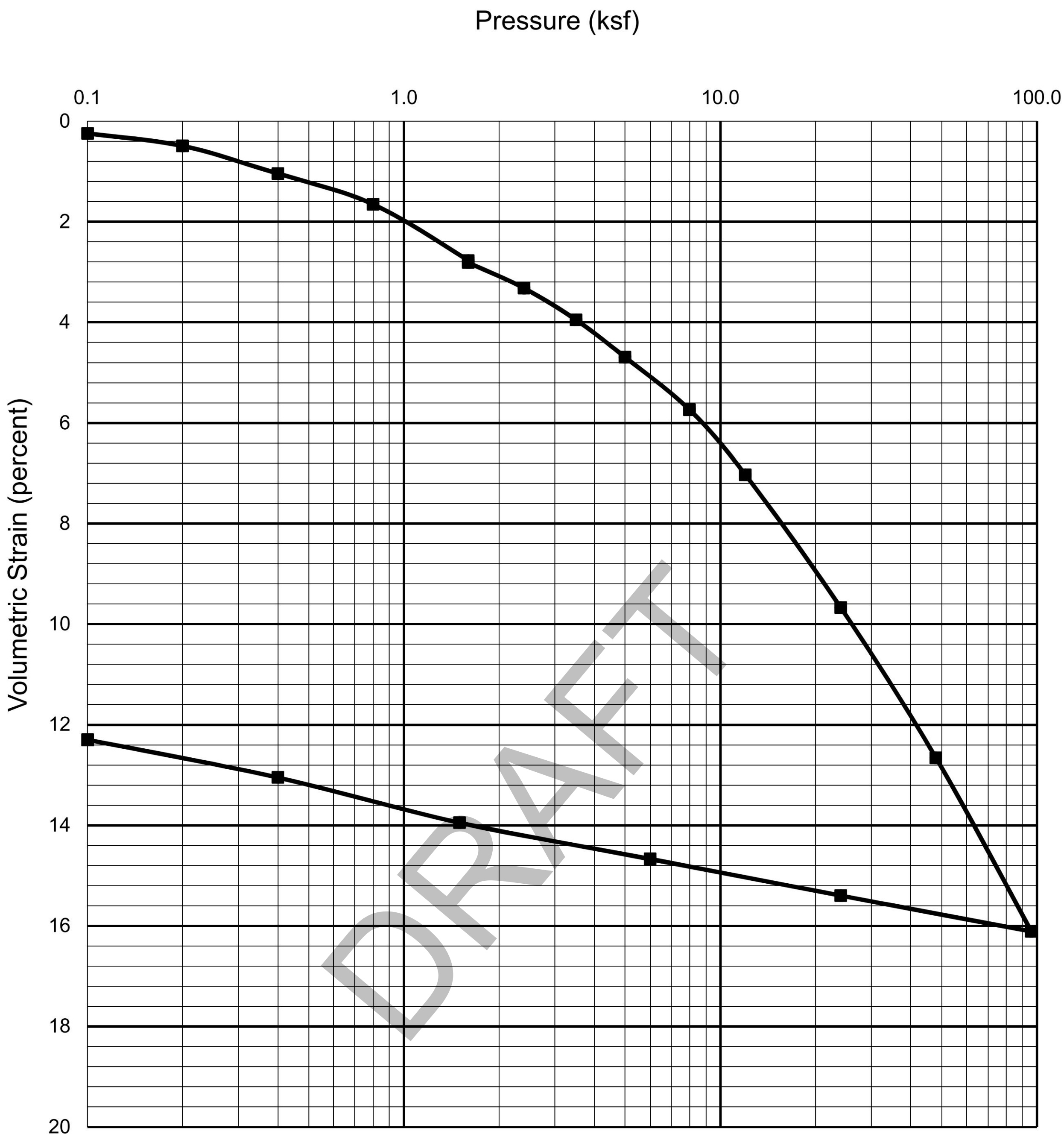

Sampler Type: Pitcher Barrel			Condition	Before Test			After Test	
Diameter (in) 2.40	Height (in) 1.00		Water Content	w_o	22.6 %		w_f	17.0 %
Overburden Pressure, p_o 7,500 psf			Void Ratio	e_o	0.66		e_f	0.46
Preconsol. Pressure, p_c 10,500 psf			Saturation	S_o	92 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$ 0.13			Dry Density	γ_d	101 pcf		γ_d	116 pcf
LL = 29	PL = 20		PI = 9			G_s	2.70 (assumed)	
Classification SANDY CLAY (CL), gray				Source	LB-2 at 110 feet			

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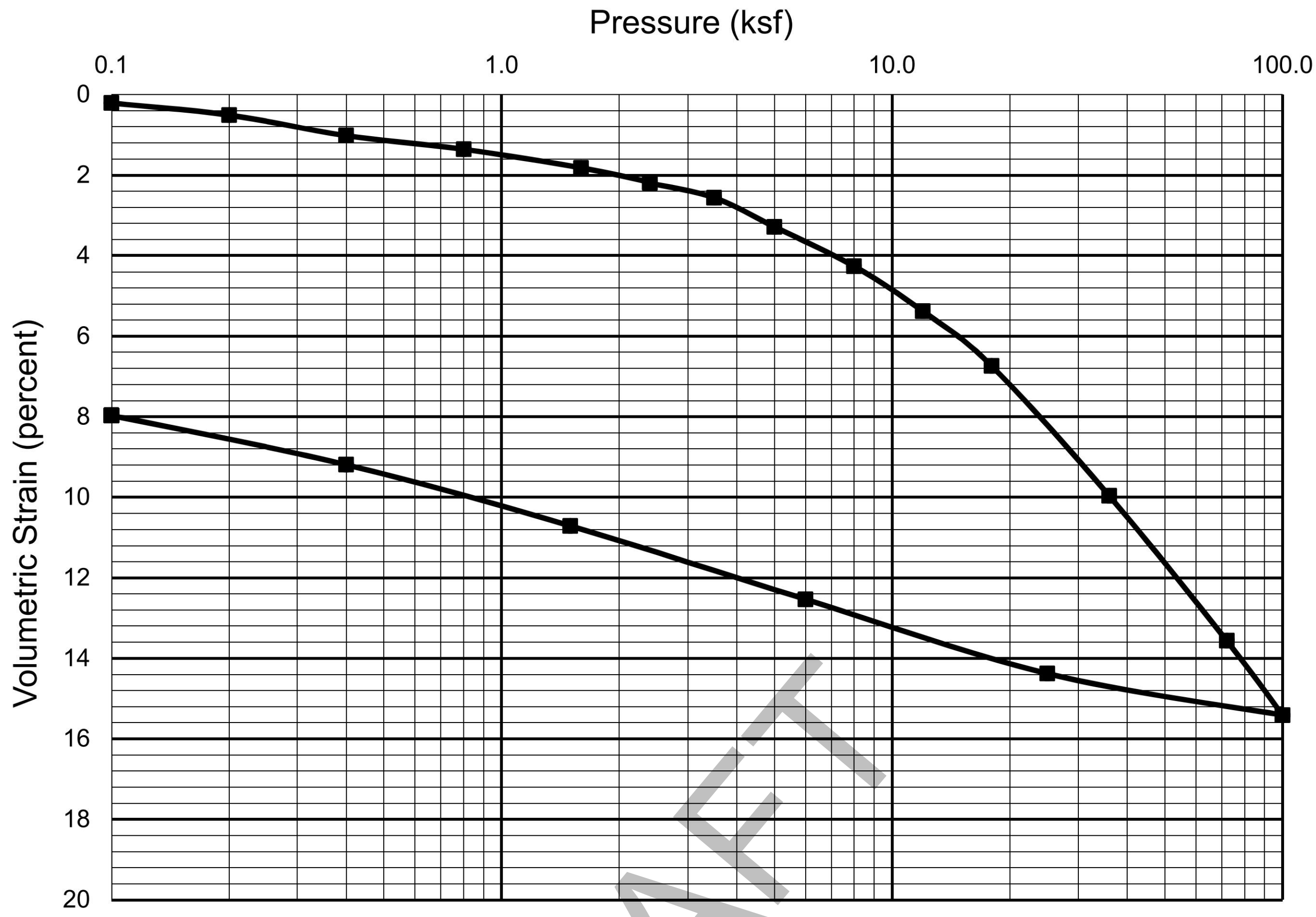

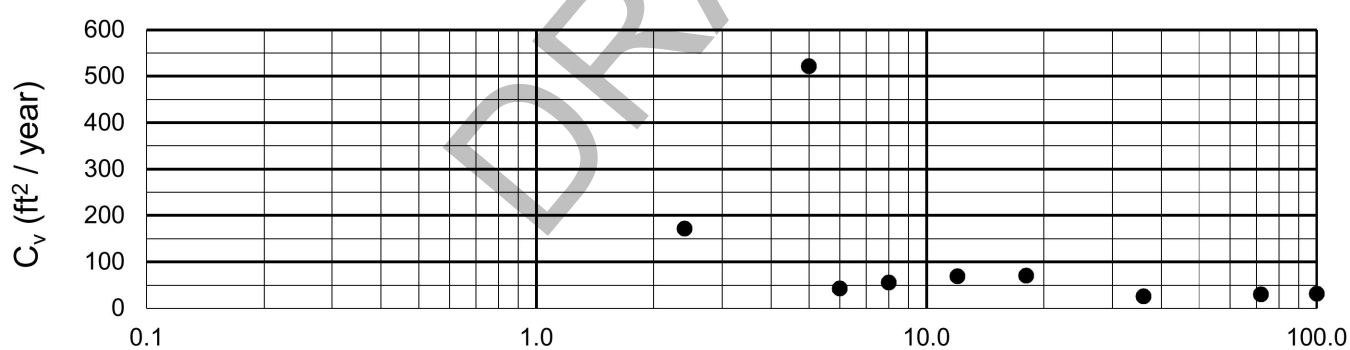

Sampler Type: Sprague & Henwood		Condition		Before Test			After Test	
Diameter (in) 2.40	Height (in) 1.00	Water Content	w_o	25.2	%	w_f	21.8	%
Overburden Pressure, p_o 11,000 psf		Void Ratio	e_o	0.72		e_f	0.59	
Preconsol. Pressure, p_c 13,500 psf		Saturation	S_o	94	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$ 0.15		Dry Density	γ_d	98	pcf	γ_d	106	pcf
LL = 49	PL = 19	PI = 30				G_s	2.70	(assumed)
Classification CLAY (CL), gray			Source	LB-2 at 171 feet				

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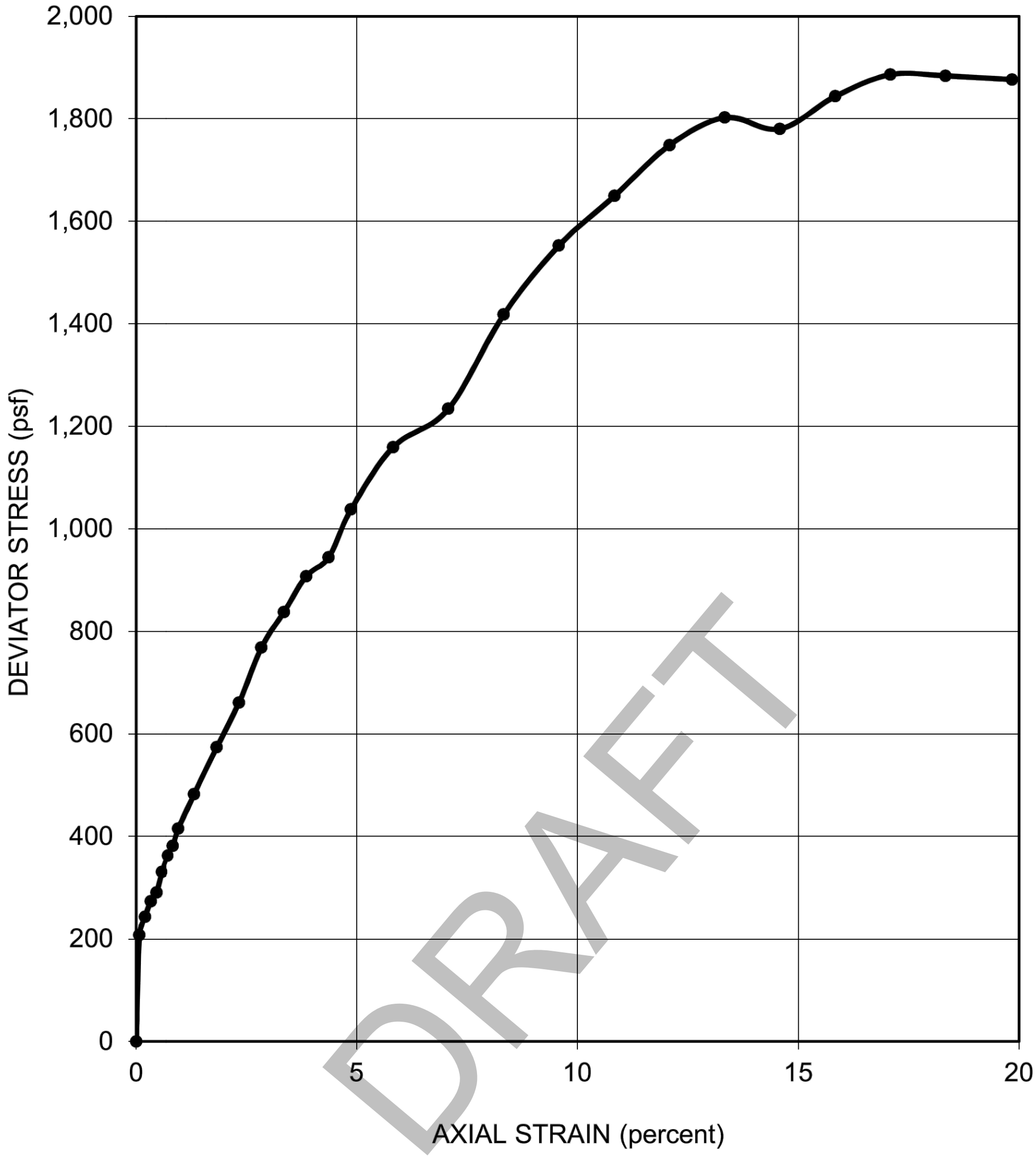
Project	Figure Title	Project No.	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	CONSOLIDATION TEST REPORT	770672701 Date 01/28/2021 Drawn By JDF Checked By TL	C-7



SAMPLER TYPE: Sprague & Henwood		SHEAR STRENGTH:	940	psf
DIAMETER (in.): 2.4	HEIGHT (in.): 5.6	STRAIN AT FAILURE:	17.1	%
MOISTURE CONTENT: 25.0 %		CONFINING PRESSURE:	3,050	psf
DRY DENSITY: 107 pcf		STRAIN RATE:	0.75	% / min
DESCRIPTION: SANDY CLAY (CL), gray-brown with orange mottling		SOURCE: LB-1 at 31 feet		

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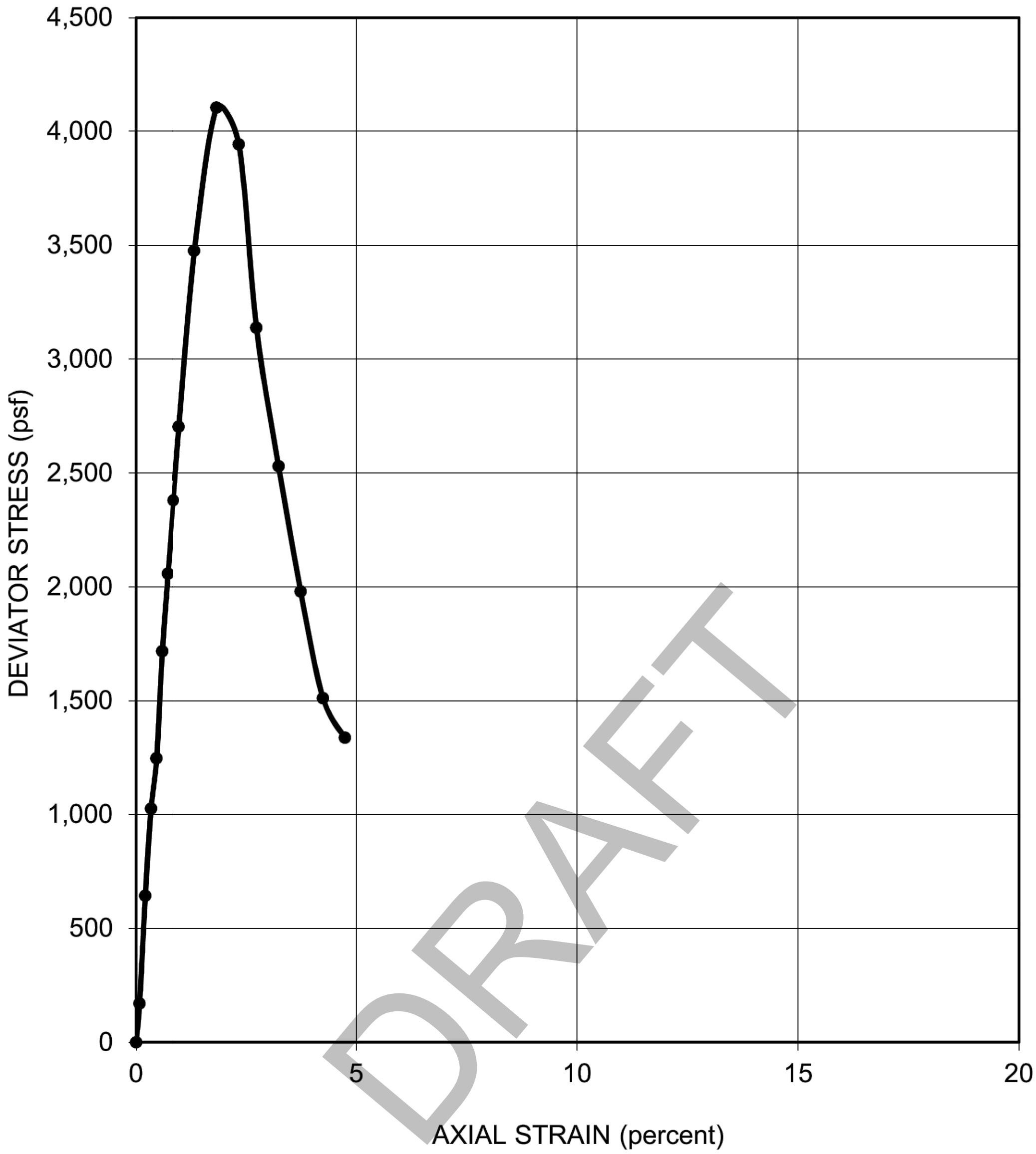

SAMPLER TYPE: Pitcher Barrel			SHEAR STRENGTH:	2,050	psf
DIAMETER (in.): 2.9	HEIGHT (in.): 6		STRAIN AT FAILURE:	1.8	%
MOISTURE CONTENT:	15.3	%	CONFINING PRESSURE:	7,600	psf
DRY DENSITY:	119	pcf	STRAIN RATE:	0.50	% / min
DESCRIPTION: SANDY CLAY (CL), gray to gray-brown			SOURCE: LB-1 at 110 feet		

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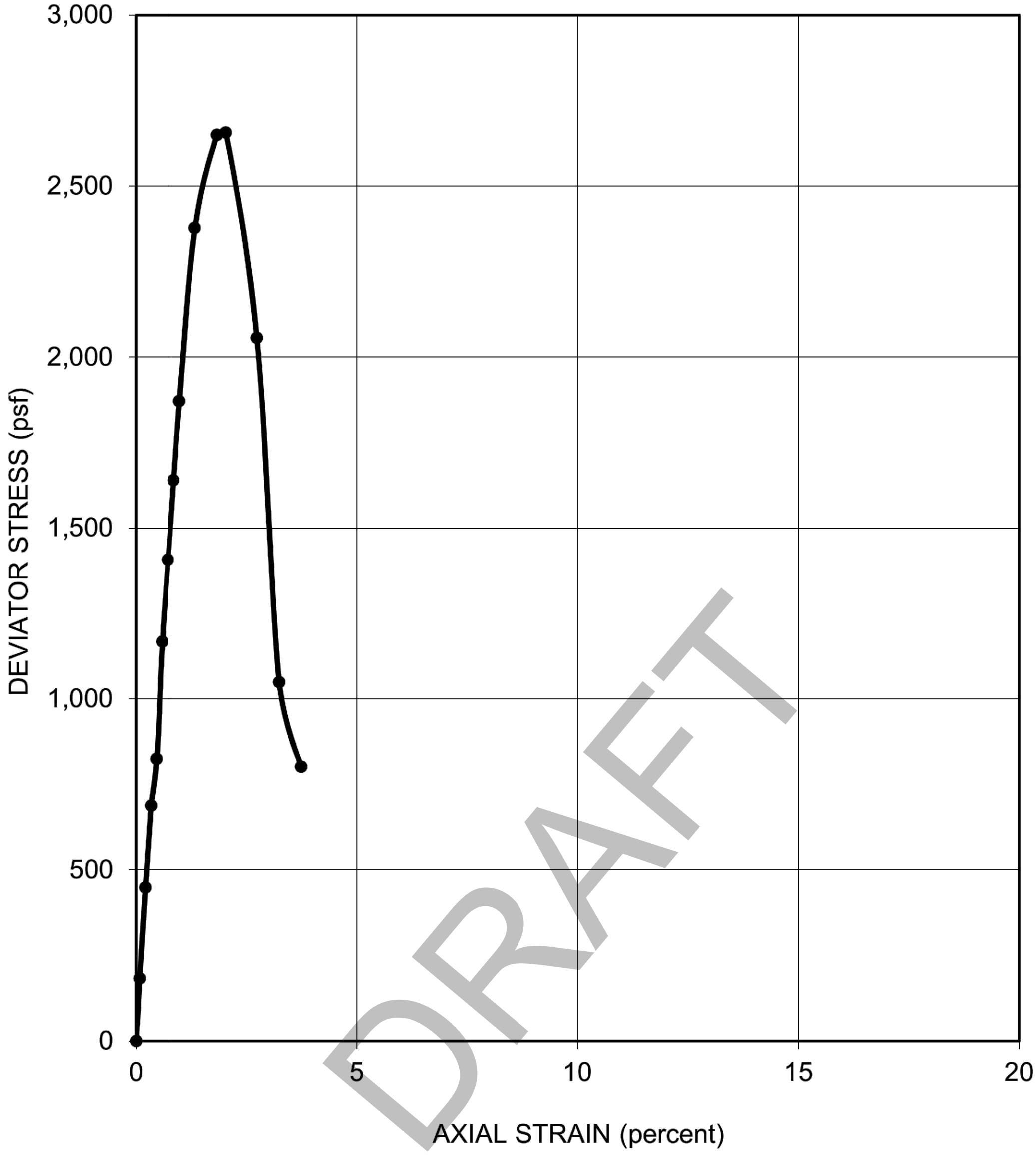

SAMPLER TYPE: Pitcher Barrel				SHEAR STRENGTH:	1,330	psf
DIAMETER (in.): 2.9	HEIGHT (in.): 6.1			STRAIN AT FAILURE:	2.0	%
MOISTURE CONTENT:		19.3	%	CONFINING PRESSURE:	9,300	psf
DRY DENSITY:		111	pcf	STRAIN RATE:	0.50	% / min
DESCRIPTION: CLAY with SAND (CL), gray				SOURCE:	LB-1 at 140 feet	

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Project
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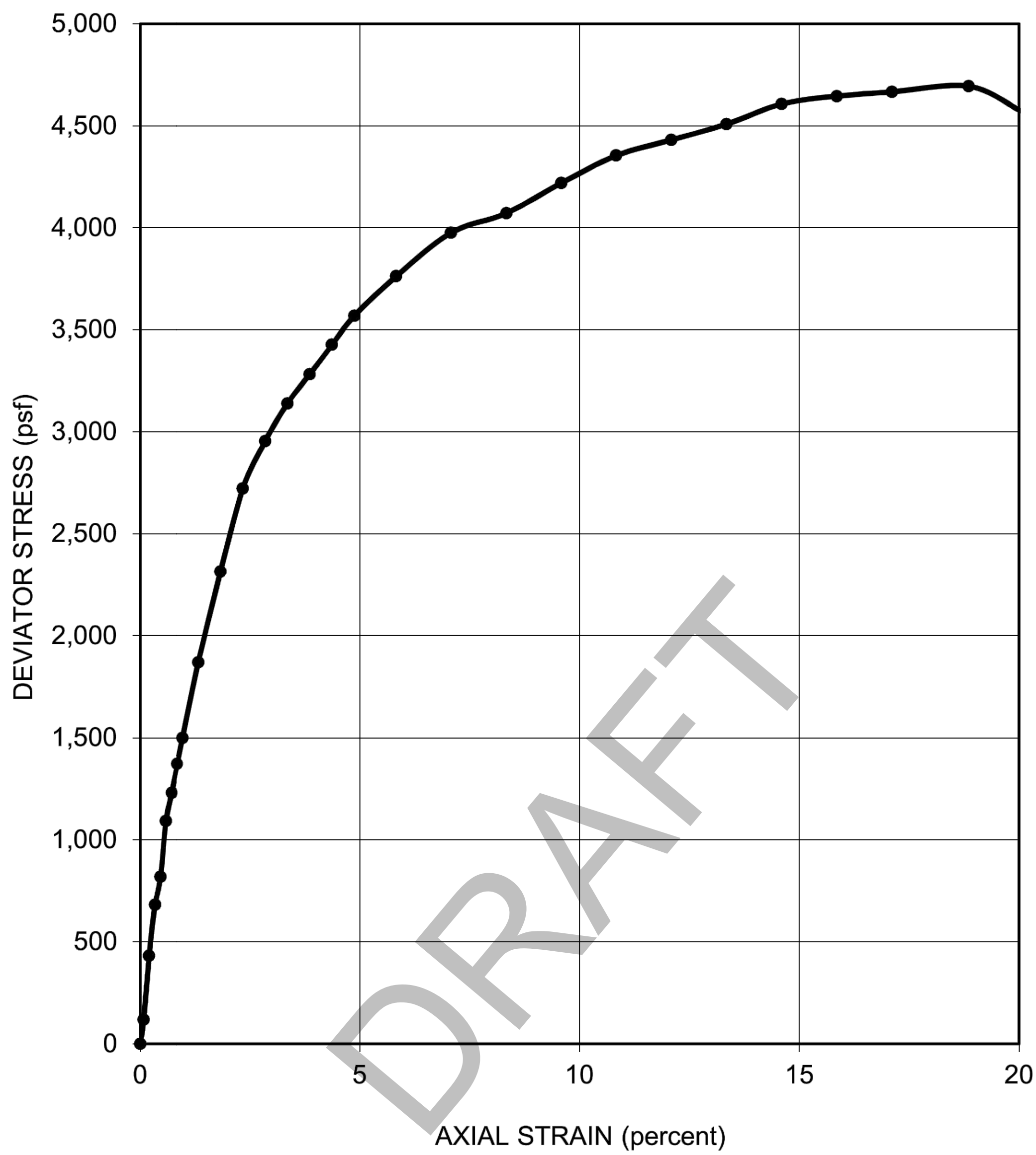
35 SOUTH 2ND STREET
SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Project No. 770672701
Date 01/28/2021
Drawn By JDF
Checked By TL

Figure
C-10



SAMPLER TYPE: Sprague & Henwood		SHEAR STRENGTH:	2,350 psf
DIAMETER (in.): 2.4	HEIGHT (in.): 5.6	STRAIN AT FAILURE:	18.9 %
MOISTURE CONTENT:	26.4 %	CONFINING PRESSURE:	6,800 psf
DRY DENSITY:	101 pcf	STRAIN RATE:	0.75 % / min
DESCRIPTION: CLAY (CH), gray-brown with orange mottling		SOURCE:	LB-2 at 96 feet

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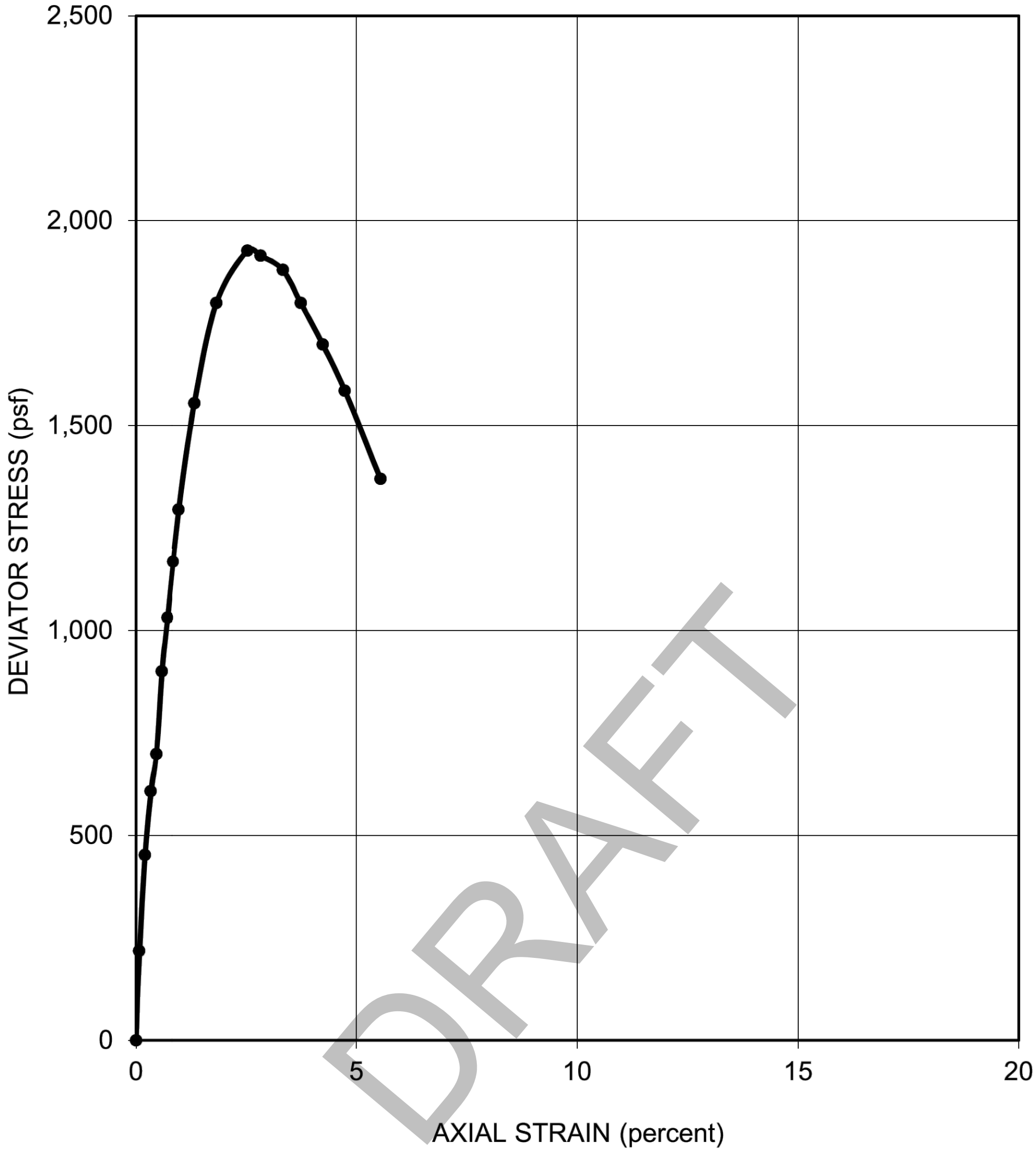
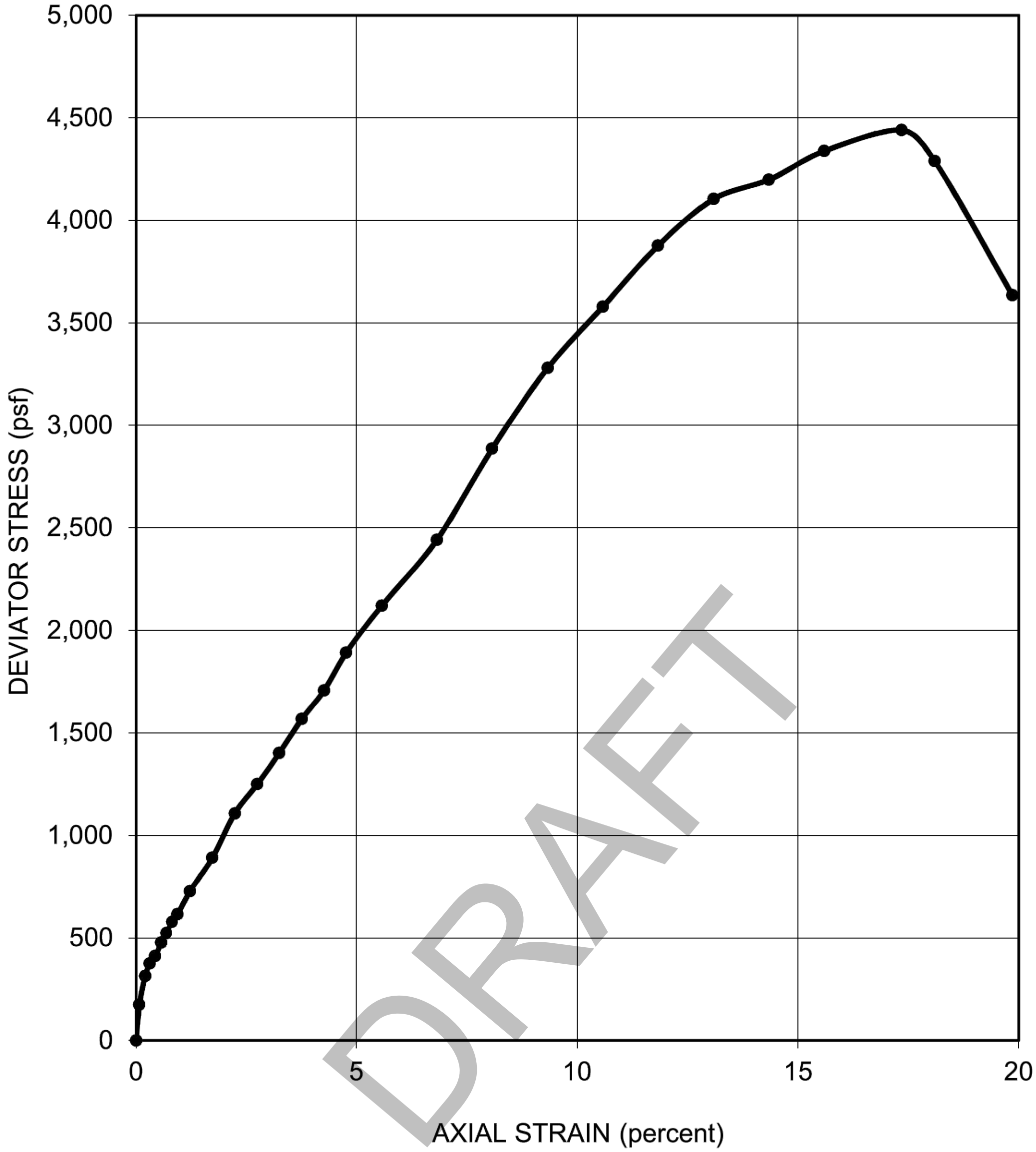


SAMPLER TYPE: Pitcher Barrel				SHEAR STRENGTH:	960	psf
DIAMETER (in.): 2.9	HEIGHT (in.): 6.1			STRAIN AT FAILURE:	2.5	%
MOISTURE CONTENT:		21.1	%	CONFINING PRESSURE:	7,600	psf
DRY DENSITY:		107	pcf	STRAIN RATE:	0.50	% / min
DESCRIPTION: SANDY CLAY (CL), gray-brown with orange mottling				SOURCE: LB-2 at 110 feet		

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	Figure Title UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST	Project No. 770672701 Date 01/28/2021 Drawn By JDF Checked By TL	Figure C-12



SAMPLER TYPE: Sprague & Henwood		SHEAR STRENGTH:	2,220	psf
DIAMETER (in.): 2.4	HEIGHT (in.): 5.6	STRAIN AT FAILURE:	17.3	%
MOISTURE CONTENT:	23.0 %	CONFINING PRESSURE:	8,800	psf
DRY DENSITY:	106 pcf	STRAIN RATE:	0.75	% / min
DESCRIPTION: CLAY (CL), gray		SOURCE:	LB-2 at 131 feet	

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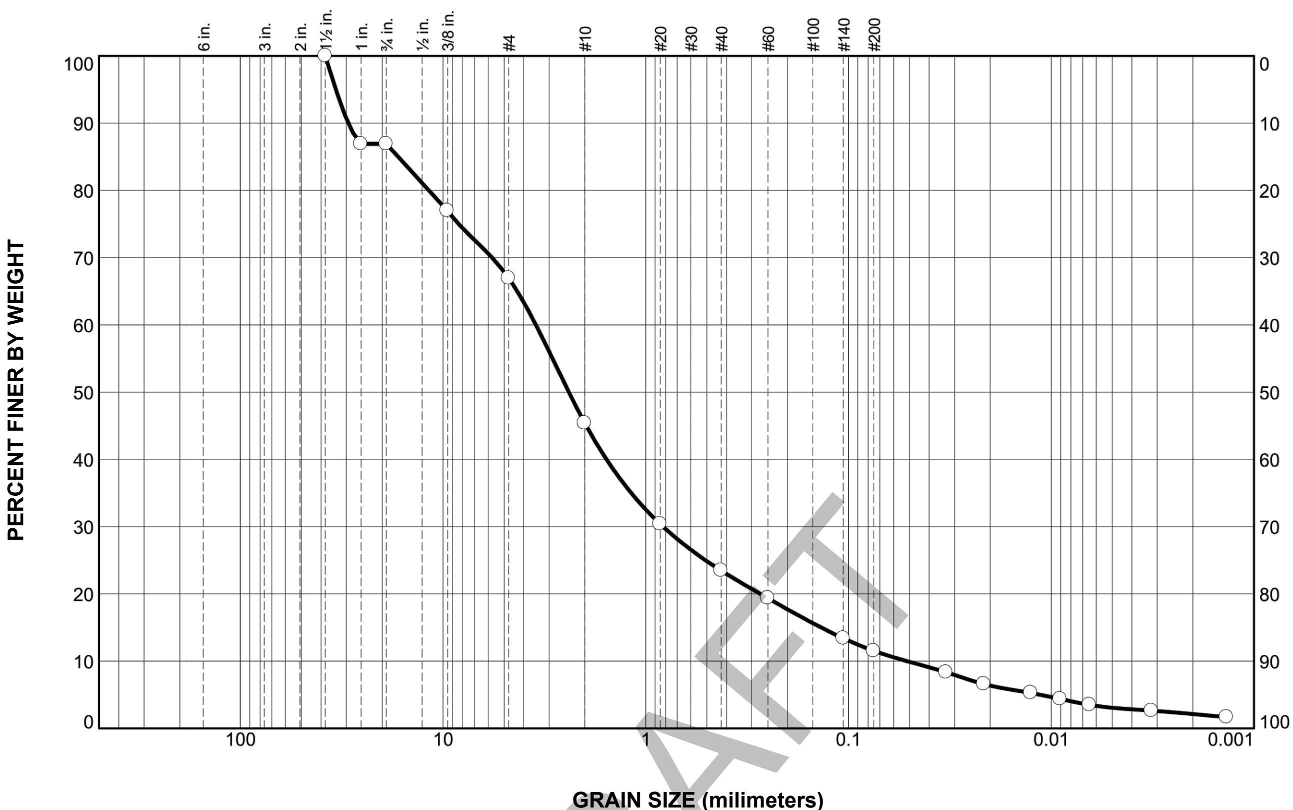
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title	Project No. 770672701	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST	Date 01/28/2021 Drawn By JDF Checked By TL	C-13



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	13	20	22	22	11	9	3

Symbol	Sample Source	Classification
○	LB-1 at 90 feet	SAND with SILT and GRAVEL (SP-SM), brown to yellow-brown

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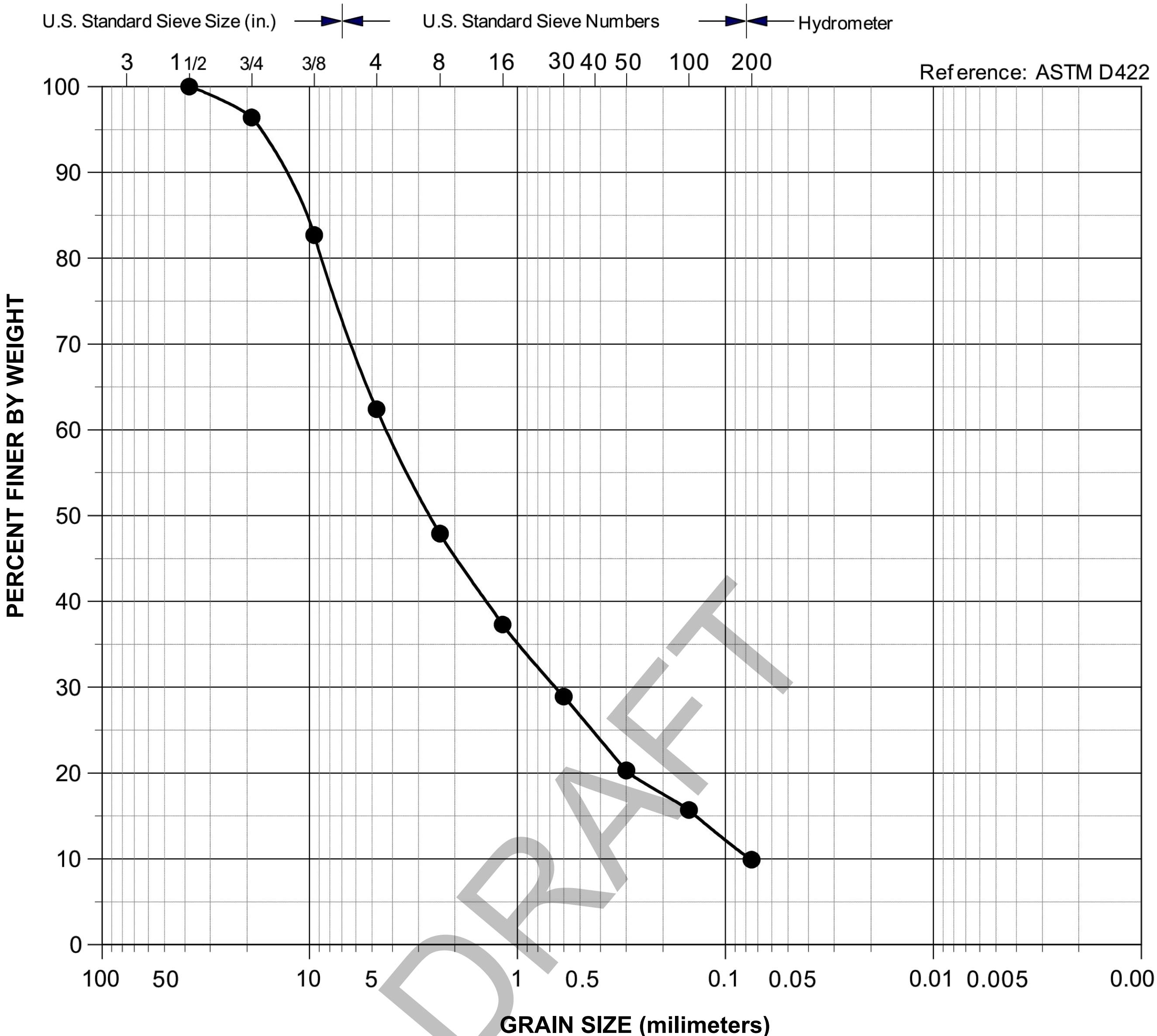
Project
PROJECT FOUNTAIN ALLEY
35 SOUTH 2ND STREET
SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
PARTICLE SIZE ANALYSIS

Project No. 770672701
Date 01/28/2021
Drawn By JDF
Checked By TL

Figure
C-14



% +3"	% Gravel	% Sand	% Fines
0.0	37.6	52.6	9.8

Symbol	Sample Source	Classification
●	LB-1 at 113.5 feet	SAND with CLAY and GRAVEL (SP-SC), yellow-brown to gray-brown

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San Jose, CA 95113
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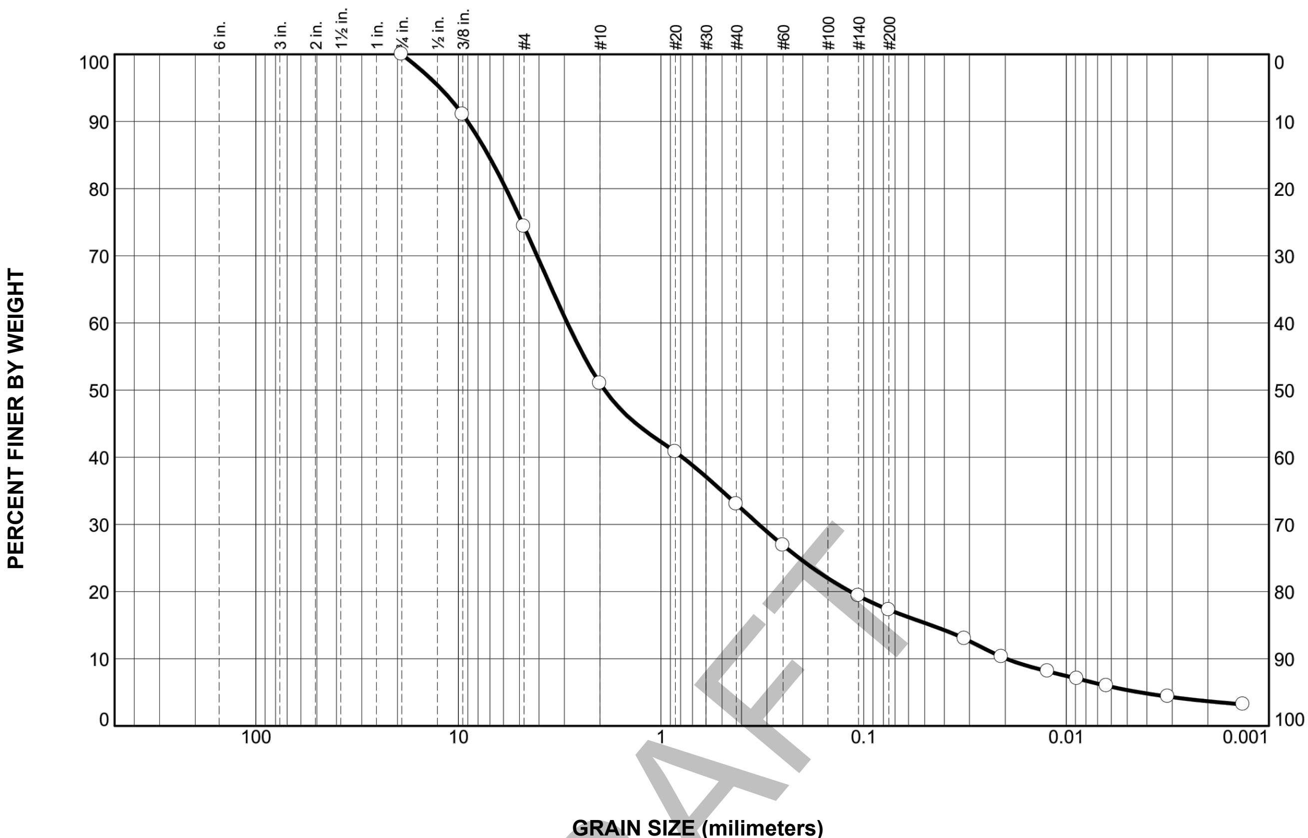
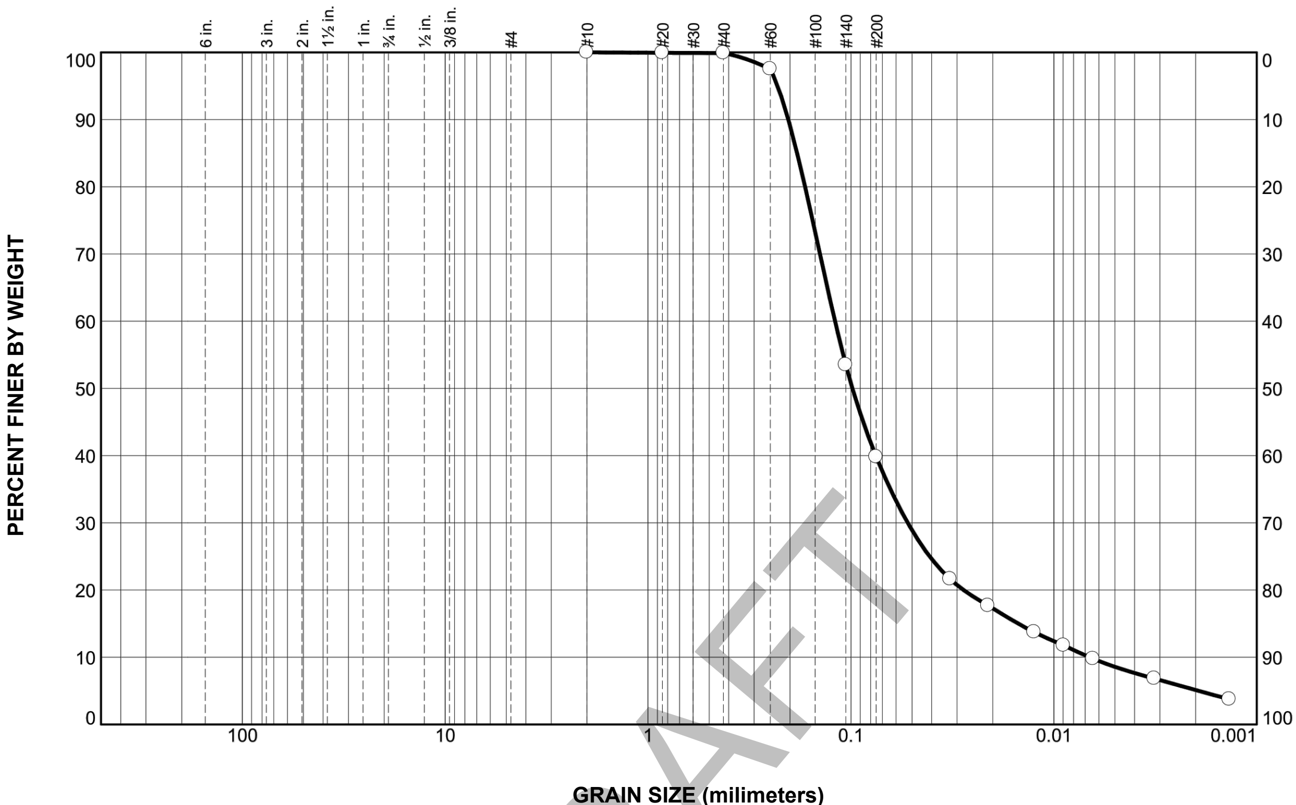
Project	Figure Title	Project No.	Figure
PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	PARTICLE SIZE ANALYSIS	770672701 Date 01/28/2021 Drawn By JDF Checked By TL	C-15



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	26	23	18	16	12	5

Symbol	Sample Source	Classification
○	LB-2 at 90 feet	SILTY SAND with GRAVEL (SM), yellow-brown

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	Figure Title PARTICLE SIZE ANALYSIS	Project No. 770672701 Date 01/28/2021 Drawn By JDF Checked By TL	Figure C-16



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	0	60	31	9

Symbol	Sample Source	Classification
○	LB-2 at 151 feet	SILTY SAND (SM), yellow-brown

LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	PROJECT FOUNTAIN ALLEY 35 SOUTH 2ND STREET SAN JOSE SANTA CLARA COUNTY CALIFORNIA	PARTICLE SIZE ANALYSIS	770672701 Date 01/28/2021 Drawn By JDF Checked By TL	C-17

Filename: \\langan.com\data\SJO\data7\770672701\Project Data\CAD\01\2D-DesignFiles\Geotechnical\FG01-770672701-B-GI0101_Lab.dwg Date: 2/1/2021 Time: 17:06 User: agekas Style Table: Langan.stb Layout: Fig C-17 Particle Size LB-2 151ft

APPENDIX D

CORRISIVITY ANALYSES WITH BRIEF EVALUATION

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

23 November, 2020

Job No. 2011124
Cust. No. 12242

Mr. Tim Light
Langan
1 Almaden Blvd., Suite 590
San Jose, CA 95113

Subject: Project No.: 770672701.700.318.0
Project Name: Fountain Alley-Hand Auger Boring
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Wong:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on November 17, 2020. Based on the analytical results, a brief evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample 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 concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration is 21 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 8.18, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 280-mV and 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

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

Client:	Langan
Client's Project No.:	770672701.700.318.0
Client's Project Name:	Fountain Alley - Hand Auger Boring
Date Sampled:	5-Nov-20
Date Received:	17-Nov-20
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 23-Nov-2020

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)*
2011124-001	LB-1, 2-3'	280	8.18	-	2,900	-	N.D.	21

	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
Date Analyzed:	20-Nov-2020	20-Nov-2020	-	19-Nov-2020	-	20-Nov-2020	20-Nov-2020

Cheryl McMillen
Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

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

Chain of Custody 2011124

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775
CERCO analytical

Job No.	CU#	Client Project I.D.
770672701/700/318.0	12242	Fountain Alley

Schedule:

Analyte:

Date Sampled	Date Due
11/5/2020	

Full Name
Tim Light, tlight@langan.com

Phone 415-955-5227 X
Fax
Cell

Company and/or Mailing Address
Langan, 1 South Almaden Blvd, #590, San Jose, CA 95113 ☒

Sample Source
Hand Auger boring

ANALYSIS — ASTM

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated			Brief Evaluation						
	LB-1, 2-3'	11/5/2020	8:50a	Soil	Bag			1	X	X	X	X	X			X						

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

SAMPLE RECEIPT
Total No. of Containers:
Rec'd Good Cond/Cold:
Conforms to Record:
Temp. a t Lab -°C:
Sampler:

		Date	Time
Relinquished By:	[signature]	2020-11-13	9:45 AM
Received By:	[signature]	11/13/20	0945
Relinquished By:			
Received By:			
Relinquished By:			
Received By:			

Comments:

THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM METAL TUBES

Email Address: tlight@langan.com

APPENDIX E

BORING AND CPT LOGS FROM PREVIOUS INVESTIGATIONS

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Fountain Alley Tower
PROJECT NUMBER 1178-3-2
PROJECT LOCATION 35 South 2nd Street, San Jose, CA

DATE STARTED 3/11/20 **DATE COMPLETED** 3/11/20
GROUND ELEVATION 87 FT +/- **BORING DEPTH** 90.9 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
∇ **AT TIME OF DRILLING** 16 ft.
NOTES
▼ **AT END OF DRILLING** 30 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
87.0 / 86.8	0	2 inches asphalt concrete over 3 inches aggregate base							
86.6		**Lean Clay with Sand (CL) [Fill]** hard, moist, brown, fine sand, low plasticity	36	MC-1B	124	13	10		○
85.0		**Lean Clay with Sand (CL)** hard, moist, brown, fine sand, low plasticity Liquid Limit = 25, Plastic Limit = 15	17	MC-2B	109	10			
82.0	5	**Silty Sand (SM)** loose, moist, brown, fine sand	14	MC-3B	93	13		57	
76.5	10	**Sandy Lean Clay (CL)** stiff, moist, brown, fine sand, low plasticity							
75.0		**Lean Clay (CL)** stiff, moist, gray with brown mottles, some fine sand, moderate plasticity	17	MC-4B	92	31			○
72.0	15	**Silty Sand (SM)** medium dense, moist, brown, fine sand	23	MC-5B		19		35	
69.5		**Lean Clay (CL)** medium stiff, moist, gray, some fine sand, moderate plasticity	11	MC-6B	94	30			△○
	20		21	MC-7B	87	33			△○
63.0									

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME Fountain Alley Tower

PROJECT NUMBER 1178-3-2

PROJECT LOCATION 35 South 2nd Street, San Jose, CA

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
63.0		**Lean Clay with Sand (CL)** stiff, moist, gray, fine sand, moderate plasticity		ST-8	108	20		
	25							
			24	MC-9B	103	23		
	30							
55.0		**Sandy Silt (ML)** medium stiff, moist, gray, fine sand						
		NP = non plastic	20	MC-10B	101	26	NP	
52.0	35	**Poorly Graded Sand with Silt (SP-SM)** medium dense, wet, brown, fine to medium sand	26	MC-11B	112	21		11
			34	SPT				
49.5		**Lean Clay (CL)** medium stiff, moist, gray, some fine sand, moderate plasticity		ST-13	82	37		
	40							
45.5		**Poorly Graded Sand with Clay (SP-SC)** medium dense, wet, brown, fine to medium sand						
43.0		**Well Graded Gravel with Silt and Sand (GW-GM)** very dense, wet, brown, fine to coarse subangular to subrounded gravel, fine to coarse sand	72	SPT-14B		9		
	45							
			50/6"	SPT				
	50							
35.3								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME Fountain Alley Tower
PROJECT NUMBER 1178-3-2
PROJECT LOCATION 35 South 2nd Street, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
35.3		**Well Graded Gravel with Silt and Sand (GW-GM)** very dense, wet, brown, fine to coarse subangular to subrounded gravel, fine to coarse sand	50/3"	MC-16	117	17			
	55								
30.5		**Poorly Graded Sand (SP)** very dense, wet, brown, fine to medium sand	64	MC-17A	112	16		4	>4.5 ○
27.5	60	**Lean Clay (CL)** hard, moist, gray, some fine sand, moderate plasticity	50/5"	MC					
26.0		**Poorly Graded Sand with Silt (SP-SM)** very dense, wet, brown, fine to medium sand, some fine subangular to subrounded gravel							
		becomes dense	52	MC-19B	112	16			
22.0	65	**Lean Clay (CL)** stiff, moist, gray, some fine sand, moderate plasticity	29	SPT					○
				ST					○
	70	becomes very stiff	64	MC-22		24			○
	75		50/5"	MC-23B	131	9			
11.5		**Clayey Gravel with Sand (GC)** very dense, wet, brown, fine to coarse subangular to subrounded gravel, fine to coarse sand							
8.5									
7.6									

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME Fountain Alley Tower
PROJECT NUMBER 1178-3-2
PROJECT LOCATION 35 South 2nd Street, San Jose, CA

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
7.6	79.4	**Well Graded Gravel with Silt and Sand (GW-GM)** very dense, wet, brown, fine to coarse subangular to subrounded gravel, fine to coarse sand							
	80		50/2"	MC					
	85		50/6"	SPT-25		14			
	90		50/5"	SPT					
-3.9	90.9	Bottom of Boring at 90.9 feet.							

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Fountain Alley Tower
PROJECT NUMBER 1178-3-2
PROJECT LOCATION 35 South 2nd Street, San Jose, CA
DATE STARTED 3/12/20 **DATE COMPLETED** 3/12/20
GROUND ELEVATION 87 FT +/- **BORING DEPTH** 71.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** 10 ft.
NOTES
▼ **AT END OF DRILLING** 38.5 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
87.0 / 86.8 / 86.4	0	2 inches asphalt concrete over 5 inches aggregate base							
		Clayey Sand with Gravel (SC) [Fill] medium dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	28	MC-1B	130	11			
			48	MC-2B	141	10			
	5								
			52	MC-3B	120	10			
77.0 ▽	10	**Lean Clay with Sand (CL)** very stiff, moist, brown with gray mottles, fine sand, low plasticity	19	MC-4B	95	28			○
			20	MC-5B	95	28			
72.0	15	**Lean Clay (CL)** stiff, moist, gray with dark gray mottles, some fine sand, some organics, moderate plasticity	17	MC-6B	80	35			○ △
	20		33	MC-7B	81	30			△ ○
65.0		**Lean Clay with Sand (CL)** medium stiff, moist, brown, fine sand, low plasticity							
63.0									

Continued Next Page

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Fountain Alley Tower

PROJECT NUMBER 1178-3-2

PROJECT LOCATION 35 South 2nd Street, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
63.0		**Lean Clay with Sand (CL)** medium stiff, moist, brown, fine sand, low plasticity		8B	105	22			▲
	25		19	MC					○
				8C	111	26			△
61.0		**Sandy Lean Clay (CL)** very stiff to stiff, moist, gray with brown mottles, fine to medium sand, low plasticity		ST-9	94	19			○
			25	MC-10A		24		51	○
57.3	30	**Lean Clay with Sand (CL)** very stiff to stiff, moist, brown, fine sand, low plasticity							
			25	MC-11B	102	23			△ ○
	35		35	MC					
51.0		**Fat Clay (CH)** medium stiff, moist, gray, some fine sand, high plasticity	19	MC-13B	69	56			△ ○
	40								
45.5		**Well Graded Gravel with Silt and Sand (GW-GM)** very dense, wet, brown, fine to coarse subangular to subrounded gravel, fine to coarse sand	50/6"	SPT-14		6			
	45								
			50/6"	SPT-15B		25			>4.5 ○
37.5	50	**Lean Clay with Sand (CL)** hard, moist, brown and gray mottled, fine sand, moderate plasticity							
35.3									

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME Fountain Alley Tower

PROJECT NUMBER 1178-3-2

PROJECT LOCATION 35 South 2nd Street, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
35.3		**Lean Clay with Sand (CL)** hard, moist, brown and gray mottled, fine sand, moderate plasticity							
	55	becomes stiff	29	SPT-16		24			△ (~3.6)
29.0		**Poorly Graded Sand with Silt (SP-SM)** dense to very dense, wet, brown, fine to medium sand, some fine subangular to subrounded gravel	65	MC-17B	108	20			
	60		48	SPT					
	65		60	MC					○ (~1.5)
21.0		**Lean Clay (CL)** stiff, moist, gray, some fine sand, moderate plasticity							
18.0		**Sandy Lean Clay (CL)** stiff, moist, gray with brown mottles, fine sand, low plasticity							
	70		60	MC-20B	96	22			○ (~2.0)
15.5		Bottom of Boring at 71.5 feet.							
	75								

BORING NUMBER EB-2A

CORNERSTONE EARTH GROUP

PROJECT NAME	Fountain Alley Tower
PROJECT NUMBER	1178-3-2
PROJECT LOCATION	35 South 2nd Street, San Jose, CA
DATE STARTED	3/12/20
DATE COMPLETED	3/12/20
GROUND ELEVATION	87 FT +/-
BORING DEPTH	10 ft.
DRILLING CONTRACTOR	Exploration Geoservices, Inc.
LATITUDE	
LONGITUDE	
DRILLING METHOD	Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:	
LOGGED BY	EA
AT TIME OF DRILLING	Not Encountered
NOTES	
AT END OF DRILLING	Not Encountered

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
87.0 / 86.8	0	2 inches asphalt concrete over 5 inches aggregate base							
86.4		**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	31	MC-1B	138	8			
		abundant brick fragments at 4 feet.	38	MC-2B	126	10			
	5								
			40	MC-3B	119	7			
77.0	10	Practical auger refusal at 10 feet. Bottom of Boring at 10.0 feet.							
	15								
	20								

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL; 1.0 2.0 3.0 4.0

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

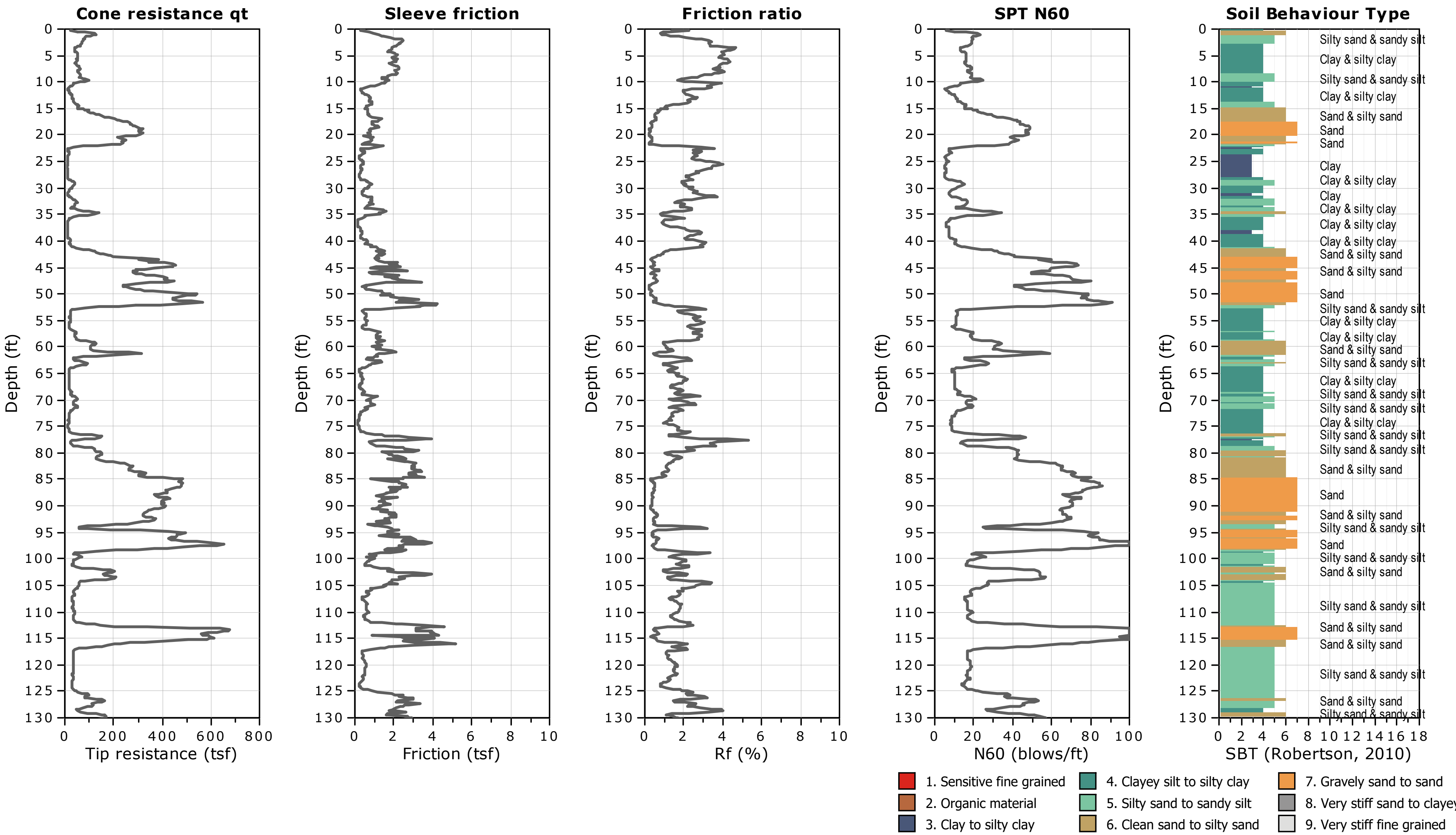
CPT: CPT-01

CLIENT: CORNERSTONE

SITE: PUBLIC PARKING LOT, SANTA CLARA, CA

FIELD REP: MATT SCHAFFER

Total depth: 130.58 ft, Date: 2/26/2020



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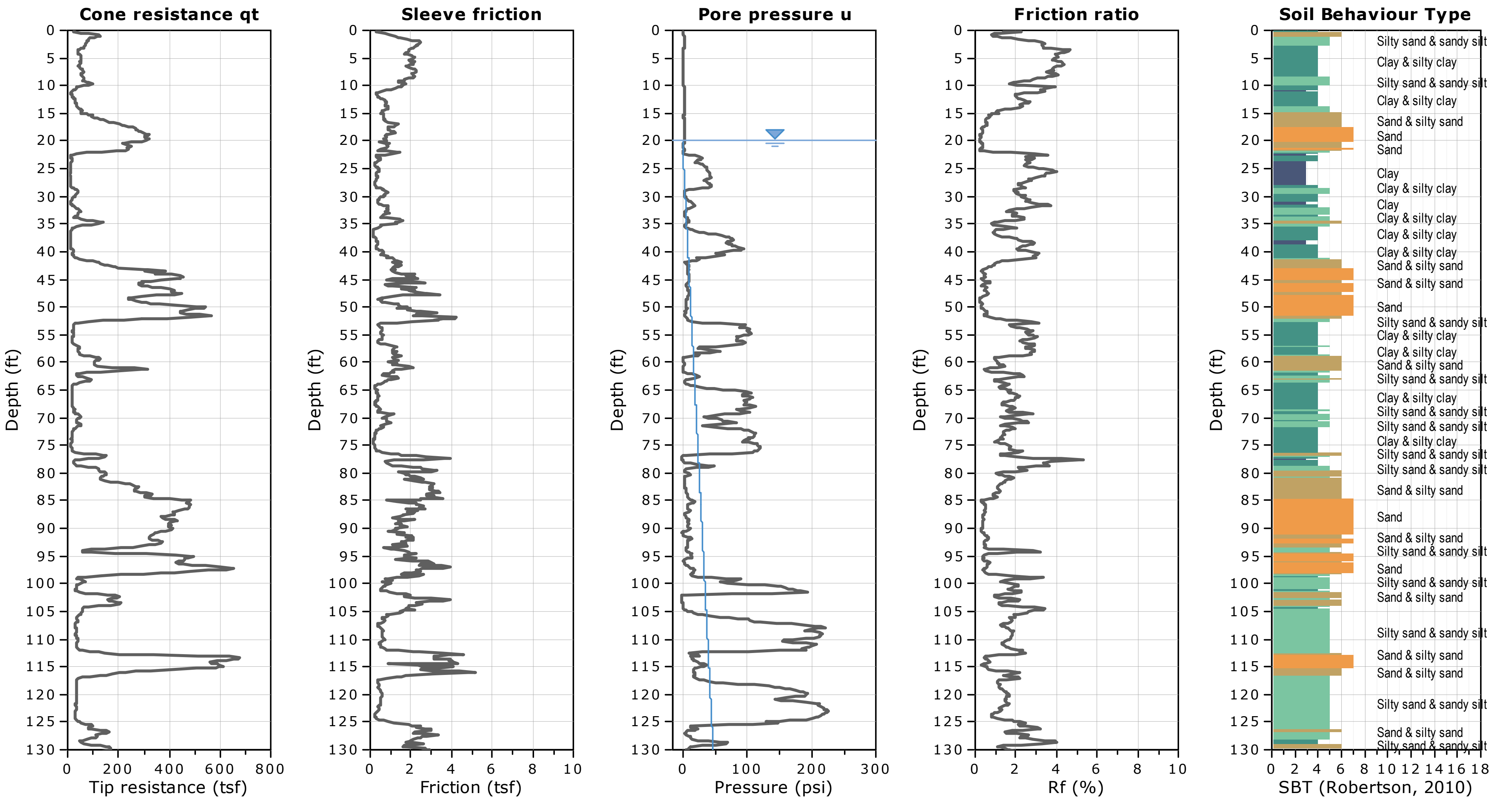


1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

Earth Systems Pacific

Boring No. 1

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

JOB NO.: 303551-001
DATE: 11/15/19

6-Story Fountain Alley Development
26-34 South 1st Street
San Jose, California

DEPTH (feet)	USCS CLASS	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
0	CL	Lean CLAY with SAND and GRAVEL; stiff, gray brown, moist [Fill]							
1.5-3.0			1.5-3.0	1-1	2.5" Mod Cal			9 8 8	4.5
3.5	SW-SC	Well-Graded SAND with CLAY and GRAVEL; medium dense, light gray, moist [Fill]							
5.0-6.5			5.0-6.5	1-2	2.5" Mod Cal	95.4	13.9	40 25 19	
8		-concrete fragments							
9.5		-color change gray brown, AC fragments							
8.5-10.0			8.5-10.0	NR	2.5" Mod Cal			11 20 50	
12	CL	SANDY Lean CLAY; stiff, gray, very moist, some oxidation staining							
13.5-15.0		[LL-35, PI=12]	13.5-15.0	1-3	2.5" Mod Cal	97.6	26.8	4 6 9	1.5
18.5-20.0			18.5-20.0	1-4	2.5" Mod Cal	92.5	23.5	7 10 11	1.25
22	CL	Lean CLAY; very stiff, blue gray, very moist, tan and gray mottling							
23.5-25.0			23.5-25.0	1-5	SPT			5 8 12	

LEGEND: ▬ 2.5" Mod Cal Sample ○ Bulk Sample ▭ Shelby Tube ● SPT ▼ Groundwater

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 1

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

JOB NO.: 303551-001
DATE: 11/15/19

6-Story Fountain Alley Development
26-34 South 1st Street
San Jose, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
26			Lean CLAY (Continued)							
27	SW-SC		Well-Graded SAND with SILTY CLAY and GRAVEL; medium dense, dark gray, wet							
28.5-30			%Fines = 6 %Sand = 69 %Gravel = 25 [LL-22, PI=4]	28.5-30.0	1-6	●			14 16 16	
33.5-35				33.5-35.0	1-7	●			16 20 23	
38	CL		SANDY Lean CLAY; hard, gray brown, wet, fine-to-coarse-grained sand, fine-grained gravel %Fines = 53 %Sand = 47	38.5-40.0	1-8	●			26 32 50/5"	
42	SW-SC		Well-Graded SAND with CLAY and GRAVEL; very dense, gray brown, wet	43.5-45.0	1-9	●			26 35 47	
47.5	SP		Poorly-Graded SAND; dense, gray brown, wet %Fines = 4 %Sand = 96	48.5-50.0	1-10	●			14 21 28	

LEGEND: 2.5" Mod Cal Sample ◯ Bulk Sample ▭ Shelby Tube SPT Groundwater

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

Boring No. 1

JOB NO.: 303551-001
DATE: 11/15/19

6-Story Fountain Alley Development
26-34 South 1st Street
San Jose, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
52	SP		Poorly-Graded SAND (Continued)							
53	SW		Well-Graded SAND; medium dense, gray brown, moist, fine-to-coarse-grained sand							
55				53.5-55.0	1-11	●			5 8 17	
60				58.5-60.0	1-12	●			14 50/5"	
65				63.5-65.0	1-13	●			47 50/5"	
66	CL		SANDY LEAN CLAY; hard, blue gray and orange brown, wet							
70				68.5-70.0	1-14	●			17 20 23	
73			-color change gray brown, highly oxidized, some gravel							
75				73.5-70.0	1-15	●			22 50/6"	

LEGEND: ■ 2.5" Mod Cal Sample ○ Bulk Sample ▭ Shelby Tube ● SPT ▼ Groundwater

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 1

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

JOB NO.: 303551-001
DATE: 11/15/19

6-Story Fountain Alley Development
26-34 South 1st Street
San Jose, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
78 - 79 - 80	CL		SANDY Lean CLAY (continued)	78.5-80.0	1-16	●			10 15 34	
80 - 81 - 82 - 83 - 84 - 85 - 86 - 87 - 88 - 89 - 90 - 91 - 92 - 93 - 94 - 95 - 96 - 97 - 98 - 99 - 100 - 101 - 102 - 103 - 104 -			Bottom of boring at 80' Groundwater was encountered at 27' below the ground surface							

LEGEND: ■ 2.5" Mod Cal Sample ○ Bulk Sample ▭ 2" Cal Sample ● SPT ▼ Groundwater

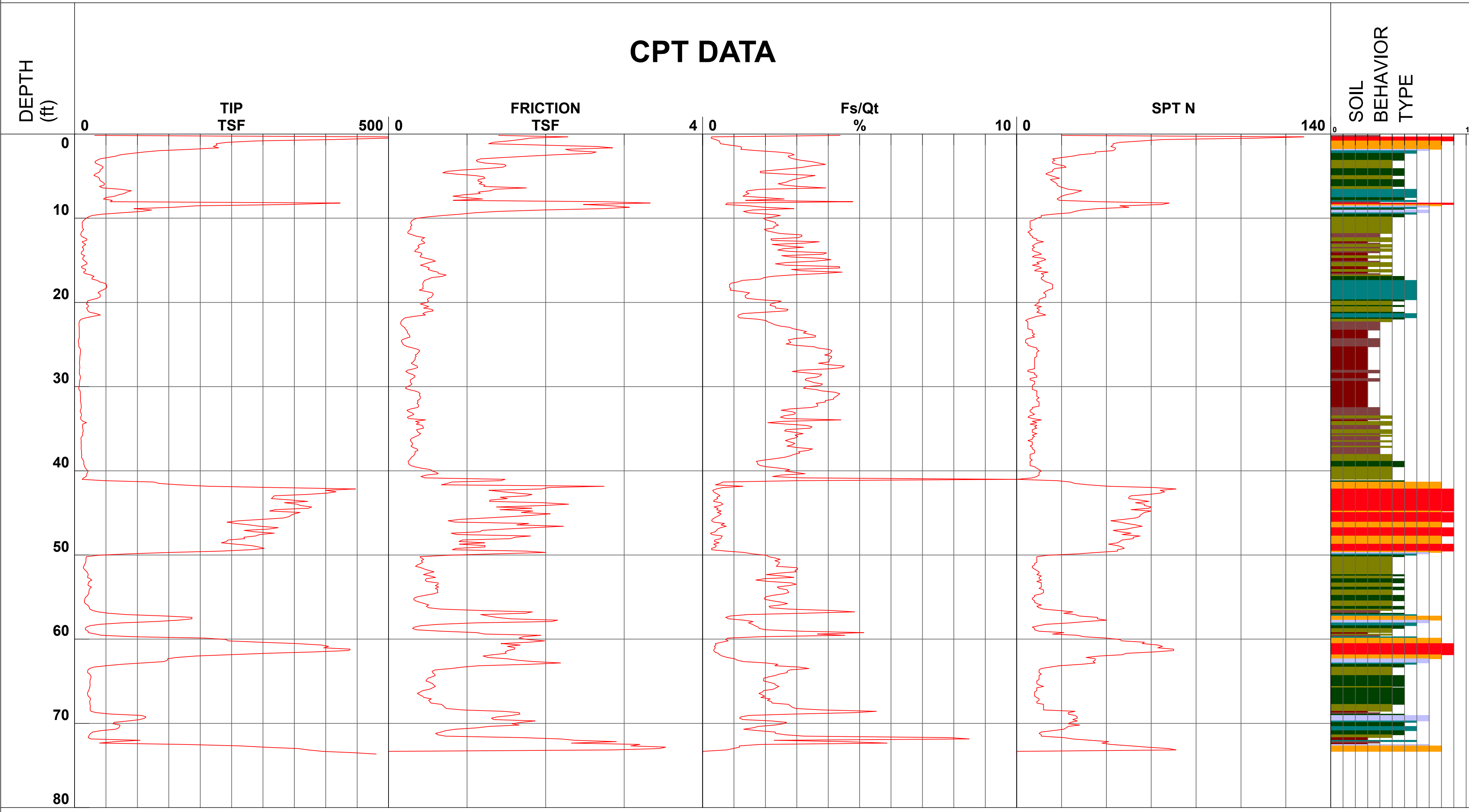
NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems

Project	Six Story Fountain Alley Development	Operator	BH-AJ	Filename	SDF(099).cpt
Job Number	303551-001	Cone Number	DDG1496	GPS	
Hole Number	CPT-01	Date and Time	11/25/2019 3:42:21 PM	Maximum Depth	73.65 ft
EST GW Depth During Test		27.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

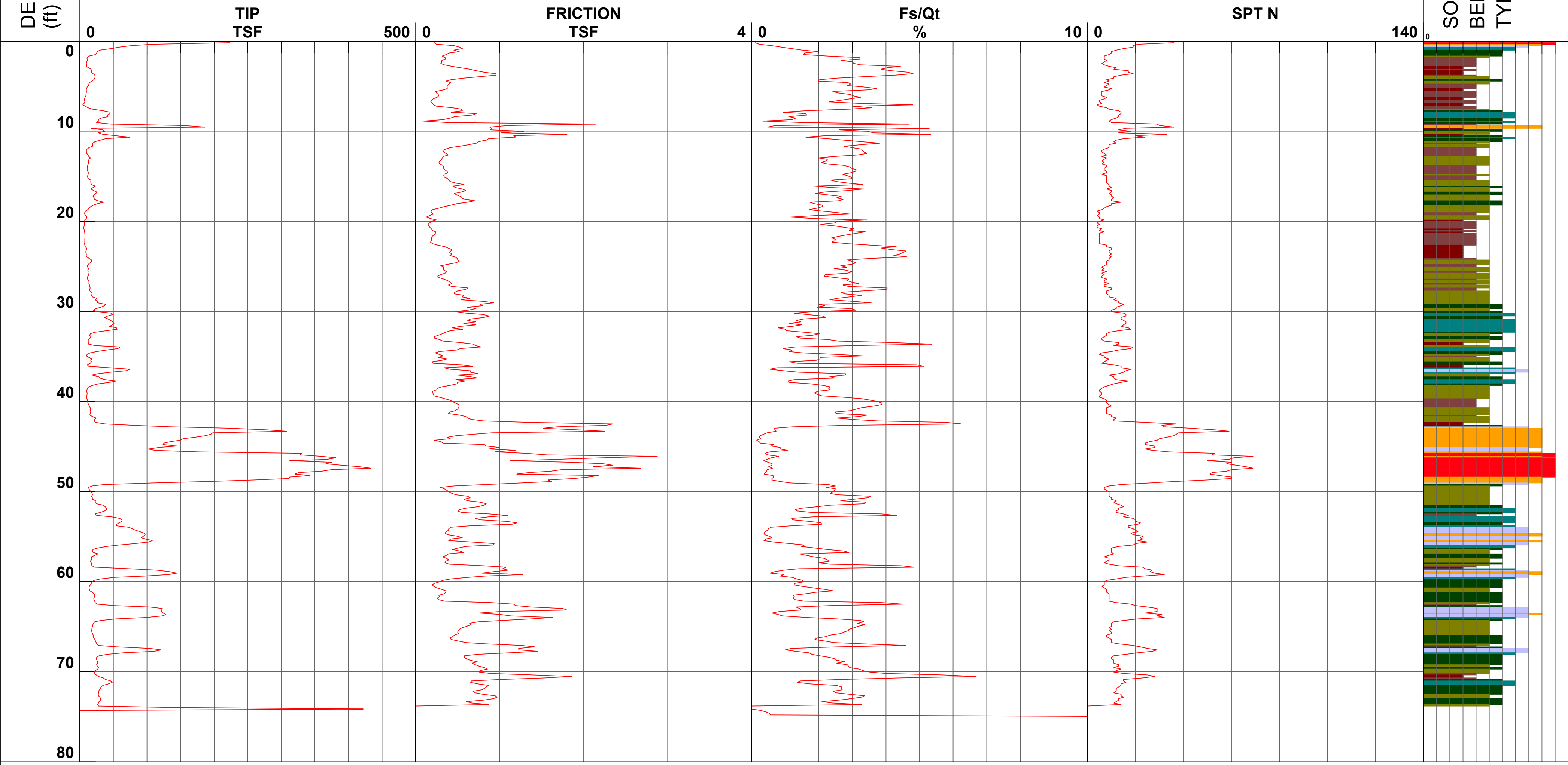
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Six Story Fountain Alley Development	Operator	BH-AJ	Filename	SDF(098).cpt
Job Number	303551-001	Cone Number	DDG1496	GPS	
Hole Number	CPT-02	Date and Time	11/25/2019 1:25:08 PM	Maximum Depth	75.29 ft
EST GW Depth During Test			27.00 ft		

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE)	CLEAN GRAVELS (Less than 5% Fines)	GW		Well graded gravels, gravel-sand mixtures, little or no fines
			GP		Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM		Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC		Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE)	CLEAN SANDS (Less than 5% Fines)	SW		Well graded sands, gravelly sands, little or no fines
			SP		Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM		Silty sands, sand-silt-mixtures, non-plastic fines
			SC		Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE)	SILTS AND CLAYS (LIQUID LIMIT IS LESS THAN 50 %)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL		Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (LIQUID LIMIT IS GREATER THAN 50 %)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH		Inorganic clays of high plasticity, fat clays
			OH		Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT		Peat and other highly organic soils

DEFINITION OF TERMS

	U.S. STANDARD SIEVE SIZE: 200	40	10	4	CLEAR SQUARE SIEVE OPENINGS: 3/4"	3"	12"	
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			
	0.08	0.4	2	5	19	76mm		

GRAIN SIZES

TERZAGHI SPLIT SPOON STANDARD PENETRATION

MODIFIED CALIFORNIA

D&M UNDERWATER SAMPLER

SHELBY TUBE

NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE A-1

EXPLORATORY BORING: EB-9

DRILL RIG: FAILING 750	PROJECT NO: 1723-2
BORING TYPE: ROTARY WASH	PROJECT: CIM MIXED-USE PROJECTS- LOT D
LOGGED BY: BAH	LOCATION: SAN JOSE, CA
START DATE: 1-15-03 FINISH DATE: 1-15-03	COMPLETION DEPTH: 50.0 FT.

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Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression. Scale: 1.0, 2.0, 3.0, 4.0

SURFACE ELEVATION:

DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH (ksf)
0	6" asphalt concrete						
	SANDY LEAN CLAY WITH GRAVEL (CL) [FILL] hard, moist, brown with olive brown mottles, fine to coarse sand, fine gravel, low plasticity	CL, FILL	29	17	102		○ >4.0
5	**SANDY LEAN CLAY (CL)** very stiff, moist, brown with gray mottles, fine sand, low plasticity	CL	32	19	101		○ ~2.5
10	**SILTY CLAY (CL-ML)** very stiff, moist, brown with gray mottles, some fine sand, low plasticity	CL-ML	15	23	100		○ ~2.0
15	**SILT WITH SAND (ML)** very stiff, moist, dark brown with gray mottles, fine sand, some clay, low plasticity	ML	15	24	97	72	○ ~2.25
20	**SILTY SAND (SM)** medium dense, moist, brown with gray mottles, fine sand	SM	22	19			
	POORLY GRADED SAND WITH CLAY (SP-SC) medium dense, moist, gray and brown, fine to medium sand, some coarse sand, some fine gravel	SP-SC	19	12		9	
25	increasing gravel		10	18			
	CLAYEY SAND (SC) medium dense, moist, brown, fine sand	SC				22	
	SANDY LEAN CLAY (CL) medium stiff, moist, bluish gray, fine sand, low plasticity	CL	280psi				○ ~0.5
30							

Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

LA CORP.GDT 2/19/03 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-9 Cont'd

DRILL RIG: FAILING 750	PROJECT NO: 1723-2
BORING TYPE: ROTARY WASH	PROJECT: CIM MIXED-USE PROJECTS- LOT D
LOGGED BY: BAH	LOCATION: SAN JOSE, CA
START DATE: 1-15-03 FINISH DATE: 1-15-03	COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

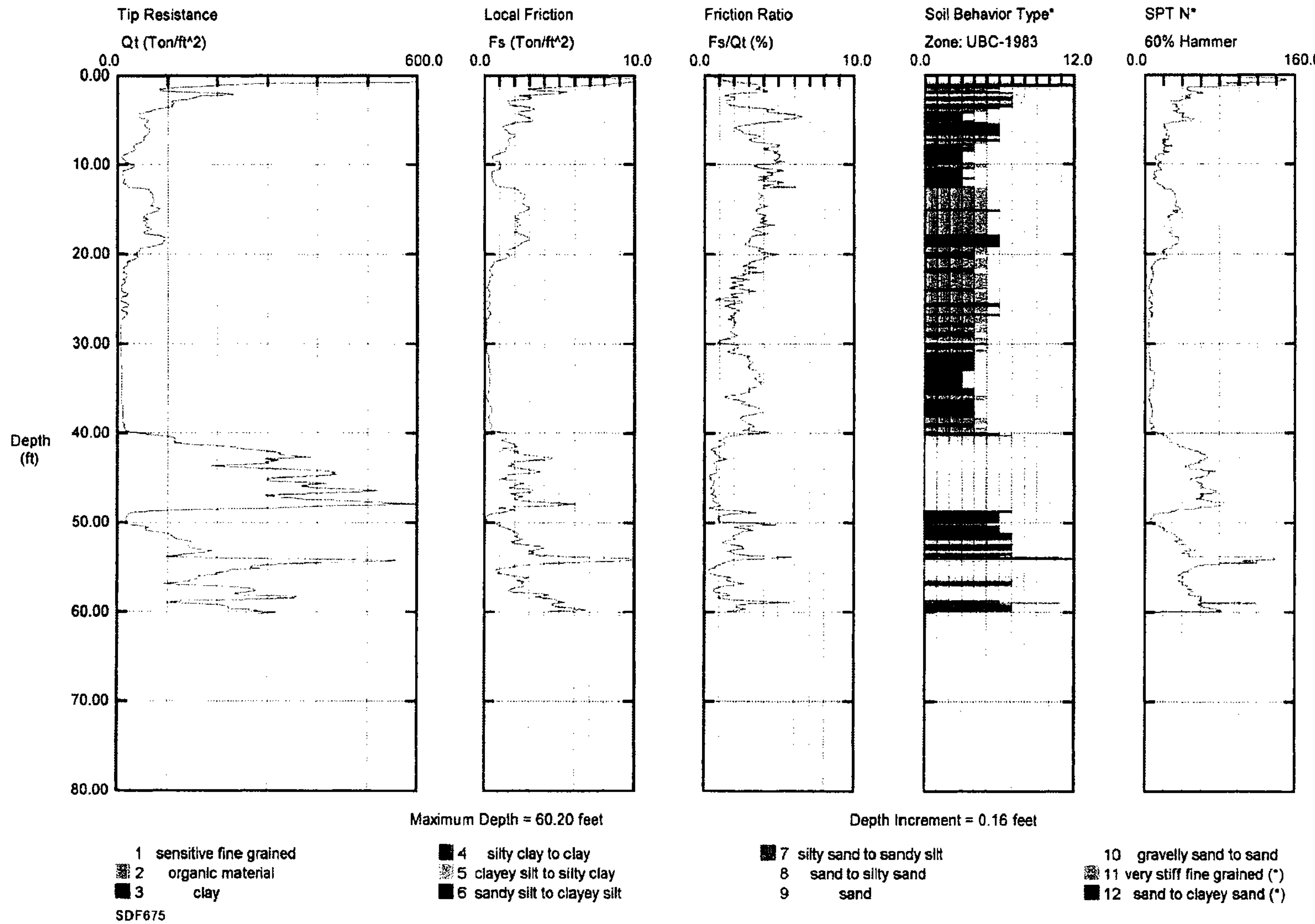
Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30	**SANDY LEAN CLAY (CL)** medium stiff, moist, bluish gray, fine sand, low plasticity	CL					
	~34.5			11	29	97		○ (~0.5)
	~37	**LEAN CLAY WITH SAND (CL)** stiff, moist, bluish gray, fine sand, low plasticity	CL					
	~39.5			17	44	80		○ (~1.75)
	~43	color grades to brown						
	~44	**POORLY GRADED SAND WITH SILT (SP-SM)** dense, moist, brown with gray, fine to medium sand, some coarse sand, some fine gravel, trace coarse gravel	SP-SM	57	20		11	
	~49.5			33	8		8	
	50	Bottom of Boring at 50 feet						

Depth scale: 30, 35, 40, 45, 50, 55, 60

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

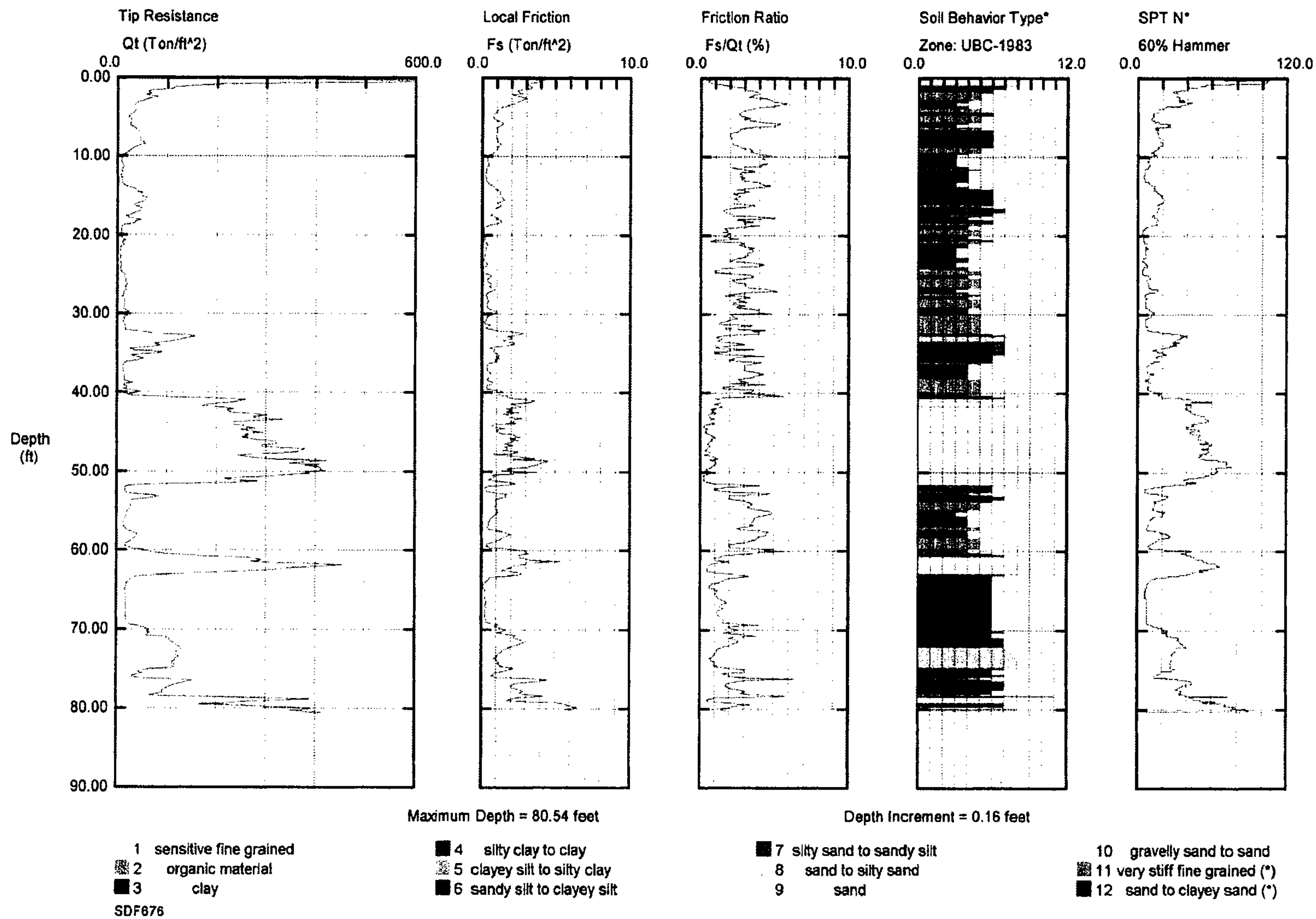


2/03*EB

CONE PENETRATION TEST - CPT-8

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

CPT-8
1723-2



2/03*EB

CONE PENETRATION TEST - CPT-10

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

CPT-10
1723-2

APPENDIX F

LABORATORY DATA FROM PREVIOUS INVESTIGATIONS

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 40 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 32 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

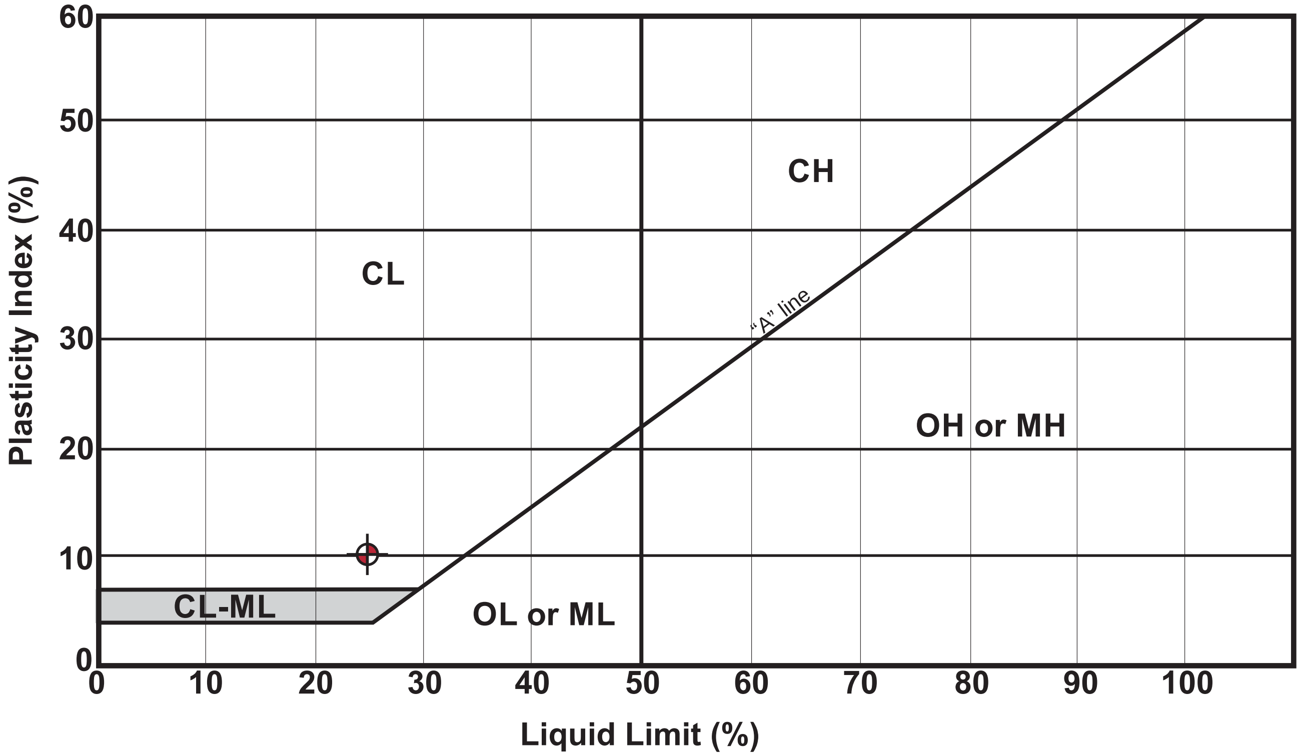
Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on five samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Two Plasticity Index determination (ASTM D4318) were performed on two samples of the subsurface soil to measure the range of water contents over which the material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

Undrained-Unconsolidated Triaxial Shear Strength: The undrained shear strength was determined on two relatively undisturbed samples by unconsolidated-undrained triaxial shear strength testing (ASTM D2850). The results of these tests are included as part of this appendix.

Consolidation: Two consolidation tests (ASTM D2435) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation tests are presented graphically in this appendix.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⌖	EB-1	2.0	13	25	15	10	—	Lean Clay with Sand (CL)
	EB-1	34.5	26	determined non-plastic			---	Sandy Silt (ML)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

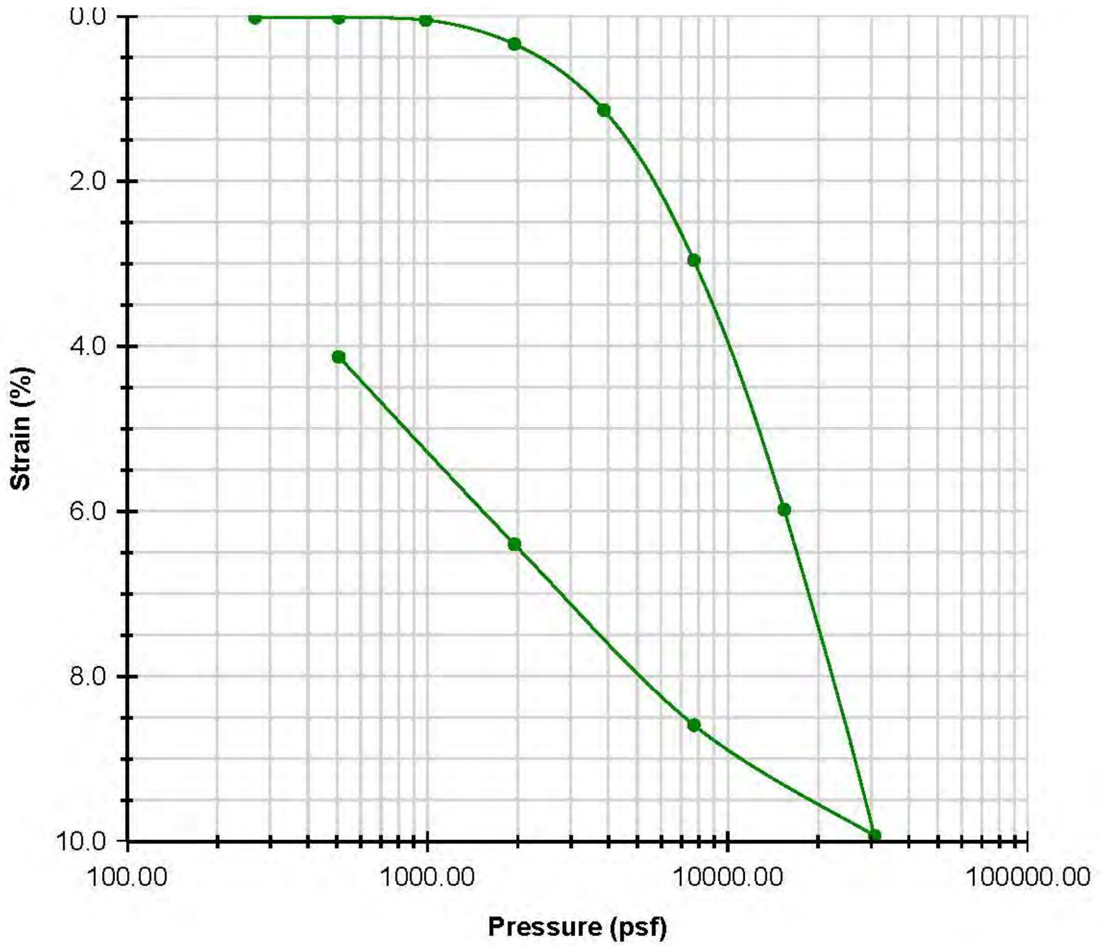
Fountain Alley Tower
35 South 2nd Street
San Jose, CA

Project Number	1178-3-2
Figure Number	Figure B1
Date	April 2020
Drawn By	FLL

Consolidation Test ASTM D2435

Boring: EB-1 **Sample:** 8 **Depth:** 25.7’

Description: Lean Clay with Sand (CL)



	BEFORE	AFTER
Moisture (%)	20.2	18.9
Dry Density (pcf)	107.6	112.2
Saturation (%)	95.1	100.0
Void Ratio	0.58	0.51

CORNERSTONE EARTH GROUP

Strain-Log Curve - EB-1 @ 25.7’

Fountain Alley Tower
35 South 2nd Street
San Jose, CA

Project Number 1178-3-2

Figure Number Figure B2

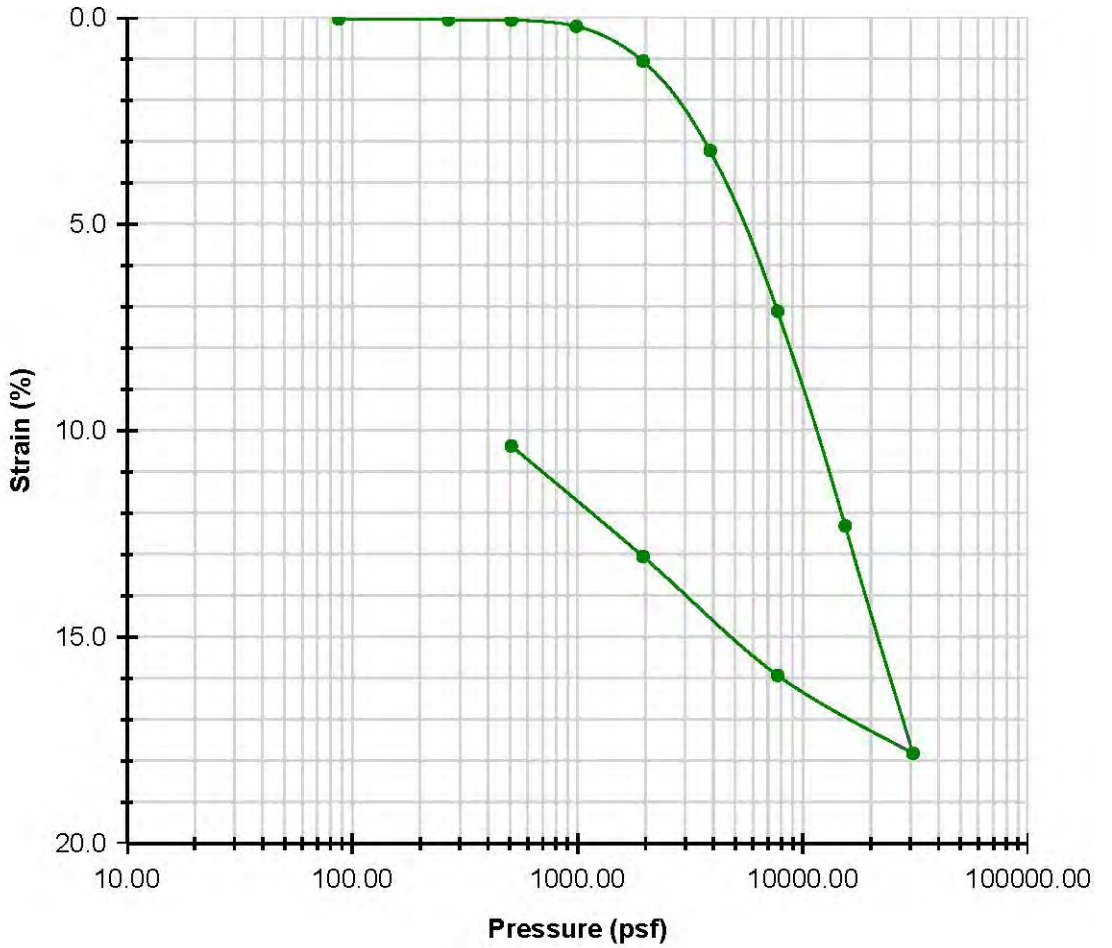
Date March 2020

Drawn By FLL

Consolidation Test ASTM D2435

Boring: EB-1 Sample: 13 Depth: 40.0'

Description: Lean Clay (CL)



	BEFORE	AFTER
Moisture (%)	37.2	34.3
Dry Density (pcf)	82.4	87.8
Saturation (%)	95.4	100.0
Void Ratio	1.06	0.93

CORNERSTONE EARTH GROUP

Strain-Log Curve - EB-1 @ 40.0'

Fountain Alley Tower
35 South 2nd Street
San Jose, CA

Project Number 1178-3-2

Figure Number Figure B3

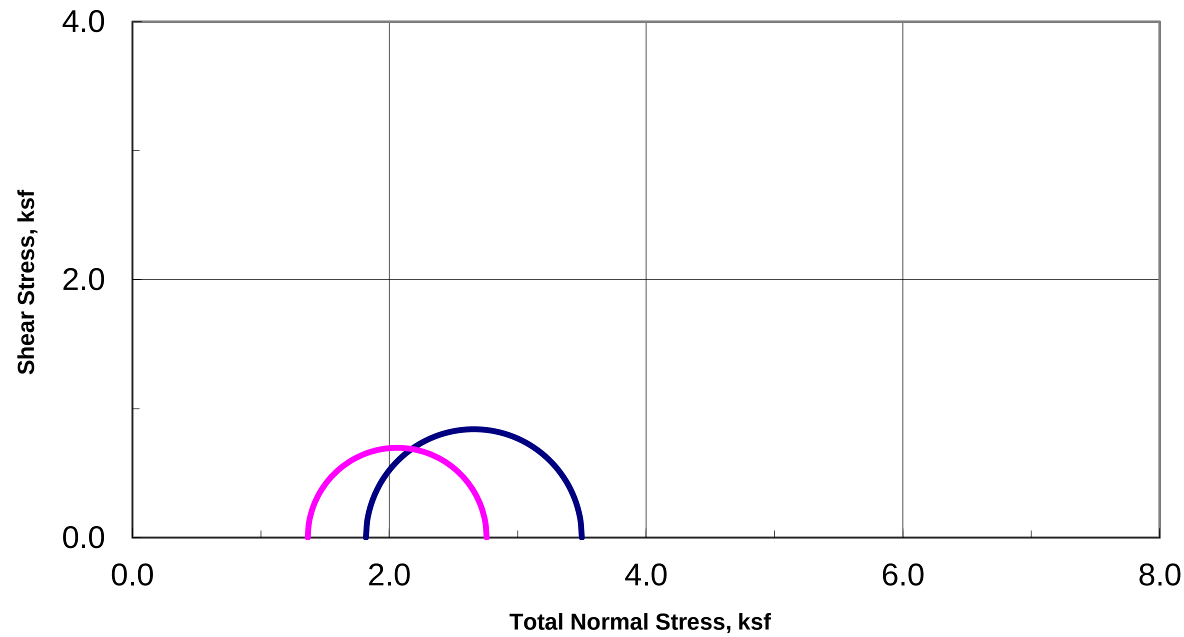
Date March 2020

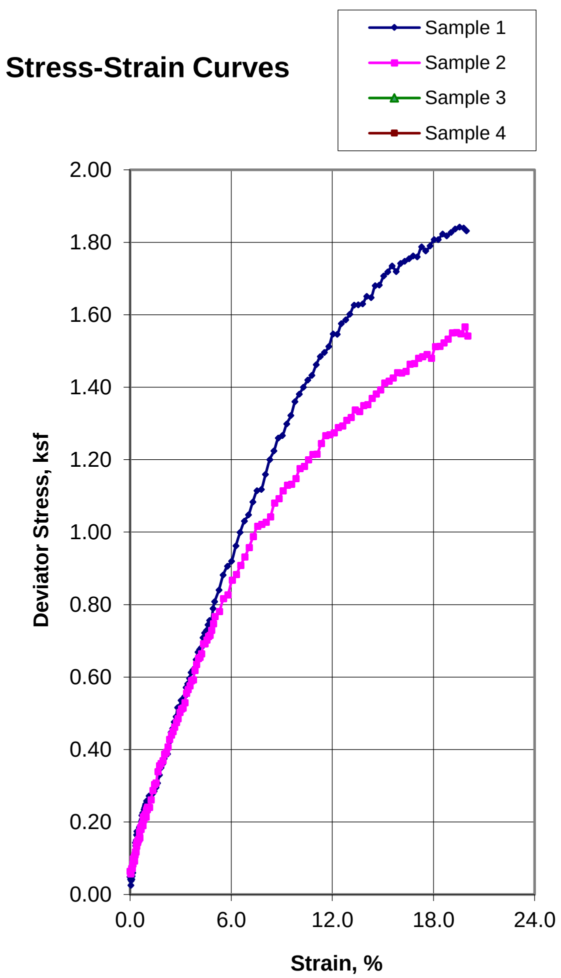
Drawn By FLL

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 %	23.6	22.2		
Dry Den,pcf	102.7	104.7		
Void Ratio	0.642	0.610		
Saturation %	99.3	98.1		
Height in	5.01	5.00		
Diameter in	2.41	2.41		
Cell psi	12.6	9.5		
Strain %	15.00	15.00		
Deviator, ksf	1.682	1.394		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	640-1385			
Client:	Cornerstone Earth Group			
Project:	1178-3-2			
Boring:	EB-1	EB-2		
Sample:	9B	8B		
Depth ft:	29.5	25.0		

Visual Soil Description

Sample #	
1	Greenish Gray Sandy CLAY
2	Olive Brown Sandy CLAY
3	
4	

Remarks:

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

Six-Story Fountain Alley Development 303551-001

BULK DENSITY TEST RESULTS

ASTM D 2937-17 (modified for ring liners)

November 27, 2019

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
B1	6.0 - 6.5	13.9	108.6	95.4
B1	14.5 - 15.0	26.8	123.8	97.6
B1	19.0 - 19.5	23.5	114.2	92.5

Earth Systems Pacific

Six-Story Fountain Alley Development 303551-001

PARTICLE SIZE ANALYSIS

ASTM D 422-63/07; D 1140-17

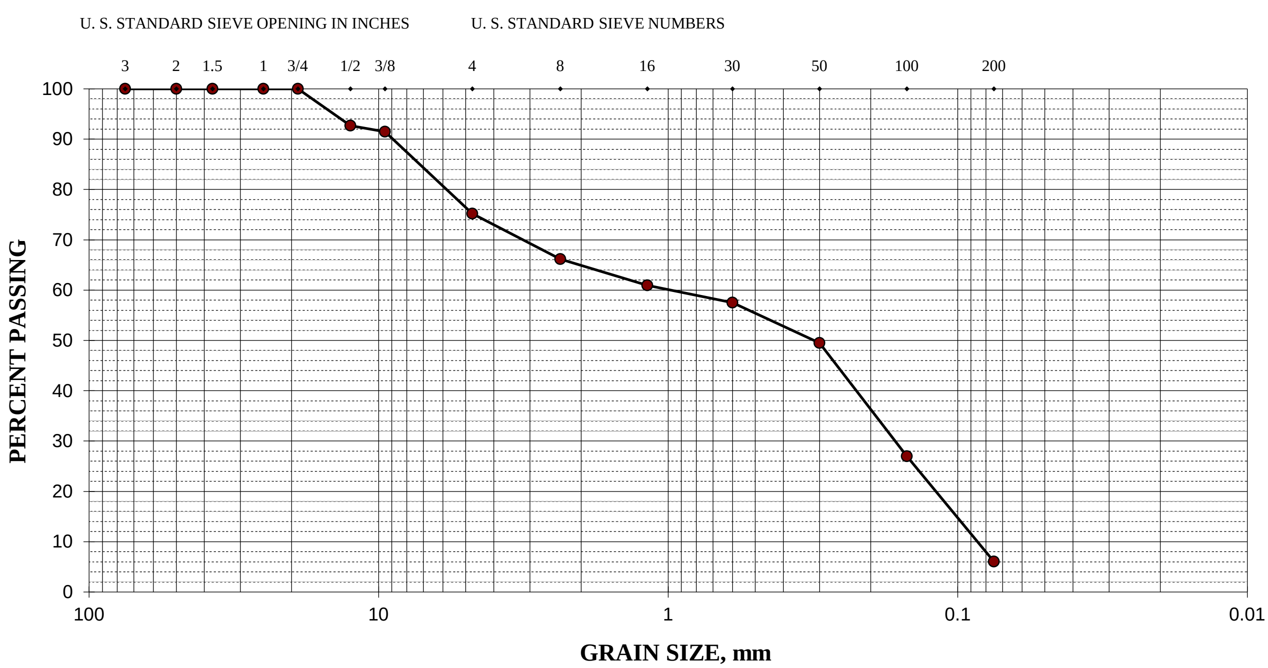
Boring #B1 @ 28.5 - 30.0' November 27, 2019

Dark Gray Brown Poorly Graded Sand with Silty Clay (SP-SC)

Cu = 11.5; Cc = 0.3

LL = 22; PL = 18; PI = 4

Sieve size	% Retained	% Passing
3" (75-mm)	0	100
2" (50-mm)	0	100
1.5" (37.5-mm)	0	100
1" (25-mm)	0	100
3/4" (19-mm)	0	100
1/2" (12.5-mm)	7	93
3/8" (9.5-mm)	9	91
#4 (4.75-mm)	25	75
#8 (2.36-mm)	34	66
#16 (1.18-mm)	39	61
#30 (600-µm)	43	57
#50 (300-µm)	51	49
#100 (150-µm)	73	27
#200 (75-µm)	94	6



Earth Systems Pacific

Six-Story Fountain Alley Development 303551-001

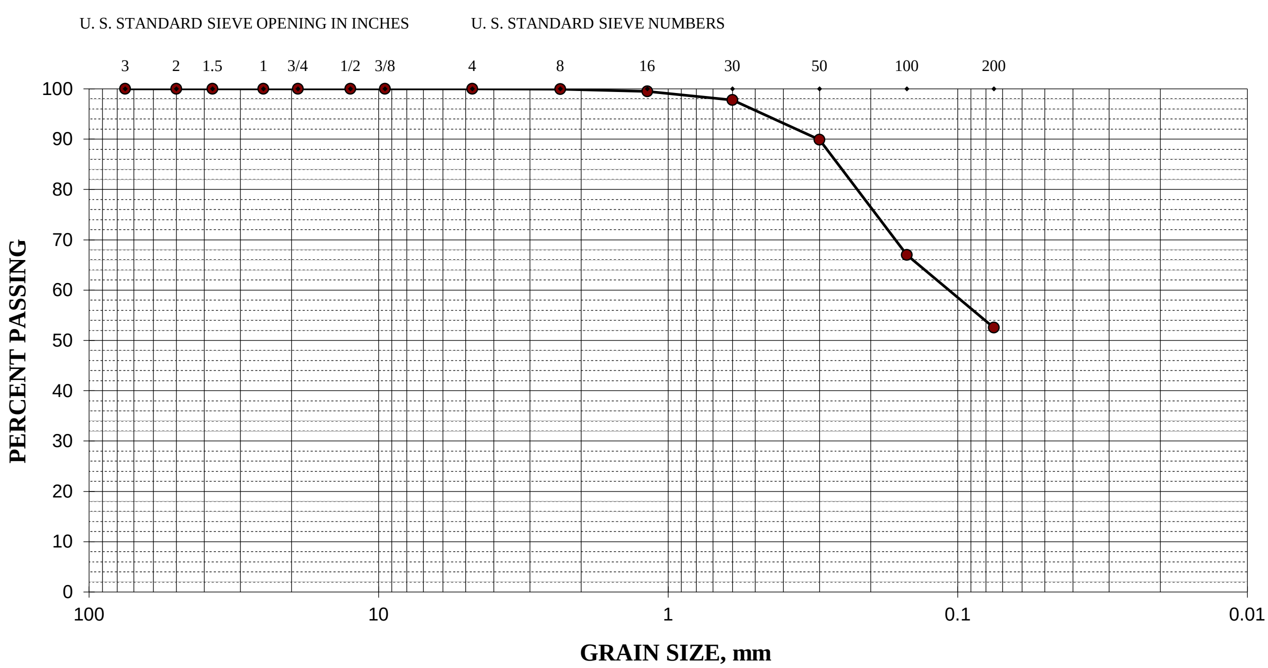
PARTICLE SIZE ANALYSIS

ASTM D 422-63/07; D 1140-14

Boring #B1 @ 38.5 - 40.0' November 27, 2019

Gray Brown Sandy Lean Clay (CL)

Sieve size	% Retained	% Passing
3" (75-mm)	0	100
2" (50-mm)	0	100
1.5" (37.5-mm)	0	100
1" (25-mm)	0	100
3/4" (19-mm)	0	100
1/2" (12.5-mm)	0	100
3/8" (9.5-mm)	0	100
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	2	98
#50 (300-µm)	10	90
#100 (150-µm)	33	67
#200 (75-µm)	47	53



Earth Systems Pacific

Six-Story Fountain Alley Development 303551-001

PARTICLE SIZE ANALYSIS

ASTM D 422-63/07; D 1140-14

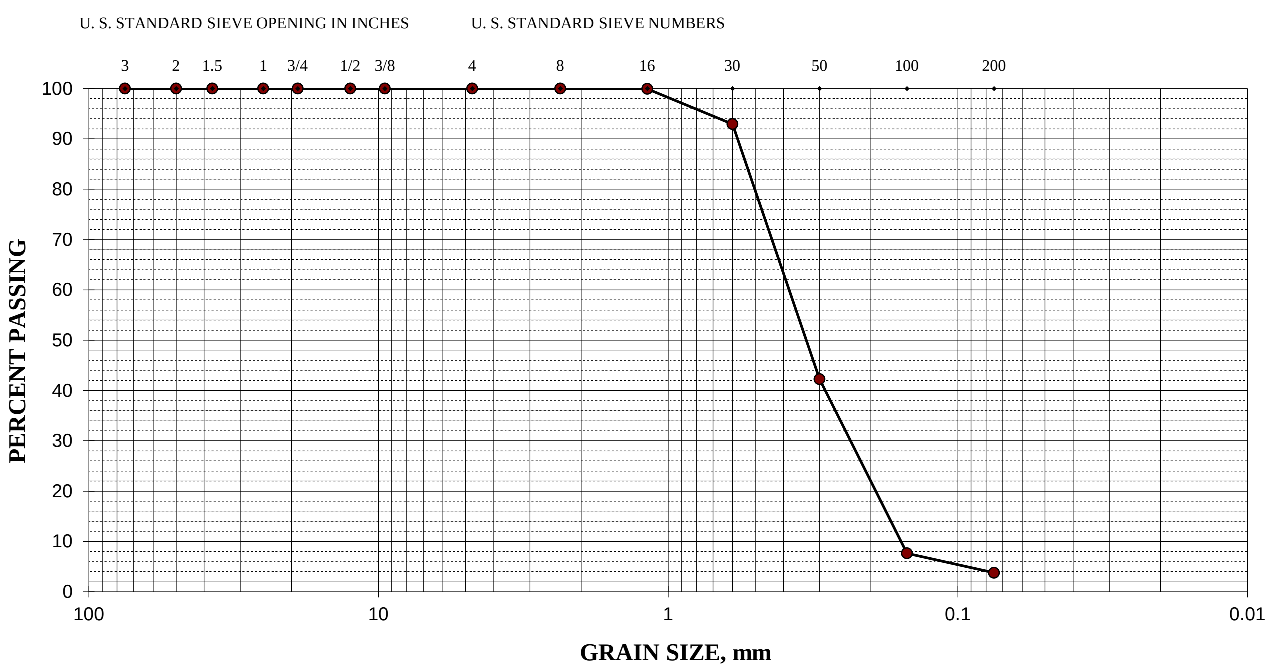
Boring #B1 @ 48.5 - 50.0'

November 27, 2019

Poorly Graded Sand (SP)

Cu = 2.4; Cc = 0.9

Sieve size	% Retained	% Passing
3" (75-mm)	0	100
2" (50-mm)	0	100
1.5" (37.5-mm)	0	100
1" (25-mm)	0	100
3/4" (19-mm)	0	100
1/2" (12.5-mm)	0	100
3/8" (9.5-mm)	0	100
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	7	93
#50 (300-µm)	58	42
#100 (150-µm)	92	8
#200 (75-µm)	96	4



Earth Systems Pacific

Six-Story Fountain Alley Development 303551-001

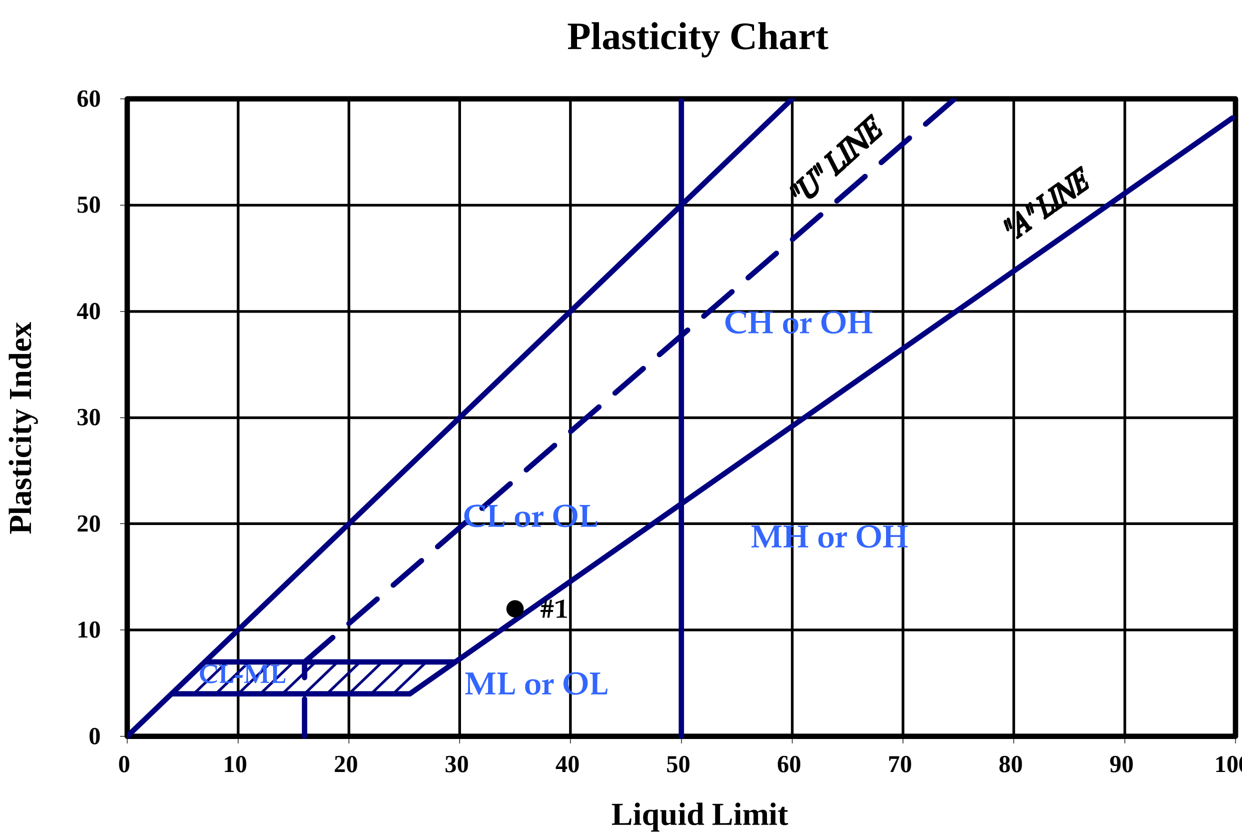
PLASTICITY INDEX

ASTM D 4318-17

Gray Brown Lean Clay (CL)

November 27, 2019

Test No.:	1	2	3	4	5
Boring No.:	B1				
Sample Depth:	14.5 - 15.0'				
Liquid Limit:	35				
Plastic Limit:	23				
Plasticity Index:	12				



Earth Systems Pacific

Six-Story Fountain Alley Development | 303551-001

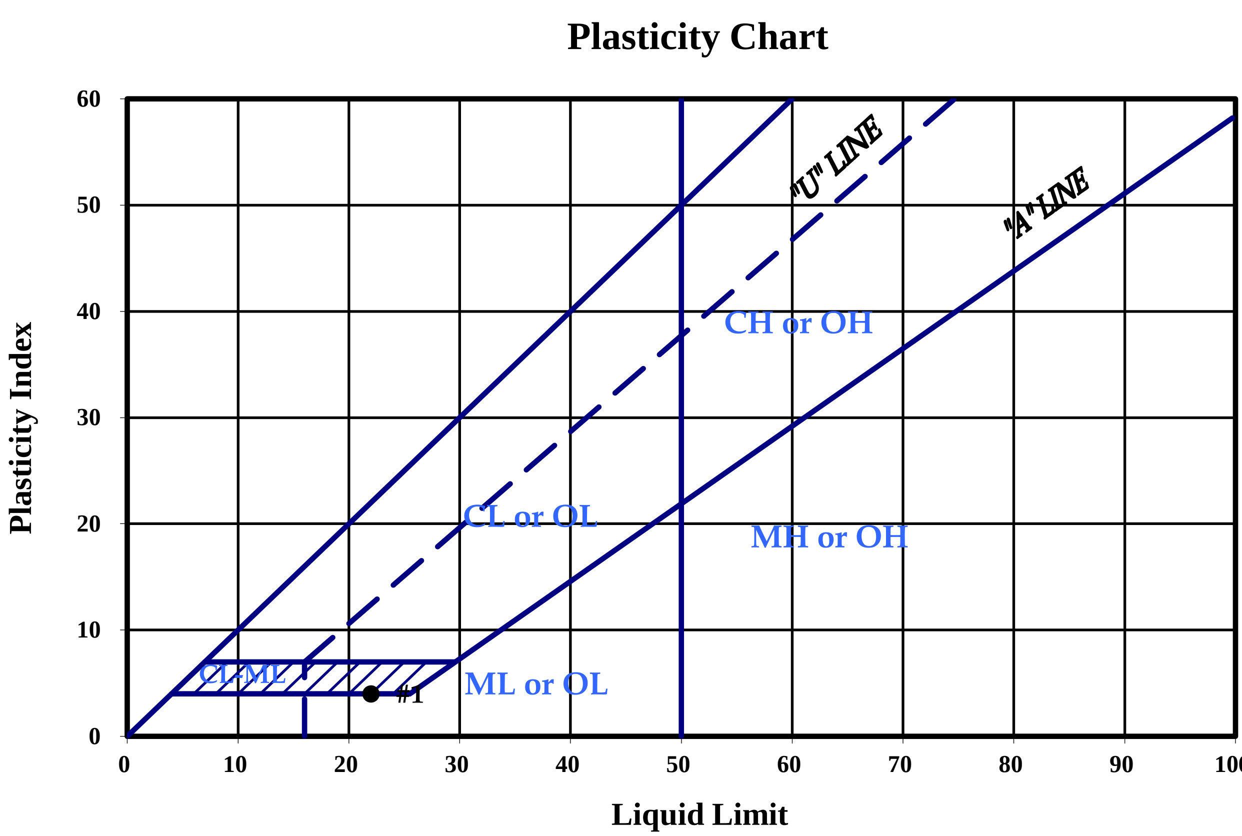
PLASTICITY INDEX

ASTM D 4318-17

Dark Gray Brown Poorly Graded Sand with Silty Clay (SP-SC)

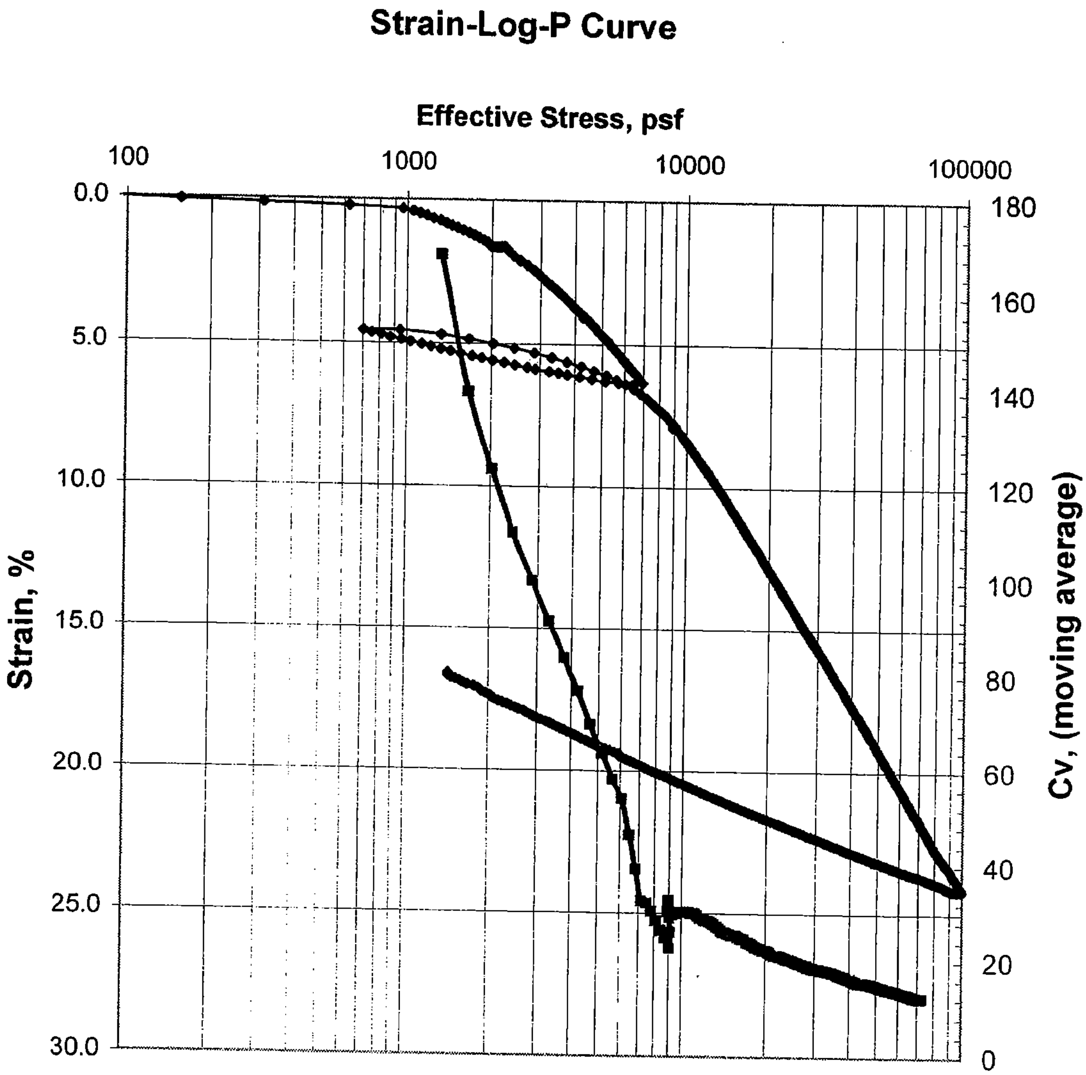
November 27, 2019

Test No.:	1	2	3	4	5
Boring No.:	B1				
Sample Depth:	28.5 - 30.0'				
Liquid Limit:	22				
Plastic Limit:	18				
Plasticity Index:	4				



Boring:	EB-9	By:	DC
Sample:	8A	Date:	2/14/2003
Depth:	27.5'		

Soil Type: gray CLAY w/sand



Ass. Gs = 2.77	Initial	Final
Moisture %:	29.4	20.8
Density, pcf:	94.2	109.8
Void Ratio:	0.835	0.575
% Saturation:	97.6	100.1
Initial Back Press., psi	50.1	

Remarks:

Max Pore Pressure Ratio	29	PI =

CONSOLIDATION TEST

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE B-1
1723-2

APPENDIX G

SITE-SPECIFIC RESPONSE SPECTRA

APPENDIX G
SITE-SPECIFIC RESPONSE SPECTRA

This appendix presents the details of our estimation of the level of ground shaking at the site during future earthquakes. Because performance based design will be used for the project, the seismic design will be performed per the PEER Tall Building Initiative (TBI) version 2.03 and the 2019 California Building Code and by reference ASCE 7-16. To develop site-specific response spectra in accordance with codes, we performed probabilistic seismic hazard analysis (PSHA) and deterministic analysis to develop smooth, site-specific horizontal spectra for three levels of shaking, namely:

- Risk-Targeted Maximum Considered Earthquake (MCE_R), which corresponds to the lesser of two percent probability of exceedance in 50 years (2,475-year return period) or 84th percentile of the controlling deterministic event both considering the maximum direction as described in ASCE 7-16.

- Design Earthquake (DE), which corresponds to 2/3 of the MCE_R.

- Service Level Earthquake (SLE), which corresponds to 50 percent probability of exceedance in 30 years (43-year return period).

G1.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a PSHA, which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

To perform a PSHA, information regarding the seismicity, location, and geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source, are needed. The assumptions necessary to perform the PSHA are that:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data

- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake

- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate.

As part of the development of the site-specific spectra, we performed a PSHA to develop a site-specific response spectrum for 2 and 50 percent probability of exceedance in 50 and

30 years, respectively. The spectra were developed using the OpenSHA platform. The approach used in PSHA is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources, and earthquake activities were assigned to the faults based on historical and geologic data. The levels of shaking were estimated using ground motion prediction equations (attenuation relationships) that are primarily dependent upon the magnitude of the earthquake and the distance from the site to the fault, as well as the average shear wave velocity of the upper 30 meters, V_{S30}.

G1.1 Probabilistic Model

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. Fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

The probability of exceedance, $P_e(Z)$, at a given ground-motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

$$V(z) = \sum_i v_i \iint P[Z > z \mid m, r] f_{M_i}(m) f_{R_i|M_i}(r; m)\, dr\, dm$$

where:

v_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source i

P [Z > z | m,r] = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z

f_{Mi} (m) and $f_{Ri|Mi}$ (r;m) = probability density functions for magnitude and distance

Z represents peak ground acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

G1.2 Source Modeling and Characterization

The segmentation of faults, maximum magnitudes, and recurrence rates were modeled using the data presented in the Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165. We also included the combination of fault segments and their associated magnitudes and recurrence rates as described in the WGCEP (2014) in our seismic hazard model. These and other faults of the region are shown on Figure 5. Table G-1 presents the distance and direction from the site to the fault, mean moment magnitude, mean slip rate, and fault length for individual fault segments in UCERF3 source model. The mean moment magnitude presented on Table G-1 was computed assuming full rupture of the segment using Hank and Bakun (2008) relationship.

TABLE G-1
Source Zone Parameters

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Moment Magnitude[1]	Mean Slip Rate (mm/yr)	Fault Length (km)
Silver Creek 2011 CFM	2.0	Northeast	6.7	0.1	48
Hayward (So) extension 2011 CFM	9.7	East	6.1	4.3	23
Total Hayward-Rodgers Creek Healdsburg	9.7	East	7.3	7.3	213
Hayward (So) 2011 CFM	10.3	Northeast	6.9	9.8	54
Monte Vista - Shannon 2011 CFM	11.8	Southwest	7.0	0.8	60
Calaveras (Central) 2011 CFM	12.1	East	6.7	10.2	52
Total Calavares	12.1	East	7.5	8.0	186
Calaveras (No) 2011 CFM	14.9	Northeast	6.8	4.8	48
Mission (connected) 2011 CFM	14.9	Northeast	6.1	0.8	28
San Andreas (Peninsula) 2011 CFM	19.3	Southwest	7.2	15.1	100
San Andreas 1906 event	19.3	Southwest	8.1	17.2	464
San Andreas (Santa Cruz Mts) 2011 CFM	20.5	Southwest	7.0	18.6	63
Sargent 2011 CFM	22.5	South	6.8	1.7	57
Butano 2011 CFM	22.5	Southwest	6.7	0.7	46
Pilarcitos 2011 CFM	24.3	West	6.7	0.7	51
Zayante-Vergeles 2011 CFM	27.6	Southwest	7.1	0.1	90
Zayante-Vergeles	28.5	Southwest	6.9	0.1	58
Las Positas	31.0	North	6.3	0.4	15
Greenville (So) 2011 CFM	34.7	East	6.5	1.8	29
Greenville (No) 2011 CFM	36.2	East	6.9	2.6	51
San Gregorio (North) 2011 CFM	42.3	West	7.3	4.6	129

1 Mean Moment Magnitude based on entire fault length rupturing using Hank and Bakun (2008)

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Moment Magnitude[1]	Mean Slip Rate (mm/yr)	Fault Length (km)
Mount Diablo Thrust South	44.2	North	6.2	1.5	11
Mount Diablo Thrust	44.3	North	6.6	1.6	25
Calaveras (So) 2011 CFM	45.3	Southeast	6.4	11.6	26
Mount Diablo Thrust North CFM	48.7	North	6.4	1.8	19
Reliz 2011 CFM	49.4	Southwest	7.3	0.3	127
Monterey Bay-Tularcitos	50.6	Southwest	7.2	0.6	86
Ortigalita (North)	52.2	East	6.6	1.8	40
Great Valley 07 (Orestimba)	53.1	Northeast	6.8	0.5	66
Hayward (No) 2011 CFM	55.9	Northwest	6.8	8.3	53
Franklin 2011 CFM	58.4	North	6.7	1.1	38
Clayton	58.6	North	6.4	0.7	16
San Gregorio (South) 2011 CFM	58.8	Southwest	7.1	2.1	90
Contra Costa (Lafayette) 2011 CFM	60.2	North	6.1	0.8	8
Contra Costa (Larkey) 2011 CFM	60.3	North	6.0	0.8	8
Great Valley 06 (Midland) 2011 CFM alt1	60.6	Northeast	7.1	0.3	69
Great Valley 06 Midland alt2	63.1	Northeast	6.7	0.3	33
Concord 2011 CFM	63.5	North	6.4	3.4	18
Quien Sabe 2011 CFM	63.5	Southeast	6.4	0.9	25
Contra Costa (Reliez Valley) 2011 CFM	63.7	North	5.9	0.2	6
Contra Costa Shear Zone (connector) 2011 CFM	66.6	North	6.6	0.9	30
Contra Costa (Briones) 2011 CFM	69.4	North	6.0	0.4	9
San Andreas (Creeping Section) 2011 CFM	69.6	Southeast	7.3	18.7	121
Contra Costa (Southampton) 2011 CFM	69.7	North	6.2	0.1	11
Calaveras (So) - Paicines extension 2011 CFM	70.5	Southeast	6.9	7.1	60
Ortigalita (South)	70.8	East	6.9	1.2	62
Great Valley 08 (Quinto)	71.1	East	6.0	0.3	19
Los Medanos - Roe Island	71.4	North	6.4	0.2	21
Point Reyes 2011 connector	73.1	West	6.5	0.1	34
Great Valley 05 Pittsburg Kirby Hills alt2	75.9	North	6.8	1.0	32
Great Valley 05 Pittsburg - Kirby Hills alt1	78.0	North	6.3	1.0	21
Contra Costa (Dillon Point) 2011 CFM	79.3	North	6.1	0.7	11
Contra Costa (Ozal - Columbus) 2011 CFM	80.4	North	6.1	0.4	9
Green Valley 2011 CFM	80.8	North	6.8	3.8	43
Great Valley 09 (Laguna Seca)	84.1	East	6.6	1.6	39
Contra Costa (Vallejo) 2011 CFM	90.7	North	5.6	0.6	4
Contra Costa (Lake Chabot) 2011 CFM	91.4	North	5.6	0.7	4

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Moment Magnitude[1]	Mean Slip Rate (mm/yr)	Fault Length (km)
San Andreas (North Coast) 2011 CFM	92.6	Northwest	7.4	18.0	171
West Napa 2011 CFM	97.4	North	6.8	1.3	44

G1.3 Attenuation Relationships

Based on the subsurface conditions, the site is classified as a stiff soil profile, Site Class D. Using the subsurface information including shear wave velocity measurements, we estimate the shear wave velocity of the upper 100 feet (30 meters), V_{S30}, is approximately 918 feet per second (i.e. 280 meters per second); the value of V_{S30} was estimated based on the shear wave velocity measurements. Furthermore, based on a review of the NGA West2 database, depths Z_1 and $Z_{2.5}$ are about 572 meters and 0.87 kilometer, respectively. These values were used in the development of site-specific spectra.

The Pacific Earthquake Engineering Research Center (PEER) embarked on the NGA-West2 project to update the previously developed ground motion prediction equations (attenuation relationships), which were mostly published in 2014. We used the relationships by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014). The value of V_{S30} is used in these attenuation relationships include the average shear wave velocity in the upper 100 feet. Furthermore, these relationships were developed from the same earthquake database, therefore, each one is equally credible and the average of the relationships (using equal weights for each attenuation relationship) was used to develop the recommended spectra.

The NGA relationships database includes the most up-to-date recorded and processed data. They were developed for the orientation-independent geometric mean of the data. Geometric mean is defined as the square root of the product of the two recorded components.

G2.0 PSHA RESULTS

Figure G-1 presents the geometric mean results of the PSHA for the 2 percent probability of exceedance in 50 years hazard level (2,475-year return period) using the four relationships discussed above as well as the average of these relationships. These results were developed using OpenSHA Hazard Spectrum Application, Version 1.5.2 (UCERF3 model).

ASCE 7-16 specifies the development of MCE_R site-specific response spectra in the maximum direction. Shahi and Baker (2014) provide scaling factors that modify the geometric mean spectra to provide spectral values for the maximum response (maximum direction). We used the scaling factors presented in Table 1 of Shahi and Baker (2014) for ratios of $Sa_{RotD100}$ / $Sa_{GMRotI50}$ to modify the average of the PSHA results for two percent probability of exceedance in 50 years. The maximum direction spectrum is also shown on Figure G-1.

Figure G-2 presents the results of the PSHA for 50 percent probability of exceedance in 30 years (43-year return period, SLE) for about 2.1% damping, which were modified using the Rezaeian et al. (2014) relationship to account for a damping ratio, β = 2.1%. We evaluated the SLE for a damping ratio of about 2.1 percent, which is based on a planned building height of about 290 feet and equation 4-1 of the PEER TBI version 2.03.

Figure G-3 presents the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. From the examination of these results, it can be seen that the Hayward and San Andreas faults dominate the hazard at the project site at different periods of interest.

G3.0 DETERMINISTIC ANALYSIS

We performed a deterministic analysis to develop the MCE_R spectrum at the site. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. On the basis of the deaggregation results we developed deterministic spectra for both scenario earthquakes:

- a Moment Magnitude of 7.3 on the Hayward fault at a distance of 9.7 kilometers from the site, and;
- a Moment Magnitude of 8.1 on the San Andreas fault at a distance of 19.3 kilometers from the site.

The deterministic MCE_R spectrum was defined as an envelop of both scenario earthquakes. This is consistent with the deaggregation results discussed in Section G2.0.

The same attenuation relationships and weighting factors as discussed in Section G1.3 were used in our deterministic analysis. Figures G-4 and G-5 present the 84th percentile deterministic results for the San Andreas and Hayward scenarios, respectively. The average of the four attenuation relationships for the geometric mean are also presented on those figures. Similar to the PSHA results, we developed the 84th percentile deterministic spectrum in the maximum direction using the Shahi and Baker (2014) ratios. Figure G-6 presents the average of the 84th percentile deterministic results in the maximum direction for both scenarios as well as the recommended envelop of both scenarios.

G4.0 RECOMMENDED SPECTRA

The MCE_R as defined in ASCE 7-16 is the lesser of the maximum direction PSHA spectrum having a two percent probability of exceedance in 50 years (2,475-year return period) or the maximum direction 84th percentile deterministic spectrum of the governing earthquake scenario and the DE spectrum is defined as 2/3 times the MCE_R spectrum. Furthermore, the MCE_R spectrum is defined as a risk-targeted response spectrum, which corresponds to a targeted collapse probability of one percent in 50 years. The USGS Risk-Targeted Ground Motion calculator was used to determine the risk coefficients for each period of interest for the probabilistic spectrum. We used these risk coefficients to develop the risk-targeted PSHA spectrum.

Furthermore, we followed the procedures outlined in Chapter 21 of ASCE 7-16 and Supplement No. 1 to develop the site-specific spectra for MCE_R and DE. Chapter 21 of ASCE 7-16 requires the following checks:

- the largest spectral response acceleration of the resulting 84th percentile deterministic ground motion response spectra shall not be less than $1.5 \times Fa$ where F_a is equal to 1.0.
- the DE spectrum shall not fall below 80 percent of Sa determined in accordance with Section 11.4.6, where F_a is determined using Table 11.4-1 and F_v is taken as 2.5 for $S_1 \geq 0.2$ (Section 21.3 of Chapter 21 ASCE 7-16).
- The site-specific MCE_R spectral response acceleration at any period shall not be taken as less than 150 percent of the site-specific design response spectrum determined in accordance with Section 21.3.

Table G-2 presents digitized values of the site-specific spectra for the PSHA 2,475 year return period (maximum direction) and the 84th percentile deterministic (maximum direction). The largest spectral response acceleration of the 84th percentile deterministic response spectrum is 1.767g and is greater than $1.5 \times F_a$ (where $F_a = 1.0$ for Site Class D); therefore, no further scaling of the 84th percentile deterministic spectra was needed.

Figure G-7 and Table G-2 present a comparison of the site-specific spectra for the risk-targeted 2,475-year return period PSHA and the 84th percentile deterministic spectra, both in the maximum direction. In this case, the 84th percentile deterministic spectrum is less than the risk-targeted PSHA spectrum for a 2 percent probability of exceedance in 50 years (2,475-year return period) for all periods of interest and therefore, the deterministic spectrum should be used as the basis for the development of the MCE_R spectrum. The DE spectrum is defined as 2/3 times the MCE_R; however, the DE spectrum should not be less than 80 percent of the DE code spectrum as determined using F_a equal to 1.0 and F_v equal to 2.5 (per Section 21.3 of ASCE 7-16). As shown on Figure G-7 and Table G-2 the DE spectrum is greater than or equal to 80 percent of the of the DE code spectrum for all periods.

TABLE G-2
Comparison of Site-specific and Code Spectra for Development of MCE_R Spectrum per ASCE 7-16
Sa (g) for 5 percent damping

Period (sec.)	Risk Targeted PSHA – 2,475-Year Return Period Max. Dir.	Deter-ministic 84th Percentile Max. Dir.	Lesser of PSHA and Deter-ministic (Initial MCE_R)	2/3 of Initial MCE_R (Initial DE)	ASCE 7-16 - 80% DE per Section 21.3 Site Class D; F_v = 2.50	Recommended Spectra	
						DE	MCE_R
0.010	1.105	0.694	0.694	0.463	0.344	0.463	0.694
0.10	1.946	1.098	1.098	0.732	0.560	0.732	1.098
0.20	2.531	1.511	1.511	1.007	0.800	1.007	1.511
0.30	2.774	1.732	1.732	1.155	0.800	1.155	1.732
0.40	2.758	1.767	1.767	1.178	0.800	1.178	1.767
0.50	2.613	1.718	1.718	1.145	0.800	1.145	1.718
0.75	2.115	1.414	1.414	0.943	0.800	0.943	1.414
1.00	1.741	1.202	1.202	0.801	0.800	0.801	1.202
1.50	1.228	0.870	0.870	0.580	0.533	0.580	0.870
2.00	0.931	0.663	0.663	0.442	0.400	0.442	0.663
3.00	0.616	0.465	0.465	0.310	0.267	0.310	0.465
4.00	0.441	0.353	0.353	0.235	0.200	0.235	0.353
5.00	0.340	0.274	0.274	0.183	0.160	0.183	0.274

The recommended MCE_R and DE spectra for 5 percent damping and SLE spectrum for 2.1 percent damping are presented on Figure G-8. Digitized values of the recommended spectra are presented in Table G-3.

TABLE G-3
Recommended MCE_R, DE, and SLE Spectra
Sa (g)

Period (seconds)	MCE_R 5% Damping	DE 5% Damping	SLE 2.1% Damping
0.01	0.694	0.463	0.221
0.10	1.098	0.732	0.485
0.20	1.511	1.007	0.691
0.30	1.732	1.155	0.691
0.40	1.767	1.178	0.626
0.50	1.718	1.145	0.558

Period (seconds)	MCE_R 5% Damping	DE 5% Damping	SLE 2.1% Damping
0.75	1.414	0.943	0.401
1.00	1.202	0.801	0.299
1.50	0.870	0.580	0.190
2.00	0.663	0.442	0.131
3.00	0.465	0.310	0.076
4.00	0.353	0.235	0.049
5.00	0.274	0.183	0.034

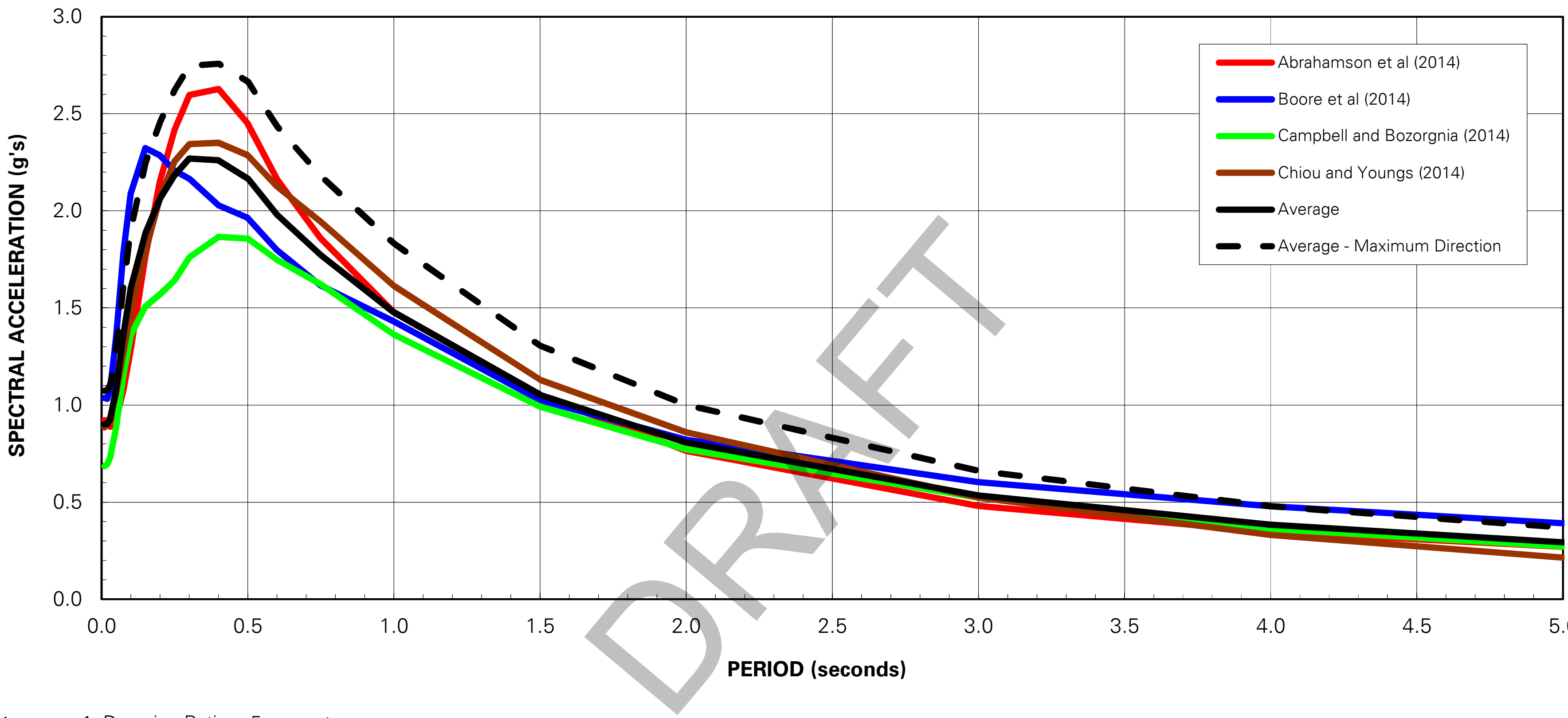
Because the site-specific procedure was used to determine the recommended response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-16 should be used as shown in Table G-4.

TABLE G-4
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.590[2]
S_{M1}	1.412[3]
S_{DS}	1.060[2]
S_{D1}	0.941[3]

[2] S_{DS} is based on the site-specific response spectra and is based on 90 percent of the maximum spectral acceleration within the period range of 0.2 to 5 seconds; it is governed by 90 percent of the spectral acceleration at a period of 0.4 seconds.

[3] S_{D1} is based on the site-specific response spectra and is the maximum of the product of period, T, and spectral acceleration, Sa, for periods from 1.0 to 5.0 seconds; it is governed by the product of the period and spectral acceleration at a period of 4.0 seconds.



Notes

1. Damping Ratio = 5 percent
2. Estimated V_{S30} = 280 m/s
3. Maximum direction factors from Shahi and Baker (2014)

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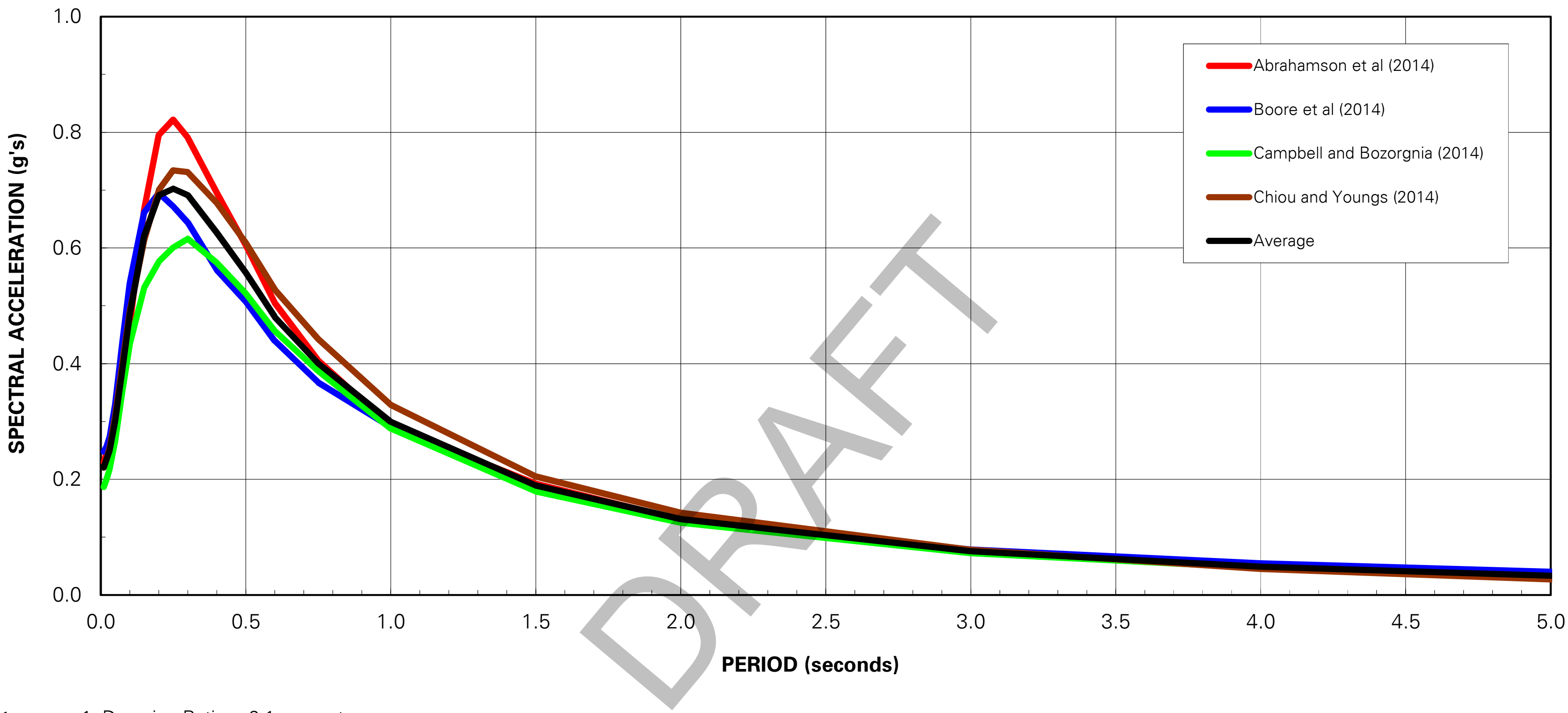

Notes

1. Damping Ratio = 2.1 percent
2. Estimated V_{S30} = 280 m/s

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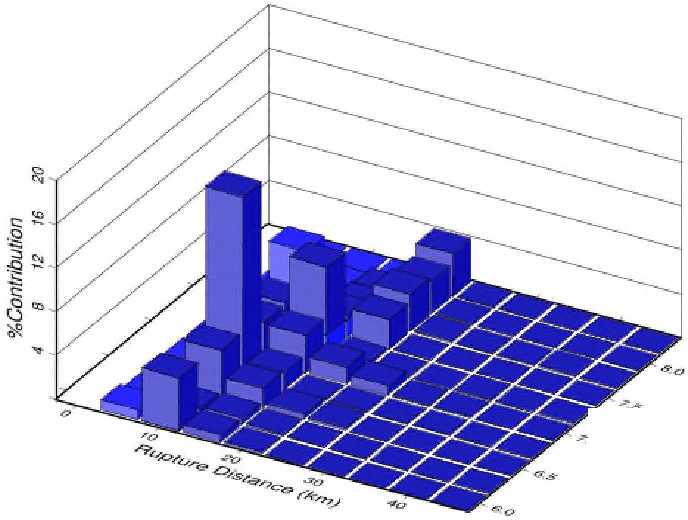
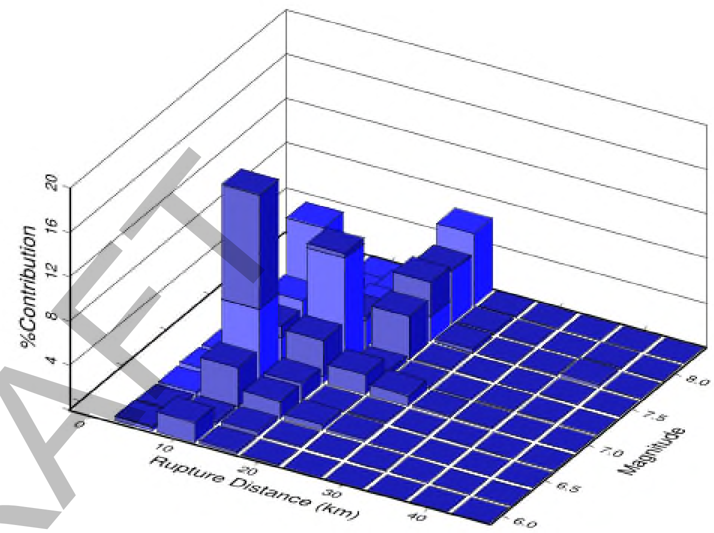
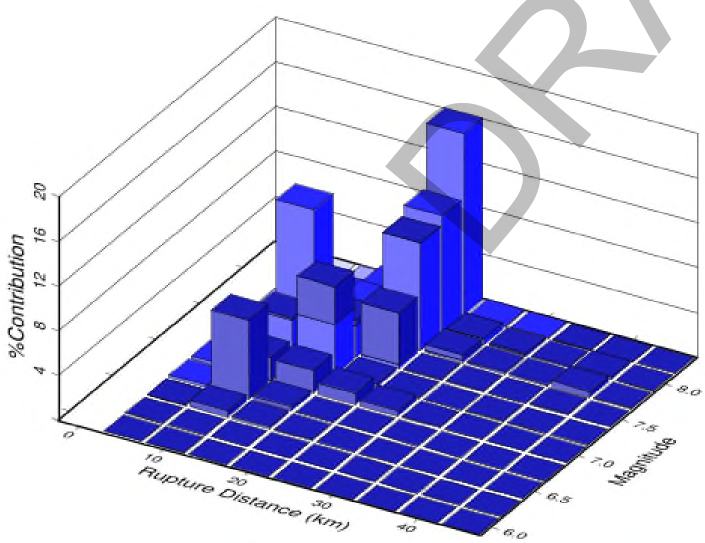


(a) PGA



(b) S_a, T = 1.0 seconds



(c) S_a, T = 4.0 seconds

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Project

PROJECT FOUNTAIN ALLEY
35 SOUTH 2ND STREET

CITY OF SAN JOSE

SANTA CLARA COUNTY CALIFORNIA

Figure Title

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS MAGNITUDE DISTANCE DEAGGREGATION PLOTS

Project No. 770672701

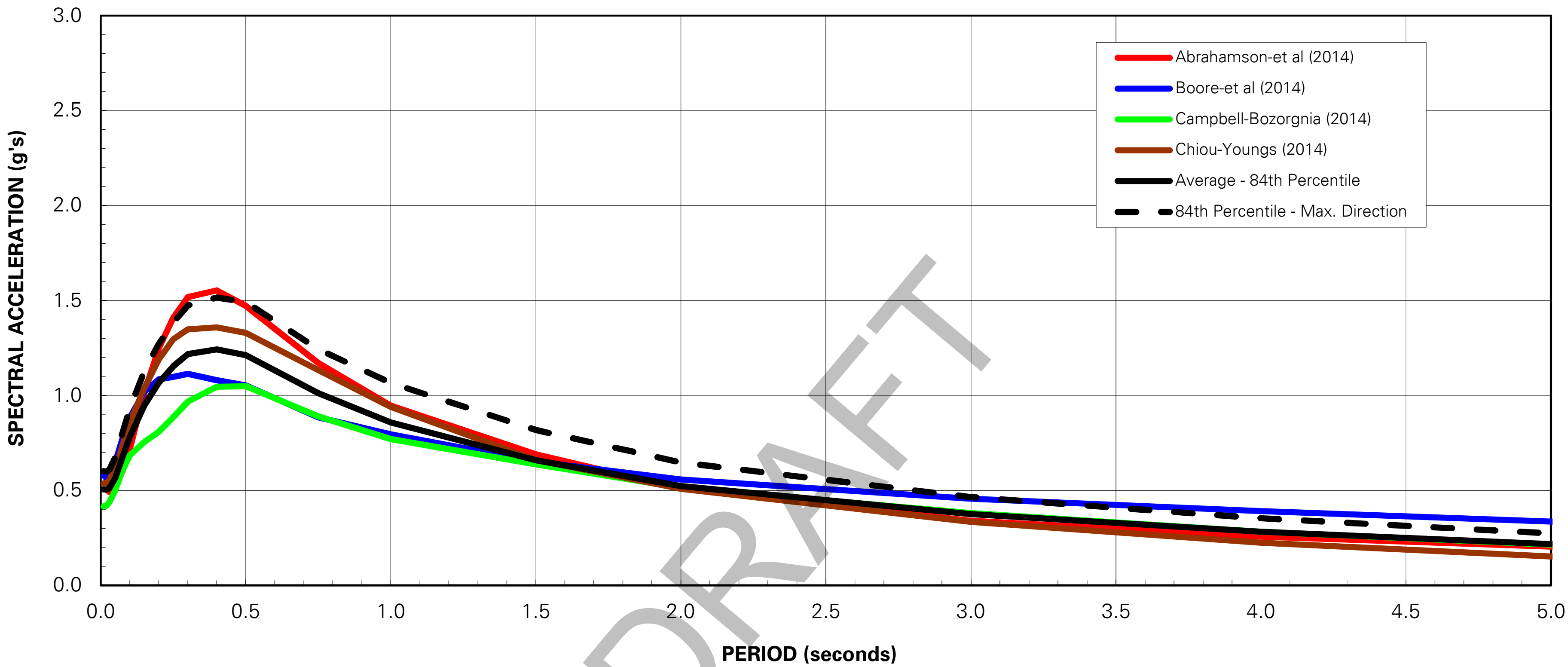
Date 12/29/2020

Scale AS SHOWN

Prepared By: TJL

Figure No.

G-3



Notes

1. Damping Ratio = 5 percent
2. Estimated V_{S30} = 280 m/s
3. Maximum direction factors from Shahi and Baker (2014)
4. Deterministic results correspond to a Moment Magnitude 8.1 occuring on the San Andreas fault about 19.3 km from the site

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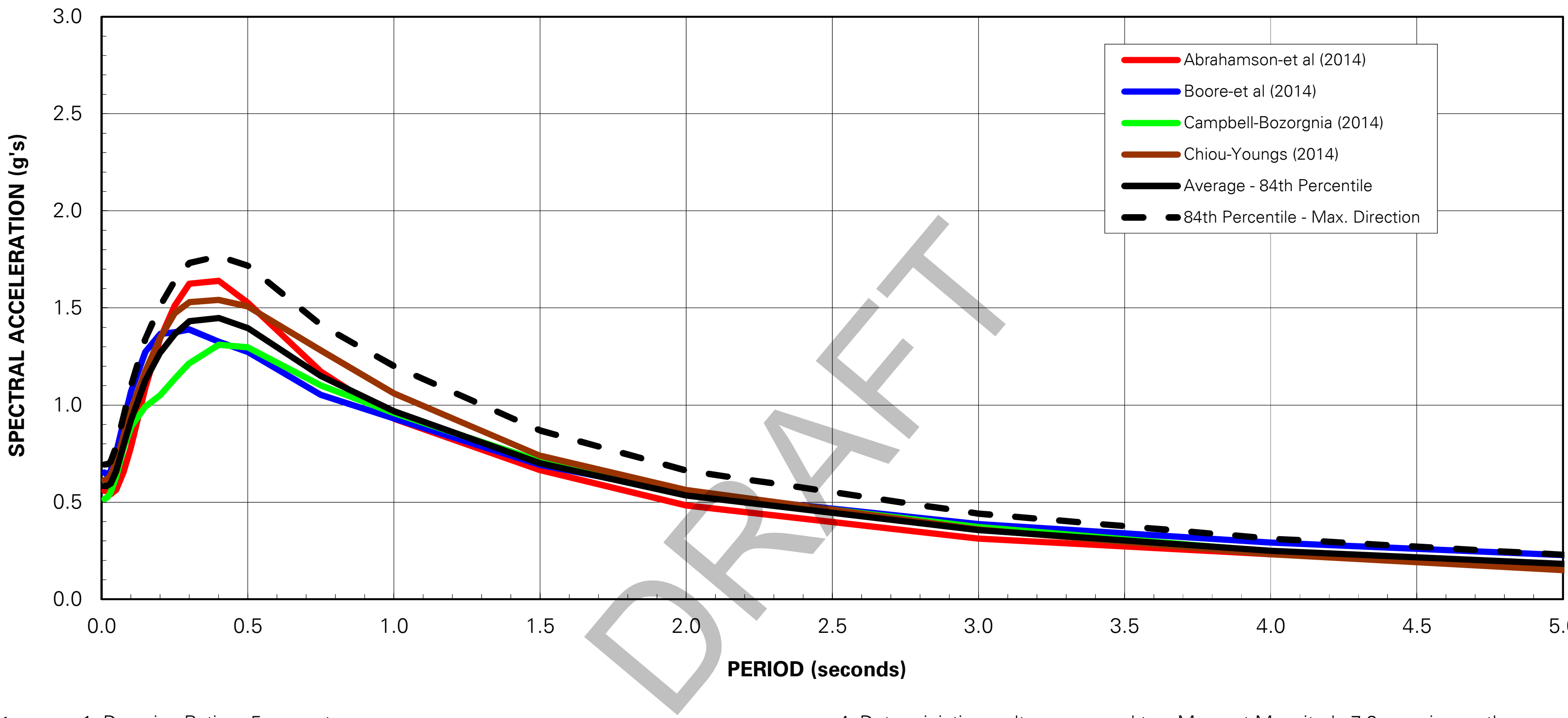

Notes

1. Damping Ratio = 5 percent
2. Estimated V_{S30} = 280 m/s
3. Maximum direction factors from Shahi and Baker (2014)
4. Deterministic results correspond to a Moment Magnitude 7.3 occuring on the Hayward-Rodgers Creek fault about 9.7 km from the site

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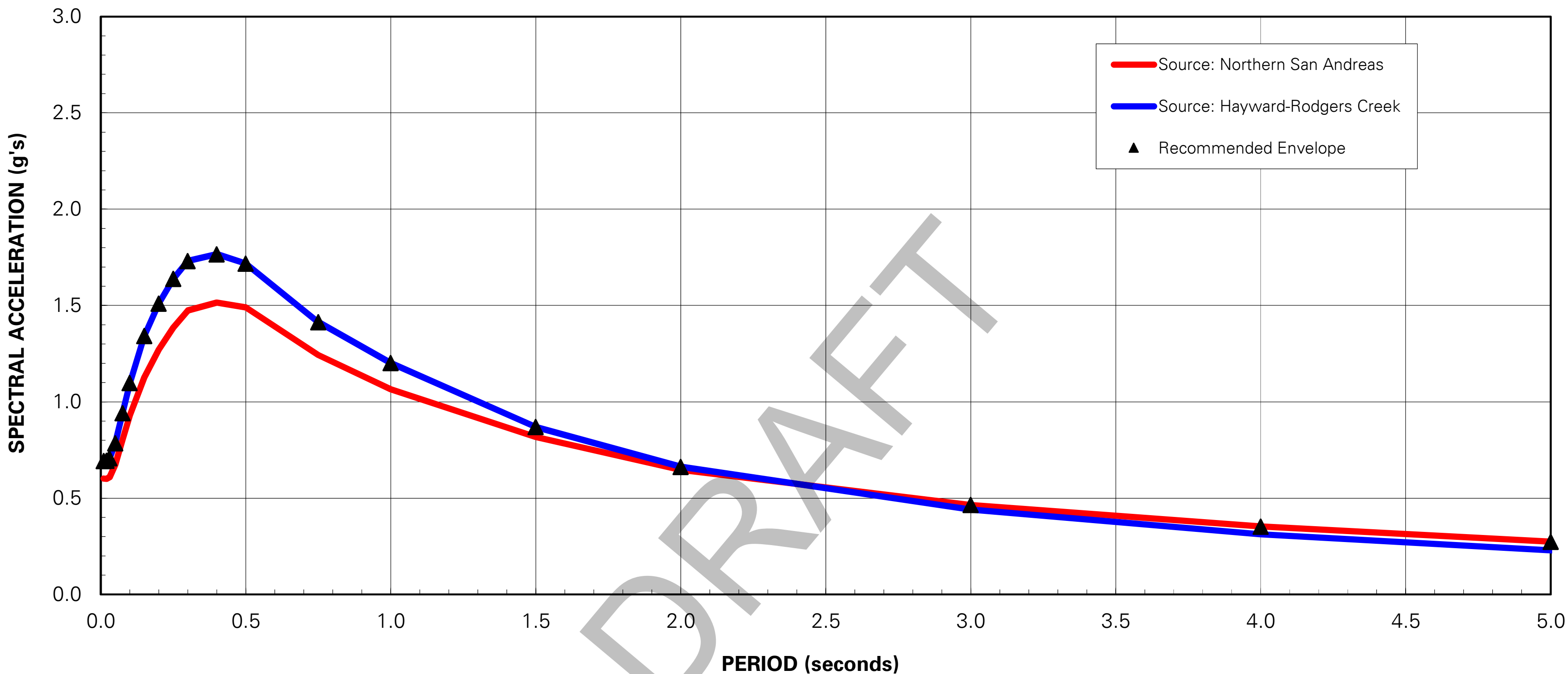

Notes

1. Damping Ratio = 5 percent
2. Estimated V_{S30} = 280 m/s
3. Maximum direction factors from Shahi and Baker (2014)
4. Deterministic results correspond to envelop of the Hayward-Rodgers Creek event (Mw = 7.3 at D = 9.7 km) and San Andreas event (Mw = 8.1 at D = 19.3 km)

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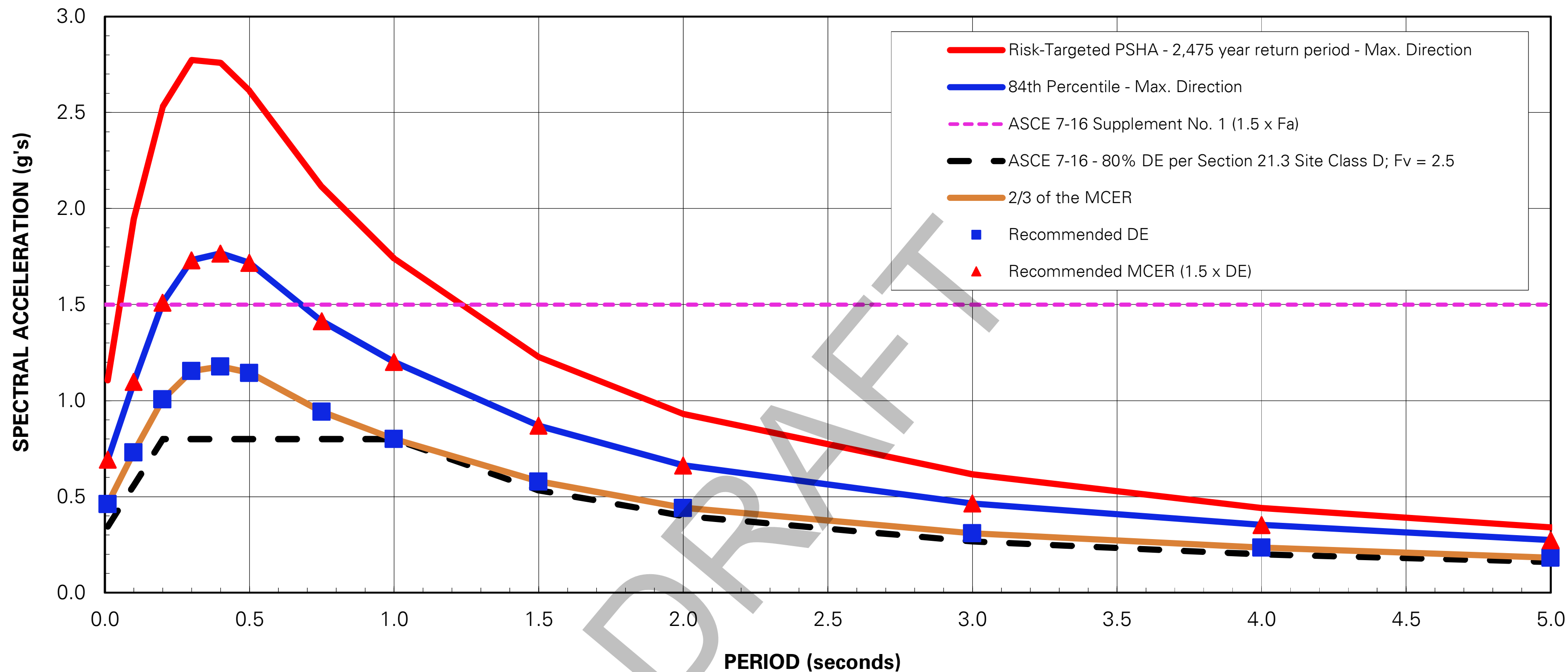

Notes

1. Damping Ratio = 5 percent
2. Estimated V_{S30} = 280 m/s
3. Maximum direction factors from Shahi and Baker (2014)
4. Deterministic results correspond to envelop of the Hayward-Rodgers Creek event (Mw = 7.3 at D = 9.7 km) and San Andreas event (Mw = 8.1 at D = 19.3 km)

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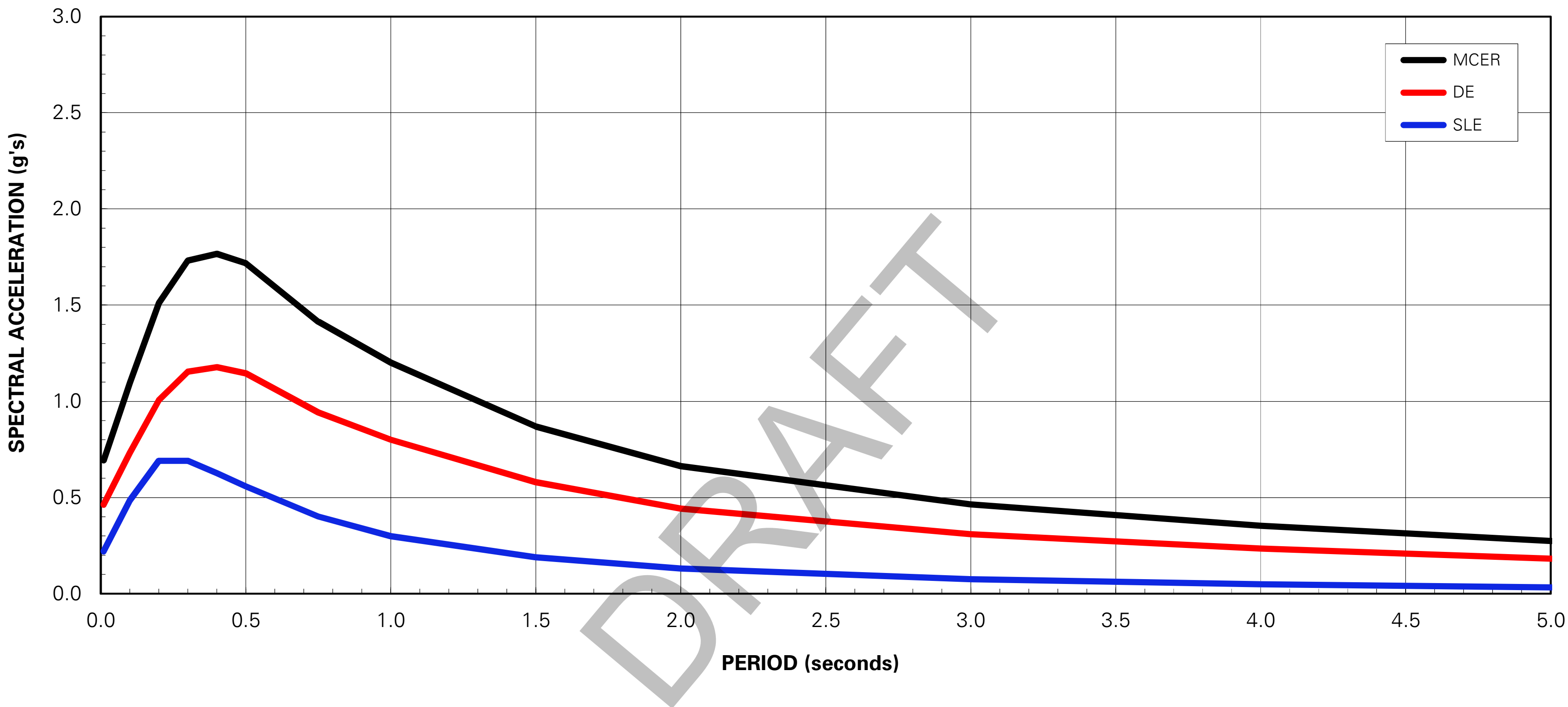
LANGAN ENGINEERING AND ENVIRONMENTAL SERVICES, INC. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	**PROJECT FOUNTAIN ALLEY** **35 SOUTH 2ND STREET** CITY OF SAN JOSE SANTA CLARA COUNTY CALIFORNIA	**COMPARISON OF DETERMINISTIC, PROBABILISTIC AND CODE SPECTRA**	770672701 Date: 12/29/2020 Scale: AS SHOWN Prepared By: TJL	**G-7**



Notes

1. Damping Ratio = 5 percent for the MCE_R and DE, and 2.1 percent for the SLE
2. Estimated V_{S30} = 280 m/s

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