

# **APPENDIX E**

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Geotechnical Report



## GEOTECHNICAL EXPLORATION

2500 & 2508 J STREET YAMANEE LOFTS  
SACRAMENTO, CALIFORNIA

The logo for ENGEEO is rendered in large, white, 3D block letters. The letters are set against a background of a green, rolling hillside under a blue sky. The 'E' and 'O' are particularly prominent. The logo is positioned in the center of the page, overlapping a blue horizontal band.

# ENGEEO

*Expect Excellence*

**Submitted to:**

Mr. Ryan Heater  
2500 J Street Owners, LLC  
3619 Winding Creek Road  
Sacramento, CA 95864

**Prepared by:**

ENGEEO Incorporated

November 13, 2015

**Project No:**

12487.000.000

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Ryan Heater  
2500 J Street Owners, LLC  
3619 Winding Creek Road  
Sacramento, CA 95864

Subject: Yamanee Lofts  
2500 and 2508 J Street  
Sacramento, California

## **GEOTECHNICAL REPORT**

Dear Mr. Heater:

ENGEO prepared this geotechnical report for the Yamanee Lofts project as outlined in our agreement dated July 15, 2015. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

We anticipate that additional consultation will be needed for refining the foundation design with your structural engineer. Once you have the structural engineer under contract, we would be glad to meet with them and discuss the findings in this report.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated



Nick Broussard, PE



Mark M. Gilbert, GE



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## **1.0 INTRODUCTION**

### **1.1 PURPOSE AND SCOPE**

ENGEO prepared this geotechnical report for design of the Yamane Lofts project located at 2500 and 2508 J Street in Sacramento, California. We prepared this report as outlined in our agreement dated July 15, 2015. The 2500 J Street Owners LLC authorized ENGEO to conduct the following scope of services:

- Service plan development
- Subsurface field exploration
- Soil laboratory testing
- Data analysis and conclusions
- Report and supplemental recommendation preparation

For our use we received the following:

1. CMS Architecture + Design; Floor Plans, Yamane Lofts, Sheets A0, A1, A2, A3, A10, A11 and A12; dated June 25<sup>th</sup>, 2015.
2. AEI Consultants; Phase I Environmental Site Assessment, 2500 J Street, Sacramento, California; Project number 329035, dated April 17, 2014.
3. AEI Consultants; Limited Phase II Subsurface Investigation, 2500-2504 J Street, Sacramento, California; Project number 329035; dated November 13, 2014.
4. RSC Engineering; Topographic Survey; dated July 2015.
5. Partner Engineering and Science Inc.; Phase I Environmental Site Assessment, Art Studio, 2508 J Street, Sacramento, California; dated December 17, 2013.

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

### **1.2 PROJECT LOCATION**

[Figure 1](#) displays a Site Vicinity Map. The site includes two separate parcels, the western parcel at 2500 J Street and the eastern parcel at 2508 J Street. Access is provided by 25<sup>th</sup> Street, Jazz Alley, and J Street.

[Figure 2](#) shows site boundaries and our exploratory locations. The site is bordered by 25<sup>th</sup> Street to the west, Jazz Alley to the south, J Street to the north, and existing buildings to the east.

### **1.3 PROJECT DESCRIPTION**

Based on our discussion with you and review of the information provided, we understand that site improvements will consist of:

1. A high-rise building covering nearly the entire site; we understand there will be 12 stories above grade with close to 2 levels below grade for parking. We anticipate the below-grade portion would extend about 15 to 20 feet below existing grade. The building construction type is unknown at this time.
2. Below-grade retaining walls.
3. Utilities and other infrastructure improvements.
4. Miscellaneous pavement and concrete flatwork.

The precise building perimeter, interior column spacings, and bay widths have not yet been developed. Additionally, structural column loads and foundation layout are under development.

## **2.0 FINDINGS**

### **2.1 SITE HISTORY**

According to the environmental site assessment documents referenced in Section 1.1, the 2500 J Street parcel has been developed since the early 1900's with residential dwellings followed by a dry cleaning facility up until 1971. The former dry cleaner, known as Red Feathers Cleaners, had a solvent tank and hazardous storage area in the southeast corner of the site. The site is now a current Leaking Underground Storage Tank site on the California Geotracker website, Case number SL0606789259. In 1971, the existing improvements, including the two-story building, were constructed on the 2500 J Street parcel.

The Phase 1 ESA for 2508 J Street indicates 2508 J Street was also originally developed in the early 1900's for residential and later commercial use. No documentation was provided to us indicating whether the former development on the site was demolished or removed completely.

### **2.2 REGIONAL AND LOCAL GEOLOGY**

The City of Sacramento is located in the Great Valley geomorphic province. The Great Valley is an elongate, northwest-trending structural trough bound by the Coast Range on the west and the Sierra Nevada on the east. The Great Valley has been and is presently being filled with sediments primarily derived from the Sierra Nevada. The impact of periodic glaciation of the Sierra Nevada during the last global climate change was strongly felt by the Sacramento Valley River systems. Huge quantities of sediments were moved through the river systems fed by alpine glaciers during the last period of glaciation. As this period of glaciation ended, rivers draining the Sierra Nevada were made even more powerful by the considerably wetter climate and

abundant meltwater. Abundant sediments left from the retreating glaciers were carried downstream into the Sacramento area by the American and Sacramento River watersheds. These sediments deposited in the last 11,000 years are mapped at the site as Holocene Alluvium (Helley, 1979). The regional geology is shown on [Figure 3](#), Regional Geologic Map.

Holocene Alluvium generally consists of young unweathered gravel, sand, and silt; these sediments can be loose or soft. To the east and south of the site, older Pleistocene Lower Riverbank Formation is mapped (Helley, 1979). Riverbank Formation consists of alluvial fans and terraces of sediments derived from the Sierra Nevada that generally slope down in elevation from the Sierra Nevada foothills and typically underlie the Holocene Alluvium in much of the Sacramento area. These older Riverbank alluvial sediments generally consist of dense to very dense sand or gravel and stiff to very stiff fine-grained soils; commonly these soils show advanced soil development and weathering as well as cemented layers.

### 2.3 Faulting and Seismicity

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

The site is located within a seismically active region, as California has numerous faults that are considered active. Generally, a fault is considered active if it has ruptured within the Holocene epoch (11,700 years before present). The following table summarizes the distances to known, mapped, active regional faults and estimated maximum magnitude within approximately 62 miles (100 kilometers) using the USGS Spatial Query tool. The query tool is based on the USGS 2008 National Seismic Hazard Maps that were used to develop the 2013 California Building Code seismic parameters. Refer to [Figure 4](#) for a Regional Faulting and Seismicity map that shows nearby USGS faults and historic earthquake magnitudes.

**TABLE 2.3-1**  
 Distances to Mapped 2008 USGS Regional Active Faults  
 (38.57453°, -121.47308°)

Fault	Distance (miles)	Maximum Moment Magnitude (Avg. of Hanks and Ellsworth)
Great Valley 4a, Trout Creek	28	6.5
Great Valley 4a, Gordon Valley	31	6.7
Great Valley 3, Mysterious Ridge	32	6.7
Great Valley 5, Pittsburg Kirby Hills	34	6.5
Hunting Creek-Berryessa	40	6.7
Green Valley Connected	40	6.6
West Napa	50	6.5
Greenville Connected	52	6.7
Great Valley 2	57	6.3
Mount Diablo Thrust	57	6.5

<b>Fault</b>	<b>Distance (miles)</b>	<b>Maximum Moment Magnitude (Avg. of Hanks and Ellsworth)</b>
Great Valley 7	58	6.6
Calaveras CN+CC+CS	60	6.8
Bartlett Springs	61	6.9
Hayward-Rodgers RC+HN+HS	62	7.2

Although the Foothill Fault System is not mapped in the USGS database, the Cleveland Hills Fault Segment (part of the Foothills Fault System) near Oroville lies approximately 59 miles from the site and produced a Magnitude 5.8 earthquake in 1975. Segments of the Foothills Fault System located as close as 30 miles from the site are not considered active, but could be capable of a large magnitude earthquake.

Historically, no significant damage in Sacramento has been caused by earthquakes; however, notable ground shaking has been felt in the past from distant events. These seismic events include the 1892 Vacaville-Winters Magnitude 6.4, the 1906 San Francisco Magnitude 7.8, and the 1989 Loma Prieta Magnitude 6.9 earthquakes.

According to the 2008 USGS Interactive Deaggregation tool for a site Class D using this shear wave velocity estimate, a mean magnitude earthquake (M<sub>w</sub>) of 6.6 is appropriate for analyzing liquefaction for the site. This is consistent with the magnitudes listed above for the nearby faults.

## **2.4 FIELD EXPLORATION**

Our field exploration included drilling one boring and advancing five Cone Penetration Test (CPT) soundings at various locations on the site and nearby with the public right-of-way. We performed our field exploration on October 19<sup>th</sup> and 20<sup>th</sup>, 2015. The location and elevations of our explorations are approximate and were estimated by pacing from features shown on the Site Plan, [Figure 2](#); they should be considered accurate only to the degree implied by the method used.

Previous explorations associated with phase II environmental assessments are also shown on the Site Plan. These explorations include a total of four soil borings in the parking lot of the 2500 J Street parcel.

### **2.4.1 Boring**

We observed drilling of one boring at the location shown on the Site Plan, [Figure 2](#). An ENGEO representative supervised the drilling and logged the subsurface conditions. We retained a truck-mounted CME75 drill rig and crew to advance the boring using a 5-inch-diameter solid-flight auger in the upper 25 feet and 4-inch-diameter mud rotary methods below 25 feet. The boring was advanced to an approximate depth of 101½ feet below existing grade. We permitted and backfilled the boring in accordance with Sacramento County requirements.

We retrieved soil samples at various depth intervals from the boring. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. In addition, soil samples were also obtained using a 3.0-inch O.D. Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows.



CME75 Drill Rig at 1-B1

We used the field logs to develop the report logs in [Appendix A](#). The logs depict subsurface conditions at the exploration location for the date of exploration; however, subsurface conditions may vary with time.

#### 2.4.2 Cone Penetration Tests

We retained a CPT rig to advance a cone penetrometer at five locations to depths between 22 and 75 feet. [1-CPT4](#) and [1-CPT4A](#) were advanced in close proximity to [1-B1](#) to allow for site specific correlations. On October 19, 2015, four of the CPTs encountered refusal in a dense gravel layer. On October 20, 2015, the drilling contractor advanced a 4-inch-diameter hollow-stem auger using their support truck to a depth of approximately 40 feet to penetrate the gravel zone. They then advanced the CPT through the hollow-stem auger and began collecting data at a depth of 40 feet.



CPT Rig

The CPT has a 20-ton compression-type cone with a 15-square-centimeter ( $\text{cm}^2$ ) base area, an apex angle of 60~ degrees and a friction sleeve with a surface area of 225  $\text{cm}^2$ . The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D-3441. Measurements include the tip resistance to penetration of the cone ( $Q_c$ ), the resistance of the surface sleeve ( $F_s$ ), and pore pressure ( $U$ ) (Robertson and Campanella, 1988). Shear wave velocity ( $V_s$ ) measurements were taken in 1-CPT4 and 1-CPT4A. CPT logs are presented in [Appendix C](#).

Shear wave velocity measurements were also recorded in one CPT. We estimated the  $V_{s30}$ , or shear wave velocity for the upper approximately 100 feet of the site, to be approximately 1,090 feet per second.

## 2.5 SURFACE CONDITIONS

A two-story building was located on the northern half of the 2500 J Street parcel with an asphalt concrete parking lot in the southern half. At 2508 J Street, there were two existing structures and associated paved parking areas. The western and northern boundary of the site consisted of City sidewalks and streets.

According to the Topographic Survey prepared by RSC Engineering dated July 2015, site grades range from approximately Elevation 19 to 21½ feet (Datum = NGVD29).

Please refer to the Site Plan, [Figure 2](#), for more information on site features.

## 2.6 SUBSURFACE CONDITIONS

Based on our subsurface exploration and review of existing data, the subsurface conditions at the site can be described in the following generalized stratigraphy:

**Very Stiff Sandy SILT to Medium Dense Silty SAND:** Our explorations generally encountered a non-plastic to low plasticity sandy silt, silty clay, and silty sand in the upper 17½ to 22 feet below existing grade; the CPTs predicted a soil behavior type consistent with the soil boring. The thickness of this layer appears to increase towards the southern end of the site based on our explorations and those performed as part of the environmental site assessments by others.

**Medium Dense Silty SAND:** Exploration 1-CPT2, 1-CPT3, and 1-CPT4 generally encountered sand to silty sand from approximately 21 to 24 feet, 19 to 21 feet, and 18 to 20 feet below existing grade, respectively. This layer was not apparent in Boring 1-B1 or 1-CPT1.

**Dense Silty Gravel with SAND and Cobble:** In Boring 1-B1, we encountered dense silty gravel with sand starting from approximately 19½ feet and extending to about 40 feet below existing grade. In 1-CPT1, 1-CPT2, 1-CPT3, 1-CPT4, and 1-CPT4A, the top of this dense gravel layer was encountered at approximately 17, 25, 22, and 20 feet below existing grade, respectively. The CPTs encountered refusal within this layer. In 1-CPT1, the rig operator was able to advance a ‘dummy’ tip through the gravel layer and estimated the bottom of the dense layer at approximately 37 feet below grade. The hollow-stem auger for 1-CPT4A encountered the gravel layer from approximately 20 to 37 feet below grade. The fine to coarse gravel that was sampled and or pushed to the surface with the drilling fluid contained some cobble sizes of approximately 3 to 5 inches. The existence of cobble was interpreted from the relatively slow advancement rate drilling through the layer, loud ‘chattering’ noise, driller comments and cobble fragments in select driven samples.

**Medium Dense Clayey SAND to Very Stiff/Hard Sandy Lean CLAY:** From approximately 40 to 46 feet below grade, 1-CPT1 encountered dense clayey sand. The lab results from Boring 1-B1 support the soil behavior type from 1-CPT1, indicating a low to medium plasticity (Plasticity Index of 16) clayey sand. From approximately 46 to 51 feet below grade this medium plasticity layer exhibited an average tip resistance value (qt) of approximately 55 tsf. The blow counts within this layer from Boring 1-B1 were deemed not representative due to cobble suspected at the bottom of the boring that was being advanced downward with the samplers.

**Hard Sandy Lean CLAY to Very Dense Silty SAND to SAND:** At a depth of approximately 55 feet below existing grade in Boring 1-B1, the exploration encountered hard sandy clay and very dense silty sand.

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

## 2.7 GROUNDWATER CONDITIONS

We encountered groundwater at the time of drilling in **Boring 1-B1** at a depth of 19.6 feet below existing grade, or approximate Elevation 0 feet (DATUM = NGVD29). Below is a summary of depth to groundwater encountered at the site based on our review of existing subsurface data:

**TABLE 2.7-1**  
 Groundwater Information Encountered At Site

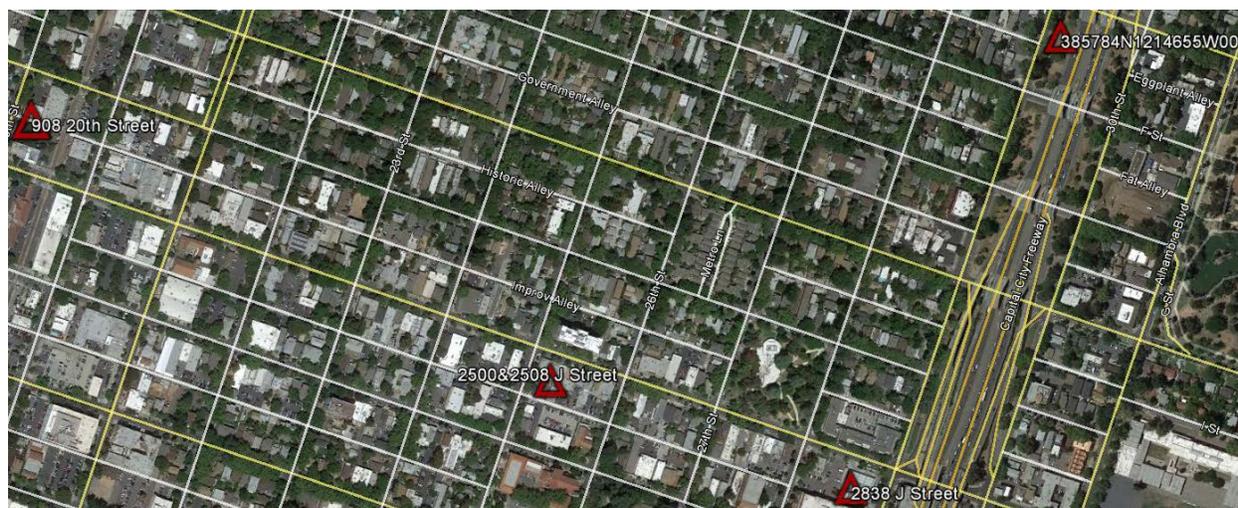
Source	Depth (Elevation, NGVD29)	Date of Reading
ENGEO	19.6 feet (~0 feet)	October 19, 2015
AEI	17.5 feet (SB-1) and 20 feet (SB-2) (~2 and ½ feet, respectively)	September 23, 2015
LUSH	20 feet (~0 feet)	March 22, 2004

To supplement the groundwater data obtained from our explorations, we reviewed publically available information from [www.geotracker.ca.gov](http://www.geotracker.ca.gov) and [www.waterdatalibrary.ca.gov](http://www.waterdatalibrary.ca.gov). Data from nearby sites suggest groundwater in the vicinity may have fluctuated between approximately Elevation 0 to 6½ feet (NGVD29) over the last approximately 20 years; this corresponds to depths of approximately 15 to 22 feet below existing grade. The data shows there were two periods when the groundwater was approximately 5 feet shallower than the general trends from the last 20 years. It is unknown if this has to do with local use of the well, but if projected to the project site it could be an indication that in extremely wet seasons the groundwater may rise to as shallow as 10 feet below the site grade. Nearby irrigation well data is summarized in the table below.

**TABLE 2.7-2**  
 Local Irrigation Well Groundwater Data

Address	Location Relative to Project Site	Groundwater Elevation (feet, NGVD29)		
		Minimum (date)	Maximum (date)	Average Over Last 20 years
2838 J Street (MW-1)	~1,000ft East	0.51 (09/2002)	6.56 (03/1998)	3.15
908 20th Street (MW-2)	~2,400ft West	0.36 (12/2000)	8.56 (05/2006)	3.17
385784N1214655W001*	~2,600ft Northeast	-4.4 (10/1972)	11.1(03/1983)	2 feet since 1995 1.11 since 1968

\*Irrigation well data may not be representative of static groundwater elevation due to local use of well.



Locations of nearby irrigation wells

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

## 2.8 LABORATORY TESTING

We performed laboratory tests on selected soil samples to determine their engineering properties. For this project, we performed moisture content, dry density, unconfined compression, plasticity index, gradation, consolidation, resistance value and soil corrosion potential testing. Moisture contents and dry densities are recorded on the boring log in [Appendix A](#). All other laboratory data are included in [Appendix B](#).

## 2.9 LIQUEFACTION ANALYSES

Soil liquefaction results from loss of strength during cyclic loading, such as that imposed by an earthquake. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded fine sands below the groundwater table. Empirical evidence indicates that loose silty sands, as well as lean silts and some clays are also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress of the soil, it is said to have liquefied, and if the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. In some cases, observed settlement has been amplified directly beneath a building, due to the cyclic rocking of the building foundation, as compared to the surrounding ground surface. This is referred to as the “ratcheting” effect, and is thought to be caused by the interaction of the building foundation and the surrounding soil during seismic shaking.

Some of our explorations encountered thin layers of medium dense silty sand or silt overlying the dense gravel layer. Using methods by Bray and Sancio (2006) and Seed (2003) for determining fine-grained soil liquefaction susceptibility, the silts were classified as being in the

category of potentially liquefiable soil. Therefore, we conservatively analyzed the silt below the groundwater table for liquefaction triggering. Below the dense sand and gravel, we generally encountered medium plasticity sandy lean clay to lean clay underlain by hard and very dense lean clay and sand; we judged these soils as not susceptible to liquefaction triggering.

Using a groundwater depth at approximately 19 feet below existing grade, we checked for potential liquefaction triggering under seismic conditions. Consistent with current California Building Code (CBC) and the ASCE 7-10 documents, we developed the maximum considered earthquake geometric mean ( $MCE_G$ ) ground motions for geotechnical analysis at the site using the  $MCE_G$  peak ground acceleration (PGA) of 0.30g and a moment magnitude ( $M_w$ ) of 6.6 for evaluation of liquefaction triggering. We estimated the earthquake magnitude by taking a weighted average of the faults that contribute to the site shaking hazard by greater than 2 percent from the USGS 2008 Interactive Deaggregation website.

We analyzed liquefaction potential using both the SPT data and CPT data; separate analyses were utilized. In performing our SPT-based liquefaction analysis of 1-B1, we used methods published by Youd et al. (2001) and Idriss & Boulanger (2008). Each recorded SPT blow count resistance (N-value) was corrected for sampler and hammer type, overburden pressure, boring diameter, and fines content. The SPT sampler had room for liners. Assigning a representative blow count of 7 and a fines content of 50 percent, we calculated a factor of safety against liquefaction of the silt from 19 to 20 feet below grade to be approximately 1.

For our CPT-based liquefaction analyses, we calculated potential liquefaction-induced triggering using the commercially available program CLiq. We utilized the method published by Robertson (2009). The analysis identified the medium dense silty sand layer, described in Section 2.7, as being potentially liquefiable in 1-CPT2 and 1-CPT3; both of these CPTs were located on the southern end of the site where the depth to the dense sand and gravel increases. In 1-CPT2, we calculated a factor of safety against liquefaction triggering of approximately 0.8 from a depth of 20 to 21 feet below existing grade or approximate elevation 0 to 1 feet. In 1-CPT3, we calculated a factor of safety against liquefaction triggering of approximately 0.75 from a depth of 20 to 22 feet below existing grade or approximately elevation 0 to -2. Our liquefaction analyses is included in [Appendix D](#).

Based on the calculated factors of safety and thickness of the layers, we estimate liquefaction-induced vertical reconsolidation settlement of approximately  $\frac{1}{2}$  to  $\frac{3}{4}$  inch is possible for the design level seismic event at the locations we encountered.

### 3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the proposed project may be designed as planned, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications. We summarize our conclusions below.

### **3.1 FOUNDATION SUPPORT**

Several types of foundation systems could be and have been used in the Sacramento area for high-rise buildings, each with disadvantages and advantages. In addition to the cost, common considerations in selecting an appropriate foundation system generally include:

- Structural loads and amount of unloading for below-grade portions.
- Acceptable total and differential settlements.
- Noise and vibration during construction.
- Depth to groundwater.
- Soil and groundwater spoils produced during construction.
- Hazardous soil and groundwater concerns.
- Quality control.

Due to the anticipated high column loads, over-excavation to accommodate the basement, and the presence of a dense gravel layer at about 20 to 25 feet below existing grade, we recommend supporting the structure on a mat foundation. As an alternative, drilled piers may be used. See Section 4 for foundation recommendations.

### **3.2 GROUNDWATER CONSIDERATIONS**

As discussed in Section 2.7, limited historic groundwater level data in the vicinity of the site indicates groundwater may have been as shallow as approximately 15 feet below existing grade over the last 20 years. One location approximately ½ mile from the site indicates groundwater may have risen to approximately 10 feet below grade in abnormally wet seasons. Subsurface explorations performed by ENGEO and others encountered groundwater at a depth of approximately 17½ to 20 feet below existing grade on the site.

Based on historic groundwater data we reviewed and the currently proposed basement depth, we anticipate the bottom of the basement may extend to or below the groundwater table and would likely require dewatering during construction. For post-construction conditions, if installation of a permanent dewatering system is not feasible, then it would be necessary to waterproof portions of the building that extend below the high groundwater elevation. The building structural design must also consider buoyancy impacts.

We recommend that waterproofing be provided below the anticipated seasonal high groundwater level, which we anticipated to be approximately 15 feet below existing grade or approximately Elevation 5 feet (NGVD29). This is based on very limited site data and the design team should consider if groundwater monitoring through the installation of a site monitoring well would be desirable.

Temporary construction dewatering would likely consist of a perimeter system of dewatering wells to temporarily lower the groundwater while below-grade construction is completed. Once the building construction has extended above the seasonal high groundwater elevation and has

sufficient dead weight to resist hydrostatic uplift, the temporary construction dewatering could be terminated.

### **3.3 SEISMIC HAZARDS**

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, soil liquefaction, lateral spreading, landslides, tsunamis, flooding or seiches is considered low to negligible at the site.

#### **3.3.1 Ground Rupture**

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

#### **3.3.2 Ground Shaking**

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

#### **3.3.3 Liquefaction**

As described in Section 2.9, limited soil deposits immediately above the dense gravel may be potentially liquefiable in a design level seismic event. Based on the calculated factors of safety and thickness of the layers, we estimate liquefaction-induced vertical reconsolidation settlement of approximately  $\frac{1}{2}$  to  $\frac{3}{4}$  inch is possible for the design level seismic event. Because the proposed structure includes a basement and we are recommending a mat foundation to extend to a minimum depth of 20 feet below grade, these liquefiable soil layers will be removed during construction.

### 3.3.4 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils that can be damaging to improvements. The potential for the formation of these cracks is considered greater at contacts between thick alluvium and shallow bedrock. Based on the depth to bedrock and vicinity to active faults, the risk of ground lurching impacts is negligible, in our opinion.

### 3.4 EXISTING FILL

Although not identified in our soil boring, existing undocumented fill is anticipated due to the former and existing site development. At this time, the proposed building footprint will cover nearly the entire site and the bottom of the basement excavation is expected to extend approximately 15 to 20 feet below existing grade. Therefore, we anticipate existing undocumented fills will be removed as part of the basement excavation. Undocumented fill in areas outside of the basement excavation that will support future improvements would require overexcavation and removal. We present fill removal recommendations in Section 5.

Our site research revealed that portions of the site had former buildings or structures that were demolished and removed. It is not known if the foundations or underground utilities associated with these structures were removed or if there were any basements or crawl spaces that may have been backfilled. In our experience, infill developments sometimes have insufficient records of previous below-grade structures that may be encountered during construction. It may be beneficial to consider non-destructive geophysical exploration such as ground penetrating radar or magnetometer surveys to search for evidence of buried structures or debris.

### 3.5 SOIL CORROSION POTENTIAL

As part of this study, we obtained one representative soil sample for sulfate testing to assist in foundation design and one representative soil sample that we submitted to a qualified analytical lab for determination of pH, resistivity, sulfate, and chloride to assist in selecting appropriate corrosion protection for underground utilities. The results are included in [Appendix B](#) and summarized in the table below.

**TABLE 3.5-1**  
 Corrosivity Test Results

Sample Location	Depth	pH	Resistivity (ohms-cm)	Chloride (mg/kg)	Sulfate (mg/kg)
1-B1	14	NT	NT	NT	<50*
1-B1	0-1	7.6	4,420	7.5	0.7**

\*As determined by ASTM C1580      NT = Not Tested

\*\*As determined by CT-417

The 2013 CBC references the 2011 American Concrete Institute Manual, ACI 318-11, Chapter 4, Sections 4.2.1 for structural concrete requirements. ACI Table 4.2.1 provides the exposure categories and classes, and concrete requirements in contact with soil based upon the exposure risk.

In accordance with the criteria presented in the above referenced table, these soils are categorized as being within S0 sulfate exposure class. Considering a ‘Not Applicable’ sulfate exposure, there is no requirement for cement type or water-cement ratio. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

The soils are classified as moderately corrosive to buried metal pipe according to the National Association of Corrosion Engineers’ 1984 “Corrosion Basics an Introduction” interpretation of resistivity. Values tested for chloride do not pose a significant impact to metals or concrete.

If desired to investigate this further, we recommend a corrosion consultant be retained to determine if specific corrosion recommendations are necessary for the project.

**TABLE 3.5-2**  
 ACI Table 4.2.1: Exposure Categories and Classes

Category	Severity	Class	Condition	
<b>F</b> Freezing and thawing	Not Applicable	F0	Concrete not exposed to freezing-and-thawing cycles	
	Moderate	F1	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture	
	Severe	F2	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture	
	Very Severe	F3	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals	
<b>S</b> Sulfate			<b>Water- Soluble Sulfate in Soil % by Weight*</b>	<b>Dissolved Sulfate in Water mg/kg (ppm)**</b>
	Not applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 \leq 1,500$ seawater
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1,500 \leq SO_4 \leq 10,000$
	Very severe	S3	$SO_4 > 2.00$	$SO_4 > 10,000$
<b>P</b> Requiring low permeability	Not applicable	P0	In contact with water where low permeability is not required.	
	Required	P1	In contact with water where low permeability is required.	

Category	Severity	Class	Condition
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources
*Percent sulfate by mass in soil determined by ASTM C1580			
**Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130			

### 3.6 2013 CBC SEISMIC DESIGN PARAMETERS

The 2013 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered and the shear wave velocity measurements from our CPT, we characterized the site as Site Class D in accordance with the 2013 CBC. We provide the 2013 CBC seismic design parameters in Table 3.6-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE<sub>R</sub>) spectral response acceleration parameters.

**TABLE 3.6-1**  
 2013 CBC Seismic Design Parameters  
 Latitude: 38.57446 Longitude: -121.47302

Parameter	Value
Site Class	D
Mapped MCE <sub>R</sub> Spectral Response Acceleration at Short Periods, S <sub>S</sub> (g)	0.66
Mapped MCE <sub>R</sub> Spectral Response Acceleration at 1-second Period, S <sub>1</sub> (g)	0.29
Site Coefficient, F <sub>A</sub>	1.27
Site Coefficient, F <sub>V</sub>	1.82
MCE <sub>R</sub> Spectral Response Acceleration at Short Periods, S <sub>MS</sub> (g)	0.84
MCE <sub>R</sub> Spectral Response Acceleration at 1-second Period, S <sub>M1</sub> (g)	0.53
Design Spectral Response Acceleration at Short Periods, S <sub>DS</sub> (g)	0.56
Design Spectral Response Acceleration at 1-second Period, S <sub>D1</sub> (g)	0.35
Mapped MCE Geometric Mean (MCE <sub>G</sub> ) Peak Ground Acceleration, PGA (g)	0.23
Site Coefficient, F <sub>PGA</sub>	1.35
MCE <sub>G</sub> Peak Ground Acceleration adjusted for Site Class effects, PGA <sub>M</sub> (g)	0.30
Long period transition-period, T <sub>L</sub>	12

### 3.7 FLOODING

The City of Sacramento is located in a historic flood plain and is protected from flooding by levee systems along the American and Sacramento Rivers. The Flood Insurance Rate Map

(FIRM) for the City of Sacramento, California, dated June 16, 2015, identifies the site in Zone X, which is mapped as “protected from the 1-percent-annual-chance or greater flood hazard by a levee system.” Along with the river levee systems, the Sacramento Area is protected from flooding by Folsom Dam located upstream on the American River. In the event of a flood larger than the flood control system is designed for, or in the event of levee or dam failure, the site would be subject to flooding.

## **4.0 FOUNDATION RECOMMENDATIONS**

We developed foundation recommendations using data obtained from our field exploration, laboratory test results, engineering analysis, and experience. We recommend that the proposed building be supported on either a mat foundation bearing in the dense gravel layer at a depth of approximately 20 to 25 feet below grade or on deep foundations.

### **4.1 MAT FOUNDATION**

The proposed building may be supported on a rigid mat foundation bearing in the dense gravel layer. We recommend that the bottom of the mat foundation bear at a minimum depth corresponding to Elevation -1 feet; this may need to be extended as much as 4 to 5 feet deeper due to variations in subsurface conditions across the building footprint.

An average allowable bearing pressure of 2,000 pounds per square foot (psf) may be used with localized increases near column loads of up to 2,500 psf; these may be increased by one-third when considering transient loads, such as wind or seismic. The structural design of the mat should be an iterative process between ENGEO and the structural engineer and will be highly dependent upon the actual bottom of mat elevation, thickness of mat determined by the structural engineer, the column loads and column spacing, and anticipated settlement estimated by ENGEO for each of these combinations. For the first iteration of design, we recommend the structural engineer analyze the mat with a uniform modulus of subgrade reaction ( $k_{v1}$ ) of 20 pounds per square inch per inch of deflection (psi/in) at this elevation.

While we were not provided any structural loads for the proposed building, we anticipate that the average mat foundation pressure for 12 stories of residential over 2 stories of parking will likely be in the range of 1,500 to 2,000 psf. The result of removing 20 feet of soil for the below-grade excavation results in very little increase in net load on the soil beneath the building. We anticipate a properly-designed rigid mat foundation would likely experience total settlements of about 1 inch with differential settlements of about one-half of the total. The actual foundation settlements will be a function of the applied loads, the stiffness of the mat, and the modulus of subgrade reaction.

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of the mat foundation. We recommend a passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf). We recommend a coefficient of friction along the base of 0.35. These values include a factor of safety of 1.5 and may be increased by one-third for the short-term effects of wind or seismic loading.

Since the depth to the dense gravel bearing layer may locally vary beneath the building footprint, we recommend that pre-construction exploration be performed to determine the actual depth to this layer within the area of the actual building footprint. This would help refine the necessary depth for the braced excavation and provide greater clarity regarding the final mat foundation depth.

## **4.2 DRILLED PIERS**

As an alternative to a mat foundation, the building may be supported drilled, cast-in-place, straight-shaft friction piers. The piers should have a minimum diameter of 24 inches and extend to a depth of at least 30 feet below the bottom of the pier cap. Design piers for an allowable downward skin friction of 400 pounds per square foot from 0 to 30 feet below the pier cap for combined dead-plus-live loads with a one-third increase allowed for either transient wind or seismic loading. The allowable downward skin friction may be increased to 600 pounds per square foot for depths greater than 30 feet below the pier cap. Resistance to uplift loads will be developed in friction along the pier shafts. We recommend an allowable uplift frictional resistance of 70 percent of the above downward values. To reduce pile group effects, space piers at least three diameters apart, center to center.

We anticipate that the tops of drilled piers will likely be at about the top of the dense gravel layer and will extend below the groundwater table; therefore, this will require placing concrete in the wet. The bottoms of drilled pier excavations should be reasonably clean and free of loose soil before reinforcing steel is installed and concrete is placed. Concrete will need to be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete at all times to avoid entrapment of water in the concrete. We recommend that drilled pier concrete have a minimum compressive strength of 3,000 psi.

Due to the potential for caving, each shaft may need to be cased. ENGEO should be onsite during drilled pier excavation to observe soil conditions encountered across the site for comparison with the soil conditions observed during our subsurface exploration. Additionally we will monitor concrete pump volumes to determine if any voids developed during excavation of the pier shaft or during casing removal.

Structural loads and the number of piers are not known at this time. On a preliminary basis, we estimate that drilled pier total settlements will be less than about 1 inch. Once loads and pier layout are determined, we should be retained to review the design and update our settlement estimates.

Lateral load resistance for drilled piers is developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon several factors including pile stiffness, pile embedment length, conditions of fixity at the pile cap, the physical properties of surrounding soil, and the magnitude of lateral deflections. We used the computer program LPILE to estimate lateral pile loads for ¼- and ½-inch pile top deflections assuming a vertical load of 100 kips. Lateral capacities and deflection characteristics were calculated using pier stiffness (EI) of  $5.08 \times 10^{10}$  for a 24-inch diameter concrete pier with an assumed 28-day concrete compressive

strength of 3,000 psi. If pile stiffness varies by no more than 20 percent of that reported above, then load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness. For pile stiffness significantly different from the values listed above, we should be contacted to provide revised lateral pile characteristics.

**TABLE 4.2-1**  
Estimated Lateral Capacities (Single 24-inch diameter pier)

Allowable Lateral Capacity (Kips)		
Pile Condition	¼-inch Deflection	½-inch Deflection
Free Head	18	23
Fixed Head	36	49

The above lateral capacities represent the probable response of a single pier under short term loading conditions and do not include a factor of safety. Suitable factors of safety should be selected based on the type of loading.

We also estimated maximum bending moments and points of fixity for drilled piers for ¼- and ½-inch pile top deflection for both fixed and free head conditions. As referenced in the table below, “point of fixity” is defined as a point of zero lateral deflection. We present the results in Table 4.2-2 below:

**TABLE 4.2-2**  
Load Deflection Characteristics  
Cast in place Drilled Piers

Pile Type	Deflection Characteristic	Pile Deflection		Pile Deflection	
		Free Head		Fixed Head	
		¼-inch	½-inch	¼-inch	½-inch
24-inch Diameter Pier	Maximum Bending Moment (in-kips)	934	1256	1850	2677
	*Depth to Maximum Bending Moment (feet)	6.9	6.9	0	0
	*1 <sup>st</sup> Point of Fixity (feet)	12.6	12.9	16.2	16.5
	*2 <sup>nd</sup> Point of Fixity (feet)	22.8	23.1	24.0	24.3

\*Below Top of Pier

Research has shown that the lateral capacity of a group of piles is generally less than that of a single pile for pile spacings less than 6 to 8 pile diameters. For pile groups with a minimum spacing of 3 pile diameters, we recommend reducing the single pile allowable lateral capacities by the percentages in the following table.

**TABLE 4.2-3**  
Group Reduction Percentages

<b>Number of Piles in Group</b>	<b>Percentage to Reduce single Pile Capacity By</b>
2	25
4	30
9	43
16	48
25	54

Please contact us if group reduction percentages are needed for additional pile group configurations. We should be provided the opportunity to refine these numbers based on the pile configuration selected.

## **5.0 BELOW-GRADE RETAINING WALLS**

### **5.1 LATERAL EARTH PRESSURES**

Design below grade retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design walls restrained from movement at the top to resist an equivalent fluid pressure of 65 pounds per cubic foot (pcf). In addition, design restrained walls to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the value recommended above. Waterproofing should be installed where permanent walls and floors extend below the groundwater elevation (see Section 3.2).

If below-grade walls are not designed for hydrostatic pressures, construct a drainage system as recommended in Section 5.3.

### **5.2 SEISMIC LOADING**

Based on the subsurface conditions encountered during our geotechnical exploration and the site specific peak horizontal ground acceleration of 0.3g, we developed seismic design parameters for restrained basement retaining walls. If the wall is restrained from movement at the top, the resultant load would be  $12 H^2$  acting at  $1/3 H$  from the wall base. In these equations, the load is in pounds per foot of wall length, and the dimension H is the height of retained earth, in feet.

### 5.3 RETAINING WALL DRAINAGE

For below grade retaining walls that are not fully water-proofed and designed for hydrostatic pressure, we recommend installation of a geosynthetic drainage composite behind the retaining wall to reduce hydrostatic lateral forces. ENGEO should review and approve geosynthetic composite drainage systems prior to use.

### 6.0 TEMPORARY SHORING

Temporary construction excavations will require shoring. Temporary shoring should be designed to resist lateral earth pressure from adjoining material and any surcharge loads from traffic, adjacent buildings, or construction equipment and materials. We provide the following design criteria for design of temporary shoring.

**TABLE 6.0-1**  
Recommended Lateral Earth Pressure for Temporary Shoring

Earth Pressure	Equivalent Fluid Density, Drained Condition (pcf)
Active	50
At-Rest	80
Passive	300

The above lateral earth pressures assume level backfill conditions and no surcharge loading. The pressures are based on an equivalent fluid pressure in pounds per cubic foot (pcf).

The choice of shoring should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. Support of adjacent structures and utilities without distress is the contractor's responsibility. The anticipated perimeter shoring may consist of a soldier beam and lagging or sheet pile wall appropriately designed by a qualified registered engineer. These will likely include tiebacks to resist horizontal earth pressures. A soil nail wall retaining system may also be considered. We recommend that ENGEO review the contractor's plan for the excavation bracing prior to construction.

### 7.0 ADJACENT BUILDING AND STREET SUPPORT

A two-story masonry building was located to the east of the site and City of Sacramento right-of-way lies to the north, west and south. Design of the planned below-grade walls will need to include potential loading from these nearby buildings and roadways, if applicable, and construction measures would need to be implemented to protect the existing improvements from any construction impacts. This typically involves:

- Research to determine the existing foundation types for adjacent buildings. This would involve City records review and possible temporary excavations to explore existing foundation types.

- Evaluate potential ground movement from the planned excavation and temporary groundwater drawdown.
- Monitor the existing buildings during construction to check that ground movements, if any, are within a tolerable range.
- Establish construction protocols for mitigating any unplanned ground deformations.

## **8.0 SLABS-ON-GRADE**

If drilled piers are selected for foundation support, a slab-on-grade will be needed for the bottom floor if it is not designed as a structural mat. We recommend a minimum concrete thickness of 5 inches with minimum steel reinforcing of No. 3 rebar on 18-inch centers each way within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures. The structural engineer should provide final design thickness and additional reinforcement for any structural and hydrostatic loads.

If desired to reduce water vapor from beneath the bottom slab, we recommend installation of a vapor retarder. The vapor retarder membrane should be sealed at all seams and pipe penetrations and connected to all grade beams. The vapor retarder should meet the minimum requirements of a Class A vapor retarder in ASTM E 1745-97 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs”.

With the use of a slab-on-grade as the bottom floor of the building, we recommend the design team consider the use of a subdrain system to control groundwater. At a minimum the subdrain system should consist of:

1. A minimum 18-inch-thick layer of washed, crushed rock below the basement slab. Crushed rock should consist of 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Place a nonwoven geotextile filter fabric such as Mirafi 140NC, or equivalent below the rock.
2. Place 4-inch-diameter perforated pipe within the rock layer at a minimum 25 foot lineal spacing. Place pipes with perforations placed down, approximately 4 inches from the bottom of the rock layer. Slope pipes at least 1 percent toward a central collector pump system.
3. Remove collected water with a suitable collector pump system.
4. Construct cleanouts for drain maintenance.

We should be retained to review the subdrainage system prior to construction.

## **9.0 EXTERIOR FLATWORK**

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum concrete flatwork thickness of 4 inches. Construct control and construction joints in accordance with current Portland Cement Association Guidelines. City of Sacramento standard details may apply to exterior flatwork where encroaching within City right-of-way.

## **10.0 EARTHWORK RECOMMENDATIONS**

The relative compaction and optimum moisture content of soil, rock, and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as determined by an ENGEO representative. As used in this report, the term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define “structural areas” in this report as any area sensitive to settlement of compacted soil. These areas include, but are not limited to the building, sidewalks, pavement areas, and retaining walls.

### **10.1 DEMOLITION AND CLEARING**

Clear improvement areas of all surface and subsurface deleterious materials including existing foundations, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots.

Our site research revealed that portions of the site had former buildings or structures that were demolished and removed. It is not known if the foundations or underground utilities associated with these structures were removed or if there were any basements or crawl spaces that may have been backfilled. Previous below-grade structures may be discovered during construction. It may be beneficial to consider non-destructive geophysical exploration such as ground penetrating radar or magnetometer surveys to search for evidence of buried structures or debris.

### **10.2 EXISTING FILL REMOVAL**

Remove all existing fill to competent native soil, as determined by ENGEO. We anticipate that existing fill within the building footprint will be removed as part of the basement construction. The lateral extent and depth of fill is expected to vary.

### **10.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS**

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. In addition, wet soil conditions may be found near the bottom of the basement excavation. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by frequent

spreading and mixing during dry weather, mixing with drier materials, or amending with chemical admixtures such as cement or lime.

#### **10.4 ACCEPTABLE FILL**

Onsite soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 8 inches in maximum dimension. Organic concentrations should be less than 3 percent by weight.

Imported fill materials should meet the above requirements and have a plasticity index less than 12, and at least 20 percent passing the No. 200 sieve. Allow ENGEO to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

#### **10.5 FILL COMPACTION**

##### **10.5.1 Grading in Structural Areas**

Perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 8 inches;
2. Moisture condition soil to at least 1 percentage point above the optimum moisture content;  
**and**
3. Compact the subgrade to at least 90 percent relative compaction. Compact the upper 6 inches of finish pavement subgrade to at least 95 percent relative compaction prior to aggregate base placement.

After the subgrade soil has been compacted, place and compact acceptable fill (defined in Section 4) as follows:

1. Spread fill in loose lifts that do not exceed 8 inches;
2. Moisture condition lifts to at least 1 percentage point above the optimum moisture content;  
**and**
3. Compact fill to a minimum of 90 percent relative compaction; compact the upper 6 inches of pavement subgrade to 95 percent relative compaction prior to aggregate base placement.

Subgrade processing is not required where competent dense gravel is exposed, as determined by ENGEO's field representative. Compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

## 10.5.2 Underground Utility Backfill

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

Place and compact trench backfill in structural areas as follows:

1. Trench backfill should have a maximum particle size of 6 inches;
2. Moisture condition trench backfill to or slightly above the optimum moisture content. Moisture condition backfill outside the trench;
3. Place fill in loose lifts not exceeding 12 inches; and
4. Compact fill to a minimum of 90 percent relative compaction (ASTM D1557).

Jetting of backfill is not an acceptable means of compaction.

## 10.6 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical.

## 11.0 PAVEMENT DESIGN

### 11.1 FLEXIBLE PAVEMENTS

We obtained a representative bulk sample of the surface soil and performed an R-value test to provide data for pavement design. The results of the test are included in [Appendix B](#) and indicate an R-value of 60. Because surface soils are expected to vary across the site, it is our opinion that an R-value of 40 is applicable for design. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

**TABLE 11.1-1**  
Recommended Asphalt Concrete Pavement Sections

Traffic Index	Section	
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5	2½	5
6	3	7

Traffic Index	Section	
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
7	4	7
8	4½	9

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

## 11.2 RIGID PAVEMENTS

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

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

## 11.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 8.7. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

## 12.0 CONSTRUCTION MONITORING

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

1. Collaborate with the structural engineer in evaluating the actual structural loads on the mat foundation to refine the subgrade modulus values for proper foundation performance.
2. Review the final grading and foundation plans and specifications prior to construction to determine whether our recommendations have been implemented, and to provide additional or modified recommendations, if necessary. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

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

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

### **13.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS**

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the 2500 & 2508 J Street Yamane Lofts project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

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

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

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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

**FIGURES**

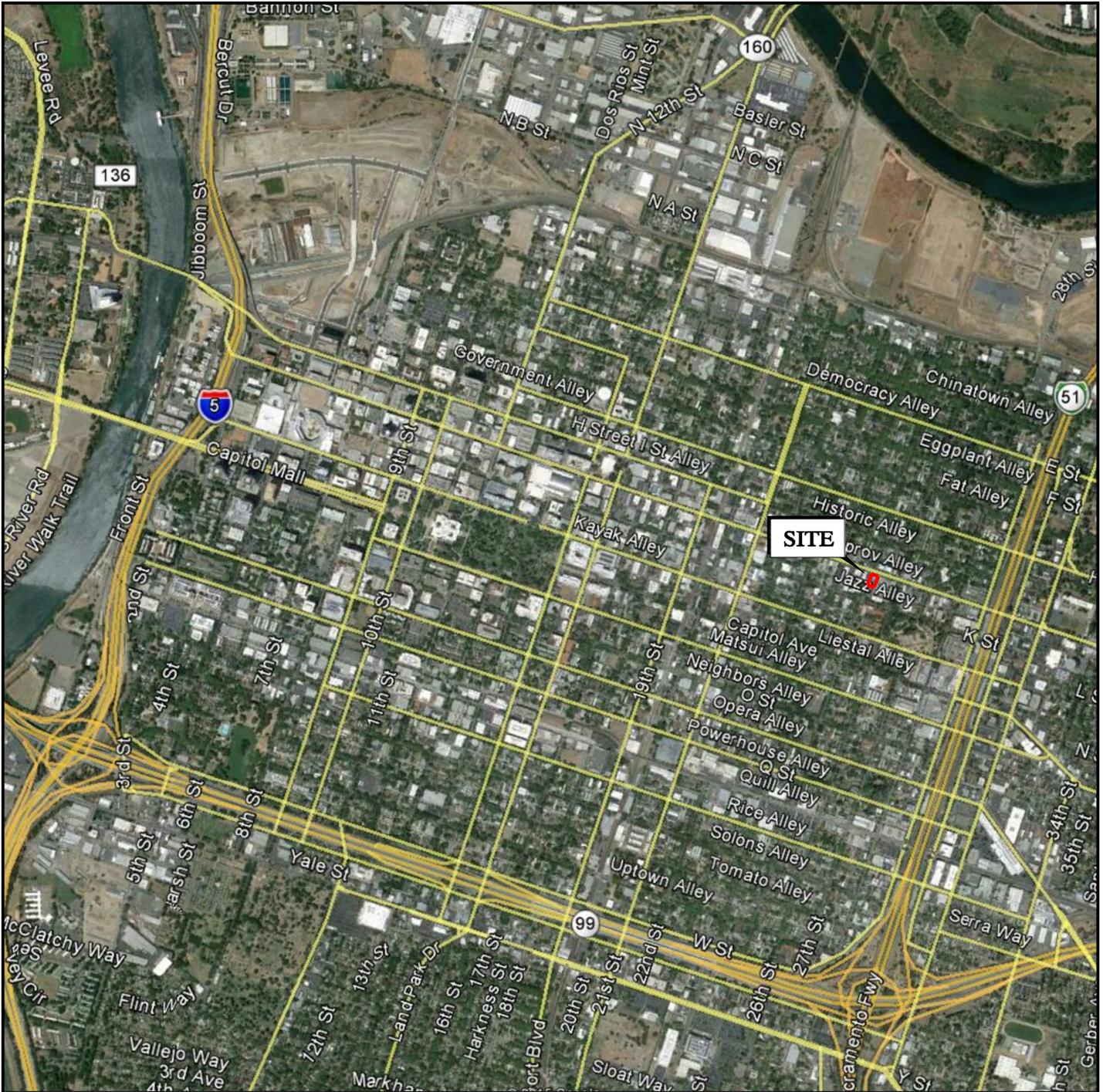
**Figure 1 - Vicinity Map**

**Figure 2 - Site Plan**

**Figure 3 - Regional Geologic Map**

**Figure 4 - Regional Faulting and Seismicity Map**





BASE MAP SOURCE: GOOGLE EARTH PRO



VICINITY MAP  
 YAMANEE LOFTS - 2500 & 2508 J STREET  
 SACRAMENTO, CALIFORNIA

PROJECT NO.: 12487.000.000

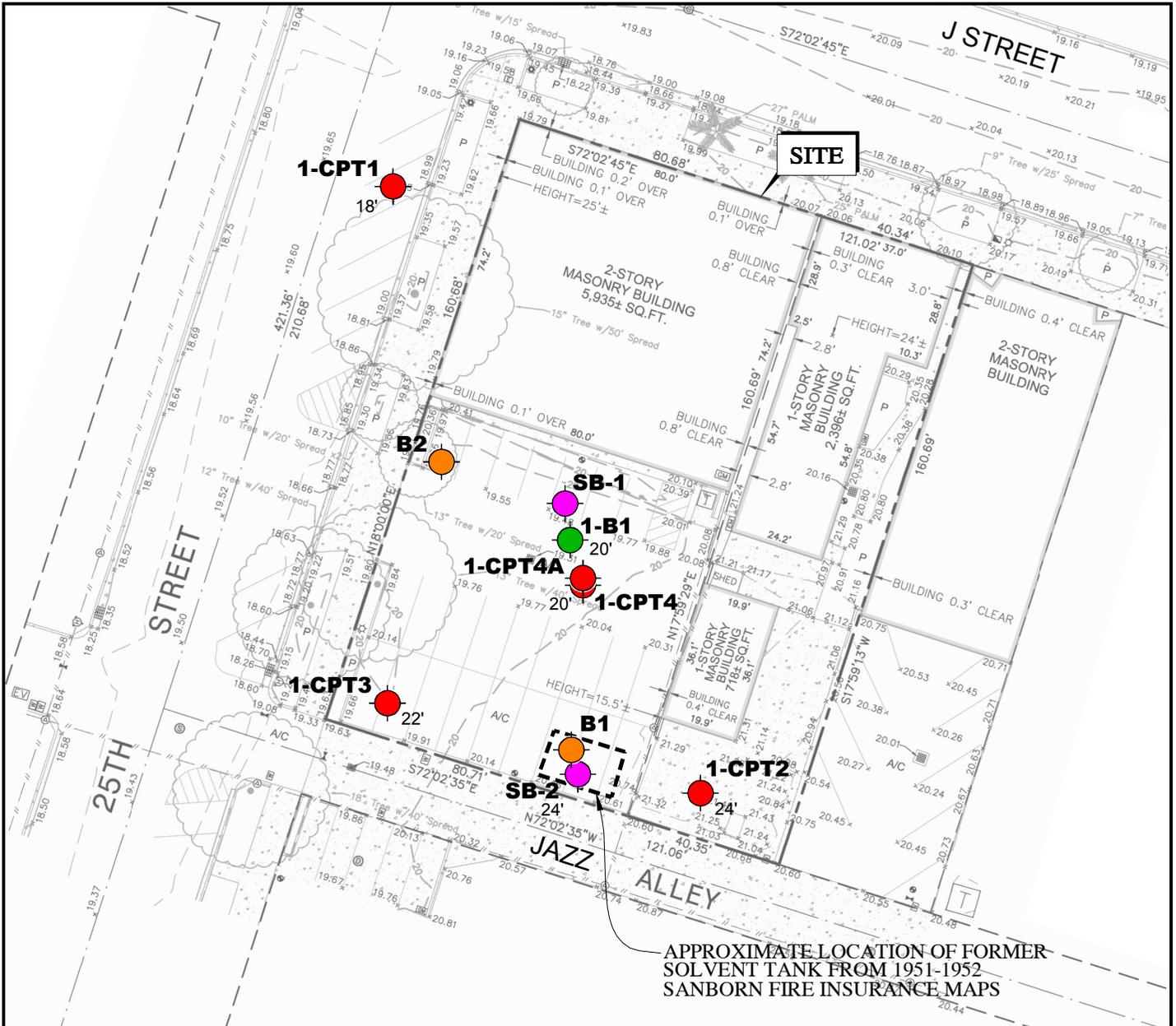
SCALE: AS SHOWN

DRAWN BY: LL

CHECKED BY: MG

FIGURE NO.

1



**EXPLANATION**

ALL LOCATIONS ARE APPROXIMATE

**1-B1** 20' BORING WITH APPROXIMATE DEPTH TO TOP OF DENSE GRAVEL ZONE (ENGEO, 2015)

**1-CPT4** 20' CONE PENETRATION TEST WITH APPROXIMATE DEPTH TO TOP OF DENSE GRAVEL ZONE (ENGEO, 2015)

**SB-2** 24' SOIL AND GROUNDWATER SAMPLING POINT WITH APPROXIMATE DEPTH TO TOP OF DENSE GRAVEL ZONE (AEL, 2014)

**B2** 24' SOIL AND GROUNDWATER SAMPLING POINT (LUSH, 2004)



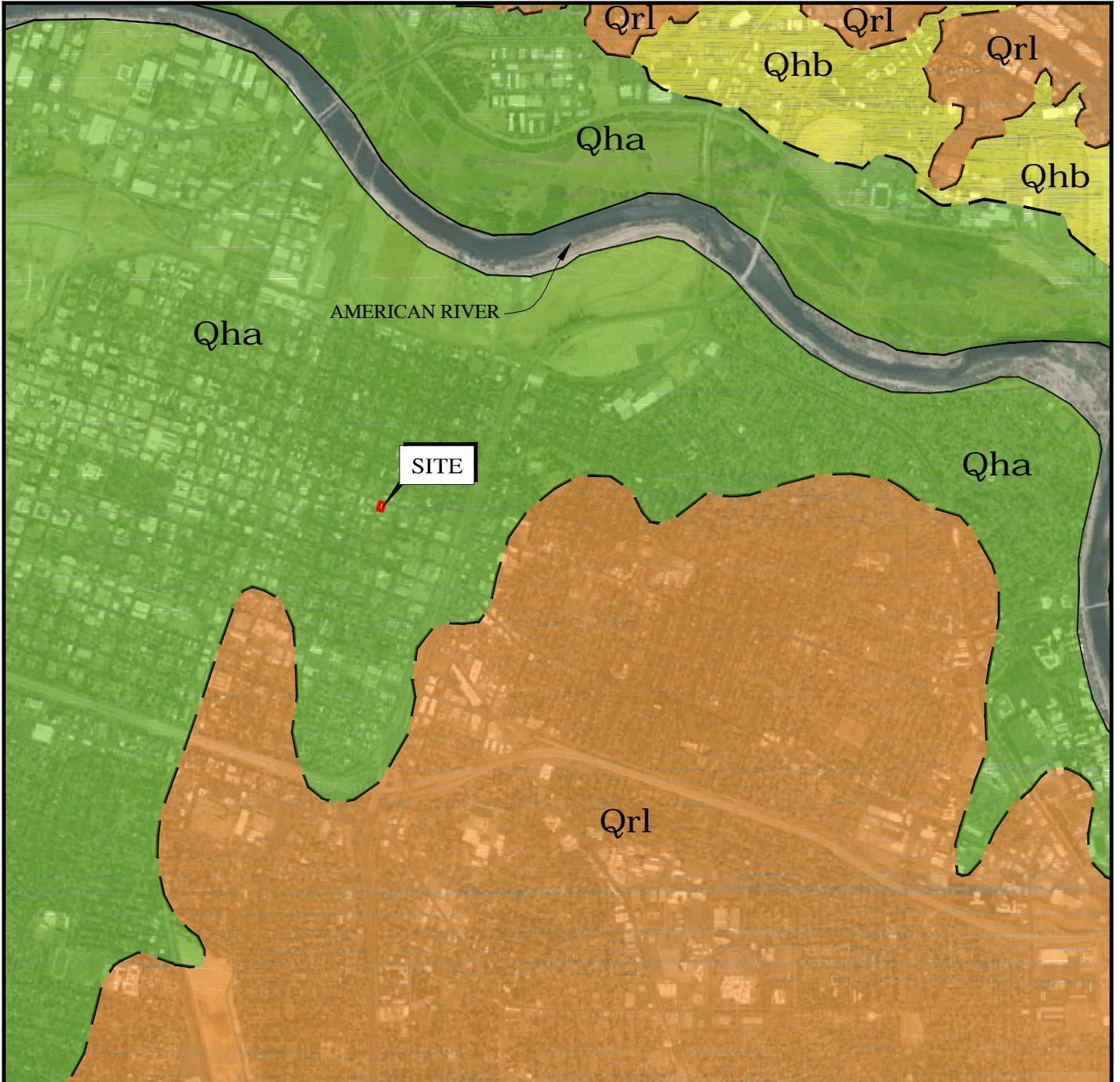
BASE MAP SOURCE: RSC ENGINEERING



SITE PLAN  
YAMANE LOFTS - 2500 & 2508 J STREET  
SACRAMENTO, CALIFORNIA

PROJECT NO.: 12487.000.000	FIGURE NO.
SCALE: AS SHOWN	2
DRAWN BY: LL	

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**EXPLANATION**

- Qha HOLOCENE ALLUVIUM
- Qhb HOLOCENE BASIN DEPOSITS
- Qrl QUATERNARY LOWER RIVERBANK FORMATION

BASE MAP SOURCE: GOOGLE EARTH PRO & HELLEY, 1979



**REGIONAL GEOLOGIC MAP**  
 YAMANEE LOFTS - 2500 & 2508 J STREET  
 SACRAMENTO, CALIFORNIA

PROJECT NO.: 12487.000.000

SCALE: AS SHOWN

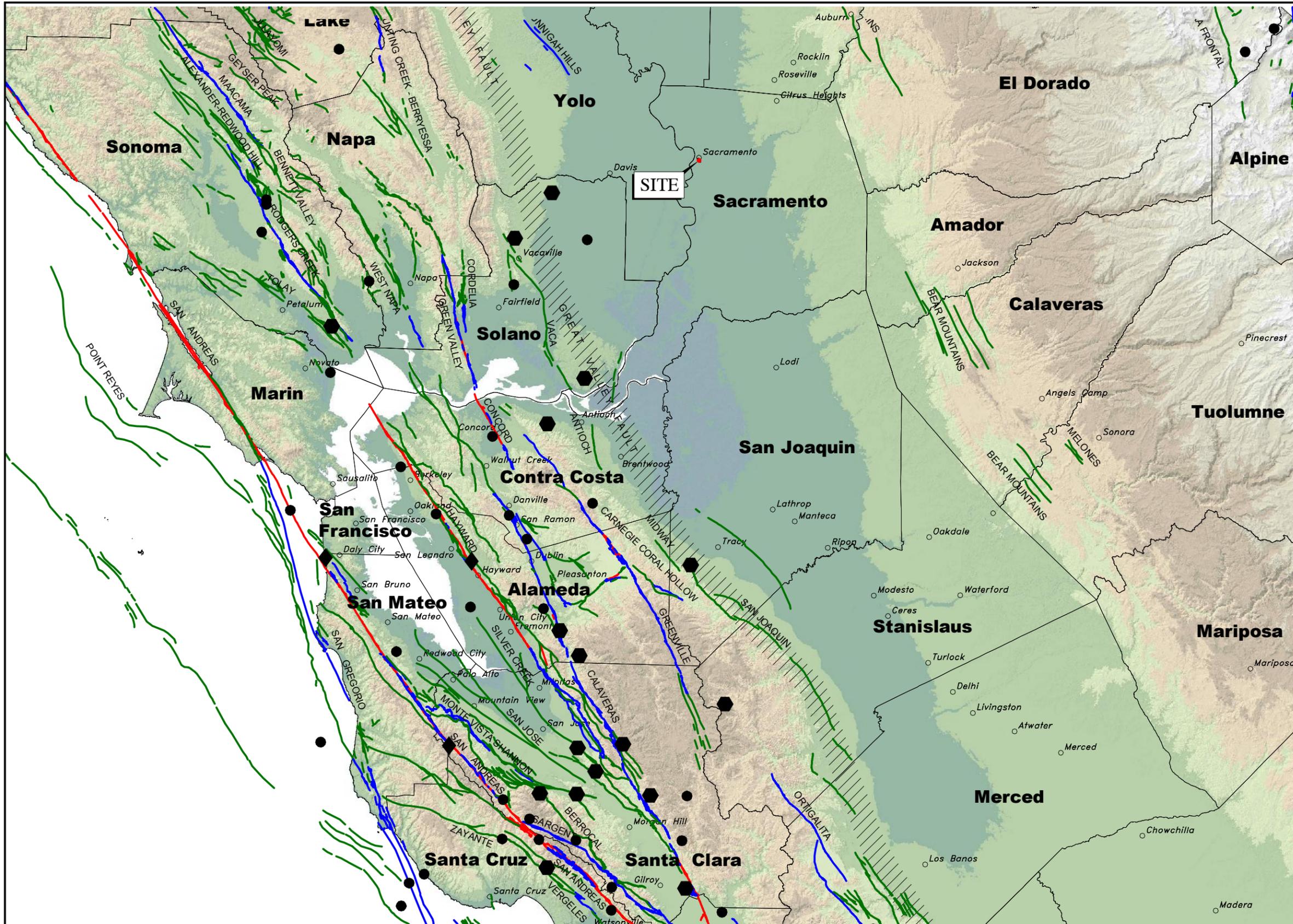
DRAWN BY: LL

CHECKED BY: MG

FIGURE NO.

3

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**EXPLANATION**

- ◆ MAGNITUDE 7+
- ⬡ MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- /// HISTORIC BLIND THRUST FAULT ZONE

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



REGIONAL FAULTING AND SEISMICITY  
 YAMANE LOFTS - 2500 & 2508 J STREET  
 SACRAMENTO, CALIFORNIA

PROJECT NO:	12487.000.000	FIGURE NO.	4
SCALE:	AS SHOWN		
DRAWN BY:	LL	CHECKED BY:	

ORIGINAL FIGURE PRINTED IN COLOR

**A  
P  
P  
E  
N  
D  
I  
X  
  
A**

**APPENDIX A**  
**Key to Boring Logs**  
**Exploration Logs**



## KEY TO BORING LOGS

MAJOR TYPES		DESCRIPTION	
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity CL - Inorganic clay with low to medium plasticity OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity CH - Fat clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

## GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE				CLEAR SQUARE SIEVE OPENINGS			
200                      40                      10                      4				3/4"                      3"                      12"			
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

### RELATIVE DENSITY

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

### CONSISTENCY

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

### SAMPLER SYMBOLS

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

### MOISTURE CONDITION

Dry	Dusty, dry to touch
Moist	Damp but no visible water
Wet	Visible freewater

### LINE TYPES

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

### GROUND-WATER SYMBOLS

	Groundwater level during drilling
	Stabilized groundwater level

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

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



# LOG OF BORING 1-B1

Geotechnical Exploration  
2500 & 2508 J St. Yamanee Lofts  
Sacramento, CA  
12487.000.000

DATE DRILLED: 10/19/2015  
HOLE DEPTH: 101.5 ft.  
HOLE DIAMETER: 5.5 in.  
SURF ELEV (NGVD29): Approx. 19½ ft.

LOGGED / REVIEWED BY: N. Broussard / MMG  
DRILLING CONTRACTOR: GeoExploration  
DRILLING METHOD: HSA/Mud Rotary  
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			2 inches of asphalt concrete over 3 inches of aggregate base										
			SANDY SILT (ML), brown, very stiff, moist, fine grained sand, poorly graded, approximately 40% non plastic fines.			8				74	15.1	89.8	3.0
5	15					9					18.2		3.0
10	10		Grades to dark brown			12	27	22	5	69	21.9	93.3	3.5
15	5		SILTY SAND to SANDY SILT (SM/ML), dark brown, loose, moist, fine grained sand, poorly graded, approximately 40-50% low plasticity silt, one 3/4" subrounded piece of gravel.			10				50			2.5
20	0		SILTY GRAVEL with SAND (GM), brown to grayish brown, wet, trace to medium grained sand, gravel rounded to subrounded, coarse gravel fragments in shoe of sample.			28				18	11.1		
			Cobble approximately 3 - 4" long by 2" wide in cuttings.										
25	-5		Grades to clean Sandy Gravel. Blow count of 5, 13, and 24. Sampler advancement slowed half way through last 6 inches with representative blowcount of 2 to 3 blows per inch.			37				5			

LOG - GEOTECHNICAL W/ELEV. 12487.000.000 J STREET BORING LOGS.GPJ ENGEO INC.GDT 11/13/15



# LOG OF BORING 1-B1

Geotechnical Exploration  
2500 & 2508 J St. Yamanee Lofts  
Sacramento, CA  
12487.000.000

DATE DRILLED: 10/19/2015  
HOLE DEPTH: 101.5 ft.  
HOLE DIAMETER: 5.5 in.  
SURF ELEV (NGVD29): Approx. 19½ ft.

LOGGED / REVIEWED BY: N. Broussard / MMG  
DRILLING CONTRACTOR: GeoExploration  
DRILLING METHOD: HSA/Mud Rotary  
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
30	-10	NR	SILTY GRAVEL with SAND (GM), brown to grayish brown, wet, trace to medium grained sand, gravel rounded to subrounded, coarse gravel fragments in shoe of sample. At 27 feet loud chattering and drill fluid bringing almost all gravel to surface with little sand. Chattering continued to approximately 40 feet. Appears to be bouncing on cobble at 29 feet.			50/3							
35	-15	NR				50/2							
40	-20		Sandy gravel with cobble (fragments of cobble in sample at 41 feet). Clayey sand appears to have pushed around cobble as sample driven. CLAYEY SAND (SC), brownish gray, medium dense, wet, medium plasticity. Blow count not representative due to cobble.			64	30	14	16	39	22.1		
45	-25		LEAN CLAY with SAND (CL), brownish gray, stiff to very stiff, medium plasticity. Sample disturbed and blow count not representative from being pushed around by what driller thinks is a cobble at bottom of hole. No recovery and blow count not representative. Driller advanced a pointed sampler down hole to push potential cobble out of the way of sampling.			54				86	44.1		
50	-30		Grades to hard, approximately 20-30% fine grained sand. SANDY LEAN CLAY (CL), brownish red, hard, white streaks throughout, approximately 30% fine grained sand.			50/4							
						39					30.3	>4.5 >4.5	

LOG - GEOTECHNICAL W/ELEV. 12487.000.000 J STREET BORING LOGS.GPJ ENGEO INC.GDT 11/13/15



# LOG OF BORING 1-B1

Geotechnical Exploration  
 2500 & 2508 J St. Yamanee Lofts  
 Sacramento, CA  
 12487.000.000

DATE DRILLED: 10/19/2015  
 HOLE DEPTH: 101.5 ft.  
 HOLE DIAMETER: 5.5 in.  
 SURF ELEV (NGVD29): Approx. 19½ ft.

LOGGED / REVIEWED BY: N. Broussard / MMG  
 DRILLING CONTRACTOR: GeoExploration  
 DRILLING METHOD: HSA/Mud Rotary  
 HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			SANDY LEAN CLAY (CL), brownish red, hard, white streaks throughout, approximately 30% fine grained sand.										
55	-35		Gray in top of sample at 55 feet, reddish at bottom. Upper 6 inches of sample is cuttings.			50/2				32.8	89.7	>4.5	
60	-40		SILTY SAND (SM), very dark gray, dense, wet, fine grained to poorly graded sand, approximately 20-30% fines.			72/11				39.3	82.3	>4.5	
			LEAN CLAY (CL), light gray, hard, low plasticity, silty, cemented.										
65	-45		Grades to more silty.			78/9				30.5	91.7	>4.5	
75	-55					75/11						>4.5	

LOG - GEOTECHNICAL W/ELEV. 12487.000.000 J STREET BORING LOGS.GPJ ENGEO INC.GDT 11/13/15



# LOG OF BORING 1-B1

Geotechnical Exploration  
2500 & 2508 J St. Yamanee Lofts  
Sacramento, CA  
12487.000.000

DATE DRILLED: 10/19/2015  
HOLE DEPTH: 101.5 ft.  
HOLE DIAMETER: 5.5 in.  
SURF ELEV (NGVD29): Approx. 19½ ft.

LOGGED / REVIEWED BY: N. Broussard / MMG  
DRILLING CONTRACTOR: GeoExploration  
DRILLING METHOD: HSA/Mud Rotary  
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
80	-60		LEAN CLAY (CL), light gray, hard, low plasticity, silty, cemented.										
85	-65		POORLY GRADED SAND (SP), dark gray, very dense, fine grained.			71						>4.5	
90	-70		Driller noted drilling behaviour consistent with the hard lean clay at 90 feet. LEAN CLAY (CL), light gray, hard, low plasticity										
95	-75												
100	-80					80/10						>4.5	
			Bottom of boring at approximately 101 1/2 feet below ground surface. Groundwater encountered at time of drilling at 19.6 feet below existing ground surface.										

LOG - GEOTECHNICAL W/ELEV. 12487.000.000 J STREET BORING LOGS.GPJ ENGEO INC.GDT 11/13/15

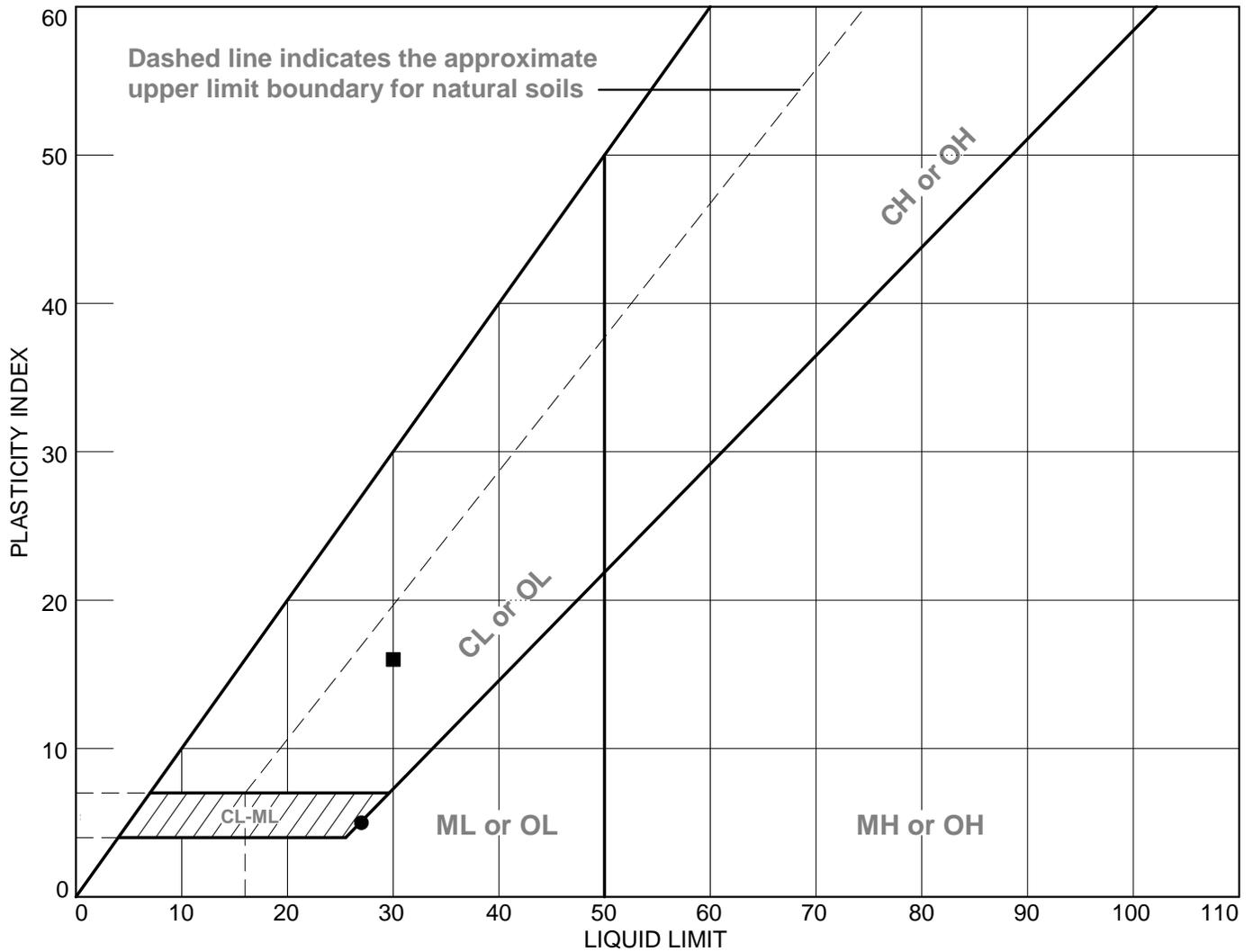
**APPENDIX B**

**LABORATORY TEST DATA**

**Liquid and Plastic Limits Test Report  
Particle Size Distribution Report (7 pages)  
Unconfined Compression Test  
R-Value Test Report  
Incremental Consolidation  
Water Soluble Sulfates in Soils  
Analytical Results of Soil Corrosion**



# LIQUID AND PLASTIC LIMITS TEST REPORT



## SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	GEX	B1@9.5	9.5 feet	21.9	22	27	5	
■	GEX	B1@41	41 feet	22.1	14	30	16	



**Client:** 2500 J Street Owner, LLC

**Project:** 2500 - 2508 J Street Yamanee Lofts

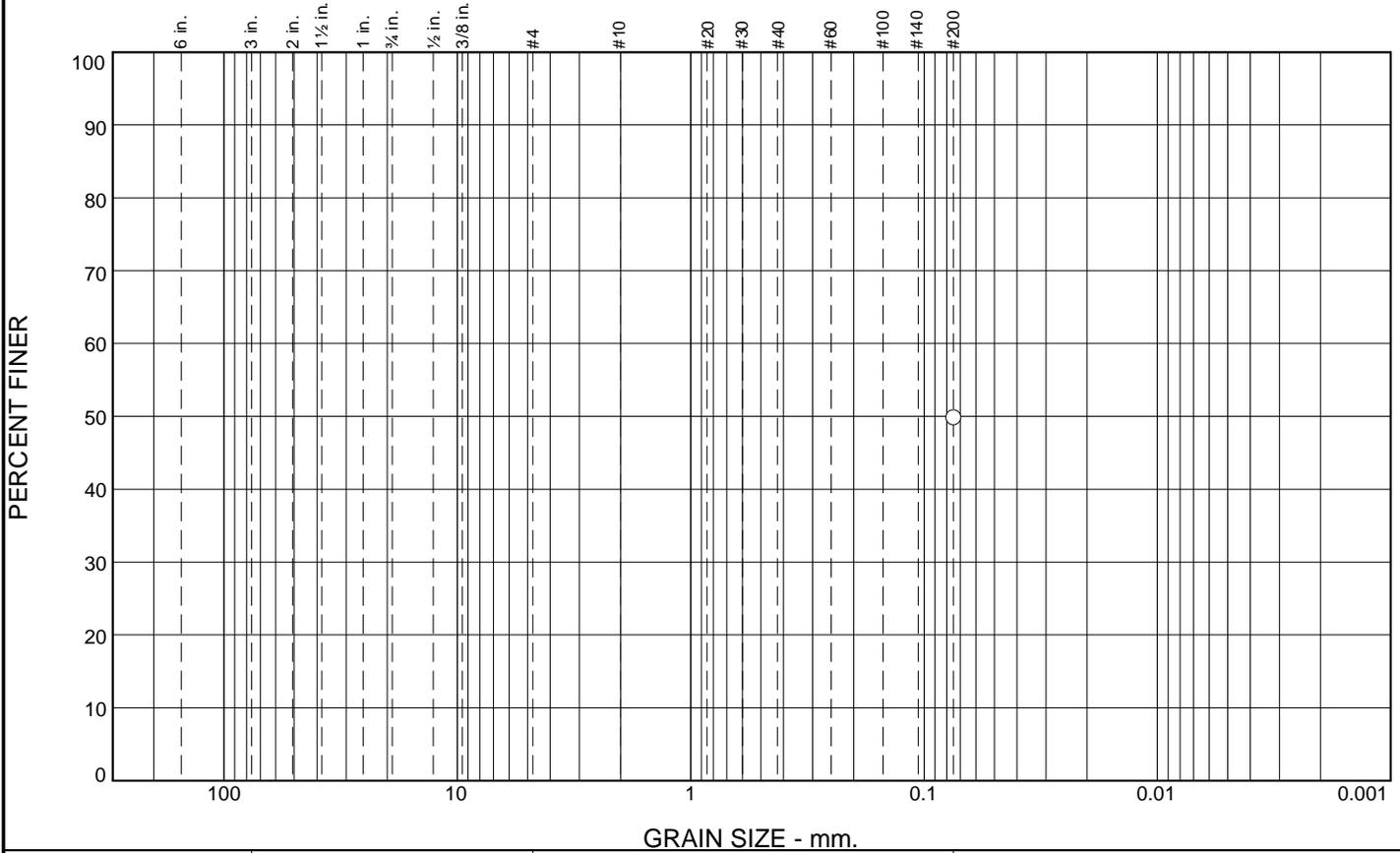
**Project No.:** 12487.000.000 PH001

**Figure**

**Tested By:** R. Montalvo

**Checked By:** M. Gilbert

# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						49.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	49.9		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL=                      LL=                      PI=

**Coefficients**

D<sub>90</sub>=                      D<sub>85</sub>=                      D<sub>60</sub>=  
D<sub>50</sub>=                      D<sub>30</sub>=                      D<sub>15</sub>=  
D<sub>10</sub>=                      C<sub>u</sub>=                      C<sub>c</sub>=

**Classification**

USCS=                      AASHTO=

**Remarks**

\* (no specification provided)

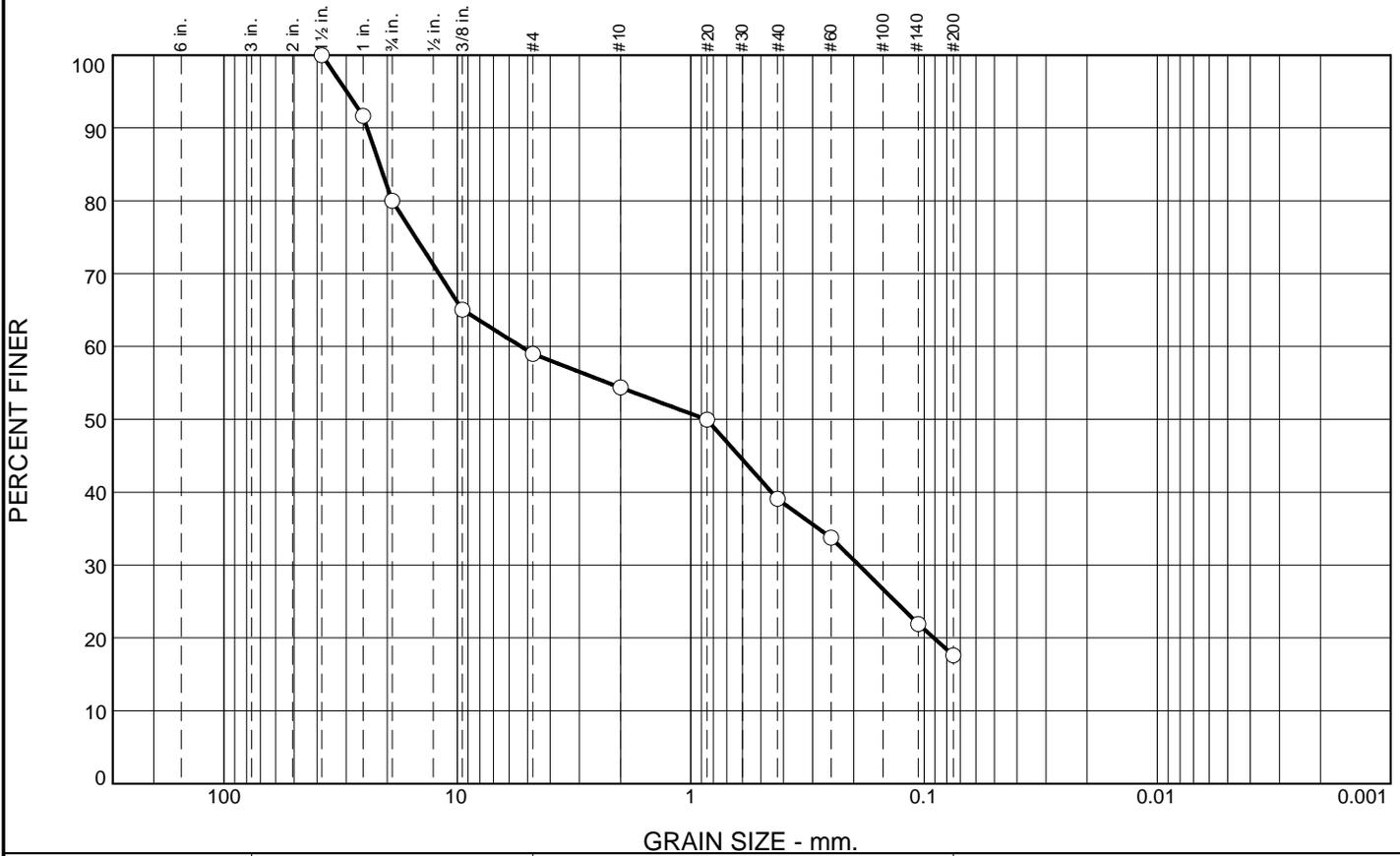
**Source of Sample:** GEX                      **Depth:** 14.5 feet  
**Sample Number:** B1@14.5

**Date:** 11-06-2015

	<p><b>Client:</b> 2500 J Street Owner, LLC  <b>Project:</b> 2500 - 2508 J Street Yamanee Lofts  <b>Project No:</b> 12487.000.000 PH001</p>
	<b>Figure</b>

**Tested By:** R. Montalvo                      **Checked By:** M. Gilbert

# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	20.0	21.0	4.6	15.3	21.5	17.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1"	91.7		
3/4"	80.0		
3/8"	65.0		
#4	59.0		
#10	54.4		
#20	49.9		
#40	39.1		
#60	33.7		
#140	21.9		
#200	17.6		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL=                      LL=                      PI=

**Coefficients**

D<sub>90</sub>= 24.3771      D<sub>85</sub>= 21.5462      D<sub>60</sub>= 5.3379  
D<sub>50</sub>= 0.8588      D<sub>30</sub>= 0.1905      D<sub>15</sub>=  
D<sub>10</sub>=                      C<sub>u</sub>=                      C<sub>c</sub>=

**Classification**

USCS=                      AASHTO=

**Remarks**

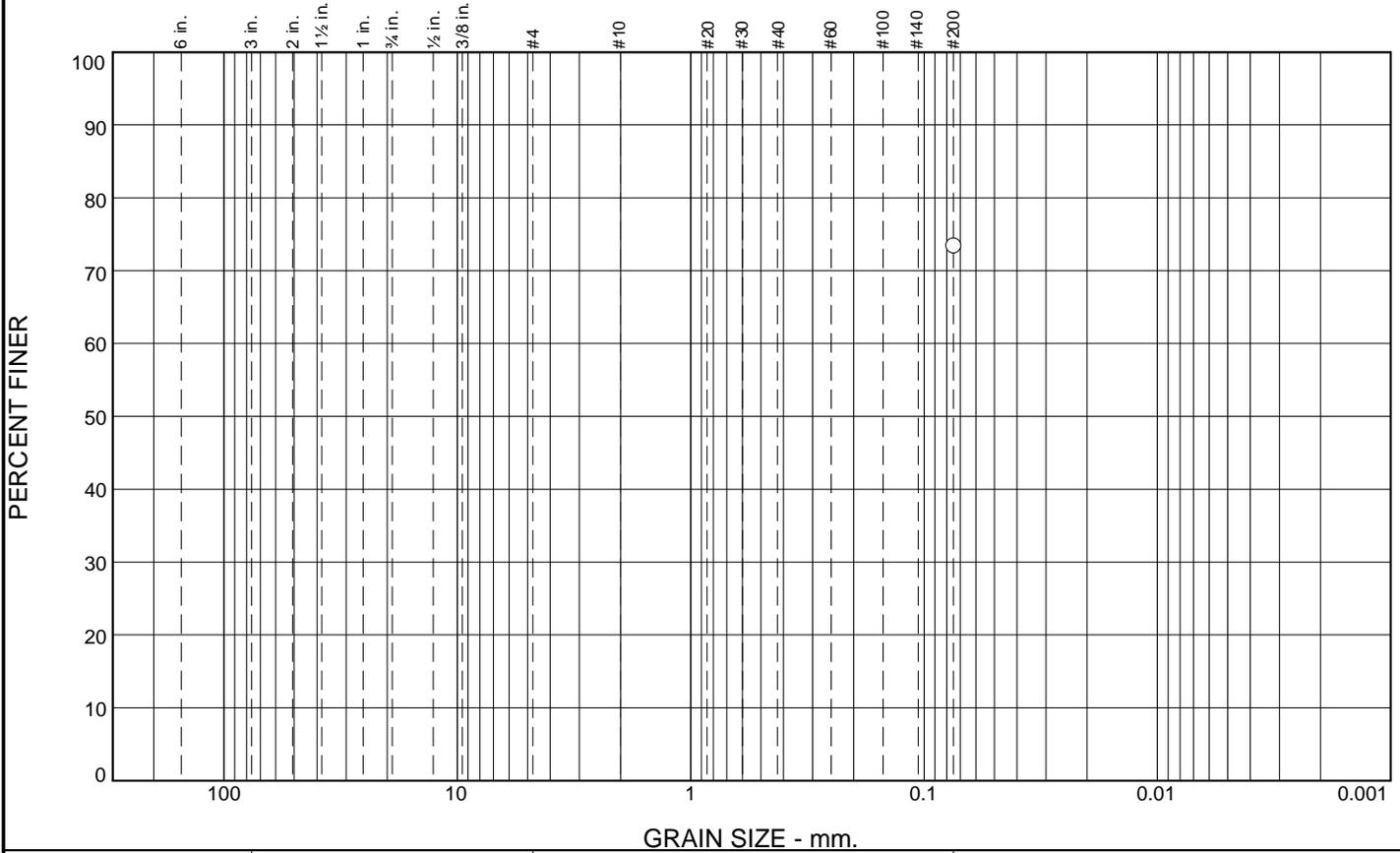
\* (no specification provided)

**Source of Sample:** GEX                      **Depth:** 19 feet                      **Date:** 11-06-2015  
**Sample Number:** B1@19

	<b>Client:</b> 2500 J Street Owner, LLC <b>Project:</b> 2500 - 2508 J Street Yamanee Lofts  <b>Project No:</b> 12487.000.000 PH001	<b>Figure</b>
--	---	---------------

**Tested By:** R. Montalvo                      **Checked By:** M. Gilbert

# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						73.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	73.5		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL=                      LL=                      PI=

**Coefficients**

D<sub>90</sub>=                      D<sub>85</sub>=                      D<sub>60</sub>=  
D<sub>50</sub>=                      D<sub>30</sub>=                      D<sub>15</sub>=  
D<sub>10</sub>=                      C<sub>u</sub>=                      C<sub>c</sub>=

**Classification**

USCS=                      AASHTO=

**Remarks**

\* (no specification provided)

**Source of Sample:** GEX                      **Depth:** 2 feet  
**Sample Number:** B1@2

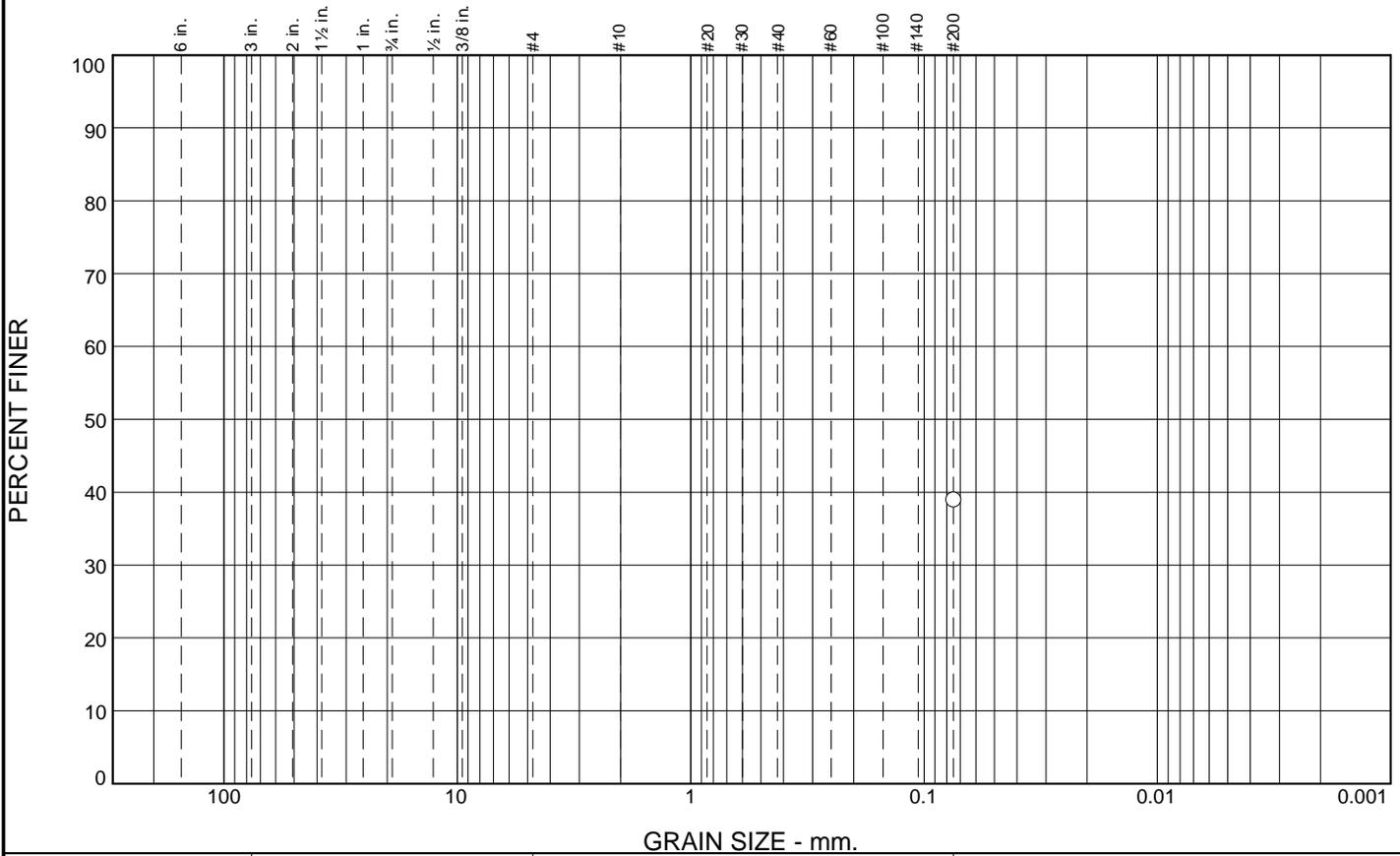
**Date:** 11-06-2015

	<p><b>Client:</b> 2500 J Street Owner, LLC</p> <p><b>Project:</b> 2500 - 2508 J Street Yamanee Lofts</p> <p><b>Project No:</b> 12487.000.000 PH001</p>	<p><b>Figure</b></p>
--	--	----------------------

**Tested By:** R. Montalvo                      **Checked By:** M. Gilbert



# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						39.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	39.0		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL= 14      LL= 30      PI= 16

**Coefficients**

D<sub>90</sub>=      D<sub>85</sub>=      D<sub>60</sub>=  
D<sub>50</sub>=      D<sub>30</sub>=      D<sub>15</sub>=  
D<sub>10</sub>=      C<sub>u</sub>=      C<sub>c</sub>=

**Classification**

USCS=      AASHTO=

**Remarks**

\* (no specification provided)

**Source of Sample:** GEX  
**Sample Number:** B1@41

**Depth:** 41 feet

**Date:** 11-06-2015



**Client:** 2500 J Street Owner, LLC  
**Project:** 2500 - 2508 J Street Yamanee Lofts

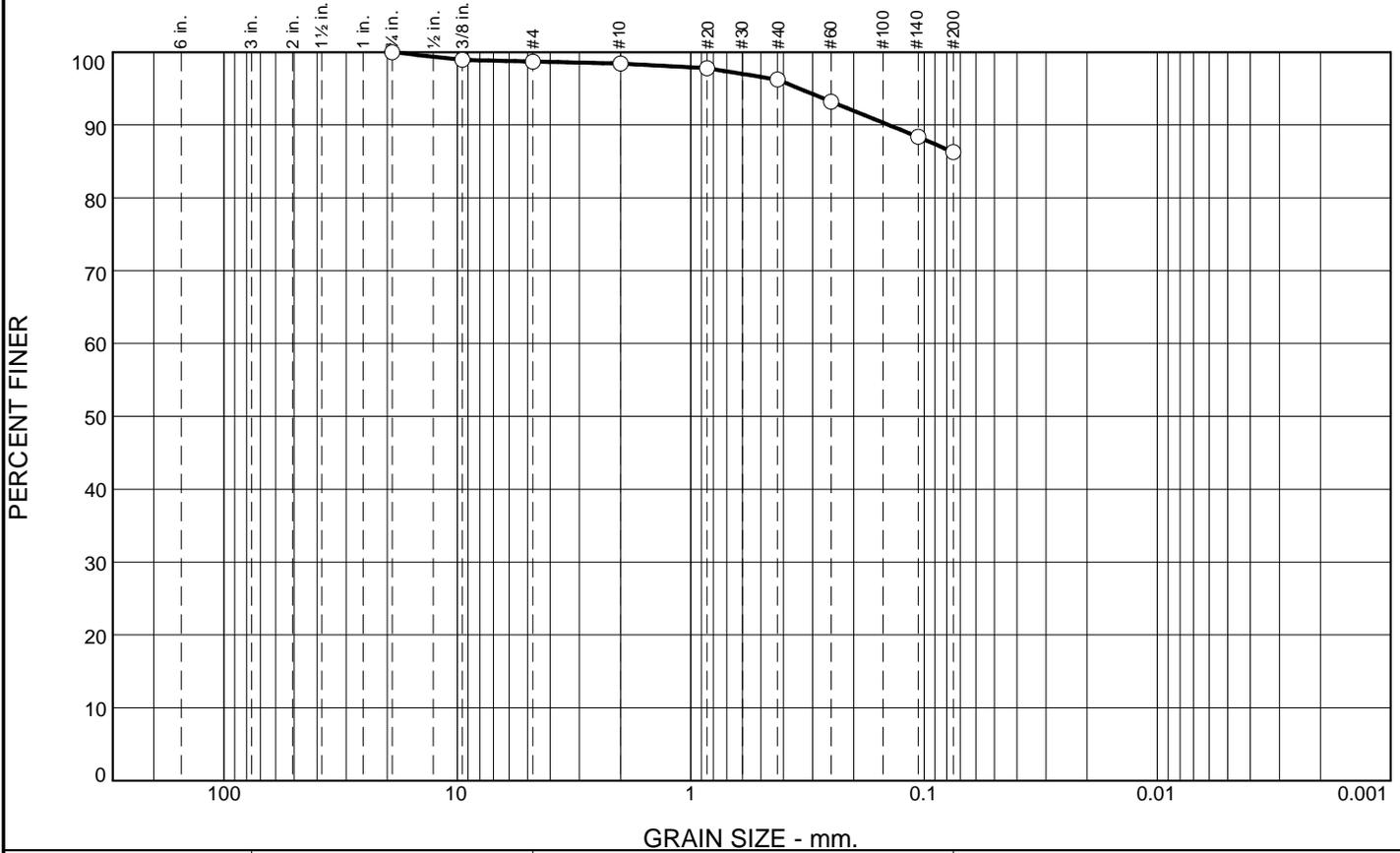
**Project No:** 12487.000.000 PH001

**Figure**

**Tested By:** R. Montalvo

**Checked By:** M. Gilbert

# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.3	0.3	2.2	9.9	86.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	98.9		
#4	98.7		
#10	98.4		
#20	97.8		
#40	96.2		
#60	93.2		
#140	88.4		
#200	86.3		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL=                      LL=                      PI=

**Coefficients**

D<sub>90</sub>= 0.1417      D<sub>85</sub>=                      D<sub>60</sub>=

D<sub>50</sub>=                      D<sub>30</sub>=                      D<sub>15</sub>=

D<sub>10</sub>=                      C<sub>u</sub>=                      C<sub>c</sub>=

**Classification**

USCS=                      AASHTO=

**Remarks**

\* (no specification provided)

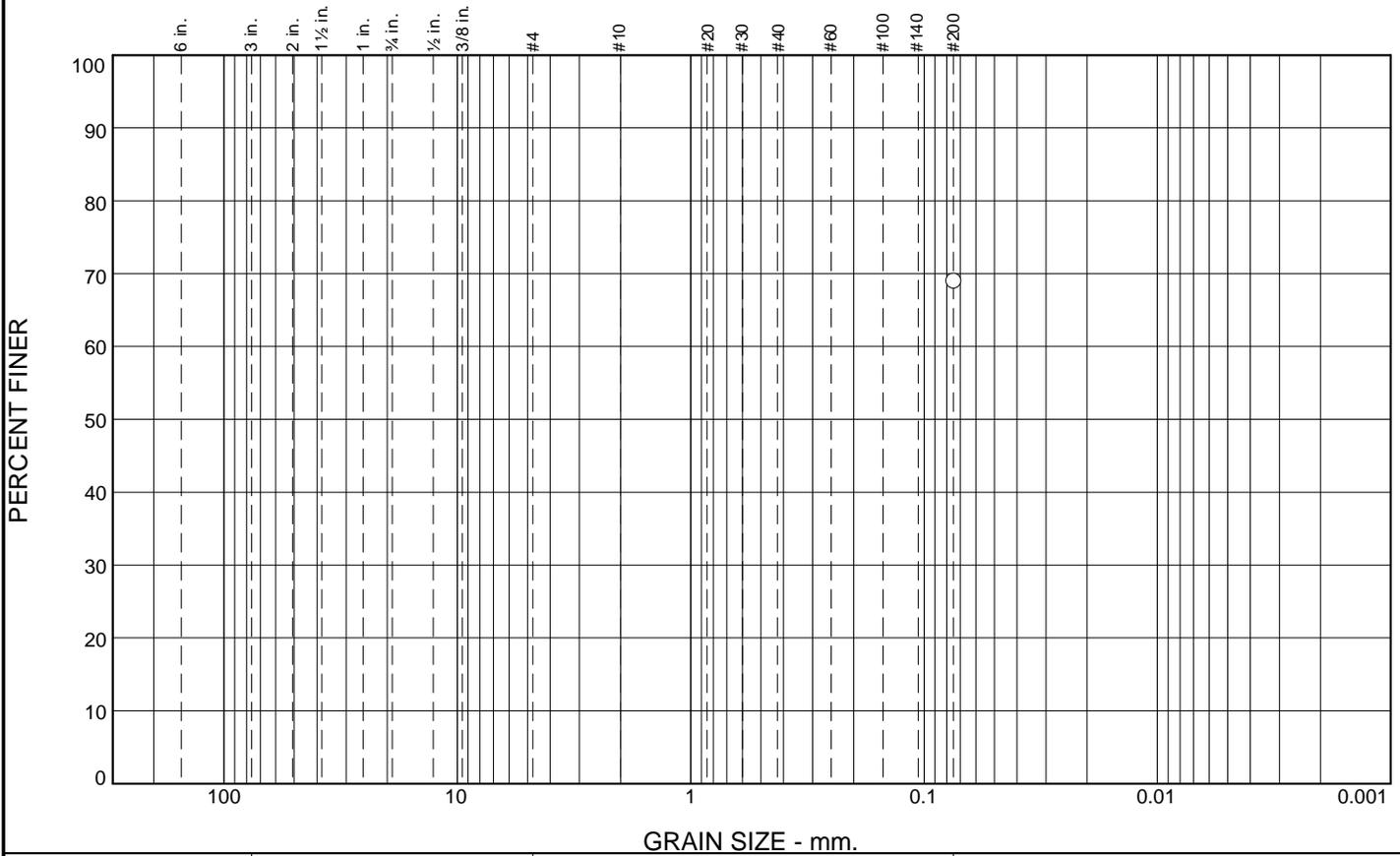
**Source of Sample:** GEX                      **Depth:** 46 feet  
**Sample Number:** B1@46

**Date:** 11-06-2015

	<p><b>Client:</b> 2500 J Street Owner, LLC</p> <p><b>Project:</b> 2500 - 2508 J Street Yamanee Lofts</p> <p><b>Project No:</b> 12487.000.000 PH001</p>
<p><b>Figure</b></p>	

**Tested By:** R. Montalvo                      **Checked By:** M. Gilbert

# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						69.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	69.0		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL= 22      LL= 27      PI= 5

**Coefficients**

D<sub>90</sub>=      D<sub>85</sub>=      D<sub>60</sub>=  
D<sub>50</sub>=      D<sub>30</sub>=      D<sub>15</sub>=  
D<sub>10</sub>=      C<sub>u</sub>=      C<sub>c</sub>=

**Classification**

USCS=      AASHTO=

**Remarks**

\* (no specification provided)

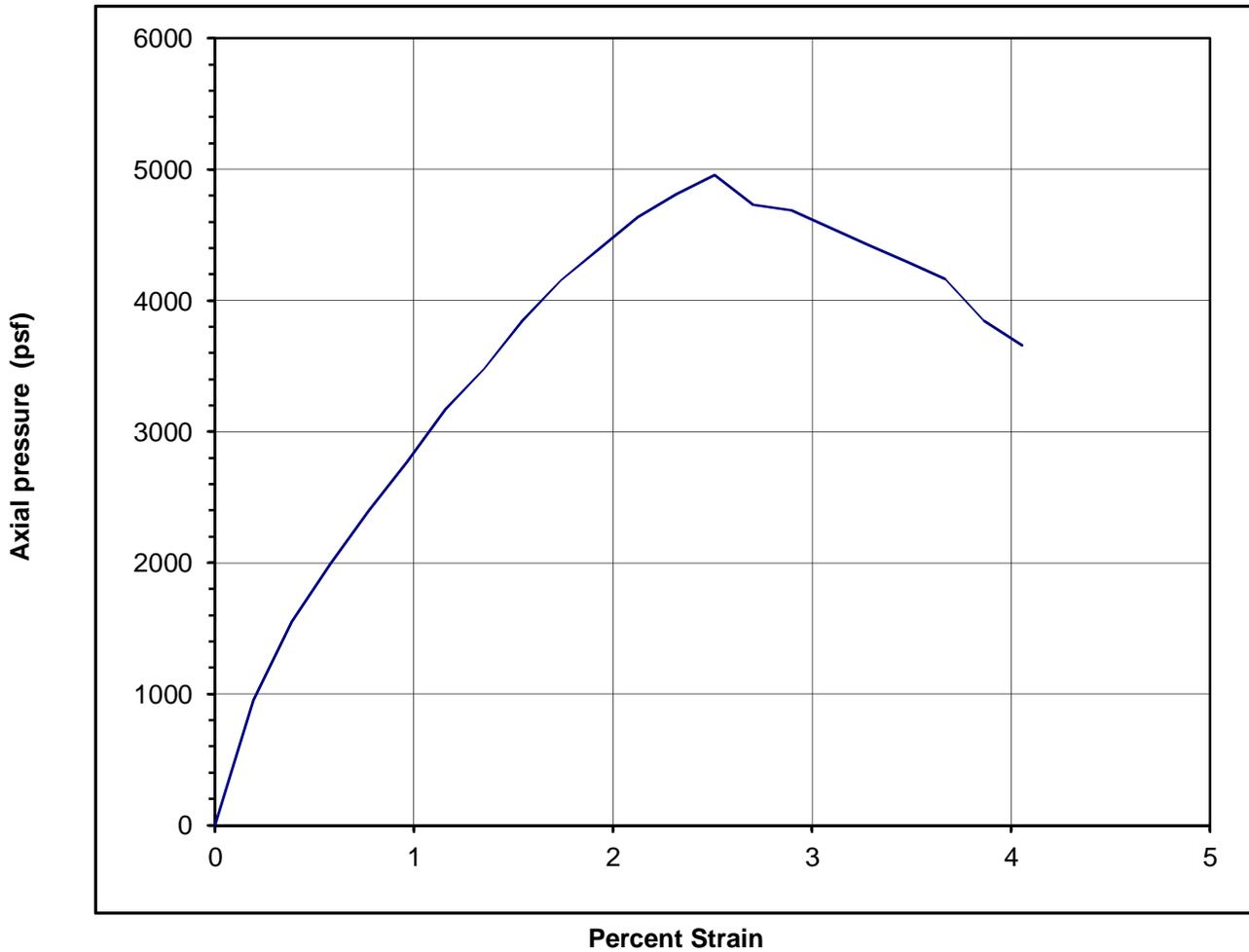
**Source of Sample:** GEX      **Depth:** 9.5 feet  
**Sample Number:** B1@9.5

**Date:** 11-06-2015

	<p><b>Client:</b> 2500 J Street Owner, LLC  <b>Project:</b> 2500 - 2508 J Street Yamanee Lofts</p> <p><b>Project No:</b> 12487.000.000 PH001</p>
	<b>Figure</b>

**Tested By:** R. Montalvo      **Checked By:** M. Gilbert

**Unconfined Compression Test  
ASTM Test Method D2166**



**Unconfined Compressive Strength: 4950 psf      2.5 tsf**

**Sample Description: (See Exploration Log)**

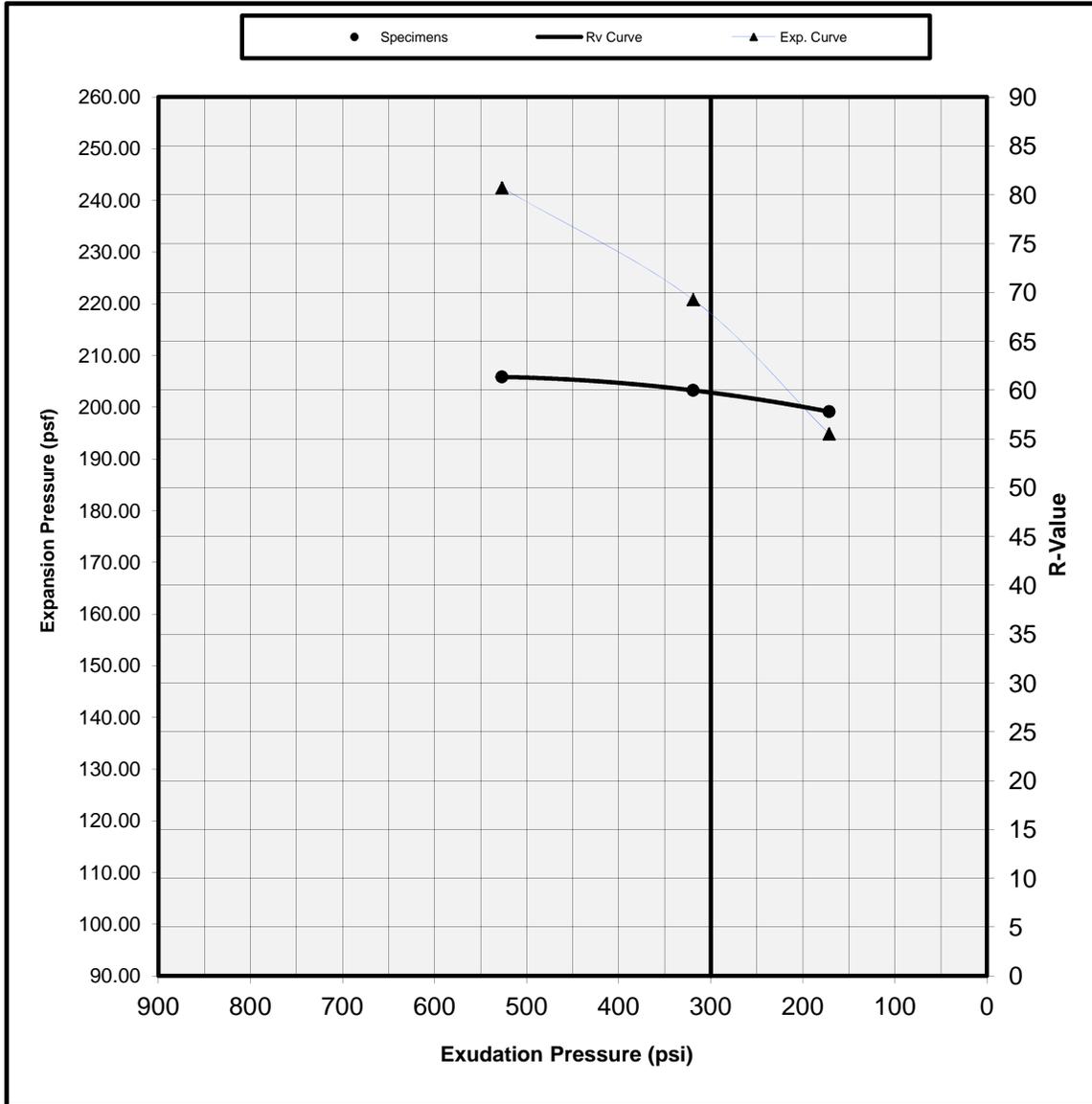
<b>Initial Diameter:</b>	<b>2.40 in.</b>	<b>Sample Number:</b>	<b>B1@65.5</b>
<b>Initial Height:</b>	<b>5.18 in.</b>	<b>Boring Number:</b>	<b>B1</b>
<b>Strain Rate:</b>	<b>0.96 %/min</b>	<b>Dry Unit Weight:</b>	<b>91.7 pcf</b>
<b>Total Strain:</b>	<b>4.05 %</b>	<b>Moisture Content:</b>	<b>30.5 %</b>
		<b>Depth of Sample:</b>	<b>65.5 ft.</b>

**ENGEO**  
INCORPORATED

**2500-2508 J Street Condo Building  
Sacramento, CA**

<b>Job No.:</b>	<b>12487.000.000</b>
<b>Sample Number:</b>	<b>B1 @65.5</b>
<b>Date:</b>	<b>11/6/2015</b>

Figure No.



Date: 11/13/15

Project Name: 2500 - 2508 J Street Condo Building

Project Number: 12487.000.000 PH001

Sample Location: S1

Description: Dark brown silty SAND

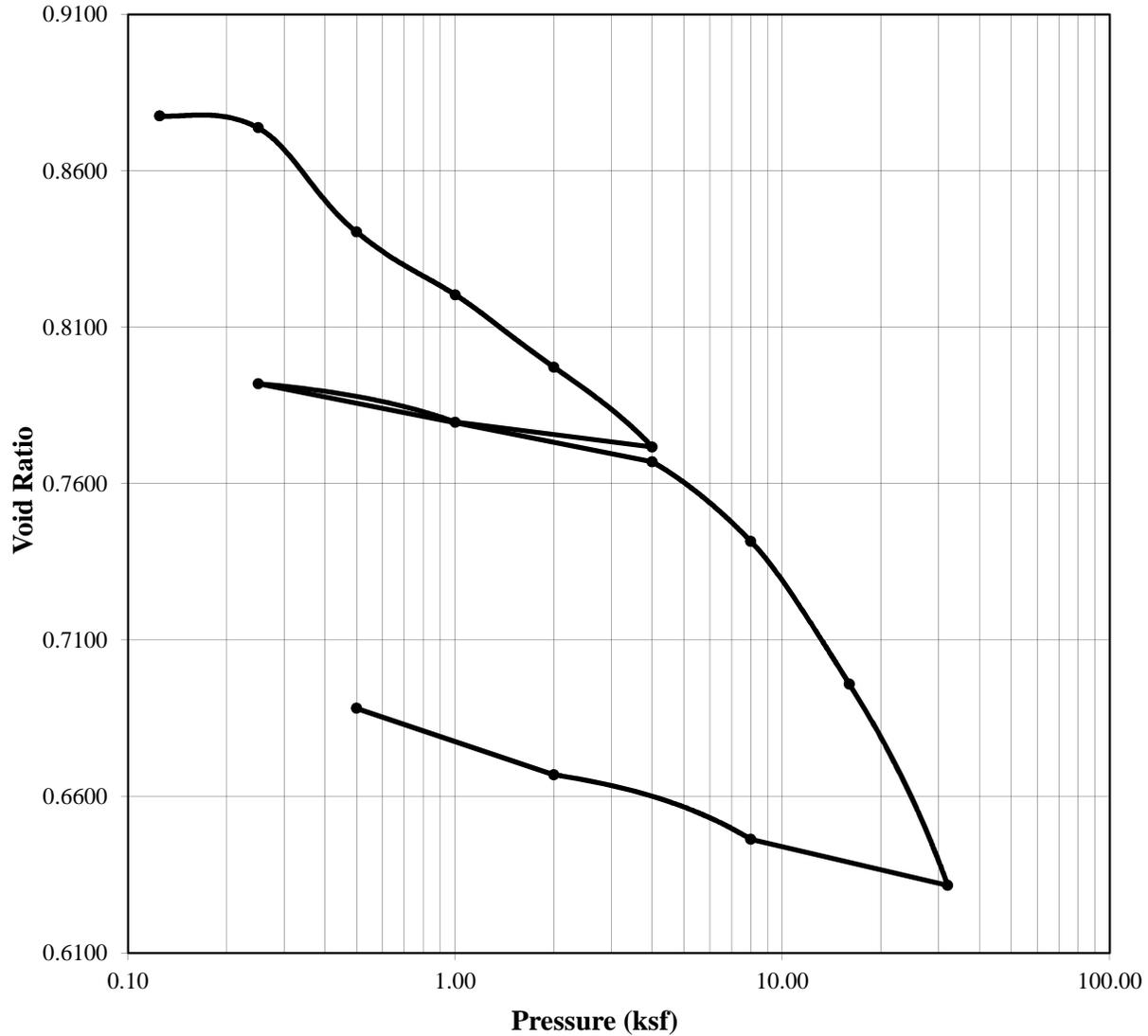
Test Performed By: R.Montalvo

Reviewed By: M. Gilbert

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	527	319	171
Expansion dial (0.0001")	56	51	45
Expansion Pressure (p.s.f.)	242	221	195
Resistance Value, "R"	61	60	58
% Moisture at Test	15.6	16.4	17.6
Dry Density at Test, p.c.f.	105.0	104.3	103.6
"R" Value at Exudation Pressure of 300 psi.	60		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	218		

# Incremental Consolidation

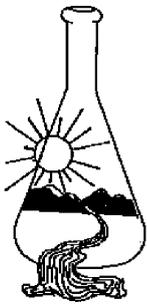
## ASTM D2435 Method B



	<b>Before</b>	<b>After</b>	<u>ASTM D4318 - Wet Method</u>	<b>Test Date:</b> 10-28-15
<b>Moisture (%):</b>	32.84	26.78	<b>Liquid Limit:</b>	
<b>Dry Density (pcf):</b>	89.69	97.92	<b>Plastic Limit:</b>	
<b>Saturation (%):</b>	100.83	100.24	<u>ASTM D854 - Assumed</u>	
<b>Void Ratio:</b>	0.8776	0.6896	<b>Specific Gravity:</b>	<b>2.700</b>
<b>Soil Description:</b>	See exploration logs		<b>Remarks:</b>	
<b>Project Number:</b>	12487.000.000		<b>Depth:</b>	55.5
<b>Sample Number:</b>	1-B1 @ 55.5		<b>Boring #:</b>	1-B1
<b>Project Name:</b>	2500 & 2508 J Street, Yamanee Lofts			
<b>Client:</b>	2500 J Street Owners, LLC			
<b>Location:</b>	1-B1			
<b>Tested By:</b> K. Lecce	<b>Checked By:</b> D. Seibold			







**Sunland Analytical**  
11419 Sunrise Gold Cir.#10  
Rancho Cordova, CA 95742  
(916) 852-8557

Date Reported 10/30/15  
Date Submitted 10/26/15

To: Nick Broussard  
Engeo, Inc.  
2213 Plaza Dr.  
Rocklin, CA, 95765

From: Gene Oliphant, Ph.D. \ Randy Horney  
General Manager \ Lab Manager

The reported analysis was requested for the following:  
Location : 12487 Site ID: B1@4FT  
Thank you for your business.

\* For future reference to this analysis please use SUN # 70725 - 147607

---

EVALUATION FOR SOIL CORROSION

Soil pH	7.60		
Minimum Resistivity	4.42	ohm-cm (x1000)	
Chloride	7.5 ppm	0.0008	%
Sulfate-S	0.7 ppm	0.0001	%

**METHODS:**

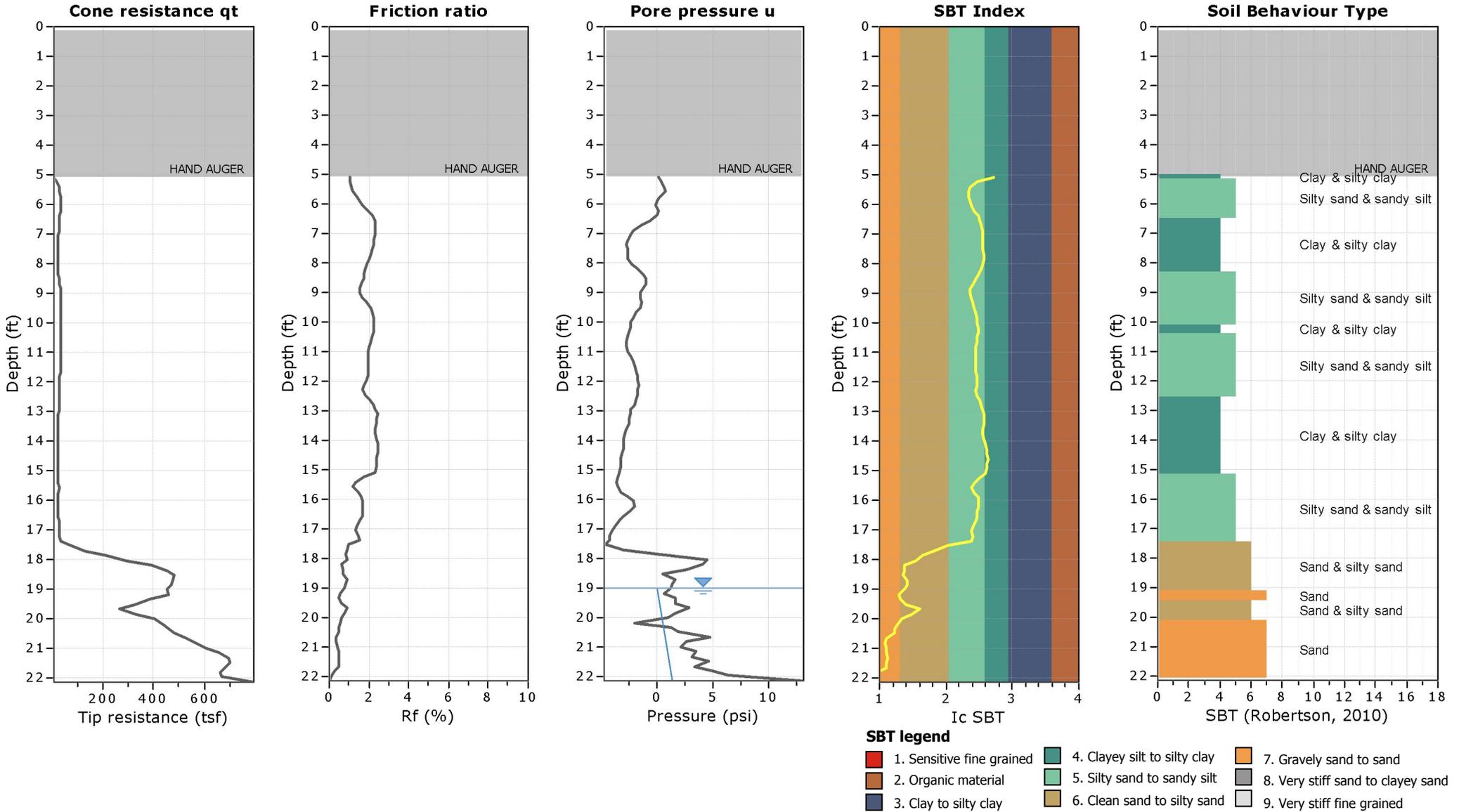
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)  
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

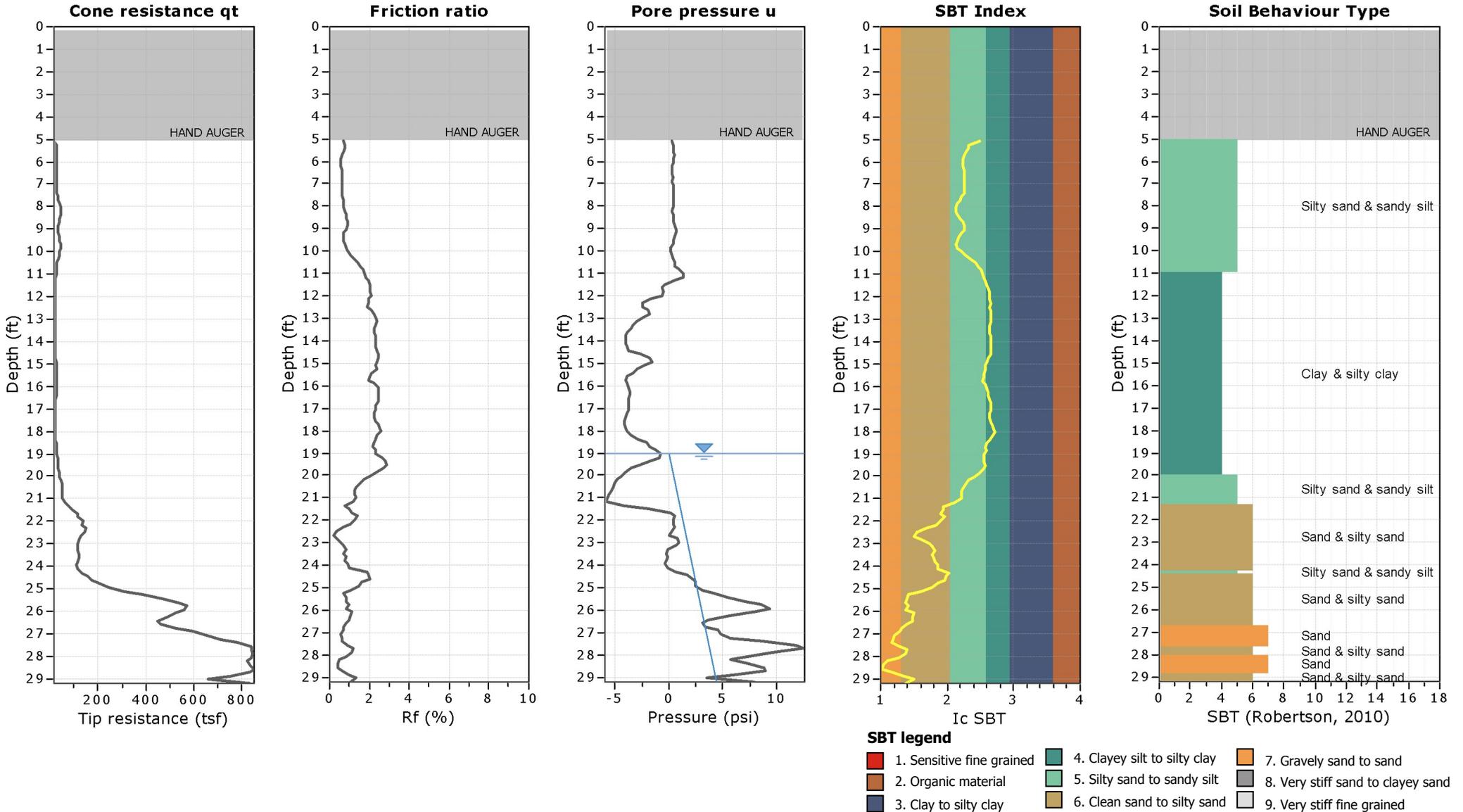
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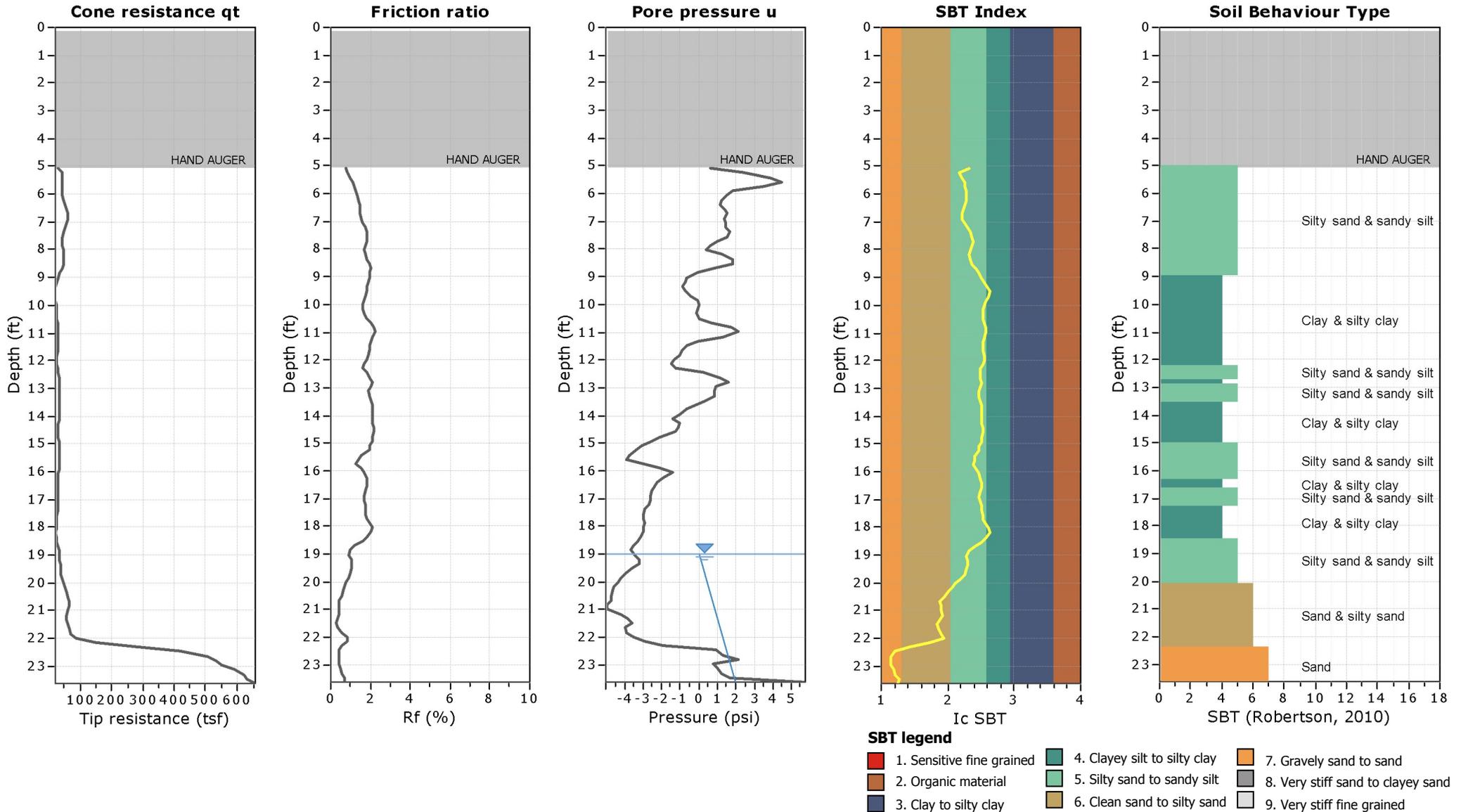
**APPENDIX C**

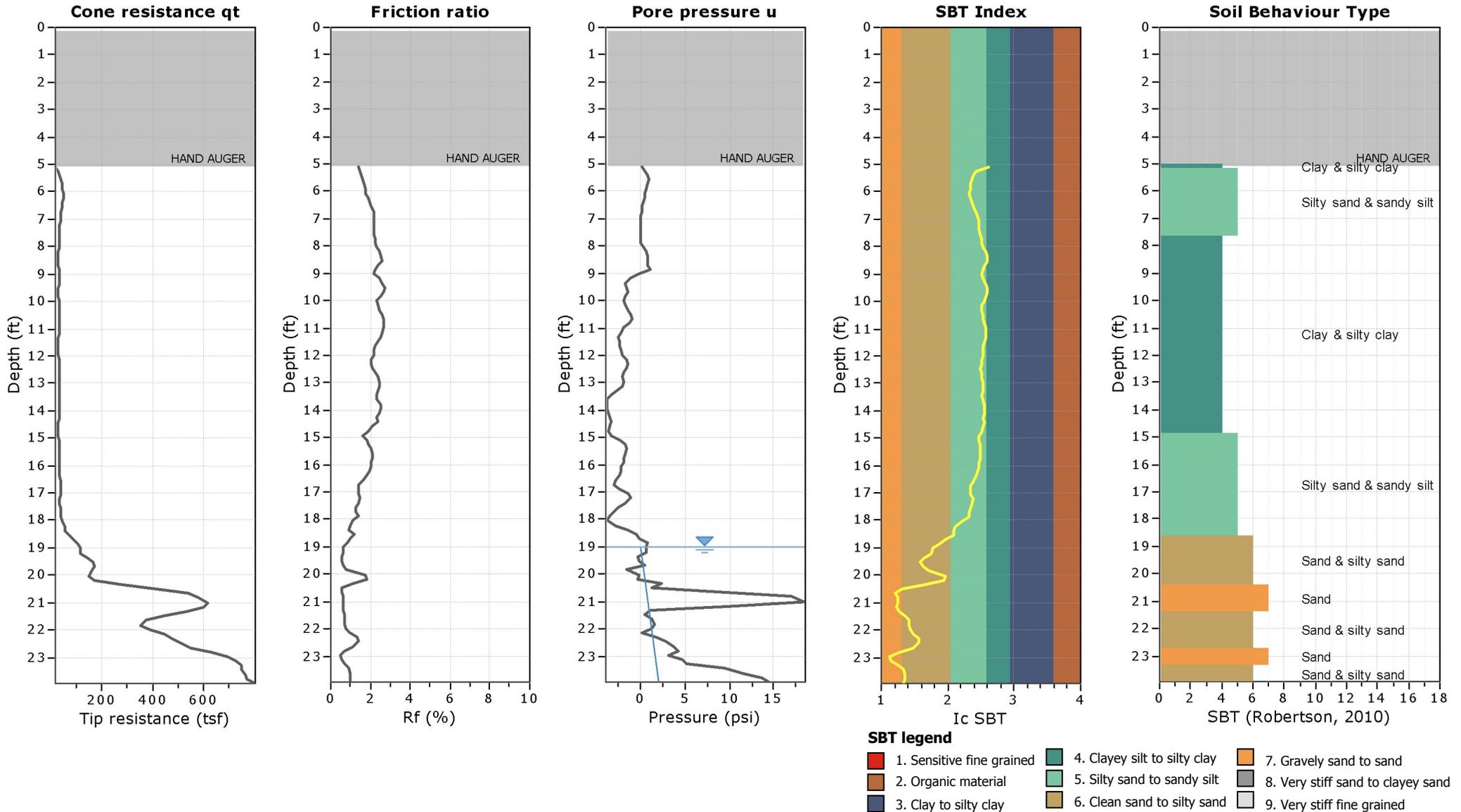
**CPT Data**

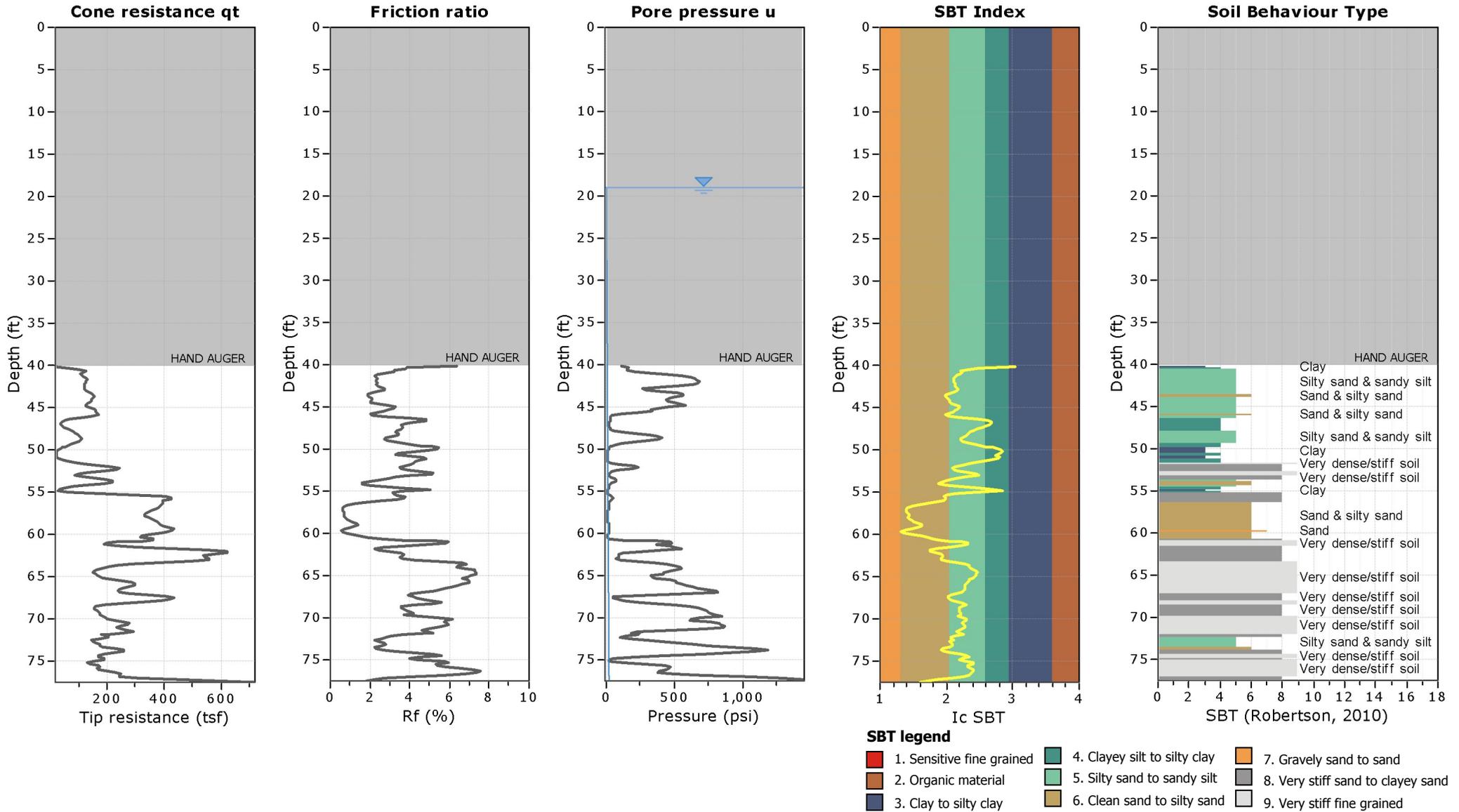










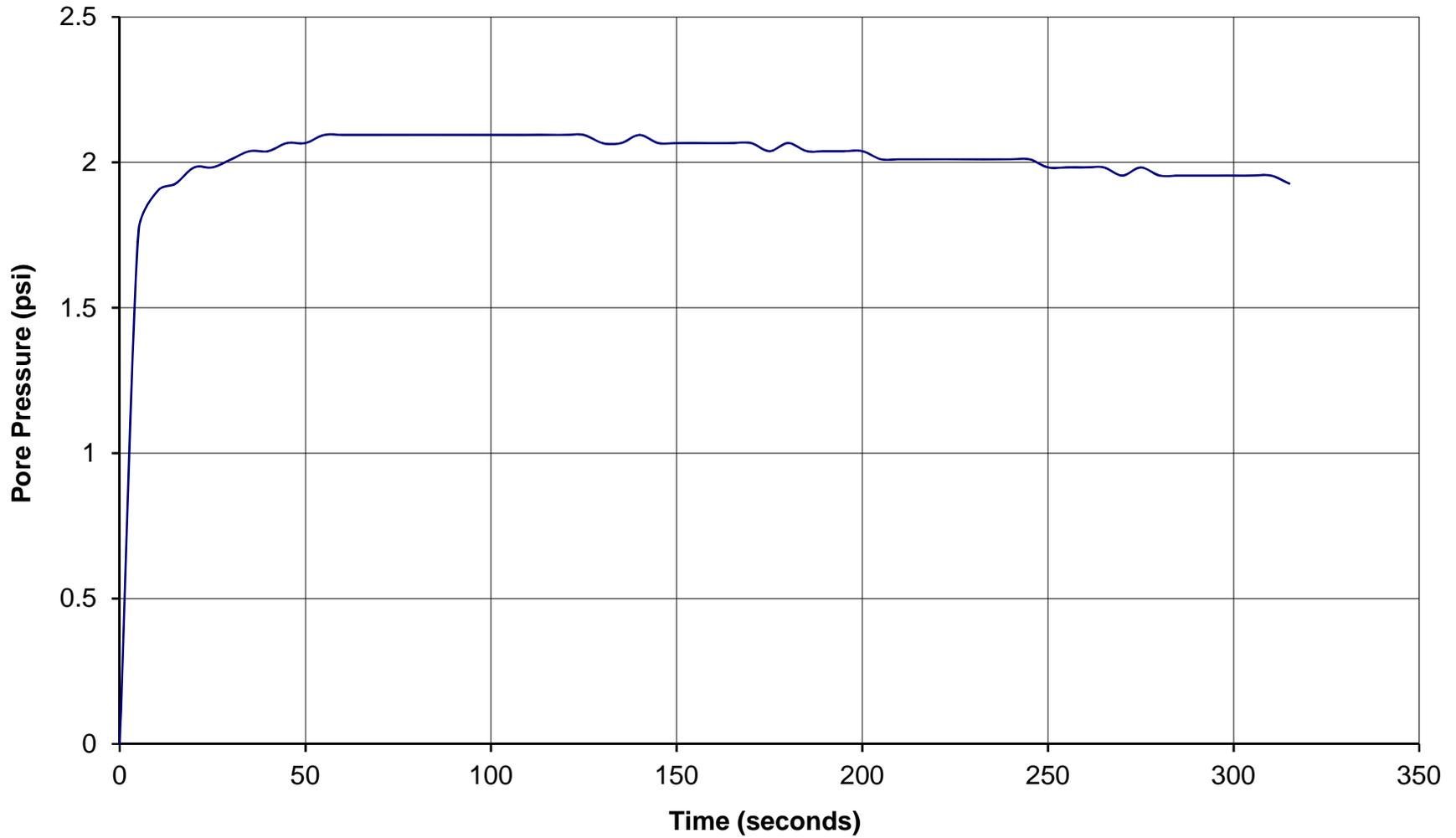




# GREGG DRILLING & TESTING

## Pore Pressure Dissipation Test

Sounding: 1cpt2  
Depth: 25.0983495  
Site: Yamanee Lofts  
Engineer: N. Broussard





# Shear Wave Velocity Calculations

Yamane Lofts Proj.

1-CPT-4

Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

10/19/15

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)
10.01	9.35	9.49	9.49	16.2000			
14.93	14.27	14.37	4.87	23.7000	7.5000	649.4	11.81
20.01	19.35	19.42	5.06	31.0500	7.3500	688.4	16.81
23.95	23.29	23.35	3.92	34.2500	3.2000	1226.5	21.32

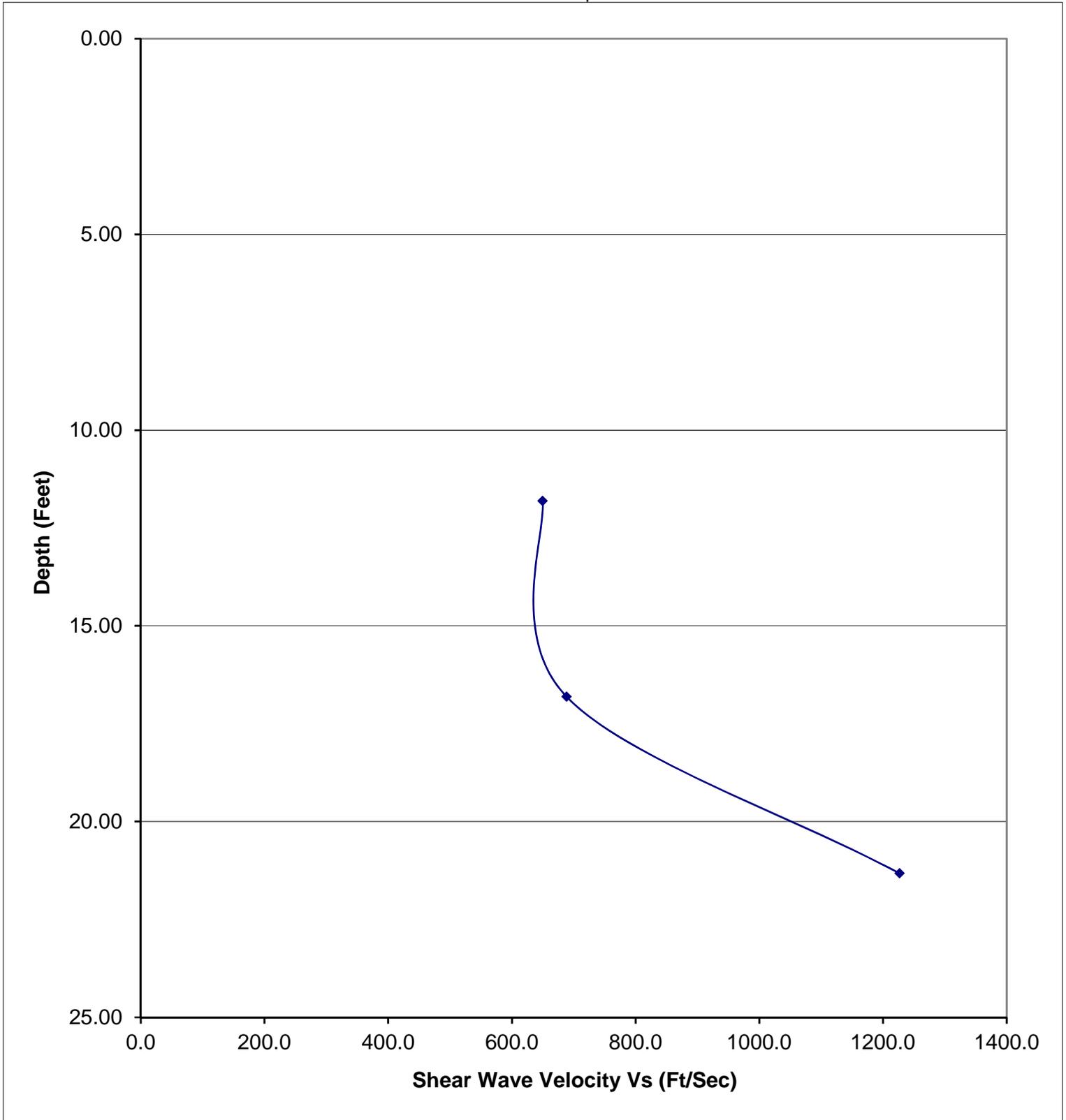


# Shear Wave Velocity Profile

Yamane Lofts Proj.

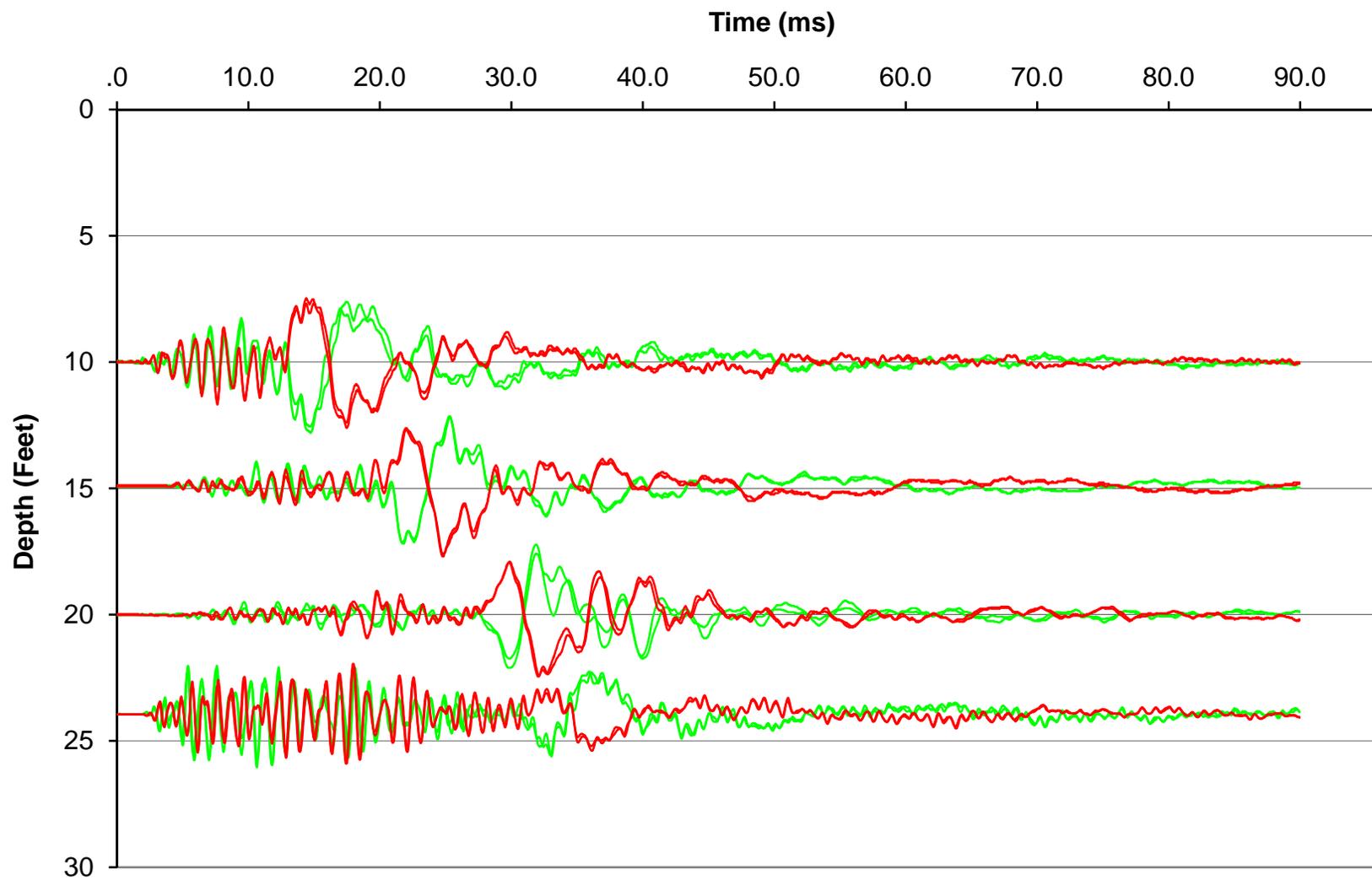
15-193MA

s1cpt4





## Waveforms for Sounding s1cpt4





# Shear Wave Velocity Calculations

Yamane Lofts Proj.

1-CPT-4A

Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

10/20/15

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)
45.11	44.45	44.48	44.48	53.2500			
50.03	49.37	49.40	4.92	57.5000	4.2500	1157.2	46.91
55.12	54.46	54.48	5.08	61.3000	3.8000	1337.5	51.92
60.04	59.38	59.40	4.92	63.5500	2.2500	2186.3	56.92
65.12	64.46	64.49	5.08	66.0500	2.5000	2033.4	61.92
70.05	69.39	69.41	4.92	68.3000	2.2500	2186.5	66.93
75.13	74.47	74.49	5.08	69.5000	1.2000	4236.6	71.93

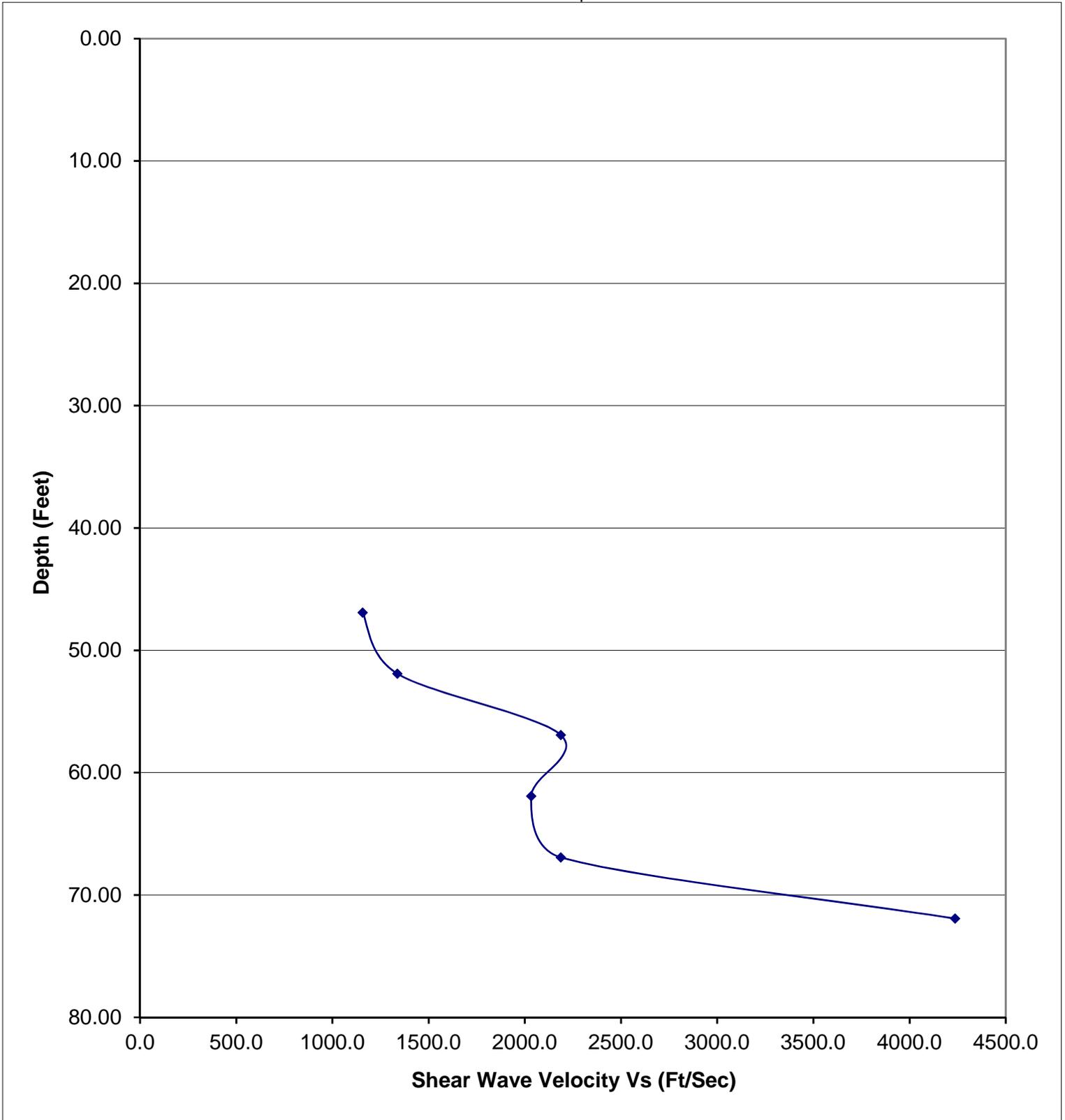


# Shear Wave Velocity Profile

Yamane Lofts Proj.

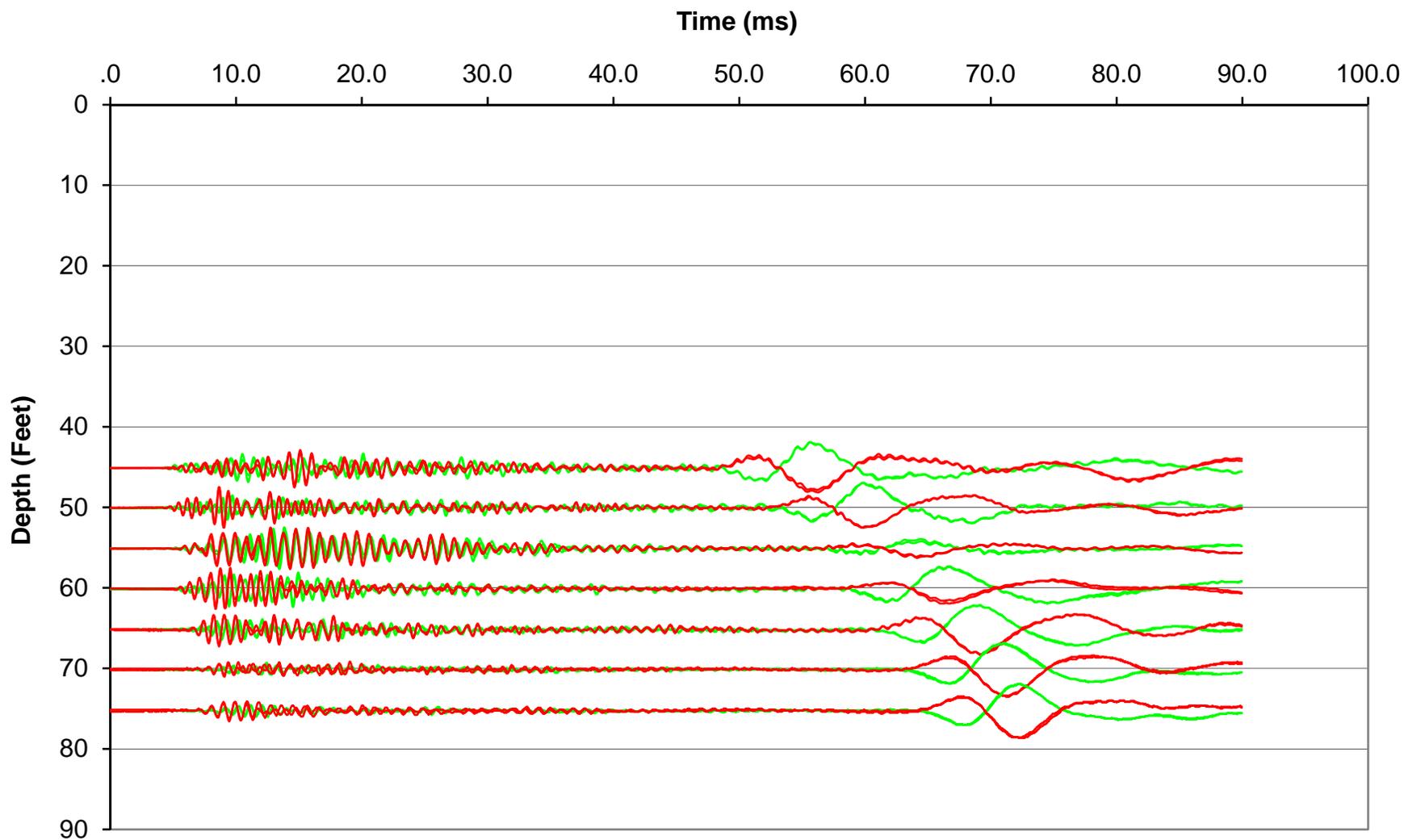
15-193MA

s1cpt4a





## Waveforms for Sounding s1cpt4a



**A  
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**APPENDIX D**  
**Liquefaction Analyses**



## LIQUEFACTION ANALYSIS REPORT

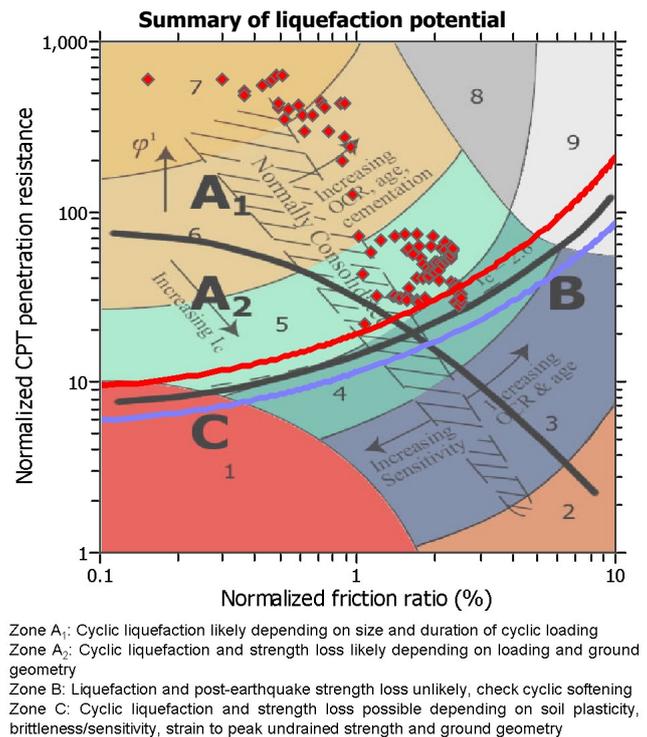
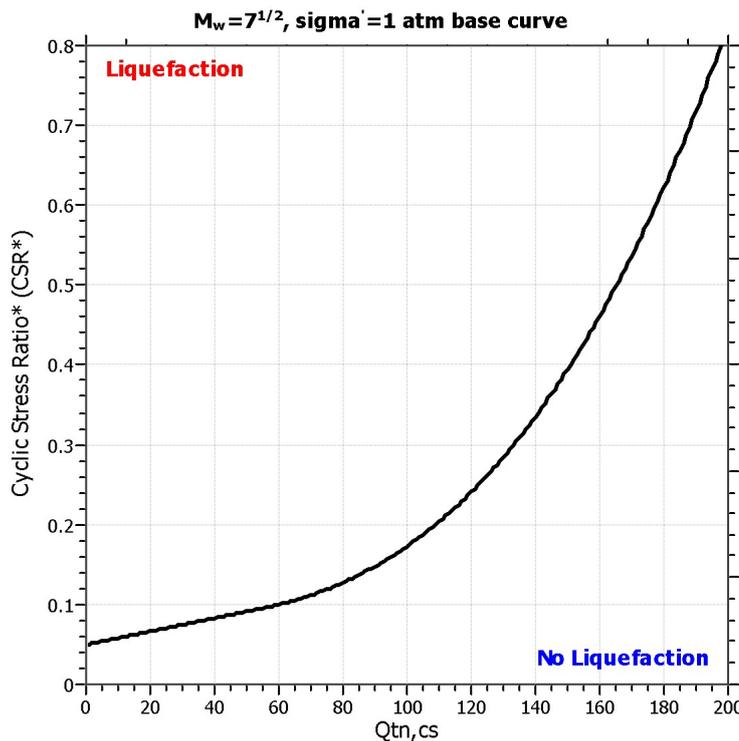
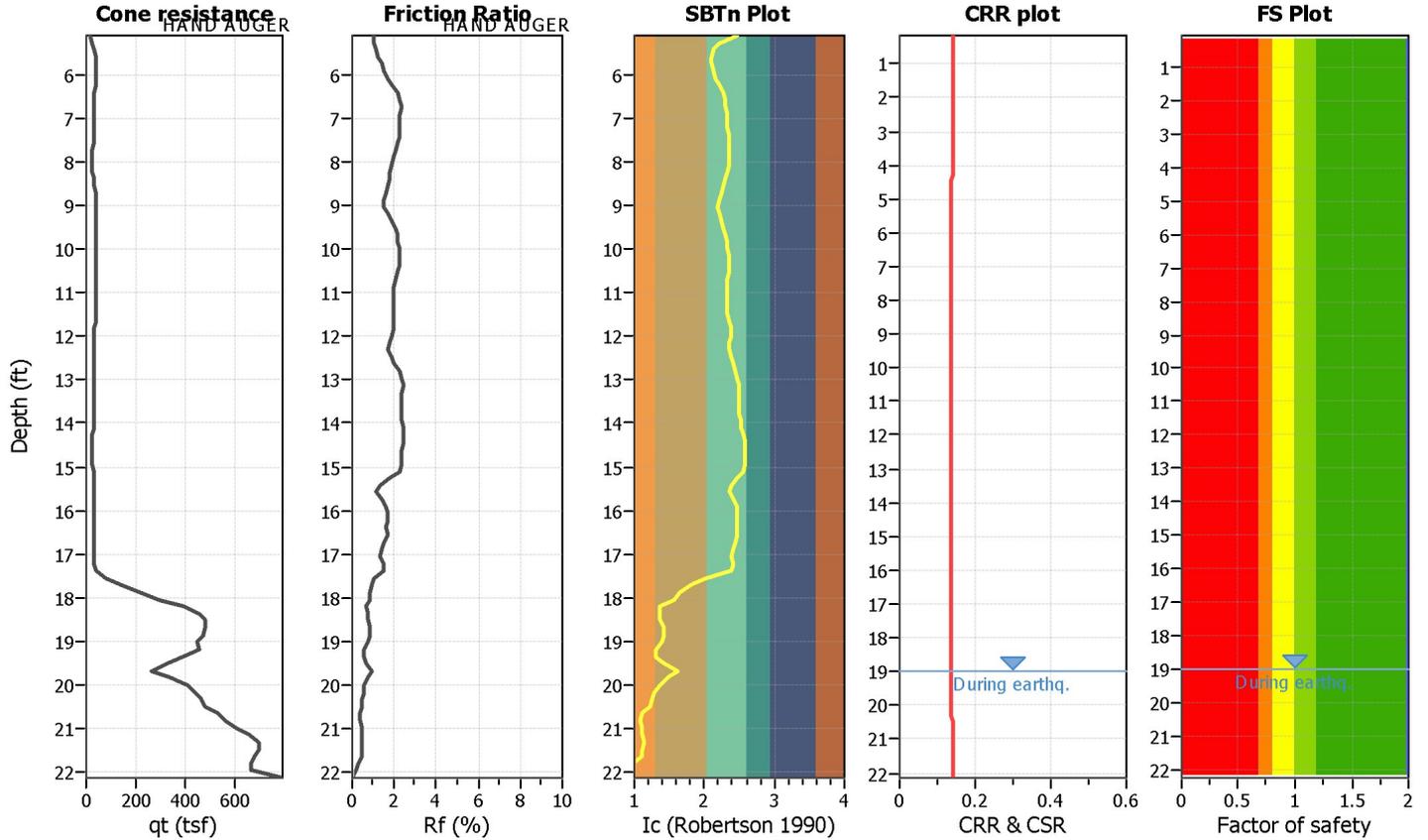
**Project title : Yamane Lofts**

**Location : 2500 & 2508 J Street, Sacramento, CA**

**CPT file : 1-CPT1**

**Input parameters and analysis data**

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	19.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	19.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude $M_w$ :	6.60	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.30	Unit weight calculation:	Based on SBT	$K_0$ applied:	No		



## LIQUEFACTION ANALYSIS REPORT

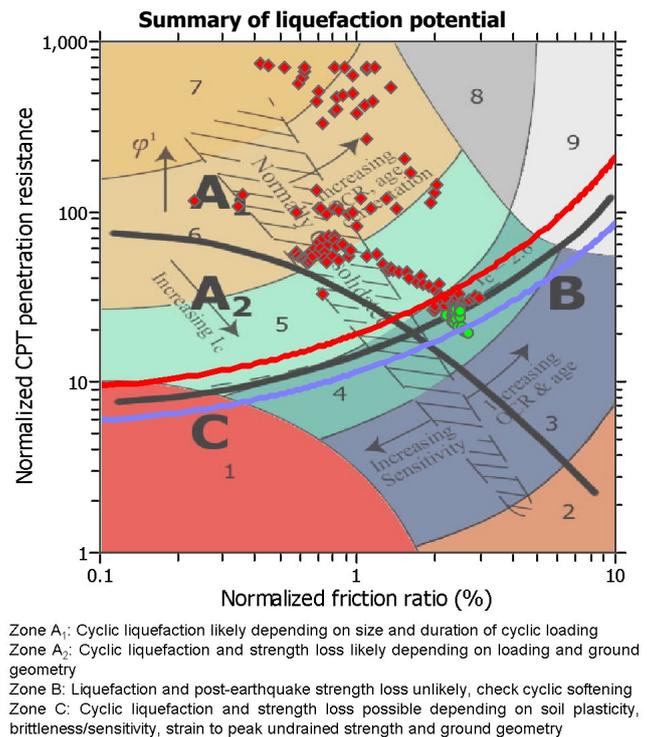
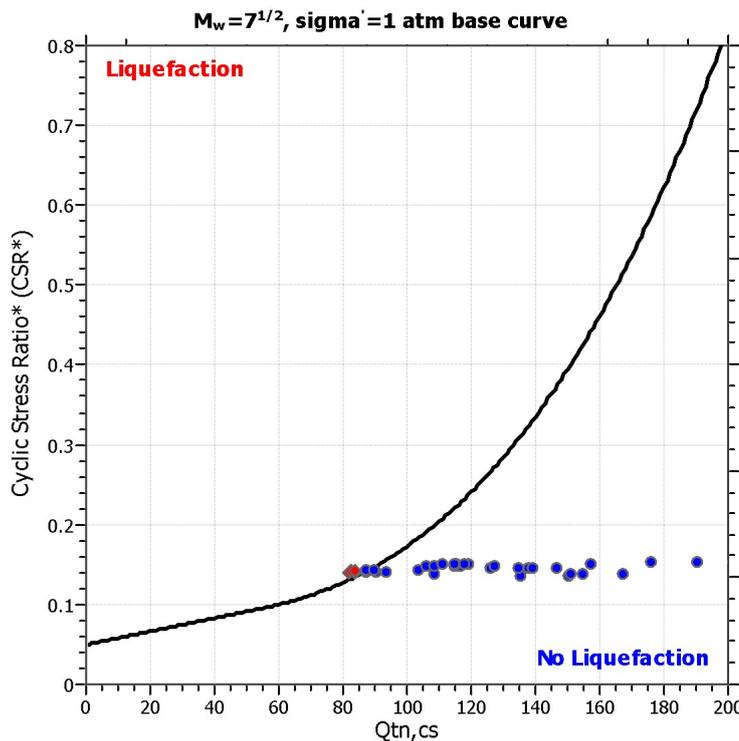
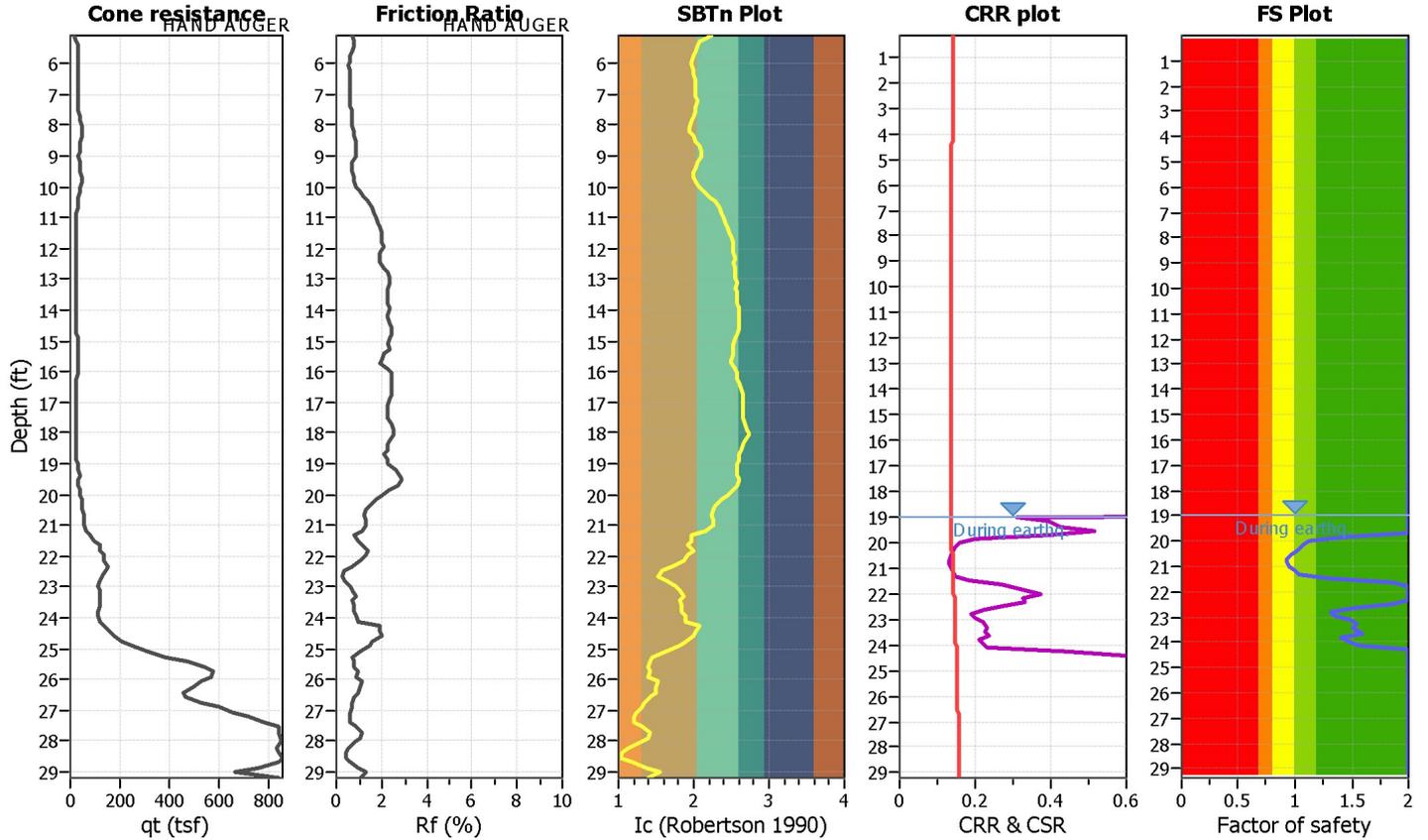
**Project title : Yamane Lofts**

**Location : 2500 & 2508 J Street, Sacramento, CA**

**CPT file : 1-CPT2**

**Input parameters and analysis data**

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	19.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	19.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude $M_w$ :	6.60	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.30	Unit weight calculation:	Based on SBT	$K_0$ applied:	No		



## LIQUEFACTION ANALYSIS REPORT

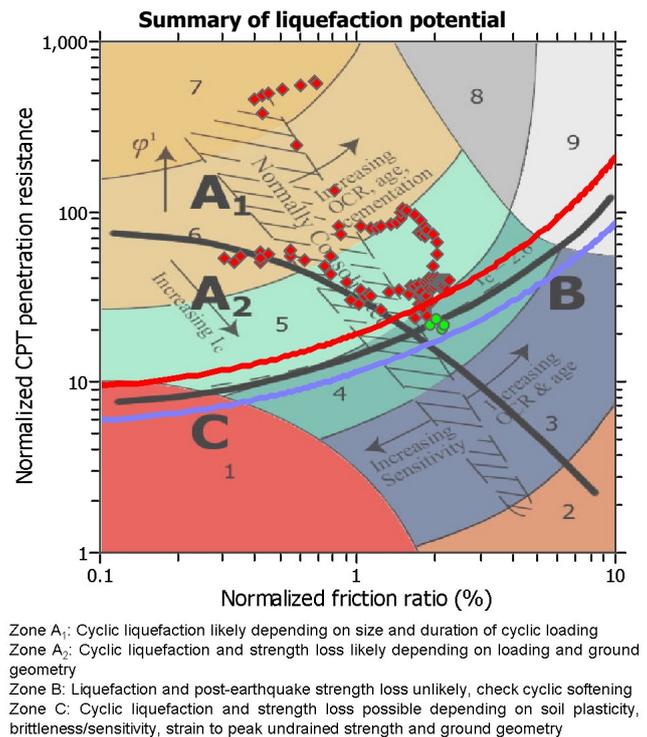
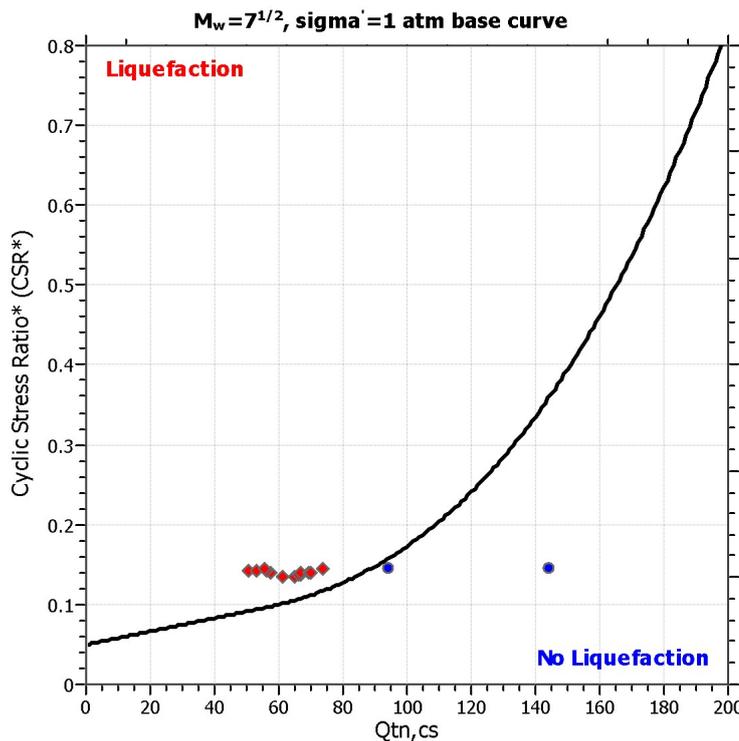
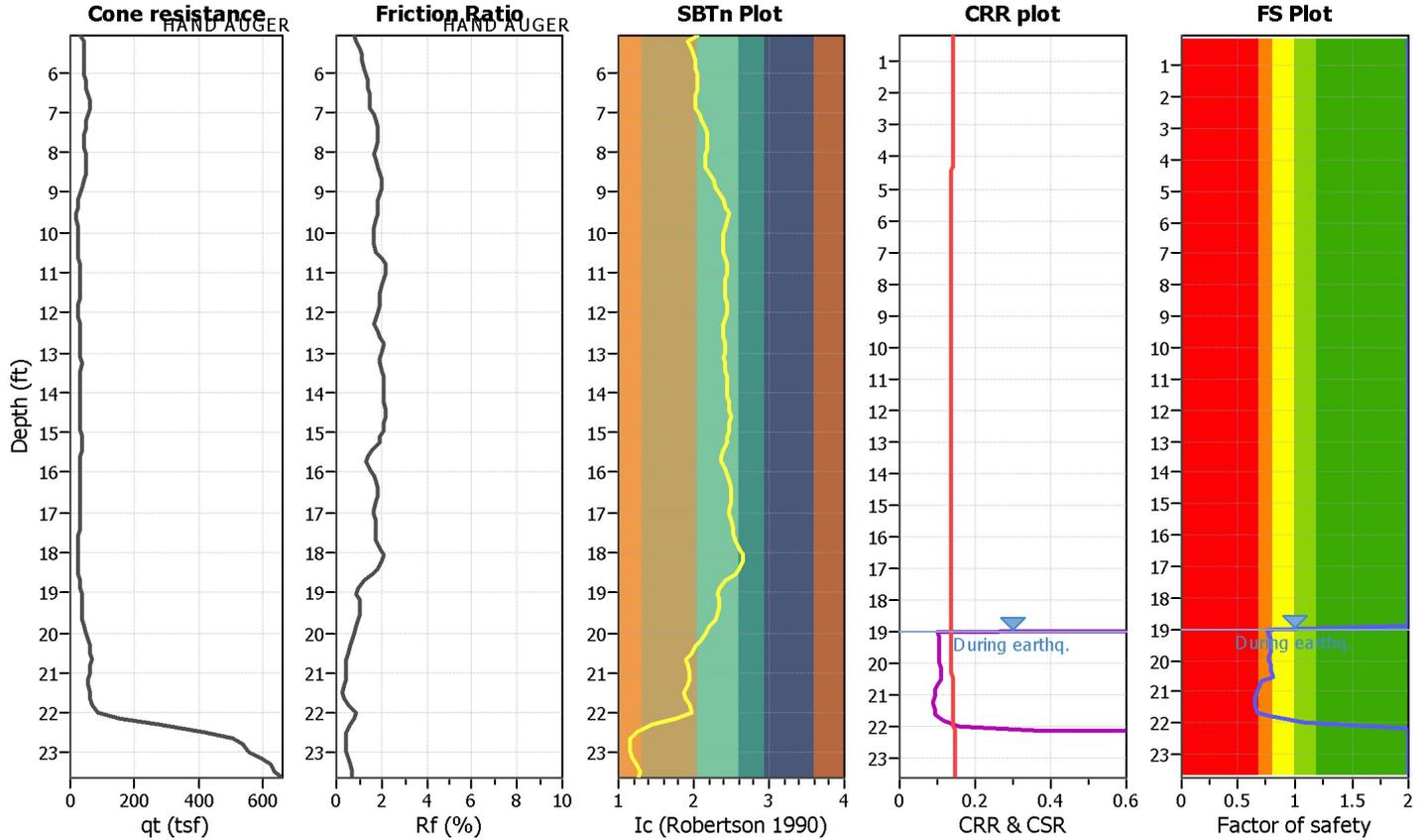
**Project title : Yamane Lofts**

**Location : 2500 & 2508 J Street, Sacramento, CA**

**CPT file : 1-CPT3**

**Input parameters and analysis data**

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	19.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	19.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude $M_w$ :	6.60	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.30	Unit weight calculation:	Based on SBT	$K_0$ applied:	No		



## LIQUEFACTION ANALYSIS REPORT

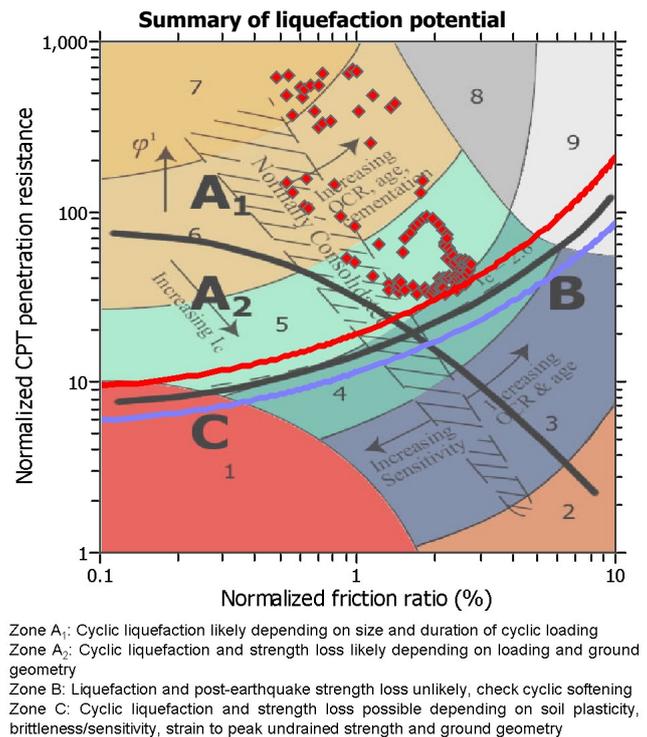
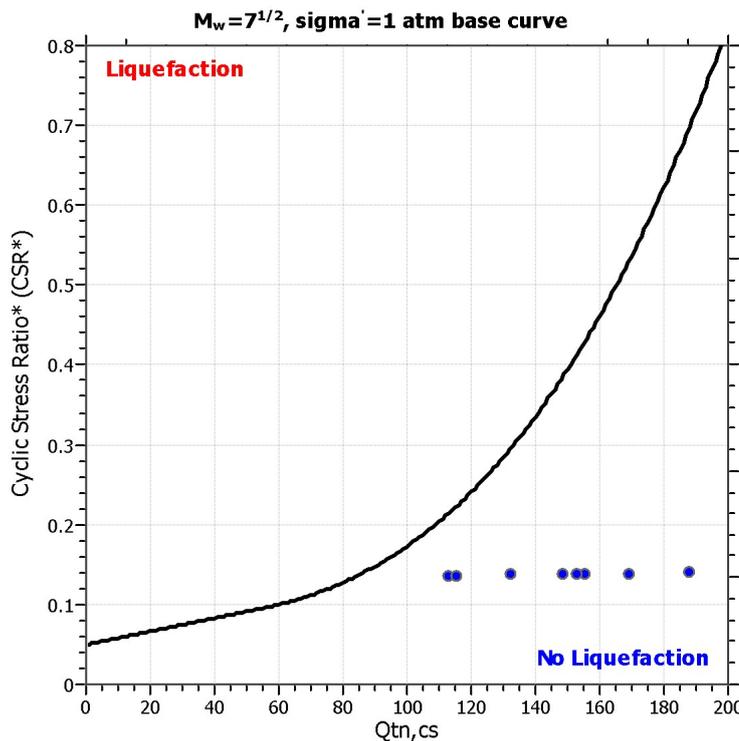
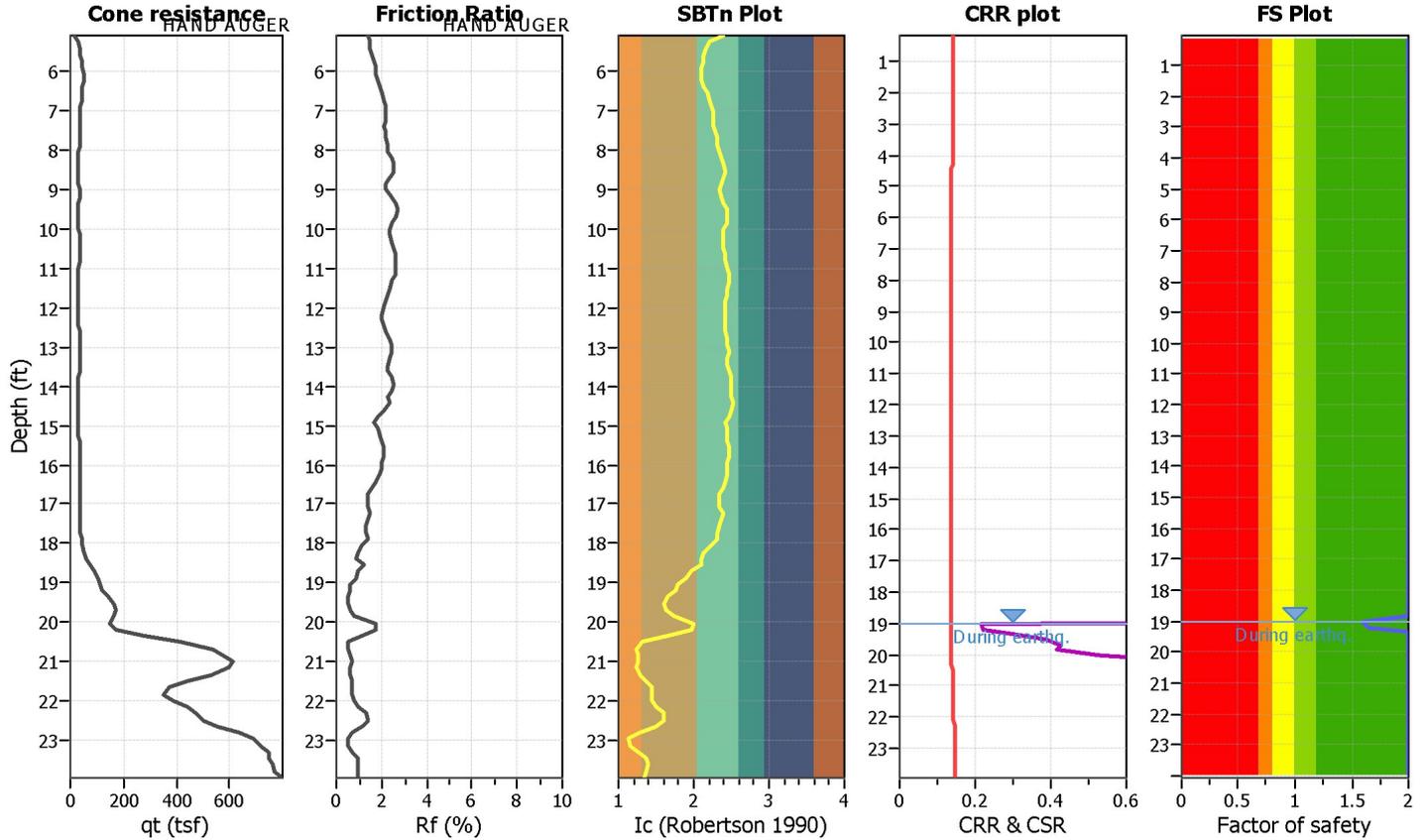
**Project title : Yamane Lofts**

**Location : 2500 & 2508 J Street, Sacramento, CA**

**CPT file : 1-CPT4**

**Input parameters and analysis data**

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	19.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	19.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude $M_w$ :	6.60	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.30	Unit weight calculation:	Based on SBT	$K_0$ applied:	No		



## LIQUEFACTION ANALYSIS REPORT

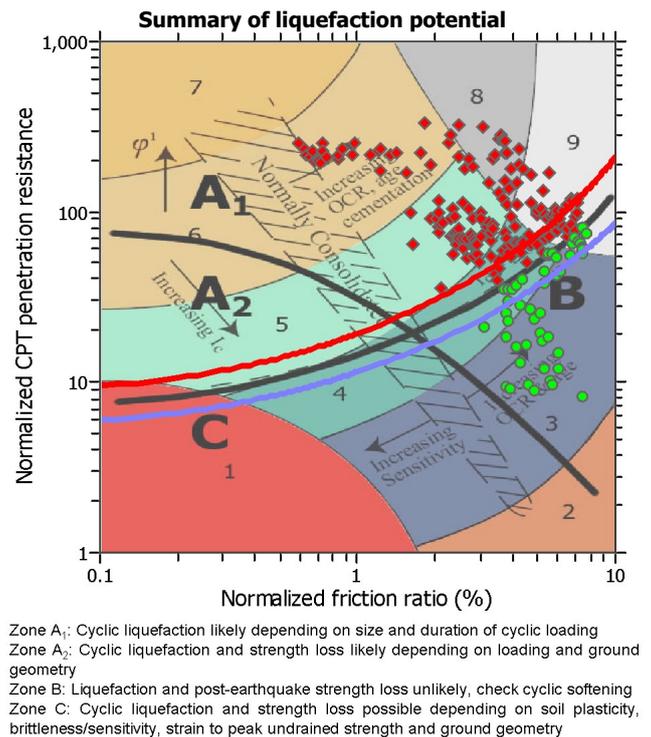
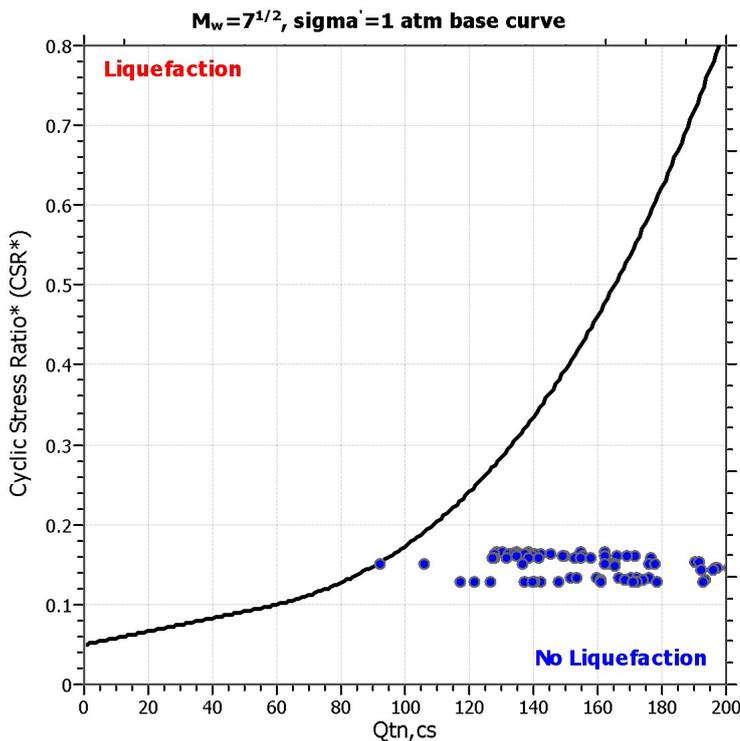
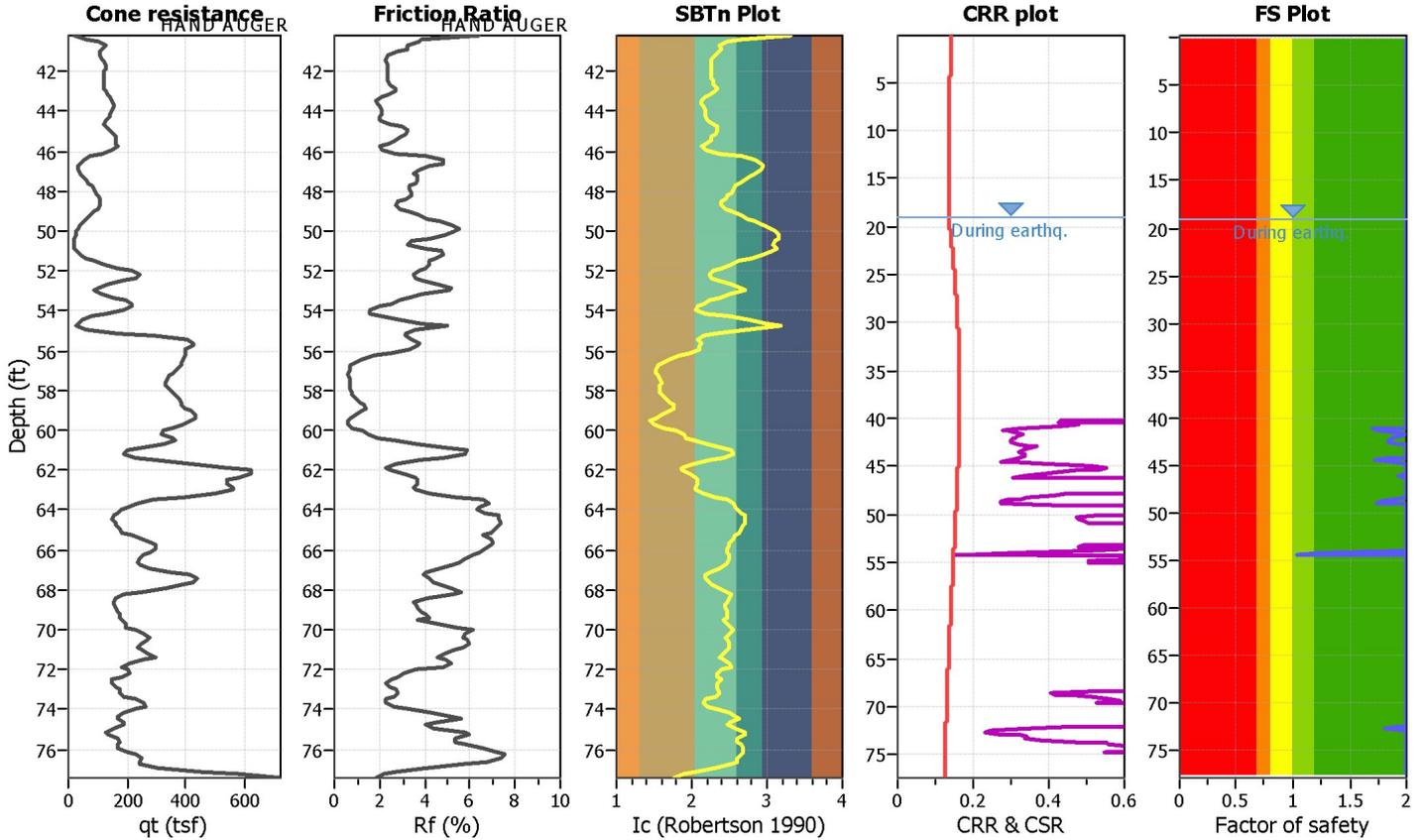
**Project title : Yamane Lofts**

**Location : 2500 & 2508 J Street, Sacramento, CA**

**CPT file : 1-CPT4A**

**Input parameters and analysis data**

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	19.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	19.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude $M_w$ :	6.60	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.30	Unit weight calculation:	Based on SBT	$K_0$ applied:	No		



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**APPENDIX E**

**Supplemental Recommendations**



# *SUPPLEMENTAL RECOMMENDATIONS*



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## GENERAL INFORMATION

### PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

### DEFINITIONS

<b>Backfill</b>	Soil, rock or soil-rock material used to fill excavations and trenches.
<b>Drawings</b>	Documents approved for construction which describe the work.
<b>The Geotechnical Engineer</b>	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
<b>Engineered Fill</b>	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
<b>Fill</b>	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
<b>Imported Material</b>	Soil and/or rock material which is brought to the site from offsite areas.
<b>Onsite Material</b>	Soil and/or rock material which is obtained from the site.
<b>Optimum Moisture</b>	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
<b>Relative Compaction</b>	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
<b>Select Material</b>	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

## **PART I - EARTHWORK**

### **1.1 GENERAL**

#### **1.1.1 WORK COVERED**

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

#### **1.1.2 CODES AND STANDARDS**

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

#### **1.1.3 TESTING AND OBSERVATION**

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

## 1.2 MATERIALS

### 1.2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

### 1.2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics, or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

**TABLE 1.2.2-1**  
Engineered Fill and Backfill Requirements

US Standard Sieve	Percentage Passing
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

**TABLE 1.2.2-2**  
Imported Fill Material Requirements

GRADATION (ASTM D-421)	SIEVE SIZE	PERCENT PASSING
	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

### 1.2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

#### 1.2.3A Pipe

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

**TABLE 1.2.3A-1**  
Perforated Pipe Requirements

Pipe Type	Standard	Typical Sizes (inches)	Pipe Stiffness (psi)
<b>Pipe Stiffness above 200 psi (Below 50 feet of Finished Grade)</b>			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
<b>Pipe Stiffness between 100 psi and 150 psi (Between 15 and 50 feet of Finished Grade)</b>			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
<b>Pipe Stiffness between 45 psi and 50 psi* (Between 0 to 15 feet of Finished Grade)</b>			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

\*Pipe with a stiffness less than 45 psi should not be used.

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

### 1.2.3B Outlets and Risers

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

### 1.2.3C Permeable Material

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

**TABLE 1.2.3C-1**  
Class 2 Permeable Material Grading Requirements

Sieve sizes	Percentage passing
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

### 1.2.3D Filter Fabric

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632) .....	180 lbs
Mass per Unit Area (ASTM D-4751).....	6 oz/yd <sup>2</sup>
Apparent Opening Size (ASTM D-4751).....	70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491).....	80 gal/min/ft <sup>2</sup>
Puncture Strength (ASTM D-4833) .....	80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer's recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

### 1.2.4 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall

encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

## ***PART II - GEOGRID SOIL REINFORCEMENT***

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength ( $T_a$ ) and Pullout Resistance, for the soil type(s) as specified on design plans.

The contractor shall submit a manufacturer's certification that the geogrids supplied meet plans and project specifications. The contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

## ***PART III - GEOTEXTILE SOIL REINFORCEMENT***

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

The contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

The contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO.

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

**TABLE III-1**  
Geotextile Soil Reinforcements

Property	Test
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec <sup>-1</sup> (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

## ***PART IV - EROSION CONTROL MAT***

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

The Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140°F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting out a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12 inches length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.