APPENDIX C

GEOTECHNICAL INVESTIGATION

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REPORT ON

GEOTECHNICAL INVESTIGATION
CITY OF SAN JOSE
9084 - FIRE DEPARTMENT TRAINING CENTER - RELOCATION
SAN JOSE, CALIFORNIA

by Haley & Aldrich, Inc.
San Jose, California

Ten Over Studio
San Luis Obispo, California

File No. 134258-002
May 2020

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Attention: Candice Wong
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Subject: Geotechnical Investigation
City of San Jose
Fire Training Center
San Jose, California

Ladies and Gentlemen:

Enclosed is our geotechnical investigation report for the Ten Over Studio team for the proposed new Fire Training Center (FTC) at the Central Service Yard (CSY) in San Jose, California. The project includes relocation of the Fire Department Training Center, Office of Emergency Management (OEM) Services, and the Emergency Operations Center (EOC) to the expanded CSY at 1661 Senter Road and 1591 Senter Road in San Jose, California.

This report contains a discussion of our findings regarding subsurface soil and groundwater conditions, site seismicity and potential seismic hazards, and foundation recommendations. The primary geotechnical issues that should be addressed during the design of the planned structures include the potential for very strong seismic shaking, the potential for liquefaction induced settlement, the presence of compressible clays subject to settlement from structural loads, and the presence of fill in the near-surface soils.

Based on our evaluation of the subsurface conditions at the site, we conclude that the proposed renovation of the existing Building D4 should continue to remain on shallow footing foundations. To address resilience during a seismic event, the EOC building, the OEM Administration and Classroom Building (Building 1), and the Fire Training Tower (Tower) should be supported on ground improvement system with shallow foundations. The shallow foundations may be a spread footing or a mat slab system. The ancillary buildings of the complex are not classified as essential service structures and are not composed of block masonry such as the Tower. Therefore, they are not subject to the same performance criteria and therefore do not require ground improvement elements. These training structures may be supported on either shallow, spread footings or on drilled piers if needed for uplift resistance. The ground improvement elements will primarily gain support in a bearing layer of alluvial gravel and gravelly sand, which generally was encountered starting at about 35 to 40 feet below the existing ground surface. Our findings and recommendations regarding foundations and other geotechnical aspects of this project are presented in the following report.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
HALEY & ALDRICH, INC.

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Enclosures

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1. Introduction

This report presents the results of our geotechnical investigation for the relocation of the Fire Department Training Center in San Jose, California. The proposed location for the new Fire Training Center (FTC) is located at 1661 Senter Rd and is bound by Excite Ballpark and Elma Avenue to the northwest, Senter Road to the northeast, Phelan Avenue to the southeast and South 10th Street to the southwest, as shown on Figures 1 and 2. The approximate coordinates of the site are 37°19'09.5"N and 121°51'37.5"W. The City has acquired the adjacent parcel at 1591 Senter Road to create an expanded campus footprint. The project is owned by and being developed by the City of San Jose. The project design team for the proposed project includes Ten Over Studio as the Architect, Biggs Cardosa Associates, Inc. (BCA) as the Structural Engineer, and BKF as the Civil Engineer.

The 1661 Senter Road site is currently the Central Service Yard (CSY) and is occupied by several existing buildings including the existing warehouse building D4 as well as by an asphalt concrete (AC) parking lot with covered with solar canopies. Within the parking lot there is landscaping with concrete curbs, light posts, and a gate system. The 1591 Senter Road site is currently vacant but has been previously developed. These properties will be referred to as the Site.

Based on the “Program and Adjacency Site Assessment Plan Phase 1 - Building 1 and Building D4” Sheet A2.0 prepared by Ten Over Studios dated 29 October 2019, the Site is generally flat with a ground elevation of approximately 110 feet National Geodetic Veridical Datum of 1929 (NGVD29).

Phase 1 of the proposed construction consists of a two-level Building 1 and a one-story Emergency Operations Center (EOC) building. The proposed 30,900 square foot (sf) Building 1 will be steel framed on a slab-on-grade. The building is not designated as an essential services building but will serve as overflow from the adjacent EOC Building in an emergency. Loading is anticipated to be less than 50 kips of dead plus live column loads. The proposed 9,100 sf EOC Building will also be steel framed with a slab-on-grade. This will be an essential services building. As provided by BCA, interior column loads are estimated to be on the order of 46 kips of dead plus live load. Exterior column loads are estimated to be on the order of 16 kips of dead plus live load.

The existing Building D4 will be seismically upgraded and will continue to be used for storage with inclusion of a fitness center and offices. The wood framed building does not need to comply with the Essential Services Act.

Phase 2 of the proposed construction includes a new five-story Training Tower, which will be constructed of concrete masonry units (CMU) with two smaller, single-story buildings attached. Below grade pits but not full floor plate basements are anticipated. The 12,600-sf tower and two 500 and 700 sf buildings are non-occupied structures and are being designed under the National Fire Protection Code (NFPC). As provided by BCA, dead plus live loads are estimated to vary on the order of 3 to 20 kips per linear foot. Ancillary buildings will be constructed for training purposes and are anticipated to have dead plus live loads of less than 3 kips per linear foot.

Additional improvements include asphalt concrete (AC) paving, pervious asphalt pavement (PAP), exterior concrete flatwork, below grade utilities, a bioswale storm water management feature and landscaping.

If the project differs significantly from that described above, we should be consulted to review the applicability of our recommendations.

2. Scope of Services

The scope of our geotechnical services was described in our proposal dated 28 August 2019 and modified in our Contract Amendment dated 29 March 2020. Our services included ten exploration points including a combination of rotary wash borings, Cone Penetrometer Tests (CPTs), and percolation tests; performing a laboratory testing program; and completing engineering analyses to develop conclusions and recommendations regarding:

- Soil and groundwater conditions at the Site;
- Site seismicity and seismic hazards including liquefaction potential;
- Foundation design criteria, including design criteria for vertical and lateral support of the essential services building structures;
- Foundation design criteria, including design criteria for vertical and lateral support of the training structures;
- Seismic design parameters in accordance with the 2016 California Building Code;
- Interior concrete slabs-on-grade;
- Retaining wall recommendations including parameters for shoring design and dewatering considerations;
- Flexible asphalt-concrete designs;
- Exterior concrete flatwork;
- Site grading, including criteria for fill quality and compaction;
- Infiltration recommendations for stormwater management; and
- Construction considerations (as appropriate).

3. Field Investigation

The subsurface conditions at the Site were investigated by using rotary wash borings, by advancing CPTs, and by performing field percolation tests. Specific details of these investigations are described below.

3.1 FIELD INVESTIGATION

In November 2019, subsurface conditions at the Site were explored by Haley & Aldrich by drilling three rotary wash borings, advancing three CPTS, performing one field percolation test, collecting soil samples, and submitting the selected samples to the laboratory for geotechnical testing. Additionally, in March 2020, two more field percolation tests were performed along with an additional CPT. Prior to performing our field investigation, we notified Underground Service Alert (USA) to check that the boring locations were clear of existing utilities, as required by law. We also retained Subtronics, a private underground utility locator, to check for buried utilities and obstructions prior to starting the fieldwork. As part of daily field work preparation, daily on-site safety meetings were held.

Upon completion of our field exploration program, the borings were backfilled with cement grout under the observation of the Santa Clara Valley Water District (Valley Water) grout inspector in accordance with the drilling permit requirements. Excess spoils were drummed for disposal as non-hazardous waste and removed from the Site. Details of the methods employed are presented below.

3.1.1 Rotary Wash Borings

The rotary wash borings (designated as HA-1 through HA-3) were drilled by Pitcher Services, LLC (Pitcher) of Palo Alto, California. The borings, including sampling, were advanced from 4 to 8 November 2019 to depths of about 51½ feet below the existing ground surface (bgs), including sampling. Each rotary wash boring was started using a solid flight auger. The rotary wash bath was introduced at about 5 to 10 feet bgs. The details of the approximate depths and elevations (NGVD29) are presented in the table below.

TABLE 1
Summary of Boring Elevations and Depths

Boring	Top of Boring Elevation (feet)	Approximate Depth of Boring (feet)	Bottom of Boring (feet)
HA-1	110	51½	58½
HA-2	110	51½	58½
HA-3	110	51½	58½

The approximate locations of the borings are presented on Figure 2.

Haley & Aldrich's field representative logged the subsurface conditions encountered in each boring during the investigation. Soil samples were obtained using a lined Modified California sampler and an unlined Standard Penetration Test (SPT) sampler. The Modified California sampler has a 3.0-inch outside diameter and a 2.43-inch inside diameter, and the SPT sampler has a 2.0-inch outside diameter and a 1.38-inch inside diameter. The locations where each sampler was used are recorded on the boring logs. Modified California and SPT soil samples were collected by driving each respective sampler 18 inches or to penetration refusal, whichever was encountered first, using a 140-pound, above-grade hammer falling 30 inches. Uncorrected blow counts were recorded for each 6-inch-long interval of sampler penetration and are presented on the boring logs, which are included in Appendix A. After the samplers were withdrawn from the test borings, the samples were removed, examined for logging purposes, labeled, and sealed to retain the natural moisture content for laboratory testing. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs in Appendix A.

3.1.2 Cone Penetration Tests (CPTs)

In addition to rotary wash borings, the study included three CPTs (designated as CPT-1 through CPT-3) performed by Gregg Drilling using truck-mounted CPT rig with a 30-ton push capacity. The CPTs were advanced on 26 November 2019 and were performed to depths between about 50 and 100 feet bgs. An additional CPT (CPT-4) was advanced on 2 March 2020 within the footprint of the planned training tower. The approximate locations are indicated on Figure 2.

The stratigraphic interpretation of the CPT data was performed based on relationships between cone bearing and sleeve friction versus penetration depth. The friction ratio (Rf), which is sleeve friction divided by cone bearing, is a calculated parameter used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone bearing and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate small excess pore water pressures. The interpretation of soil properties from the cone data has been carried out using recent correlations developed by Robertson.[1] A pore pressure dissipation test was performed at each CPT location. Based on the CPT pore pressure dissipation tests, groundwater was estimated at a depth of 22.5 to 34.3 feet bgs. CPT-1 is a seismic CPT, or SCPT. An SCPT is a method of calculating the small strain shear modulus of the soil by measuring shear wave velocity through the soil. The small strain modulus is an important quantity for determining the dynamic response of soil during earthquakes. The results of the CPT exploration are presented in Appendix B.

TABLE 2

Summary of CPT Elevations and Depths

Boring	Top of CPT Elevation (feet)	Approximate Depth of Boring (feet)	Bottom of Boring (feet)
CPT-1	110	100	10
CPT-2	110	50	60
CPT-3	110	50	60
CPT-4	110	50	60

[1] P.K. Robertson, 1990, Soil Classification Using the Cone Penetration test, Canadian Geotechnical Journal, 27(1), pp. 151-158.

3.1.3 Percolation Tests

One downhole percolation test[2] was performed at test location PT-1 on 26 November 2019. Two additional percolation tests were performed at the PT-2 and PT-3 locations on 2 March 2020. The percolation tests consisted of installing a 2-inch-diameter slotted PVC pipe in the center of the approximately 4.6-foot-deep hand-auger boring. The area around the pipe was backfilled with screening sand to a depth of approximately 3 feet bgs. Hydrated bentonite was used to fill the remainder of the area around the pipe to the ground surface. Each test location was presoaked prior to starting the percolation tests. The test was performed by allowing the water to drop for equal time increments of 30 minutes. The drop in the water level was recorded for each increment. At the PT-1 test location the water level was not refilled between the time increments to estimate the dependency of infiltration rate with water head. At the PT-2 and PT-3 test locations water drained fully within 10 minutes, therefore water was refilled between the trials. Testing at each location was halted once percolation rate stabilized, as determined by consistent readings over three consecutive test intervals (defined as less than 10 percent change in the rate of percolation during three consecutive tests). The direct percolation rates measured in the field were adjusted in accordance with the percolation method guidelines. The results of our testing are discussed in Section 6.12, "Stormwater Infiltration," of this report and the percolation test data are presented in Appendix C.

TABLE 3

Summary of Percolation Test Elevations and Depths

Permeability Test	Top of Percolation Test Elevation (feet)	Approximate Test Depth (feet)
PT-1	110	3½
PT-2	110	5
PT-3	110	5

3.2 LABORATORY TESTING

Soil samples were collected from each boring and transported to Inspection Services, Inc. (ISI) in Berkley, California for geotechnical laboratory testing. The samples were tested for sieve analysis and fine content, liquid and plastic limits (Atterberg limits), Unconfined Compression (UC) strength test, and Resistance (R)-Value. The geotechnical laboratory test results are presented in Appendix D. Additional near-surface soil samples were submitted to CERCO Analytical, Inc. in Concord, California and tested for corrosion properties, including pH, resistivity, sulfate content, and chloride content. The corrosion test results are also presented in Appendix D.

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2 Percolation tests were performed in general conformance with the guidelines presented in the County of Los Angeles Department of Public Works Administrative Manual GS200.1.

4. Existing Conditions

4.1 REGIONAL SEISMICITY

The major active faults in the area are the Hayward, Calaveras, Monte Vista, and San Andreas faults. For each of the active faults within 100 kilometers (km) of the Site, the distance and direction from the Site and estimated maximum Moment magnitude,[3] M_w, are presented on Table 4.

TABLE 4
Active Faults within 100 km of the Site

Fault Name	Distance (km)	Direction	Maximum Moment Magnitude
Hayward - South East Extension	10.0	Northeast	6.5
Monte Vista	11.8	Southwest	6.5
Hayward - South	14.5	Northeast	6.9
Hayward - South East Extension	10.0	Northeast	6.5
Hayward - Total	14.5	Northeast	7.1
Calaveras (North of Calaveras Reservoir)	14.5	Northeast	6.8
Calaveras (South of Calaveras Reservoir)	14.7	Northeast	6.2
San Andreas - 1906 Rupture	18.4	Southwest	7.9
San Andreas - Peninsula	18.4	Southwest	7.0
San Andreas - Santa Cruz Mountains	20.8	South	7.0
Sargent	23.8	South	6.8
Zayante-Vergeles	29.6	South	6.8
Greenville	37.3	East	6.9
San Gregorio	40.9	Southwest	7.3
Great Valley - 6	44.8	Northeast	6.7
Great Valley - 7	45.2	Northeast	6.7
Hayward - North	46.1	Northwest	6.9
Monterey Bay - Tularcitos	51.0	South	7.1
Ortigalita	57.6	East	6.9
Great Valley - 8	58.7	East	6.6
Concord - Green Valley	59.6	North	6.9
Palo Colorado	64.5	South	7.0
Quien Sabe	68.6	Southeast	6.5
Great Valley - 5	76.6	North	6.5
Great Valley - 9	81.8	Southeast	6.6
Healdsburg - Rodgers Creek	89.5	Northwest	7.0
West Napa	94.4	North	6.5

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 occurred

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

east of Monterey Bay, reportedly on the San Andreas Fault (Toppozada and Borchardt, 1998). The estimated M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred on the San Andreas Fault 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 approximately 470 km in length along the San Andreas Fault from Shelter Cove to San Juan Bautista. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 km away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of 17 October 1989 occurred on the San Andreas Fault in the Santa Cruz Mountains. It had an M_w of 6.9 and was approximately 31 km 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 an 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 third Uniform California Earthquake Rupture Forecast (UCERF3) prepared by the U.S. Geological Survey (USGS) reports a 72 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay region (which includes the Site) by the year 2044 (WGCEP, 2015).

4.2 SUBSURFACE SOIL CONDITIONS

Subsurface conditions at the Site were investigated by performing three rotary wash borings, advancing four CPTs, performing three percolation tests, collecting samples of soils for visual evaluation, and performing laboratory testing on select samples. Based on these data, we conclude that the Site is blanketed with fill which is underlain by interbedded layers of soft to medium stiff clays and silts with varying amounts of loose to dense sands and gravels.

Underlying the asphalt paving, undocumented fill was encountered. The surficial fill soils in about the upper 3 to 8 feet at the Site are of low to medium plasticity and are composed of fill from existing and previous site developments. In our subsurface exploration, we encountered medium dense clayey sand and soft lean clay. The clays encountered have a low to moderate expansion potential.

Underlying the fill layer, gravel and sand with varying amounts of clay and silt were encountered, except in HA-1 where interbedded layers of medium to fine sand were encountered. This sand and gravel layer extends to depths of about 18 to 20 feet.

Below the sand and gravel layer clay and clayey silt were encountered. The clay and clayey silts were soft to medium stiff. The thickness of these layers ranged from about 6 to 10 feet in thickness. A consistent layer of clayey silt was found between all borings starting from 27 to 29 feet bgs and extending 2½ to 7½ feet in thickness.

A layer of poorly graded, medium dense gravel and sand with varying amounts of silt was found from about 31 to 35½ feet bgs. The highly variable, interbedded layers extended to depths of about 40 to 44 feet bgs. Within this layer an approximately 1½ foot layer of poorly graded sand was encountered in HA-1 at about 31 feet.

Below this depth, we generally encountered high plasticity clay and silty clay of medium stiffness, except in HA-3 where medium dense gravel was found. HA-3 is nearest to the Coyote Creek channel. This stratum extends to the depths of approximately 50 to 52 feet.

Following these depths, a 1½- to 2-foot layer of medium dense silty sand was encountered which was followed by interbedded layers of clay and silty clay. The clay layer is present to about 88 feet. Dense to very dense sandy soil was encountered below the cohesive soils to the maximum depth explored, about 101½ feet. A subsurface profile depicting the above general conditions and the assumed boundaries is presented Figure 3.

4.3 GROUNDWATER CONDITIONS

The depth to groundwater at the Site was estimated using groundwater level measurements from the rotary wash borings and pore pressure dissipation test data. Groundwater levels were masked in borings HA-1 through HA-3 due to the use of rotary wash methods. The depth to groundwater was estimated by using pore pressure dissipation tests performed at the CPT locations. Groundwater was estimated to be present at the depths between 22.5 and 34.3 feet bgs during our CPT investigation.

Groundwater levels can fluctuate based on seasonal rainfall amounts and perched groundwater conditions. Groundwater levels in the Santa Clara Valley are also influenced by regional overdraft. Although Valley Water has implemented recent programs for recharging the aquifer, the Valley is still recovering from the overdrafting. As a result, historical groundwater elevations can be higher than current conditions. We reviewed the historical high groundwater levels reported by the California Division of Mines and Geology (CDMG) Seismic Hazard Zone Report for the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California (CDMG, 2000). This depth-to-groundwater value mapped between 10 and 20 feet below grade corresponds to a design groundwater elevation of approximately 15 feet including adjustments for the placement of small amounts of fill in the past.

5. Discussion and Conclusions

Based on the results of our subsurface exploration, it is our opinion that the proposed Site is geotechnically suitable for the planned improvements, provided our recommendations are followed. The primary geotechnical concerns for this project are:

- the Site seismicity and potential hazards including liquefaction induced settlement,
- excessive settlement of the compressible clay layers, and
- the presence of surficial fill soils.

5.1 SEISMIC HAZARDS

During a major earthquake, very strong shaking has the potential to occur at the Site, as experienced during the 1989 Loma Prieta event. Shaking during an earthquake can result in ground failure, such as that associated with soil liquefaction, lateral spreading, and cyclic densification. Haley & Aldrich's assessment of these potential seismic hazards are presented in the following sections.

5.1.1 Site Seismicity

A probabilistic seismic hazard analysis (PSHA) was performed using the USGS deaggregation website (https://earthquake.usgs.gov/hazards/interactive/). The USGS deaggregation utilizes the 2014 USGS Conterminous U.S. 2014 hazard model. A Site Class D soil profile was selected for this analysis, which corresponds to an average shear wave velocity over the upper 100 feet (30 meters) of the Site (V_{s30}) of 259 meters per second (m/s; comparable to V_{s30} = 230 m/s calculated at Seismic CPT-1). The deaggregation analysis was performed for the Maximum Considered Earthquake (MCE), defined as an event with a 2 percent probability of exceedance in 50 years (return period of approximately 2,500 years). The MCE event is expected to produce a seismic event with a mean M_w of 6.84.

The risk-based site-modified peak ground acceleration (PGA_M) for the Site is 0.628g; this value was computed based on procedures outlined in ASCE 7-16.

5.1.2 Soil Liquefaction and Associated Hazards

Liquefaction is the process in which saturated, cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure during cyclic loading resulting from earthquake ground motions. The type of soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded sand and silt that have low clay content. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of liquefaction.

The Site lies within a potential liquefaction hazard zone, as defined by the CDMG Seismic Hazard Zone Report for The San Jose East 7.5-Minute Quadrangle, Santa Clara County, California (CDMG, 2000). The Site lies within a zone of up to 5 percent probability of liquefaction during an M7.8 earthquake on the San Andreas fault according to the Liquefaction Probability for M7.8 San Andreas Fault Earthquake Scenario, Santa Clara County (Holzer, 2008). Depth to groundwater at the Site during the investigation was estimated as high as 22.5 feet bgs based on data from pore pressure dissipation tests performed at the CPTs on 26 November 2019. The historical high groundwater level at the Site is approximately 15 feet bgs (CDMG, 2000).

We evaluated the potential for soil liquefaction at the Site by performing analyses in accordance with the methodology in publications prepared by Robertson and Wride (1998) for the National Center for Earthquake Engineering Research (NCEER). Based on our analyses, we conclude that the potential for on-site liquefaction to occur within the upper 50 feet of the Site and to adversely impact the planned structures is high. We compute that potentially liquefiable soil layers may result in about 1½ to 5 inches of total liquefaction-induced settlement in the upper 50 feet without mitigation measures such as ground improvement. The summary of the analysis is presented in the Table 5 below.

To consider the impacts of liquefaction below the ground improvement, we considered the potential from 40 to 60 feet. Our rotary wash borings were terminated at 51.5 feet. Therefore, we are only reflecting the CPT data in Table 5 below. We anticipate that the deeper soils, from about 40 to 60 feet may have the potential to generate liquefaction induced settlement on the order of 1 inch or less. Over a majority of the Site, liquefaction-induced differential settlement during a major seismic event is estimated to be on the order of 1 to 2½ inches over a horizontal distance of 50 feet.

TABLE 5

Estimated Liquefaction Settlement

Exploration Location	Estimated Liquefaction Settlement Upper 50 feet bgs	Estimated Liquefaction Settlement Between 40 and 60 feet bgs (1)
	(inches)	(inches)
Boring HA-1	1½	(2)
Boring HA-2	3½	(2)
Boring HA-3	4½	(2)
Seismic CPT-1	4½	<1
CPT-2	2¼	(2)
CPT-3	5	(2)
CPT-4	3¾	(2)

(1) Assumes that ground improvement elements extend to a maximum depth of about 40 feet.
(2) Exploration terminated before reaching 60 feet.

5.1.3 Cyclic Densification

Seismically-induced compaction or densification of non-saturated granular soil (such as sand above the groundwater table) due to earthquake vibrations can result in settlement of the ground surface. We analyzed the Site using the procedure outlined by Tokimatsu and Seed (1987). Based on the results of this investigation, we conclude that the soils above the groundwater table primarily consist of interbedded layers of clays and silts with varying amounts of sand and gravel. Therefore, we judge that seismic densification is unlikely at this Site.

5.1.4 Lateral Spreading

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occurs as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes and creek channels. This site is not adjacent to any free faces. Therefore, we conclude that the potential for global lateral spreading to occur under the footprint of the FTC project during a major earthquake is nil.

5.1.5 Sand Boils

Based on Ishihara (1985), we believe that the potential for ground surface disruption (such as sand boils, ground fissures, etc.) to occur at this site is low. Another major concern during an earthquake is some form of ground surface disturbance or ground failure. The ground failure can be in the form of sand boils, small ground fissures, ground oscillation such as buckled pavements, curbs, broken pipelines, etc., and lateral ground displacement. One of the major reasons for ground surface disruption is insufficient cover thickness of a non-liquefiable layer over a liquefiable layer (Ishihara, 1985; Youd and Garris, 1995). Ground surface disruption estimates have been performed using Ishihara (1985). Due to a relatively thick cap of a non-liquefiable layer above the potentially liquefiable layer, we anticipate ground surface disruption due to be low.

5.1.6 Tsunami

This Site is mapped in the San Jose-West Quad. Based on maps published by California Emergency Management Agency (2009), the Quad is not in an area predicted to be affected by tsunamis. Therefore, we judge that the potential for a seismically induced wave to impact the site to be very low.

5.1.7 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The Site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act. Based on this information, we conclude the risk of surface faulting and secondary ground failure to be very low.

5.2 EXPANSION POTENTIAL

The results of our field investigation indicate the surficial layer of clay soil in the proposed building areas has a low to moderate expansion potential and is composed of fill. Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, perched groundwater, drought or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs supported on-grade or pavements supported on these materials. To mitigate the presence of medium plasticity clays and the low to moderate expansive potential, we have recommended moisture conditioning of the clayey soils and use of a “non-expansive” fill section under concrete slabs-on-grade. For this project, the capillary break may serve as the “non-expansive” fill section under the concrete slabs-on-grade.

5.3 UNDOCUMENTED FILL

The current Site has undergone previous development, some of which is still present. We encountered undocumented fill within our borings and understand that there were previously railroad tracks and facilities on the property that includes the auxiliary buildings. To address the presence of fill, the shallow foundations systems not supported on ground improvement will need to be supported on a minimum of 36 inches of engineered fill or into competent native material.

5.4 FOUNDATIONS AND SETTLEMENT

Typically, conventional reinforced concrete spread footings with soil-supported slabs-on-grade is the most economical foundation system for most buildings, if a competent soil or bedrock bearing stratum exists at shallow depth. The undocumented fills, compressible clays, and liquefiable sandy and silty soil beneath the proposed structures are unsuitable as building foundation bearing strata in their current state. For the essential structures and Tower, we recommend the uppermost suitable natural bearing stratum be the alluvial gravel and gravelly sand, which generally was encountered starting at about 35 to 40 feet bgs. For the auxiliary buildings, we recommend that the foundations for the buildings be supported on 36 inches of engineered fill or extended into competent native material.

Various foundation support alternatives and approaches were evaluated to reduce the potential for bearing capacity failure and building settlement. The evaluation included supporting the proposed building loads on either stiffened shallow foundations underlain by improved ground or on deep foundations. Based on our evaluations of the Site and building configurations, conventional reinforced concrete spread footings or a mat slab, after ground improvement, have been identified as the most cost-effective, technically feasible approach to provide foundation support for Building 1, the EOC and the Tower.

Due to their unoccupied classification and building material type, the existing D4 and the ancillary training buildings do not need to be supported on ground improvement elements. They should be supported on engineered fill or competent native soil. It is likely that they will experience significant total and differential settlement following a design level seismic event.

6. Recommendations

Our geotechnical recommendations for the FTC building (Building 1), the EOC building, the existing Building D4 retrofit upgrades, the Tower and auxiliary buildings, as well as other site improvements, are presented in this section of the report.

6.1 FOUNDATIONS

The proposed new Building 1, the EOC building, and the Tower may be supported on either a mat foundation or shallow footing foundation system over ground improvement elements.

The auxiliary buildings may be supported on a shallow footing foundation system without ground improvement elements due to their unoccupied classification.

Light poles, canopies, and other structures needing more lateral support than shallow foundations provide may be supported on drilled piers.

These foundation options are discussed below.

6.1.1 Mat Slab Bearing on Improved Ground

We understand that the building mat will be founded at least 3 feet below existing grade with ground improvement elements. At this depth, we recommend a maximum net allowable bearing capacity of 1,000 pounds per square foot (psf) for the entire mat, with local maximum net allowable bearing capacities of 1,500 and 2,500 psf for areas no greater than 2,500 square feet and 500 square feet, respectively. This net allowable bearing capacity includes a safety factor of at least 3 with respect to shear failure of the foundation soils. The mat should be designed using a modulus of subgrade reaction of 100 pounds per square inch (psi) per inch of deflection pending confirmation from the ground improvement specialty contractor. For transient loading conditions, such as wind and earthquake, the net allowable bearing pressure may be increased by a factor of one-half.

The mat should be placed neat against native soil or engineered fill. It is critical that the mat excavation not be allowed to dry before placing concrete. If shrinkage cracks appear in the excavation, the excavation should be thoroughly moistened to close all cracks prior to concrete placement. The excavation should be monitored by a representative of Haley & Aldrich, Inc. for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. If soft or loose materials are encountered at the bottom of the excavation, they should be removed and replaced with either engineered fill or lean concrete. In addition, if dry or soft materials are encountered at the bottom of excavation, the material should be removed and also backfilled with lean concrete as soon as possible following excavation. Depending on the time of year of construction and the contractor's sequencing, consideration should be given to pouring a 2- to 3-inch lean concrete "rat slab." The use of lean concrete reduces the disturbance of the soils exposed at the bottom of the excavation to weather and construction activities following excavation.

Lateral loads may be resisted by a combination of friction between the bottom of the mat and by passive pressure against the sides of the mat. For friction resistance, we recommend a coefficient of friction of 0.30 be used for design. Passive resistance may be computed based on an equivalent fluid weight of 300 pounds per cubic foot (pcf) for mats constructed against competent, native soils or engineered fill. The passive pressures can be assumed to act starting at grade in paved areas and at a depth of 1 foot below grade in unpaved areas. It should be noted that the lateral load resistance values discussed above are only applicable where the concrete for the mat is either placed directly against competent, native soil, engineered fill, or the voids created from the use of forming are backfilled with properly compacted soil. For design purposes, the friction resistance and the passive resistance may be assumed to act simultaneously.

To maintain the desired support, foundations adjacent to utility trenches or other existing foundations should be deepened during the design of the project so that their bearing surfaces are below an imaginary plane having an inclination of 1.5 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent foundations or utility trenches.

6.1.2 Shallow Foundations Bearing on Improved Ground

We recommend ground improvement consist of installing rigid inclusions or similar vertical elements through the unsuitable soil to create a stiffened mass suitable for footing bearing. Ground improvement systems are designed and constructed by specialty contractors with proprietary equipment and/or proprietary construction techniques. Typically, ground improvement options to mitigate the compressible clays and the liquefiable soil encountered at the project site include rigid inclusions using non-driven, non-vibratory methods, such as drilled displacement elements consisting of columns of unreinforced sand-cement slurry and/or lean concrete (i.e., rigid inclusions), or deep soil mix (DSM) columns, which consist of cementitious grout that is blended into the underlying soil to form soil-cement columns.

The detailed final design and installation of ground improvement is typically performed by specialty subcontractors, in accordance with performance criteria established by the Owner's Geotechnical Engineer (Haley & Aldrich). Proposals by perspective specialty contractors bidding the work will be reviewed by the Geotechnical Engineer (Haley & Aldrich) and the Structural Engineer (BCA) for suitability of the proposed system and compliance with the project requirements.

The following design criteria are recommended for footings installed after ground improvement is performed:

- Design footings using allowable bearing pressures of 4 kips per square foot (ksf). The allowable bearing pressure used for final footing design should be verified by the ground improvement designer. For transient loading conditions, such as wind and earthquake, the net allowable bearing pressure may be increased by a factor of one-half. Increases to the allowable bearing capacity would require redesign of the foundations and are not considered desirable.
- Design footings to have a lateral dimension of 24 inches (in.) or greater.
- Locate bottoms of footings at least 18 inches below the bottom of the adjacent ground floor slab or the adjacent grade, whichever is deeper.
- Ground improvement elements should not be relied upon for uplift resistance unless permanent hold-down elements are incorporated into the design of the ground improvement elements. Haley & Aldrich recommends using an average modulus of vertical subgrade reaction value of 100 pounds per cubic inch (pci) for evaluating shallow foundations on improved ground pending confirmation from the specialty contractor.

- Lateral loads can be resisted through a combination of friction along the tops of the ground improvement elements and passive soil resistance against the embedded vertical faces of the stiffened shallow foundations. For computing frictional resistance, Haley & Aldrich recommends using a friction factor of 0.30 times the compressive load applied against the tops of the ground improvement elements.
- Additional lateral load resistance can be obtained by passive resistance acting against the embedded vertical faces of the proposed shallow foundations. To compute passive resistance, Haley & Aldrich recommends using an allowable equivalent fluid weight of 300 pcf applied against embedded faces of the shallow foundation elements. This equivalent fluid weight value contains a factor of safety of 1.5. The upper 1 foot of soil should be ignored unless the soil adjacent to the shallow foundations are covered by slabs or pavements.
- Design footings to bear below a reference line drawn upward and outward on a 1.5 horizontal to 1 vertical (1.5H:1V) slope from the bottom of any adjacent utilities or other underground structures including drainage basins.

6.1.3 Ground Improvement

Ground improvement systems are designed and constructed by specialty contractors with proprietary equipment and/or proprietary construction techniques. Therefore, Haley & Aldrich can only provide preliminary recommendations in this report. Final ground improvement design-build drawings, calculations, and specifications should be provided by the ground improvement contractor and reviewed and approved by Haley & Aldrich. Typically ground improvement options to mitigate the seismic hazards encountered at the project site include rigid inclusions using non-driven, non-vibratory methods, such as drilled displacement elements consisting of columns of unreinforced sand-cement slurry and/or lean concrete (i.e., rigid inclusions), or DSM columns, which consists of cementitious grout that is blended into the underlying soil to form soil-cement columns.

Recommendations specific to ground improvement design are provided as follows:

- Due to the presence of liquefiable soil, ground improvement elements should be grouted or consist of concrete. The designer should evaluate the grouted/concrete zone adjacent to the footing bottoms, but ground improvement elements should not be in contact with footings.
- The ground improvement design should be developed to limit settlements of footings under design working loads due to compression of the improved soil to 1 inch or less (including settlement in underlying soil from which ground improvements derive their support), with a maximum differential settlement over a 50-foot distance of ½ inch. Settlement calculations should be provided in the Contractor’s design submittals. For design purposes, it can be assumed that about one-half of foundation settlements will likely occur during construction as structure dead loads are placed on the foundations. The remaining settlements are anticipated to occur within about 5 years after building construction unless additional loads are added to the currently planned structure.
- The ground improvement design should be capable of transferring the building loads below soil layers that are susceptible to liquefiable settlement and into competent granular soil encountered at depths of about 35 to 40 feet below the existing grade.
- The ground improvement does need to be designed to provide support to slabs-on-grade, unless a mat slab is selected.

- The ground improvement does not need to be design to provide support for sub-floor utilities.
- If ground improvement elements are rigid full-height, a “footing pad” must be provided as a transition layer between the ground improvement elements and the bottoms of footings. The thickness and composition of the footing pad should consider the element capacity, stresses induced on the footing bases, compression of the pad, and other factors.
- Other details of the ground improvement including footing pad requirements should be outlined in the project specifications and specialty contractors’ proposals. We recommend a minimum footing pad thickness of 8 inches, assuming it is constructed of crushed stone or similar aggregate/granular material. Detailed design submittals should be provided for the ground improvement system, for review by Haley & Aldrich.
- Testing (modulus and/or load testing) should be performed on ground improvement elements to confirm the design assumptions. A minimum of three elements will need to be tested prior to the start of production installation. The elements will need to be taken up to a minimum of 2 times the allowable design capacities.
- The ground improvement contractor(s) should provide: 1) design capacity calculations associated with their proprietary ground improvement system(s); 2) a plan showing the ground improvement element locations and identification numbers; 3) details showing the material specifications, depths and diameters of the ground improvement elements; 4) details showing the material specifications, locations and thickness of the load transfer layers; and 5) details and descriptions of the load testing program and the quality control/quality assurance program used to confirm the design capacities and settlement behavior of the ground improvement elements.

6.1.4 Mat Slab Bearing without Improved Ground

We understand that the ancillary building mats will be founded at least 2 feet below existing grade and supported on a minimum of 3 feet of engineered fill. At this depth, we recommend a maximum net allowable bearing capacity of 1,000 pounds per square foot (psf) for the entire mat, with local maximum net allowable bearing capacities of 1,500 and 2,500 psf for areas no greater than 2,500 square feet and 500 square feet, respectively. This net allowable bearing capacity includes a safety factor of at least 3 with respect to shear failure of the foundation soils. The mat should be designed using a modulus of subgrade reaction of 55 pounds per square inch (psi) per inch of deflection pending confirmation from the ground improvement specialty contractor. For transient loading conditions, such as wind and earthquake, the net allowable bearing pressure may be increased by a factor of one-half.

The mat should be placed neat against native soil or engineered fill. It is critical that the mat excavation not be allowed to dry before placing concrete. If shrinkage cracks appear in the excavation, the excavation should be thoroughly moistened to close all cracks prior to concrete placement. The excavation should be monitored by a representative of Haley & Aldrich, Inc. for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. If soft or loose materials are encountered at the bottom of the excavation, they should be removed and replaced with either engineered fill or lean concrete. In addition, if dry or soft materials are encountered at the bottom of excavation, the material should be removed and also backfilled with lean concrete as soon as possible following excavation. Depending on the time of year of construction and the contractor’s sequencing, consideration should be given to pouring a 2- to 3-inch lean concrete “rat slab.” The use of lean concrete reduces the disturbance of the soils exposed at the bottom of the excavation to weather and construction activities following excavation.

Lateral loads may be resisted by a combination of friction between the bottom of the mat and by passive pressure against the sides of the mat. For friction resistance, we recommend a coefficient of friction of 0.30 be used for design. Passive resistance may be computed based on an equivalent fluid weight of 300 pcf for mats constructed against competent, native soils or engineered fill. The passive pressures can be assumed to act starting at grade in paved areas and at a depth of 1 foot below grade in unpaved areas. It should be noted that the lateral load resistance values discussed above are only applicable where the concrete for the mat is either placed directly against competent, native soil, engineered fill, or the voids created from the use of forming are backfilled with properly compacted soil. For design purposes, the friction resistance and the passive resistance may be assumed to act simultaneously.

To maintain the desired support, foundations adjacent to utility trenches or other existing foundations should be deepened during the design of the project so that their bearing surfaces are below an imaginary plane having an inclination of 1.5 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent foundations or utility trenches.

6.1.5 Shallow Footings without Ground Improvement

The buildings may be supported on continuous perimeter and isolated interior spread-type footings supported over 36 inches of engineered fill or extended through the fill such as for deep pits. The bottom 12 inches of engineered fill may be reworked in place without over-excavation. Foundations should be bottomed at least 18 inches below the lowest adjacent soil subgrade and should bear on subgrade prepared as discussed in the “Subgrade Preparation” section of this report. Continuous and isolated footings should be at least 24 inches wide and 30 inches square, respectively. We recommend the proposed footings be designed using an allowable bearing pressure of 2,500 psf for dead plus live load conditions. This value contains a factor of safety of at least 2 and may be increased by one-half for total loads, including wind or seismic forces. We estimate that the total and differential foundation movement of new footings under static loading conditions should be less than 1 inch and ½ inch over a 50-foot horizontal distance, respectively.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be maintained in a moist condition until concrete is placed. We should check footing excavations prior to placement of reinforcing steel. Any loose or soft soil exposed beneath footing excavations should be removed, and the resulting overexcavations should be backfilled with compacted fill in accordance with the “Subgrade Preparation” section of this report. Alternatively, the overexcavations may be backfilled with lean concrete or sand/cement slurry with 28-day unconfined compression strength of at least 50 psi.

Lateral loads may be resisted by a combination of friction between the bottom of the footing and by passive pressure against the sides of the footing. For friction resistance, we recommend a coefficient of friction of 0.30 be used for design. Passive resistance may be computed based on an equivalent fluid weight of 300 pcf for footings constructed against competent, native soils or engineered fill. The passive pressures can be assumed to act starting at grade in paved areas and at a depth of 1 foot below grade in unpaved areas. It should be noted that the lateral load resistance values discussed above are only applicable where the concrete for footings is either placed directly against competent, native soil, engineered fill, or the voids created from the use of forming are backfilled with properly compacted soil.

For design purposes, the friction resistance and the passive resistance may be assumed to act simultaneously.

To maintain the desired support, foundations adjacent to utility trenches or other existing foundations should be deepened during the design of the project so that their bearing surfaces are below an imaginary plane having an inclination of 1.5 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent foundations or utility trenches.

6.1.6 Drilled Piers

Drilled piers can be used to provide bearing capacity and resistance to lateral and uplift loads. Drilled shafts should consist of circular, straight shaft, cast-in-place reinforced concrete elements designed to develop their load carrying capacity from shaft friction in alluvial soils. Allowable skin friction values to resist downward loads may be considered as 500 psf acting against the embedded length over the circumferential area. Lateral resistance for the canopies, light standards, fence foundations may be taken as an equivalent fluid weight of 300 pcf with a triangular distribution acting against the embedded length over a width of two diameters. Lateral resistance and skin friction of the upper 1 foot of soil should be disregarded when sizing drilled shaft foundations. The piers should have a minimum depth of 5 feet, a minimum diameter of 12 inches, and a center-to-center spacing of at least three (3) pier diameters. For resistance to uplift loads, the weight of the drilled pier and the reduced skin friction between the piers and native soils or compacted, engineered fill may be used. To resist uplift, 60 percent of the allowable skin friction may be used. A factor of safety of 3.0 was used. A one-third increase is permitted for wind and/or seismic loading.

We recommend steel reinforcement and concrete be placed the same day as the holes are drilled and, ideally, within about 4 to 6 hours upon completion of each drilled hole. The steel reinforcement should be centered in the drilled hole. Concrete used for pier construction should be discharged vertically into the holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction. Based on our subsurface exploration, groundwater is not anticipated within the planned depths of the piers. However, if water more than 10 inches deep is present during concrete placement, either the water needs to be pumped out or the concrete placed into the hole using tremie methods. If tremie methods are used, the end of the tremie pipe must remain below the surface of the in-place concrete at all times. In order to develop the design skin friction value previously provided, concrete used for pier construction should have a slump of 6 to 8 inches. Although not anticipated, casing may be required where the piers extend into loose, sandy soils. The drilling contractor should have casing on hand during drilling operations.

The bottom of the drilled holes should be clean such that no more than 3 inches of loose soil remains in the hole prior to placement of concrete. A representative from the Geotechnical Engineer-of-Record should be present to observe drilled holes to confirm bottom conditions prior to placing steel reinforcement.

6.2 SEISMIC DESIGN

For seismic design in accordance with the provisions of the 2019 California Building Code, we recommend using the seismic design parameters presented on Table 6.

TABLE 6 Seismic Design Parameters	
Categorization/Coefficient	**Design Value**
Standard	ASCE/SEI 7-16
Site Class	D – Stiff Soil
Risk Category	IV
Short Period Spectral Acceleration S_s	1.5
1-Second Period Spectral Acceleration S_1	0.6
Site Coefficient F_a	1.0
Short Period MCE* Spectral Acceleration SM_S*	1.5
Short Period Design Spectral Acceleration S_{DS}*	1
Peak Ground Acceleration PGA (g)	0.571
Site Modified Peak Ground Acceleration PGA_M (g)	0.628
Mean Moment Magnitude M_w**	6.84

* Values obtained from ASCE 7 Hazards Report which uses USGS Seismic Design Maps and is based on the ASCE-7-16 and the 2019 California Building Code.

** Values obtained from the USGS Unified Hazards Tool. Site coordinates of 37.373781°N and 121.931567°W are used.

6.3 SLABS-ON-GRADE

Concrete slabs are anticipated to consist of floor slabs and exterior walkways. The exposed subgrade soils should be moisture conditioned to 2 to 5 percent over optimum moisture and recompacted as discussed in the "Fill Placement and Compaction" section of this report.

Earthquake-induced ground surface settlement may distort and crack the proposed building floor slab and exterior hardscape improvements, such as entryways, exterior slabs and sidewalks, which are not supported on a mat foundation system. Repairs to the floor slab, exterior slabs-on-grade and other site improvements should be expected after a major earthquake. If desired, to reduce this potential damage, mastic joints or other positive separations may be provided to permit relative movements between exterior slabs and proposed structures supported on ground improvement elements. An articulated, ramp-like slab could be provided at areas adjacent to building entries. These slabs would need to be designed to accommodate up to 2½ inches of earthquake-induced settlement between the entryway slab and proposed structures supported on a mat foundation with ground improvement elements, to reduce vertical offsets at the entries.

6.3.1 Interior Floor Slabs

The floor slabs can be supported on grade if the differential settlement from liquefaction, up to 2½ inches over 50 feet, is acceptable. If this potential differential settlement exceeds the structural integrity of the floor, a mat foundation system or a structural slab-on-grade should be used.

Interior concrete slabs subject to vehicle loading should be supported on at least 6 inches of Class 2 Aggregate Base or a mixture of baserock and asphalt provided that they meet the gradation requirements for Caltrans Class 2 Aggregate Base. Thickness and reinforcing of the slab should be designed by the project Structural Engineer, but we suggest a minimum thickness of 5 inches of concrete be used. Special care should be taken to ensure that reinforcement is placed at the slab mid-height, particularly when using welded wire fabric. The slabs should be separated from footings and walls. If this is not possible from a structural standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

For slabs-on-grade that have a moisture sensitive surfacing, we recommend use of a capillary break section. For a capillary break, we recommend that an impermeable membrane (10 mil or thicker) be placed over 6 inches of crushed rock to reduce the migration of moisture vapor through the concrete slab. In order to promote more uniform curing of the slab and to provide protection of the vapor membrane, it is advisable to place 2 inches of fine sand on top of the membrane prior to placing the concrete. The sand should be moisture conditioned slightly prior to placing concrete. The sand may replace an equivalent thickness of capillary break.

For slabs-on-grade without vehicle traffic or moisture sensitive surfacing, we recommend use of a 6-inch section of aggregate base, CalTrans Class 4 or better is acceptable, or an imported “non-expansive” fill.

6.3.2 Exterior Flatwork

Where exterior flatwork is to be constructed, the subgrade surface should be prepared by scarifying the subgrade to a depth of 12 inches. The scarified soil should then be moisture conditioned and recompacted as specified in the “Earthwork” section of this report. Alternately, to assist with a winter construction season, chemical stabilization may be used as described in the “Lime Treatment” section of our report. For more uniform support, 4 inches of sand or gravel can be used beneath the flatwork. Where exterior flatwork will be subjected to vehicle loading, a minimum of 6 inches of Caltrans Class 2 Aggregate Base or a mixture of on-site baserock and asphalt should be placed beneath the flatwork.

6.4 UTILITIES

Utilities may experience differential settlement on the order of 1 to 2½ inches over 50 feet as a result of a design level seismic event. Where this exceeds the capacity of the utility and it is of critical support, the utility should be hung off of the structural slab or mat slab foundation. Consideration should also be given to flexible connections where critical utilities exit or enter a building supported on ground improvement.

6.5 RETAINING WALLS

Deep pits with retaining walls are anticipated for the Tower and landscaping walls for the project. They should be designed to resist both static lateral earth pressures, lateral pressures cause by seismic loading, and additional surcharge pressures associated with vehicular traffic (if appropriate). We recommend that the retaining walls be designed for the more critical of either:

- An at-rest equivalent fluid weight (triangular distribution) of 70 pcf, plus a traffic surcharge as a uniform (rectangular distribution) lateral pressure of 100 psf applied to the entire vertical face of the retaining wall, where vehicular parking, streets and/or driveways are located within a horizontal distance of H, where H is the height of the adjacent retaining wall in feet; or
- An active equivalent fluid weight (triangular distribution) of 50 pcf.

Although not anticipated, where retaining walls are anticipated to be more than 10 feet high, seismic forces will need to be considered.

The above lateral design pressures are based on fully drained walls. Even though the retaining walls will be above the groundwater table, water can still accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines. For walls greater than 4 feet in height, prefabricated drainage material (such as Miradrain® or an approved alternate) may be used behind below-grade and retaining walls. Prefabricated drainage material should be installed in accordance with the manufacturer's recommendations. Retaining walls less than 4 feet in height do not need to consider groundwater.

As an alternative to prefabricated drainage material, a drain rock layer may be used. The drain rock layer should be 1 to 2 feet thick and extend to within 1 foot of the ground surface. Four-inch diameter perforated plastic pipe should be installed (with the perforations facing down) along the base of the walls on a 4-inch-thick bed of drain rock. The pipe should be sloped to drain by gravity to a sump or other drainage facility. Weep holes may also be used if water seepage is permissible in the building. The weep holes should be a minimum of 3 inches in diameter located at no more than 10 feet apart, and a screen placed at the back of the holes if drain rock is used.

Drain rock should conform to Caltrans Class 2 permeable material. Alternatively, locally available, clean, ½- to 3/4-inch maximum size crushed rock or gravel could be used, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi® 140N or an approved alternative.

Although not likely, even with the back drain system, localized wet spots may occur in the walls. If this is undesirable, then the wall should be waterproof.

6.6 EARTHWORK

6.6.1 Site Clearing and Stripping

We understand that some of the existing site improvements will be demolished prior to construction of the new buildings. Site clearing should include removal of existing improvements such as asphalt concrete pavements, deleterious materials, obstructions and underground utilities to be abandoned or relocated. Due to the current site use as a parking lot with solar canopies, it is feasible that concrete foundations or other buried obstructions from the previous site developments may be present below grade and will need to be demolished. Depressions, voids and holes resulting from removal of underground improvements or obstructions should be cleaned and backfilled with engineered fill compacted to the requirements given under the section entitled "Fill Placement and Compaction." A geotechnical engineer should be commissioned to observe during the site clearing and backfilling work.

After clearing, surface vegetation and organic laden soil in existing landscaping areas should be stripped. Soils with an organic content of more than 3 percent by weight or with visible organic matter deemed excessive by Haley & Aldrich should be considered organic. The actual required stripping depth should be determined in the field at the time of construction. For planning purposes, an average stripping depth of 3 inches may be assumed. The stripped, organic-rich material may be stockpiled and used for landscaping purposes, if approved by the project Landscape Architect.

6.6.2 Subgrade Preparation

Soil surfaces to receive engineered fills, concrete slabs-on-grade and pavements should be scarified to a depth of 12 inches, moisture conditioned and compacted in accordance with the recommendations given in the section entitled "Fill Placement and Compaction." In proposed building areas, subgrade preparation should extend at least 5 feet beyond the limits of the proposed exterior of the building foundations and any adjoining flatwork such as canopies. In exterior concrete slab and pavement areas, subgrade preparation should extend at least 2 feet beyond the limits of these improvements.

Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or equipment of similar weight. Moisture conditioning of subgrade soils should consist of adding water if the soils are too dry and allowing the soils to dry if the soils are too wet. After the subgrades are properly prepared, the areas may be raised to design grades by placement of engineered fill.

Wet and/or soft soils should be anticipated during site earthwork construction, especially in areas of existing slabs, pavements and landscaping, and/or during rainy seasons. Wet and soft soil conditions encountered during construction should be stabilized prior to placement of new fill and further construction. Methods for stabilization may include lime treatment and use of geotextile fabric and granular fill. A representative of Haley & Aldrich should evaluate the method of stabilization at the time of construction. Unit cost for such stabilization method should be included in the bid documents.

6.6.3 "Non-Expansive" Fill

Moderately expansive soil was found at the Site. Concrete slabs-on-grade should be constructed on a layer of "non-expansive" fill meeting the requirements presented in the table below. The "non-expansive" fill layer should be at least 6 inches thick and should extend at least 3 feet laterally beyond the outer limits of the slabs. The aggregate base under the building slab and exterior flatwork may be considered as the "non-expansive" fill.

TABLE 7
"Non-expansive" Fill Grading Requirements

Sieve Size	**Percentage Passing Sieve**
3 inch	100
1½ inch	85-100
-#200 Screen	8-40
Atterberg Limits	**Percent**
Plasticity Index	12 or less
Liquid Limit	Less than 30

Highly pervious materials such as pea gravel or clean sands are not recommended because they permit transmission of water to the underlying soils. All on-site or import fill material should be compacted to the recommendations provided for engineered fill in the "Fill Placement and Compaction" section of the report.

Due to the low to moderately expansive nature of the on-site clayey soils, proper moisture conditioning is important. The moisture conditioning should be performed in accordance with the "Fill Placement and Compaction" section. Where low expansion potential soils or baserock in paved areas is used, it should be placed immediately over the prepared subgrade to avoid drying of the subgrade. Prior to the placement of the capillary break or crushed rock material over the engineered fill subgrade for the building pads, the subgrade should be conditioned to the moisture content indicated in the "Fill Placement and Compaction" section. The subgrade for exterior concrete flatwork should be conditioned to the required moisture content prior to their construction and may require additional conditioning if allowed to dry.

Fill materials should be approved by the project geotechnical engineer prior to placement and delivery to the Site. At least five (5) working days prior to importing to the Site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

6.6.4 Material for Fill

Except for organic laden soil, the on-site soil is suitable for use as general engineered fill if it is free of deleterious matter. Soils for use in engineered fill should be inorganic, and free of deleterious materials and hazardous substances. For this project, inorganic soils are soils with an organic content of less than 3 percent by weight or without visible organic matter deemed excessive by Haley & Aldrich.

TABLE 8
General Engineered Fill Grading Requirements

Sieve Size	Percentage Passing Sieve
3 inch	100
1½ inch	85-100

6.6.5 Re-Use of On-Site Material

Any existing asphalt or aggregate base that is removed during demolition may be suitable to be pulverized and mixed with the underlying base for use as engineered fill if it has an organic content of less than 2 percent by dry weight and meets the following requirements presented under the "Material for Fill" Section 6.6.4 of this report.

The processed asphalt concrete/base material may be used as Class 2 Aggregate base if it meets the following requirements:

TABLE 9
Class 2 Aggregate Base Grading Requirements

Sieve Size	Percentage Passing
1 inch	100 min.
¾ inch	35 – 60
No. 4	40 – 90
No. 30	10 – 30
No. 200	2 – 9

Note Quality Requirements:
Sand Equivalent: 25 min R-value: 78 min

Site recycled material may be processed and reused as engineered fill, "non-expansive" fill, or aggregate base if it meets the requirements presented in this report for the specific materials.

6.6.6 Fill Placement and Compaction

Fill materials should be placed and compacted in horizontal lifts, each not exceeding 8 inches in uncompacted thickness. Compaction of fill should be performed by mechanical means only. Due to equipment limitations, thinner lifts may be necessary to achieve the recommended degree of compaction. Placement of fill should be in accordance with Table 10, Summary of Compaction Recommendations.

TABLE 10

Summary of Compaction Recommendations

Area	**Compaction Recommendations** (See Notes 1 through 4)
Subgrade Preparation and Placement of General Engineered Fill, (5) Including Imported "Non-expansive" Fill	Compact upper 12 inches of subgrade and entire fill to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content.
Subgrade Preparation Including Bottom of Footing Excavations	Compact upper 12 inches of subgrade and entire fill to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content.
Trenches (6)	Compact trench backfill to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture. Where trenches will be under the pavement section, flatwork, or other improvements, the upper 12 inches, measured from finished grade of the trench backfill, should be compacted to a minimum of 95 percent compaction.
Exterior Flatwork	Compact upper 12 inches of subgrade to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content. Compact aggregate base to a minimum of 90 percent compaction at or above optimum moisture content. Where exterior flatwork is exposed to vehicular traffic, compact aggregate base to a minimum of 95 percent compaction.
Asphalt Concrete and Concrete Paved Areas	Compact upper 12 inches of subgrade to a minimum of 95 percent compaction at a minimum of 2 percent over optimum moisture content. Compact aggregate baserock to a minimum of 95 percent compaction at near optimum moisture content.
Pervious Asphalt Paved Areas	Compact upper 12 inches of subgrade to a between 88 and 92 percent compaction at a minimum of 2 percent over optimum moisture content. Compact aggregate baserock to a minimum of 95 percent compaction at near optimum moisture content.

Notes:

(1) Depths are below finished subgrade elevation.
(2) All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D-1557 (latest version). All lifts to be compacted shall be a maximum of 8 inches loose thickness.
(3) All compacted surfaces, such as fills, subgrades, and backfills need to be firm and stable, and should be unyielding under compaction equipment.
(4) Where fills, such as backfill placement after removal of existing underground utility lines, are greater than 7 feet in depth, the portion of the fill deeper than 7 feet should be compacted to a minimum of 95 percent compaction.
(5) Includes building pads.
(6) In landscaping areas, this percent compaction in trenches may be reduced to 85 percent. Water jetting or flooding to obtain compaction of backfill should not be permitted.

6.6.7 Trench Excavation and Backfill

We anticipate that excavation of utility trenches can be readily made with conventional excavation equipment. The walls of utility trenches in clayey soils and less than 5 feet in height should be able to stand near vertical with minimal bracing, provided proper moisture content in the soil is maintained. Where excavations are deeper than 5 feet or extend into sandy soils with little or no cohesion, shoring or sloping of the sidewalls at a safe inclination will be required to increase stability. In addition, excavations should be located so that no structures, existing or new, are located above a plane projected 45 degrees upward from any point in the excavation, regardless of whether the trenches are shored or not. All excavations should be constructed in accordance with OSHA safety standards and local jurisdictions. Safety in and around the Site is the responsibility of the general contractor.

Groundwater was encountered in our CPTs at depths as high as 22.5 feet bgs. Historical groundwater elevations for the area on the order of 15 feet below existing grade. Although not anticipated, excavations extending below groundwater will require dewatering. Dewatering should lower the groundwater level to a minimum of 2 feet below the bottom of the excavation. If the soil exposed in the bottom of the excavation is soft or wet, it will be necessary to over-excavate the soil and replace it with crushed rock to create a working platform. The depth of over-excavation should be determined in the field at the time of construction; but for planning purposes, a depth of 12 inches may be assumed.

6.7 SURFACE DRAINAGE

Site grading should provide surface drainage away from the proposed structures and concrete slabs-on-grade to reduce the percolation of water into the underlying soils. Surface water should not be allowed to collect adjacent to structures and along edges of concrete slabs or pavements. Grades should be sloped away from the structures as required in the California Building Code (current edition). Surface water should be directed away from exposed soil slopes. Rainwater on the roof of buildings should be conveyed through gutters, downspouts and closed pipes which discharge directly into the Site storm water collection system or pavement. If discharging onto the pavement, safety of pedestrian traffic should be considered.

6.8 SEEPAGE CONTROL

Where utility lines extend through or beneath perimeter foundations or curbs at pavement areas, permeable backfill should be terminated at least 1 foot from the footings or curbs. Concrete or compacted clayey soil should be used around the pipes to act as a seepage cutoff. Beneath footings, the pipes should be “sleeved” through concrete cutoffs, and the annular space around the pipes should be filled with waterproof caulk. This will help reduce the amount of water seeping through the pervious trench backfill and collecting under the building or pavements.

Where slabs or pavements abut against landscaped areas, the base rock and subgrade soil should be protected against saturation. If landscape water or surface runoff is allowed to seep into the pavement section or subgrade, the service life of the pavement will be reduced. Subdrains behind curbs in landscape areas or vertical cut-off structures may be used to reduce lateral seepage under pavements or slabs from adjacent landscaped areas. Vertical cut-off structures may consist of deepened curb sections, or equivalent, extending at least 2 inches below the baserock/subgrade interface. Subdrains should discharge to a proper outlet or through weep holes in the vertical curbs as determined by the project civil engineer. Cut-off structures should be carefully constructed such that they extend below the base section and are poured neat against undisturbed native soil or compacted clayey fill. The cut-off structures should be continuous. Utility trenches (irrigation lines, electrical conduit, etc.) that extend through or under the curbs should be sealed with compacted clayey soil or poured in-place concrete. In addition, care should be taken to prevent over-watering of landscaped areas.

6.9 WET WEATHER CONSTRUCTION

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations. The current construction plans include building a lime treated pad for winterization; recommendations are provided in the "Lime Treatment" section below.

Earthwork during rainy months will require extra effort and caution by the contractors. The grading contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

Unit cost for stabilization and mitigation of wet soil conditions, such as over-excavation as well as placement of geotextile fabric and engineered fill, should be included in the bid documents.

An addition mitigation measure for reducing the potential for saturated footings is to over-excavate the footings by 2 to 3 inches and place lean concrete, or a "rat slab." Footing bottoms should be approved by the project Geotechnical Engineer-of-Record prior to placement of the rat slab.

6.10 LIME TREATMENT

Stabilization of expansive clays during the winter can be obtained with chemical treatment such as use of lime. This can also substantially increase the strength of the treated material and may be used to reduce pavement sections. A site-specific study for determining the actual percent lime to be used should be performed once an earthworks contractor is chosen and the source of lime selected. For preliminary planning, similar sites have used between 3 to 6 percent lime by weight for stabilization. For winterization, we recommend an 18-inch section of material be processed for lime treatment.

Where excavations for utility lines penetrate the lime treated layer, the utility should be located below the lime treated soil and the resulting excavation should be backfilled to the top of the adjacent lime treated soil with lean concrete or controlled density fill. The remainder of the excavation above the top of the lime treatment should be backfilled as discussed in the applicable sections of these recommendations.

If Dolomitic Quicklime is used, it may be used without admixtures. The same type of lime provided for this study, Dolomitic Quicklime, should be used during construction and should be from the same source. Alternate material sources may be used; however, we recommend additional confirmatory laboratory testing before use during site construction.

If landscaped areas are planned adjacent to the building, measures will be required to remove the lime from these areas immediately following treatment. It should be emphasized that the lime treated pad needs to be covered within 2 weeks following completion of treatment. This time frame can be extended by keeping the treated soil moist on a daily basis. The purpose of covering the treatment is to reduce excess drying and cracking of the lime treated soil. Covering of the pad can consist of 4 to 6 inches of the capillary break material, aggregate base, or engineered fill placed over the subgrade.

The lime treatment should be placed in accordance with the current Caltrans Specifications, but with a minimum compaction of 90 percent compaction for building pads based on ASTM D-1557. The geotechnical engineer should monitor the treatment operations during construction for conformance with the specifications and the recommendations in the "Earthwork" section of this report.

6.11 STORMWATER INFILTRATION

Based on our analysis of percolation test data in general accordance with percolation method guidelines[4] (see "Percolation Tests," Section 3.1.3), the adjusted long-term infiltration rate for stormwater infiltration features with invert depths at about 4.5 feet bgs is 0.1 inches per hour (see results in Appendix C). The adjusted long-term infiltration rates include a factor of safety (CFv) of 2 for moderate site variability, and a factor of safety (CFs) of 2 for long-term siltation of infiltration basins, in addition to an adjustment factor (Rf) specific to the borehole percolation test method. This is consistent with a poor drainage material including a sandy, silty, clay matrix. Tests from PT-2 and PT-3 were performed in fill near the former railroad tracks are not considered representative of the site general conditions. The results are included in Appendix C but are not reflected in our recommendation.

The effective infiltration rate of finished stormwater infiltration features can vary significantly from the rates estimated from preliminary percolation tests. The following activities may diminish the infiltration rate of proposed stormwater features and should be avoided:

- Placement of artificial fill within the stormwater infiltration feature during grading, especially placement of fill materials with poor drainage properties.
- Allowing construction runoff containing fine-grained soils to drain into the feature and cause siltation during site grading.
- Grading methods that result in smearing or compaction of soils at the feature or basin invert, including compaction by driving of heavy equipment over the area.
- Design and siting of infiltration features at locations or elevations significantly different from those tested.

6.12 FLEXIBLE PAVEMENT DESIGN

Pavements for this project will consist of asphalt concrete roadway extensions. The pavement design presented herein assumes the pavement subgrade soil will be similar to the near surface soils described in the boring logs. This assumption is based on our anticipation that grading and soil removal in the paved areas will be minimal. If site grading exposes soil other than that assumed, or import fill is used to construct pavement subgrades, we should perform additional tests to confirm or revise the recommended pavement sections for actual field conditions.

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4 Percolation tests were performed in general conformance with the guidelines presented in the County of Los Angeles Department of Public Works Administrative Manual GS200.1.

Asphalt pavement sections for this project have been calculated using Caltrans Flexible Pavement Design Method.[5] Based on the materials encountered during the field exploration borings and laboratory testing an R-Value of 10 was used to develop recommendations for the pavement sections. Alternative pavement sections for different Traffic Indices (TIs) are presented below. Each TI represents a different level of use. The owner or designer should determine which level of use best reflects the project and select appropriate pavement sections.

TABLE 11
Flexible Pavement Section Recommendations R-Value = 10

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Total Thickness (inches)
5.0	2.5	10.0	12.5
6.0	3.0	12.5	15.5
7.0	4.0	14.0	18.0
8.0	4.5	17.0	21.5
9.0	5.5	19.0	24.5

For pavement sections supported on 12 inches of existing subgrade soil scarified and compacted in-place to a minimum of 95 percent relative compaction (based on ASTM 1557 – latest edition) at a moisture content a minimum of 2 percent over optimum.

Note: AC = Type A or B Asphalt Concrete
AB = Class 2 Aggregate Base (Minimum R-Value = 78)

We recommend that the subgrade soil, over which the pavement sections are to be placed, be moisture conditioned and compacted according to the recommendations in the "Fill Placement and Compaction" section of this report. Subgrade preparation should extend a minimum of 2 feet laterally beyond the back of curb or edge of pavement. Paved areas should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water ponding should not be allowed anywhere on the site during or after construction. We recommend that the pavement section be isolated from non-developed areas and areas of intrusion of irrigation water from landscaped areas. Concrete curbs should extend a minimum of 2 inches below the baserock and into the subgrade to provide a barrier against drying of the subgrade soils, and a reduction of migration of landscape water into the pavement section. Weep holes with 4 feet on-center spacing should also be provided. In lieu of the weep holes, a more effective system is to install subdrains behind the curbs.

6.13 PERVIOUS ASPHALT PAVEMENT

Pervious asphalt pavements for this project are planned for portions of the parking areas. The pavement design presented herein assumes the pavement subgrade soil will be similar to the near surface soils described in the boring logs. This assumption is based on our anticipation that grading and soil removal in the paved areas will be minimal. If site grading exposes soil other than that assumed, or import fill is used to construct pavement subgrades, we should perform additional tests to confirm or revise the recommended pavement sections for actual field conditions.

[5] Caltrans Highway Design Manual, Chapter 630, Flexible Pavements, 2017.

Pervious asphalt pavement sections for this project have been calculated using Caltrans Pervious Pavement Design Guidance.[6] These sections include consideration of the structural elements to support the traffic loading only and <u>do not</u> account for reservoir capacity needed for storm water infiltration. Additional capacity may be generated from increasing the thickness of the reservoir layer. Based on the materials encountered during the field exploration borings and laboratory testing an R-Value of 10 was used to develop recommendations for the pavement sections. A Traffic Index of 5 with a Category B for parking areas for passenger vehicles. The owner or designer should confirm these are an appropriate level of use best reflects the project.

TABLE 12
Pervious Asphalt Pavement Section Recommendations R-Value = 10

Traffic Index	OGFC (inches)	ATPB (inches)	AB Reservoir (inches)	Total Thickness (inches)
5.0	1.2	7.0	8.0	16.2

For pervious pavement sections supported on layer of Mirafi 140N (or equivalent) geotextile fabric over 12 inches of existing subgrade soil scarified and compacted in-place to between 88 and 92 percent relative compaction (based on ASTM 1557 – latest edition) at a moisture content a minimum of 2 percent over optimum.

Note: OGFC = Open Graded Friction Course (Caltrans Specifications Section 39-2.04)
ATPB = Asphalt Treated Permeable Base (Caltrans Specifications Section 29-2)
AB = Class 4 Aggregate Base (Caltrans Specifications 26 with a Minimum R-Value = 50)

We recommend that the subgrade soil, over which the pavement sections are to be placed, be moisture conditioned and compacted according to the recommendations in the “Fill Placement and Compaction” section of this report. Subgrade preparation should extend a minimum of 2 feet laterally beyond the back of curb or edge of pavement. The following conditions should be considered for areas designated for pervious asphalt pavement:

- Paved areas should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water ponding should not be allowed anywhere on the site during or after construction.
- Pavement sections be isolated from landscape areas reduce the potential for debris and sediment leading to clogging.
- Paved areas be outside of close proximity to structural foundations to avoid infiltration of the soils below the foundations.
- Routine and long-term maintenance which as vacuuming will be needed to maintain the hydraulic function.
- Concrete curbs should be included for edge support.
- A concrete curb should be placed between asphalt concrete sections and pervious asphalt pavement to saturation of the subgrade below the asphalt concrete.
- Concrete curbs should extend a minimum of 2 inches below the baserock and into the subgrade to provide a barrier against drying of the subgrade soils, and a reduction of migration of landscape water into the pavement section.

6 Caltrans Storm Water Quality Handbook, Pervious Pavement Design Guidance, October 2013.

6.14 CORROSIVITY

Previous testing by CERCO Analytical of Concord, California resulted in classifying the soil as "corrosive." The results of the corrosion testing and a copy of CERCO's brief analysis of the results are presented in Appendix D. Since we are not corrosion specialists, if corrosion is a concern, a corrosion testing firm should be contacted for specific design details.

7. Supplemental Geotechnical Services

The final project plans and specifications should be reviewed by Haley & Aldrich prior to construction to check that they are in general conformance with the intent of our recommendations. During construction, we should observe and document the pre-production pilot test programs for the ground improvement elements, the installation of the temporary shoring system and the condition of the building foundation subgrade. In addition, we should observe and test the compaction of the exposed soil subgrade and any new fill placed at the Site. These observations will allow us to check that the contractor's work conforms to the geotechnical aspects of the plans and specifications and ensure that the foundation system(s) are constructed in accordance with the project plans and specifications and our design recommendations.

8. Limitations

This report has been prepared for specific application to the proposed construction as understood at this time. In the event that changes in the nature, design, or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid, unless the changes are reviewed by Haley & Aldrich and the conclusions and recommendations of this report are modified or verified in writing.

The geotechnical analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations appear at that time, it may be necessary to re-evaluate the recommendations of this report.

This report is prepared for the exclusive use of Ten Over Studio and their subconsultants in connection with the design and construction of the Fire Training Center. There are no intended beneficiaries other than Ten Over Studio and their subconsultants. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the agreement or the report. Use of this report by any person or entity other than Ten Over Studio and their subconsultants for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from Ten Over Studio and from Haley & Aldrich. Use of this report by such other person or entity without the written authorization of Ten Over Studio and Haley & Aldrich shall be at such other person's or entity's sole risk and shall be without legal exposure or liability to Haley & Aldrich.

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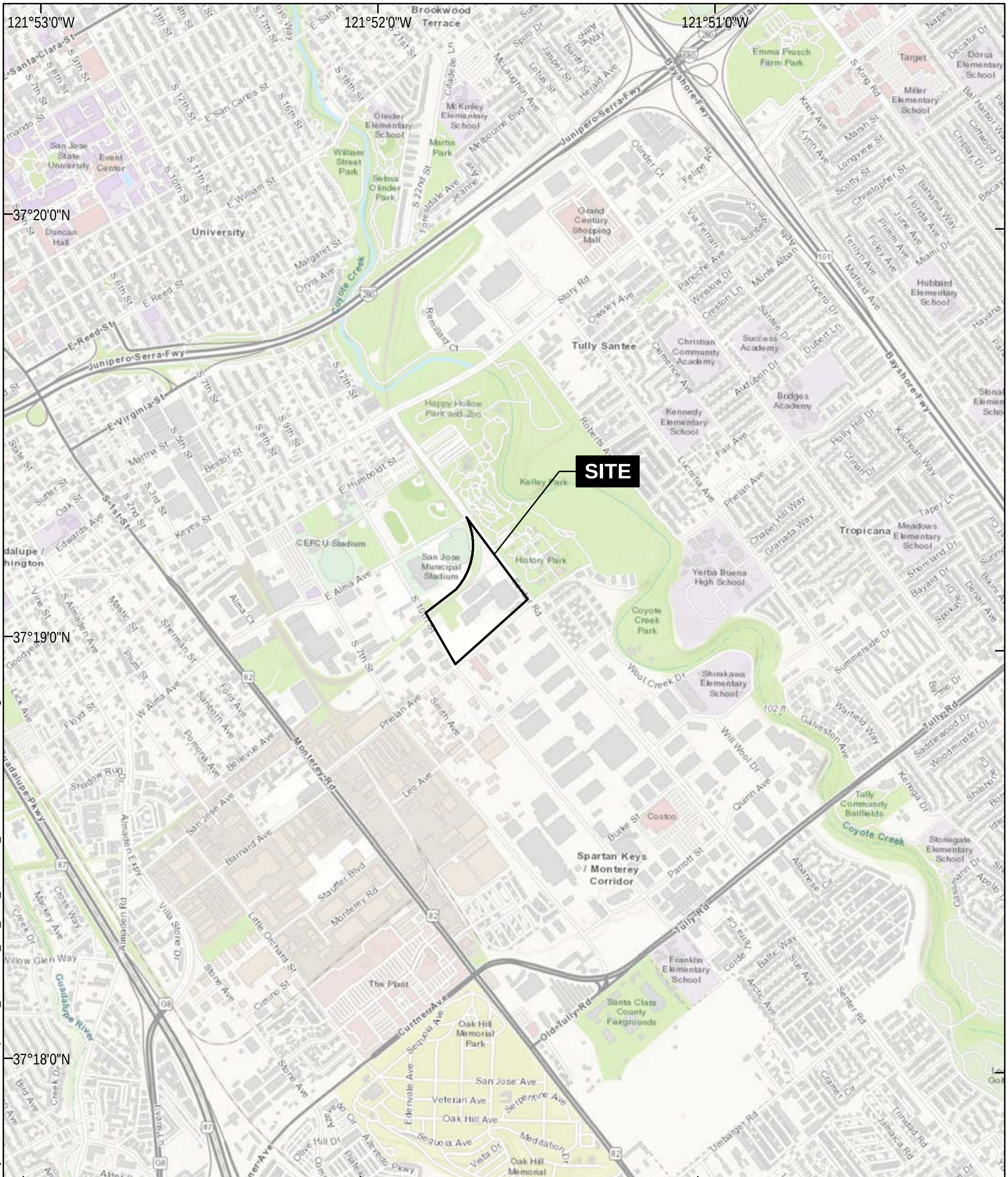
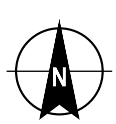
FIGURES





MAP SOURCE: ESRI
SITE COORDINATES: 121°55'53.53"W 37°22'25.60"N

HALEY ALDRICH

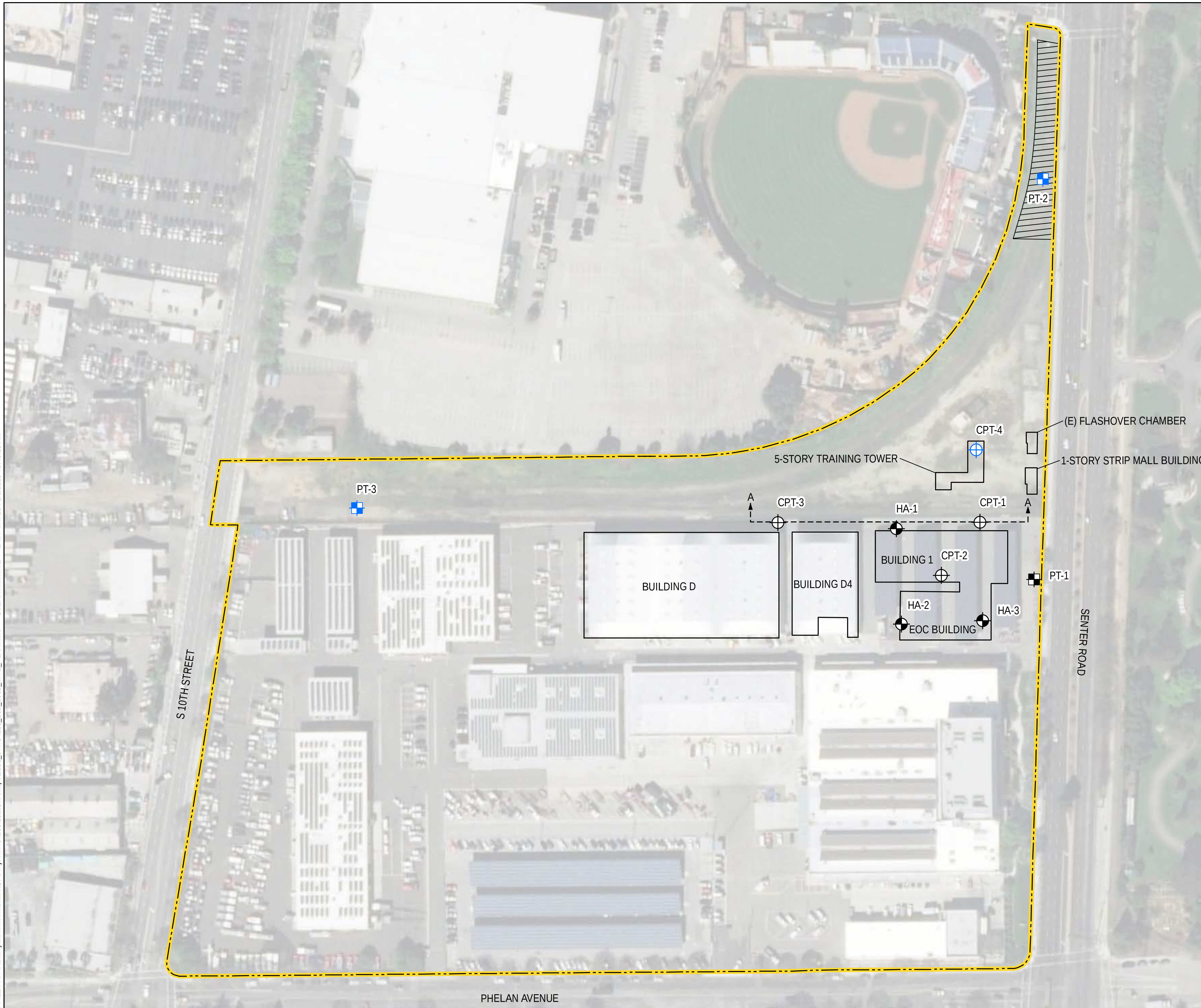
SAN JOSE FIRE TRAINING CENTER RELOCATION
SAN JOSE, CALIFORNIA

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
JANUARY 2020

FIGURE 1

GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\134258\002\GIS\Maps\2020_01\134258_002_0001_PROJECT_LOCUS.mxd — USER: afeigl — LAST SAVED: 1/14/2020 10:37:16 AM



LEGEND

SOIL BORING
CONE PENETRATION TEST (CPT) [PHASE 1]
PERMEABILITY TEST [PHASE 1]
CONE PENETRATION TEST (CPT) [PHASE 2]
PERMEABILITY TEST [PHASE 2]
SITE BOUNDARY
BUILDING OUTLINE
POTENTIAL LOCATION OF BIOSWALE

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. PARCEL DATA SOURCE: SANTA CLARA COUNTY
3. AERIAL IMAGERY SOURCE: ESRI

0 160 320
SCALE IN FEET

HALEY ALDRICH

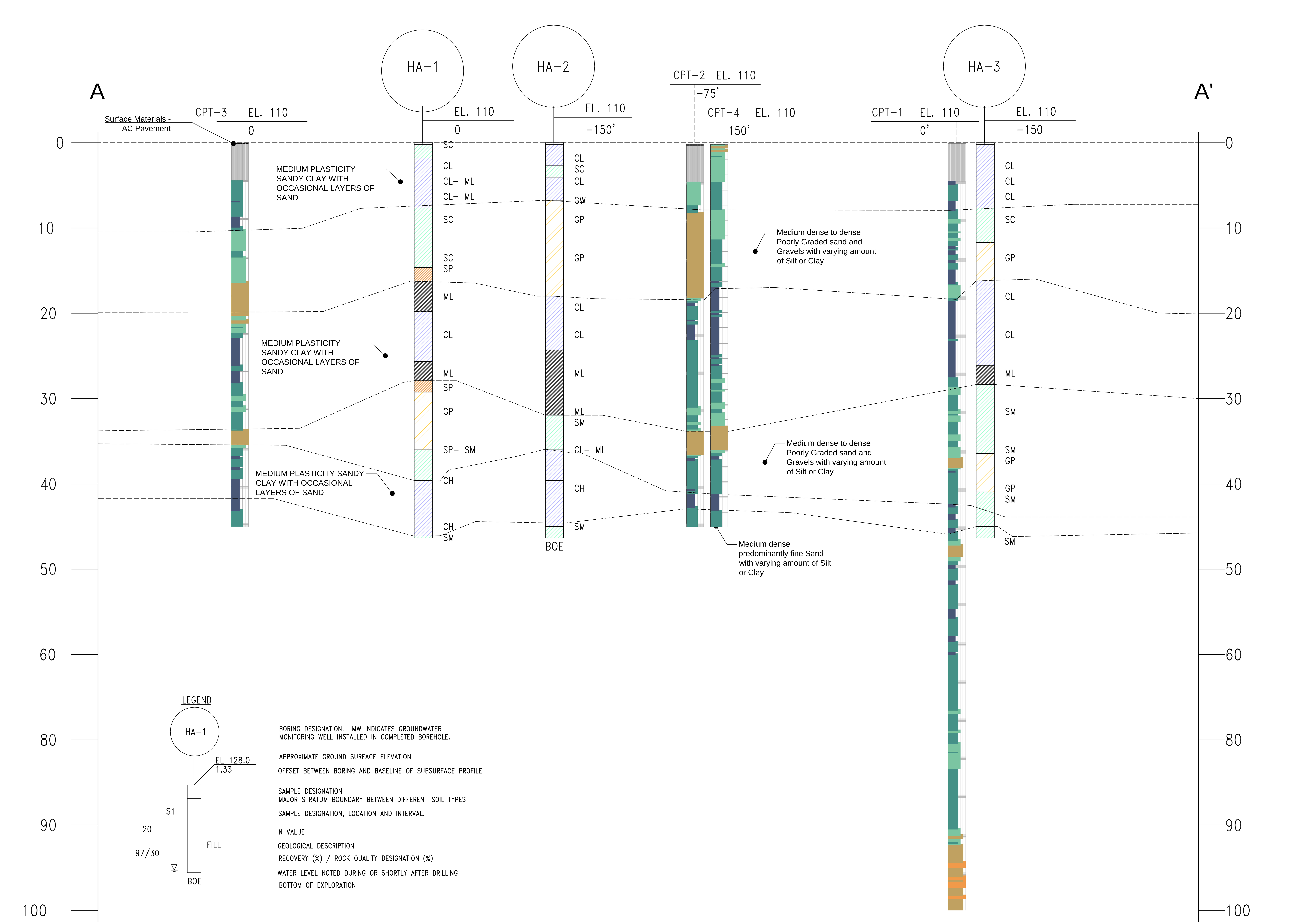
SAN JOSE FIRE TRAINING CENTER RELOCATION
SAN JOSE, CALIFORNIA

SITE PLAN

MAY 2020

FIGURE 2

GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\134258\002\GIS\Maps\2020_04\134258_002_0002_SITE_PLAN.mxd — USER: hwachholz — LAST SAVED: 5/6/2020 10:46:29 AM



NOTES

1. Soil Profile reflects generalizations and assumptions based on CPT and Rotary wash boring data. All elevations and depths are approximate.

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San Jose Airport Aircraft Fire Fighting Facility (ARFF)
San Jose, CA

Subsurface Soil Profile
A:A'

SCALE: NTS
March 2020

FIGURE 3

APPENDIX A

Boring Logs

GEOTECHNICAL TEST BORING REPORT

Boring No. HA-1

Project	San Jose Fire Training Center Relocation	File No.	134258-002
Client	City of San Jose	Sheet No.	1 of 2
Contractor	Pitcher Drilling Services, Inc.	Start	18 November 2019
		Finish	18 November 2019
		Driller	W. Halai
		H&A Rep.	R. Mandzulashvili
		Elevation	0.0 ft
		Datum	
		Location	See Boring Location Plan
			N 1
			E 0

		Drilling Equipment and Procedures
Boring Diameter (in.)	4.0	Rig Make & Model: Fraste XL Track-Mounted
Hammer Type	Automatic Hammer	Bit Type: Tricone Roller Bit
Hammer Weight (lb)	140	Drill Mud: Water/Bentonite
Hammer Fall (in.)	30	Casing:
		Hoist/Hammer: Automatic Hammer
		PID Make & Model: N/A

Depth (ft)	Sample Type	Recovery (in.)	Sampler Blows per 6 in.	Stratum Change Elev/Depth (ft)	USCS Symbol	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (GROUP NAME, density/consistency, color, max. particle size*, structure, odor, moisture, optional descriptions GEOLOGIC INTERPRETATION)	Dry Density (pcf)	Moisture (%)	Fines (%)	Elevation (ft)
0				-0.3 0.3		3-in. ASPHALT PAVEMENT				0
					SC	Brown coarse to fine SAND with gravel				
				-2.0 2.0	CL	Dark brown CLAY (high plasticity) with sand, moist				
5	MCS	10	2 5 6	-5.0 5.0	CL- ML	Stiff brown SILT & CLAY (nonplastic) with medium to fine sand, moist PI=5 LL=25				-5
	MCS	9	3 4 5		CL- ML	Medium stiff brown SILT & CLAY (nonplastic) with medium to fine Sand, moist				
10	SPT	5	3 5 5	-8.5 8.5	SM	Medium dense brown medium to fine Slity SAND, predominantly fine sand, moist			41	-10
15	SPT	4	2 2 4		SM	Loose brown medium to fine SAND with clay and silt, moist				-15
				-16.3 16.3	SP	Loose brown coarse to fine SAND, few silt at SPT tip				
20	SPT	13	0 0 2	-18.0 18.0	ML	Soft brown mottled lean clayey SILT (low plasticity) with trace fine gravel, wet PP=0.75 tsf			94	-20

Water Level Data						Sampler Type Legend	Summary
Date	Time	Elapsed Time (hr.)	Depth (ft) to: Bottom of Casing	Bottom of Hole	Water	SPT - Standard Penetration Test Sampler (1.38-in ID) MCS - Modified California Sampler (2.43-in ID) SHELBY TUBE - Thin-walled Sampler (3-in ID) GRAB - Grab Sample	Overburden (ft) 51.5 Rock Cored (ft) 0.0 Samples 11
							Boring No. **HA-1**

Field Tests: **Dilatancy**: R - Rapid S - Slow N - None **Toughness**: L - Low M - Medium H - High **Plasticity**: N - Nonplastic L - Low M - Medium H - High **Dry Strength**: N - None L - Low M - Medium H - High V - Very High

***Note: Maximum particle size (mps) is determined by direct observation within the limitations of sampler size.**

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

GEOTECHNICAL TEST BORING REPORT

HALEY ALDRICH

Boring No. HA-1
File No. 134258-002
Sheet No. 2 of 2

Depth (ft)	Sample Type	Recovery (in.)	Sampler Blows per 6 in.	Stratum Change Elev/Depth (ft)	USCS Symbol	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (GROUP NAME, density/consistency, color, max. particle size*, structure, odor, moisture, optional descriptions GEOLOGIC INTERPRETATION)	Dry Density (pcf)	Moisture (%)	Fines (%)	Elevation (ft)
25	MCS	12	4 6 7		CL	Stiff to very stiff dark brown CLAY (medium plasticity), trace fine sand, wet PP=2.0 tsf				-25
				-28.5 28.5						
30	MCS	15	2 2 8		ML	Medium stiff brown clayey SILT, little medium to fine sand, wet PP=1.25 tsf			91	-30
				-31.0 31.0	SP	Brown coarse to fine SAND at tip of SPT				
				-32.5 32.5		Gravel Cuttings				
35	SPT	9	10 11 12		GP	Medium dense brown coarse to fine GRAVEL, some coarse to fine sand, trace silt, wet				-35
						Gravel caving in, install 4-in. FJ casing				
40	SPT	7	13 14 7	-40.0 40.0	SP- SM	Medium dense brown coarse to fine SAND with medium to fine gravel, few to little silt, grading with gravel, wet			11	-40
					GP	Rig chatter while advancing from 41.5 to 44.0 ft.				
				-44.0 44.0	CH	Clay cuttings on the drill bit				
45	SPT	14	4 4 6			Stiff gray fat CLAY (highly plastic) PP=1.0 tsf				-45
50	MCS	14	4 7 11		CH	Soft to stiff gray fat CLAY (highly plastic), grading to little to trace sand PP=1.5 tsf				-50
				-51.3 51.3	SM	Brown medium to fine SAND, little silt				
				-51.5 51.5		BOTTOM OF EXPLORATION 51.5 FT				
						Notes: 1. Tremie grouted.				
55										-55
60										-60

NOTE: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

Boring No. HA-1

GEOTECHNICAL TEST BORING REPORT

Boring No. HA-2

Project	San Jose Fire Training Center Relocation	File No.	134258-002
Client	City of San Jose	Sheet No.	1 of 2
Contractor	Pitcher Drilling Services, Inc.	Start	18 November 2019
		Finish	19 November 2019
		Driller	W. Halai
		H&A Rep.	R. Mandzulashvili
		Elevation	0.0 ft
		Datum	
		Location	See Boring Location Plan N 0.000000001 E 0

		Drilling Equipment and Procedures
Boring Diameter (in.)	4.0	Rig Make & Model: Fraste XL Track-Mounted
Hammer Type	Automatic Hammer	Bit Type: Tricone Roller Bit
Hammer Weight (lb)	140	Drill Mud: Water/Bentonite
Hammer Fall (in.)	30	Casing:
		Hoist/Hammer: Automatic Hammer
		PID Make & Model: N/A

Depth (ft)	Sample Type	Recovery (in.)	Sampler Blows per 6 in.	Stratum Change Elev/Depth (ft)	USCS Symbol	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (GROUP NAME, density/consistency, color, max. particle size*, structure, odor, moisture, optional descriptions GEOLOGIC INTERPRETATION)	Dry Density (pcf)	Moisture (%)	Fines (%)	Elevation (ft)
0				-0.3 0.3		3-in. ASPHALT PAVEMENT				0
					CL	Clay while hand augering				
	MCS	2	8 6 9	-3.0 3.0	SC	Medium dense brown clayey SAND, little gravel, dry to moist				
5	SPT	4	1 1 2	-4.5 4.5	CL	Soft brown CLAY (medium to high plasticity), moist to wet (trapped water?) PI=17 LL=37				-5
	MCS	12	6 20 20	-7.5 7.5	GW	Dense brown coarse to fine GRAVEL with coarse to fine sand, little silt, moist				
10	SPT	8	8 17 28		GP	Dense brown coarse to fine GRAVEL with coarse to fine sand, little silt, moist Rig chatter from 11.5 to 15.0 ft Cave in 3 ft, install 4-in. FJ				-10
15	SPT	9	11 20 21		GP	Dense brown poorly-graded GRAVEL with coarse to fine sand, little silt, moist Install 4-in. FJ casing				-15
20	SPT	9	3 1 1	-20.0 20.0	ML	Soft brown Clayey Silt (medium plasticity), wet PP=0.75 tsf				-20

Water Level Data						Sampler Type Legend	Summary
Date	Time	Elapsed Time (hr.)	Depth (ft) to: Bottom of Casing	Bottom of Hole	Water	SPT - Standard Penetration Test Sampler (1.38-in ID) MCS - Modified California Sampler (2.43-in ID) SHELBY TUBE - Thin-walled Sampler (3-in ID) GRAB - Grab Sample	Overburden (ft) 51.5 Rock Cored (ft) 0.0 Samples 12

Boring No. HA-2

Field Tests: **Dilatancy**: R - Rapid S - Slow N - None **Plasticity**: N - Nonplastic L - Low M - Medium H - High
Toughness: L - Low M - Medium H - High **Dry Strength**: N - None L - Low M - Medium H - High V - Very High

***Note: Maximum particle size (mps) is determined by direct observation within the limitations of sampler size.**

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

Depth (ft)	Sample Type	Recovery (in.)	Sampler Blows per 6 in.	Stratum Change Elev/Depth (ft)	USCS Symbol	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (GROUP NAME, density/consistency, color, max. particle size*, structure, odor, moisture, optional descriptions GEOLOGIC INTERPRETATION)	Dry Density (pcf)	Moisture (%)	Fines (%)	Elevation (ft)
25	MCS	9	2 3 3		CL	Medium stiff grayish brown CLAY (medium plasticity), trace fine sand, wet PI=18 LL=34				-25
				-27.0 27.0						
30	SPT	12	1 3 3		ML	Soft to medium stiff brown SILT, some medium to fine silty SAND, wet				-30
35	SPT	16	2 3 8	-35.5 35.5	ML SM	Similar to above top 6 in. Medium dense brown medium to fine SAND, trace silt/clay, wet			39	-35
40	SPT	14	2 2 3	-40.0 40.0	CL- ML	Medium stiff brown Silty SAND with Gravel				-40
				-42.0 42.0	GP	Drill rig chatter and gravel cuttings from 42.0 to 44.0 ft				
				-44.0 44.0		Clay cuttings				
45	MCS	16	3 11 12		CH	Stiff to very stiff dark brown fat CLAY (high plasticity), wet PP=1.75 tsf				-45
50	MCS	12	4 8 11	-50.0 50.0	SM	Medium dense brown medium to fine SAND, few to little silt/clay				-50
				-51.5 51.5		BOTTOM OF EXPLORATION 51.5 FT Notes: 1. Tremie grouted.				
55										-55
60										-60

GEOTECHNICAL TEST BORING REPORT

HALEY ALDRICH

Boring No. HA-3

Project	San Jose Fire Training Center Relocation	File No.	134258-002
Client	City of San Jose	Sheet No.	1 of 2
Contractor	Pitcher Drilling Services, Inc.	Start	19 November 2019
		Finish	19 November 2019
		Driller	W. Halai
		H&A Rep.	R. Mandzulashvili
		Elevation	0.0 ft
		Datum	
		Location	See Boring Location Plan
		N 0	
		E 1	

		Drilling Equipment and Procedures
Boring Diameter (in.)	4.0	Rig Make & Model: Fraste XL Track-Mounted
Hammer Type	Automatic Hammer	Bit Type: Tricone Roller Bit
Hammer Weight (lb)	140	Drill Mud: Water/Bentonite
Hammer Fall (in.)	30	Casing:
		Hoist/Hammer: Automatic Hammer
		PID Make & Model: N/A

Depth (ft)	Sample Type	Recovery (in.)	Sampler Blows per 6 in.	Stratum Change Elev/Depth (ft)	USCS Symbol	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (GROUP NAME, density/consistency, color, max. particle size*, structure, odor, moisture, optional descriptions GEOLOGIC INTERPRETATION)	Dry Density (pcf)	Moisture (%)	Fines (%)	Elevation (ft)
0				-0.3 / 0.3		3-in. ASPHALT PAVEMENT				0
	MCS	10	5 7 6		CL	Stiff brown CLAY with fine sand (medium to high plasticity), moist UC=3338 psf				
5	MCS	6	3 3 5		CL	Medium stiff brown CLAY (medium plasticity), trace sand PI=17 LL=37				-5
	MCS	6	2 3 5	-8.5 / 8.5	CL	Medium stiff brown sandy CLAY (low plasticity) PP=1.25 tsf				
10	SPT	8	2 2 6		SC	Loose brown clayey SAND, trace gravel, wet			30	-10
				-13.0 / 13.0		Install 15.0 ft casing before sampling at 15.0 ft				
15	SPT	2	10 10 15		GP	Medium dense brown GRAVEL with coarse to fine sand, trace silt, wet Advance casing to 20.0 ft				-15
				-18.0 / 18.0						
20	SPT	8	0 1 1		CL	Soft brown fat CLAY (high plasticity), wet				-20

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (ft) to: Bottom of Casing	Bottom of Hole	Water

Sampler Type Legend

SPT - Standard Penetration Test Sampler (1.38-in ID)
MCS - Modified California Sampler (2.43-in ID)
SHELBY TUBE - Thin-walled Sampler (3-in ID)
GRAB - Grab Sample

Summary

Overburden (ft)	51.5
Rock Cored (ft)	0.0
Samples	12

Boring No. HA-3

Field Tests: **Dilatancy**: R - Rapid S - Slow N - None **Toughness**: L - Low M - Medium H - High **Plasticity**: N - Nonplastic L - Low M - Medium H - High **Dry Strength**: N - None L - Low M - Medium H - High V - Very High

***Note: Maximum particle size (mps) is determined by direct observation within the limitations of sampler size.**

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

Depth (ft)	Sample Type	Recovery (in.)	Sampler Blows per 6 in.	Stratum Change Elev/Depth (ft)	USCS Symbol	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (GROUP NAME, density/consistency, color, max. particle size*, structure, odor, moisture, optional descriptions GEOLOGIC INTERPRETATION)	Dry Density (pcf)	Moisture (%)	Fines (%)	Elevation (ft)
25	MCS	9	1 5 7		CL	Stiff brown CLAY (medium plasticity), trace fine sand, wet PP=1.25 tsf				-25
				-29.0 29.0						
30	MCS	18	1 2 2		ML	Medium stiff brown SILT with fine sand, dilatancy rapid, wet UC=1445 psf				-30
				-31.5 31.5						
35	MCS	14	1 4 4		ML	Medium stiff brown SILT with fine sand, dilatancy rapid, wet			88	-35
40	SPT	12	2 12 11	-40.5 40.5	ML	Similar to above				-40
					GP	Medium dense brown coarse to fine GRAVEL with coarse to fine sand, little silt				
45	SPT	10	12 3 6	-45.5 45.5	GP	Similar to above				-45
					SM	Loose to medium dense gray medium to fine SAND, few silt, wet				
50	SPT	18	9 11 14		SM	Medium dense brown medium to fine Silty SAND, little to some silt				-50
				-51.5 51.5		BOTTOM OF EXPLORATION 51.5 FT				
						Notes: 1. Tremie grouted.				
55										-55
60										-60

NOTE: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

APPENDIX B

Cone Penetration Test Logs

GREGG DRILLING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

November 27, 2019

Haley & Aldrich
Attn: Rati Mandzulashvili

Subject: CPT Site Investigation
SJ Central Service Yard
San Jose, California
GREGG Project Number: 194114MA

Dear Mr. Mandzulashvili:

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 714-863-0988.

Sincerely,
Gregg Drilling, LLC.

CPT Reports Team
Gregg Drilling, LLC.

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)
CPT-02	11/26/2019	CPT-02	-	-	-
CPT-03	11/26/2019	CPT-03	-	-	-
SCPT-01	11/26/2019	SCPT-01	-	-	22.5

Bibliography

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

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, “Geo-Environmental Application of Penetration Testing”, Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

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.

DeGroot, D.J. and A.J. Lutenegger, “Reliability of Soil Gas Sampling and Characterization Techniques”, International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, “Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT”, 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, “Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers” BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

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

GREGG DRILLING, INC.
www.greggdrilling.com

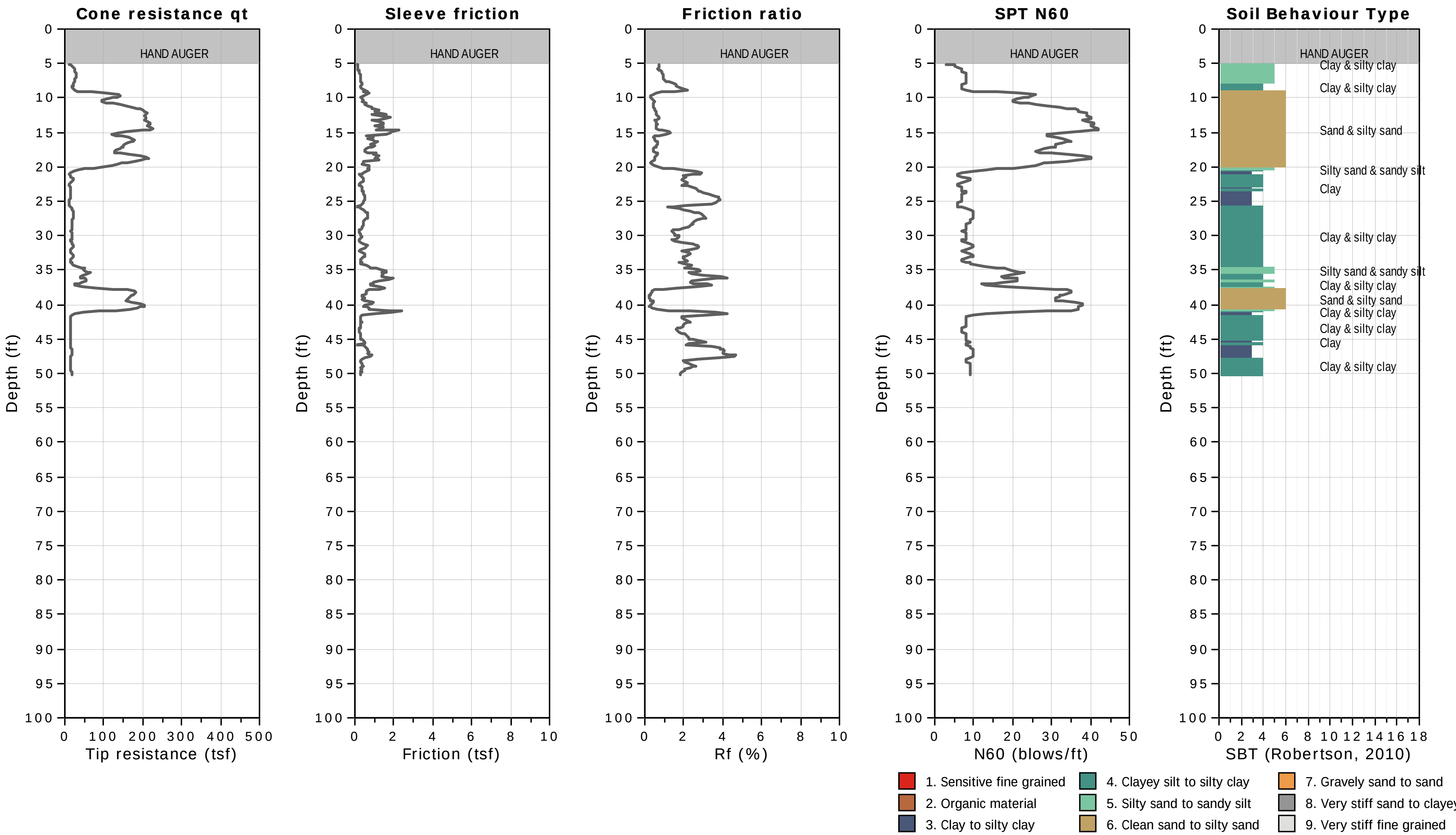
CPT: CPT-02

CLIENT: HALEY & ALDRICH

SITE: SJ CENTRAL SERVICE YARD, SAN JOSE, CA

FIELD REP: RATI MANDZULASHVILI

Total depth: 50.20 ft, Date: 11/26/2019

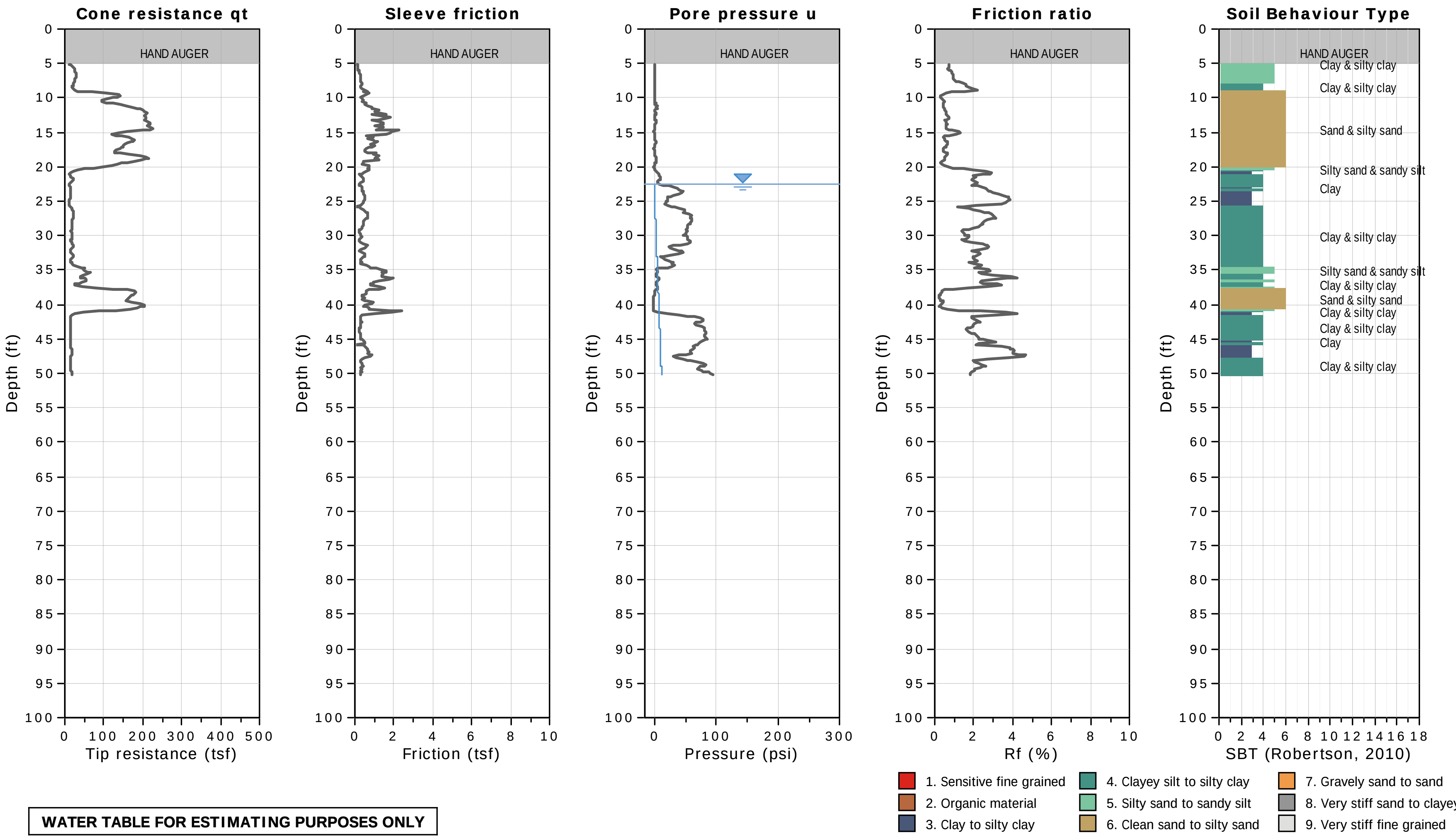


GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-02

CLIENT: HALEY & ALDRICH
SITE: SJ CENTRAL SERVICE YARD, SAN JOSE, CA

FIELD REP: RATI MANDZULASHVILI
Total depth: 50.20 ft, Date: 11/26/2019



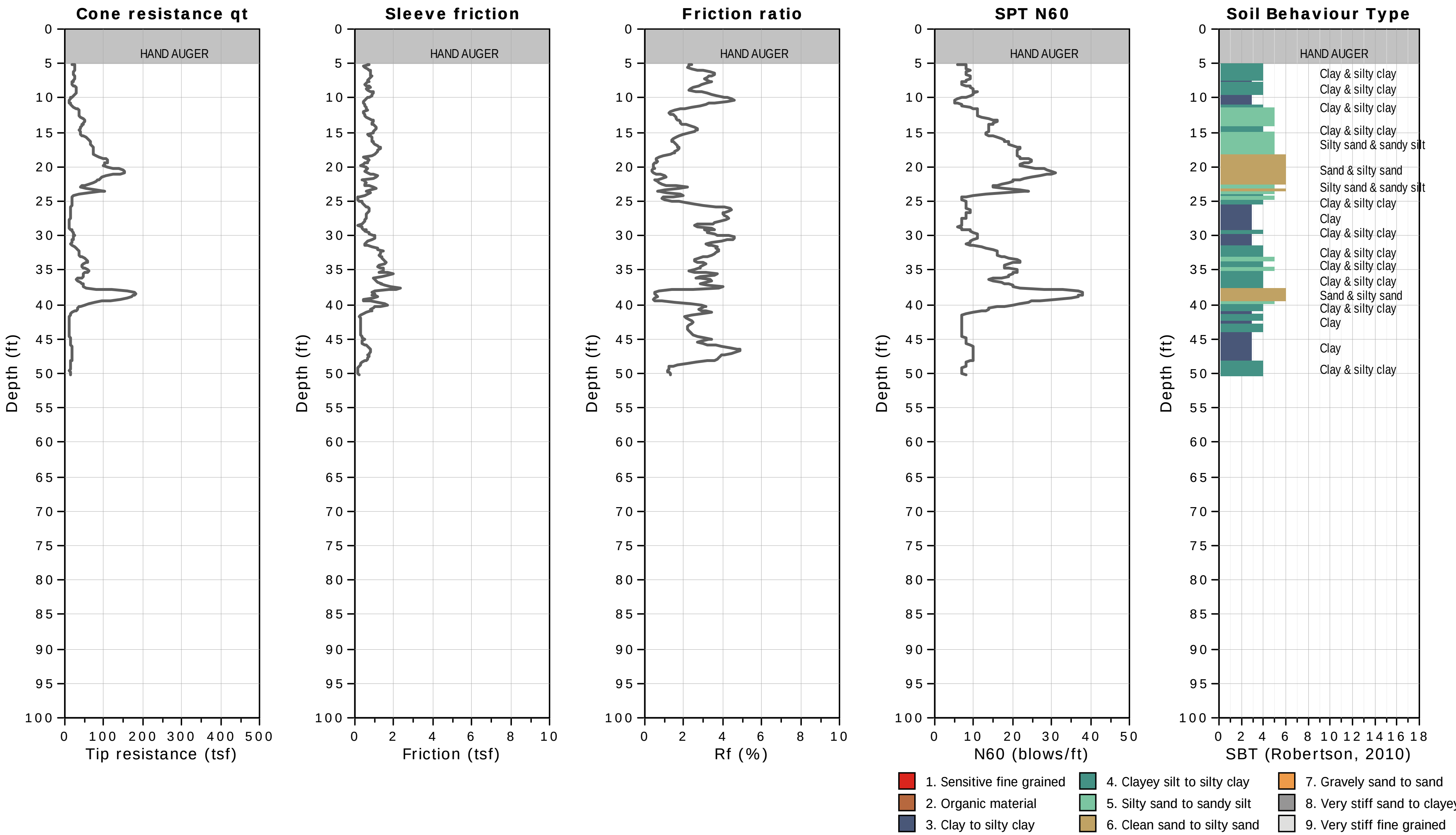
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

CPT: CPT-03

CLIENT: HALEY & ALDRICH
SITE: SJ CENTRAL SERVICE YARD, SAN JOSE, CA

FIELD REP: RATI MANDZULASHVILI
Total depth: 50.20 ft, Date: 11/26/2019



GREGG DRILLING, INC.
www.greggdrilling.com

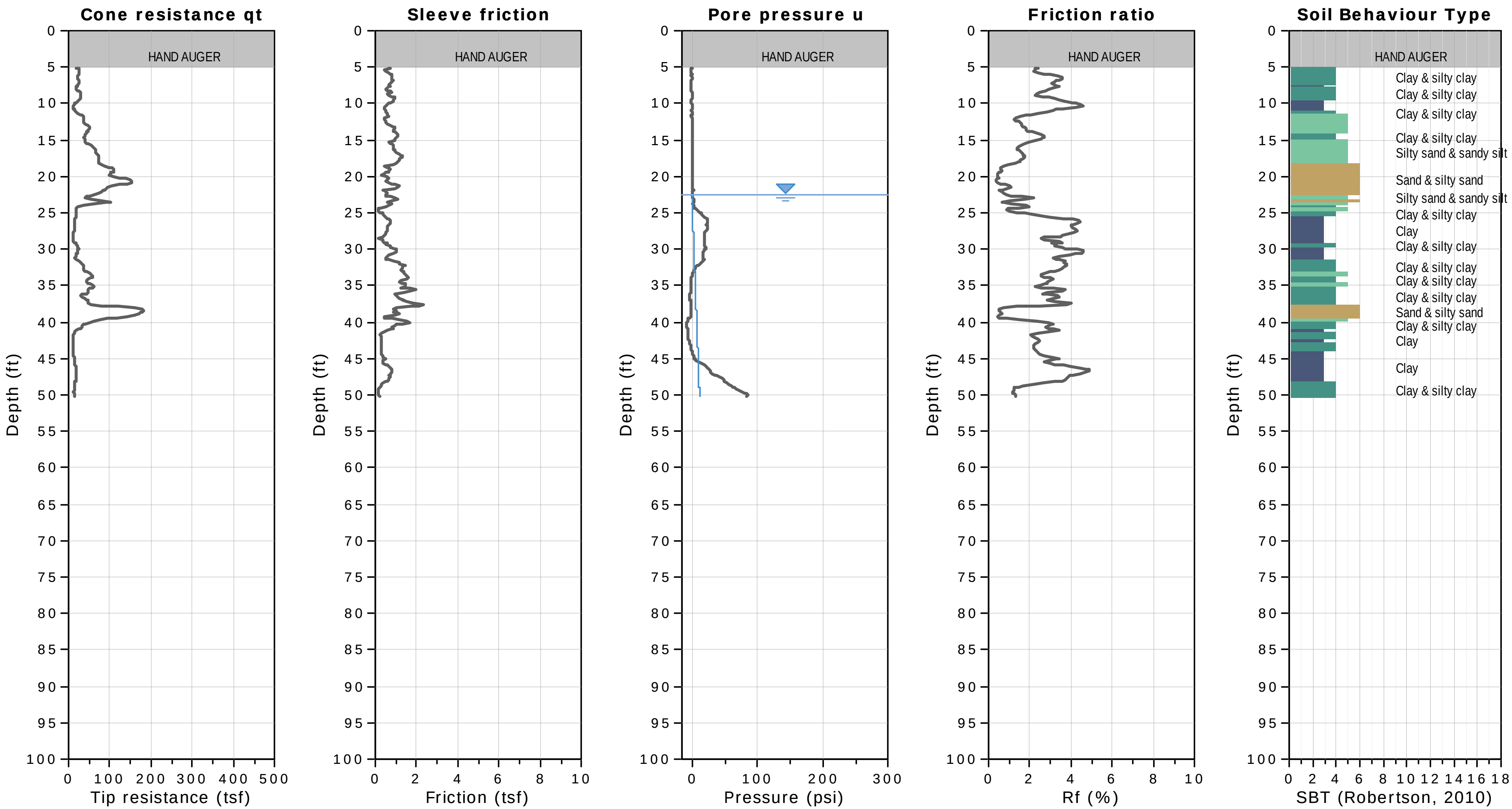
CPT: CPT-03

CLIENT: HALEY & ALDRICH

SITE: SJ CENTRAL SERVICE YARD, SAN JOSE, CA

FIELD REP: RATI MANDZULASHVILI

Total depth: 50.20 ft, Date: 11/26/2019



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, INC.
www.greggdrilling.com

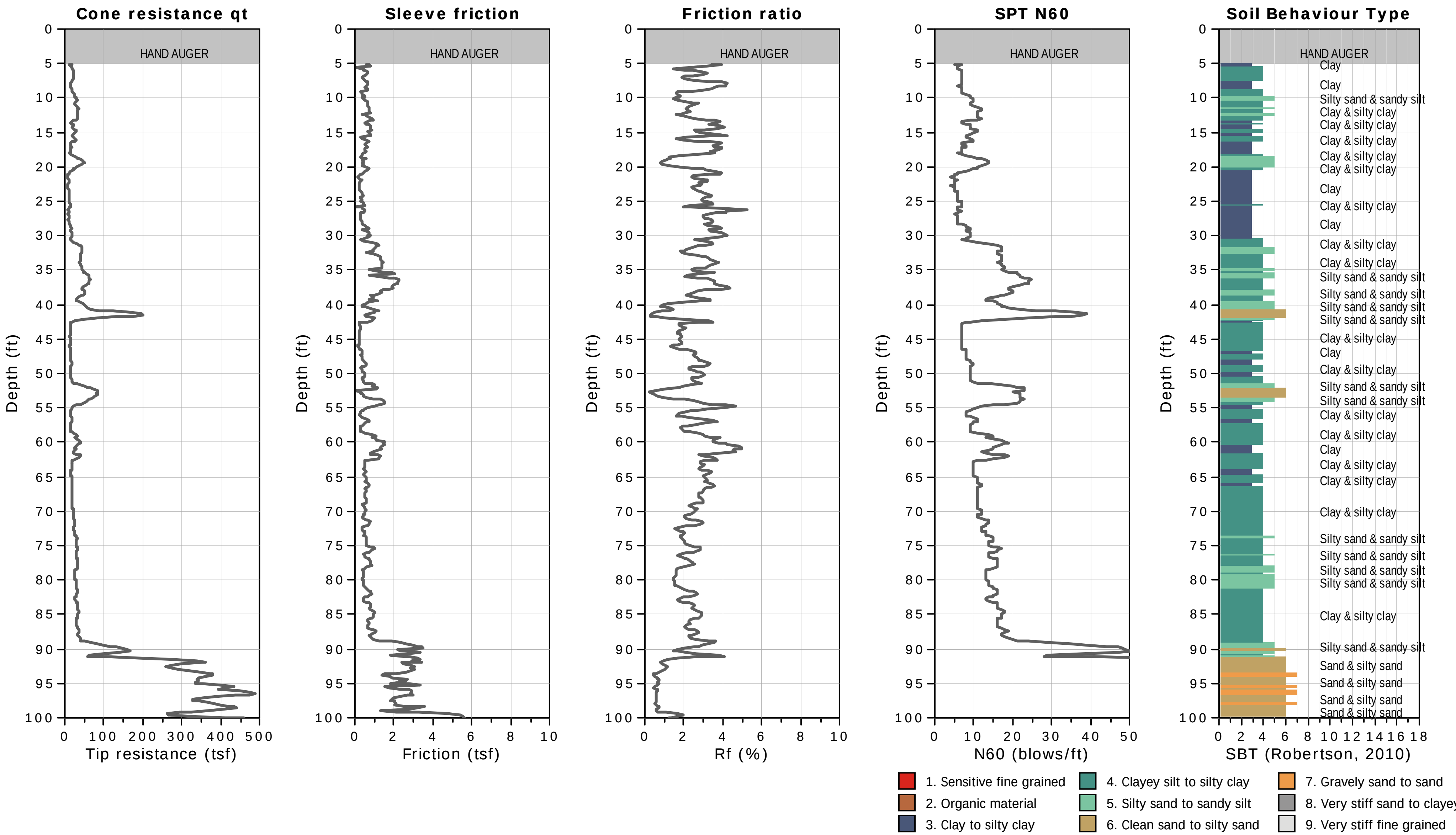
CPT: SCPT-01

CLIENT: HALEY & ALDRICH

SITE: SJ CENTRAL SERVICE YARD, SAN JOSE, CA

FIELD REP: RATI MANDZULASHVILI

Total depth: 100.07 ft, Date: 11/26/2019

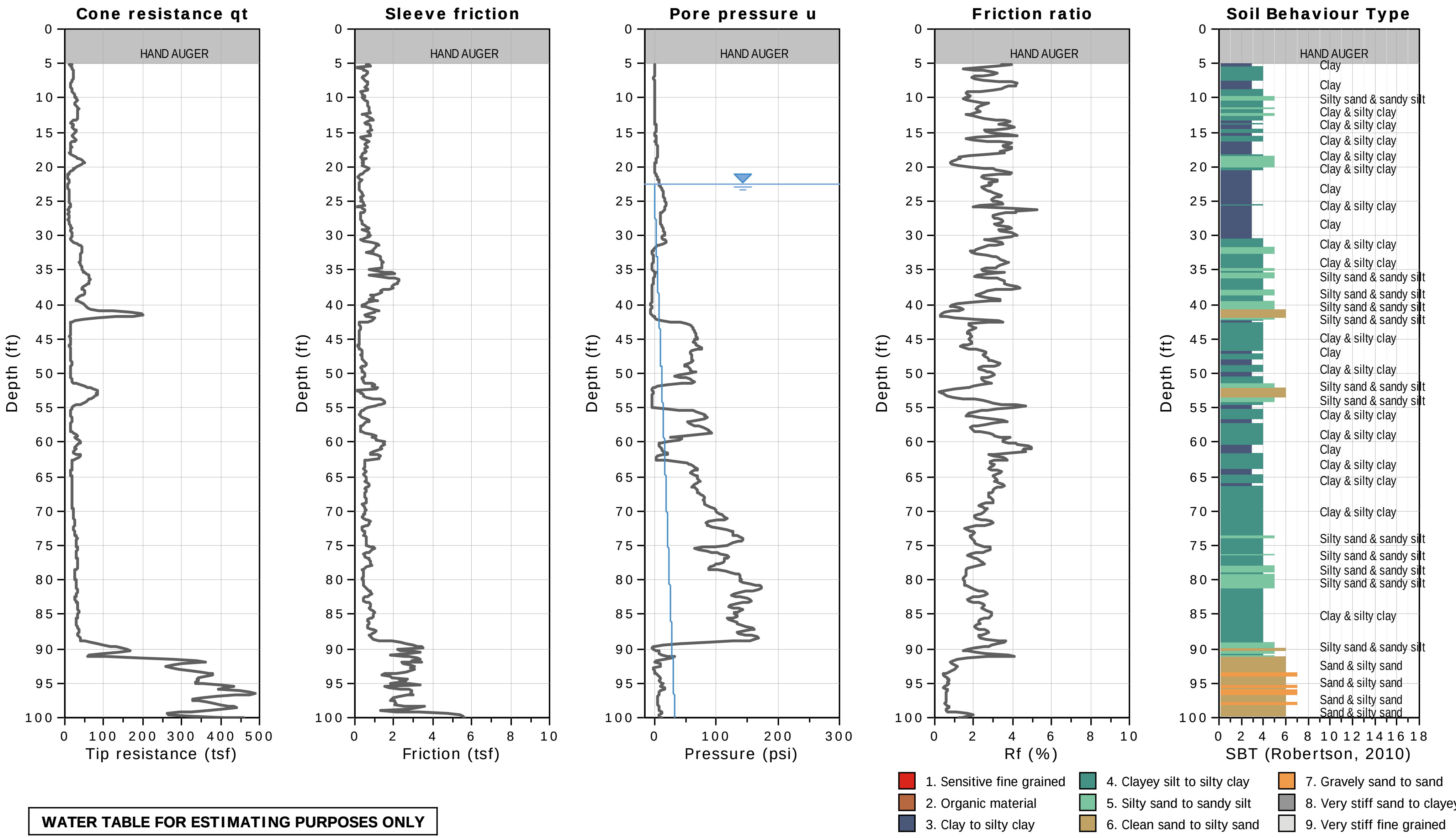


GREGG DRILLING, INC.
www.greggdrilling.com

CPT: SCPT-01

CLIENT: HALEY & ALDRICH
SITE: SJ CENTRAL SERVICE YARD, SAN JOSE, CA

FIELD REP: RATI MANDZULASHVILI
Total depth: 100.07 ft, Date: 11/26/2019



WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

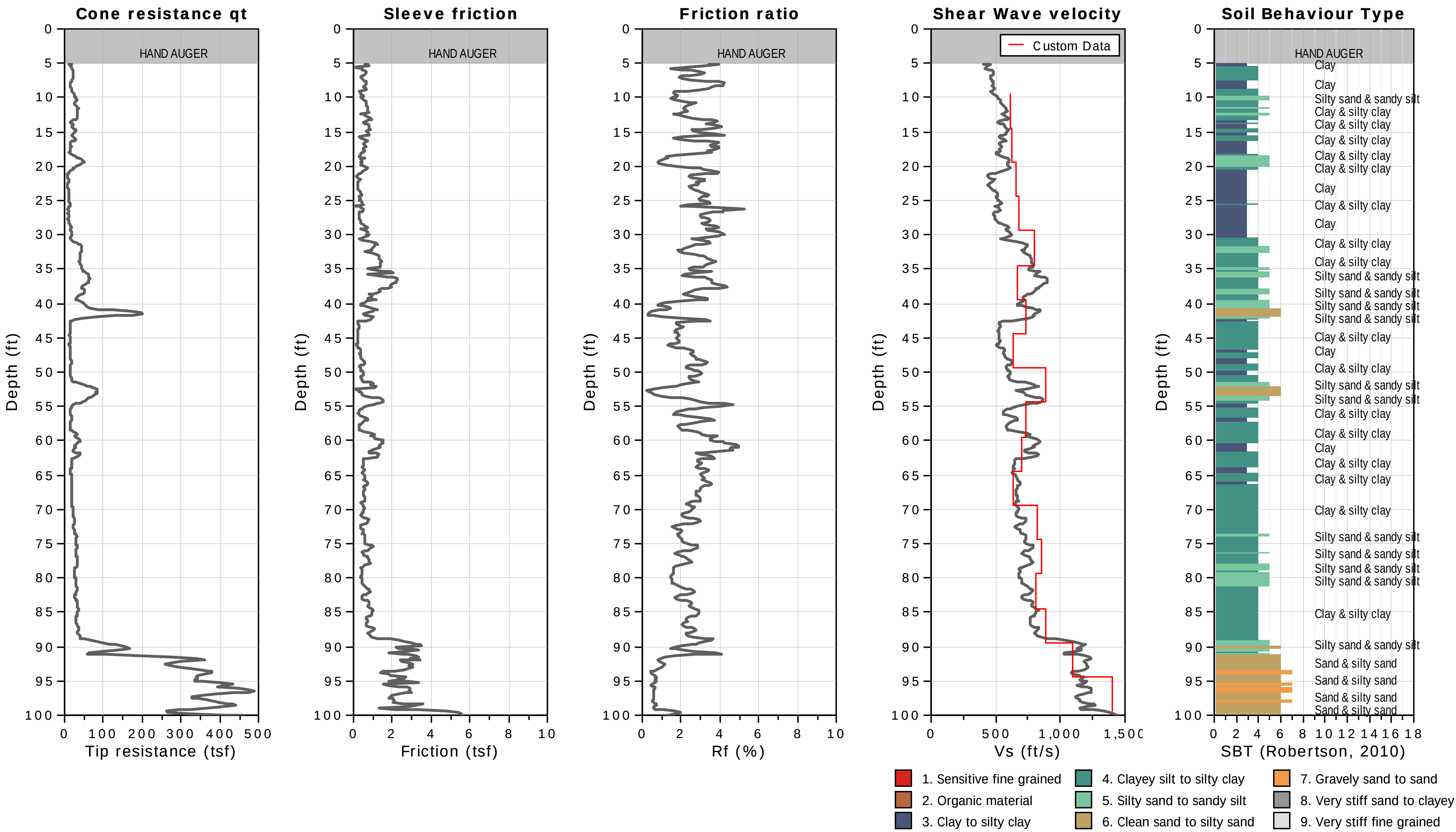
CPT: SCPT-01

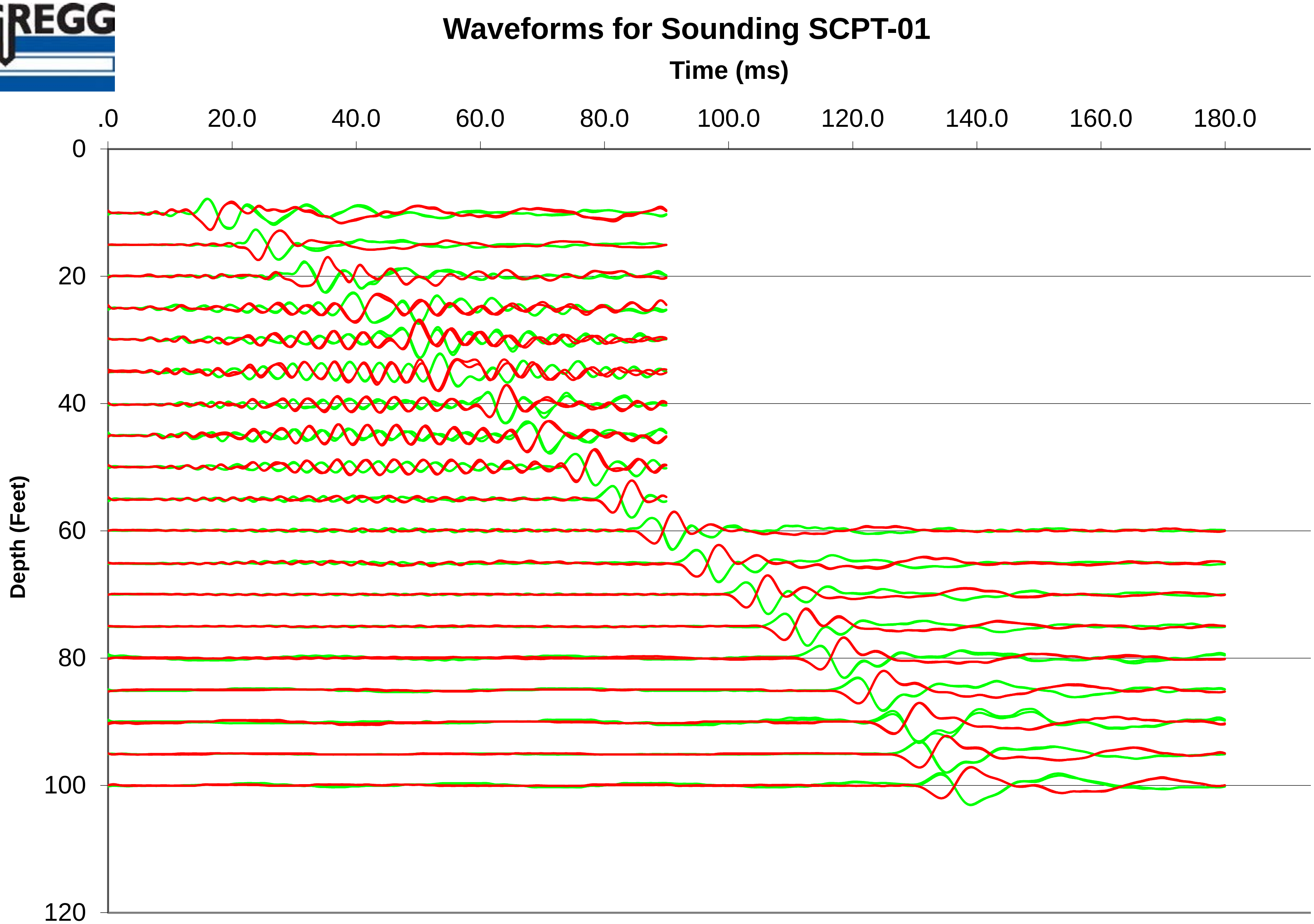
CLIENT: HALEY & ALDRICH

SITE: SJ CENTRAL SERVICE YARD, SAN JOSE, CA

FIELD REP: RATI MANDZULASHVILI

Total depth: 100.07 ft, Date: 11/26/2019





Waveforms for Sounding SCPT-01
Time (ms)
.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
Depth (Feet)
0
20
40
60
80
100
120

Shear Wave Velocity Calculations

SJ Central Service Yard

SCPT-01

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

11/26/19

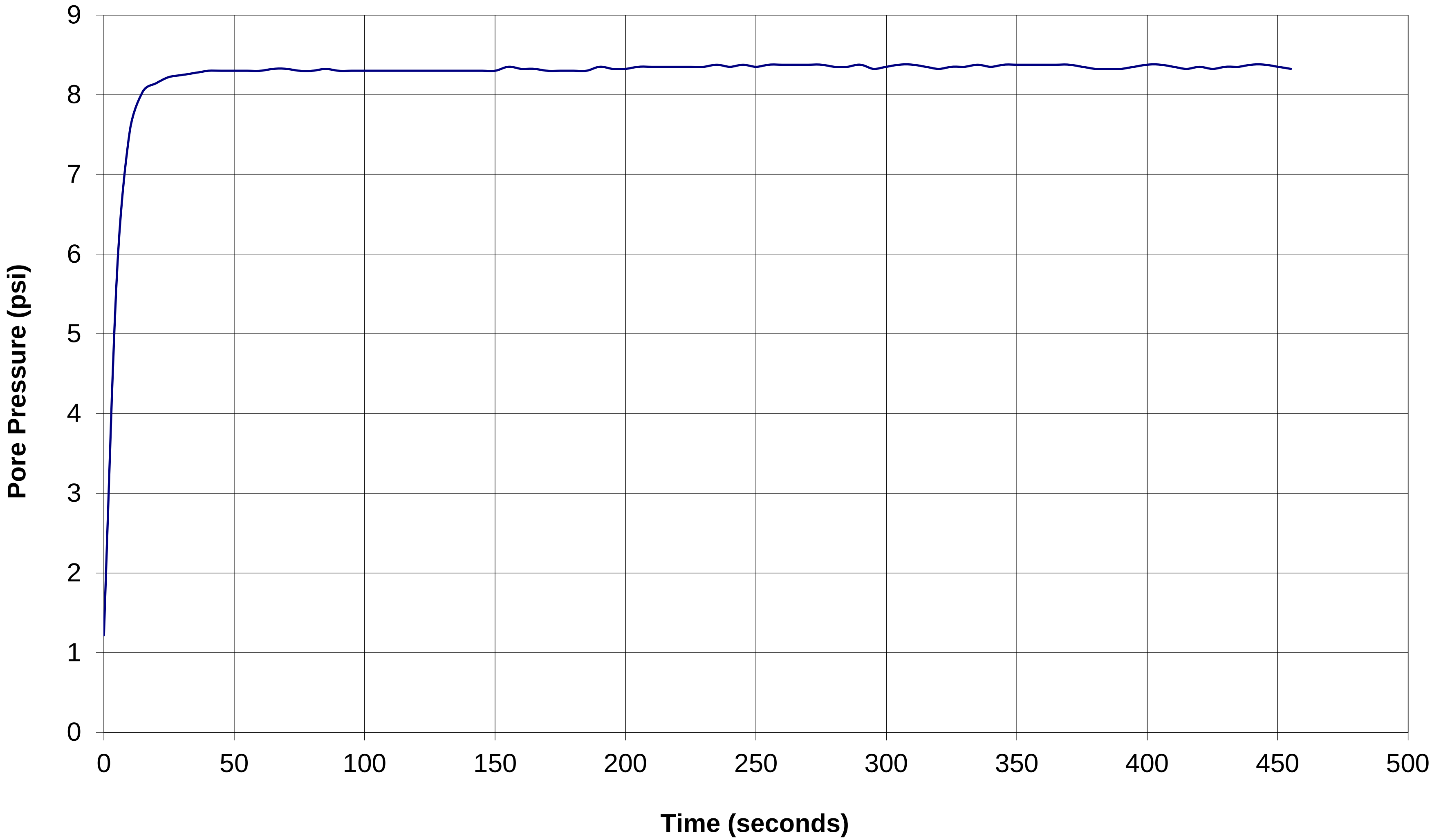
Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.17	9.51	9.66	9.66	17.8000			
15.09	14.43	14.53	4.87	25.7500	7.9500	612.8	11.97
20.01	19.35	19.42	4.90	33.6000	7.8500	623.8	16.89
25.10	24.44	24.50	5.07	41.3500	7.7500	654.2	21.90
30.02	29.36	29.41	4.91	48.5500	7.2000	682.2	26.90
35.10	34.44	34.49	5.08	54.9000	6.3500	799.7	31.90
40.19	39.53	39.57	5.08	62.5000	7.6000	668.4	36.99
45.11	44.45	44.48	4.92	69.2500	6.7500	728.5	41.99
50.03	49.37	49.40	4.92	77.0000	7.7500	634.6	46.91
55.12	54.46	54.48	5.08	82.7000	5.7000	891.7	51.92
60.04	59.38	59.40	4.92	89.4500	6.7500	728.8	56.92
65.12	64.46	64.49	5.08	96.6500	7.2000	706.0	61.92
70.05	69.39	69.41	4.92	104.4000	7.7500	634.8	66.93
75.13	74.47	74.49	5.08	110.6000	6.2000	820.0	71.93
80.05	79.39	79.41	4.92	116.3500	5.7500	855.7	76.93
85.14	84.48	84.49	5.08	122.6000	6.2500	813.5	81.93
90.22	89.56	89.58	5.08	128.3000	5.7000	892.0	87.02
95.14	94.48	94.50	4.92	132.8000	4.5000	1093.4	92.02
100.07	99.41	99.42	4.92	136.3000	3.5000	1405.9	96.94

GREGG

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding:	SCPT-01
Depth (ft):	41.67
Site:	SJ Central Service
Engineer:	Rati Mandzulashvili



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.

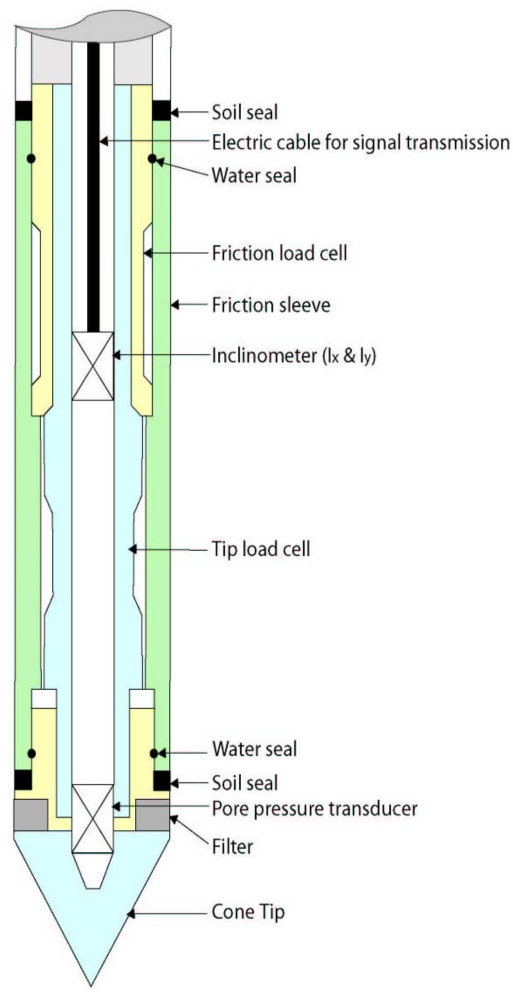


Figure CPT

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)

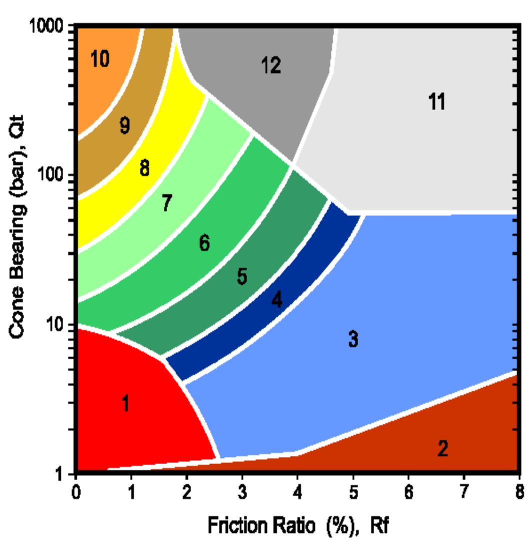
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

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) $\quad q_t = q_c + u\,(1-a)$

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.

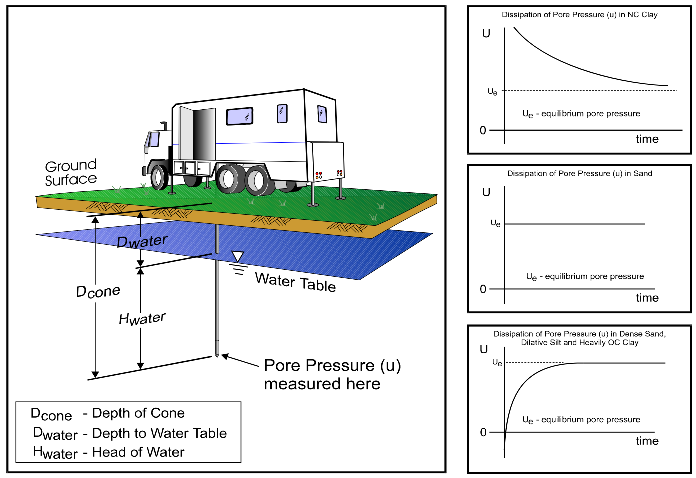
Pore pressure dissipation data can be interpreted to provide estimates of:

- 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 = Δd/Δt

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.

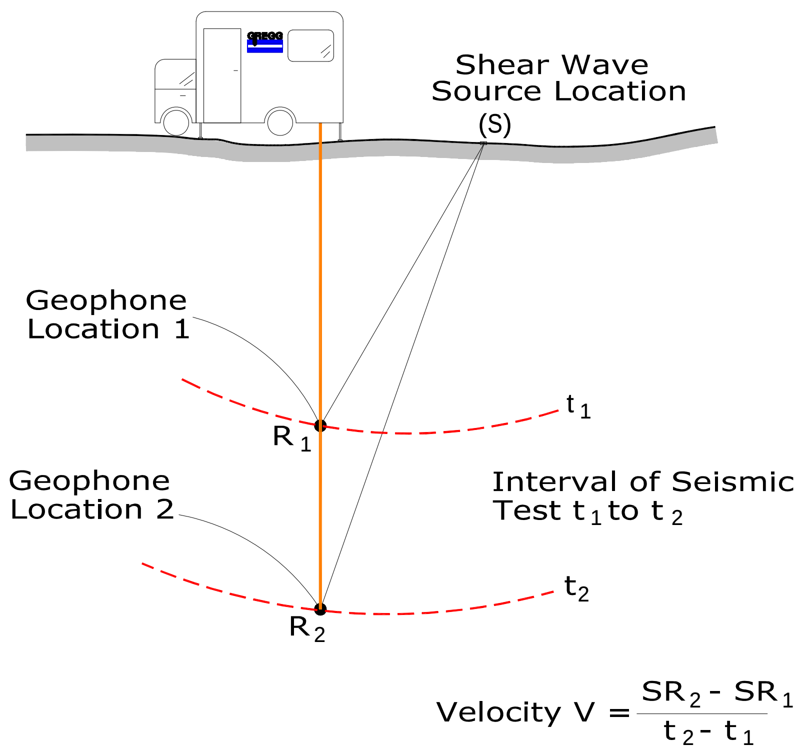


Figure SCPT

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.

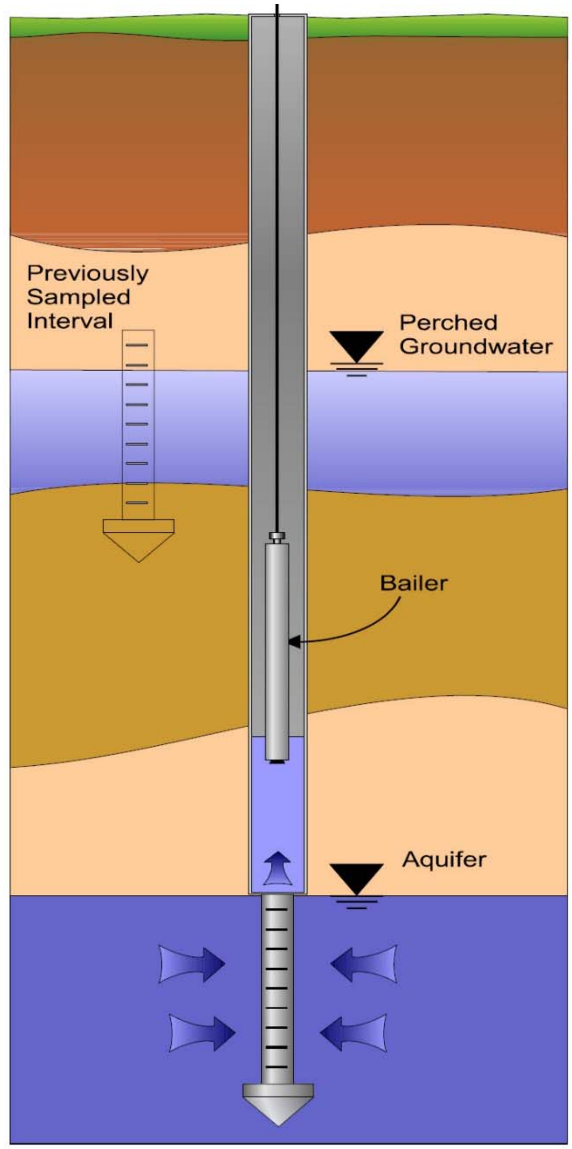


Figure GWS

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.

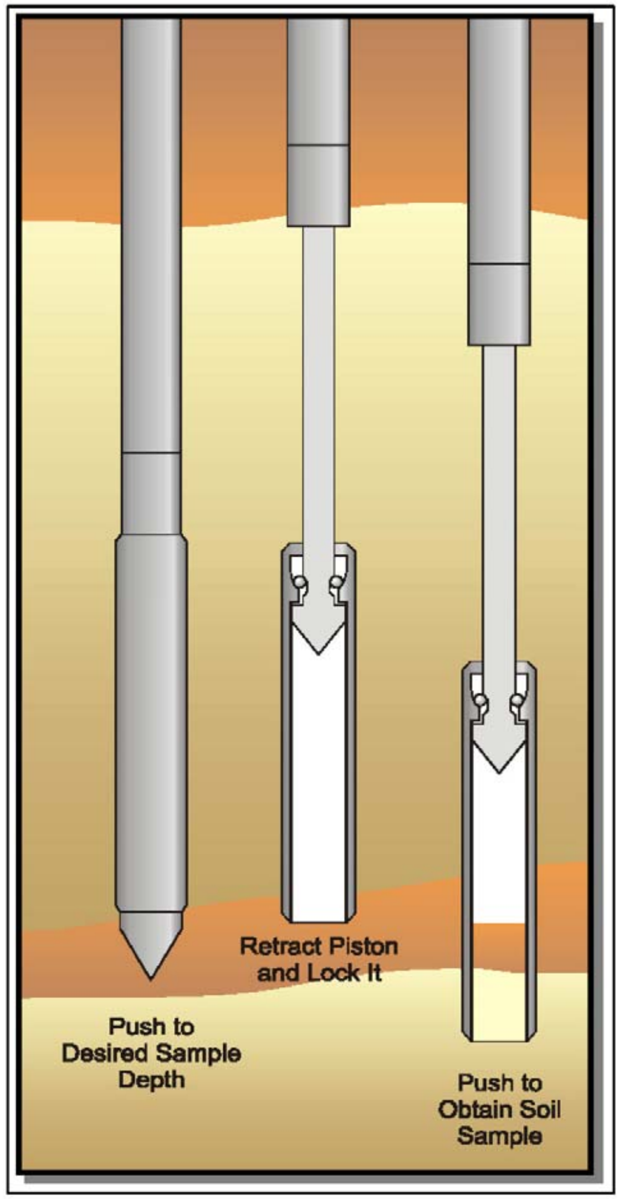


Figure SS

GREGG DRILLING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

March 3, 2020

Haley & Aldrich
Attn: Rati M.

Subject: CPT Site Investigation
San Jose Yard
San Jose, California
GREGG Project Number: D2209056

Dear Rati:

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 714-863-0988.

Sincerely,
Gregg Drilling, LLC.

CPT Reports Team
Gregg Drilling, LLC.

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)
CPT-04	03/02/2020	50.52	-	-	34.3

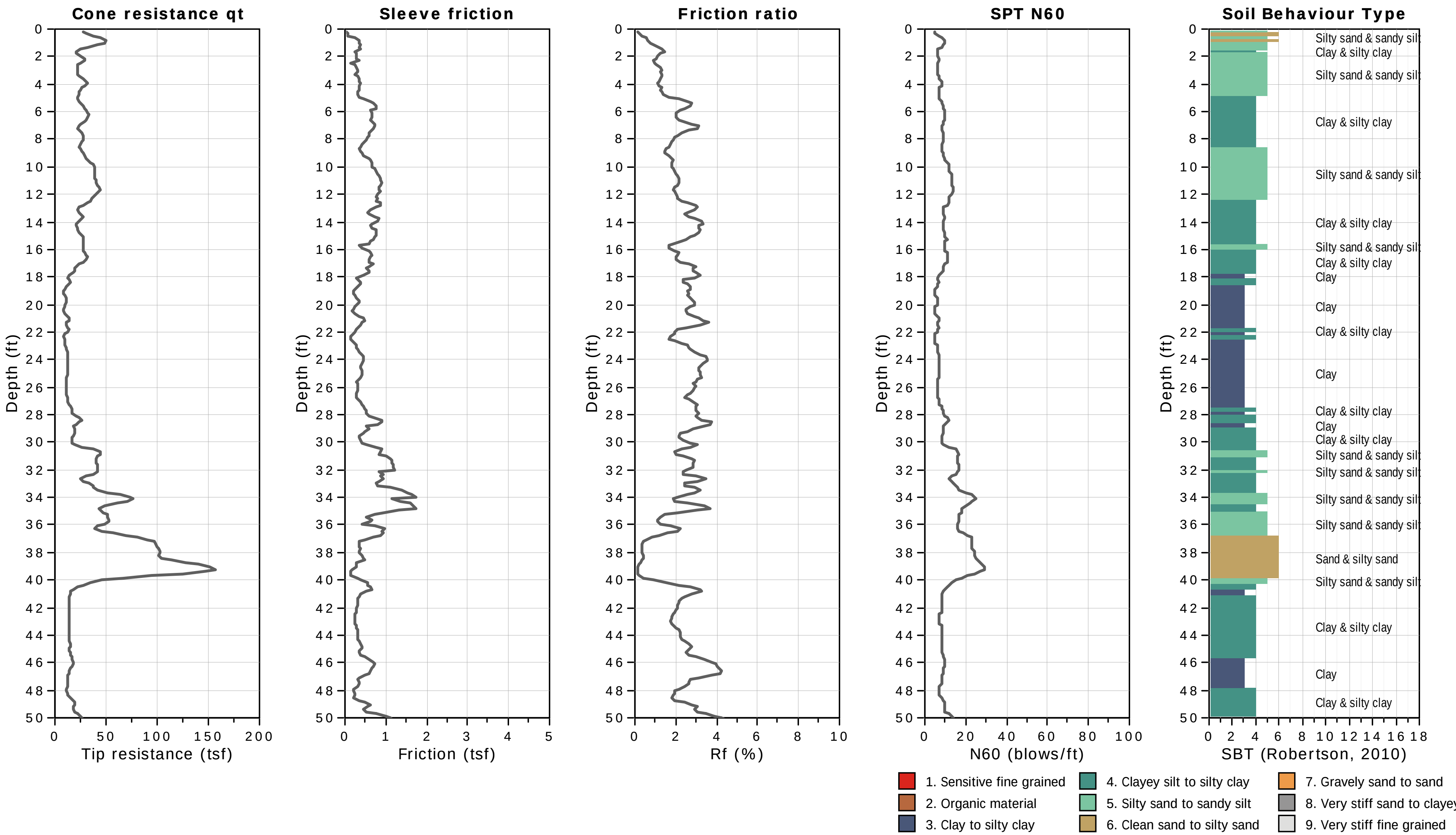
2726 Walnut Ave. • Signal Hill, California 90755 • (562) 427-6899 • FAX (562) 427-3314
950 Howe Road. • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-04

CLIENT: HALEY & ALDRICH
SITE: SAN JOSE YARD, SAN JOSE, CA

FIELD REP: R A T I M.
Total depth: 50.52 ft, Date: 3/2/2020

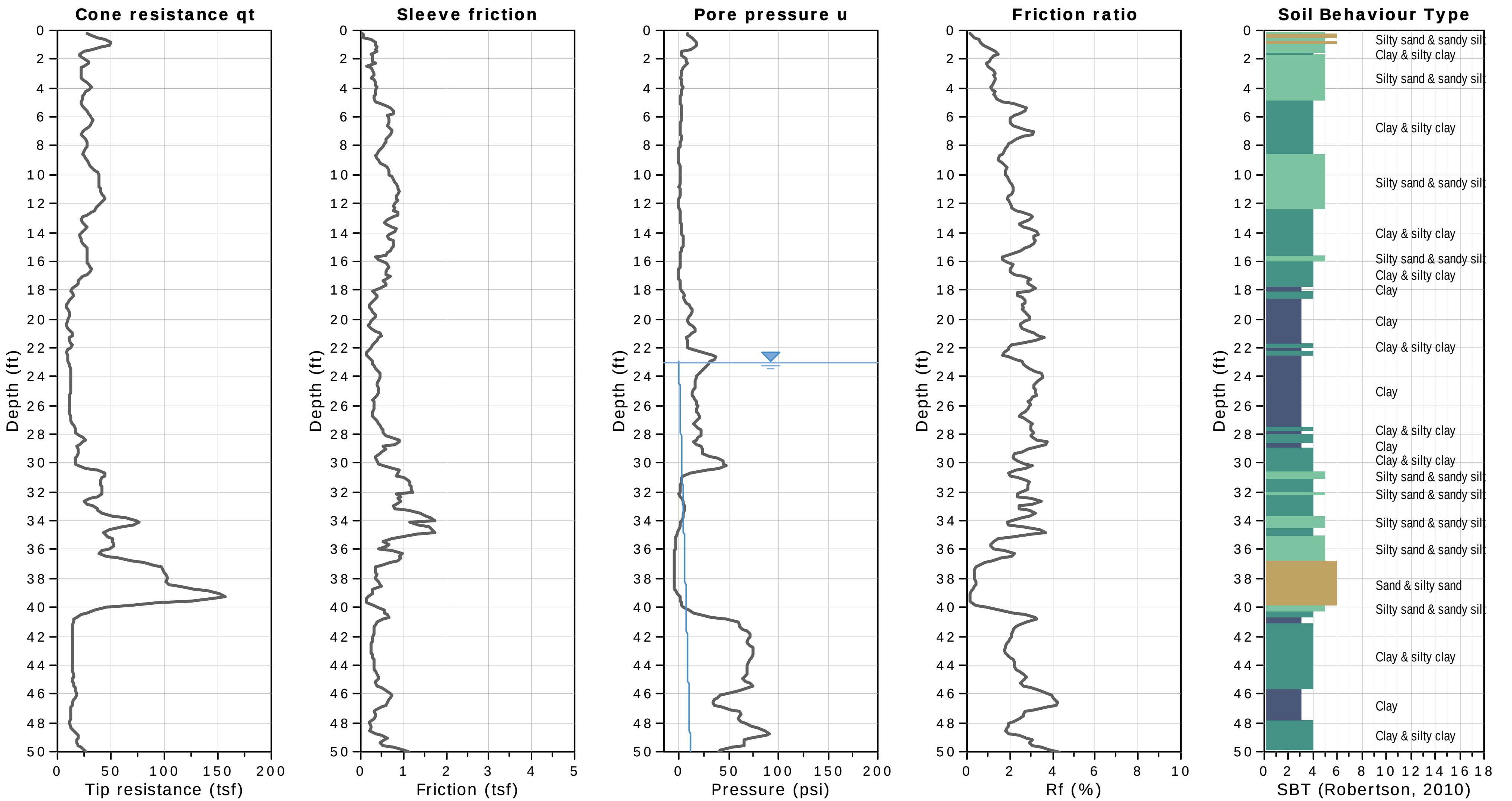


GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-04

CLIENT: HALEY & ALDRICH
SITE: SAN JOSE YARD, SAN JOSE, CA

FIELD REP: R A T I M.
Total depth: 50.52 ft, Date: 3/2/2020



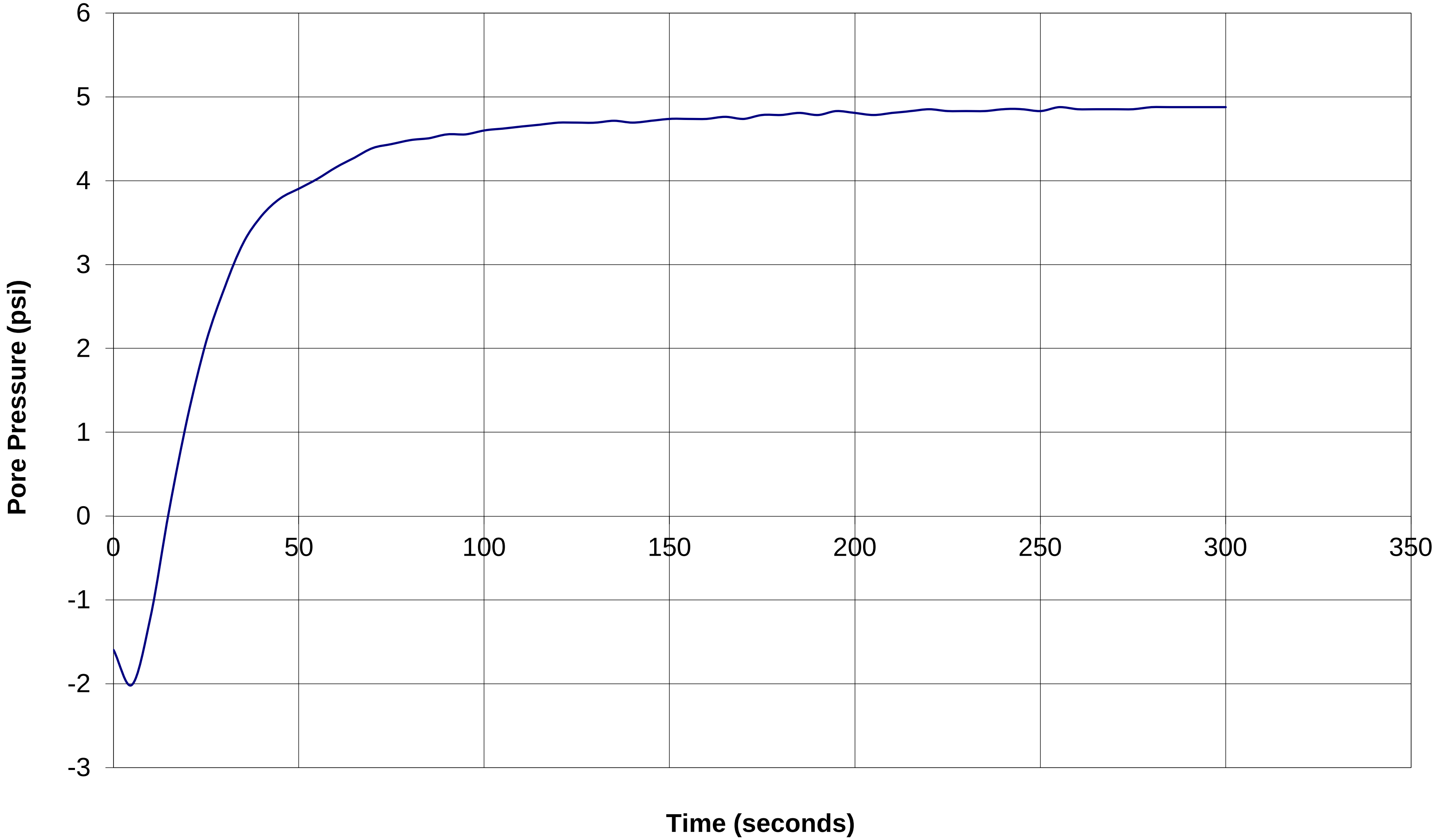
1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-04
Depth (ft): 34.28
Site: SAN JOSE YARD
Engineer: R A T I M.



APPENDIX C

Percolation Test Data

HALEY ALDRICH

PERCOLATION TEST DATA

Client:	Ten Over Studio	File No.:	134258-002
Project:	San Jose Fire Training Center	Sheet:	1 of 1
Subject:	Boring/Excavation Percolation Testing Field Log	Date:	19-Nov-2019
		Field Rep.	RM

Boring/Test Number	PT-1		Diameter of Casing	2	in.
Diameter of Boring	4	in.	Length of Casing	77	in.
Depth of Casing b.g.s	3.42	ft.	Initial Water Depth (d1)	47	in.

Reading Number	Time Start/End	Elapsed Time (min)	Final Water Depth (in)	Water Drop (in)	Direct Percolation Rate (in/hr)	Reduction Factor	Adjusted Percolation Rate (in/hr)
1	1220 1250	30	55.00	55.00	110.00	2.25	48.89
2	1320 1350	30	58.50	3.50	7.00	15.13	0.46
3	1350 1420	30	62.00	3.50	7.00	15.13	0.46
4	1420 1450	30	64.00	2.00	4.00	15.50	0.26
5	1450 1520	30	66.00	2.00	4.00	15.50	0.26
6	1520 1550	30	67.75	1.75	3.50	15.56	0.22
7	1550 1620	30	69.00	1.25	2.50	15.69	0.16
				Average of Last 3 Readings:	3.33	15.58	0.21

Reduction Factors

Rf = 15.58 *(Calculated as Rf = [(2d1 - Δd) / DIA] + 1)*

CF_v = 2 *(moderate site variability, low number of tests (1 per 11 acres))*

CF_s = 2 *(moderate long-term siltation)*

CF_{total} = 62.3 *(product of Rf, CFv, and CFs)*

Design Percolation Rate

Unadjusted Percolation Rate (P_R) = 3.33 in/hour *(average of last three)*

Design Percolation Rate (P_R / Cf_{total}) = 0.1 in/hour
(7.1E-05 cm/s)

HALEY ALDRICH	PERCOLATION TEST DATA	File No.:	134258-002
		Sheet:	1 of 1
Client:	Ten Over Studio	Date:	2-Mar-2020
Project:	San Jose Fire Training Center	Field Rep.	RM
Subject:	Boring/Excavation Percolation Testing Field Log		

Boring/Test Number	PT-2		Diameter of Casing	2	in.
Diameter of Boring	3	in.	Length of Casing	90	in.
Depth of Casing b.g.s	5.00	ft.	Initial Water Depth (d1)	0	in.

Reading Number	Time Start/End	Elapsed Time (min)	Final Water Depth (in)	Water Drop (in)	Direct Percolation Rate (in/hr)	Reduction Factor	Adjusted Percolation Rate (in/hr)
1	1030 1033	3.3	90.00	90.00	1636.36	31.00	52.79
2	1033 1037	3.5	90.00	90.00	1542.86	31.00	49.77
3	1037 1041	3.75	90.00	90.00	1440.00	31.00	46.45
4	1041 1044	3.9	90.00	90.00	1384.62	31.00	44.67
5	1044 1048	4	90.00	90.00	1350.00	31.00	43.55
6	1048 1052	4	90.00	90.00	1350.00	31.00	43.55
				Average of Last 3 Readings:	1350.00	31.00	43.55

Reduction Factors

Rf = 31.00 *(Calculated as Rf = [(2d1 - Δd) / DIA] + 1)*

CF_v = 2 *(moderate site variability, low number of tests (1 per 11 acres))*

CF_s = 2 *(moderate long-term siltation)*

CF_{total} = 124.0 *(product of Rf, CFv, and CFs)*

Design Percolation Rate

Unadjusted Percolation Rate (P_R) = 1350.00 in/hour *(average of last three)*

Design Percolation Rate (P_R / Cf_{total}) = 10.9 in/hour

(7.7E-03 cm/s)

NOTES: Casing fully drained during the testing.

HALEY ALDRICH	PERCOLATION TEST DATA	File No.:	134258-002
		Sheet:	1 of 1
Client:	Ten Over Studio	Date:	2-Mar-2020
Project:	San Jose Fire Training Center	Field Rep.	RM
Subject:	Boring/Excavation Percolation Testing Field Log		

Boring/Test Number	PT-3		Diameter of Casing	2	in.
Diameter of Boring	3	in.	Length of Casing	90	in.
Depth of Casing b.g.s	5.00	ft.	Initial Water Depth (d1)	0	in.

Reading Number	Time Start/End	Elapsed Time (min)	Final Water Depth (in)	Water Drop (in)	Direct Percolation Rate (in/hr)	Reduction Factor	Adjusted Percolation Rate (in/hr)
1	1235 1238	2.8	90.00	90.00	1928.57	31.00	62.21
2	1238 1241	3	90.00	90.00	1800.00	31.00	58.06
3	1241 1244	3.1	90.00	90.00	1741.94	31.00	56.19
4	1244 1247	3.3	90.00	90.00	1636.36	31.00	52.79
5	1247 1251	3.5	90.00	90.00	1542.86	31.00	49.77
6	1251 1254	3.5	90.00	90.00	1542.86	31.00	49.77
				Average of Last 3 Readings:	1542.86	31.00	49.77

Reduction Factors

Rf =	31.00	*(Calculated as Rf = [(2d1 - Δd) / DIA] + 1)*
CF_v =	2	*(moderate site variability, low number of tests (1 per 11 acres))*
CF_s =	2	*(moderate long-term siltation)*
CF_{total} =	124.0	*(product of Rf, CFv, and CFs)*

Design Percolation Rate

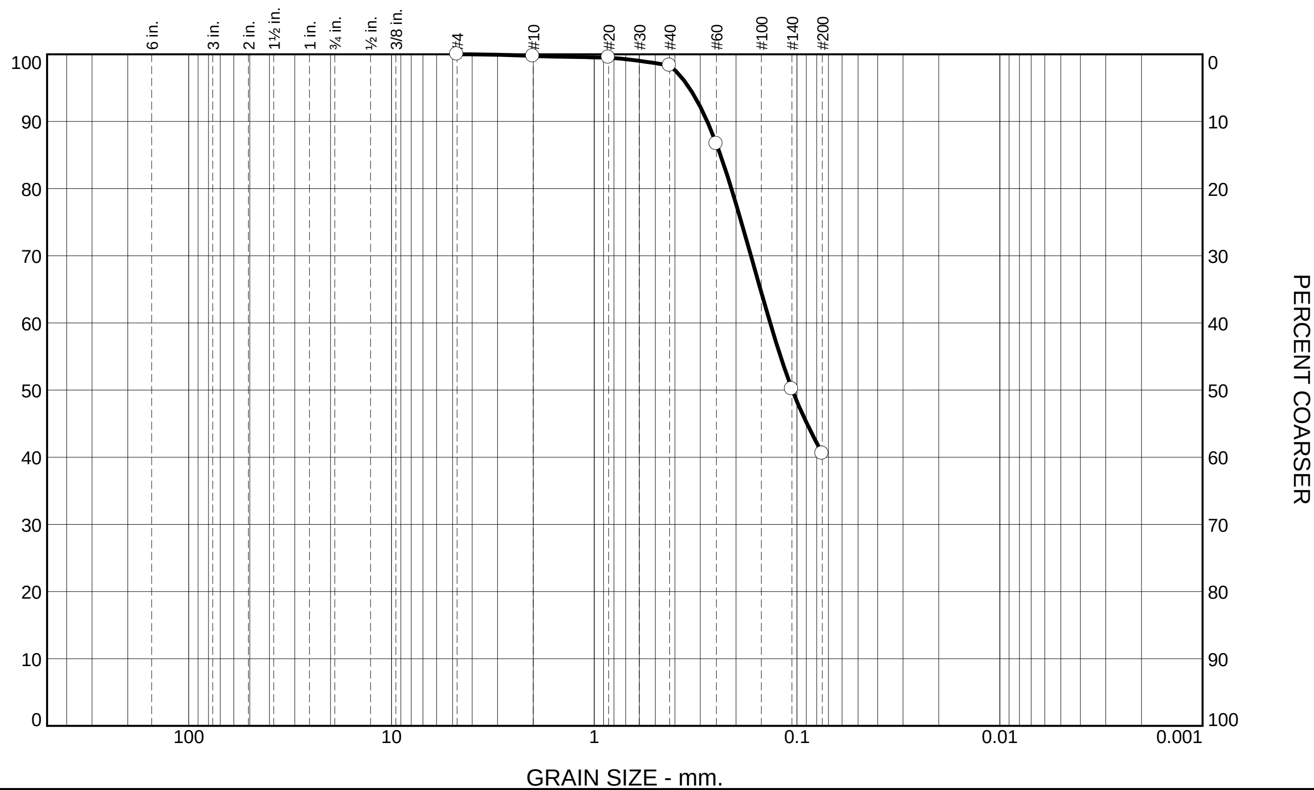
Unadjusted Percolation Rate (P_R) =	1542.86	in/hour *(average of last three)*
Design Percolation Rate (P_R / Cf_{total}) =	12.4	in/hour
	(8.7E-03	*cm/s)*

NOTES: Casing fully drained during the testing.

APPENDIX D

Laboratory Test Results

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	2	57	41	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#10	100		
#20	100		
#40	98		
#60	87		
#140	50		
#200	41		

* (no specification provided)

Soil Description

Brown sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2772 D_{85}= 0.2387 D_{60}= 0.1357
D_{50}= 0.1055 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

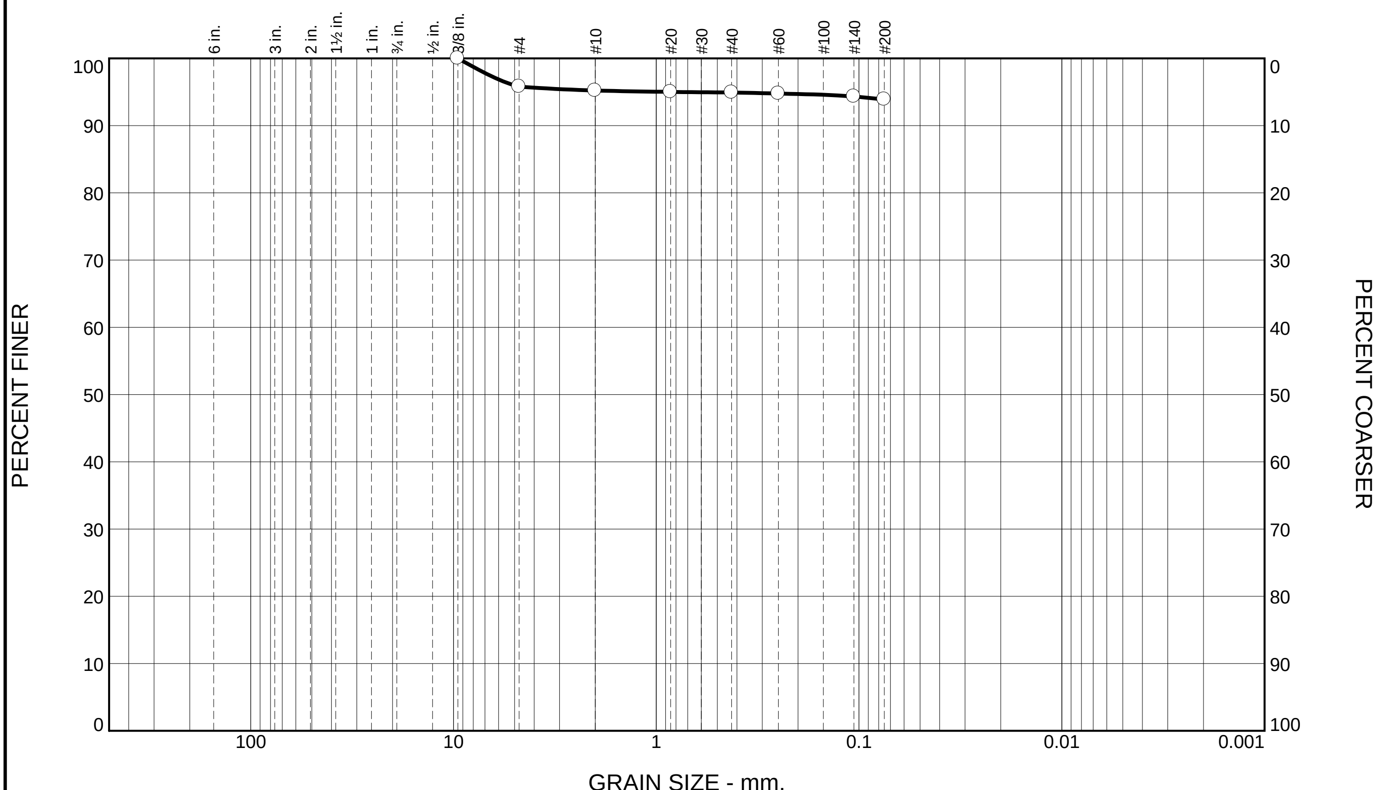
Source of Sample: HA-1 **Depth:** 10

Date: 12-12-19

Client: Haley & Aldrich, Inc.
Project: San Jose Fire Training Center
134258-002
Project No: 2831-028.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	4	1	0	1	94	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100		
#4	96		
#10	95		
#20	95		
#40	95		
#60	95		
#140	94		
#200	94		

* (no specification provided)

Soil Description

Brown silt

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= ML AASHTO=

Remarks

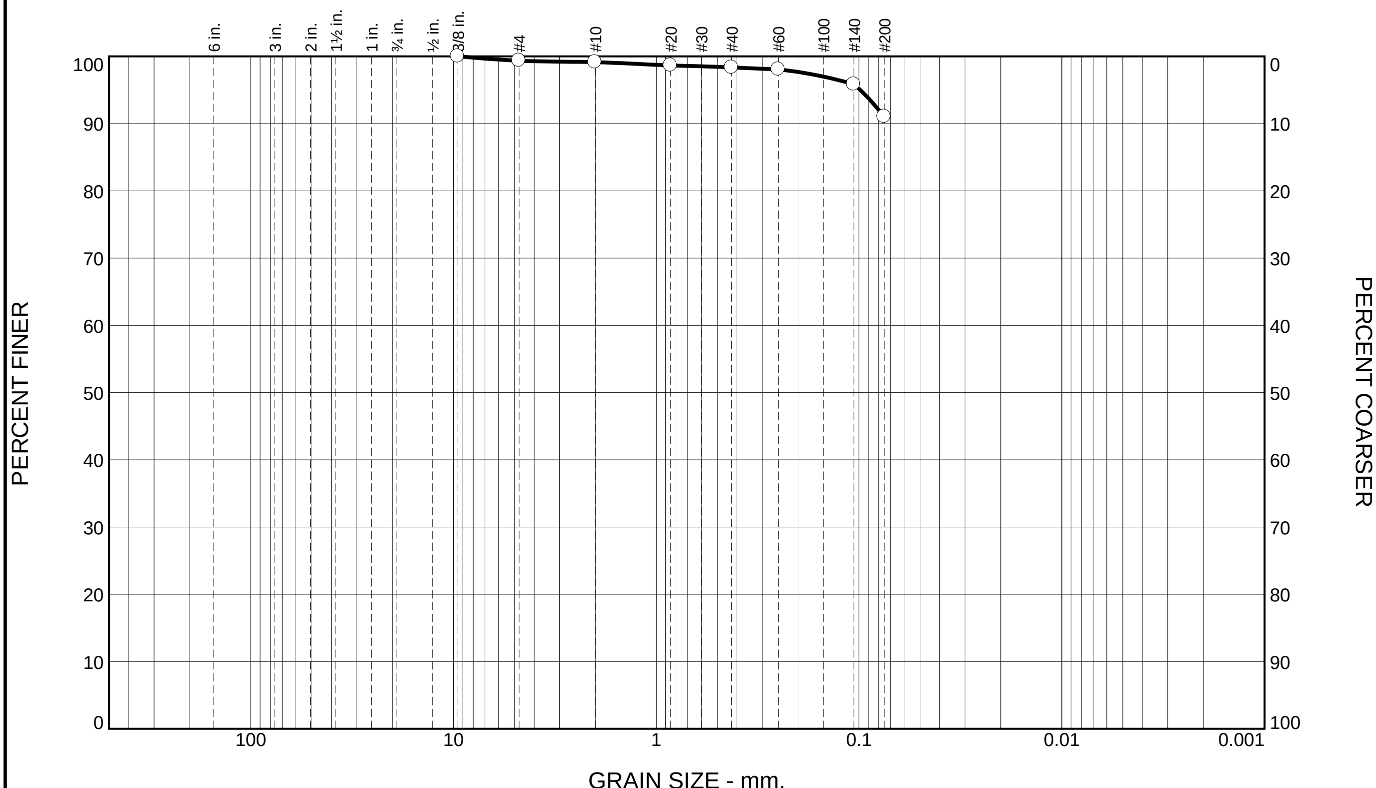
Source of Sample: HA-1 **Depth:** 20

Date: 12-12-19

Client: Haley & Aldrich, Inc.
Project: San Jose Fire Training Center
134258-002
Project No: 2831-028.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	1	0	1	7	91	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100		
#4	99		
#10	99		
#20	99		
#40	98		
#60	98		
#140	96		
#200	91		

* (no specification provided)

Soil Description

Brown silt

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= ML AASHTO=

Remarks

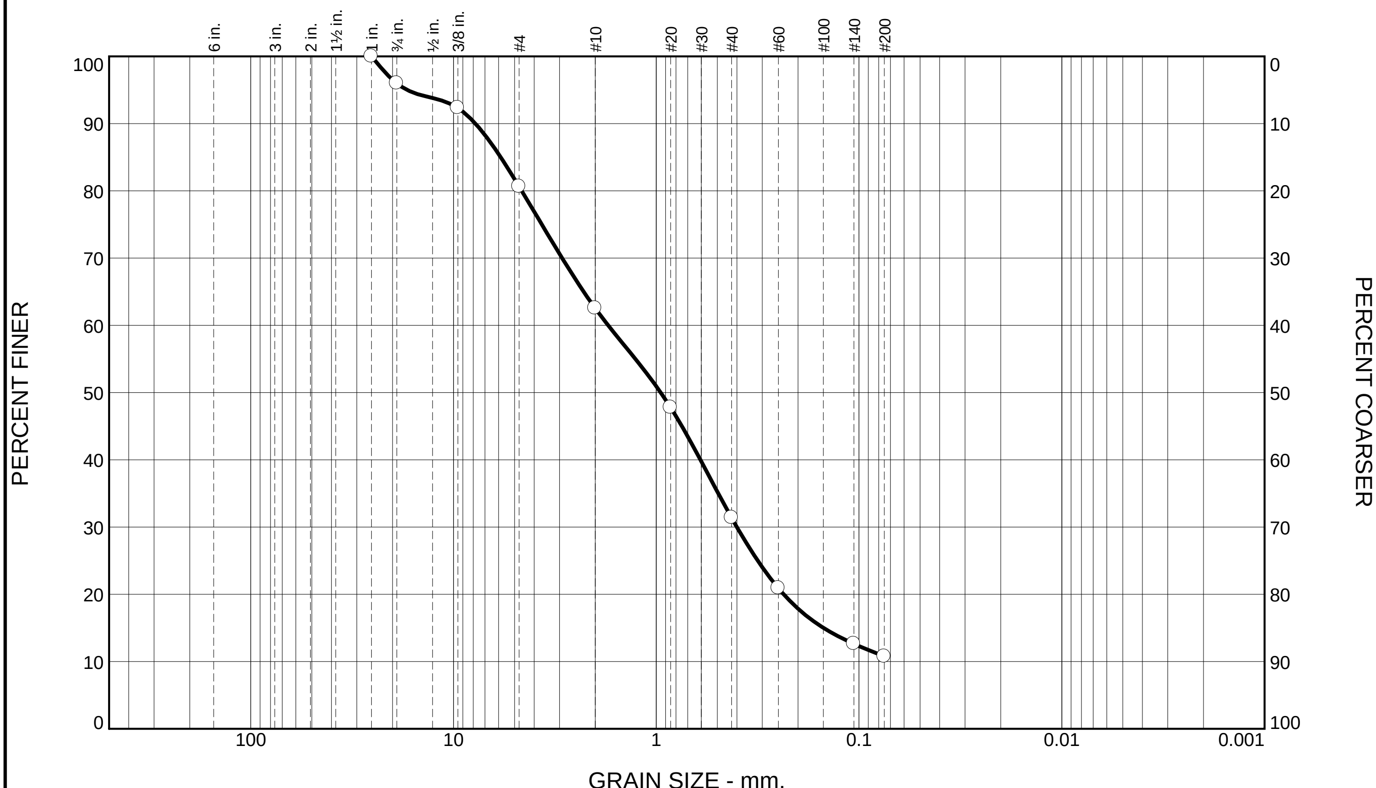
Source of Sample: HA-1 **Depth:** 30.5

Date: 12-12-19

Client: Haley & Aldrich, Inc.
Project: San Jose Fire Training Center
134258-002
Project No: 2831-028.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	4	15	18	32	20	11	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100		
3/4	96		
3/8	92		
#4	81		
#10	63		
#20	48		
#40	31		
#60	21		
#140	13		
#200	11		

* (no specification provided)

Soil Description

Gray sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.8048 D_{85}= 5.8569 D_{60}= 1.7282
D_{50}= 0.9510 D_{30}= 0.4003 D_{15}= 0.1496
D_{10}= C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

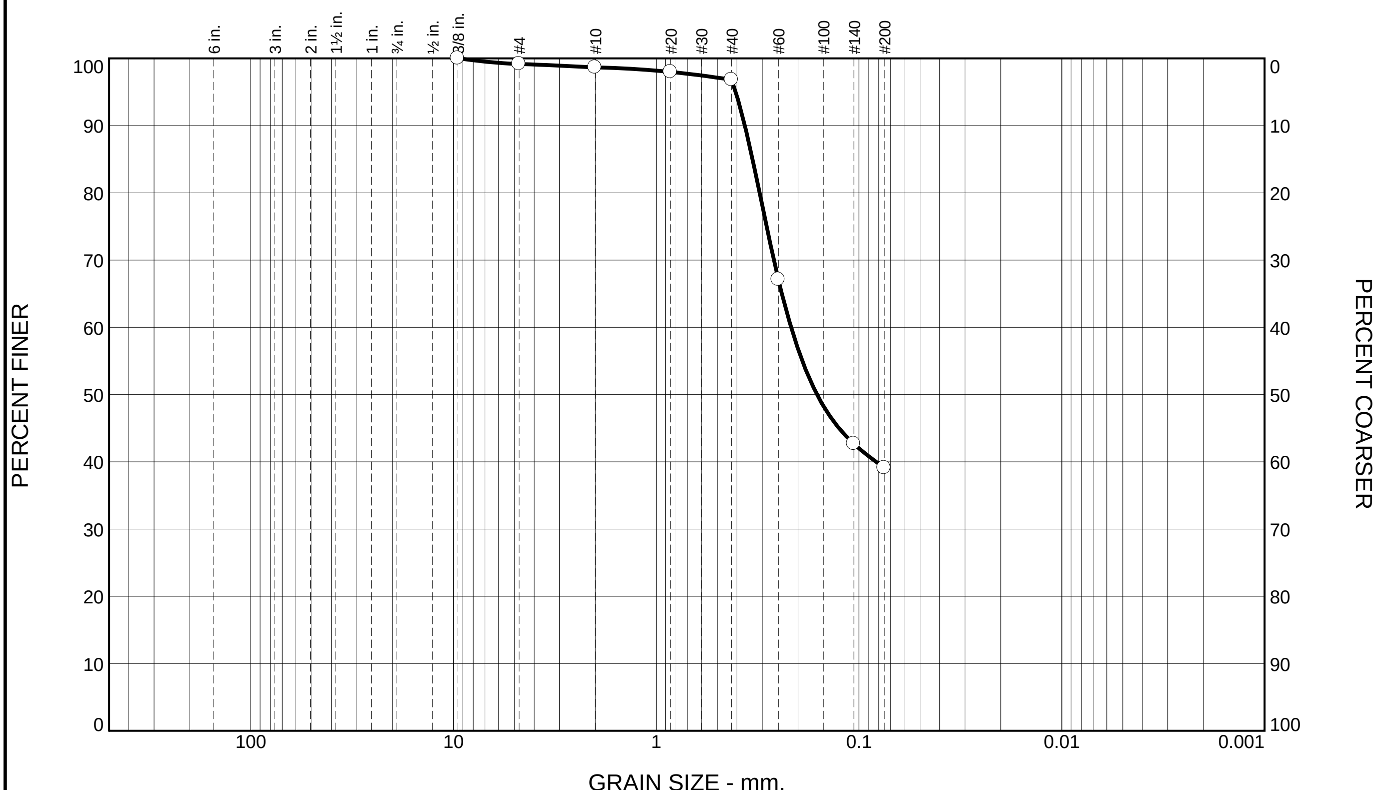
Source of Sample: HA-1 **Depth:** 40

Date: 12-12-19

Client: Haley & Aldrich, Inc.
Project: San Jose Fire Training Center
134258-002
Project No: 2831-028.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	1	0	2	58	39	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100		
#4	99		
#10	99		
#20	98		
#40	97		
#60	67		
#140	43		
#200	39		

* (no specification provided)

Soil Description

Brown sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3658 D_{85}= 0.3352 D_{60}= 0.2164
D_{50}= 0.1611 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

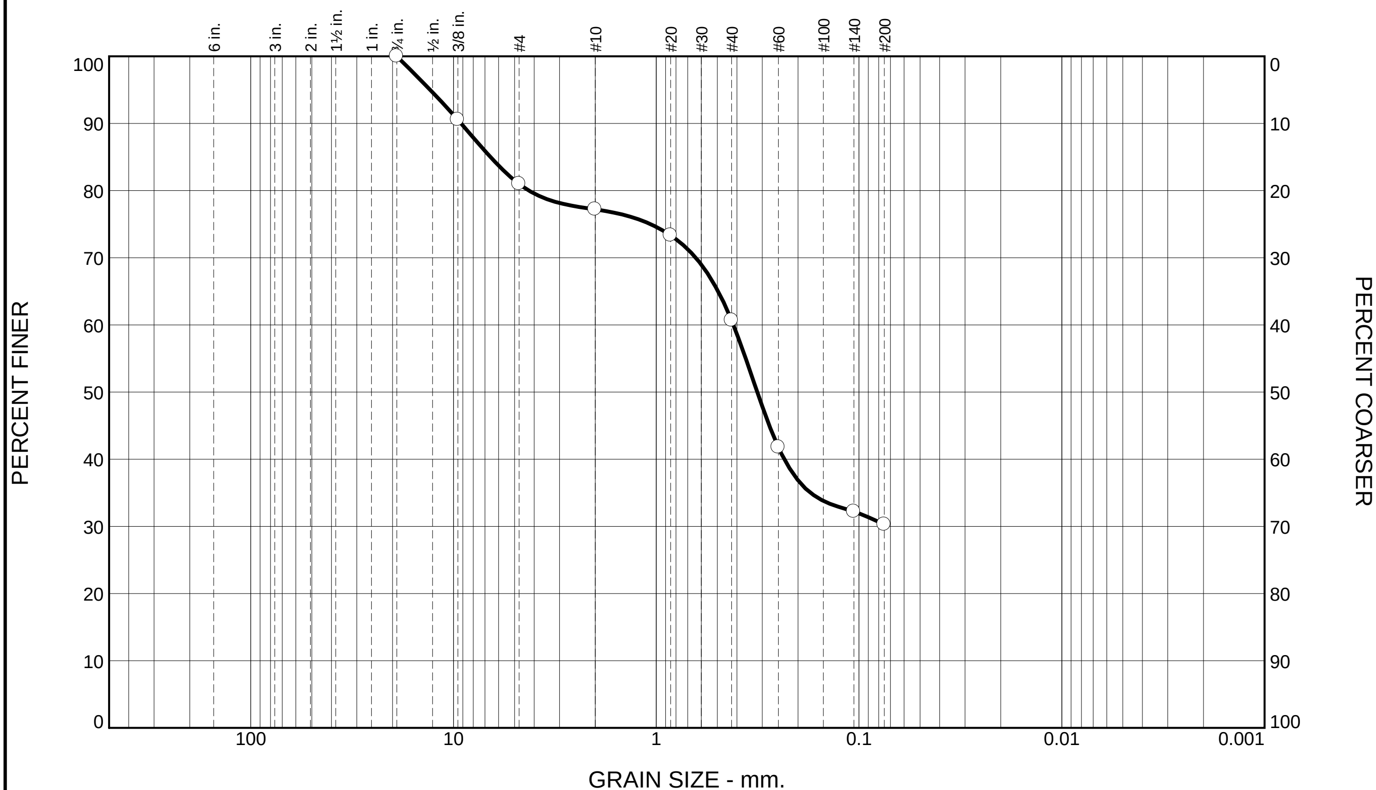
Source of Sample: HA-2 **Depth:** 35

Date: 12-12-19

Client: Haley & Aldrich, Inc.
Project: San Jose Fire Training Center
134258-002
Project No: 2831-028.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	19	4	16	31	30	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100		
3/8	91		
#4	81		
#10	77		
#20	73		
#40	61		
#60	42		
#140	32		
#200	30		

* (no specification provided)

Soil Description

Grayish brown clayey sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 9.1839 D_{85}= 6.5773 D_{60}= 0.4165
D_{50}= 0.3177 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

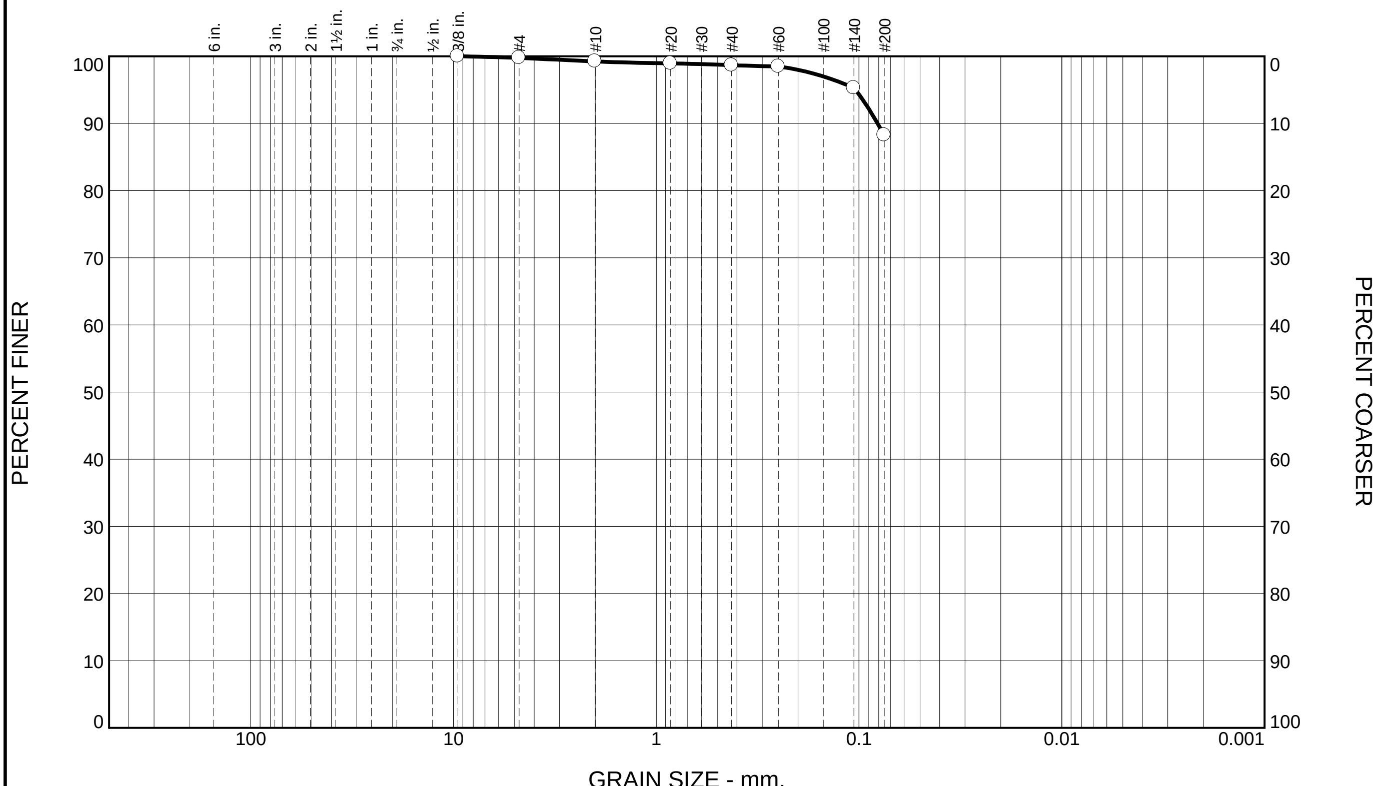
Source of Sample: HA-3 **Depth:** 10

Date: 12-12-19

Client: Haley & Aldrich, Inc.
Project: San Jose Fire Training Center
134258-002
Project No: 2831-028.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	1	0	11	88	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100		
#4	100		
#10	99		
#20	99		
#40	99		
#60	98		
#140	95		
#200	88		

* (no specification provided)

Soil Description

Grayish brown clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.0810 D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

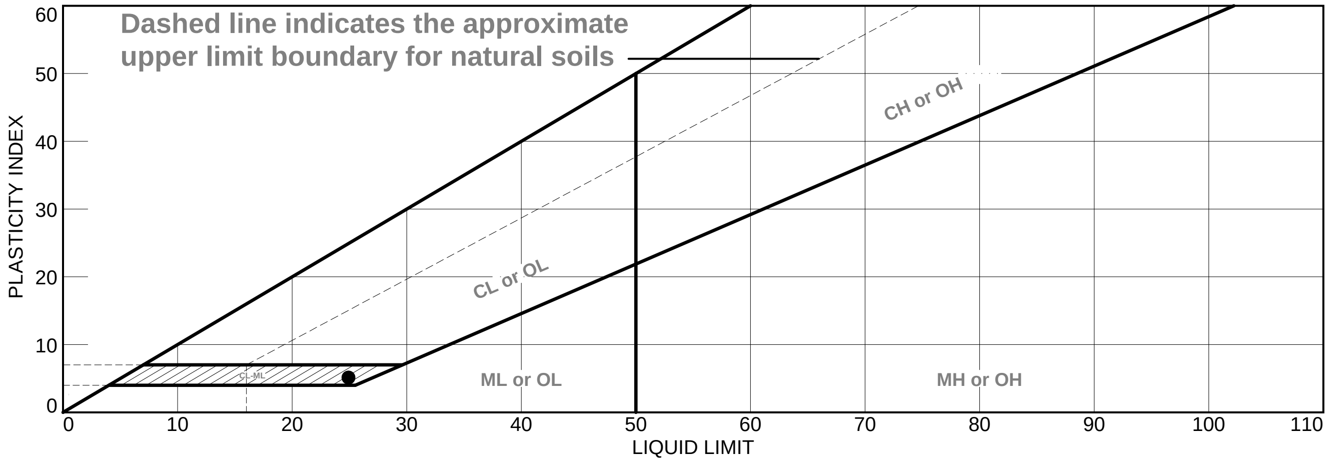
Source of Sample: HA-3 **Depth:** 36

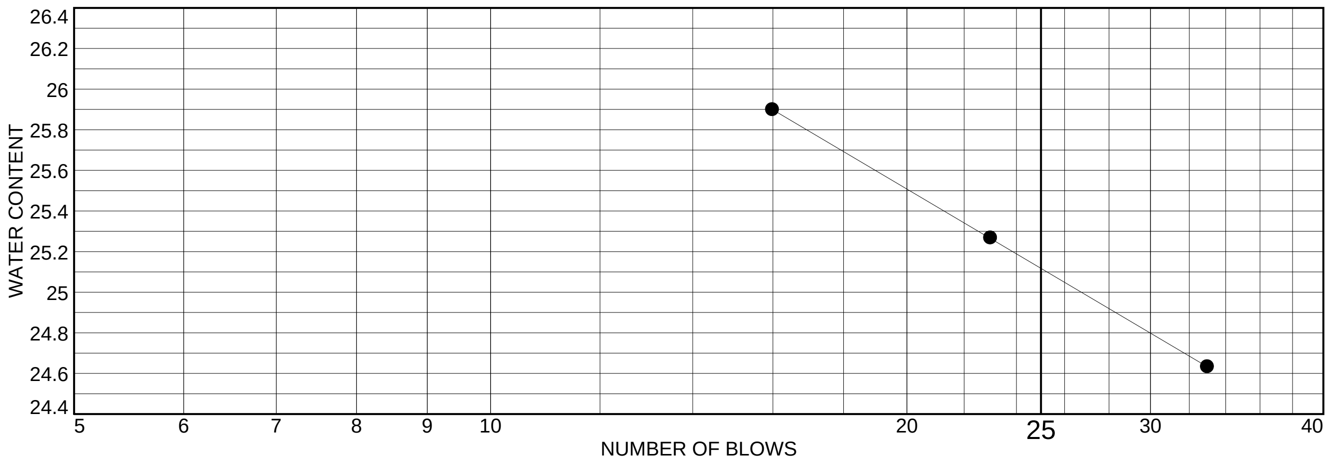
Date: 12-12-19

Client: Haley & Aldrich, Inc.
Project: San Jose Fire Training Center
134258-002
Project No: 2831-028.0 **Figure**

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown silty clay with sand	25	20	5			ML-CL

Project No. 2831-028.0 **Client:** Haley & Aldrich, Inc.

Project: San Jose Fire Training Center

134258-002

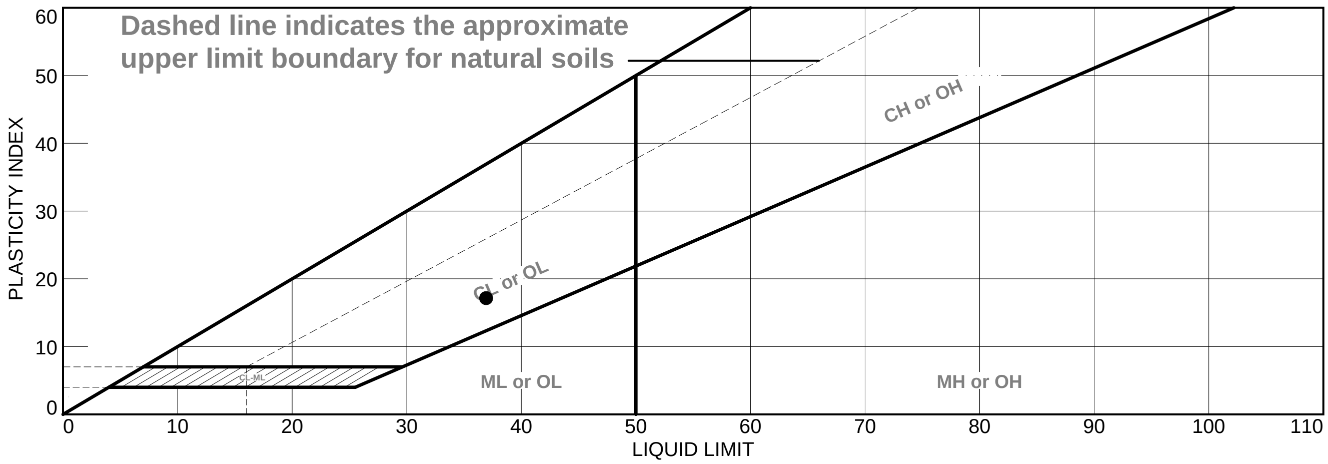
● **Source of Sample:** HA-1 **Depth:** 6

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown clay	37	20	17			CL

Project No. 2831-028.0 **Client:** Haley & Aldrich, Inc.

Project: San Jose Fire Training Center

134258-002

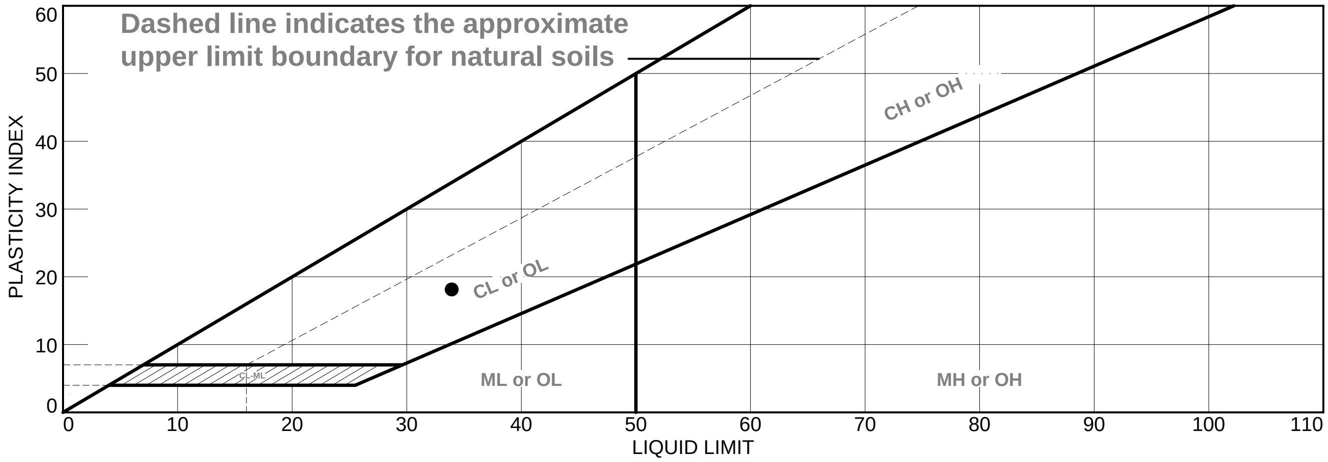
● **Source of Sample:** HA-2 **Depth:** 5

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish brown clay	34	16	18			CL

Project No. 2831-028.0 **Client:** Haley & Aldrich, Inc.

Project: San Jose Fire Training Center

134258-002

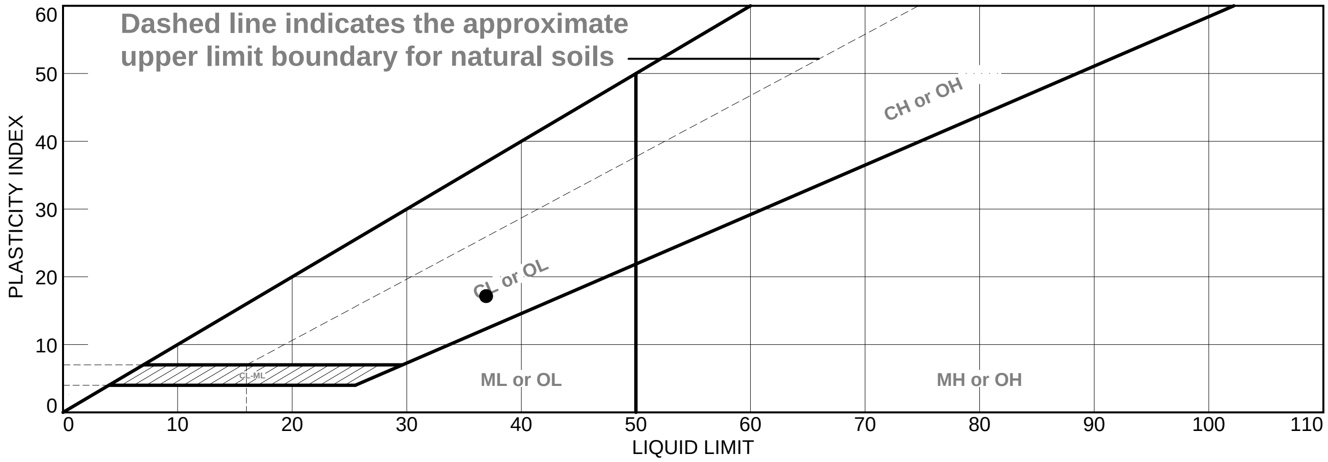
● **Source of Sample:** HA-2 **Depth:** 26

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish brown clay	37	20	17			CL

Project No. 2831-028.0 **Client:** Haley & Aldrich, Inc.

Project: San Jose Fire Training Center

134258-002

● **Source of Sample:** HA-3 **Depth:** 6

Remarks:

Figure

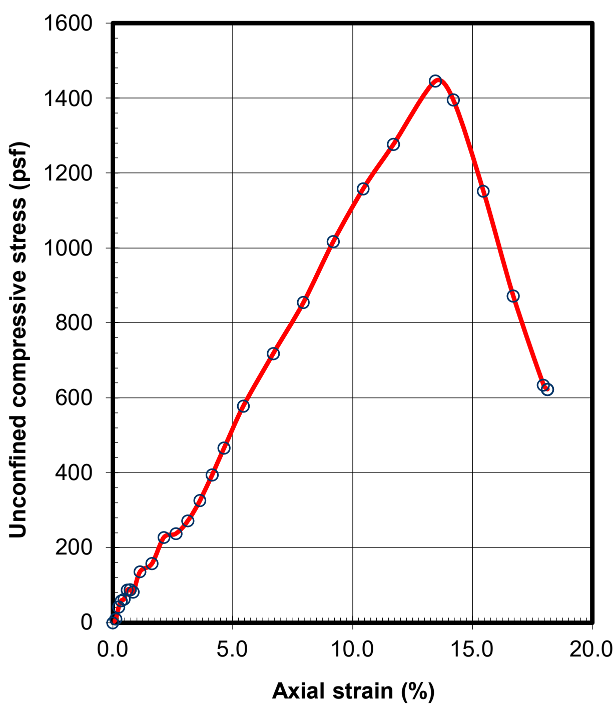
Tested By: JH **Checked By:** JH

UNCONFINED COMPRESSION TEST
ASTM D-2166

Client : Haley & Aldrich, Inc.
Project Name : San Jose Fire Training Center
Project Number : 134258-002
Boring Number HA-3
Sample Number :
Depth (ft) : 31
Date tested : 12/08/19
Soil : Brown silt (soft & saturated)

Specimen:			
	Total wt. =	791.37	gms
	Ht. =	5.38	in
	Ave dia. =	2.40	in
	Area =	4.51	sq.in
	Volume =	397.82	c.c.
	Shearing rate =	0.07	inch/min
	Shearing rate =	0.75	%/min
	Gs (assumed) =	2.70	

Test Report:			
	Void ratio =	0.742	
	Ht/Dia ratio =	2.24	
	Moisture =	28.33	%
	Total density =	124.19	pcf
	Dry density =	96.77	pcf
	Saturation =	103.1	%
	Unconfined compress. strength =	1445	psf
	Strain @ failure =	13.45	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0039	1.52	0.0	0.00
0.0106	1.83	10.2	0.12
0.0174	2.83	41.9	0.25
0.0227	3.37	58.8	0.35
0.0294	3.51	63.2	0.47
0.0365	4.28	87.6	0.61
0.0432	4.29	88.0	0.73
0.0499	4.10	81.6	0.86
0.0648	5.84	136.5	1.13
0.0917	6.55	157.8	1.63
0.1186	8.81	227.6	2.13
0.1455	9.17	237.8	2.63
0.1724	10.31	271.8	3.13
0.1994	12.13	326.2	3.63
0.2266	14.42	394.6	4.14
0.2535	16.84	466.2	4.64
0.2967	20.68	578.0	5.45
0.3640	25.65	718.4	6.70
0.4313	30.61	854.4	7.95
0.4986	36.63	1016.9	9.20
0.5658	42.07	1158.4	10.45
0.6331	46.83	1276.4	11.70
0.7273	53.87	1445.4	13.45
0.7677	52.50	1395.2	14.20
0.8350	44.21	1151.3	15.45
0.9022	34.35	872.4	16.71
0.9699	25.73	633.7	17.96
0.9787	25.36	622.6	18.13
0.9787	25.36	622.6	18.13

UNCONFINED COMPRESSION TEST
ASTM D-2166

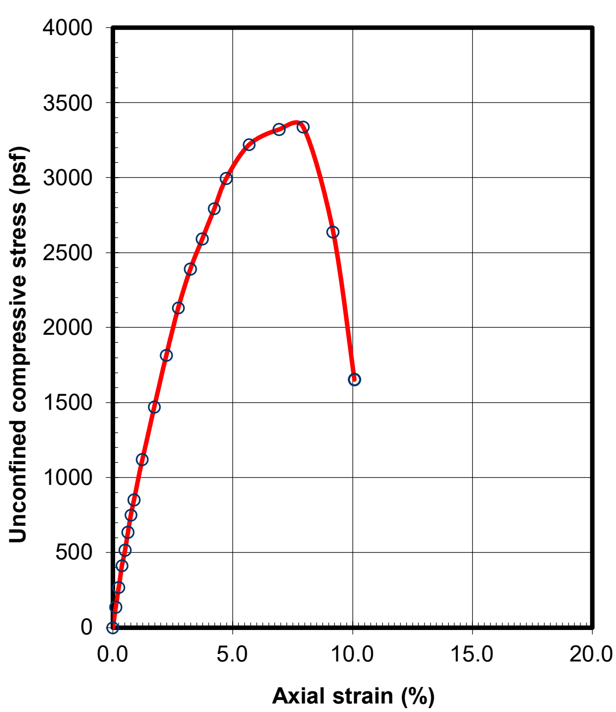
Client : Haley & Aldrich, Inc.
Project Name : San Jose Fire Training Center
Project Number : 134258-002
Boring Number HA-3
Sample Number :
Depth (ft) : 4
Date tested : 12/08/19
Soil : Brown clay

Specimen:

Total wt. =	834.67	gms
Ht. =	5.36	in
Ave dia. =	2.40	in
Area =	4.51	sq.in
Volume =	395.70	c.c.
Shearing rate =	0.07	inch/min
Shearing rate =	0.75	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.483	
Ht/Dia ratio =	2.24	
Moisture =	15.84	%
Total density =	131.69	pcf
Dry density =	113.68	pcf
Saturation =	88.6	%
Unconfined compress. strength =	3338	psf
Strain @ failure =	7.94	%



(see worksheet for sketch of failure)

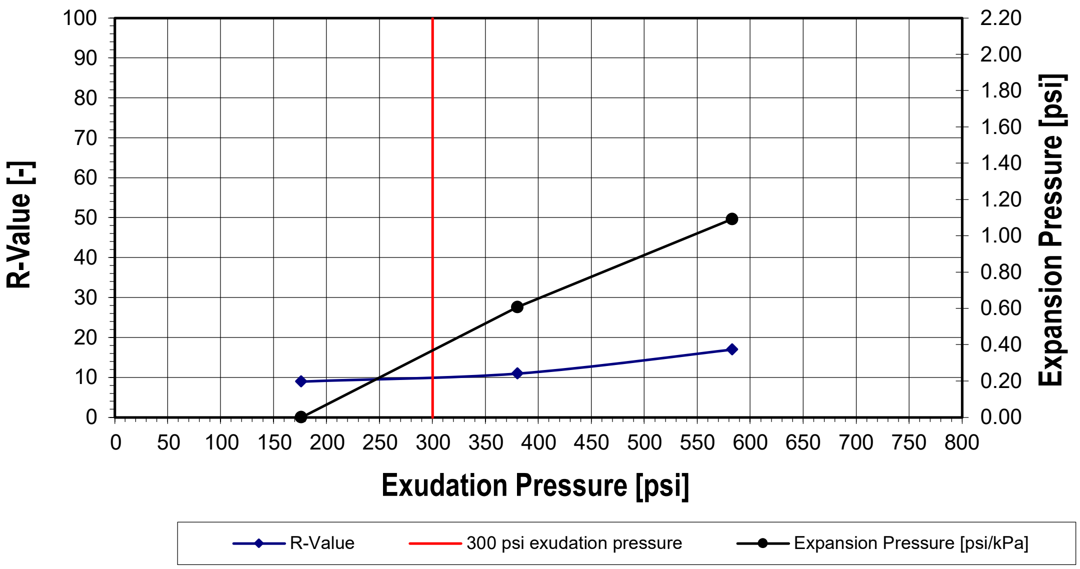
Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0037	5.49	0.0	0.00
0.0104	9.80	137.6	0.12
0.0171	13.86	267.1	0.25
0.0241	18.44	412.5	0.38
0.0308	21.71	515.8	0.51
0.0375	25.49	635.4	0.63
0.0442	29.17	751.2	0.76
0.0509	32.39	852.3	0.88
0.0696	40.95	1119.7	1.23
0.0965	52.30	1470.4	1.73
0.1233	63.61	1816.3	2.23
0.1502	74.05	2131.6	2.73
0.1770	82.78	2390.8	3.23
0.2038	89.70	2591.2	3.73
0.2306	96.76	2793.8	4.23
0.2574	103.85	2995.2	4.73
0.3086	112.29	3219.8	5.69
0.3756	117.15	3321.6	6.94
0.4292	118.92	3338.0	7.94
0.4962	96.37	2638.1	9.19
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08
0.5439	63.04	1654.2	10.08

R-Value ASTM D2844 / CT301

Project Name:	San Jose Fire Training Facility	**Clients Project No.:**	134258-002
Client Name:	Haley & Aldrich, Inc.	**ISI Project No.:**	2831-028.0
Descripton (Visual):	Brown sandy clay with gravel	**ISI Lab No.:**	G-64199
Boring:	Grab	**Test Date:**	12/17/19
Sample No.:	1	**Run By:**	JH
Depth (ft):	1-5	**Checked By:**	JH

Specimen #	1		2		3	
Compaction Pressure [**psi**/kPa]	**120**	827	**185**	1276	**265**	1827
Total Moisture [%]	15.3		14.0		12.7	
Density[pcf]	113.9		116.6		118.5	
Expansion Pressure [**psi**/kPa]	**0.00**	0.00	**0.61**	4.18	**1.09**	7.53
Horizontal Pressure at 160 psi [**psi**/kPa]	**141**	972	**134**	924	**125**	862
Number of Turns D [-]	**4.08**		**3.76**		**3.36**	
Sample Height [in./mm]	**2.58**	65.5	**2.51**	63.8	**2.50**	63.5
Exudation Pressure [**psi**/kPa]	**176**	1215	**380**	2623	**583**	4019
R-Value [-]	7.6		11.4		17.2	
Corrected R-Value [-]	9		11		17	



Corrected R-Value at 300 psi / 2.07 MPa Exudation Pressure = 10

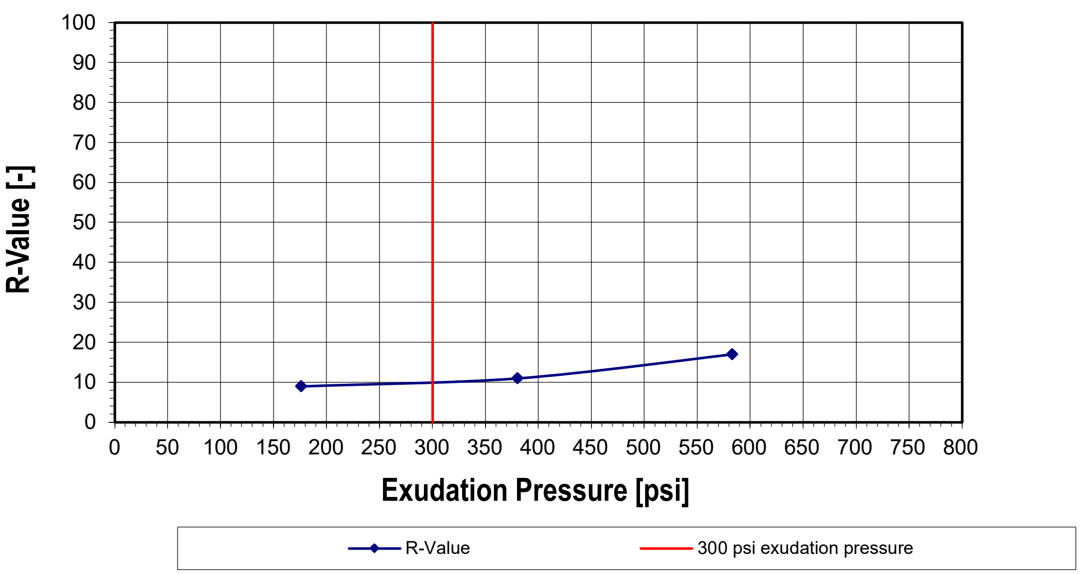
R-Value ASTM D2844 / CT301

Clients Project No.: 134258-002
ISI Project No.: 2831-028.0
ISI Lab No.: G-64199

Project Name:	San Jose Fire Training Facility
Client Name:	Haley & Aldrich, Inc.
Descripton (Visual):	Brown sandy clay with gravel
Boring:	Grab
Sample No.:	1
Depth (ft):	1-5

Test Date: 12/17/19
Run By: JH
Checked By: JH

Specimen #	1		2		3	
Compaction Pressure [**psi**/kPa]	**120**	827	**185**	1276	**265**	1827
Total Moisture [%]	15.3		14.0		12.7	
Density[pcf]	113.9		116.6		118.5	
Expansion Pressure [**psi**/kPa]	**0.00**	0.00	**0.00**	0.00	**0.00**	0.00
Horizontal Pressure at 160 psi [**psi**/kPa]	**141**	972	**134**	924	**125**	862
Number of Turns D [-]	**4.08**		**3.76**		**3.36**	
Sample Height [in./mm]	**2.58**	65.5	**2.51**	63.8	**2.50**	63.5
Exudation Pressure [**psi**/kPa]	**176**	1215	**380**	2623	**583**	4019
R-Value [-]	7.6		11.4		17.2	
Corrected R-Value [-]	9		11		17	



Corrected R-Value at 300 psi / 2.07 MPa Exudation Pressure = **10**

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

20 December, 2019

Job No. 1912032
Cust. No. 12468

Mr. Rati Mandzulashvili
Haley & Aldrich
2033 No. Main Street, Suite 309
Walnut Creek, CA 94596-7260

Subject: Project No.: 134258-002
Project Name: 1661 Senter Rd., (SJ Fire Training Center)
Corrosivity Analysis – CalTrans Test Methods

Dear Mr. Mandzulashvili:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on December 02, 2019. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "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 is 24 mg/kg and is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 31 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 7.99 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

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.

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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

Client:	Haley & Aldrich
Client's Project No.:	134258-002
Client's Project Name:	1661 Senter Rd. (SJ Fire Training Center)
Date Sampled:	18-Nov-19
Date Received:	2-Dec-19
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 20-Dec-2019

Job/Sample No.	Sample I.D.	Moisture (%)	pH	Min.Resistivity (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1912032-001	HA-2-3.0	-	7.99	1,400	-	24	31
Method:		CT 226 [(a)]	CT 643 [(b)]	CT 643 [(b)]	-	CT 422 [(c)]	CT 417 [(c)]
Reporting Limit:		-	-	-	50	15	15
Date Analyzed:		-	13-Dec-2019	19-Dec-2019	-	13-Dec-2019	13-Dec-2019

* Results Reported on an "As Received" Basis

(a) Rev. July 2010 (b) Rev. June 2007 (c) Rev. November 2006

Cheryl McMillen
Laboratory Director

<u>Quality Control Summary</u> - **All laboratory quality control parameters were found to be within established limits**

Chain of Custody

Page of

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

Job No.	CU#	Client Project I.D.	Schedule	Date Sampled	Date Due
1911033	12448	134258-002	Analyte: CalTrans w/Brief Evalation	11/18/19	

Full Name Rati Mandzulashvili **Fax** **Phone** 408-961-4815

Company Haley & Aldrich **Cell** 7323066910 ☒

Sample Source 1661 Senter Rd (SJ Fire Training Center)

ANALYSIS

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	pH	Sulfate	Chloride	Resistivity-Minimum	Brief Evaluation
001	HA-2-3.0	~~11/27/19~~ 11/18/19		S					X	X	X	X	X

MATRIX	ABBREVIATIONS	SAMPLE RECEIPT
DW - Drinking Water	HB - Hosebib	Total No. of Containers
GW - Ground Water	PV - Petcock Valve	Rec'd Good Cond/Cold
SW - Surface Water	PT - Pressure Tank	Conforms to Record
WW - Waste Water	PH - Pump House	Temp. a t Lab -°C
Water	RR - Restroom	Sampler
SL - Sludge	GL - Glass	
S - Soil	PL - Plastic	
Product	ST - Sterile	

		Date	Time
Relinquished By:	Rati Mandzulashvili		
Received By:	Diang Smith	11/18/19	1030
Relinquished By:			
Received By:			
Relinquished By:			
Received By:			

Comments:
THERE IS AN ADDITIONAL CHARGE FOR METAL/POLY TUBES