SOIL MECHANICS ANALYSIS AND COMPARISON TO IN SITU TEST METHODS OF SOILS FOUND IN POTRERO CANYON

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I. Abstract

Cone Penetration Test (CPT) data is widely accepted as the best option for subsurface investigation in determining sequence of subsurface strata, groundwater conditions, and mechanical properties of subsurface strata. While the CPT is very useful for geo-environmental purposes, the significance of widely varying data within a substrate is still rather unknown. In analyzing the soil properties of Potrero Canyon, this paper discusses the standard CPT test procedures and compares this data to data obtained from standard lab test procedures for soil mechanics analysis; the lab tests include visual description and classification of soils, moisture content, Atterberg limits, hydrometer analysis, and sieve analysis. Future study for this project includes the susceptibility of liquefaction in fine-grained soils and those issues are also discussed.

II. Introduction

Site Background

Potrero Canyon is located in the San Fernando Valley and is a 5-km-long, 200-m-wide, east traveling valley (Winterer and Durham, 1962). During the Northridge earthquake of 1994, the soil in the canyon was greatly affected by the loading of a large magnitude (M6.7) (Hall, 1994). In a reconnaissance report by Rymer et. al., liquefaction in the area had not been verified; however, there were large amounts of ground fractures as well as sand boils that were noted in the region (2001). The paper also reported that it was possible that much of the pipe breaks that occurred in the area were possibly caused by liquefaction. In a preliminary geological and geotechnical report (Allan Seward engineering geology), the soils in the canyon were described as generally unsuitable for the support of structures. The reason being that the layering of soil in areas containing steep slopes is malformed unlike those in the neighboring area of Potrero Mesa.

The presence of soils that are seemingly susceptible to liquefaction combined with the planning of an approximately 20,000 single-family home development sparked major interest in the properties of the soil for both engineers and investors. In February 2007, a seismic mitigation program began to assess the risk of liquefaction occurring in Potrero Canyon and, if needed, implement an engineering program. This seismic mitigation program is scheduled to last until November 2007. In this short time, a collaborative team of engineers and researchers are to perform a heavy regiment of in situ and lab testing to have an in-depth understanding of the soil strength and its susceptibility to liquefaction or cyclic failure in the area.

The first set of tests is all part the test fill program portion of the seismic mitigation program. In this program, there are four procedures: (1) instrumentation monitoring, (2) laboratory testing, (3) geotechnical review and analysis, (4) and a final written report. The in situ test data taken for this project is a direct result of the test fill program. Logs and cone penetration test data were taken from beneath the test fill early in the seismic mitigation program.

Leighton and Associates Inc. following construction guidelines provided by ENGEO performed the instrument implementation and drilling. The installed instrumentation included three settlement plates, four vibrating wire piezometers, one magnetic extensometer, and one groundwater monitoring well. Upon constructing the test fill pad at the test site, cone penetration test data was taken to compare readings taken from under the test pad to adjacent readings taken without fill above them.

Cone Penetration Test

Cone Penetration Test (CPT) data is widely accepted as the best option for subsurface investigation in determining sequence of subsurface strata, groundwater conditions, and mechanical properties of subsurface strata (Robertson, 2006). However, while the CPT is very useful for geo-environmental purposes, the significance and understanding of widely varying data within a substrate is still rather unknown.

The typical design of the cone penetrometer consists of three main components. The first component is the cone tip, which measures the tip resistance of the cone using strain gages. This parameter is the most commonly used in engineering applications. The second main component is the friction sleeve. It uses local friction strain gages to measure the soil's texture, which then can be used to calculate the soil behavior. The last main component of the cone penetrometer is the pore water pressure (CPTU) transducer. With the measurement of pore water pressure it became apparent that it was necessary to correct the cone resistance for pore water pressure effects, especially in clay (Rowe, 2001).

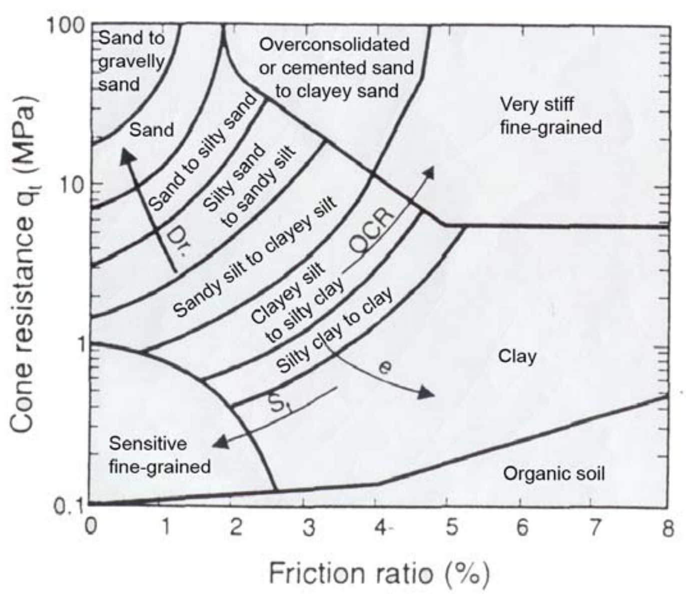

Figure 1: Soil behavior type classification adapted by Robertson et al. (1989)

The CPT is useful for determining overall soil strength and behavior indeed; it is especially useful to classify a soil using the tip resistance (q_c) and friction ratio (R_f) (Robertson, 1989). Robertson et al. developed a simplified chart shown in Figure 1 to identify stratigraphic features of a subsurface stratum using the two parameters mentioned above, which is widely used today in drilling and subsurface exploration.

For this research, the CPT data taken from SCPT boring 2a was used as a guide to help find what depths to further investigate. With a sample tube that had such a widely varying CPT plot, one of the main objectives was to investigate —by performing standard lab tests— depths at which the CPT showed large transitions. These depths, based on the CPT data, were taken to be loose estimates because of the known limitations of CPT when analyzing at very small increments.

Limitations

There are a few limitations of the cone penetration test, especially when looking specifically at transitions that are less than 5 cone diameters apart from each other. That was the case for this instance. The ASTM D 5778 advises that regardless of the type of CPT probe used, the results are average values of the soil resistance over a length of about

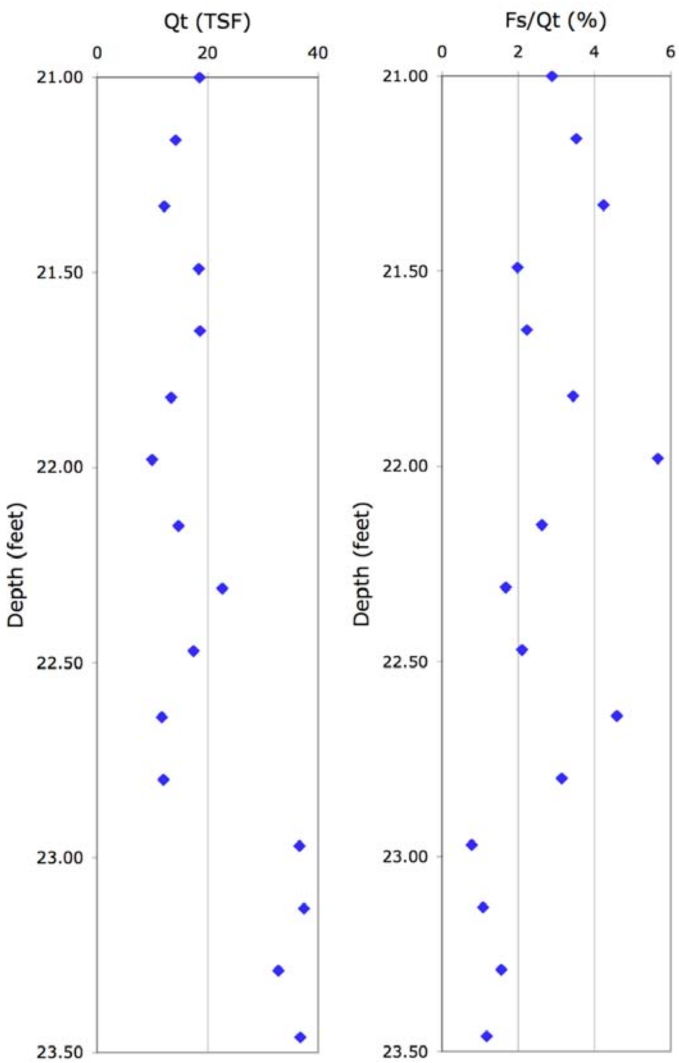

Figure 2: Extrapolated CPT data for SCPT boring 2a - Tube 7

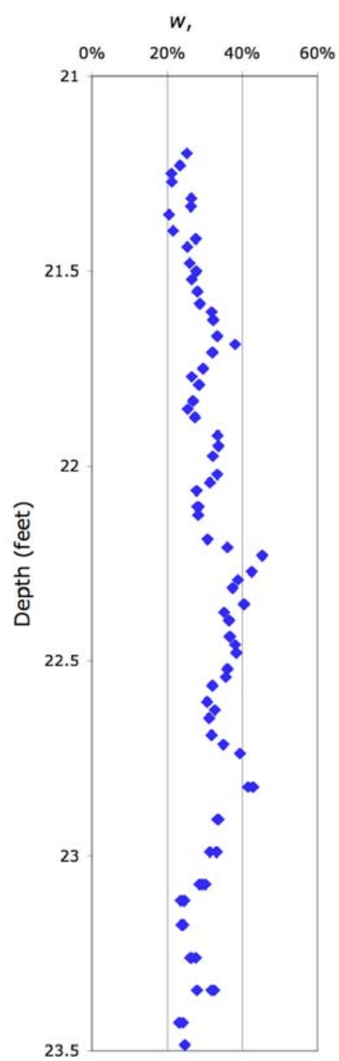

Figure 3: Water content data tested in laboratory

10 cone diameters—about 5 diameters above the tip plus about 5 diameters below the tip (2000). This "zone of influence" affects the result of the CPT to a relatively small degree when analyzing soil behavior to get an idea of the average strength and behavior of the soil; however, when exploring virtually every inch of a standard Shelby tube, these values really need to be taken as a lead to direct further investigation.

Another limitation of the CPT is that the penetration is restricted to dense sands. Evaluation of properties in soft and medium stiff fine-grained soils should be made with caution (Robertson and Powell, 1989).

Problems

In drilling procedures, the incremental distance can vary widely depending on how much data is desired by the engineers. In the CPT logs provided by ENGEO, the incremental distance was 0.164 feet (5 cm). A problem arises when the data that is being taken in the laboratory is at the one-third inch to one-inch increments, which is at a higher resolution than the CPT data. Specifically, multiple data points are being gathered in the lab to represent or correlate to one point on the provided CPT logs. This problem can be illustrated by comparing the Figures 2 and 3 side by side.

Link to Project

The basis for using the cone penetration test along with a detailed logging of this standard Shelby tube is to form a deeper understanding of the behavior of soils when there is a large amount of variability in a small amount of material. Having several sudden changes in the cone tip resistance (q_c) in one soil sample, an analysis of the "zone of influence" can be made and in this case, check all possible sources for affecting the parameters of the CPT.

Liquefaction Susceptibility

A field that is gaining continued interest amongst geotechnical engineers is soil liquefaction. During monotonic and cyclic undrained shear loading, a soil can liquefy-losing strength and causing damage to structures above it. Although a major cause of damage during earthquakes, engineering practices are still evolving to deal with the problems posed by liquefaction.

Today, there are many methods to treat liquefiable soils. When an area is deemed to run a risk of liquefaction during an earthquake, a mitigation process can take place, which will usually reduce the risk sufficiently. Current research is attempting to be able to classify liquefiable soils more easily using simple soil parameters to evaluate liquefaction susceptibility.

Earlier studies on liquefaction phenomena were on sands and fine-grained soils such as silts, clayey silts and even sands with fines were considered non-liquefiable (Prakash, 1999). More recently, after such earthquakes as Haicheng (1975), Tangshan (1976) (Wang, 1979) and Kocaeli in 1999 (Bray, 2006), it was suggested that fine-grained soils could be liquefiable. Even today, there is little lab test data to be able to determine the likelihood of a fine-grained soil to be liquefiable.

After analyzing soil that liquefied in China, Wang states that any soil containing less than 15-20% particles by weight, smaller than 0.005 mm, and having a water content (w_c) to liquid limit (LL) ratio greater than 0.9 is susceptible to liquefaction (1979). In response, using the data from China provided by Wang, Seed and Idriss (1982) stated that clayey soils could be susceptible to liquefaction *only if all three* of the following conditions are met: (1) percent of particles less than 0.005 mm <15%, (2) LL<35%, and (3) w_c/LL>0.9.

After the establishment of the Chinese criteria, there was a movement to promote simple criteria based on "key' soil parameters to deduce the susceptibility of liquefaction in fine-grained soils. Andrews and Martin pointed out that because the grain size of silts fall between that of sand and clay, it is often assumed that the susceptibility of silts must also fall somewhere between the high susceptibility of sands and non-susceptibility of clays and that there is added confusion because silts and clays are coupled under the same "fines" heading (2000).

In a report by Boulanger and Idriss, in order to distinguish the major loss of strength in soils during undrained cyclic loading, a working definition of liquefaction and cyclic failure are established. Because strength loss in these fine-grained soils can occur for different reasons, "the term 'liquefaction' is used to describe the onset of excess pore water pressures and large shear strains during undrained cyclic loading of sand-like soils and the term 'cyclic failure' is used to describe the corresponding behavior of clay-like soils" (2004). The need for this establishment illustrates the dual properties that fine-grained soils can display, especially those with a high percentage of silt content. The report continues to establish the distinctions between sand-like and clay-like fine-grained soils. They also have recommendations for the evaluation for each of the respective soil types and discuss them.

III. Methods

In Situ Testing

All of the in situ testing for the Shelby tube utilized in this project was performed by Leighton and Associates Incorporated according to guidelines provided by ENGEO Incorporated. The performed detailed logging of borings along with cone penetration test data provided a strong basis for soil classification prior to lab testing.

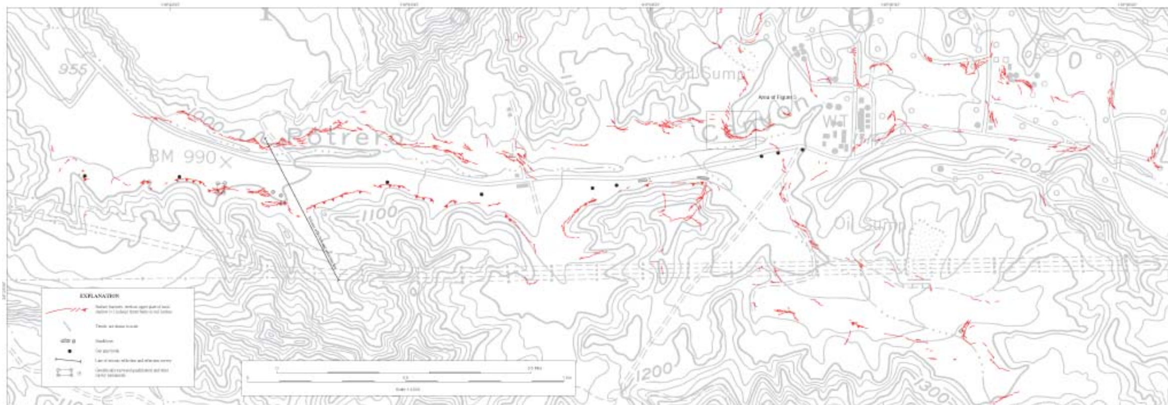

Figure 4: Surface fractures shown in red provided by USGS

United States Geological Survey Reconnaissance

Shortly after the Northridge earthquake in 1994, the United States Geological Survey (USGS) visited Potrero Canyon to perform a reconnaissance of any geologic activity following the earthquake. The nature of this mission was one from a geologic standpoint and not one of an engineering perspective. In a report released by the USGS, numerous surface fractures were observed in the area and taken note of. This may have been caused by the canyon being situated in the up-dip projection of the seismographic rupture plane of the main shock (Winterer and Durham, 1962).

The surface fractures found in this investigation were not associated with primary faulting or with triggered, secondary, surface faulting on a deep seismographic fault, but rather the term “surface fractures” was used to describe general ground breakage (Rymer et al., 2001). These surface fractures can be seen in Figure 4 shown as red lines spread throughout the canyon. The USGS report also states that several of the fractures were open or had been filled with loose sand. Normal displacement of up to 0.066 feet (2 cm) was observed across these fractures within the trenches.

There were also some landslides that occurred during the earthquake. There were thousands of landslides triggered by the 17 January earthquake (Harp and Jibson, 1995), including landslides in the Potrero Canyon area. The volume of individual earthquake-induced landslides in the Potrero Canyon area varied, but most commonly was small, less than 10 cubic meters (Rymer et al., 2001).

There were also sand blows observed following the Northridge earthquake. Also mentioned by the report released by the USGS, these sand blows formed cones, about one to three meters in diameter, which locally coalesced into zones tens of meters long. (2001). While the report did not verify the occurrence of liquefaction they could not cite anyone who could, the amount of earthquake-induced ground activity was remarkable and the report did mention the possibility of liquefaction being the cause of multiple pipe breaks in the area.

Test Fill Compaction Program

Fill compaction is one of the fundamental concepts in geotechnical engineering. It is the process to increase the density of soil mass by mechanical means (Gue and Liew). The reason there was a need for a test fill compaction program in Potrero Canyon was because of the unpredictable behavior of the soil in the area after the 1994 Northridge earthquake. The program began in February 2007 and the testing and monitoring process is scheduled until September 2007.

As part of the test fill program, dry densities, moisture contents, and compaction of the fill were all measured. When moisture content gradually increases, the soil skeleton structures will tend to collapse and rearrange easily to a more compact state under compaction as the effect of surface tension reduces with increasing water content. When the dry density of the compacted soil mass reaches a peak, the corresponding moisture content is called the optimum moisture content (OMC) (Gue and Liew, 2001).

The compaction attained at the end of the test is compared to the compaction required, which will determine the success of the test and test method.

Cone Penetration Test

As aforementioned, the CPT is widely accepted as a practical means for subsurface investigation. The type of CPT instrument used in this study was a seismic cone penetrometer. This penetrometer incorporates a triaxial package of small seismometers into a standard penetrometer (Campenella et al.). This enables the cone to read shear wave velocities in situ while obtaining CPT data. A cone of 10 cm^2 base area with an apex angle of 60 degrees is generally accepted as standard and has been specified in the European and American standards (ASTM, 1979). A friction sleeve, located above the conical tip, has a standard area of 150 cm. The friction sleeve has the same diameter as the conical tip, e.g., 35.7 mm.

The tip of the cone is suited with strain gages that enable the cone to measure the resistance of the soil (q_c), or tip resistance. The friction sleeve obtains another reading (f_s). From these two parameters, the friction ratio can be calculated by dividing the friction sleeve reading by the tip resistance ($R_f = f_s/q_c \ x$ 100%). Plotting the friction ratio versus the cone resistance on the soil classification chart established by Robertson will establish the soil behavior/classification.

The cone was pushed and a log was taken on 5 April 2007. Readings were taken at 0.164 feet (5 cm) intervals and soil was classified by soil behavior type onsite. The cone was advanced at the standard rate of 2 cm/s. The boring was then taken on 16 April 2007 utilizing the mud rotary method. The accuracy of the vertical precision of the SCPT cone reading and the tube sampling is unknown. While it would be possible to attempt to line up the CPT data to the results of the lab tests of the Shelby tube, it is not recommended due to the possibility of skewing data to a personal bias.

Laboratory Testing

The bulk of this NEESreu project was spent in the lab obtaining soil characteristics in order to compare and contrast with the in situ data that was obtained from various sources. All lab testing was performed in the soil interactions laboratory at University of California at Davis.

The objective in performing lab tests on Seismic Cone Penetration Test Boring 2a - Tube 7 sampled from Potrero Canyon was to obtain as much soil mechanics data possible for soil mechanics analysis and to make further use of data taken for future use. This would ultimately be used as part of a larger study of liquefaction susceptibility in fine-grained soils. Lab testing was performed under the specifications pointed out by the American Society for Testing and Materials (ASTM) via the visual manual procedure, moisture content, Atterberg limits, and hydrometer analysis.

Visual Manual Procedure and Soil Classification

To begin laboratory testing, logging of the tube sample by means of the visual manual procedure as defined by ASTM Designation D 2488. This began by cutting small sections of the Shelby tube (approximately 5 in.) and extruding the soil from the tube. After extruding the sample, a longitudinal cross section was created using a wire saw, taking extreme care to minimize any disturbances to the soil. Detailed photos (Figure 5) and drawings were made of these cross sections, and the visual description of the soil was taken.

Descriptive information required in the visual manual procedure cannot only be used for engineering purposes, but also as a scientific method to compare different forms of data acquired through different means. The primary procedure used for this process can be found in Section 10 and 14 of ASTM Designation D 2488; descriptive information for soils and the procedure for identifying fine-grained soils, respectively.

Color was typically the first point made when describing a soil. This can play an important role when comparing to similar materials in the area. Also, there were some changes in color, which made some thin layers visible. The moisture condition of the sample was also made note of as well as the measured moisture content of the soil later discussed in this paper. Other soil characteristics notated were hardness of larger particles, organic material content, and the soil's

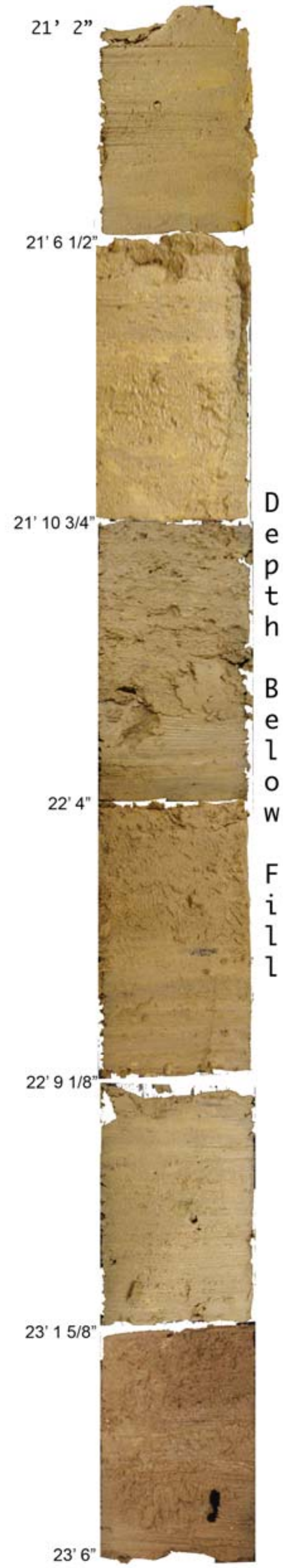

Figure 5: Cross section of Shelby tube sample

reaction to *hydrochloric aci*d (HCl). Actually, much of the soil contained in tube 7 reacted very strong to HCl. This implies that that calcium carbonate is present in high concentrations since it is a common cementing agent.

More specific to fine grained soils, notes were taken on the dry strength of the soil. This was done by rolling a small sample into a ball approximately ½ inch in diameter and crushing it, then measuring the pressure required to force the ball to crumble. The dilatancy of the soil was also remarked by applying small forces to the surface of the sample repeatedly, then describing the amount of water liberated from the soil. This is useful for visual classification because more silty soils will release more water when tested then a clay will.

Moisture Content

Moisture content data was taken for every 1/3 inch of the Shelby tube. This procedure was performed in accordance with the ASTM Designation D 2216: the Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass. This method, most commonly used to test moisture content of soils, utilizes a drying oven to extract all water from the sample.

In this procedure, a test specimen is selected to be representative of the entire sample. By taking samples every 1/3 of an inch, a precise representation of the sample can be assumed. These test specimens were also taken from the center of the tube, where the soil is the most undisturbed, in its natural state.

After a test specimen is selected, the mass of a clean and dry tare is determined and recorded. The moist test specimen is placed into the tare, and the combined mass is determined and recorded. The moist sample and the tare are placed in the oven at approximately 110° C and dried for a minimum time of 12 to 16 hours or until the mass remains constant. The sample and container are then removed from the oven and allowed to cool to room temperature so as to not affect the mass reading due to convection currents.
The water content can now be calculated using all of the recorded data using the formula as follows:

$$w = [M_{cws} - M_{cs})/(M_{cs} - M_c)] \text{ x } 100 = \frac{M_w}{M_s} \text{ x } 100 \qquad \text{(Eq. 1)}$$

where:
w = water content, %,
M_{cws} = mass of container and wet specimen, g,
M_{cs} = mass of container and oven dry specimen, g,
M_c = mass of container, g,
M_w = mass of water, g, and
M_s = Mass of solid particles, g.

All of the preceding data is reported by including everything in a data sheet. In the data sheet, the sample is identified by including the boring number, sample number, test number, container number, etc in the header and tables. For this project, water contents were calculated the nearest 0.1%

Atterberg Limits

While initially developed for use in ceramics, this seemingly arbitrary test has come to be very useful in geotechnical engineering uses. The Atterberg limits consist of the liquid limit and the plastic limit. These two limits can then be used to determine the plasticity index, which in turn gives us the ability to classify the soil sample's behavior type. Because the test results of this test can somewhat vary depending on the experience of the operator, it is appropriate and important to adhere to the ASTM Designation D 4318.

The liquid limit apparatus is consisted of a hard rubber base, rubber feet, a brass cup, a mechanical cam, and a flat grooving tool. The hard rubber base is located beneath the brass cup and the cup is dropped from a height of 10 mm onto it. This rubber should have a D Durometer hardness of approximately 80 to 90. The rubber feet attached to the underside of the hard rubber base provide support and isolation of the base from the work surface. The brass cup of the liquid limit apparatus carries the grooved specimen and should weigh approximately 185 to 215 grams including the hanger but not the specimen. The rotating cam connected to the crank of the device and the cup hanger provides a smooth ascension and drop for the specimen. Finally, the flat grooving tool imprints a groove in the specimen and has the dimensions as specified by the ASTM.

When sampling, extra care needs to be taken as to not mix stratum. In this project, the Shelby tube was mostly silt throughout, but it did have many very think layers that could not have been extracted from the sample and tested. Also, any gravel pieces or course sand particles should be removed prior to testing. This is because these larger particles may influence the test and are not accurately measured with the testing apparatus. Any tested specimen should have never dried below its natural moisture content and should be prepared at least 16 hours before testing to a blow count of 25 to 35.

In obtaining the first of the two Atterberg limits, the liquid limit, Method A in the ASTM was performed. In this method, multiple blow counts are obtained and the moisture contents corresponding to those blow counts are recorded. In this experiment, two blow counts were taken between 12 and 25, and two more blow counts were taken between 25 and 40. Using linear regression analysis, the best-fitted line to these points was taken and valued at a blow count of 25. The moisture content at this blow count is considered the liquid limit of the specimen. In order to obtain these blow counts, a soil pat was placed into the brass cup of the liquid limit device and dropped from a height of 10 mm and the drop that closes the groove, made by the flat grooving tool. The blow that closes the groove at least 13 mm is the recorded blow count and moisture content. If it is necessary to add water to the specimen to manipulate the blow count, only distilled water is to be used.

The second Atterberg limit, the plastic limit, the only required test equipment is consisted of a ground glass plate, a spatula, and a wash bottle containing distilled water. In this procedure, a specimen weighing approximately 5 g is rolled on to the glass plate between the palm and fingers with sufficient pressure to force the ball into a cylindrical shape. The rolled thread should be rolled to a diameter of 3.2 mm. After this is performed, the piece(s) are combined together into a ball once again and the procedure is repeated until the specimen crumbles at 3.2 mm. The water content of the specimen at this occurrence is considered the plastic limit.

The plasticity index is then calculated from the liquid limit and the plastic limit. The calculation is as follows:

$$PI = LL - PL \qquad \text{(Eq. 2)}$$

where:
LL = liquid limit, and
PL = plastic limit.

Both the plastic limit and liquid limit are to be taken as whole numbers, and if either the liquid limit or plastic limit could not determined by the test procedure, then the soil is to be considered nonplastic, denoted NP.

Hydrometer Analysis

This project also used hydrometer analysis to further correlate the soil mechanics properties to the cone penetration test. This method applies Stokes' law of free falling spherical particles in a continuous viscous fluid. Because the hydrometer procedure is only useful for distinguishing the percentage of silts and clays, sieve analysis is also necessary in this project, especially because there are some relatively large percentages of fine and medium sands in the Shelby tube tested. The purpose of this test is to determine the percentage of soil passing a particular particle diameter.
For the preparation of the sample to be tested, a dispersing agent, *sodium hexametaphosphate* $(NaPO_3)_6$,was applied to the sample at a concentration of 125 g per liter of distilled water and was allowed to soak overnight. The purpose of the dispersing agent and the soaking is to force the clay particles apart by neutralizing the Van der Waal forces that keep them grouped together. If the dispersing agent is not applied, the grouped clay particles will be considered one large particle in this experiment.

After the dispersing agent is allowed to soak overnight into a thick slurry , the sample is then poured into a dispersing cup along with 125 mL of distilled water and is mixed vigorously for about 1 to 2 minutes. The slurry sample is then poured into a cylinder that is filled water until it reaches 1 liter. The temperature of the water in the cylinder should be allowed to cool or reach the room temperature that will be prevalent throughout the experiment since the hydrometer readings can be affected by temperature.

The cylinder should then be turned upside down, then upright repeatedly for about 1 minute or until the soil sample inside is distributed evenly throughout the volume of the cylinder. The cylinder should then be placed right side up on a hard surface while simultaneously starting time at 0. Readings are then taken from an ASTM hydrometer at 1, 2, 4, 8, 16, 32, 64, 128, 256, etc. until readings are obtained for at least 48 hours. Any error in temperature and in the meniscus of the slurry can be corrected by having a separate cylinder containing only distilled water and the dispersing agent in the same proportions as the cylinder with the soil sample. After every reading taken from the hydrometer, a second reading should be taken from the control solution. The difference between these two readings is the corrected hydrometer reading.

Once all hydrometer readings are taken, the soil solution is passed through a No. 200 sieve. All particles not passing the No. 200 sieve are considered to be sand based on the Unified Soil Classification System (USCS). The sample not passing the No. 200 sieve is then passed through a No. 40 sieve. The sample passing the No. 40 sieve is considered to be fine sand particles and the particles not passing are medium sand particles. The separated soil samples are collected into different tares and oven dried. The dry masses are then taken and the total mass of the sample is recorded.

To calculate what percentage of soil passes a particular particle diameter, the percentage of soil remaining in the solution at each time interval needs to be calculated. This percentage of soil remaining in suspension at which the hydrometer is measuring the density of the suspension can be calculated by the following equation:

$$P = (Ra/W) \times 100 \qquad \text{(Eq. 3)}$$

where:

a = correction factor to be applied to the reading of hydrometer 152H. (Values shown on the scale are computed using a specific gravity of 2.65.

P = percentage of soil remaining in suspension at the time at which the hydrometer measures the density of the suspension.

R = hydrometer reading with composite correction applied

W = oven-dry mass of soil in a total test sample

Next, in order to obtain the diameter of the particles corresponding to the percentage indicated by a given hydrometer reading, we apply Stokes' law assuming that a particle of this diameter was at the surface of the suspension at $t = 0$ and had settled at the level at which the hydrometer is measuring the density. According the Stokes' law:

$$D = \sqrt{[30n/980(G - G_1)] \times L/T} \qquad \text{(Eq. 4)}$$

where:

D = diameter of particle, mm

n = coefficient of viscosity of the suspension medium (in this case water) in poises (varies with changes in temperature of the suspending medium),

L = distance form the surface of the suspension to the level at which the density of the suspension is being measured, cm

T = interval of time from beginning of sedimentation to the taking of the reading, min

G = specific gravity of soil particles (taken to be 2.65), and
G_1 = specific gravity (relative density) of suspending medium (value may be used as 1.000 for all practical purposes)

The above calculation can be simplified for convenience in the form as follows:

$$D = K\sqrt{L/T} \quad \text{(Eq. 5)}$$

where:
K = constant depending on the temperature of the suspension and the specific gravity of the soil particles.

The above calculation can be graphed on a plot with particle diameter versus percent passing with the respective units in mm and %. In order to determine the percent passing the particle diameter at which the USCS considers clay particles, the value at 0.002 mm is evaluated. This value is the percentage of total soil sample passing that is considered clay.

The percentage of the total soil sample that is considered by the USCS to be medium-grained and fine-grained sand can be computed more directly. After oven-drying the samples, the masses of the sample retained by the No. 200 and No. 40 sieves separately. The percentage passing each can be calculated by the following equations:

$$P_S = M_{<200}/M_S - P_C \quad \text{(Eq. 6)}$$
$$P_{FS} = M_{<40}/M_S \quad \text{(Eq. 7)}$$
$$P_{MS} = M_{>200}/M_S - P_{FS} \quad \text{(Eq. 8)}$$

where:
P_S =percentage of soil sample considered to be silt by USCS,
P_{FS} = percentage of soil sample considered to be fine sand by USCS,
P_{MS} = percentage of soil sample considered to be medium sand by USCS,
P_C = percentage of soil sample considered to be clay by USCS,
$M_{<200}$ = mass of soil sample having a particle diameter less than a No. 200 sieve, g,
$M_{>200}$ = mass of soil sample having a particle diameter greater than a No. 200 sieve, g,
$M_{<40}$ = mass of soil sample having a particle diameter less than a No. 40 sieve, g, and
M_S = mass of total soil sample, g,

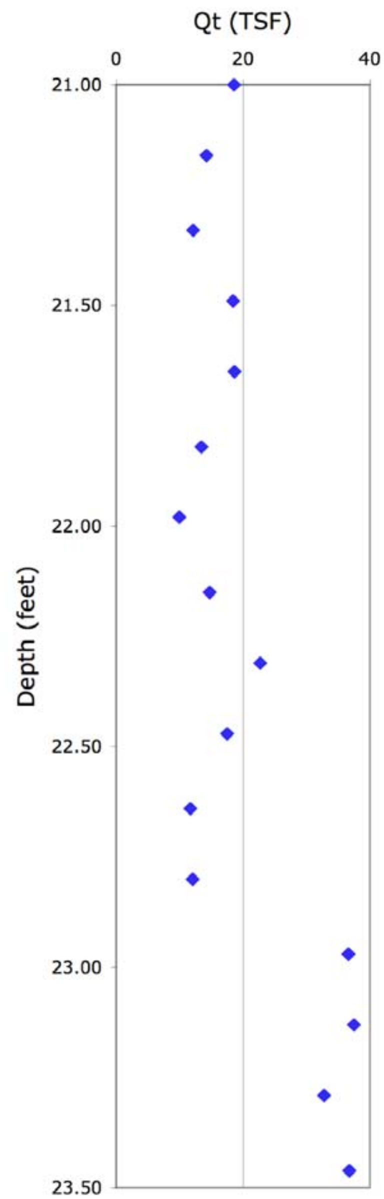

Figure 6: Plot of CPT data provided with transition at ~22.9 ft

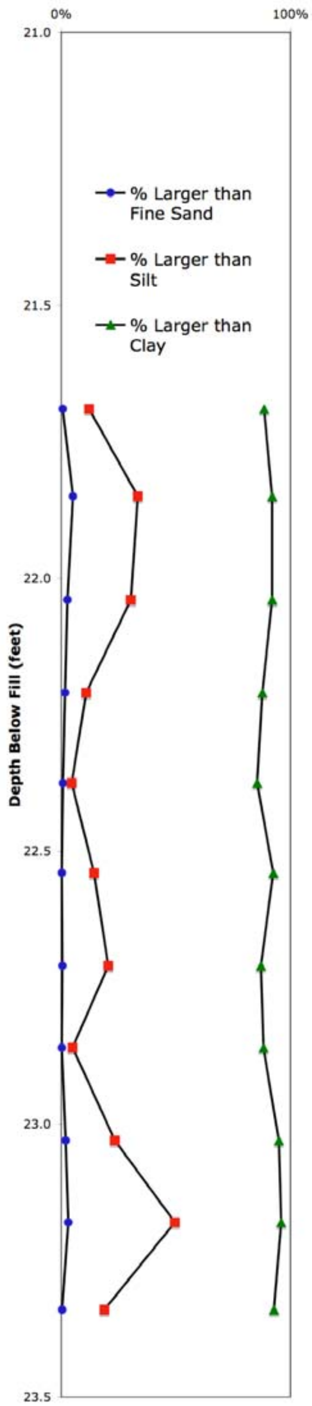

Figure 7: Grain Size distribution results

In reporting all of these parameters in a data sheet, all background of the soil should be included such as the boring number, sample number, sample depth, dispersing agent (either sodium hexametaphosphate or sodium metaphosphate), date of testing, a soil description, and hydrometer number. An example of a data sheet is included in Appendix 1.

IV. Results

The first of all the test methods, the cone penetration test results, had a fair amount of variability in it. When classifying the soil based on normalized CPT data, the tube sample varies in the top three-quarters of the tube changing from "silty clay to clay" to " clayey silt to silty clay." Just by looking at the CPT lo provided by ENGEO, it visually looks like there is much more changes in soil behavior type, but when classifying the soil based in the normalized Robertson chart, that portion of the tube is still fairly consistent. The bottom quarter of the tube; however, is classified as "sandy silt to clayey silt" and "silty sand to sandy silt." While still not a major definitive change in strata, this is where the most dramatic change in CPT data occurs for this tube sample. This change occurs where readings were at taken at 22.80 ft and 22.97 ft. The corresponding Qt values were 12.01 and 36.59, respectively. This can be seen in Figure 6. All CPT data provided for this study can be found in Appendix 2. This change in CPT data would dictate the location in the soil mechanics investigation of the Shelby tube.

In the visual-manual procedure portion of the testing of the Shelby tube, it seems as though the soil visually seemed to be sandier than the CPT revealed. There is definitely some agreement between the CPT and the visual description in the cases where clay is identified. In spite of this, because the soil was being analyzed visually at a higher resolution that the CPT took readings, understandably, there are sandy silt areas where clay is identified and vise-versa in clayey silt areas where sand is identified. Examples of these cases arises at approximately 23.27 ft and 22.5 ft, respectively.

When observing the trends in the water content results taken from every 1/3 inch of the tube, the first thing that can be seen is the range of water contents; these water contents range from about 20% to the middle 40% scope. In analyzing the water contents and comparing the results to the CPT data in Appendix 3, in the area of interest — approximately 22.9 ft, the water contents take a noteworthy drop in water content. Again, at this point the CPT reads the soil to transition from a clayey silt to a sandy silt, so this may be caused by a change in dilatancy in the soil. The sandier soil is not capable of retaining as much water as its clayey counterpart. So it can be assumed that there is agreement in the CPT results and the laboratory tested water contents.

The hydrometer test results were much less dramatic than expected by only looking at the CPT test results. In total, 11 hydrometer labs were conducted, and they all determined that the tube was silt in majority. In agreement with both the CPT and the visual description of the sample, there was a large increase in the percentage of sample containing sand at approximately 22.9 ft. Not enough sample was available for testing the entire zone of influence (10 cone diameters above and below CPT reading) of the transition point of the CPT log. The grain size distribution can be seen in Figure 7.

The most interesting correlation to be examined in this project is that of the Atterberg limits to the CPT. Atterberg test results classified the bottom 1/3 portion of the tube as nonplastic. The range of nonplastic behavior according to the Atterberg limit tests performed in the laboratory, is slightly above the transition point in the CPT data. This can be caused by a number of things. One possibility is the zone of influence of the CPT as discussed earlier. If the soil progressively gets sandier below the Shelby tube, then we can expect an overestimation of sand behavior type at the recorded transition point. Another possibility is that the CPT results and Shelby tube depth are slightly skewed. In this case, if the CPT data were to be shifted up slightly, then the tests would tend to agree more. For the latter, it would behoove the party anxious for a conclusion to perform a shift in the data; however, this can be an incorrect assumption and skewing data to match desired results is not in scientific interest.

V. Discussion and Conclusions

In this project, multiple soil mechanics parameters were tested and compared to the most common in situ test method used today for subsurface investigation, the cone penetration test. Although there were very few inconsistencies between the CPT data and the laboratory test results, all tests, being the cone penetration test, visual-manual description, hydrometer analysis, sieve analysis, Atterberg limits, and water content seemed to be in overall agreement. The transition in CPT data that occurs at approximately 22.9 ft was also near any transition in data in all laboratory tests.

Future Study

As previously discussed, the remodeling of the "Chinese criteria," which is most commonly used today, liquefaction susceptibility of fine-grained soils first introduced by Wang has been underway in the recent years. As part of current research being conducted at he University of California at Davis, the soils of Potrero Canyon have been chosen as a practical candidate for this research, mainly because of the soft and silty geology of the area. The soils tested in this project did fill most of the criteria specified by Seed based on data obtained from subsurface investigation after the Adapazari earthquake in Turkey presented by Bray. In the criteria, the zone on the Atterberg plasticity chart in Figure 8, labeled as potentially liquefiable, is noted that the water content of the soil must be greater than 0.8 times the liquid limit; in this project all of the soil tested from boring SCPT 2a Tube 7 met that criteria. The chart also specifies that the criteria established is only applicable for soils with a fines content of less than or equal to 20% or 35% depending on the soil's plasticity index. The clay content of these soils was consistently about 10%; however, the "fines" content, that is including clay and silt content, averaged for the entire tube sample is about 80%.

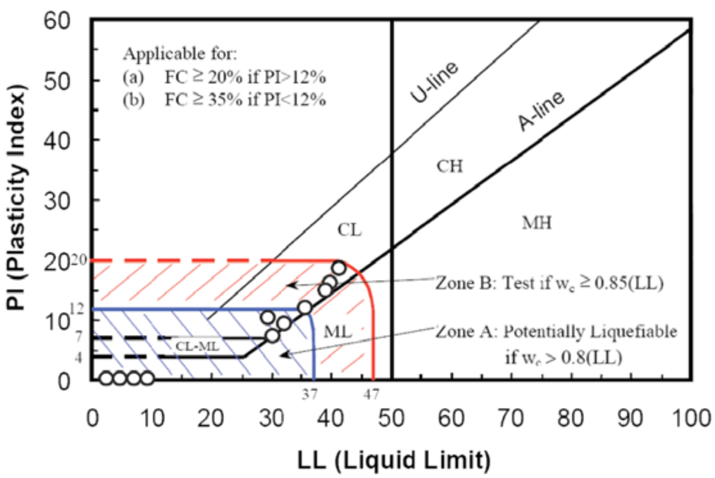

Figure 8: Plasticity chart with zones recommended by Seed et al. with Atterberg results plotted

While the fines content of the soils tested in this study are well over the criteria established by Seed, the data from this NEES project can still be used in order to further investigate the correlation of percent silt in a soil to its susceptibility to liquefy or experience cyclic failure under cyclic loading. In Figure 8, points from laboratory tests from this project have been plotted to illustrate how practical these soils pertain to its future study. The data provided by this NEES project can also be used to distinguish, if it is susceptible to liquefaction, whether the soil liquefies (that is it behaves like a sand under cyclic loading) or if it experiences cyclic failure (meaning that the soil behaves like a fine-grained soil under cyclic loading).

Acknowledgments

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APPENDIX 1

Example of a hydrometer data sheet with proper headings

University of California Davis
Geotechnical Engineering Laboratory

Hydrometer Test

Project: Potrero Canyon
Boring No.: SCPT 2a
Sample No.: H-11 S-23
Sample Depth: 21.69 ft. (6.61 m)
Specific Gravity:
Dispersing Agent: $(NaPO_3)_6$

Date: 7/30/2007
Tested By: RMF
Soil Description: SANDY SILT (ML)

Hydrometer Number: 529488
Meniscus Correction:

Table 1: Sieve Analysis

	Tare #	Wt. Dry Soil + Tare (g)	Wt. Tare (g)	Wt. Dry Soil (g)
Wt. Dry Soil > #200 Sieve (g)	C3	204.26	198.45	5.81
Wt. Dry Soil < #200 Sieve (g)	B	294.42	252.27	42.15
Total Wt. Dry Soil (g)				47.96
Wt, Dry Soil < #40 Sieve (g)	P1	210.32	204.79	5.53

Table 2: Experimental Results

Time (min)	Soil Hydrometer Reading	Reference Liquid Reading	Corrected Hydrometer Reading, R
1	36	4	32
2	32	4	28
4	28	4	24
8	24	4	20
16	20	3.5	16.5
32	18	2	16
64	16	2	14
96	14	2.5	11.5
128	12.5	2	10.5
256	12	2	10
610	9	3.5	5.5
1440	8	3.5	4.5

Table 3: Experimental Calculations

Time (min)	Effective Depth, L (cm)	Particle Diameter, D (mm)	Percent Passing, P
1	10.4	0.04160125	66.7222686
2	11.1	0.03039038	58.381985
4	11.7	0.02206239	50.0417014
8	12.4	0.01606037	41.7014178
16	13	0.0116279	34.4036697
32	13.3	0.0083165	33.3611343
64	13.7	0.00596843	29.1909925
96	14	0.00492627	23.9783153
128	14.25	0.0043042	21.8932444
256	14.3	0.00304886	20.8507089
610	14.8	0.00200935	11.4678899
1440	15	0.0013166	9.38281902

*K value taken to be 2.65

Table 4: Grain Size Classification

% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				0.6%	11.5%	76.4%	11.5%

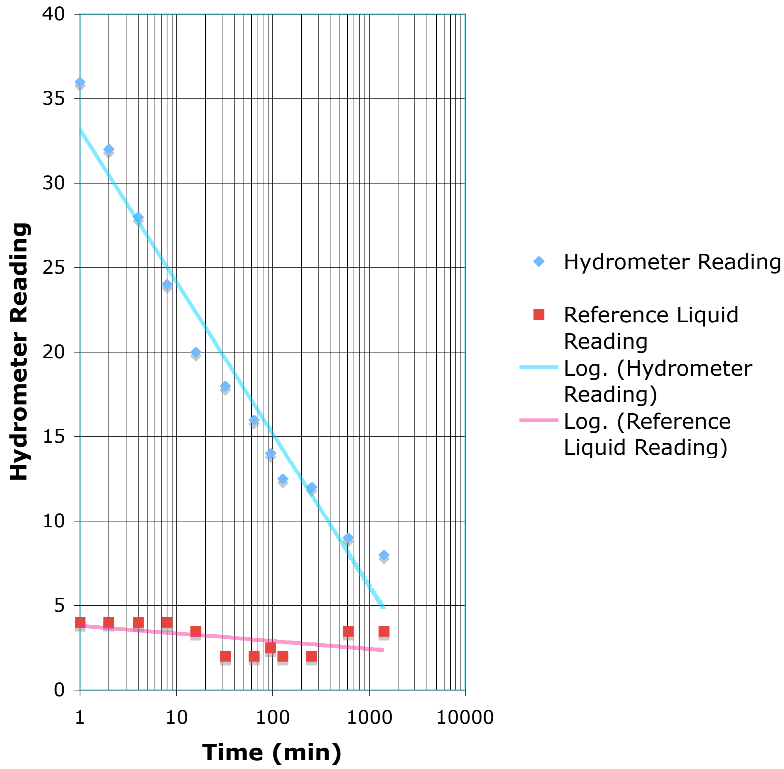


Figure 1: Plot of hydrometer readings

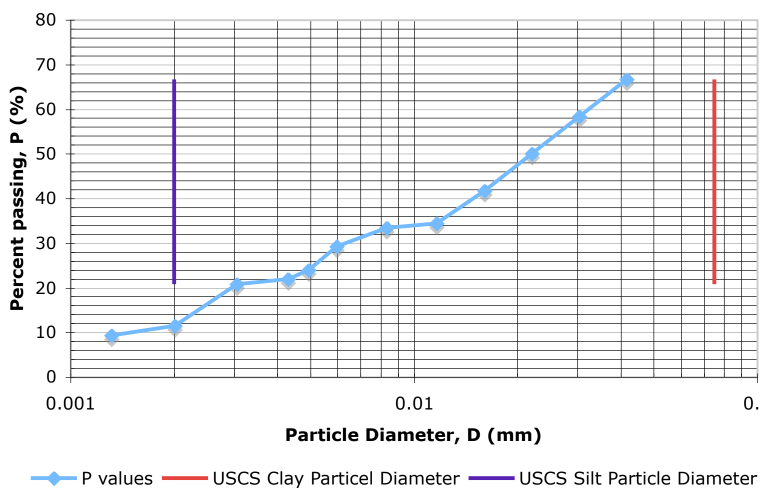


Figure 2: Grain Size Distribution

APPENDIX 2

CPT data and boring log provided by ENGEO Inc.

ENGEO
Expect Excellence

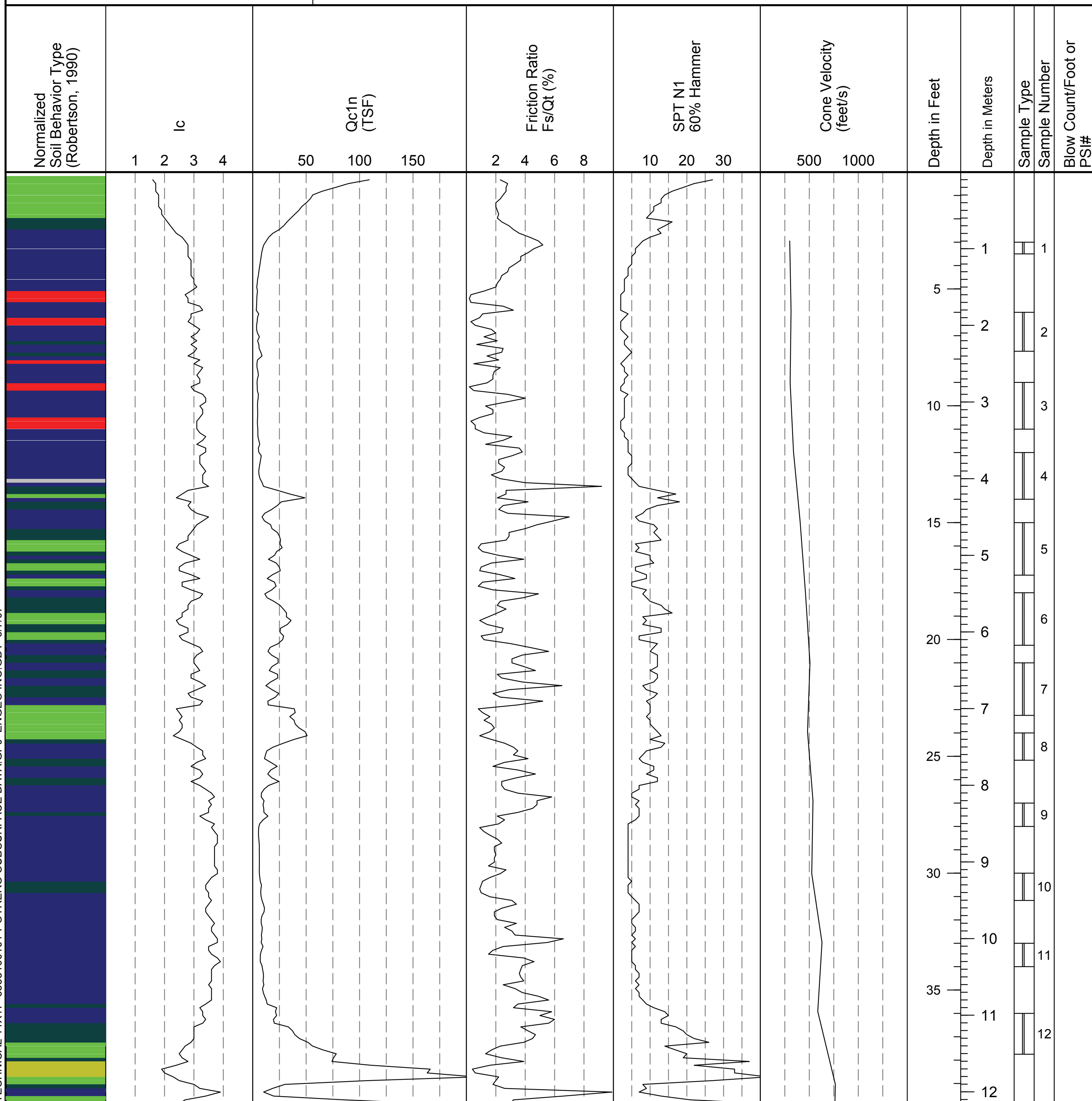
LOG OF BORING SCPT-2a

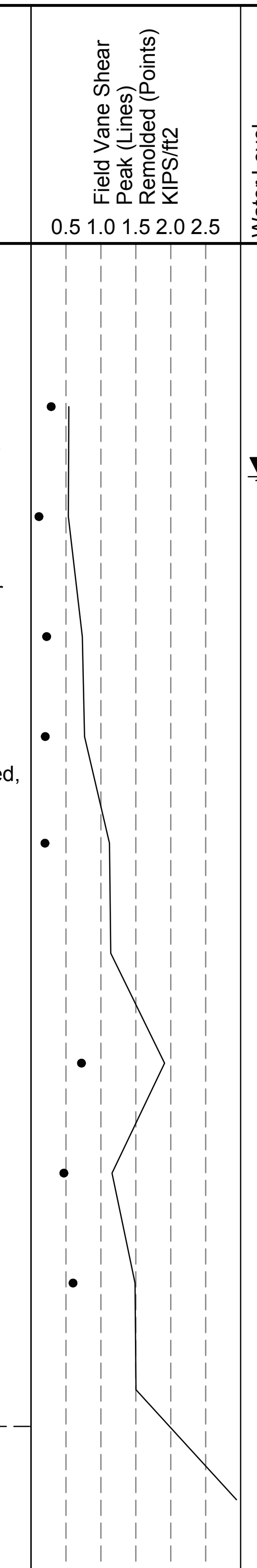
Newhall Land
Potrero Canyon
Los Angeles County, CA
6538.1.001.01

DATE DRILLED: 4/16/2007
HOLE DEPTH: Approx. 70¼ ft.
HOLE DIAMETER: 2.0 in.
SURF ELEV (FT): 1017 ft.

CPT PUSHED: 4/5/2007
CPT CONTRACTOR: Holguin, Fahan & Associates
CPT DEPTH: 64 ft.
WATER LEVEL: 7 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: WDC Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip





Sample Number	DESCRIPTION	Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx	Additional Tests
	SILT (CL/ML), pale brown, dry, fine- to coarse-grained sand on shaker table								
1	SILTY CLAY (CL/ML), dark brown, driller reports soft push								
2	SILTY CLAY (CL/ML), dark brown, fine-grained sand at bottom of sampler, hole in bottom of sample that drained water	32 32 30	18 16 20	14 16 10	79 68 73				0.0 / 2.5 / 18.3 / 60.7 / 18.5 (Co/M/F/S/Cl) 1.8 / 7.0 / 19.3 / 50.2 / 17.7 (Co/M/F/S/Cl) 1.0 / 7.4 / 17.8 / 63.1 / 10.2 (Co/M/F/S/Cl)
3	SILTY CLAY (CL/ML), dark brown, coarse-grained sand in shaker table								
4	SILTY CLAY (CL/ML), dark brown, with fine- to medium-grained sand, stiffer pushing than previous, recovery in the middle of the run	33 24 34	17 16 17	16 8 17	80 70 89				0.1 / 2.0 / 17.8 / 61.3 / 18.8 (Co/M/F/S/Cl) 0.0 / 0.3 / 30.0 / 57.3 / 12.4 (Co/M/F/S/Cl) 0.0 / 2.2 / 8.8 / 72.4 / 16.6 (Co/M/F/S/Cl)
5	SILT (ML), dark yellowish brown, trace gravel, may be compressed, sampler continued to advance on own weight after pushing								
6	SILTY CLAY (CL/ML), dark yellowish brown								
7	SILT (ML), dark yellowish brown, with fine-grained sand, driller reports increased push resistance								
8	SILTY CLAY (CL/ML), dark yellowish brown								
9	SILTY CLAY,CL/ML,DARK,YELLOWISH,BROWN (CL/ML), dark yellowish brown, last 2" fell out, very soft, driller instructed to inspect check valve								
10	SILTY CLAY (CL/ML), dark yellowish brown, very soft, with fine-grained sand	45	17	28	86				1.1 / 1.9 / 10.5 / 58.5 / 27.6 (Co/M/F/S/Cl)
11	SILTY CLAY (CL/ML), dark yellowish brown								
12	SILTY SAND (SM), dark yellowish brown, refusal during push, sample allowed to drain								
	Subrounded gravel up to 1 cm-diameter in shaker table								

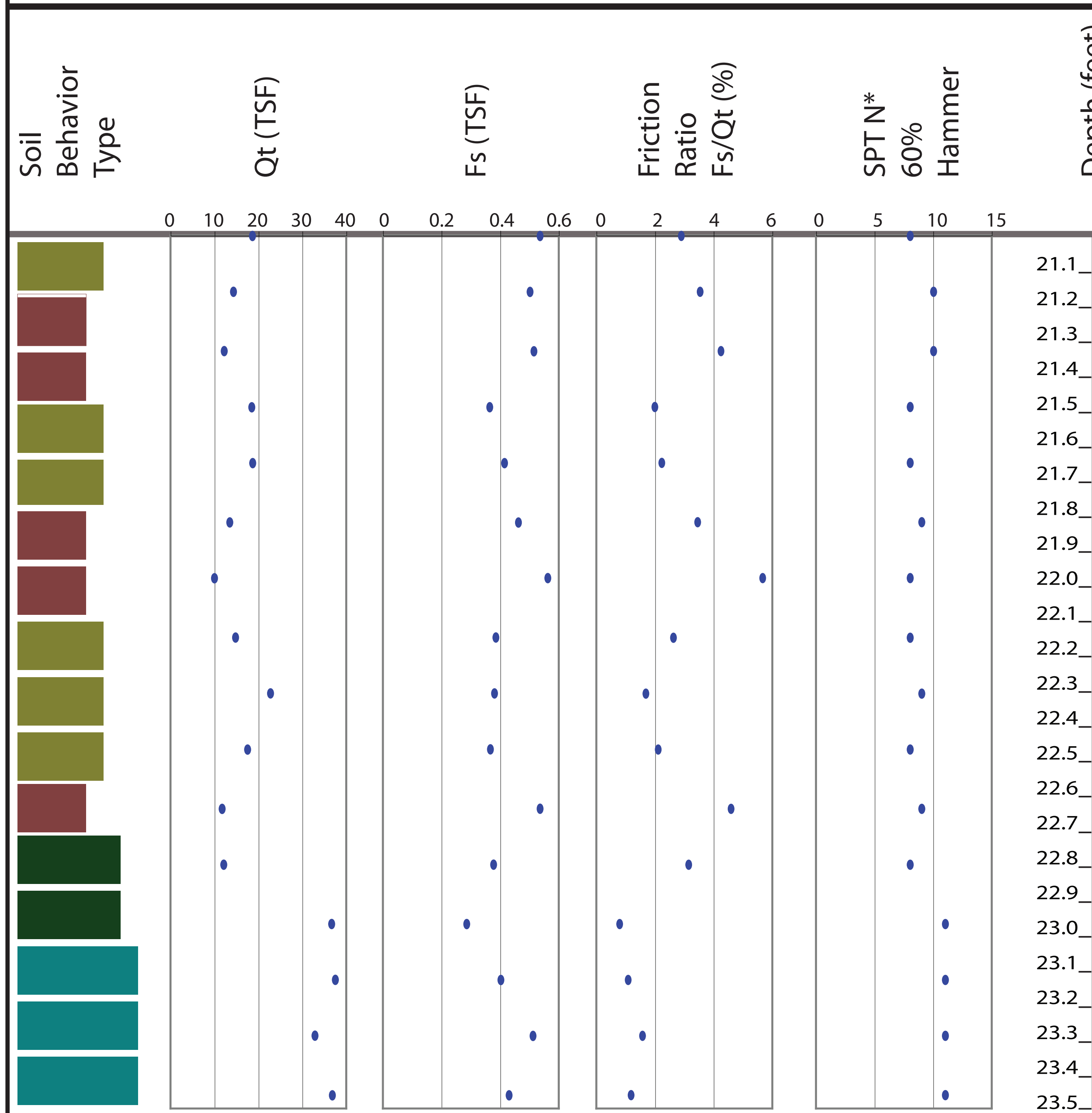
APPENDIX 3

Extrapolated CPT data and boring log for SCPT 2a – Tube 7

Log of Boring SCPT 2a Tube 7

Date Drilled:	4/16/2007	CPT Pushed:	4/5/2007
Hole Depth:	Approx. 70.25 ft	Water Level:	7 ft.
Hole Diameter:	2.0 in.	Drilling Method:	Mud Rotary
Surf Elevation (ft):	1017 ft.	Hammer Type:	140 lb. Auto Trip
Logged/Reviewed by:	P. Lam/PJS	Lab Tests Performed by:	R. M. Fernandez

NEES



Description

SILT (ML), dark yellowish brown, lower moisture with rust modeling, top disturbed

SANDY SILT (ML), dark yellowish brown with lighter stripe through sample, dark“smearing throughout

SANDY SILT, dark yellowish brown, seemingly more wet

Darker SILT with CLAY (CL-ML)T

CLAYEY SILT (CL-ML), dark yellowish brown

SANDY SILT (ML), dark yellowish brown, large black organic chunk present

CLAYEY SILT (CL-ML), dark yellowish brown

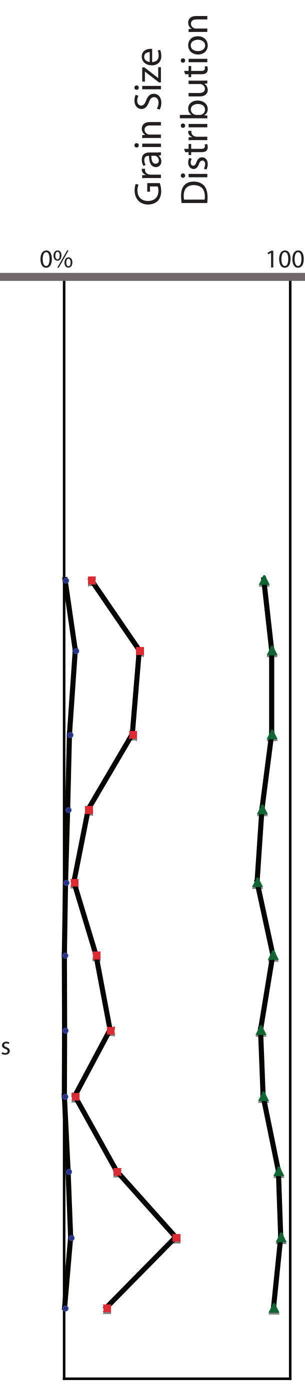
CLAYEY SILT (CL-ML) and SANDY SILT, dark yellowish brown with sandier areas noticeably darker than clayey material

SANDY SILT (ML), traces of organics

SANDY SILT with CLAYEY SILT regions, small rocks present ~1/16”

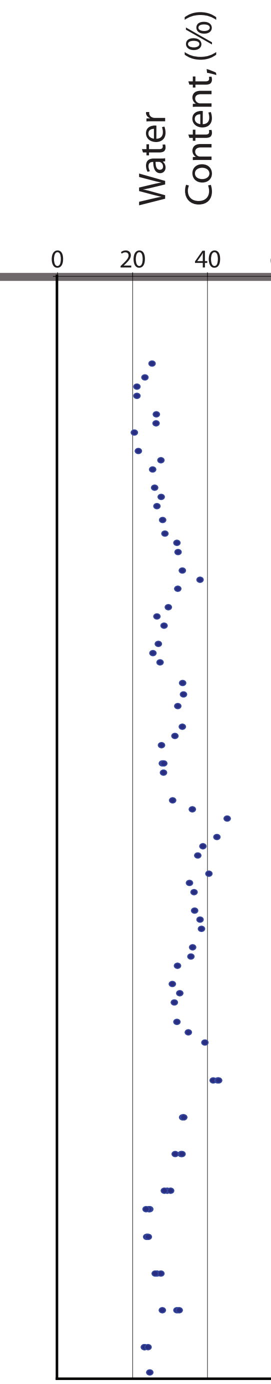
CLAYEY SILT (CL-ML), dark yellowish brown

SANDY SILT (ML) with CLAYEY SILT(CL-ML), dark yellowish brown, large void ~0.75 inches in diameter, small clcium deposits present



Atterberg Limits (%)

LL	PL	PI
38.9	23.9	15.0
29.3	19.3	10.4
32.0	22.6	9.4
35.5	23.6	12.0
41.2	22.6	18.6
30.1	22.8	7.3
39.7	23.5	16.2
NP	20.8	NP
NP	23.2	NP
NP	24.2	NP
NP	20.7	NP



1 silty clay to clay
2 clayey silt to silty clay
3 sandy silt to clayey silt
4 silty sand to sandy silt

▲ Silt/Clay Boundary
■ Fine Sand/Silt Boudary
● Med Sand/Fine Sand Boundary