ROCKRIDGE GEOTECHNICAL

Prepared for **First Community Housing**

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
280 McEVOY STREET
SAN JOSE, CALIFORNIA

April 27, 2018
Project No. 18-1465

270 Grand Avenue
Oakland, CA 94610
www.rockridgegeo.com
510 420-5738 tel
510 652-3096 fax

ROCKRIDGE GEOTECHNICAL

April 27, 2018
Project No. 18-1465

Ms. Jessica de Wit
Senior Project Manager
First Community Housing
75 East Santa Clara Street, Suite 1300
San Jose, California 95113

Subject: Geotechnical Consultation
Proposed Residential Building
280 McEvoy Street
San Jose, California

Dear Ms. de Wit:

We are pleased to present our geotechnical consultation report, dated April 27, 2018, for the proposed residential building to be constructed at 280 McEvoy Street in San Jose. Our geotechnical consultation was provided in accordance with our proposal dated February 22, 2018. A geotechnical investigation for a previously proposed project at this site was performed by Stevens Ferrone & Bailey Engineering Company, Inc. (SFB), the results of which were presented in a report titled *Geotechnical Investigation, 699, 740 & 777 West San Carlos Street, San Jose, California*, dated October 1, 2015. We relied on the boring logs and laboratory test data provided in the SFB report in developing the conclusions and recommendations presented in our report.

The trapezoidal-shaped project site encompasses an area of about 1.13 acres and is bordered by McEvoy Street to the west, Dupont Street to the east, W. San Carlos Street to the south, and a light industrial development to the north. The southern portion of the site is occupied by a one-story metal industrial building. The remainder of the lot is occupied by a small, one-story garage and a paved area used for parking and storage.

Plans are to construct an at-grade residential building that would occupy most of the property except for the triangular-shaped parcel along W. San Carlos Street, which will be a common open area. The proposed building will be of reinforced concrete construction and will include 10 stories of residential units above two stories of parking. The residential units will be constructed in two structures, a rectangular-shaped tower containing studios and an L-shaped tower containing 2- and 3-bedroom units, separated by open space. Preliminary structural loading information provided by Vertech

Engineering indicates the typical dead-plus-live column loads will be 1,620 kips and the average bearing pressure imposed by the building (i.e., total building weight divided by the building footprint area) for dead-plus-live load conditions will be approximately 2,320 pounds per square foot (psf).

Based on the results of our engineering analyses, we conclude there are no major geotechnical issues that would preclude development of the site as proposed. The primary geotechnical issues affecting the proposed development include: 1) the presence of highly expansive near-surface soil that is susceptible to large volume changes with changes in moisture content; 2) the potential for up to 3/4 inch of liquefaction-induced settlement at the site following a major earthquake; and 3) providing adequate foundation support to limit the total and differential settlement for the proposed building.

We estimate total settlement of the proposed building supported on a properly designed and constructed mat foundation bearing on unimproved native soil will be between 4 and 6 inches and differential settlement will be on the order of 1-1/2 to 2 inches over a horizontal distance of 30 feet. Because this estimated differential settlement exceeds typical structural and architectural tolerances, we conclude ground improvement should be performed to a depth of about 40 feet below a mat foundation for the building to reduce total settlement to less than four inches and differential settlement to less than 3/4 inch over a horizontal distance of 30 feet. Alternatively, the proposed building could be supported on deep foundations gaining support through skin friction.

The previous investigation performed by SFB was for a significantly lighter, low-rise structure and, therefore, the borings and CPTs did not extend below a depth of 50 feet. Considering the large footprint and weight of the proposed structure, the settlement of the soil below a depth of 50 feet needs to be considered in the overall settlement analysis. For the purposes of this report, we assumed the stiff to very stiff clay encountered between depths of 40 to 50 feet bgs extends down to a depth of 150 feet; however, this assumption needs to be confirmed with additional borings and CPTs that extend to a depth of at least 150 feet below the existing ground surface.

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

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
CRAIG S. SHIELDS
GE2116
3/31/19
GEOTECHNICAL
STATE OF CALIFORNIA

Craig S. Shields, P.E., G.E.
Principal Geotechnical Engineer

Enclosure

ROCKRIDGE GEOTECHNICAL

TABLE OF CONTENTS

REFERENCES

Boulanger, R.W and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures", Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, Report No. UCD/CGM-14/01, April.

California Building Code (2013).

California Division of Mines and Geology (1996), Probabilistic seismic hazard assessment for the State of California, DMG Open-File Report 96-08.

California Geological Survey (2006a), State of California Seismic Hazard Zones, San Jose West Quadrangle, Official Map, February 7, 2002.

California Geological Survey (2002), State of California Seismic Hazard Zone Report for the San Jose West 7.5-Minute Quadrangle, Plate 1.2, revised October 10, 2005.

Cao, T., Bryant, W. A., Rowshandel, B., Branum D. and Wills, C. J. (2003). "The Revised 2002 California Probabilistic Seismic Hazard Maps"

Helley, E.J., Graymer, R.W., Phelps, G.A., Showalter, P.K., and Wentworth, C.M. (1994), "Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California: A digital database," May 1994.

GeoLogismiki, 2014, CLiq, Version 1.7.

U.S. Geological Survey, (2008), The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): prepared by the 2007 Working Group on California Earthquake Probabilities, U.S. Geological Survey Open File Report 2007-1437.

Zhang, G., Robertson. P.K., Brachman, R., (2002), "Estimating Liquefaction Induced Ground Settlements from the CPT", Canadian Geotechnical Journal, 39: pp 1168-1180

LIST OF FIGURES

GEOTECHNICAL CONSULTATION
PROPOSED RESIDENTIAL BUILDING
280 McEVOY STREET
San Jose, California

1.0 INTRODUCTION

This report presents the results of the geotechnical consultation provided by Rockridge Geotechnical for the proposed residential building to be constructed at 280 McEvoy Street (also referred to as 699 W. San Carlos Street) in San Jose, California. The project site is on the northern side of W. San Carlos Street between McEvoy and Dupont streets, as shown on the Site Location Plan (Figure 1).

The trapezoidal-shaped project site encompasses an area of about 1.13 acres and is bordered by McEvoy Street to the west, Dupont Street to the east, W. San Carlos Street to the south, and a light industrial development to the north. The southern portion of the site is occupied by a one-story metal industrial building. The remainder of the lot is occupied by a small, one-story garage and a paved area used for parking and storage.

Plans are to construct an at-grade residential building that would occupy most of the property except for the triangular-shaped parcel along W. San Carlos Street, which will be a common open area. The proposed building will be of reinforced concrete construction and will include 10 stories of residential units above two stories of parking. The residential units will be constructed in two structures, a rectangular-shaped tower containing studios and an L-shaped tower containing 2- and 3-bedroom units, separated by open space. Based on preliminary structural loading information provided by Vertech Engineering, we understand the typical dead-plus-live column loads will be 1,620 kips and the average bearing pressure imposed by the building (i.e., total building weight divided by the building footprint area) for dead-plus-live load conditions will be approximately 2,320 pounds per square foot (psf).

2.0 PREVIOUS GEOTECHNICAL INVESTIGATION

A geotechnical investigation for a previously proposed project at this site was performed by Stevens Ferrone & Bailey Engineering Company, Inc. (SFB), the results of which were presented in a report titled *Geotechnical Investigation, 699, 740 & 777 West San Carlos Street, San Jose, California*, dated October 1, 2015. As part of their investigation, SFB drilled two borings, designated as SFB-1 and SFB-4, and performed two cone penetration tests (CPTs), designated as CPT-4 and CPT-5, on the subject property at the approximate locations shown on the Site Plan, Figure 2. The borings were drilled to depths ranging from 21.5 to 36.5 feet below the ground surface (bgs) and the CPTs were each advanced to a depth of 50 feet bgs. The borings were drilled using a truck-mounted drill rig equipped with solid-stem continuous flight augers and the CPTs were performed with a 25-ton truck-mounted CPT rig.

SFB also performed laboratory tests on selected samples from the borings to measure moisture content, dry density, Atterberg limits, gradation, and unconfined compressive strength. The logs of the borings and CPTs and the results of laboratory tests performed by SFB are presented in Appendices A and B of this report, respectively.

3.0 SCOPE OF SERVICES

Our consultation was provided in accordance with our proposal dated February 22, 2018. Our scope of work consisted of reviewing the existing subsurface data and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s)
- estimates of foundation settlement
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- site grading and excavation, including criteria for fill quality and compaction
- subgrade preparation for concrete slab-on-grade floors and concrete flatwork

- 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

4.0 SUBSURFACE CONDITIONS

As presented on the Regional Geologic Map (Figure 3), the site is mapped as being underlain by Holocene-age alluvial deposits (Qha). The CPTs and borings advanced by SFB indicate the site is blanketed by about 3 to 7 feet of fill consisting of stiff to hard clay below the pavement section. An Atterberg limits test on a sample of the fill indicates the clay is highly expansive[1] with a plasticity index (PI) of 28. At the SFB-4 location and at both CPT locations, the fill is underlain by very stiff native clay that extends to a depth of about eight feet bgs. The clay fill and native clay (where present) is underlain by a layer of medium dense to very dense sand with varying silt content that extends to depths ranging from about 20 to 24 feet bgs. Below the sand layer is stiff to very stiff clay interbedded with occasional lenses or layers of medium dense to very dense sand with varying fines content that extend to the maximum depth explored of 50 feet bgs.

Groundwater was measured in the SFB borings and CPT holes at depths of about 35 to 44 feet bgs; however, the borings and CPT holes were backfilled with neat cement grout on the same day they were advanced and, therefore, the measurements may not represent a stabilized groundwater level. Further, the measurements were taken after several years of drought and, therefore, the groundwater level would be expected to be significantly below the historic high groundwater level. The seismic hazard zone report prepared by the California Geological Survey (CGS) for the San Jose West Quadrangle (2002) indicates the historic high groundwater level is about 25 feet bgs. The depth to groundwater is expected to vary several feet seasonally, depending on the amount of rainfall.

---

[1] Highly expansive soil undergoes large volume changes with changes in moisture content.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity and Faulting

The site is located in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent shearing along the San Andreas Fault system. Movements along this plate boundary in the Northern California region occur along right-lateral strike-slip faults of the San Andreas Fault system.

The major active faults in the area are the San Andreas, Hayward, and Calaveras faults. These and other faults in the region are shown on Figure 4. Active faults within a 50-kilometer radius of the site, the distance from the site and mean characteristic moment magnitude[2] [2007 Working Group on California Earthquake Probabilities (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

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

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	10	Southwest	6.50
Total Calaveras	15	East	7.03
Total Hayward	15	Northeast	7.00
Total Hayward-Rodgers Creek	15	Northeast	7.33
N. San Andreas - Peninsula	18	Southwest	7.23
N. San Andreas (1906 event)	18	Southwest	8.05
N. San Andreas - Santa Cruz	19	South	7.12
Zayante-Vergeles	27	South	7.00
Greenville Connected	38	East	7.00
San Gregorio Connected	41	West	7.50
Mount Diablo Thrust	46	Northeast	6.70
Monterey Bay - Tularcitos	49	Southwest	7.30

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 east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 32 kilometers southeast of 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 U.S. Geological Survey's 2014 Working Group on California Earthquake Probabilities has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Region during the next 30 years (starting from 2014) is 72 percent. The highest probabilities are assigned to the Hayward fault, Calaveras fault, and the northern segment of the San Andreas fault. These probabilities are 14.3, 7.4, and 6.4 percent, respectively.

5.2 Seismic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture, liquefaction[3], lateral spreading[4] and cyclic densification.[5] We used the results of the CPTs and borings advanced by SFB for their investigation to evaluate the potential of these phenomena occurring at the project site. The results of our analyses and evaluation are presented in the following sections.

---

3 Liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences temporary reduction in strength during cyclic loading such as that produced by earthquakes.

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

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

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the San Andreas and Hayward Faults, although ground shaking from future earthquakes on other faults, including the Monte Vista-Shannon and Calaveras Faults, will also be felt at the site. These and other faults in the region are shown in relation to the site on Figure 4. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, and magnitude and duration of the earthquake. We judge that strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

5.2.2 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

As shown on Figure 5, the site has been mapped within a zone of liquefaction potential as shown on the map titled *State of California, Seismic Hazard Zones, San Jose West Quadrangle, Official Map,* prepared by the California Geological Survey (CGS, 2006a), dated February 7, 2002.

Liquefaction susceptibility was assessed using the software CLiq v1.7.6.49 (GeoLogismiki, 2015). CLiq uses measured field CPT data and assesses liquefaction potential, including post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). Our liquefaction analyses were performed using the methodology proposed by Boulanger and Idriss (2014). We also used the relationship proposed by Zhang, et al (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; a relationship that is an extension of the work by Ishihara and Yoshimine (1992).

Our liquefaction analyses were performed using an assumed "during earthquake" groundwater depth of 25 feet bgs. In accordance with the 2016 California Building Code (CBC), we used a peak ground acceleration of 0.50 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a moment magnitude 8.05 earthquake, which is consistent with the mean characteristic moment magnitude for the Northern San Andreas Fault (1906 rupture), as presented in Table 2.

Our liquefaction analyses indicate there is an approximately 1- to 3-foot-thick layer of potentially liquefiable soil underlying the site between depths of about 28 and 36 feet bgs. Based on the CPTs performed for this project, we estimate total settlement resulting from liquefaction (referred to as post-liquefaction reconsolidation) during an MCE event generating a PGA_M of 0.50g will be less than about 3/4 inch and differential settlement will be less than approximately 1/2 inch over a horizontal distance of 30 feet.

Ishihara (1985) presented empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced ground failure, such as sand boils, would be expected to occur under a given level of shaking for a liquefiable layer of given thickness overlain by a resistant, or protective, surficial layer. Our analysis indicates the non-liquefiable soil overlying the potentially liquefiable soil layer is sufficiently thick and the potentially liquefiable layer is sufficiently thin such that the potential for surface manifestations of liquefaction, such as sand boils, is nil.

Considering the relatively flat site grades and the absence of a free face in the site topography, as well as the depth and relative thickness of the potentially liquefiable layer, we conclude the risk of lateral spreading is nil.

5.2.3 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The CPTs and borings indicate the soil above the groundwater at the site is not susceptible to cyclic densification due to its cohesion or high relative density. Accordingly, we conclude the potential for ground surface settlement resulting from cyclic densification is low.

5.2.4 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, and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

6.0 DISCUSSION AND CONCLUSIONS

Based on the results of our engineering analyses, we conclude there are no major geotechnical issues that would preclude development of the site as proposed. The primary geotechnical issues affecting the proposed development include:

- the presence of highly expansive near-surface soil that is susceptible to large volume changes with changes in moisture content;
- the potential for up to 3/4 inch of liquefaction-induced settlement at the site following a major earthquake;
- providing adequate foundation support to limit the total and differential settlement for the proposed building.

Our conclusions and recommendations regarding these issues are presented in the following sections.

6.1 Expansive Soil

Atterberg limits tests performed on samples of the near-surface clay fill indicate the material is highly expansive. Highly expansive near-surface soil is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause movement and cracking of foundations, pavements, slabs, and below-grade walls. Therefore, pavements, slabs, and below-grade walls should be designed and constructed to mitigate the effects of the expansive soil. In general, the effects of expansive soil can be mitigated by moisture-conditioning the expansive soil, providing select, non-expansive fill or lime-treated soil below interior and exterior slabs and behind retaining walls. Our conclusions and recommendations regarding appropriate the foundation types for the proposed building are presented below in Section 6.2.

Considering the expansive clay that will be exposed at subgrade level is susceptible to disturbance from equipment traffic, soil subgrade stabilization methods may be needed to limit disturbance of the prepared subgrade. In addition, at expansive soil sites it is critical to properly manage surface and subsurface drainage to prevent water from collecting beneath pavements and slabs or behind below-grade walls, where it can lead to cyclic swelling and shrinking of the subgrade soil and can cause subgrade instability under vehicular loads. If permeable pavements, tree wells, irrigated landscaped zones, and storm water infiltration basins will be constructed near the proposed buildings, they should incorporate design elements that prevent saturation of the soil below foundations. While the objective of permeable pavement systems and infiltration basins is to allow for water storage and infiltration, we conclude that infiltration into the subgrade soil is not feasible at this site due to the low permeability of the highly expansive clay. Furthermore, from a geotechnical standpoint, water should not be allowed to collect alongside or beneath pavements and flatwork. This can be achieved by providing subdrain systems beneath permeable surfaces and installing vertical barriers between permeable surfaces underlain by subdrains and non-permeable surfaces underlain by conventional aggregate base.

6.2 Foundations and Settlement

We estimate total settlement of the proposed building supported on a properly designed and constructed mat foundation bearing on unimproved native soil will be between 4 and 6 inches and differential settlement will be on the order of 1-1/2 to 2 inches over a horizontal distance of 30 feet. Because this estimated differential settlement exceeds typical structural and architectural tolerances, we conclude ground improvement should be performed below a mat foundation for the building to reduce total settlement to less than four inches and differential settlement to less than 3/4 inch over a horizontal distance of 30 feet. Alternatively, the proposed building could be supported on deep foundations gaining support through skin friction.

6.2.1 Mat Foundation with Ground Improvement

There are several types of ground improvement that may be utilized to reduce the differential settlement of the building to a tolerable amount (typically considered to be 3/4 inch between columns), as well as to increase the allowable bearing pressures, which can result in more economical and better performing foundations. We consider soil-cement mix (SMX) columns or drilled displacement sand-cement (DDSC) columns to be the most appropriate ground improvement methods for this project. SMX columns are installed by injecting and blending cement into the soil using a drill rig equipped with single or multiple augers. DDSC columns are installed by advancing a hollow-stem auger that mostly displaces the soil and then pumping a sand-cement mixture into the hole under pressure as the auger is withdrawn. This system results in low vibration during installation and generates fewer drilling spoils for off-haul. DDSC columns and SMX columns are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of columns should be determined by the contractor, based on the desired level of improvement. If soil improvement is to be considered, we recommend a preliminary design, including calculations of static and seismic settlement, be prepared by the ground improvement contractor and submitted for our review.

We performed preliminary settlement analyses for a mat foundation bearing on ground improved with DDSC or SMX columns that extend to a depth of 40 feet bgs. Because the available subsurface data from the SFB investigation only extends to a depth of 50 feet bgs and the proposed building (with or without ground improvement) will impose a significant pressure on the soil below a depth of 50 feet, we assumed the stiff to very stiff clay encountered below a depth of 40 feet bgs extends to a depth of at 150 feet bgs for the purposes of our settlement analyses. The results of our preliminary settlement analysis indicate installation of ground improvement elements to a depth of 40 feet below the mat foundation would reduce total settlement to less than about four inches. The static settlements are primarily due to recompression of the underlying, overconsolidated clay, which occurs relatively quickly. Therefore, we estimate about 80 to 90 percent of the total settlement will be complete by the end of construction.

6.2.2 Deep Foundations

Alternatively, the proposed building could be supported on deep foundations that derive support through skin friction and end bearing in the underlying alluvium. We evaluated the feasibility of the following deep foundation systems:

- drilled piers
- driven concrete or steel piles
- torque-down piles
- auger cast-in-place piles

We conclude drilled piers are not desirable for the site because of the presence of sandy and gravelly soil that are susceptible to caving. Installation of drilled piers would require casing and/or drilling slurry where caving soil is encountered. In addition, construction of drilled piers will generate a large volume of soil for off-haul.

We conclude driven concrete or steel piles are also not desirable for the site because of the relatively high vibrations and noise generated during pile driving. We anticipate the noise would be disruptive to the nearby businesses, as well as the occupants of the residential buildings on the south side of W. San Carlos Street.

We believe more appropriate deep foundation systems are proprietary pile types, such as torque-down piles (TDPs) or auger cast-in-place piles (ACIP) piles; provided these piles can be successfully installed to lengths of 70 feet to limit static settlement discussed later in this section.

A TDP is a steel pipe pile with a closed conical end with pitched flights that allow the pipe pile to be "screwed-in" to the soil, resulting in displacement and densification of the surrounding soil. The pipe typically used for the TDPs has an outside diameter of 12.75 inches and a wall thickness of 0.375 (3/8) inches. When the pipe pile is advanced to the design tip elevation, it is filled with structural concrete to provide additional bending resistance. TDPs are displacement piles installed with little spoils created to reduce off-haul. An advantage of the TDPs is they can be installed with minimal vibration and noise, as compared to driven piles.

ACIP piles are installed by advancing a continuous flight, hollow-stem, auger into the ground to a specified depth. Sand-cement grout or concrete is pumped into the hole under pressure as the auger is removed, eliminating the need for temporary casing or slurry. After the auger is removed, reinforcement can be installed while the cement grout or concrete is still fluid. Unlike driven piles, very little noise and vibrations are generated during the installation of the ACIP piles. ACIP piles are available with variable diameters; however, 16-inch-diameter is typical. Partial displacement ACIP piles may be installed by using specially manufactured augers to reduce spoils and off-haul. However, use of partial displacement auger will increase the potential of the ACIP pile encountering early refusal in very dense sand layers.

Assuming a pile length of 70 feet, we estimate total settlement of the building would be less than about two inches and differential settlement will be less than 3/4 inch over a horizontal distance of 30 feet. If the building is supported on pile foundations that are 70 feet long, we anticipate settlement of the building during an earthquake would be negligible. Therefore, the floor slab could settle as much as 3/4 inch differentially relative to the pile caps and grade beams. If this potential seismically induced differential settlement is not acceptable, the floor slab should be designed to span between pile caps and grade beams.

6.3 Soil Corrosivity

Corrosivity testing was previously performed by Cerco Analytical of Concord, California on a samples of soil obtained during SFB's field investigation from Borings SFB-3 and SFB-4 at depths of 6 and 3.5 feet bgs, respectively. The results of the tests are presented in Appendix B of this report. Based on the resistivity test results, the samples are classified as corrosive (resistivity of 1,000 ohm-cm) to highly corrosive (resistivity of 420 ohm-cm) to buried steel. The pH environment is not corrosive to buried metallic and concrete structures. The test results indicate that sulfate ion concentrations are insufficient to damage reinforced concrete structures below ground, and the chloride concentration of the soil does not present a problem with buried metallic structures and reinforcing steel in concrete structures.

7.0 PRELIMINARY RECOMMENDATIONS

Our preliminary recommendations for site grading, foundation design, below-grade walls, and seismic design are presented in this section of the report.

7.1 Site Preparation, Grading and Fill Placement

Site clearing should include removal of all existing pavements, former foundation elements, and underground utilities. The concrete and asphalt can be reused as engineered fill provided it is acceptable from an environmental standpoint and the materials are broken into pieces smaller than four inches in greatest dimension. These materials should be mixed with sufficient fine-grained material to minimize the presence of voids. Existing utility lines may be abandoned in place by grouting. If the lines will interfere with new construction, they should be removed.

Excavation for the building subgrade will likely expose highly expansive clay. Care should be taken to minimize disturbance to the mat subgrade during excavation. Heavy rubber-tired equipment should not be driven on the subgrade to reduce the potential for subgrade "pumping". If soft areas are encountered at the mat subgrade elevation, subgrade stabilization measures may be required.

In areas that will receive improvements (i.e. building pad, exterior concrete flatwork, etc.) or fill, the soil subgrade exposed following stripping and clearing should be scarified to a depth of at least eight inches, moisture-conditioned to at least four percent above optimum moisture content, and compacted to between 88 and 93 percent relative compaction (RC)[6]. Where the building will be supported on a mat foundation bearing on ground improvement, the subgrade of the mat foundation should be scarified, moisture-conditioned, and compacted to between 88 and 93 percent RC prior to installing ground improvement elements.

7.1.1 Fill Quality and Compaction

Where the on-site expansive clay is placed as fill or compacted, such as behind the below-grade walls or subgrade beneath flatwork, the material should be moisture-conditioned to at least four percent above optimum moisture content, and compacted to between 88 and 93 percent RC. Moisture conditioning may require aerating the soil to lower its moisture content or adding water if the material is too dry.

If imported fill is required, it should consist of select fill material that is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 15, and be approved by the geotechnical engineer.

Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent RC. Select fill greater than five feet in thickness, as well as the upper six inches of soil subgrade beneath vehicular pavements, should be compacted to at least 95 percent RC. Samples of proposed select fill material should be submitted to the geotechnical engineer at least five business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not provided, a minimum of two weeks will be required to perform any necessary analytical testing.

---

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

7.1.2 Utilities

The thickness and type of bedding material required for utilities outside the building footprint will depend on the soil conditions at the utility trench bottom. As a minimum, bedding should extend at least D/4 (with D equal to the outside pipe diameter) below the bottom of the pipe. However, the bedding should be at least four inches thick. This minimum bedding thickness and either clean sand, rod mill or pea gravel bedding material is adequate for shallow trenches above the groundwater level.

Backfill for utility trenches should be compacted according to the recommendations presented for general site fill presented in Section 7.1.1. Jetting of trench backfill should not be permitted. The soil excavated from the trenches can be reused to backfill the trenches, provided the material can be compacted to the required compaction. If sand or gravel with less than 10 percent fines (particles passing the No. 200 sieve) is used or more than five feet of backfill will be placed, it should be compacted to at least 95 percent relative compaction. Pea gravel, drain rock, and rod mill should be mechanically tamped in 12-inch lifts where placed beneath pavements. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Where utility trenches enter the building pad, an impermeable plug consisting of lean concrete or sand-cement slurry, at least three feet in length, should be installed. Furthermore, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

Utility conduits should be designed to have sufficient flexibility to resist differential settlement related to static loads and seismically-induced settlement.

7.1.3 Exterior Flatwork Subgrade Preparation

We recommend a minimum of four inches of Class 2 aggregate base be placed over six inches of non-expansive soil (i.e. select fill) beneath proposed exterior concrete flatwork; the non-expansive soil should extend at least six inches beyond the slab edges. Non-expansive soil beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned above optimum moisture content and compacted to at least 90 percent RC. Class 2 aggregate base beneath concrete flatwork should be compacted to at least 90 percent relative compaction.

Even with 10 inches of non-expansive soil (including aggregate base layer), exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slab edges and adding additional reinforcement will control this cracking to some degree. Where slabs are adjacent to landscaped areas, thickening the concrete edge will help control water infiltration beneath the slabs. In addition, where slabs provide access to building, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

We do not recommend the use of pervious interlocking pavers at this site, due to the expansive nature of the subgrade soil. If pavers must be incorporated into this project, we should provide additional recommendations for proper subgrade preparation and subsurface drainage.

7.2 Foundations

Provided the estimated settlements presented in Section 6.2 of this report are acceptable, the proposed building may be supported on a mat foundation bearing on ground improved with DDSC or SMX columns. Proprietary deep foundations, TDPs and ACIP piles, are also appropriate foundation systems for the proposed building.

7.2.1 Mat Foundation on Improved Ground

For preliminary design of a mat foundation bearing on improved ground, we recommend ground improvement elements extend to a minimum depth of 40 feet below the mat foundation. We anticipate the ground improvement systems described in Section 6.2.1, if properly designed and constructed, should be capable of increasing the maximum allowable bearing pressures to 9,000 pounds per square foot (psf) for dead-plus-live loads and 12,000 psf for total loads. The final design allowable bearing pressures, estimated settlements, and modulus of vertical subgrade reaction should be provided by the design-build ground improvement contractor, as these values will be dependent on the diameter, depth, and spacing of the ground improvement elements.

Lateral forces can be resisted by friction along the base of the mat and passive pressure against the sides of the mat foundation. To compute lateral resistance, we recommend using an allowable uniform pressure of 1,500 psf (rectangular distribution) for transient loads and an equivalent fluid weight (triangular distribution) of 270 pounds per cubic foot (pcf) for sustained loads. The upper foot of soil should be ignored for passive resistance unless it is confined by a slab or pavement. The allowable friction factor will depend on the type of waterproofing or vapor retarder placed below the mat, if any. If no waterproofing membrane or vapor retarder is installed below the mat foundation, an allowable base friction coefficient of 0.30 may be used. If Preprufe or a vapor retarder is placed below the mat foundation, a base friction factor of 0.20 should be used. For bentonite-based water proofing membranes, such as Paraseal or Voltex, a friction factor of 0.12 should be used (assumes a bentonite friction angle of 10 degrees). Friction factors for other types of waterproofing membranes can be provided upon request. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without further reduction.

The mat subgrade should be free of standing water, debris, and disturbed materials prior to placing concrete. The subgrade should be maintained in a moist condition until concrete is placed. We should inspect the mat subgrade prior to placement of reinforcing steel.

7.2.2 Deep Foundations

As an alternative to a mat foundation, the proposed building may be supported on proprietary pile types, such as TDPs or ACIP piles. In the San Francisco Bay Area, TDPs and ACIP piles are typically installed under a design-build contract by specialty foundation contractors. Therefore, the recommendations provided below may be modified by the pile subcontractor;

however, any design recommendations should be confirmed with at least two pile load tests in compression. In evaluating pile lengths, the reduced shear strength and settlement resulting from liquefaction should be taken into account. All piles should be at least 70 feet long and should be spaced at least three pile diameters, center-to-center, to prevent vertical capacity reductions due to pile interaction effects.

The corrosion potential of the soil, as discussed above in Section 6.3, should be taken into account in the design of the concrete, steel shell (for TDPs), and reinforcement for the piles. For TDPs, corrosion potential is typically addressed by either placing an epoxy coating on the upper portion of the pile or with a corrosion allowance. For the soil types on this project, a typical corrosion allowance is on the order 1/10 inch for a 50-year design life.

Axial Capacity

The piles for this project will gain support primarily from skin friction in the interbedded alluvial soil underlying the site. Recommendations for vertical capacities for 12.75-inch-diameter TDPs and 16-inch-diameter ACIP piles are presented in Table 2 below.

TABLE 2
Recommended Vertical Pile Capacities

Pile Type	Pile Length (feet)	$Q_{ultimate}$ Axial Capacity (kips)	$Q_{allowable}$ Dead Plus Live Load (kips)	$Q_{allowable}$ Total Design Load (kips)	$Q_{allowable}$ Uplift (kips)
12.75" TDP	70	500	250	330	200
16" ACIP	70	600	300	400	240

The capacities recommended above include factors of safety of 2.0 for compression and tension loading and 1.5 for total loads.

Lateral Capacities

The piles should develop lateral resistance from the passive pressure acting on the upper portion of the piles and their structural rigidity. The allowable lateral capacity of the piles depends on:

- the pile stiffness
- the strength of the surrounding soil
- axial load on the pile
- the allowable deflection at the pile head
- the allowable moment capacity of the pile.

We computed the allowable lateral loads that can be applied to the pile head to limit deflection to 0.5 inch or less for both free-head (zero moment) and fixed-head (zero slope) conditions. To account for the pile cap thickness, we assumed the tops of the piles would be three feet below the existing ground surface. Recommended lateral capacities for single piles (i.e., no group effects) are presented in Table 3 below. Deflection, moment and shear profiles for the piles can be provided upon request.

TABLE 3
Recommended Lateral Pile Capacities

Pile Type	Free-Head Conditions			Fixed-Head Conditions		
	Lateral Capacity (kips)	Maximum Moment (ft-kips)	Depth to Maximum Moment (feet)	Lateral Capacity (kips)	Maximum Moment (ft-kips)	Depth to Maximum Moment (feet)
TDP	24	84	6.0	54	215	0
16” ACIP	25	83	5.5	57	203	0

Research has shown that developing full fixity at the top of the pile is very difficult. Therefore, we believe it would be more reasonable to assume partial fixity. For partial fixity, the lateral load and maximum moment can be interpolated linearly between the values for free- and fixed-head conditions. To develop 50-percent fixity, we recommend the piles be embedded at least 12 inches into the pile cap or grade beam.

The lateral capacities presented in Table 3 above are for single piles only. For pile groups where the center-to-center spacing is three diameters in the direction of loading, the single-pile lateral capacities should be reduced using the appropriate reduction factors in Table 4.

TABLE 4
Pile Group Reduction Factors for Lateral Loading

Number of Piles in Pile Group	Center-to-Center Spacing Between Piles (inches)	Reduction Factor
2	3 pile diameters	0.90
4	3 pile diameters	0.75
6	3 pile diameters	0.65
9+	3 pile diameters	0.65

Where piles have center-to-center spacing of at least six pile diameters in the direction of loading, no group reduction factors need to be applied. Reduction for other pile group spacing can be provided once the number and arrangement of piles are known. Reduction factors apply only to lateral capacity; the bending moments should not be reduced.

Additional lateral load resistance can be obtained from passive resistance acting against the faces of pile caps and grade beams. For calculating passive resistance to dynamic (transient) loads, we recommend using a uniform pressure of 1,500 psf. For calculating passive resistance to sustained loads, we recommend using an equivalent fluid weight (triangular distribution) of 270 pcf. These values include a factor of safety of at least 1.5. The upper 12 inches should be ignored unless the ground surface is confined by a slab or pavement.

Indicator Piles

We recommend an indicator pile program be performed to provide data for production pile installation. To evaluate the potential for variations throughout the site and to evaluate whether predrilling will be required for piles, we recommend at least 12 indicator piles be installed prior to production piles. The indicator piles should be installed with the same equipment that will be used to install the production piles. The indicator piles may be used as production piles provided the recommended length is achieved. If the indicator torque down piles will be removed after installation, they should be located at least six pile diameters from the location of production piles and should the resulting holes should be filled with grout or concrete immediately after the piles are removed.

Load Testing

In addition, we recommend pile load tests of the TDPs or ACIP piles be performed to confirm the axial compressive and tensile pile capacities. For TDPs and ACIP piles, we recommend a minimum of two compressive and one uplift load tests be performed. The test piles should be selected by the Geotechnical Engineer and approved by the Structural Engineer. The load tests should be performed in accordance with ASTM D1143 (Standard Test Methods for Deep Foundations Under Static Axial Compressive Load) and ASTM D3689 (Standard Test Methods for Deep Foundations Under Static Axial Tensile Load). Equipment used for the test (load frame, jacks, ad reaction piles) should be capable of applying at least 2.5 times the allowable dead plus live design loads. The Davisson Method or 90% Criterion (Brinch-Hanson) Method should be used to interpret the ultimate capacities of the piles.

7.3 Floor Slab

If water vapor moving through the floor slab for the pile foundation option is considered detrimental, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

If required by the Structural Engineer, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. However, excess water trapped in the sand could eventually be transmitted as vapor through the slab. Therefore, if rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced. The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 5.

TABLE 5
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
¾ inch	30 – 100
½ inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slabs should have a low w/c ratio - less than 0.50. If the concrete is poured directly over the vapor retarder (no sand layer), we recommend the w/c ratio of the concrete not exceed 0.45 and water not be added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slabs should be properly cured. Before floor coverings, if any, are placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.4 Permanent Below-Grade Walls

Permanent below-grade walls, such as elevator pit walls, should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, and traffic loads (if vehicular traffic is expected within 10 feet of the wall). We recommend the permanent below-grade walls be designed for the more critical of the following criteria:

- at-rest equivalent fluid weight of 60 pcf, or
- active equivalent fluid weight of 40 pcf, plus a seismic equivalent fluid weight of 21 pcf.

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads from vehicles or adjacent building foundations. Where the below-grade wall is subject to vehicular loading within 10 feet of the wall, an additional uniform lateral pressure of 50 psf applied to the upper 10 feet of the wall.

The lateral earth pressures recommended are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the walls. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). The collector pipe should be sloped to drain to a sump or another suitable outlet.

To protect against moisture migration, below-grade walls should be waterproofed and water stops should be placed at all construction joints. If backfill is required behind retaining walls, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.5 Temporary Cut Slopes

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. We judge that temporary cuts in on-site soil which are less than eight feet high, and inclined in accordance to OSHA guidelines for Type A soil will be stable provided that they are not surcharged by equipment or building material. Temporary shoring will be required where temporary slopes are not possible because of space constraints. If shoring will be required, we can provide recommendations for shoring design upon request.

7.6 Ground Improvement

We conclude viable ground improvement systems include DDSC or SMX columns. Ground improvement systems are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the ground improvement elements should be determined by the contractor based on the proposed structural loads and the desired level of improvement. For planning purposes, we recommend the ground improvement elements extend to at least 40 feet below the bottom of the mat foundation. The length and spacing of the DDSC or SMX columns should be sufficient to limit total and differential static settlement to less than 4 inches and 3/4 inches across a horizontal distance of 30 feet and liquefaction-induced total settlement to less than 1/4 inch; these settlement requirements should be confirmed by the Structural Engineer prior to bidding.

Our geotechnical report should be provided to potential design-build ground improvement contractors and we should be retained to provide technical input and review the geotechnical aspects of their final design prior to construction.

7.7 Seismic Design

We understand the proposed building will be designed using the seismic provisions in the 2016 CBC. The latitude and longitude of the site are 37.3243° and -122.9039°, respectively. Although the 2016 CBC calls for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude the potentially liquefiable soil layer beneath the site is relatively thin with a moderately high relative density, such that the potentially liquefiable soil layer will not incur significant nonlinear behavior during strong ground shaking. Therefore, for seismic design we recommend Site Class D be used. For seismic design in accordance with the 2016 CBC, we recommend the following for a Site Class D:

- S_S = 1.50g, S_1 = 0.60g
- S_{MS} = 1.50g, S_{M1} = 0.90g
- S_{DS} = 1.00g, S_{D1} = 0.60g
- Seismic Design Category D for Risk Categories I, II, and III.

8.0 FUTURE GEOTECHNICAL STUDY AND LIMITATIONS

The previous investigation performed by SFB was for a significantly lighter, low-rise structure and, therefore, the borings and CPTs did not extend below a depth of 50 feet. Considering the large footprint and weight of the proposed structure, the settlement of the soil below a depth of 50 feet needs to be considered in the overall settlement analysis. For the purposes of this report, we assumed the stiff to very stiff clay encountered between depths of 40 to 50 feet bgs extends down to a depth of 150 feet; however, this assumption needs to be confirmed with deep borings and CPTs.

The preliminary conclusions and recommendations presented within the report should be confirmed with additional borings and CPTs and are not intended for final design. Prior to final design, we should be retained to provide a final geotechnical report based on a supplemental field investigation. Once our final report has been completed, the design team has selected a foundation system, and prior to construction, we should review the project plans and specifications to check their conformance with the intent of our final recommendations. During construction, we should observe site preparation, foundation installation, ground improvement, and the placement and compaction of fill. These observations will allow us to compare the actual with the anticipated soil conditions and to check if the contractor's work conforms with the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface soil and groundwater conditions do not deviate appreciably from those disclosed in the exploratory borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

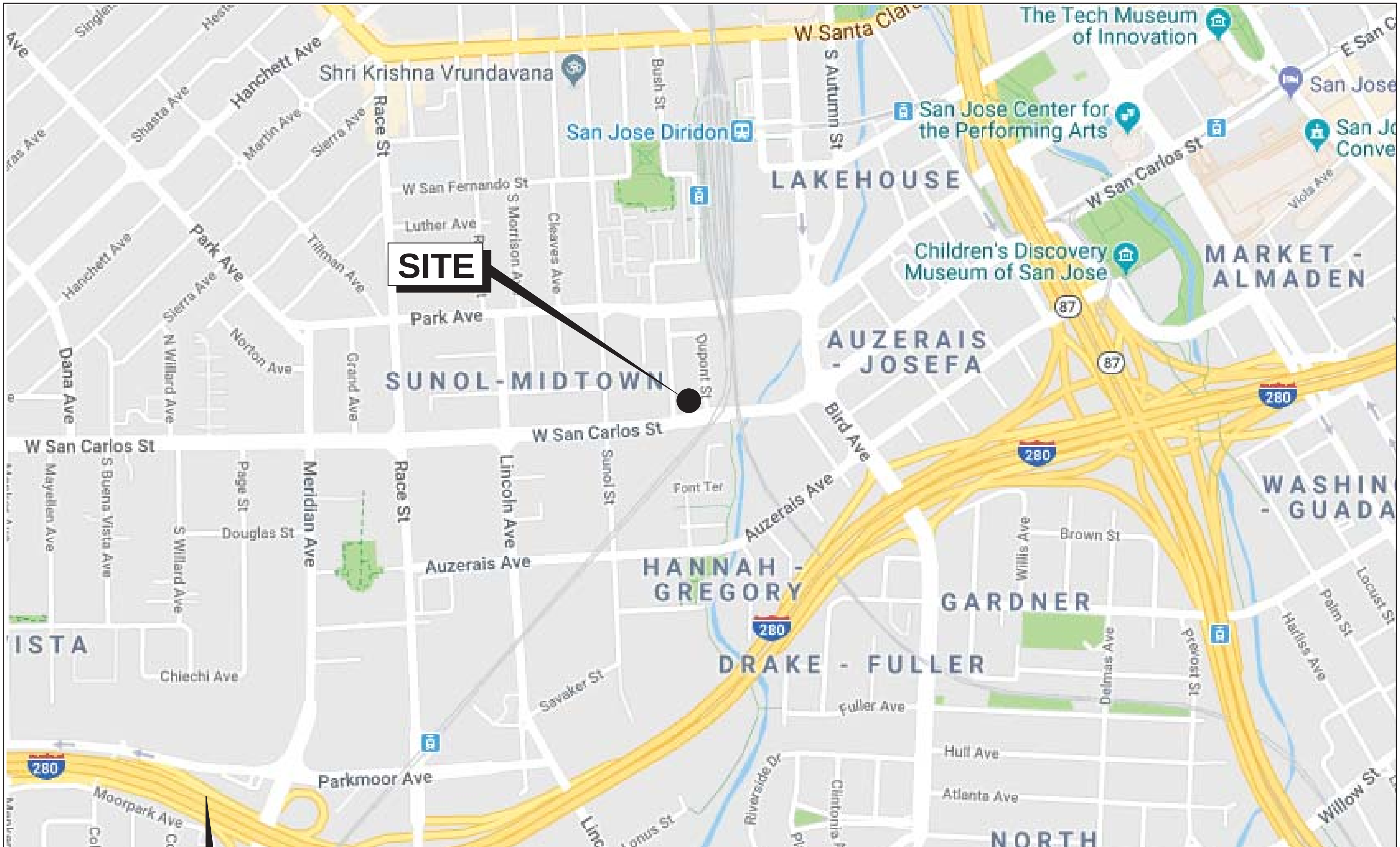
ROCKRIDGE GEOTECHNICAL

FIGURES

APPENDIX A
Logs of Borings and Cone Penetration Test Results by SFB

ROCKRIDGE GEOTECHNICAL

APPENDIX B
Laboratory Test Results by SFB



Base map: Google Map, 2017

0 1,000 2,000 Feet

Approximate scale

280 McEVOY STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 04/24/18	Project No. 18-1465	Figure 1

Base map: Google Earth Pro, 2017.

EXPLANATION

CPT-4 — Approximate location of cone penetration test by Stevens, Ferrone & Bailey, September 16, 2015

SFB-1 — Approximate location of boring by Stevens, Ferrone & Bailey, September 16, 2015

Project limits

0 60 Feet

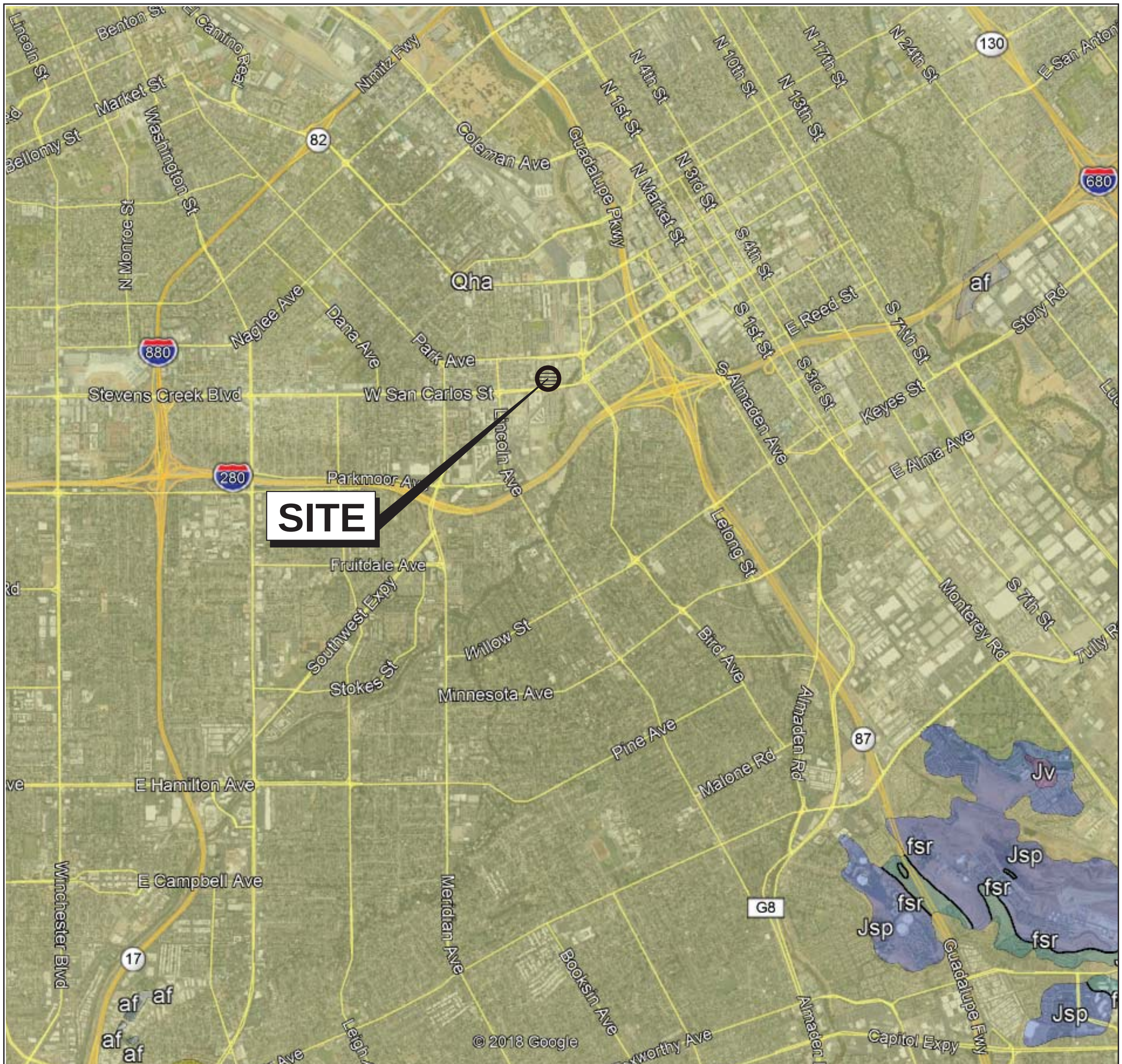
Approximate scale

280 McEVOY STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

SITE PLAN

Date 04/24/18 | Project No. 18-1465 | Figure 2



Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2017.

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
fsr	Franciscan Complex melange (Eocene, Paleocene, and (or) Late Cretaceous)
Jv	Great Valley complex volcanic rocks (Jurassic)
Jsp	Great Valley complex serpentinite (Jurassic)

Geologic contact:
dashed where approximate and dotted where concealed, queried where uncertin

0 4000 8000 Feet

Approximate scale

280 McEVOY STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 04/24/18	Project No. 18-1465	Figure 3

Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2008.

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

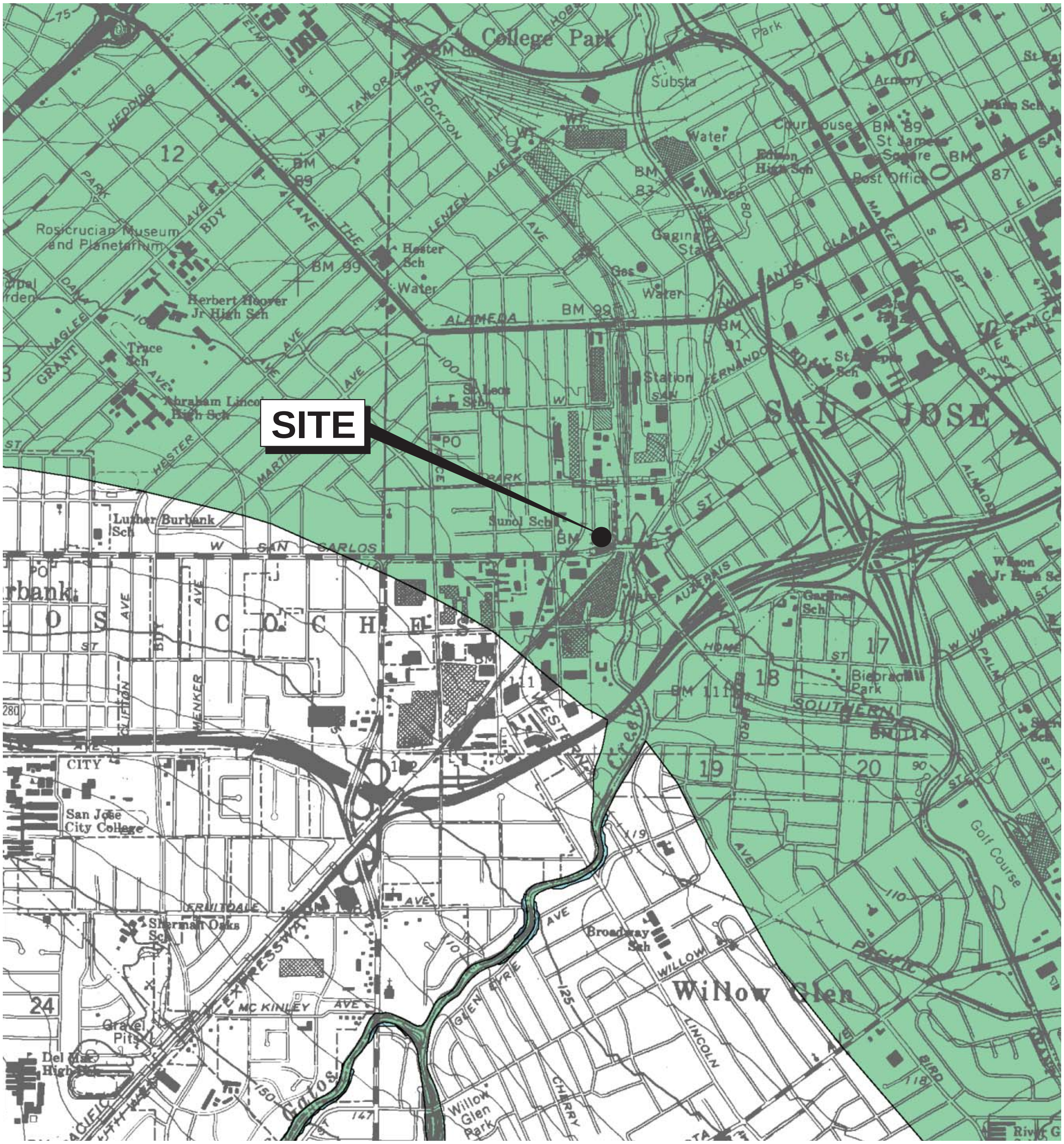
Approximate scale

280 McEVOY STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 04/24/18 | Project No. 18-1465 | Figure 4



EXPLANATION

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

Reference:
State of California "Seismic Hazard Zones"
San Jose West Quadrangle.
Released on February 7, 2002

0 2000 4000 Feet

Approximate scale

280 McEVOY STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

Date 04/24/18	Project No. 18-1465	Figure 5

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		grf	ltr	Description	Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines	Soils	Silts And Clays LL < 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			GM	Silty gravels, gravel-sand-silt mixtures				OL	Organic silts and organic silt-clays of low plasticity
			GC	Clayey gravels, gravel-sand-clay mixtures		Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines				CH	Inorganic clays of high plasticity, fat clays
			SP	Poorly-graded sands or gravelly sands, little or no fines				OH	Organic clays of medium to high plasticity
			SM	Silty sands, sand-silt mixtures	Highly Organic Soils			PT	Peat and other highly organic soils
			SC	Clayey sands, and-clay mixtures					

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200 — 40 — 10 — 4 — 3/4" — 3" — 12"

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.
**Unconfined compressive strength.

SYMBOLS & NOTES

- Standard Penetration sampler (2" OD Split Barrel)
- Modified California sampler (3" OD Split Barrel)
- California Sampler (2.5" OD Split Barrel)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Ground Water level initially encountered
- Ground Water level at end of drilling

PI = Plasticity Index
LL = Liquid Limit
R = R-Value

Increasing Visual Moisture Content

Saturated
Wet
Moist
Damp
Dry

Constituent Percentage

trace	<5%
some	5-15%
with	16-30%
-y	31-49%

Stevens, Ferrone & Bailey Engineering Company, Inc.

1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

KEY TO EXPLORATORY BORING LOGS

WEST SAN CARLOS STREET
San Jose, CA

PROJECT NO.	DATE	FIGURE NO.
648-6	October 2015	A-1

KEY 648-6.GPJ STEVENS FERRONE BAILEY.GDT 10/2/15

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION 101 feet	LOGGED BY TC
DEPTH TO GROUND WATER 35 feet	BORING DIAMETER 4-inch	DATE DRILLED 09/16/15

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
FILL: CLAY (CL), mottled gray yellowish brown, silty, sandy(fine- to coarse-grained), some gravel(fine to coarse, angular to subrounded), dry to damp.	stiff hard		0	100		39	9	115		
FILL: CLAY (CL), mottled olive dark gray, silty, with sand(fine- to medium-grained), dry to damp.	hard					45				
			5	95		36	17	110	12.6	At 6': Liquid Limit = 44 Plasticity Index = 28 Coarse Sand = 1% Medium Sand = 6% Fine sand = 15% Silt = 29% Clay = 49%
Some gravel(fine, subangular to subrounded).										
SAND (SM), yellowish brown, fine- to medium-grained, some coarse-grained, some gravel(fine to coarse, subangular to rounded), with silt, dry.	medium dense									
			10	90		21	7	101		At 11': Coarse Gravel = 3% Fine Gravel = 6% Coarse Sand = 7% Medium Sand = 11% Fine Sand = 48% Silt & Clay = 25%
Gravels up to 2" diameter at 12.5'.										
Hole caved, gravel up to 3" diameter.			15	85						
Hole caved to 13'.			20	80						
CLAY(CL)/SILT(ML) bluish gray, silty, with sand(fine-grained), damp to moist. Interbedded with thin sand lenses(fine- to coarse-grained), with silt, some gravel(fine, subangular to subrounded), moist.	stiff		25	75		23				
			30	70						
SAND (SM), bluish gray, fine- to medium-grained, silty, moist.	medium dense									

Stevens, Ferrone & Bailey Engineering Company, Inc.
1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

EXPLORATORY BORING LOG

WEST SAN CARLOS STREET
San Jose, CA

PROJECT NO.	DATE	BORING NO.
648-6	**October 2015**	**SFB-1**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION 101 feet	LOGGED BY TC
DEPTH TO GROUND WATER 35 feet	BORING DIAMETER 4-inch	DATE DRILLED 09/16/15

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
SAND (SM), continued.	medium dense		35	65		18				Passing #200 Sieve = 31%
CLAY (CL), bluish gray, silty, some sand(fine-grained), damp.	very stiff									
Bottom of Boring = 36.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			40	60						
			45	55						
			50	50						
			55	45						
			60	40						
			65	35						

Stevens, Ferrone & Bailey Engineering Company, Inc.
1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

EXPLORATORY BORING LOG

WEST SAN CARLOS STREET
San Jose, CA

PROJECT NO.	DATE	BORING NO.
648-6	**October 2015**	**SFB-1**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION 101 feet	LOGGED BY TC
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4-inch	DATE DRILLED 09/16/15

DESCRIPTION AND REMARKS	CONSIST	DEPTH (FEET)	ELEVATION	SAMPLER N-VALUE (SPT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
Asphalt Concrete (AC) 3.5" thick. Aggregate Base (AB).		0	101					
FILL: CLAY (CL), mottled gray brown, silty, sandy(fine- to medium-grained), with gravel(fine to coarse, subangular to subrounded), dry to damp. Pieces of glass at 3'.	very stiff		100	16 30				
CLAY (CH), mottled olive dark gray, silty, some sand(fine- to medium-grained), dry to damp.	very stiff	5	95	33				
SAND (SP-SM), mottled gray brown, fine- to medium-grained, some coarse grained, some gravel(fine to coarse, subangular to rounded), some silt, dry. Gravels up to 2" diameter.	medium dense	10	90	18				At 11': Fine Gravel = 17% Coarse Sand = 10% Medium Sand = 20% Fine Sand = 45% Silt & Clay = 8%
SAND (SW-SM), mottled gray yellowish brown, fine- to coarse-grained, gravelly(fine to coarse, angular to rounded), some silt and clay, dry to damp. Gravels up to 3" diameter.	very dense dense	15 20	85 80	40 30				At 16': Coarse Gravel = 8% Fine Gravel = 35% Coarse Sand =15% Medium Sand = 20% Fine Sand = 14% Silt & Clay = 8%
Bottom of Boring = 21.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.		25 30	75 70					

Stevens, Ferrone & Bailey Engineering Company, Inc.
1600 Willow Pass Court
Concord, CA 94523
Tel: 925-688-1001

EXPLORATORY BORING LOG

WEST SAN CARLOS STREET
San Jose, CA

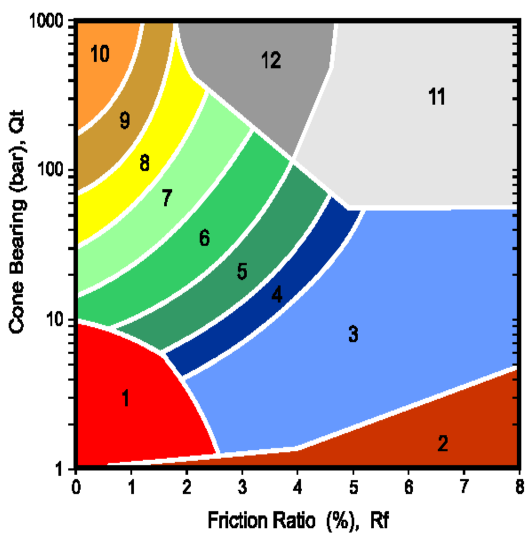
PROJECT NO.	DATE	BORING NO.
648-6	**October 2015**	**SFB-4**

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

GREGG

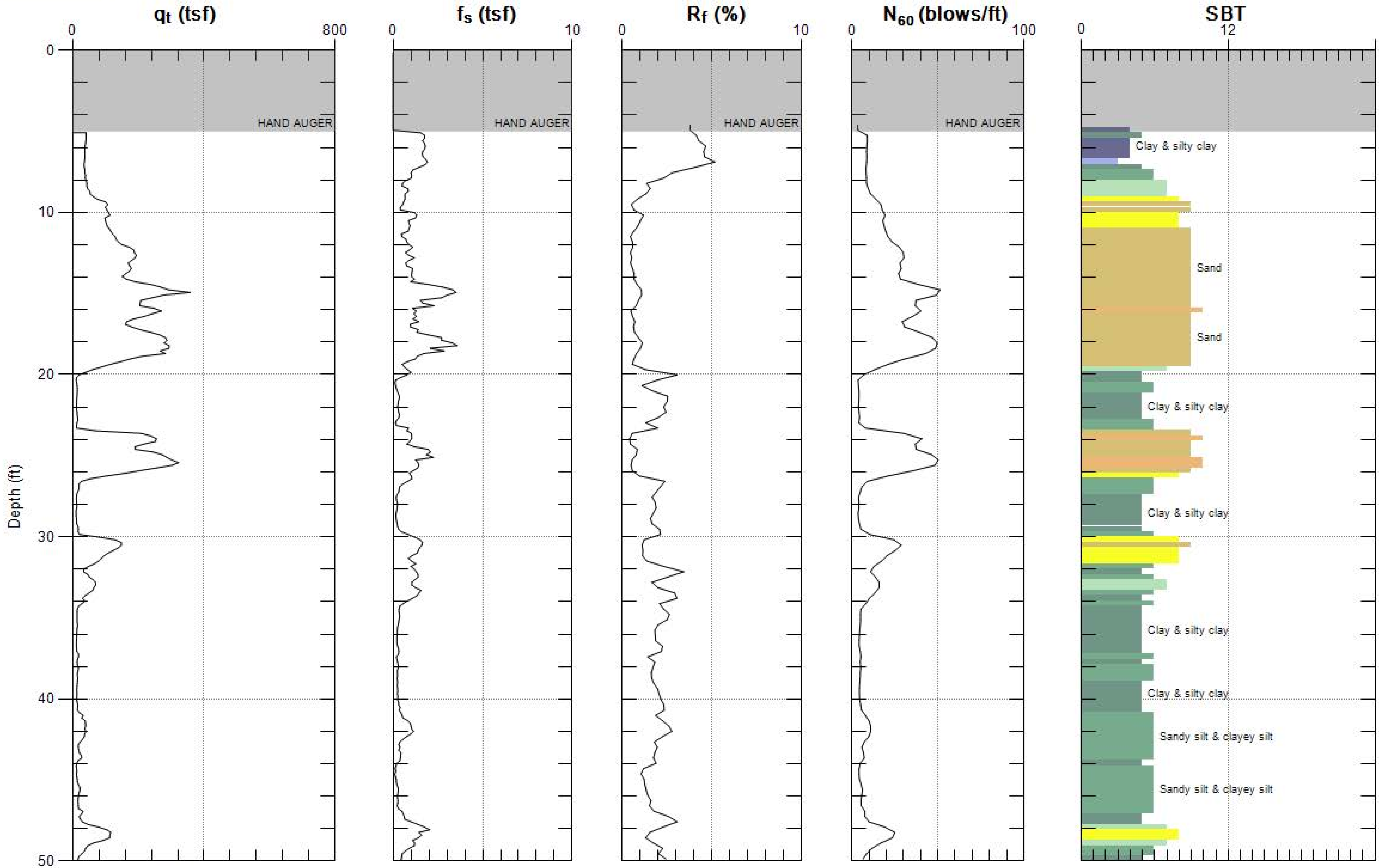
SF&B

Site: W. SAN CARLOS ST.

Sounding: CPT-4

Engineer: T.CHEN

Date: 9/16/2015 01:02



Max. Depth: 50.033 (ft)

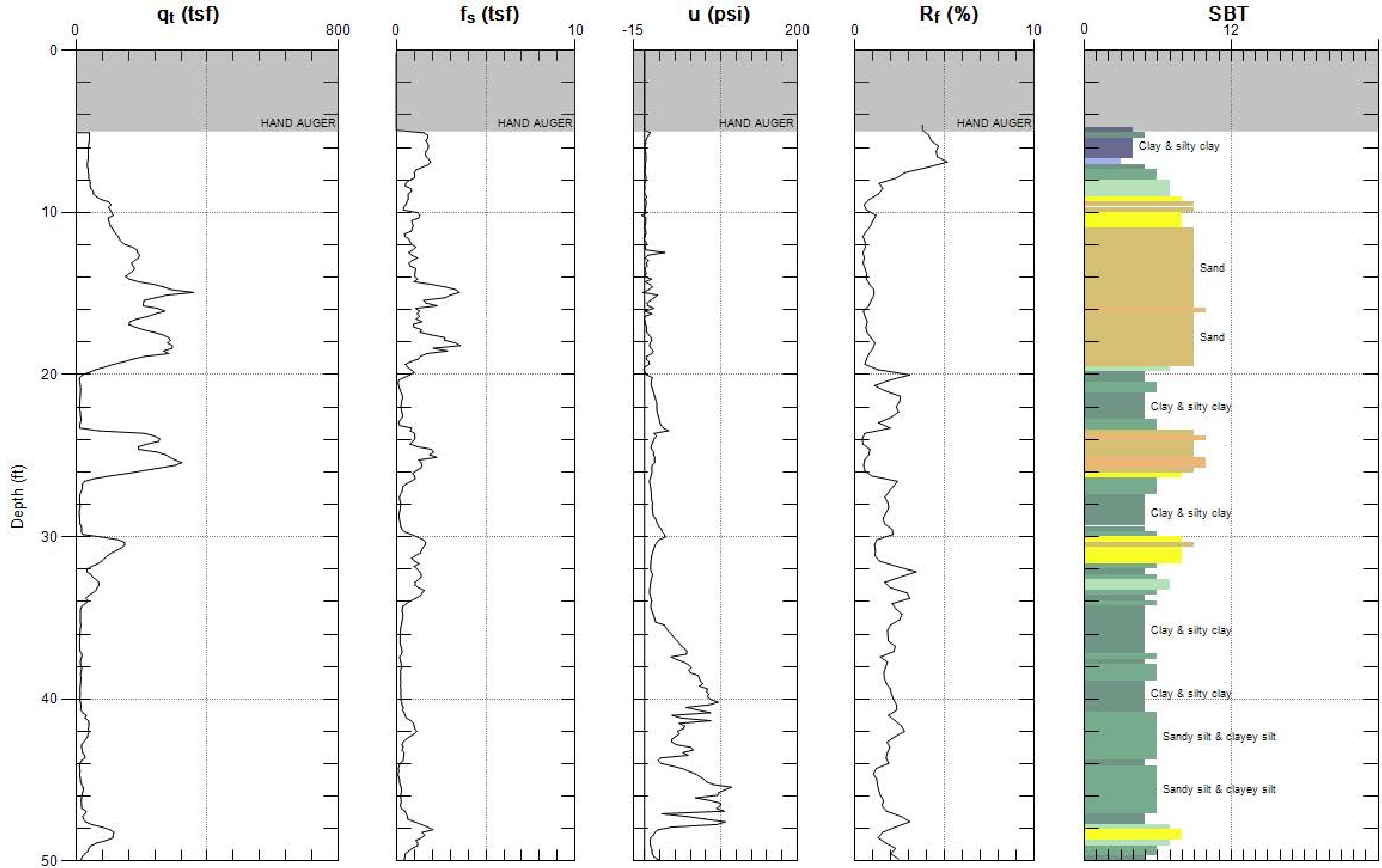
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

SF&B

Site: W. SAN CARLOS ST.
Sounding: CPT-4

Engineer: T.CHEN
Date: 9/16/2015 01:02



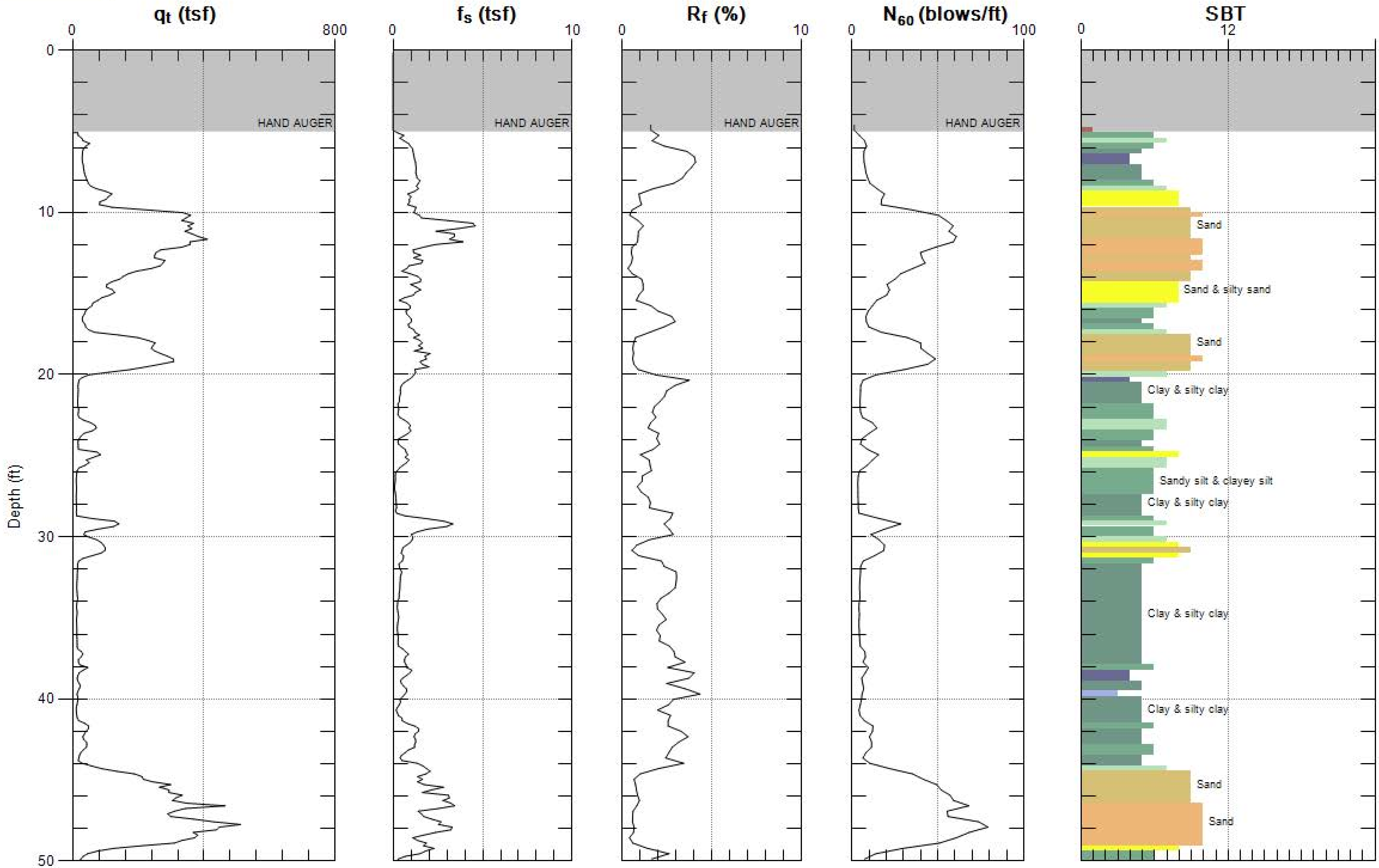
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG SF&B

Site: W. SAN CARLOS ST. Engineer: T.CHEN

Sounding: CPT-5 Date: 9/16/2015 02:00



Max. Depth: 50.033 (ft)

Avg. Interval: 0.328 (ft)

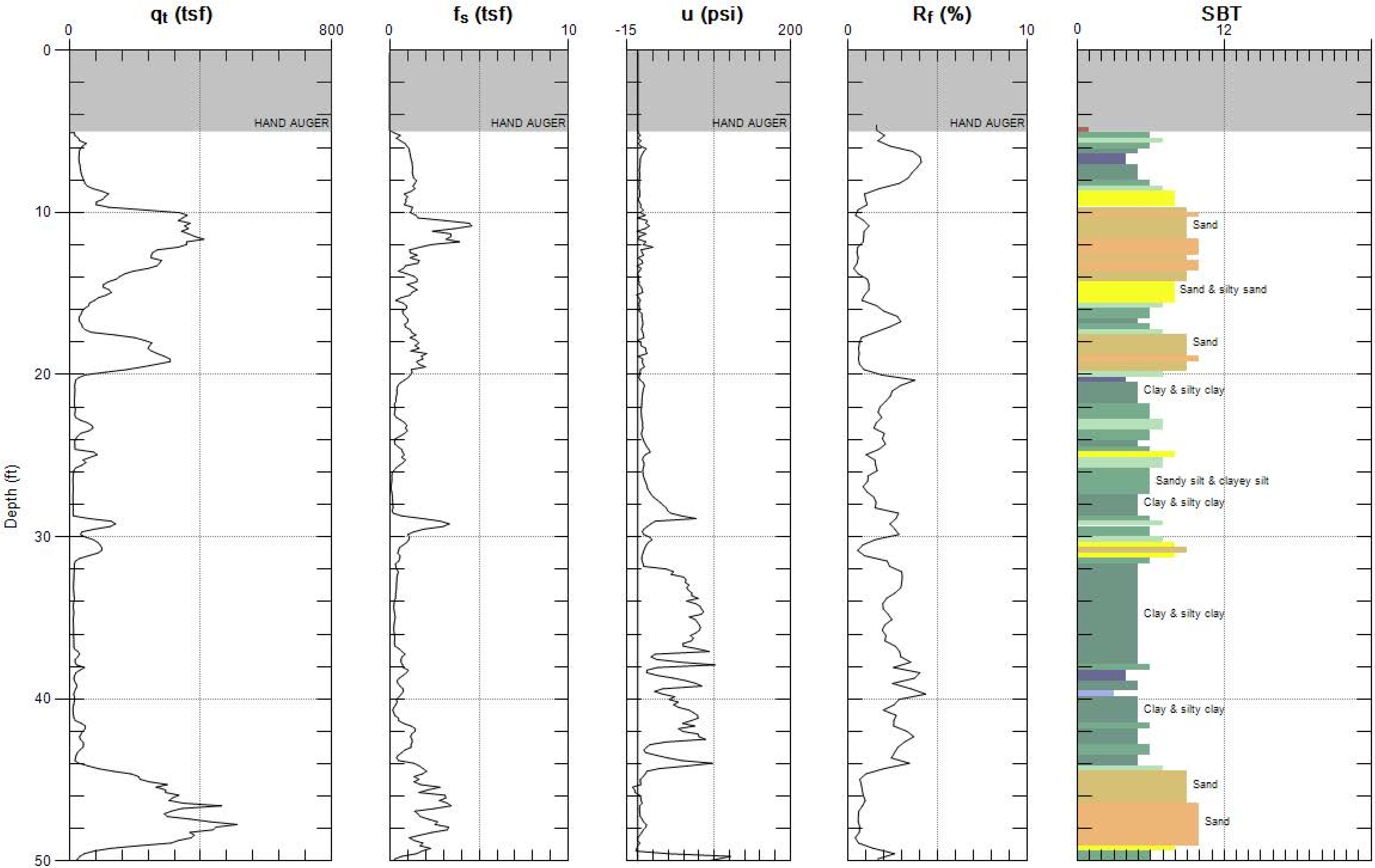
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SF&B

Site: W. SAN CARLOS ST.
Sounding: CPT-5

Engineer: T.CHEN
Date: 9/16/2015 02:00



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

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: Stevens, Ferrone & Bailey
Client's Project No.: 648-6
Client's Project Name: West San Carlos Street, San Jose
Date Sampled: 16-Sep-15
Date Received: 17-Sep-15
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 24-Sep-2015

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1509149-001	SFB-3 @ 6'	390	7.45	-	1,000	N.D.	N.D.	84
1509149-002	SFB-4 @ 3.5'	360	7.83	-	420	N.D.	22	140

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	21-Sep-2015	21-Sep-2015	-	18-Sep-2015	24-Sep-2015	21-Sep-2015	21-Sep-2015

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen

Cheryl McMillen
Laboratory Director

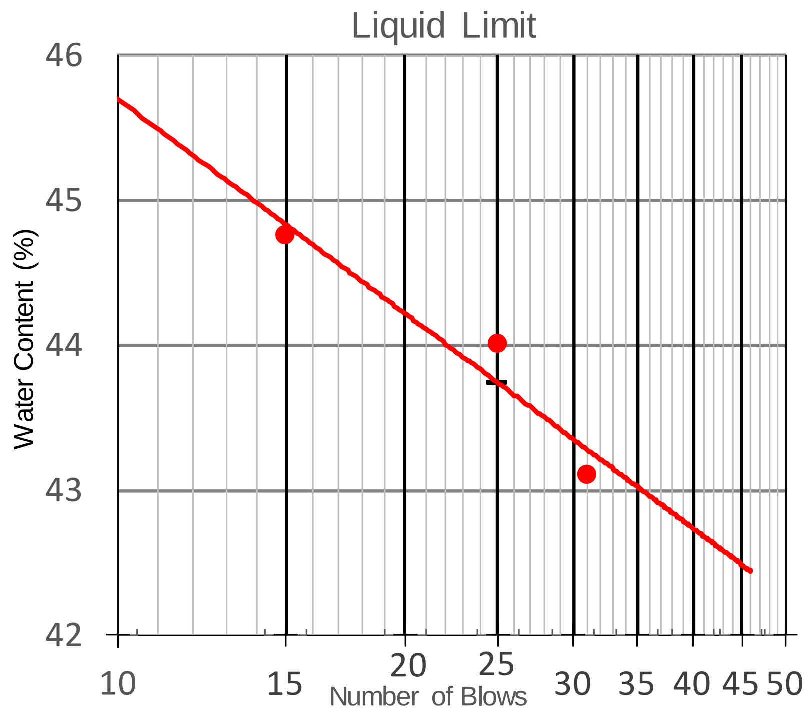
<u>Quality Control Summary</u> - All laboratory quality control parameters were found to be within established limits

Atterberg Limits Test – ASTM D4318

Project Number: 648-6 **Project Name:** West San Carlos

Boring/Sample Number: SFB-1 **Depth:** 6 **Date:** 09-24-15

Description of Sample: Dark brown silty CLAY with sand (CL) **Tested By:** R

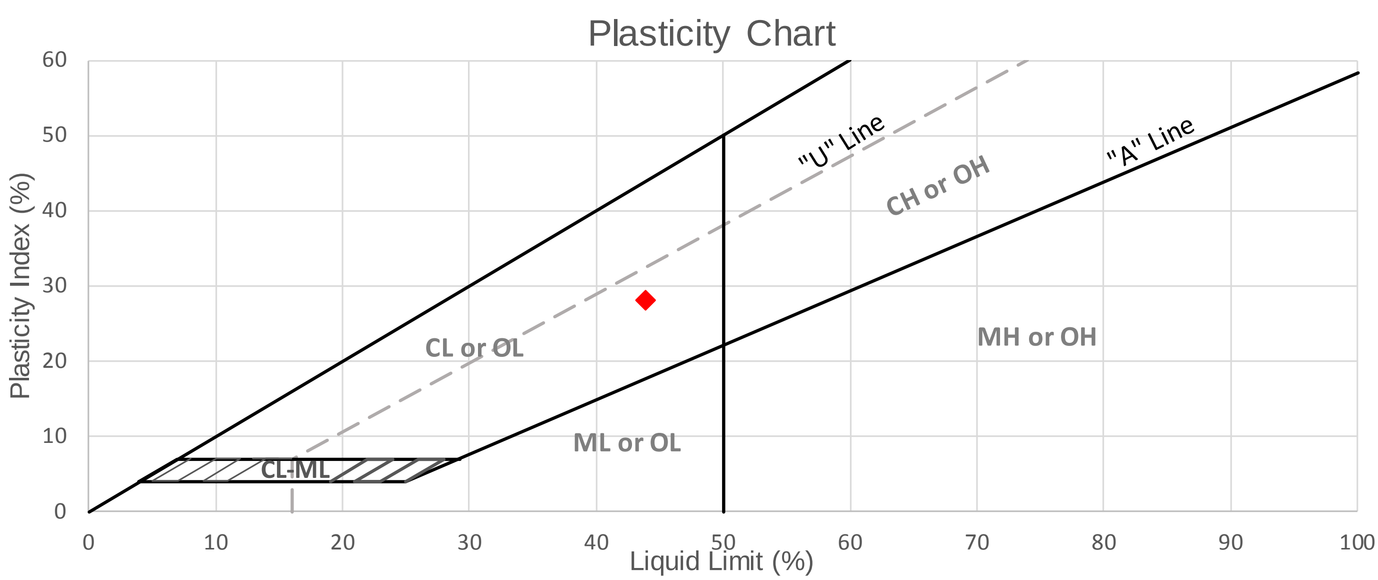


Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	15.8	16.1	16

Data Summary

Liquid Limit	**44**
Plastic Limit	**16**
Plasticity Index	**28**
Natural Water Content	17.2
Liquidity Index	0.041
% Passing #200	**78.4**



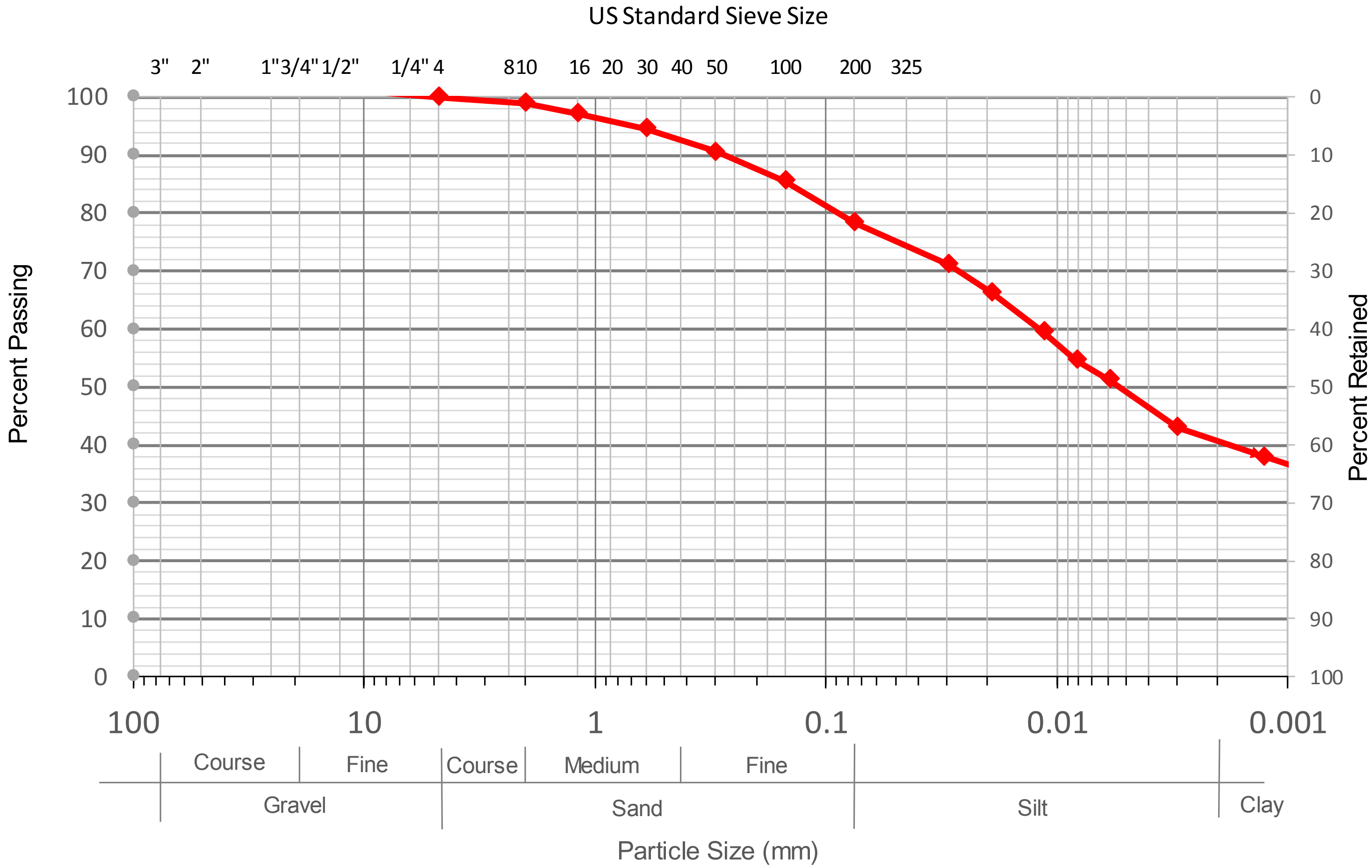
Stevens Ferrone & Bailey Engineering Company, Inc

Hydrometer Analysis – ASTM D422

Project Number: 648-6 **Project Name:** West San Carlos Street

Sample Number: SFB-1 **Description:** Dark brown silty CLAY with sand (CL)

Depth: 6 **Test Date:** 09-22-15 **Tested By:** R



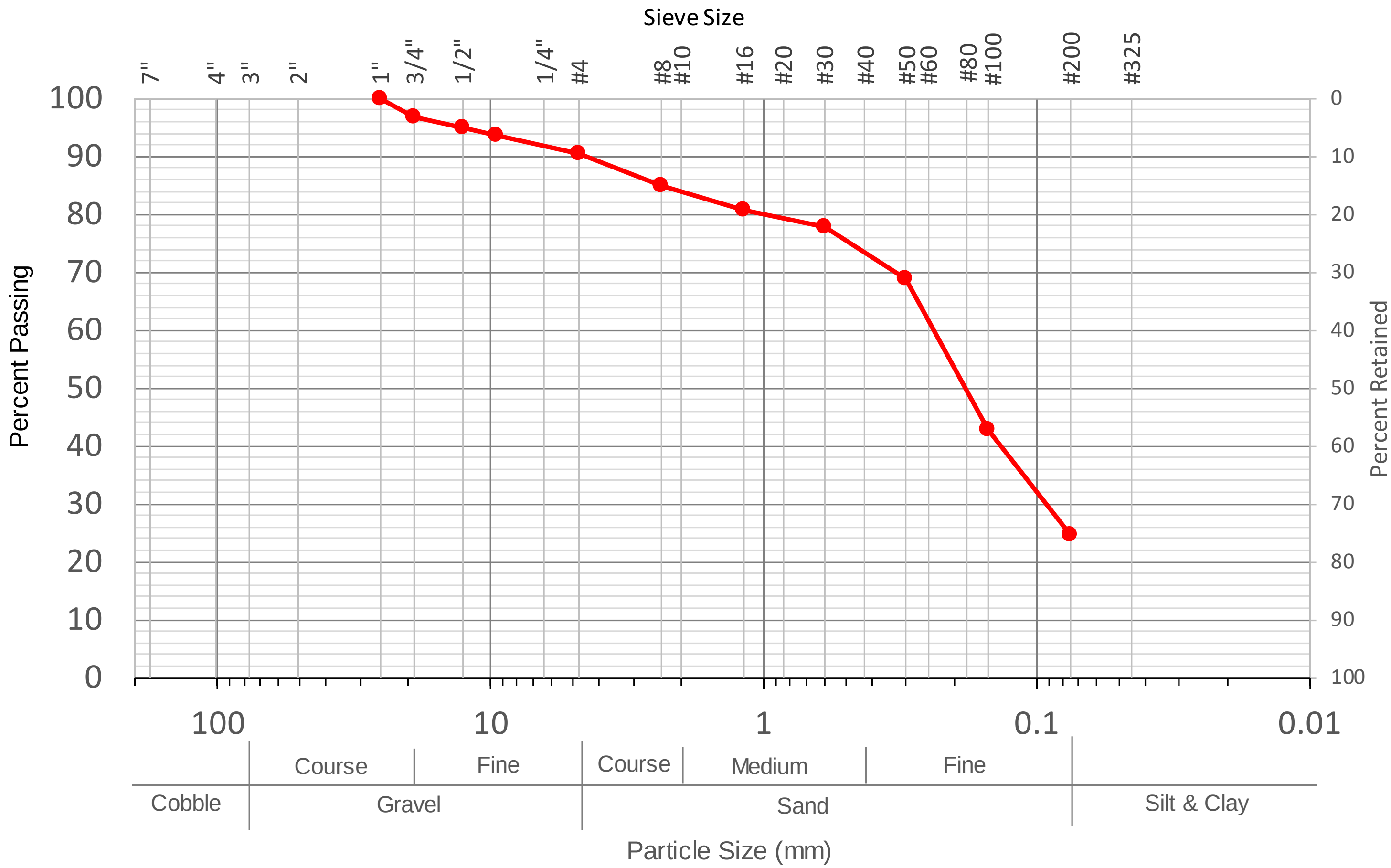
Composite Sieve Data

Standard Sieve Size	Percent Passing
3"	
1.5"	
3/4"	
3/8"	
#4	100.0
#10	99.0
#16	97.2
#30	94.6
#50	90.6
#100	85.7
#200	78.4

Particle Diameter (mm)	Percent Soil in Suspension
0.0293	71.2
0.0190	66.2
0.0114	59.6
0.0082	54.6
0.0059	51.3
0.0030	43.0
0.0013	38.1

Sieve Analysis – ASTM C136

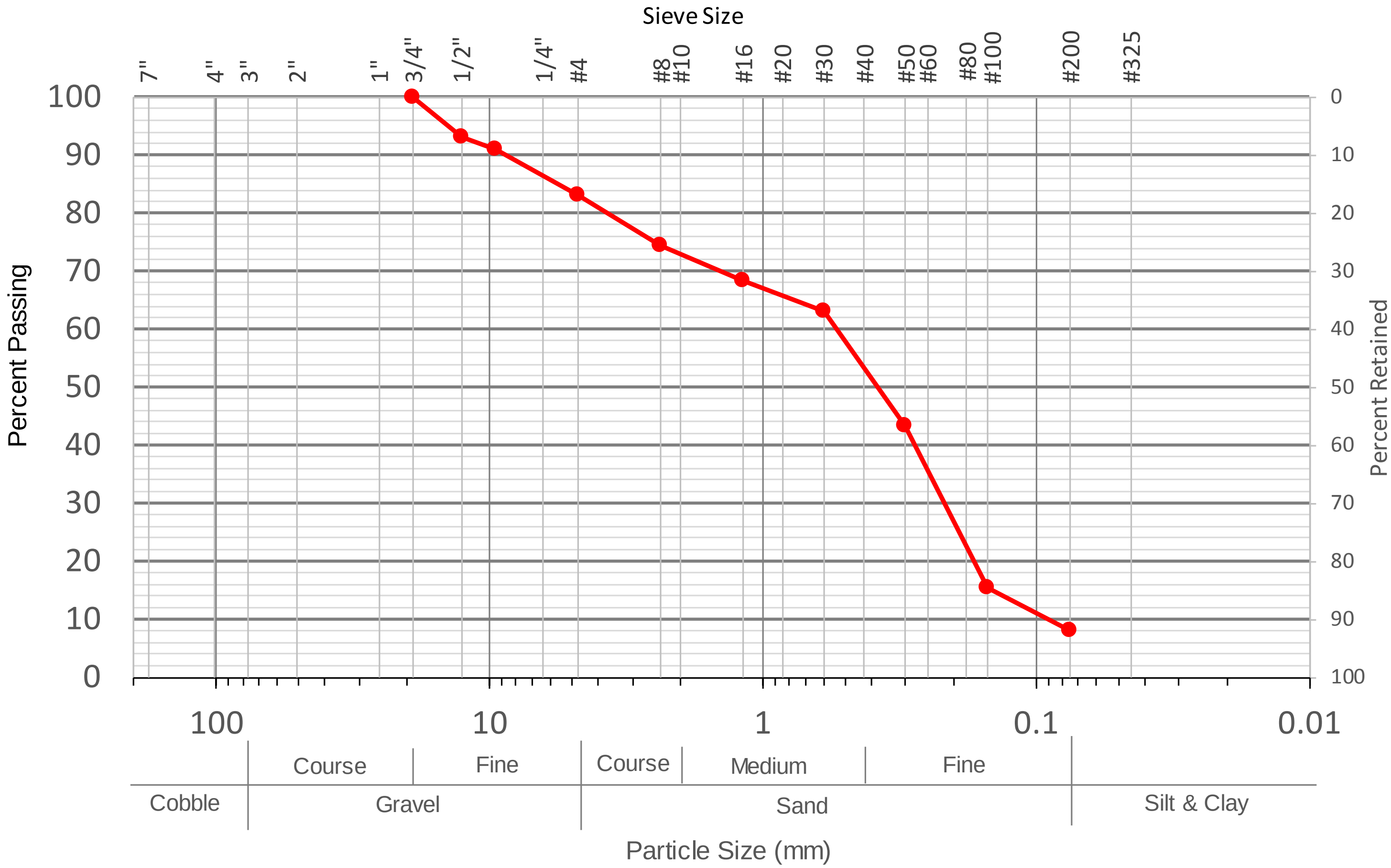
Project Number:	648-6	**Project Name:**	West San Carlos Street	**Sampling Date:**	9/16/2015
Sample Number:	SFB-1 @ 11	**Description:**	Light brown fine SAND with silt some gravel (SM)	**Test Date:**	9/23/2015
Sampled By:	TC	**Source:**	Onsite	**Tested By:**	R



Composite Sieve Data		
Standard Sieve Size	Percent Passing	Specs
3”		
2.5”		
2”		
1.5”		
1”	100.0	
3/4”	96.9	
1/2”	94.8	
3/8”	93.7	
#4	90.5	
#8	85.0	
#16	80.7	
#30	77.7	
#50	68.9	
#100	42.7	
#200	24.8	

Sieve Analysis – ASTM C136

Project Number:	648-6	**Project Name:**	West San Carlos Street	**Sampling Date:**	9/16/2015
Sample Number:	SFB-4 @ 11	**Description:**	Brown fine SAND with gravel some silt (SM)	**Test Date:**	9/23/2015
Sampled By:	TC	**Source:**	Onsite	**Tested By:**	R



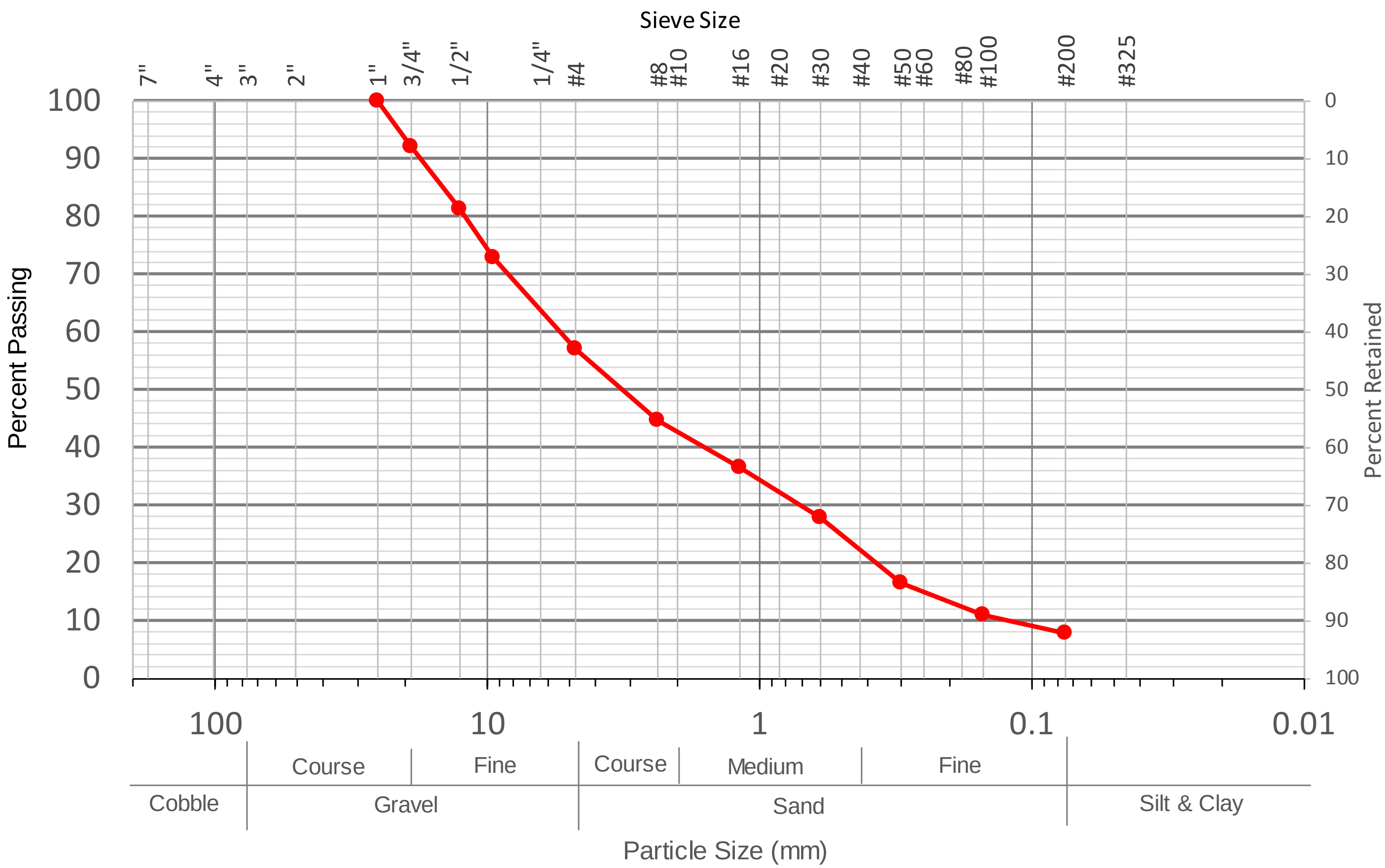
Composite Sieve Data		
Standard Sieve Size	Percent Passing	Specs
3”		
2.5”		
2”		
1.5”		
1”		
3/4”	100.0	
1/2”	93.2	
3/8”	90.9	
#4	83.2	
#8	74.5	
#16	68.3	
#30	63.2	
#50	43.2	
#100	15.3	
#200	8.2	

Sieve Analysis – ASTM C136

Project Number: 648-6 **Project Name:** West San Carlos Street **Sampling Date:** 9/16/2015

Sample Number: SFB-4 @ 16 **Description:** Brown gravelly SAND some silt (SM) **Test Date:** 9/23/2015

Sampled By: TC **Source:** Onsite **Tested By:** R



Composite Sieve Data		
Standard Sieve Size	Percent Passing	Specs
3”		
2.5”		
2”		
1.5”		
1”	100.0	
3/4”	91.9	
1/2”	81.2	
3/8”	72.9	
#4	57.1	
#8	44.8	
#16	36.6	
#30	27.9	
#50	16.5	
#100	11.0	
#200	7.8	

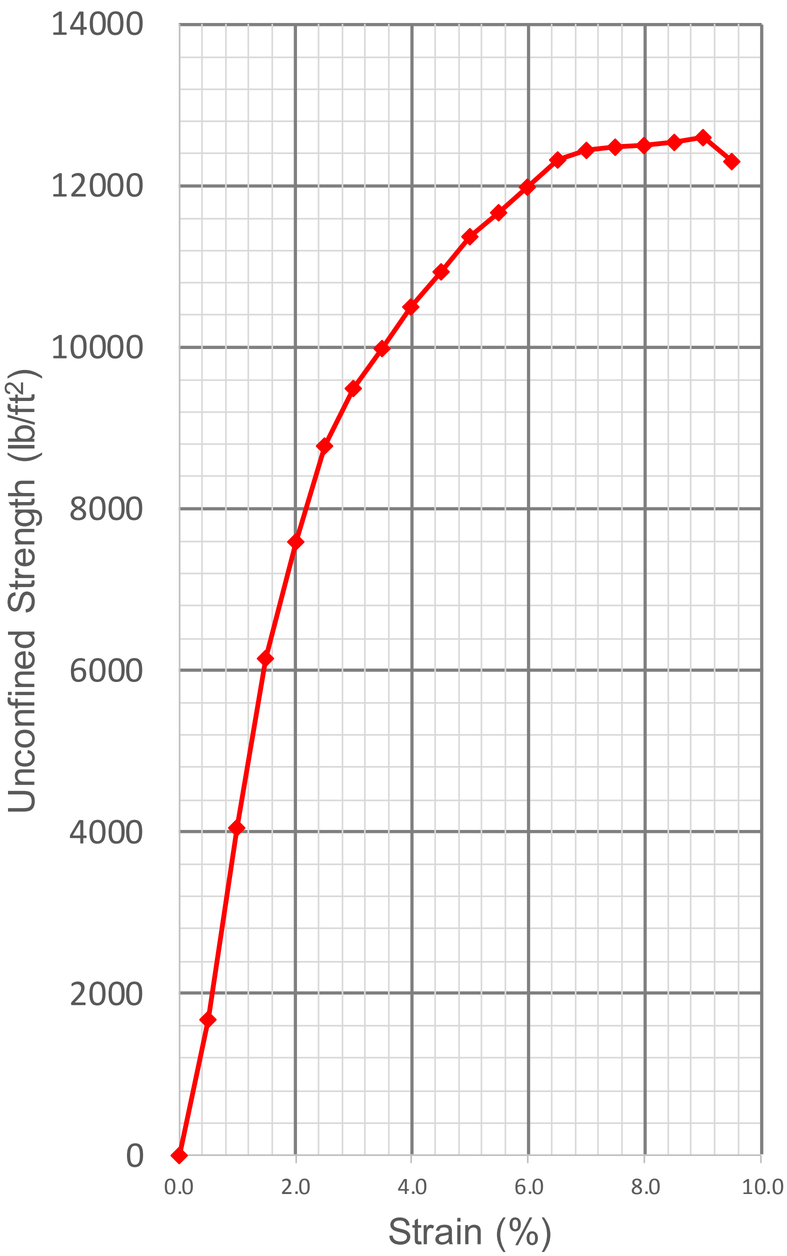
Stevens Ferrone & Bailey Engineering Company, Inc

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 648-6 **Boring #:** SFB-1 **Depth:** 6

Project Name: West San Carlos Street **Date:** 9/18/2015

Description: Dark brown silty CLAY some sand and gravel (CL) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5 in
Volume	0.01331 ft^3
Water Content	17.2
Wet Density	129.4 pcf
Dry Density	110.4 pcf

Max Unconfined Compressive Strength	
Elapsed Time	9 min
Vertical Dial	0.45 in
Strain	9.0 %
Area	0.03510 ft^2
Axial Load	442.6 lbs
Compressive Strength	12610 psf