CORNERSTONE EARTH GROUP

Transmittal

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Address:	777 California Avenue Palo Alto, CA 94304		
Project:	Lester Property Residential Development		
Project No.:	168-6-2		

Hello Vince,

Please find our enclosed updated geotechnical report for your use. Hard copies will follow. Please let us know if you have any questions. Thank you Vince and hope to catch up with you soon.

Sincerely,

Cornerstone Earth Group

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TYPE OF SERVICES	Updated Geotechnical Investigation Report
PROJECT NAME	Lester Property Residential Development
LOCATION	Blossom Hill Road and Southcrest Way San Jose, California
CLIENT	SummerHill Homes
PROJECT NUMBER	168-6-2
DATE	July 30, 2010

CORNERSTONE EARTH GROUP

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Client	**SummerHill Homes**
Client Address	**777 California Avenue** **Palo Alto, California**
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Date	**July 30, 2010**

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APPENDIX A: FIELD INVESTIGATION
APPENDIX B: LABORATORY TEST PROGRAM

Type of Services	**Updated Geotechnical Investigation Report**
Project Name	**Lester Property Residential Development**
Location	**Blossom Hill Road and Southcrest Way** **San Jose, California**

SECTION 1: INTRODUCTION

This geotechnical report has been prepared for the sole use of SummerHill Homes for the Lester Property Residential Development project referenced above. The site location is shown on the Vicinity Map, Figure 1.

For our use, we have been provided a Conceptual Site Plan, Sheet 3.0, titled, "General Development Plan, Exhibit C, PDC -, Lands of Lester, Summerhill Homes," prepared by HMH, Project No. 3566.10, dated April 28, 2010.

1.1 PROJECT DESCRIPTION

Based on the information provided, the project will consist of redeveloping the approximately 8½-acre, multiple-parcel site for a new residential development. The quantity and type of units have not been finalized at this time. However, new structures will likely consist of single-family structures. Based on our experience with similar SummerHill Homes projects, new structures will be two-story, wood-framed construction supported on mat foundations. New interior access roads, utilities, and appurtenant improvements also are planned. We understand an alternative for a future park trail is being considered for the parcel currently owned by the City. No below-grade basements or depressed garages are anticipated at this time.

1.2 SCOPE OF SERVICES

Our original scope of services was presented in our proposal dated January 19, 2010 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 LITERATURE REVIEW

Published geologic maps were researched and reviewed for this investigation and are listed in the “References” section of this report.

1.4 SITE RECONNAISSANCE AND GEOLOGIC MAPPING

Our geologist performed a site reconnaissance on March 16, 2010 to map the aerial extent of geologic deposits and obtain other details regarding the site geologic conditions and potential geologic and geotechnical hazards at and immediately adjacent to the site. The results of the reconnaissance and mapping are discussed in the following sections.

1.5 FIELD EXPLORATION PROGRAM

Field exploration consisted of two exploratory borings drilled on March 22, 2010 with truck-mounted, hollow-stem auger drilling equipment and five Cone Penetration Tests (CPTs) advanced on March 16, 2010. The borings were drilled to depths of approximately 29 to 35 feet; the CPTs were advanced to depths of 25 to 50 feet.

The borings and CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan and Geologic Map, Figure 2. Details regarding our field program are included in Appendix A.

1.6 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation and pavement design. Testing included moisture contents (ASTM D2216), dry densities (ASTM D2937), washed sieve analyses (ASTM D1140), Plasticity Index (ASTM D4318) and R-value testing (Caltrans Test Method 301). Details regarding our laboratory program are included in Appendix B.

1.7 CORROSION EVALUATION

Three samples from our borings at depths of 4 to 7 feet were tested for resistivity, (ASTM G57), pH (ASTM G51), sulfate (Caltrans 417 modified), and chloride (Caltrans 422 modified) to evaluate and provide preliminary corrosion screening for buried metallic and Portland concrete cement structures. Laboratory test results and recommendations are provided in Section 3.5.

1.8 ENVIRONMENTAL SERVICES

Environmental services were not requested or performed by Cornerstone Earth Group for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL GEOLOGIC SETTING

The site is located within the relatively flat alluvial plain of the Santa Clara Valley, a northwest-southeast trending valley within the Coast Range Geomorphic Province. The Santa Clara Valley is within the San Francisco Bay Block, which is bounded to the east by the Hayward and Calaveras faults and to the west by the San Andreas Fault. According to McLaughlin et al. (1999), a Neogene age range-front thrust system, which includes the Shannon, Sargent, Hooker Gulch, Berrocal, and Monte Vista faults, lies in the foothills of the Santa Cruz Mountains along the western boundary of the valley. The subject site lies in the southwestern portion of the valley where it abuts the Santa Teresa Hills, a small northwest trending spur off the Santa Cruz Mountains further to the south.

More locally, the Almaden Block represents a further subdivision of the San Francisco Bay block (Wentworth et al., 1999). The New Almaden structural block forms the northeastern flank of the southern Santa Cruz Mountains between the Sierra Azul block and the southern Santa Clara Valley, with its northeastern boundary concealed beneath the valley and probably close to its northeastern margin. It abuts the San Andreas fault northwest of the Sierra Azul block. The block consists largely of masses of Franciscan greenstone and graywacke of the Permanente and Marin Headlands terranes that are immersed in abundant melange, all belonging to the Franciscan Central belt. These rocks, together with long seams and patches of serpentinite here considered part of the Coast Range ophiolite, have been tectonically imbricated and interleaved. Subsequently, within the last 3 to 5 MY, these rocks and unconformably overlying marine Miocene and nonmarine Plioene to middle-Pleistocene Santa Clara Formation were folded into a series of open to tightly-compressed folds and repeated across northeast-vergent reverse faults of the Sargent, Berrocal, and Shannon fault zones. The eastern edge of the valley is bounded by active and potentially active faults, such as the Calaveras fault.

The broad alluvial plain of the Santa Clara Valley surrounding the site consists of Holocene and Pleistocene alluvial deposits (Helley and Wesling, 1990) that are comprised of a deep section of unconsolidated and semi-consolidated stream and basin deposits that were deposited largely by ancestral Coyote Creek and Guadalupe River on top of the Franciscan Complex rocks that form the bottom of the basin (Dibblee, 1972; Rogers and Williams, 1974; and the Dibblee Foundation, 2005).

The tectonic regime in the San Francisco Bay region is primarily translational, expressed by mostly right-lateral strike-slip movement along the faults of the San Andreas fault system, including the nearby Calaveras and Hayward faults. A small component of compression is active in the region, resulting in continued folding and faulting of the geologic units. Compressional reverse or thrust faulting has occurred along the Monte Vista-Shannon fault to the southwest of the site, but its' present activity at this location is poorly understood. Similar evidence of compression is evident along the Silver Creek fault in the hills bordering the northeast side of Santa Clara Valley.

At the subject site the quaternary deposits consist of alluvial fan deposits (Crittenden, 1951; Davis and Jennings, 1954; Helley and Brabb, 1971; Dibblee, 1972, 2005; Rogers and Williams, 1974; Helley, 1990; Helley & Wesling, 1990; Helley et al., 1994; Wentworth et al., 1999, Knudsen et al., 2000). A vicinity geologic map is shown in Figure 4 (a portion of the map by Dibblee, 2005) and Figure 5 (a portion of Helley & Wesling, 1990).

2.2 REGIONAL SEISMICITY

The project site is located within the seismically active San Francisco Bay region that has been subjected to recurring large earthquakes. The Working Group on California Earthquake Probabilities (2003) developed estimates of earthquake probabilities in the San Francisco Bay area for the period from 2002 to 2031. The probability of a magnitude 6.7 earthquake being generated on an unknown fault or any of the potentially active faults in the San Francisco Bay region including, the Berryessa, Crosley, Clayton, Quimby, Shannon, Evergreen, and Silver Creek faults in San Jose is 14 percent. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2007) forecast a 99.7 percent chance of a magnitude 6.7 or greater earthquake somewhere in California before 2038. During such an earthquake the danger of fault ground rupture is limited to sites immediately adjacent to these fault zones (the project site is not located next to these fault zones), but strong ground shaking would occur city-wide, in San José, California.

The major active faults that could impact the project area include the San Andreas Fault, the Hayward Fault, the Monte Vista-Shannon Fault, and the Calaveras Fault. The San Andreas fault produced the 7.1 magnitude 1989 Loma Prieta earthquake, and the Calaveras fault produced the 1984 magnitude 6.2 Morgan Hill earthquake. It can be expected that earthquakes could produce strong ground shaking at the Project site during the lifetime of the structures built there.

Seismologists and Geologists recognize the City of San José and the entire South Bay to be within one of the most seismically active areas in the United States. The 2007 California Building Code (Section 1613) provides a classification system termed "Site Class", where each site is classified based on the soil types and their engineering properties as defined in Section 1613.5.2 and Table 1613.5.2. There are six site classifications, Site Class A through Site Class F. In general, much of the City of San José would be classified as Site Class D (stiff soil) unless the geotechnical investigation provides data that justifies classifying the site otherwise. The California Geological Survey (CGS) has an ongoing program to map Seismic Hazard Zones at 7.5 minute quadrangle scale (1:24,000) in the Bay Area. These maps show areas having a potential for earthquake-induced liquefaction (Figure 6). The City of San José has also established Geologic Hazard Zones that identify potential geologic hazard areas within the City's Sphere of Influence. Areas within Geologic Hazards Zones may require special study prior to development.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. Table 1 presents the State-considered active faults (CDMG, 1997) within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Monte Vista/Shannon	2.7	4.4
Hayward (Southeast)	6.7	10.8
Calaveras (S)	9.3	14.9
Sargent	9.3	15.0
San Andreas	9.7	15.6
Zayante	13.4	21.5
Hayward	13.5	21.7
Calaveras (N)	15.4	24.8

A regional fault map is presented as Figure 7, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE HISTORY AND AIR PHOTO REVIEW

We reviewed aerial photos taken during the period when the outlying areas of San Jose made it greatest transition in terms of land use from agricultural to residential/commercial. Photos reviewed were taken in 1960, 1963, 1965, 1968, 1974 and 1981. Our review of aerial photos that extend back to 1960 indicate that the site has been used for agricultural purposes since that time. As of 1960 the subject site and the adjacent sites extending from the Canoas Creek on the east, to the current Cahalan Street contained cultivated row crops throughout. An agricultural water well was located where the present Highway 85 crosses the area. By 1968 the site was no longer being actively cultivated (open fields of grasses and weeds) and a residence had been constructed in the eastern portion of the larger parcel near present Chesbro Avenue (just east of the current eastern property line). By 1974 the site was once again in row crop and the adjacent parcels were increasingly becoming developed for residential purposes. By 1981 the site and surrounding areas were as they are currently in terms of configuration and development. The site existed as open, undeveloped land.

3.2 SURFACE DESCRIPTION AND TOPOGRAPHY

The generally rectangular site is approximately 8½ acres in area and consists of an open field which is fenced on all sides. A fence also separates the City of San Jose parcel to the west. A short paved access road extends into the northern portion of the site that provides access to a small brick building owned by the Valley Transit Authority (Substation #12). Although in a larger sense topographically flat, the site has an uneven ground surface, which undulates resulting in changes of elevation on the order 1 to 3 feet (locally) due to disturbance during past agricultural activities. The site contains no other structures or evidence suggesting past construction. The

access road in the northern portion of the site has longitudinal cracks along its edges suggesting.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional studies by Dibblee, 1972, and 2005 (see Figure 4) and Helley and Wesling, 1990 (see Figure 5) suggest Quaternary age alluvial deposits as prevalent in the area surrounding the site. We have adopted the nomenclature of Helley and Wesling, (1990) in assigning geologic unit names for our characterization of the site. As shown on the Site Plan and Geologic Map (Figure 2) and Cross Section A-A' (Figure 3), the site as underlain by Quaternary basin deposits ("Qhb"). Alluvial fan levee deposits ("Qhl") are mapped in areas located just west of the subject site. The basin deposits were deposited in topographic lows such as at the distal end of alluvial fans, between fan levees, and adjacent to floodplains. Typically organic rich, dark-colored clay and silt that settles out of seasonal standing water collected in the basins. Liquefaction susceptibility is mapped as high (Figure 6). Our subsurface investigation revealed the basin deposits consist of a relatively thick surficial layer of fat clay, which is underlain by gravels and sands with varying proportions of silt and clay. Corrected blow counts and pocket penetrometer values indicate the soils are stiff to hard (cohesive soils) and loose to very dense (cohesionless soils). Some silt and clay beds were encountered at depth in Boring EB-2. These sediments varied from very stiff to hard (cohesive soils) and loose to very dense (cohesionless soils).

We performed one Plasticity Index (PI) test on a surficial sample from EB-1 at a depth of 3½ feet. The test results were used to evaluate expansion potential of surficial soils. The test resulted in a PI of 41, indicating a high expansion potential to wetting and drying cycles.

3.4 GROUND WATER

Based on available ground water map (CGS, 2000), the historic ground water table in this immediate area is estimated between 10 to 20 feet below the ground surface. According to the generalized characterization by Helley and Wesling (1990), areas with basin deposits typically have a water table within 10 ft of the surface and are (or were) subject to flooding. Our subsurface investigation revealed depths to ground water at approximately 10 feet, which stabilized over the brief period of drilling. Fluctuations in ground water levels could occur due to many factors including recharge from nearby creeks, perched water, and regional ground water variations, and rainfall or irrigation.

3.5 CORROSION EVALUATION

We tested three samples from our borings collected at depths ranging from 4 to 7 feet for in-situ resistivity, pH, soluble sulfates, and chlorides. Field and laboratory test results are summarized in Table 2.

Table 2: Summary of Corrosion Test Results

Sample/Test Location Number	Depth (feet)	Soil pH	Minimum Resistivity[1] (ohm-cm)	Chloride (mg/kg)[2]	Sulfate (mg/kg)[2]
EB-1	4	7.2	969 /	29	< 5
EB-1	7	7.1	1,451 /	4	< 5
EB-2	4	7.4	1,496 /	15	< 5

Note: 1 Laboratory resistivity measured at 100% saturation
2 mg/kg = milligrams per kilogram = parts per million (ppm)

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

3.5.1 Preliminary Soil Corrosion Screening

Based on field resistivity and laboratory test results summarized in Table 2, soils are considered severely corrosive to buried metallic improvements. Other corrosion parameters (pH, chloride and sulfate ion content) do not indicate a significant corrosion potential to buried metallic or concrete structures. In accordance with the 2007 CBC, Chapter 19, Section 1904.3, sulfate concentrations are “negligible”; therefore, Type II cement may be used.

3.5.2 PG&E Subsurface Transformers Requirements

We have obtained and reviewed PG&E “Soils Report Requirements” for design of underground transformers. Based on those requirements, subsurface transformers “shall not” be used when subsurface soils are generally corrosive and any of the following conditions exist:

- soil resistivity is less than 2,001 ohms-cm (ASTM G57)
- or two of the following criteria
  - pH > 8.5 or pH < 5.0 (ASTM G51)
  - sulfate > 1,000 ppm (ASTM D516)
  - chloride > 100 ppm (ASTM D512)
  - sulfate – test positive
- heavy erosion occurs which may fill the enclosure with soil
- a high water table exists causing repeated flooding of the enclosure
- heavy snowfall occurs (generally above 3,000 foot elevation)
- a severe mosquito problem exists

Based on current PG&E requirements and corrosion results referenced above, it appears that underground transformer is not feasible at the site without soil modification and/or prior approval from PG&E. In addition, additional lab testing may be required by PG&E at actual transformer locations and depths prior to approval. We recommend a corrosion engineer and/or utility consultant be retained to provide additional recommendations, as required.

SECTION 4: GEOLOGIC HAZARDS

Based on our review, we understand that the site is not located within a State of California earthquake-induced Landslide Hazard Zone, but is located in a State of California Liquefaction Hazard Zone as well as the City of San Jose Hazard Zone of Liquefaction.

4.1 FAULT SURFACE RUPTURE

The available published regional geologic maps covering the area of the site indicate no Faults are mapped as transecting the site or immediately adjacent to the site (Crittenden, 1951; Davis and Jennings, 1954; Helley and Brabb, 1971; Dibblee,1972, 2005; Rogers and Williams, 1974; Helley & Wesling, 1990; Helley et al., 1994; Wentworth et al., 1999, Knudsen et al., 2000). Accordingly, the site is not located within a State Designated Alquist Priolo Earthquake Fault Zone nor is it in a City of San Jose Fault Hazard Zone or a Santa Clara County Fault Rupture Hazard Zone.

Our review of stereo aerial photo pairs and our site reconnaissance did not reveal any evidence suggestive of active surface fault traces at or adjacent to the site.

The 2007 California Building Code has assigned near-source factors to mitigate potential seismic shaking should an earthquake occur along this fault. The subject site is not located within a CBC-designated near-source zone.

Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (>M7) earthquakes have been recorded and can be expected to occur in the future. Figure 5 shows the approximate locations of these faults and significant historic earthquakes recorded within the larger Bay Area Region.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. As discussed, the site is located in a highly seismically active region of Northern California, and the Monte Vista/Shannon Fault is located less than 4½ kilometers from the site. A Probabilistic Seismic Hazard Analysis (PSHA) was performed, utilizing EZ-FRISK version 7.37 by Risk Engineering Inc., to evaluate the likelihood of various ground motion levels at the site as reflected in peak horizontal ground acceleration (PHGA). This approach takes into account the geological slip rate of all active faults and background seismicity within 100 km (75 miles) of the site and the site-specific response characteristics.

The PSHA results are based on ground motions, which corresponds to the anticipated response at a free field (i.e., ground motions are not influenced by the presence of a structure, topographic features, or ground failure).

The site coordinates of 37.252403° and -121.845647° were utilized in this analysis. The site is underlain by deep alluvium. The average shear wave velocity for the upper 100 feet of soil deposit was estimated to be approximately 280 meters per second based on the correlations proposed by Wills and Silva (1998). Next generation attenuation (NGA) relationships by Campbell-Bozorgnia (2008), Chiou-Youngs (2007), and Boore and Atkinson (2008) were utilized in the analyses. The average of these attenuation relationship results was utilized in our analyses. These attenuation relationships are based on the geometric mean of the peak horizontal accelerations.

The average PHGA corresponding to a 10 percent probability of exceedance in 50 years (475-year ARP event) is 0.42g.

4.3 LANDSLIDING

The site is essentially level and located far from any mapped landslides or moderate slopes (Cooper-Clark and Associates 1974; Dibblee, 1972 & 2005; Nilsen, 1975; Helley & Wesling, 1990; California Geological Survey, 2000). Therefore, in our opinion, there is a low potential for landsliding impacting the site and planned residential development.

4.4 LIQUEFACTION

The site is within a State-designated Liquefaction Hazard Zone (CGS, San Jose East Quadrangle, 2001) as well as a City of San Jose mapped Liquefaction Hazard Zone. Our field and laboratory programs addressed this issue by sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT correlations, and performing various tests to further classify the soil properties.

4.4.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 3 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.4.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 10 feet. Following the procedures in the 2008 monograph, Soil Liquefaction During Earthquakes (Idriss and Boulanger, 2008) and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation.

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the "Estimated Ground Shaking" section above, and is corrected for overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are typically considered unreliable in sands below ground water. The tip pressures are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers. Selected soil samples collected at the site were tested to evaluate grain size, as well as visually observed for confirmation of CPT soil behavior types.

The results of our CPT analyses (CPT-1 through CPT-5) are presented as Figures 8 through 12 of this report.

4.4.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement ranging from approximately ⅓ to 1½ inches based on the Ishihara and Yoshimine (1990) method. As discussed in the SCEC report, differential movement for level ground sites over deep soil sites will be about half of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ¼- to ¾-inch over a horizontal distance of 25 feet.

4.5 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone D, described as an area of "undetermined, but possible, flood hazards." The "Qhb" is described by Helley and Wesling as being subject to flooding.

The Association of Bay Area Governments has compiled a database of Dam Failure Inundation Hazard Maps (ABAG, 1995). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is not located within a dam failure inundation area.

We recommend the project civil engineer be retained to confirm the above information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 PRIMARY GEOTECHNICAL CONCERNS

From a geotechnical viewpoint, the proposed development is feasible provided the project design is performed in accordance with recommendations presented in this report. Primary geotechnical concerns are:

- Potentially liquefiable soils
- Highly expansive soil
- Shallow ground water
- Disturbed surface soils

A brief discussion of these concerns is presented herein with additional recommendations following in subsequence sections of this report.

5.1.1 Potentially Liquefiable Soils

The site currently is mapped within a State of California Seismic Hazard Zone for liquefaction (CGS, San Jose East Quadrangle, October 2001) and the City of San Jose liquefaction zone.

Our liquefaction analysis indicates that there is a potential for liquefaction of localized sand and silt layers during a significant seismic event. Based on our analysis, total liquefaction-induced settlements on the order of ⅓ to 1½ inch is possible with potential differential settlements on the order of ¼ to ¾ inch. Additional recommendations are presented in the "Foundations" section of this report.

5.1.2 Highly Expansive Soil

As discussed, the surficial clays are highly expansive. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to planned structures and at-grade improvements, footings should extend below the zone of seasonal moisture fluctuation and slab-on-grades are underlain by a layer of non-expansive fill or use a stiffer foundation system such as mat foundations to accommodate the potential movement due to expansive soils. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from footings as well as limiting landscaping watering. Additional recommendations regarding this concern are provided in subsequent sections of this report.

5.1.3 Shallow Ground Water

Our explorations encountered ground water at depths ranging from approximately 10 to 13½ feet below existing site grades at the time of drilling. In addition, historic high ground water in the area is mapped at depths on the order of 15 to 20 feet below the ground surface. As a result, temporary dewatering and subgrade stabilization will likely be required for deeper underground utility installations and excavations.

5.1.4 Disturbed Surface Soils

The upper 1 to 3 feet of soils in localized areas across the site have been disturbed due to past agricultural activities. As a result, those soils are loose and will have a moderate potential for post-construction settlement and should be adequately compacted in areas where structures and site improvements are planned. Loose soils will require partial over-excavation and replacement as engineered fill. Recommendations addressing this concern are provided in the "Earthwork" section.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review geotechnical aspects of project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between small-diameter and widely-spaced borings and CPTs, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and

testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION, CLEARING AND PREPARATION

6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and any existing surface and subsurface improvements. Demolition of existing improvements is discussed in detail below. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. At the time of this investigation, the site is mostly covered by tall vegetation and we recommend surficial stripping extend about 2 to 3 inches below existing grade.

6.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the rootballs and any roots larger than ½-inch diameter removed completely. Grade depressions resulting from rootball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

6.1.3 Demolition of Existing Slabs, Foundations and Pavements

Where applicable, all slabs, foundations, and pavements scheduled for demolition should be completely removed from within planned improvement areas. Slabs, foundations, and pavements that extend into planned flatwork or pavement areas may be left in place provided there is at least 2 feet of engineered fill overlying the remaining materials and they are determined not to conflict with new utilities. A discussion of recycling existing improvements is provided later in this report.

6.1.4 Abandonment of Existing Utilities

All utilities should be completely removed from within planned improvement areas. For any utility line to be considered acceptable to remain within an improvement area, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the improvement area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within improvement areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the improvement areas may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risks associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

6.2 DISTURBED SURFACE SOILS

Localized areas of loose surficial soils should be expected across the site. To provide a more uniform bearing surface for structural support and to minimize the potential for significant differential settlement beneath new structures, the loose soils should be replaced with engineered fill.

We recommend loose soils within new structures be over-excavated and replaced as engineered fill. Prior to fill placement, the bottom of excavations should be scarified a minimum of 6 inches, moisture conditioned, and compacted. Over-excavation limits should extend at least 5 feet beyond building footprint and foundations, whichever is greater. Subgrade preparation and compaction should be performed in accordance with the following sections. If the over-excavated materials meet requirements in the “Materials for Fill” section, they can be reused as engineered fill.

6.3 REMOVAL OF UNDOCUMENTED FILLS

Our exploratory borings did not encountered undocumented fill. However and based on our experience with similar projects and site conditions, localized areas of undocumented fill should be anticipated and properly accounted for during design and planning. If localized fills are encountered during site grading they should be removed and replaced as engineered fill. Any fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, fills may be reused when backfilling excavations. Materials that do not meet these requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and not be reused. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below. Side slopes of fill removals in building and pavement areas should be sloped at inclinations of 1:1 (horizontal to vertical) or flatter to minimize abrupt variations in fill thickness.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Site B materials. A Cornerstone representative should be retained to confirm the preliminary site classification.

6.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

6.6 MATERIAL FOR FILL

6.6.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.6.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Imported and general fill to be used in conjunction with post-tensioned mats should be inorganic with a PI of 25, or less. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant's review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.6.3 Non-Expansive Fill Using Chemical Treatment

6.6.3.1 General

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the clay content and high expansion potential of the surficial soil, it is not likely that sufficient quantities of non-expansive fill can be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. It has been our experience that highly plastic clay will likely need to be mixed with quicklime, or approved equivalent, to fully react with the clay and adequately reduce the PI of the on-site soils to 15 or less.

6.6.3.2 Preliminary Chemical Treatment Recommendations

For your project budgeting and preliminary design purposes, we estimate the native expansive clayey soil will need to be mixed with approximately 4 to 5 percent quicklime (CaO) by weight to reduce the PI down to 15 or less. Depth of chemical treatment can be expected to be on the order of 12 to 18 inches thick. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of chemical treatment required.

6.7 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in Table 3. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General guidelines for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill	On-Site Expansive Soils	87 – 92	>3
(within upper 5 feet)	On-Site Low Expansion Soils	90	>1
General Fill	On-Site Expansive Soils	92	>3
(below a depth of 5 feet)	On-Site Low Expansion Soils	95	>1
Wall Backfill	Without Surface Improvements	90	>1
Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	On-Site Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of pavement subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Near Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	On-Site Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Near Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	On-Site Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Near Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)
4 – Using lightweight compaction or walls should be braced

6.8 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted

to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

To reduce the potential for water migration into building and pavement areas through the granular shading materials, we recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.]

6.9 SITE DRAINAGE

Surface water runoff should not be allowed to pond adjacent to building foundations, slabs-on-grade, or pavements. New hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

Based on our experience and understanding with similar SummerHill Homes projects, the preferred foundation alternative is post-tensioned mat foundations (PT mat). Based on the subsurface conditions encountered and our engineering judgment, in our opinion, post-tensioned mat foundations are feasible provided recommendations in the "Earthwork" section and herein are followed. If an alternative foundation system such as spread footing is desired, we can provide additional recommendations, as needed. Detailed foundation recommendations are provided below.

7.2 SEISMIC DESIGN CRITERIA

The project structural design will be based on the 2007 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped

spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Based on our borings, CPTs, and review of available local geology, the site is underlain by stiff to hard and medium dense to very dense alluvial soils to depths of at least 50 feet with SPT "N" values generally from 20 to greater than 50 blows per foot; therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program Earthquake Ground Motion Parameters, Version 5.0.8, revision date November 20, 2007, based on the site coordinates presented below and the site classification. Table 4 lists the various factors used to determine the seismic coefficients.

Table 4: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.252403°
Site Longitude	-121.845647°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.732g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g
Short-Period Site Coefficient – Fa	1.00
Long-Period Site Coefficient – Fv	1.50
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.732g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.900g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.155g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.600g

Note 1: For Site Class B, 5 percent damped.

7.3 MAT FOUNDATIONS

7.3.1 Post-Tensioned Mats

New residential structures can be supported on post-tensioned (PT) concrete mat foundations designed in accordance with the parameters in Table 5 below and based on procedures developed by the Post-Tensioning Institute (2004) and as referenced in the 2007 California Building Code. The PT mat foundations may bear on natural soil or engineered fill prepared in accordance with the "Earthwork" section of this report.

TABLE 5: POST-TENSIONED MAT DESIGN CRITERIA

Differential Movement Condition	Center Lift	Edge Lift
Edge Moisture Variation (feet)	7.3	3.8
Differential Soil Movement (inches)	0.8	1.5

To reduce potential differential movement, all mats should be designed for a maximum average allowable bearing pressure of 1,000 psf for dead plus live loads; at column or wall loading, the maximum localized bearing pressure should be limited to 3,000 psf. When evaluating wind and seismic conditions, allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Additional reinforcing steel should be included, as required by the structural engineer, to help span irregularities and differential settlement.

7.3.2 MAT FOUNDATION SETTLEMENT

In addition to estimated differential static settlements of less than ½ inch across the mat area for reinforced concrete mats and the differential movement listed in Table 5, the mats should be designed to accommodate an estimated seismic differential movement of ½ inch over a horizontal distance of 25 feet.

7.3.3 LATERAL LOADING

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.35 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.4 MAT FOUNDATION CONSTRUCTION CONSIDERATIONS

Due to the presence of highly expansive soils, mat subgrade areas should be kept moist until concrete placement by regular sprinkling to prevent desiccation. If deep drying is allowed to occur, several days of moisture conditioning (flooding of the pads is not recommended) may be required to allow the moisture to re-penetrate the subgrade. If severe drying occurs, reworking and moisture conditioning of the pad may be required. Prior to placement of any vapor retarder and mat construction, the subgrade should be proof-rolled and visually observed by a Cornerstone representative to confirm stable subgrade conditions. The pad moisture should also be checked at least 24 hours prior to vapor barrier or mat reinforcement placement to confirm that the soil has a moisture content of at least 3 to 5 percent over optimum in the upper 12 inches.

SECTION 8: EXTERIOR PEDESTRIAN PAVEMENTS

8.1 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base underlain by a 6-inch of non-expansive fill layer. As an alternative, exterior concrete flatwork can also be underlain entirely by 12 inches of Class 2 aggregate base in-lieu of the non-expansive fill layer. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 9: INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

For your consideration, interior slabs-on-grade including post-tensioned mat foundations may be underlain by a moisture barrier consisting of a vapor retarder and crushed rock capillary break layer. The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the mat foundation performance.

- Place a minimum 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements.
- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels should not be allowed versus light broom or limited trowel finishing.
- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

We collected a bulk sample of the surficial soil at the estimated pavement subgrade elevation for Resistance-Value (R-Value) testing in accordance with California Department of Transportation Test Method 301. The test resulted in an R-value of less than 5 for the surficial soil sampled. Based on our engineering judgment and to account for variation in soil conditions, we judge an R-value of 5 is applicable for design.

Table 6 provides asphalt concrete pavement recommendations generally based on Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indicies (TI) for various pavement-loading conditions. We recommend the project civil engineer determine the appropriate TI and corresponding pavement section alternatives based on anticipated loading conditions.

Table 6: Asphalt Concrete Pavement Recommandations
Estimated Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	12.5	16.0
6.5	4.0	14.0	18.0
7.0	4.0	15.5	19.5

Note: 1 Caltrans Class 2 aggregate base; minimum R-value of 78

10.1.1 Temporary Pavement Section during Construction

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be use the pavements.

10.1.2 Asphalt Concrete Pavement on Expansive Subgrade

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

10.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). Recommendations for garage slabs-on-grade were provided in the "Concrete Slabs and Pedestrian Pavements" section above. We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 7: PCC Pavement Recommendations, Design R-value = 5

Allowable ADTT	Minimum PCC Thickness (inches)
0.8	5
13	5½
130	6

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the "Earthwork" section, and laterally restraining the PCC with curbs or concrete shoulders. The slab for the trash enclosure should be at least 6 inches thick and supported ion at least 12 inches of Class 2 aggregate base. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

10.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduce to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or “Deep-Root Moisture Barriers” that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of SummerHill Homes specifically for the Lester Property Residential Development in San Jose, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

SummerHill Homes have provided Cornerstone with plans, reports and other documents prepared by others. SummerHill Homes understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone’s control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work

has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 12: REFERENCES

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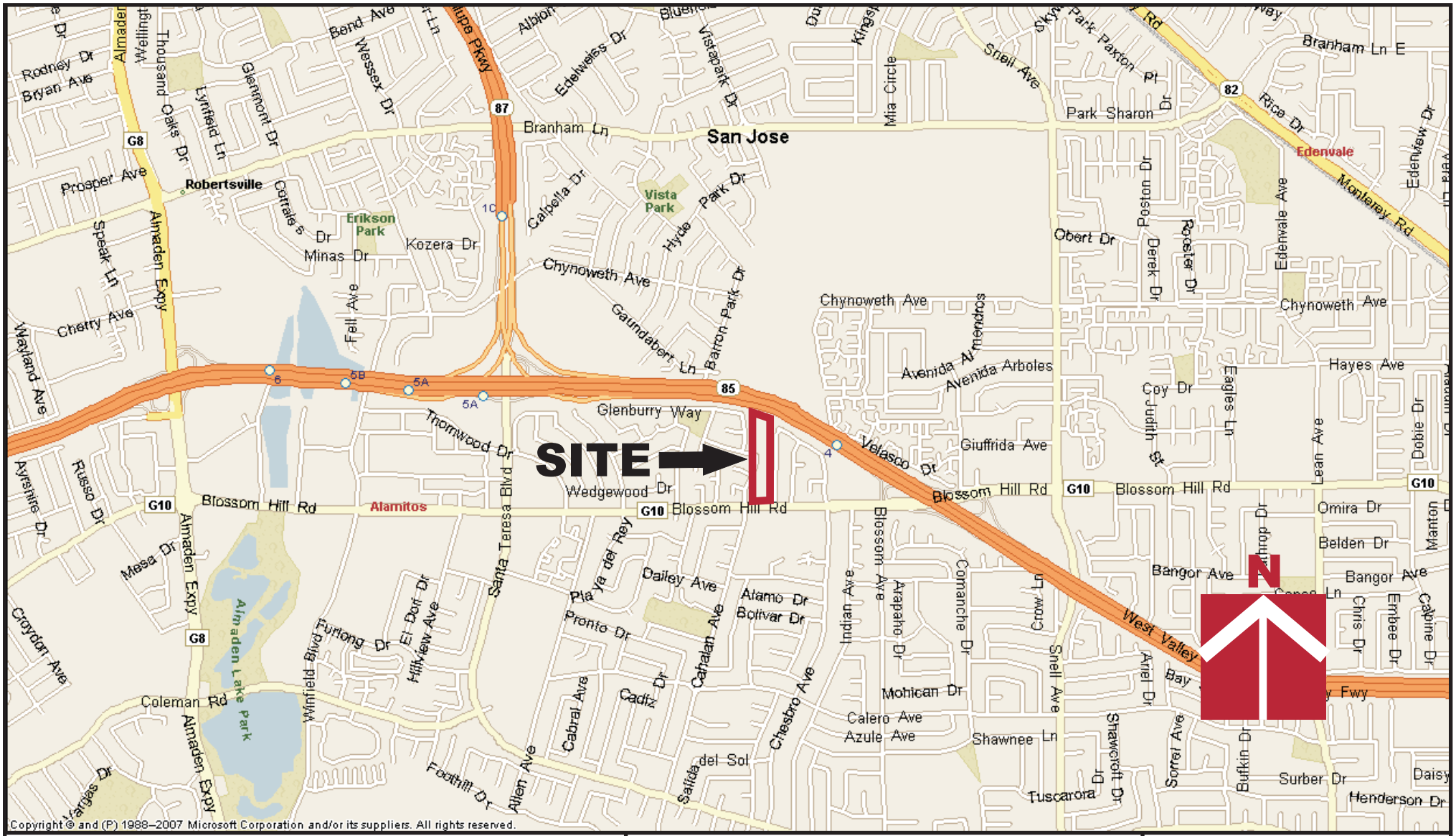
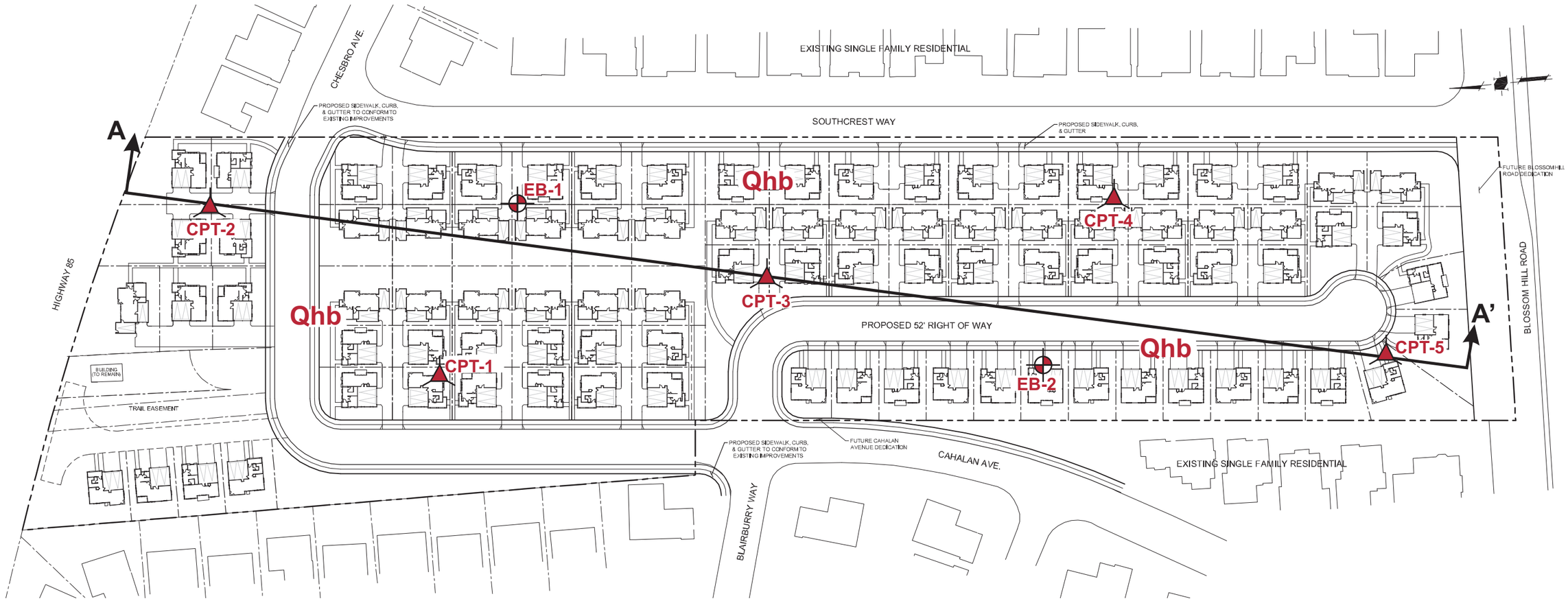


CORNERSTONE EARTH GROUP

Vicinity Map

Lester Property Residential Development
Blossom Hill Road
San Jose, CA

Project Number	168-6-2
Figure Number	Figure 1
Date	July 2010
Drawn By	FLL/MGV



Explanation

Geologic units

Qhb Holocene basin deposits

Symbols

Approximate location of exploratory boring

Approximate location of Cone Penetration Test (CPT)

A A' Approximate location of geologic cross-section (see Figure 3)

N

0 100 200

APPROXIMATE SCALE (FEET)

Site Plan and Geologic Map

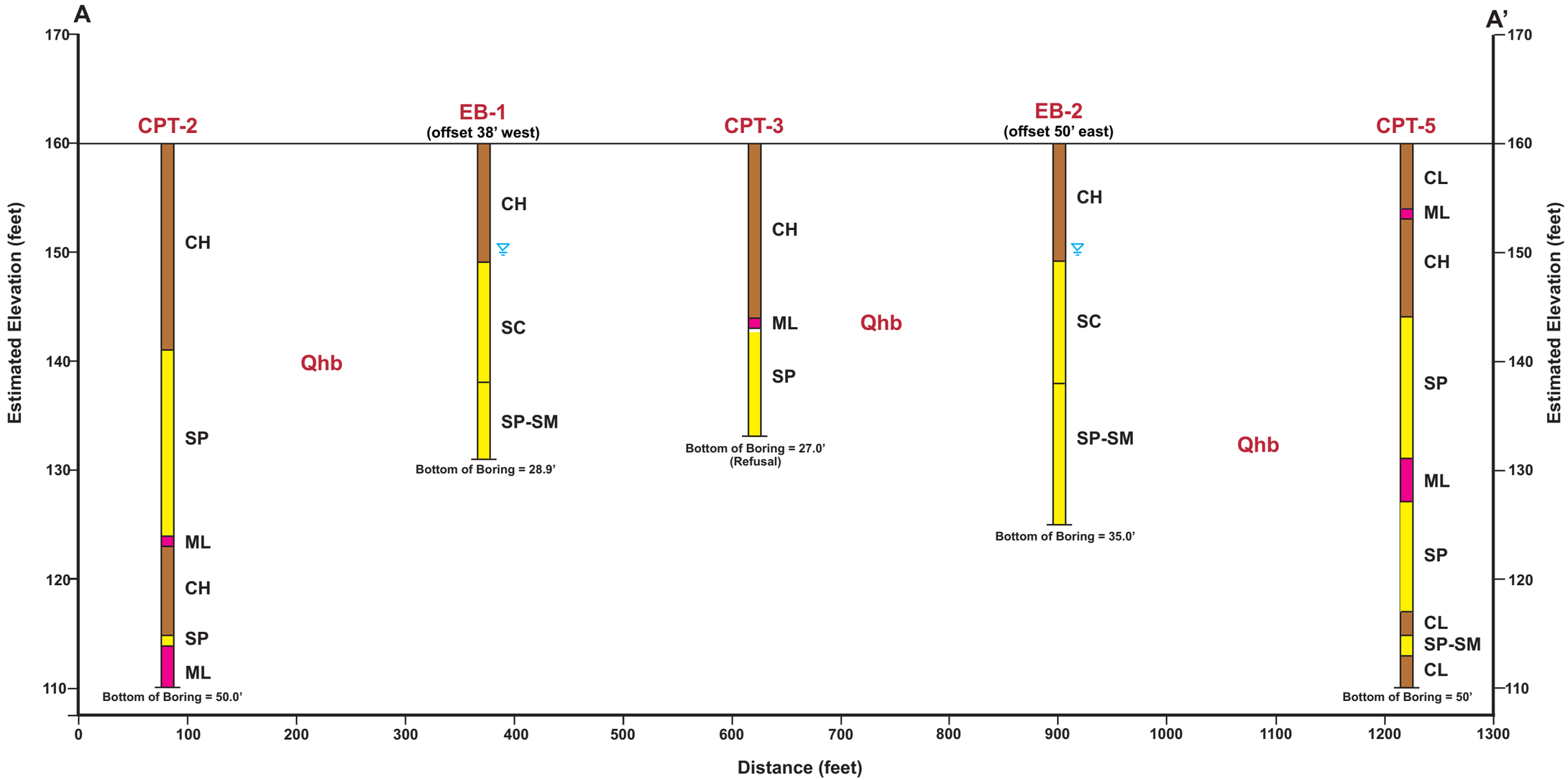
Lester Property Residential Development
Blossom Hill Road
San Jose, CA

CORNERSTONE EARTH GROUP

Figure Number
Figure 2

Project Number
168-6-2

July 2010



Explanation

Geologic units

Qhb — Holocene basin deposits

Symbols

CH	Fat Clay
CL	Lean Clay
ML	Silt
SC	Clayey Sand
SP-SM	Poorly Graded Sand with Silt
SP	Poorly Graded Sand
▽	Groundwater depth

Section A-A'

(View Looking East)
Horizontal Scale 1"=100'
Vertical Scale 1"=10'

Graphical Scale
Vertical Exaggeration = 10X

0 — 100 feet
0 — 10 feet

Notes: 1) Topographic information not available. Estimated elevation is 160'. Existing site grade is relatively level based on visual observations.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

Geotechnical Cross-Section A-A'

Lester Property Residential Development
Blossom Hill Road
San Jose, CA

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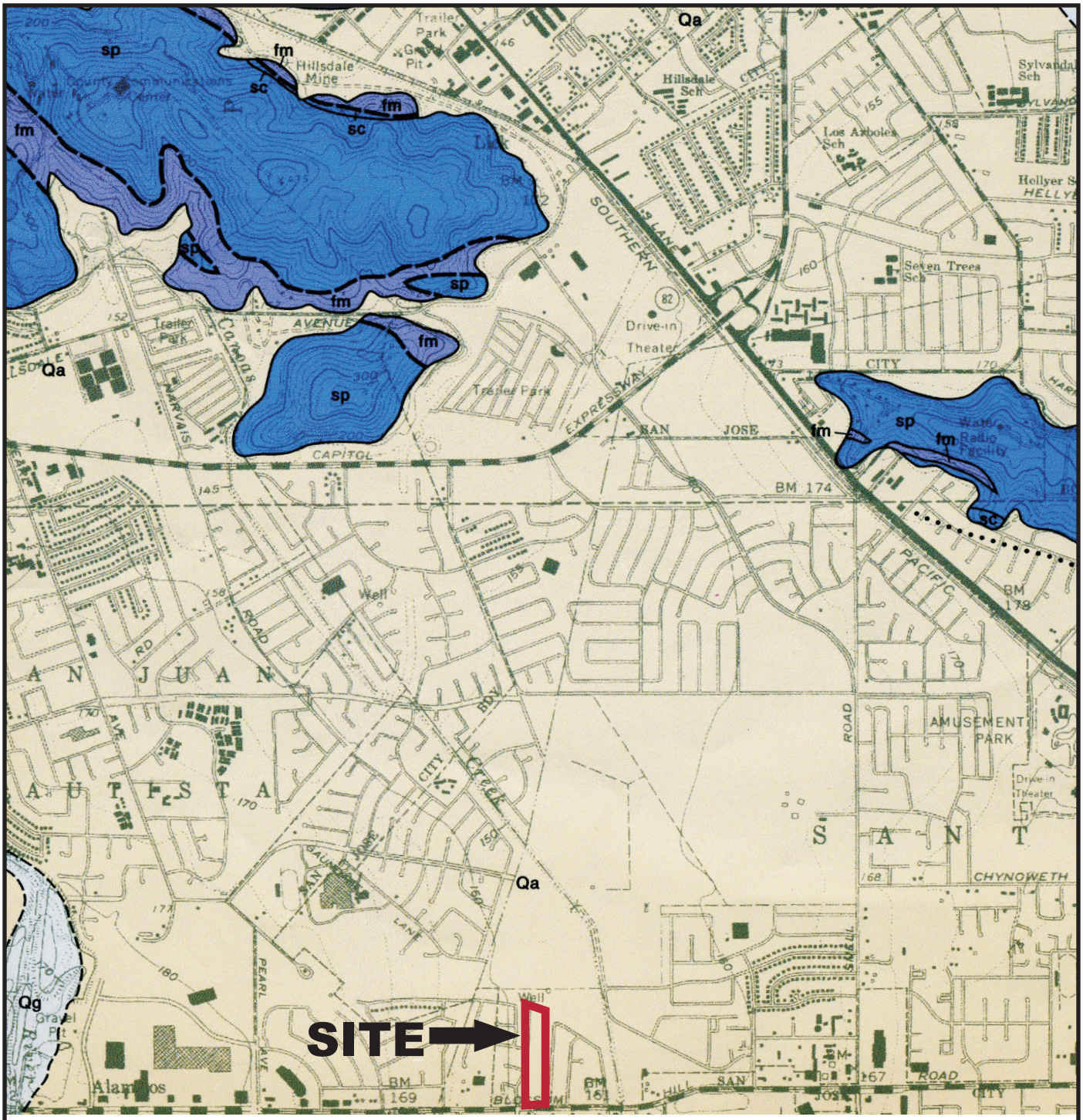
Figure Number
Figure 3

Project Number
168-6-2

July 2010



Local Geologic Units

Qg	Sand and gravel of Coyote Creek Channel (Holocene)
Qa	Alluvial Sediments (Holocene)
Qls	Landslide rubble (Pleistocene)
Qoa	Older alluvial sediments (Pleistocene)
Qts	Santa Clara Formation gravel/conglomerate (Pleistocene to late Pliocene)

Symbols

——— Geologic contact, dashed where approximately located

N

0 2000 4000

APPROXIMATE SCALE (FEET)

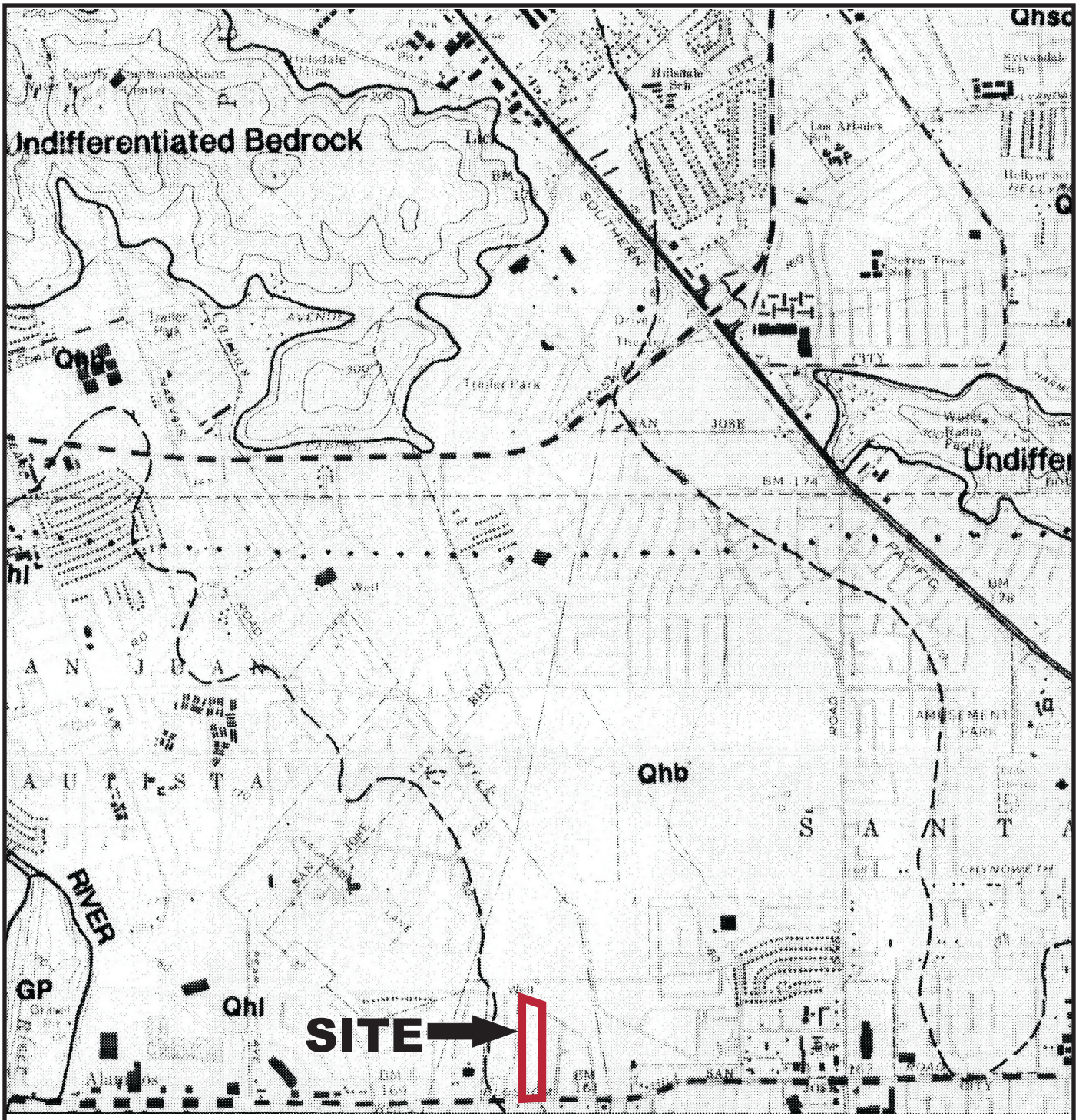
Base by Dibblee, 2005

CORNERSTONE EARTH GROUP

Regional Geologic Map - 2005

Lester Property Residential Development
Blossom Hill Road
San Jose, CA

Project Number	168-6-2
Figure Number	Figure 4
Date	July 2010
Drawn By	FLL/MGV



Local Geologic Units

Qhb Holocene basin deposits

Qhl Holocene alluvial fan levee deposits

Symbols

——— Geologic contact, dashed where approximately located

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base by Helley and Wesling, 1990

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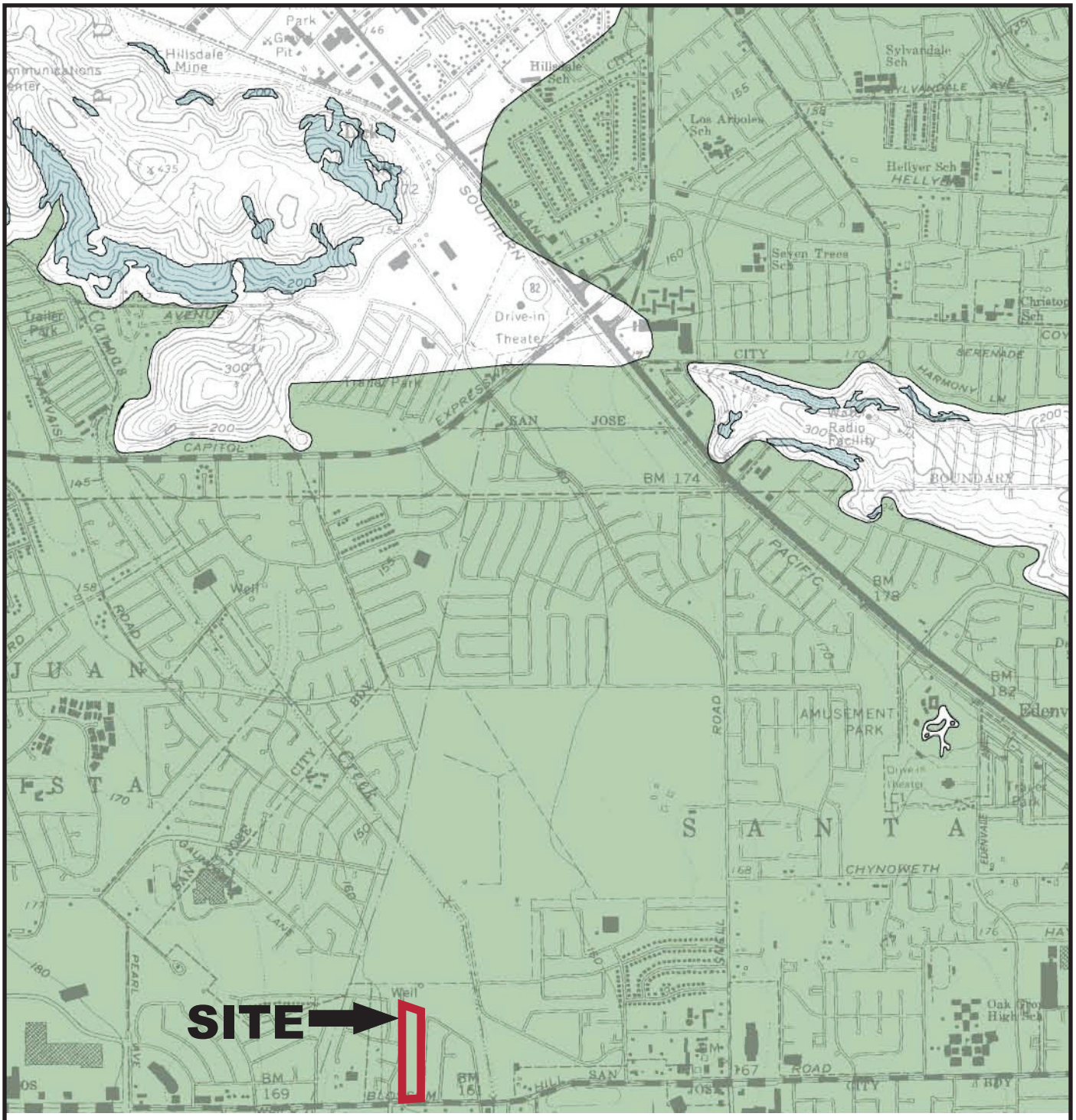
Regional Geologic Map - 1990

Lester Property Residential Development
Blossom Hill Road
San Jose, CA

Project Number	168-6-2
Figure Number	Figure 5
Date	July 2010
Drawn By	FLL/MGV



Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base by State of California, Seismic Hazard Zones, San Jose East Quadrangle, dated January 17, 2001

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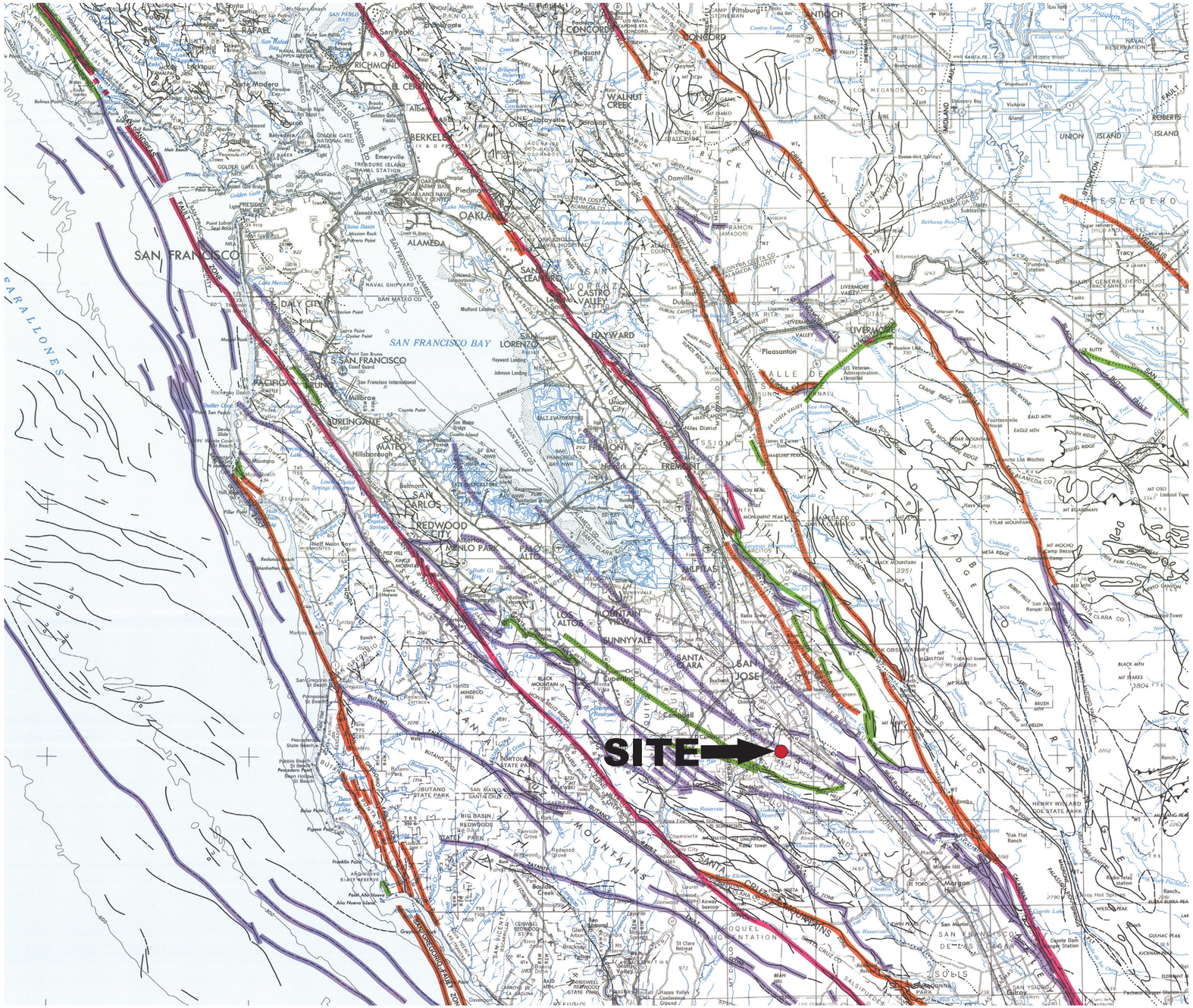
CGS Seismic Hazard Zone Map

Lester Property Residential Development
Blossom Hill Road
San Jose, CA

Project Number	168-6-2
Figure Number	Figure 6
Date	July 2010
Drawn By	FLL/MGV



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	200			Displacement during Holocene time.[2]
			10,000			Faults showing evidence of displacement during late Quaternary time.[3, 4]
	Early Quaternary	Pleistocene	700,000			Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		2,000,000			
	Miocene		5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

N

0 10 20

APPROXIMATE SCALE (MILES)

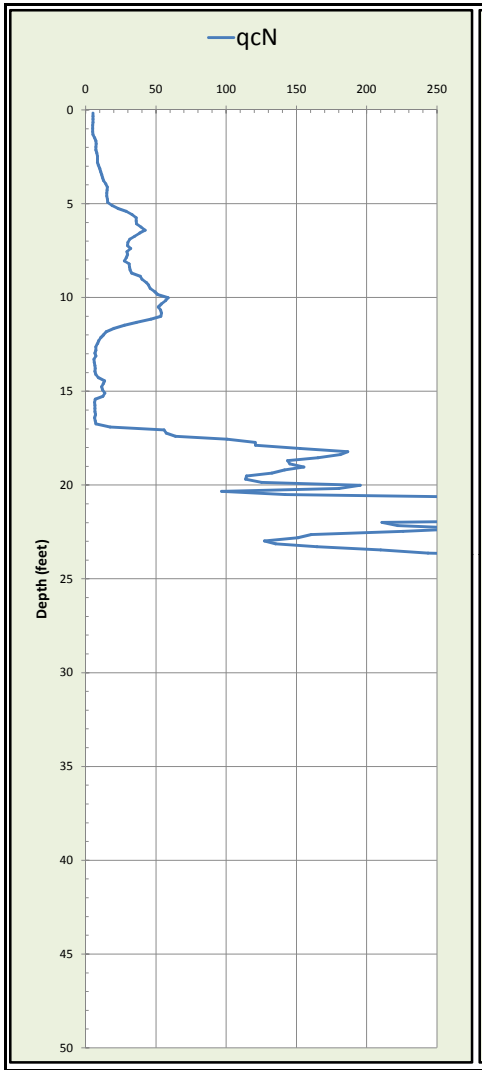
Base Map: "Map showing recency of faulting, San Francisco - San Jose Quadrangle California," by Bortugno, et.al. (California Division of Mines and Geology) dated 1991.

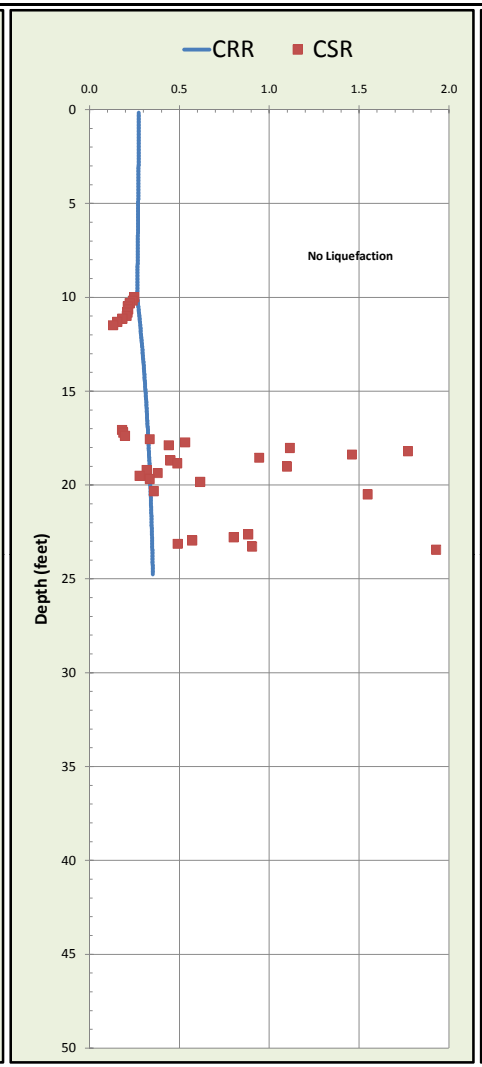
Project Number	168-6-2
Figure Number	Figure 7
Date	July 2010
Drawn By	FLL/MGV

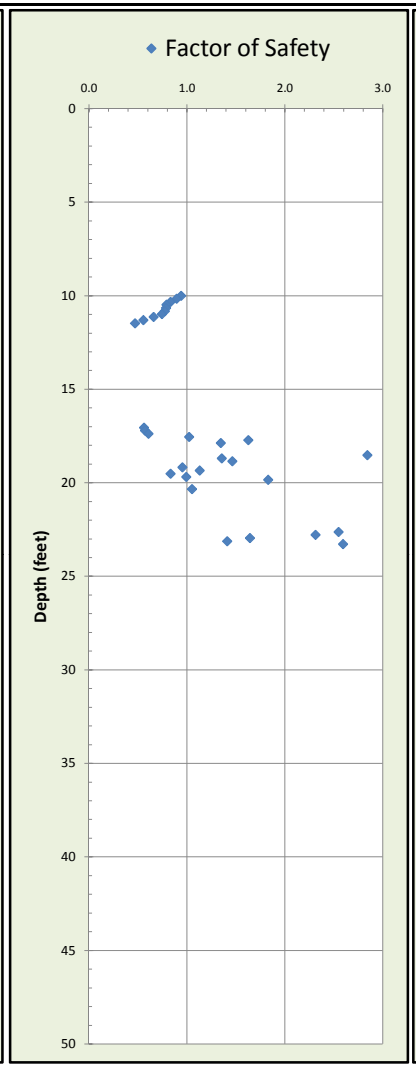
Regional Fault Map

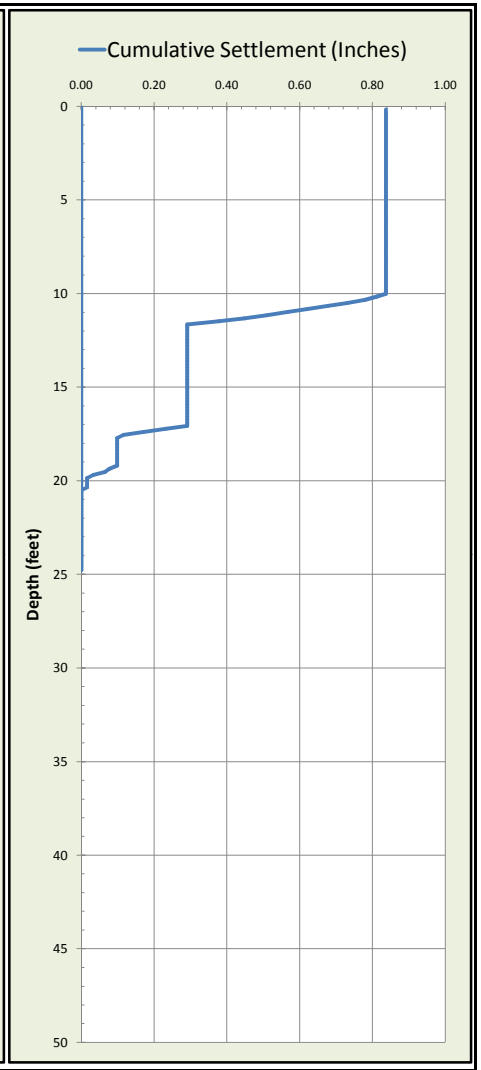
Lester Property Residential Development
Blossom Hill Road
San Jose, CA

CORNERSTONE EARTH GROUP









CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary

Lester Property Residential

San Jose, California

Project Number

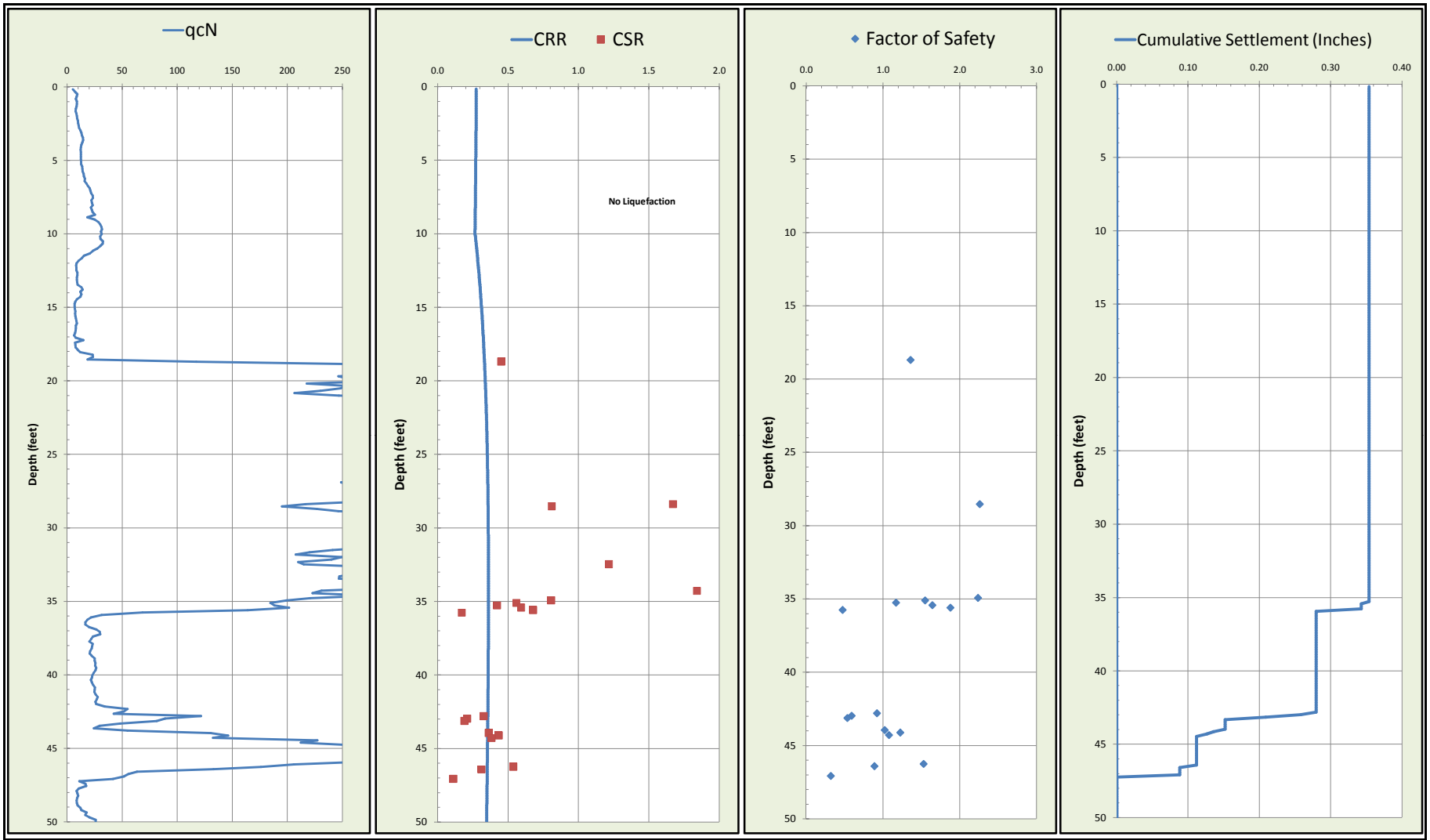
168-6-1

Figure Number

Figure 8

4/8/2010

CPT No. 1



CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary

Lester Property Residential

San Jose, California

Project Number

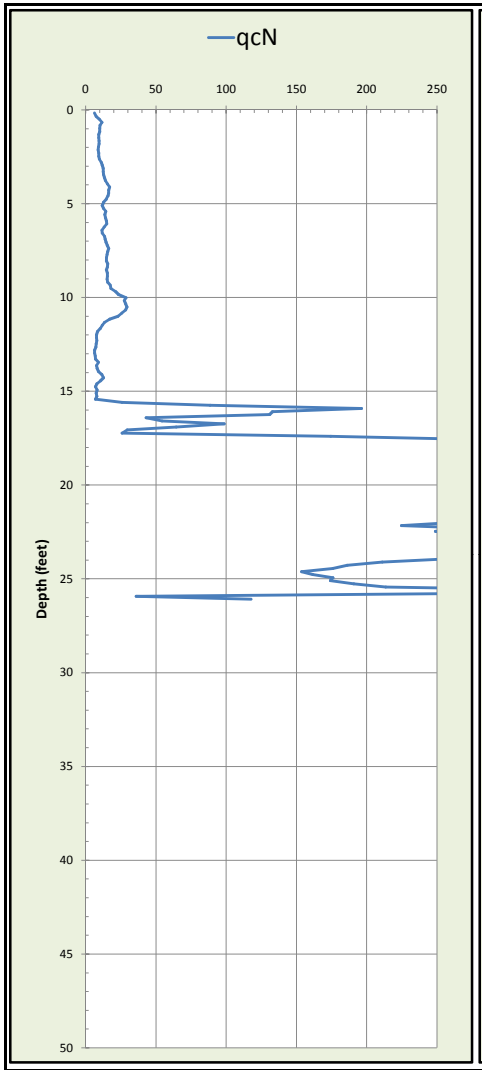
168-6-1

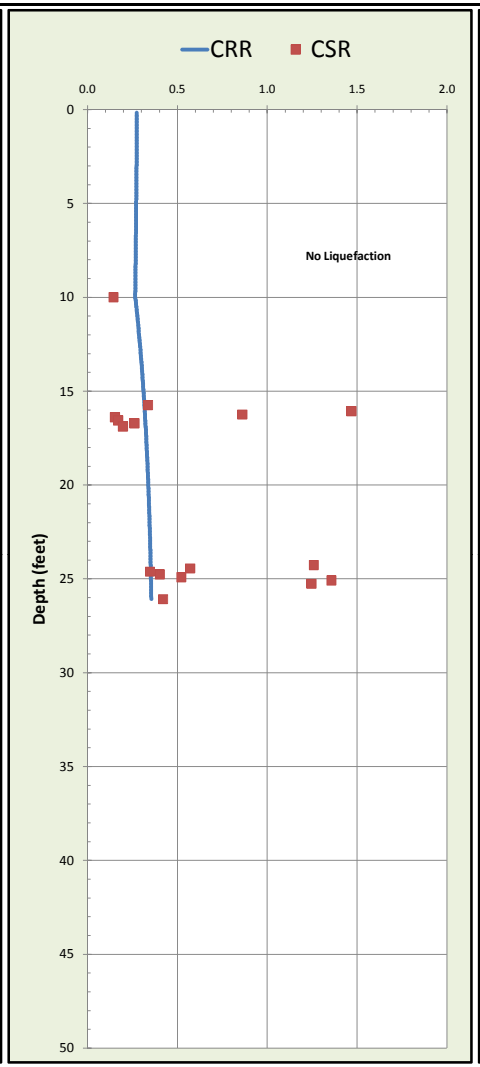
Figure Number

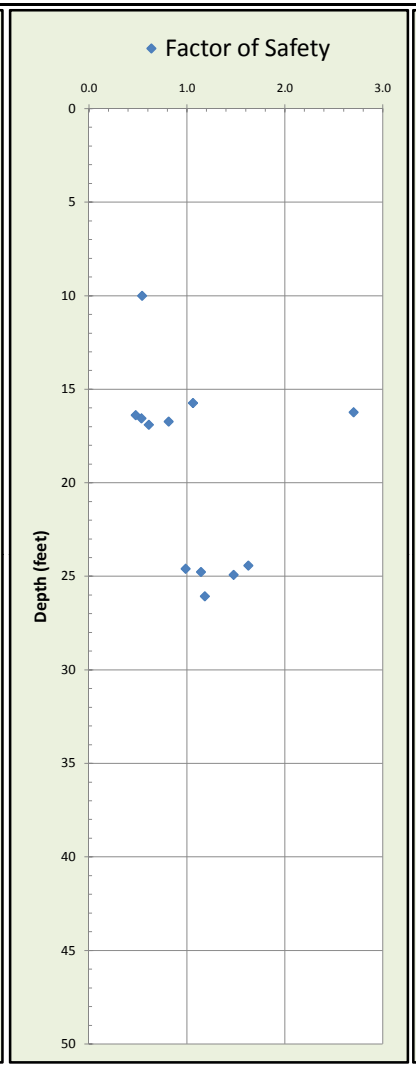
Figure 9

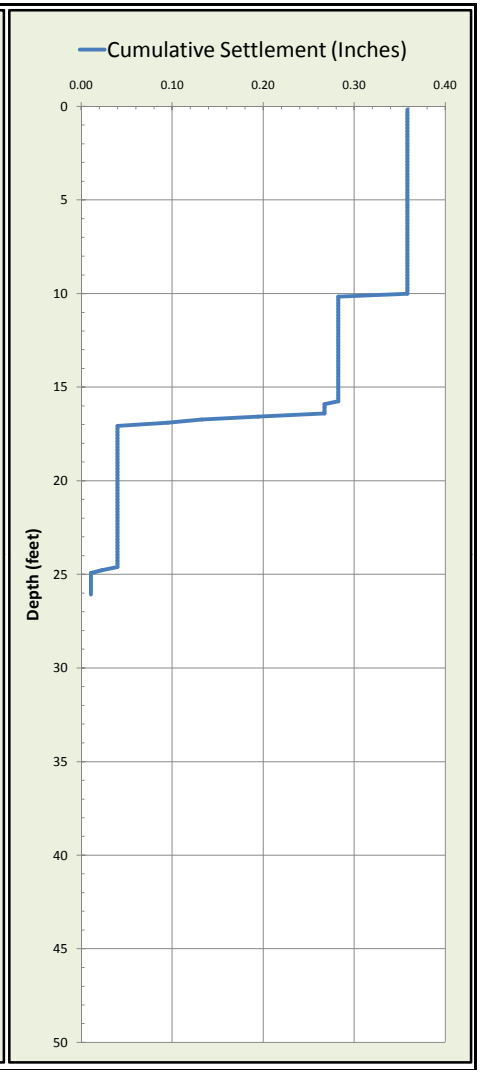
4/8/2010

CPT No. 2









CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary

Lester Property Residential

San Jose, California

Project Number

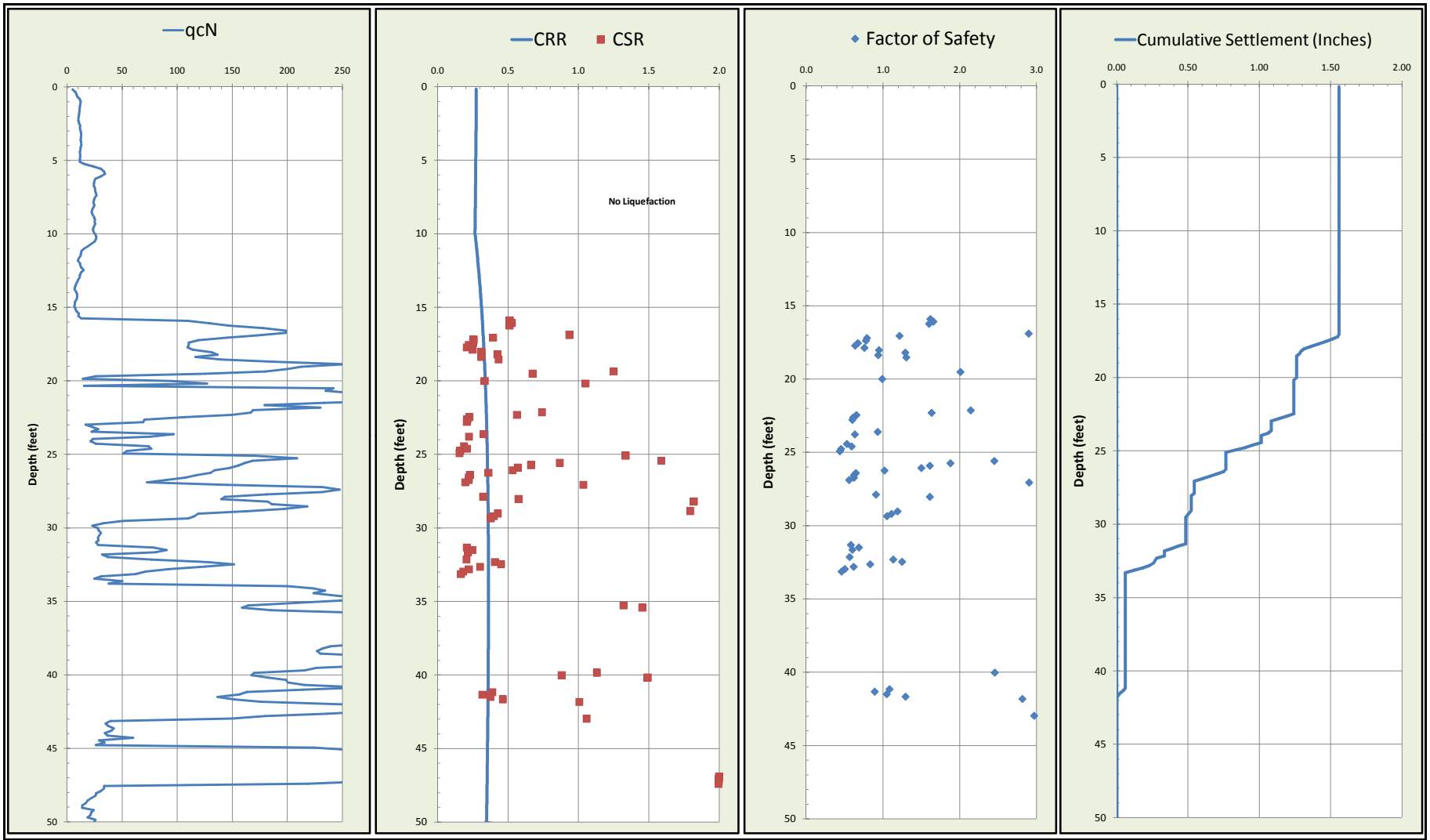
168-6-1

Figure Number

Figure 10

4/8/2010

CPT No. 3



CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary

Lester Property Residential

San Jose, California

Project Number

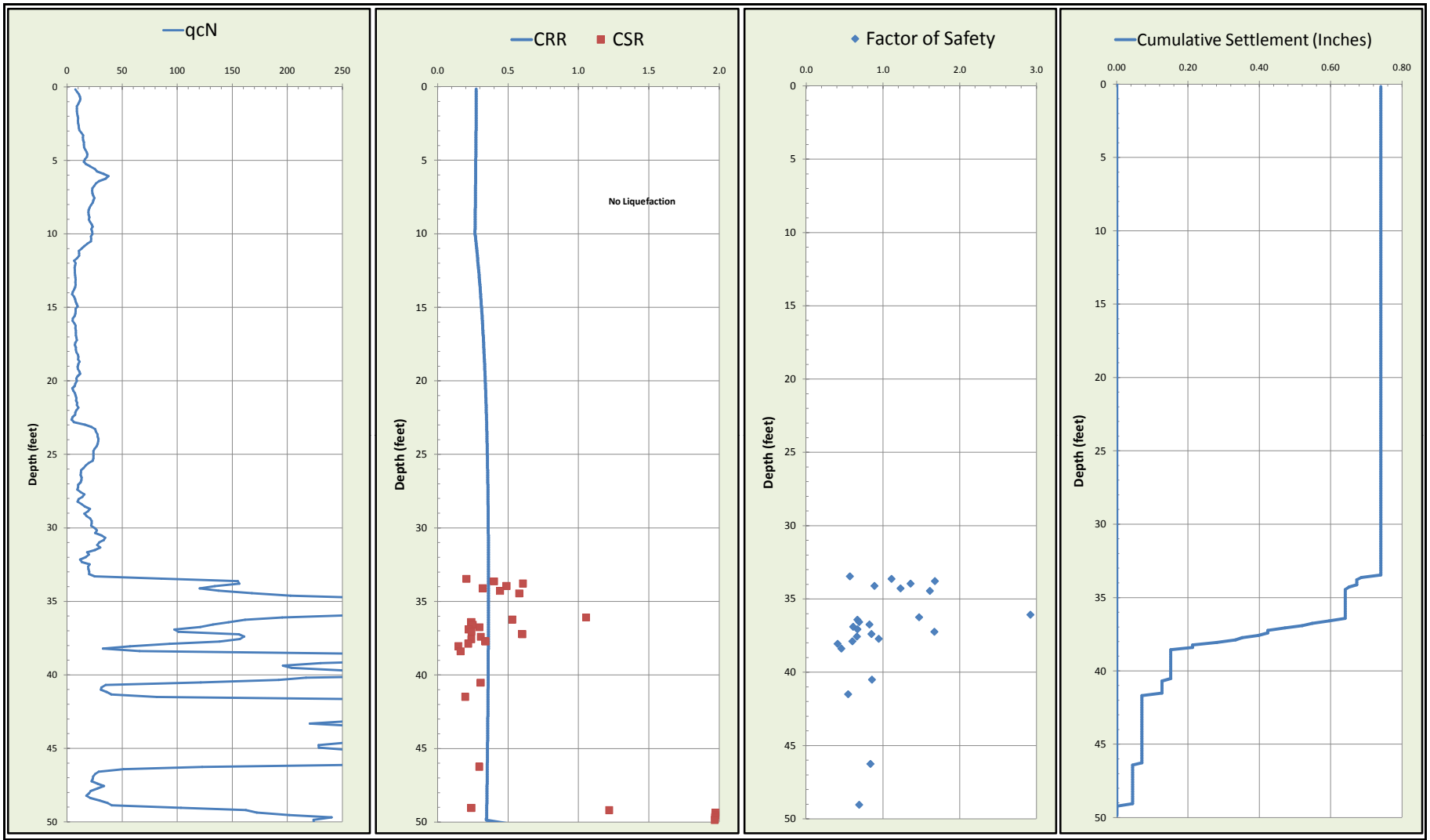
168-6-1

Figure Number

Figure 11

4/8/2010

CPT No. 4



CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary

Lester Property Residential

San Jose, California

Project Number

168-6-1

Figure Number

Figure 12

4/8/2010

CPT No. 5

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and a 20- to 25-ton truck-mounted Cone Penetration Test equipment. Two 8-inch-diameter exploratory borings were drilled on March 22, 2010, to depths ranging from approximately 29 to 35 feet. Five CPTs were also performed in accordance with ASTM D 5778-95 (revised, 2002) on March 16, 2010 to a maximum depth of 50 feet. Approximate boring and CPT locations are shown on the Site Plan, Figure 2. Soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries and other site features as references. Boring and CPT elevations were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. 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 (ASTM D1586). 2.5-inch I.D. samples were obtained using a 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 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data is included as part of this appendix.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. Results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER MATERIAL SYMBOLS

- Poorly-Graded Sand with Clay
- Clayey Sand
- Sandy Silt
- Artificial/Undocumented Fill
- Poorly-Graded Gravelly Sand
- Topsoil
- Well-Graded Gravel with Clay
- Well-Graded Gravel with Silt
- Sand
- Silt
- Well Graded Gravelly Sand
- Gravelly Silt
- Asphalt
- Boulders and Cobble

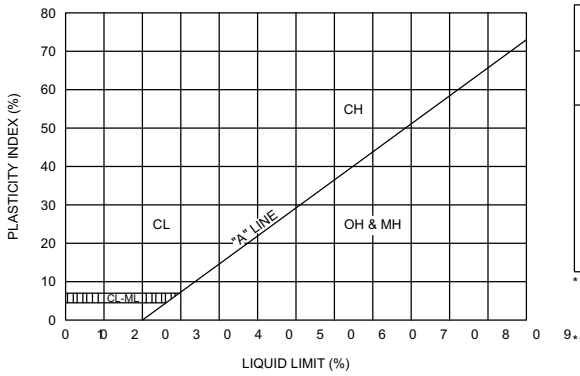
SAMPLER TYPES

- SPT
- Modified California (2.5" I.D.)
- Rock Core
- Shelby Tube
- No Recovery
- Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	PI	-	PLASTICITY INDEX
CD	-	CONSOLIDATED DRAINED TRIAXIAL	SW		SWELL TEST
CN	-	CONSOLIDATION	TC	-	CYCLIC TRIAXIAL
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TV	-	TORVANE SHEAR
DS	-	DIRECT SHEAR	UC	-	UNCONFINED COMPRESSION
PP	-	POCKET PENETROMETER (TSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)		-	
RV	-	R-VALUE	UU		UNCONSOLIDATED UNDRAINED TRIAXIAL
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE		-	
	-	WATER LEVEL			

PLASTICITY CHART



PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Lester Property Residential Development
PROJECT NUMBER 168-6-1
PROJECT LOCATION San Jose, CA

DATE STARTED 3/22/10 **DATE COMPLETED** 3/22/10
GROUND ELEVATION **BORING DEPTH** 28.9 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37°15.196' **LONGITUDE** -121°50.719'
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EMT
AT TIME OF DRILLING 13.5 ft.
NOTES
AT END OF DRILLING 10 ft.

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
	0		**Fat Clay (CH)** very stiff, moist, dark gray, some fine sand, high plasticity	21	MC-1	88	23		
			Liquid Limit = 64, Plastic Limit = 23	21	MC-2A	93	26	41	
	5		**Fat Clay (CH)** hard, moist, gray and brown mottled, some fine to medium sand, moderate to high plasticity	31	MC-3B		21		
				28	MC-4A	95	19		
	10		**Clayey Sand (SC)** medium dense, moist to wet, brown, fine to medium sand, some fine subrounded to subangular gravel	11	MC-5B	112	15		31
	15								
			Liquid Limit = 27, Plastic Limit = 16	17	MC-6B	113	14	11	25
	20								
			Poorly Graded Sand with Silt and Gravel (SP-SM) very dense, wet, gray and brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	50 / 6"	SPT				
	25			50 / 5"	SPT				
			Bottom of Boring at 28.9 feet.						
	30								
	35								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 4/8/10 08:00 - P:\DRAFTING\GINT FILES\168-6-1 LESTER PROPERTY.GPJ

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Lester Property Residential Development
PROJECT NUMBER 168-6-1
PROJECT LOCATION San Jose, CA

DATE STARTED 3/22/10 **DATE COMPLETED** 3/22/10
GROUND ELEVATION **BORING DEPTH** 35 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37°15.112' **LONGITUDE** -121°50.759'
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EMT
AT TIME OF DRILLING 12.5 ft.
NOTES
AT END OF DRILLING 10 ft.

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (1.0 2.0 3.0 4.0 5.0)
	0		**Fat Clay (CH)** very stiff, moist, dark gray with brown mottles, some fine sand, high plasticity	13	MC-1	87	27			
				18	MC-2B	98	20			
	5		**Fat Clay (CH)** hard, moist, brown and gray mottled, some fine to medium sand, moderate to high plasticity	21	MC-3B	97	17			
			Sandy Lean Clay with Gravel (CL) hard, moist, brown, fine to medium sand, fine subrounded to subangular gravel, low to moderate plasticity	36	MC-4B	107	15			
	10		**Poorly Graded Sand with Silt and Gravel (SP-SM)** loose, wet, gray and brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	9	SPT-5				7	
	15		**Poorly Graded Gravel with Silt and Sand (GP-GM)** dense, wet, gray and brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	39	SPT					
	20			49	NR					
	25		**Sandy Silt (ML)** hard, moist, brown, fine sand, low plasticity	38	SPT					
	30		**Sandy Lean Clay with Gravel (CL)** very stiff, moist, brown, fine to medium sand, fine subrounded to subangular gravel, low to moderate plasticity	43	MC					
	35		**Clayey Sand with Gravel (SC)** medium dense, moist to wet, gray and brown, fine to coarse sand, fine subrounded gravel							
	40		Bottom of Boring at 35.0 feet.							

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

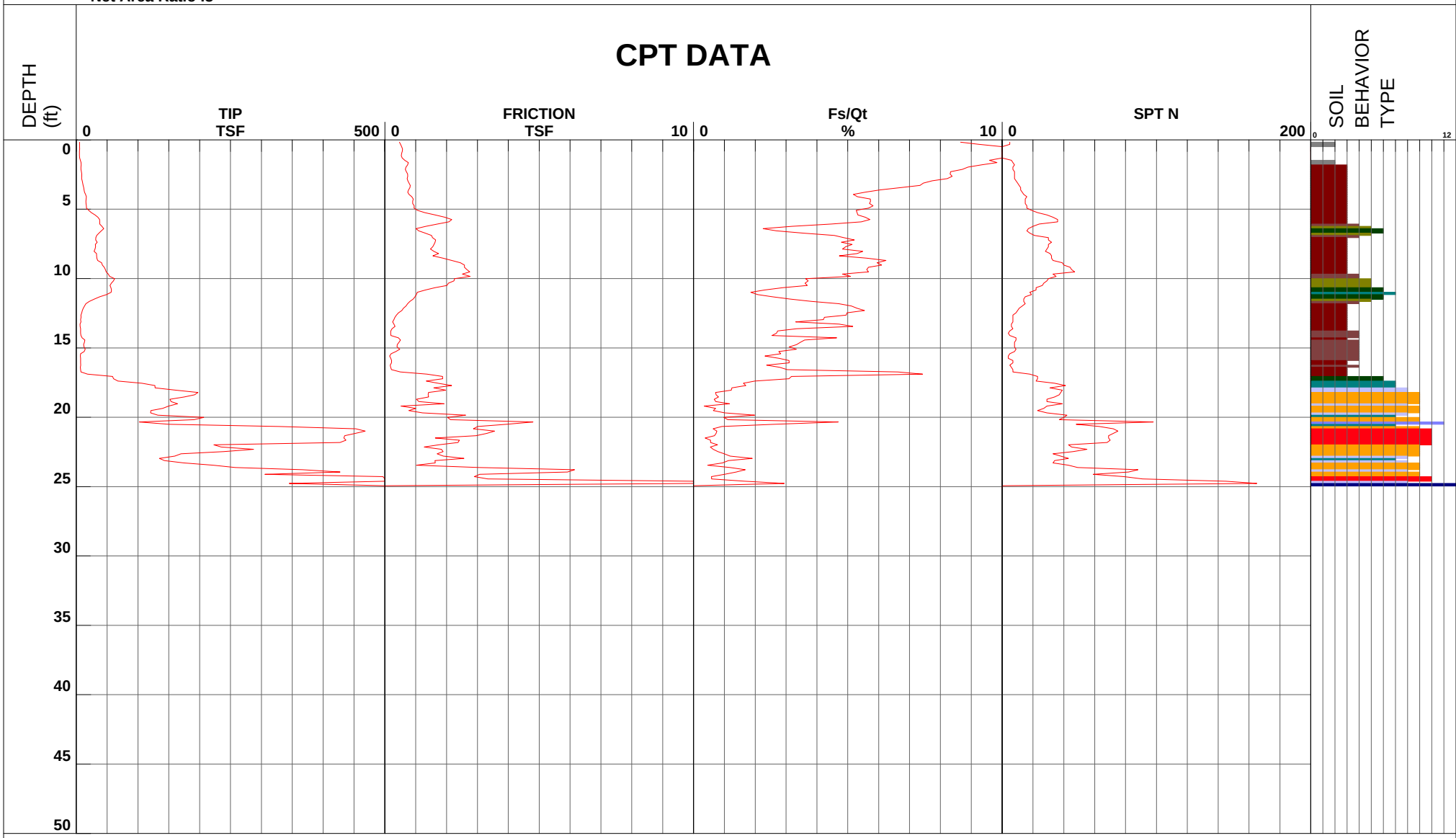
CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 4/9/10 13:21 - P:\DRAFTING\GINT FILES\168-6-1 LESTER PROPERTY.GPJ

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(207).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-01	Date and Time	3/16/2010 10:58:39 AM	Maximum Depth	25.10 ft
Water Table Depth	13.00 ft				

Net Area Ratio .8

CPT DATA



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

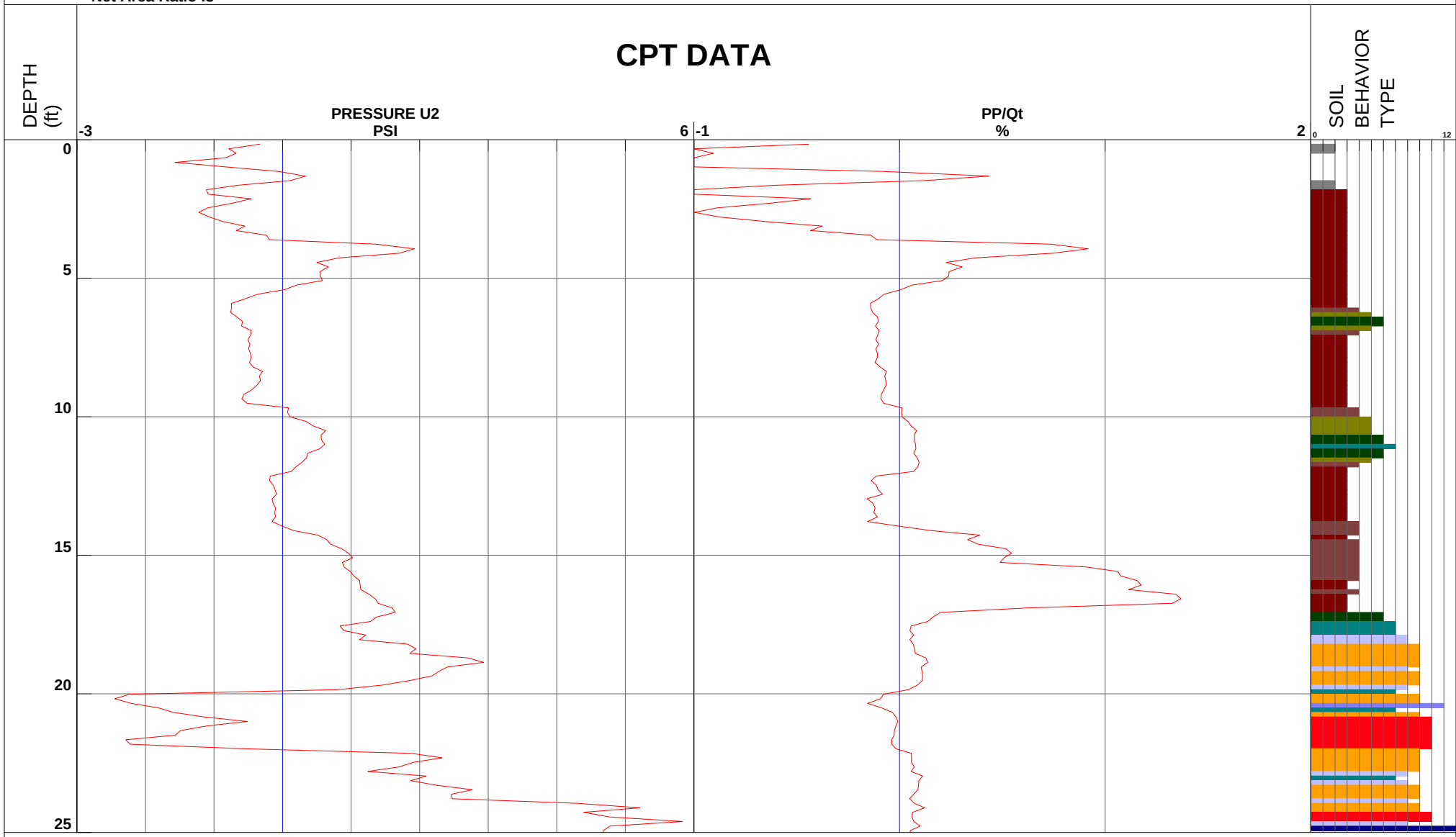
Cone Size 10cm squared

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

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(207).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-01	Date and Time	3/16/2010 10:58:39 AM	Maximum Depth	25.10 ft
Water Table Depth	13.00 ft				

Net Area Ratio .8



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

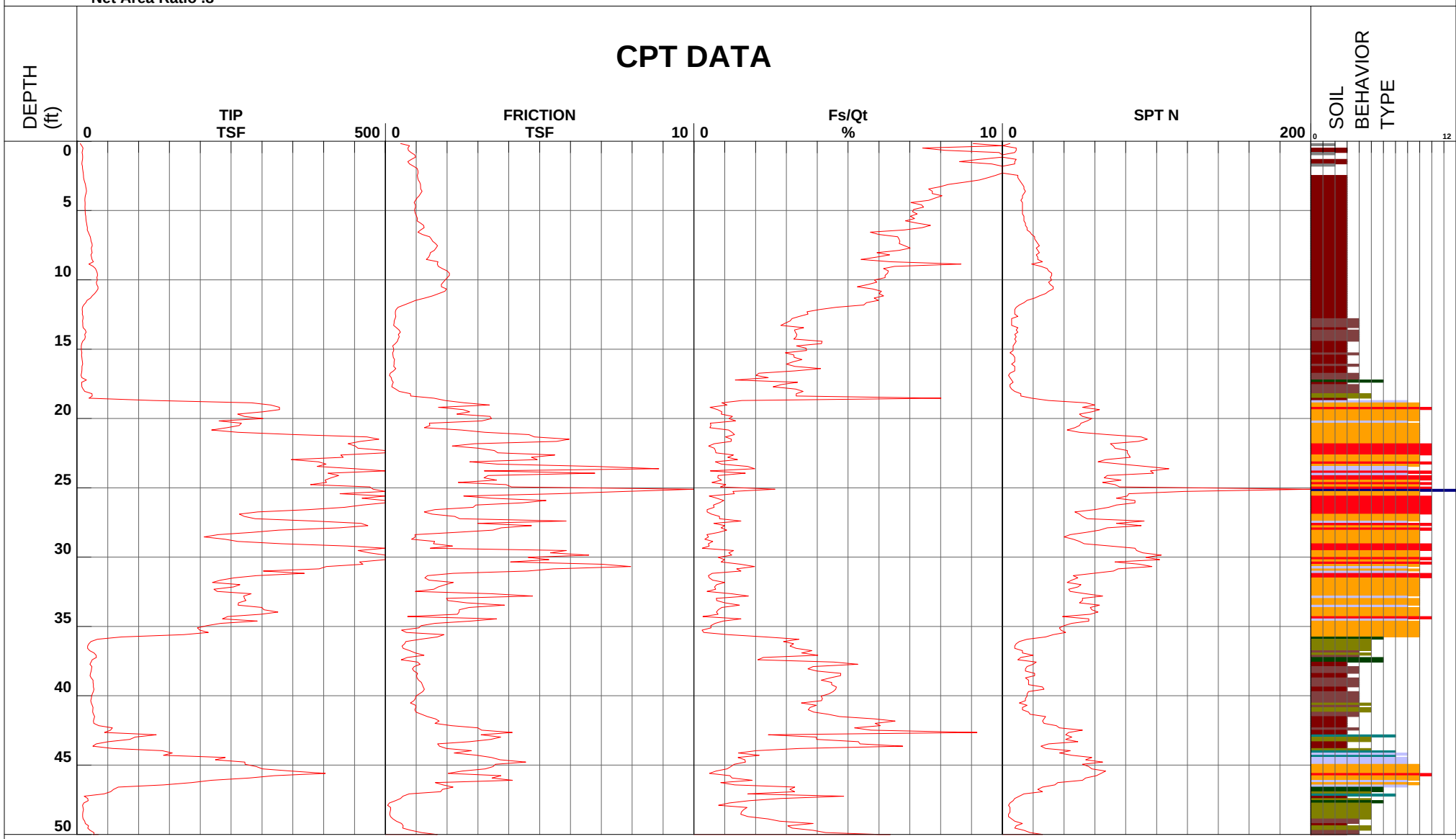
Cone Size 10cm squared

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

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(206).cpt
Job Number	168-5-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-02	Date and Time	3/16/2010 8:43:22 AM	Maximum Depth	50.52 ft
Water Table Depth	13.00 ft				

Net Area Ratio .8



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

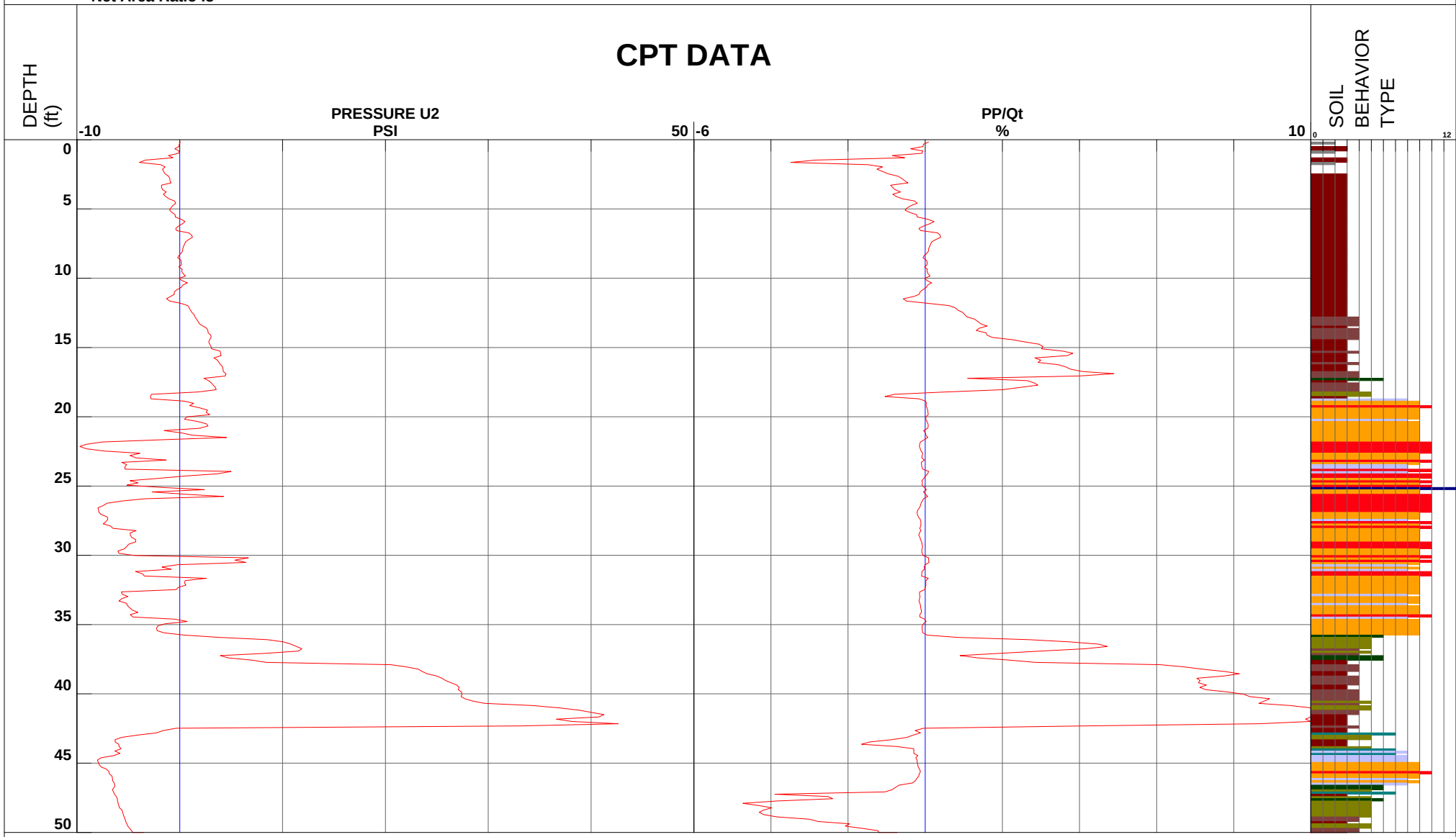
Cone Size 10cm squared

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

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(206).cpt
Job Number	168-5-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-02	Date and Time	3/16/2010 8:43:22 AM	Maximum Depth	50.52 ft
Water Table Depth	13.00 ft				

Net Area Ratio .8



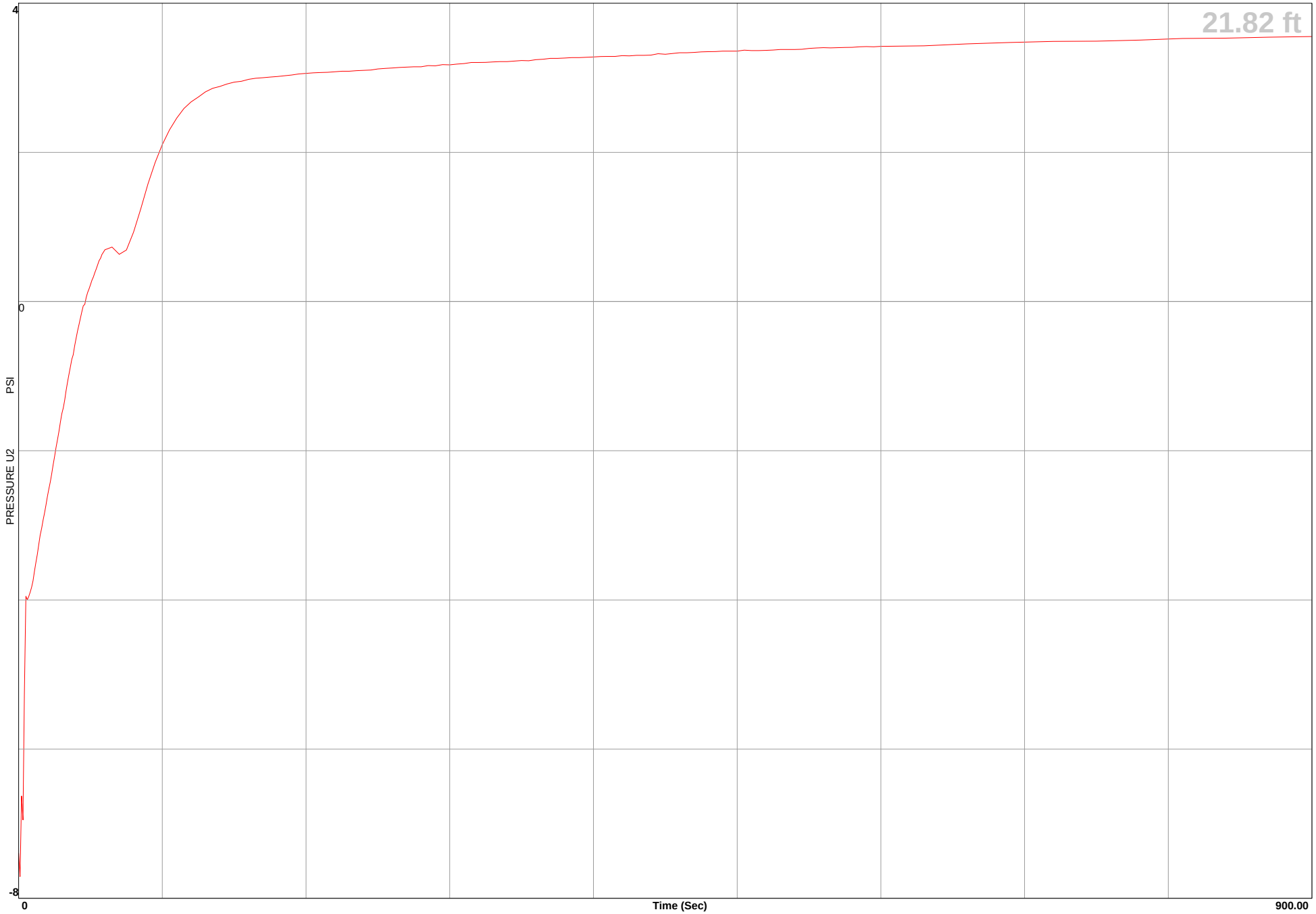
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

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

Cornerstone Earth Group

Location	Lester Properties	Operator	BH-MA		
Job Number	168-5-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-02	Date and Time	3/16/2010 8:43:22 AM		
Equilized Pressure	3.5	Ground Water Depth	13.7		

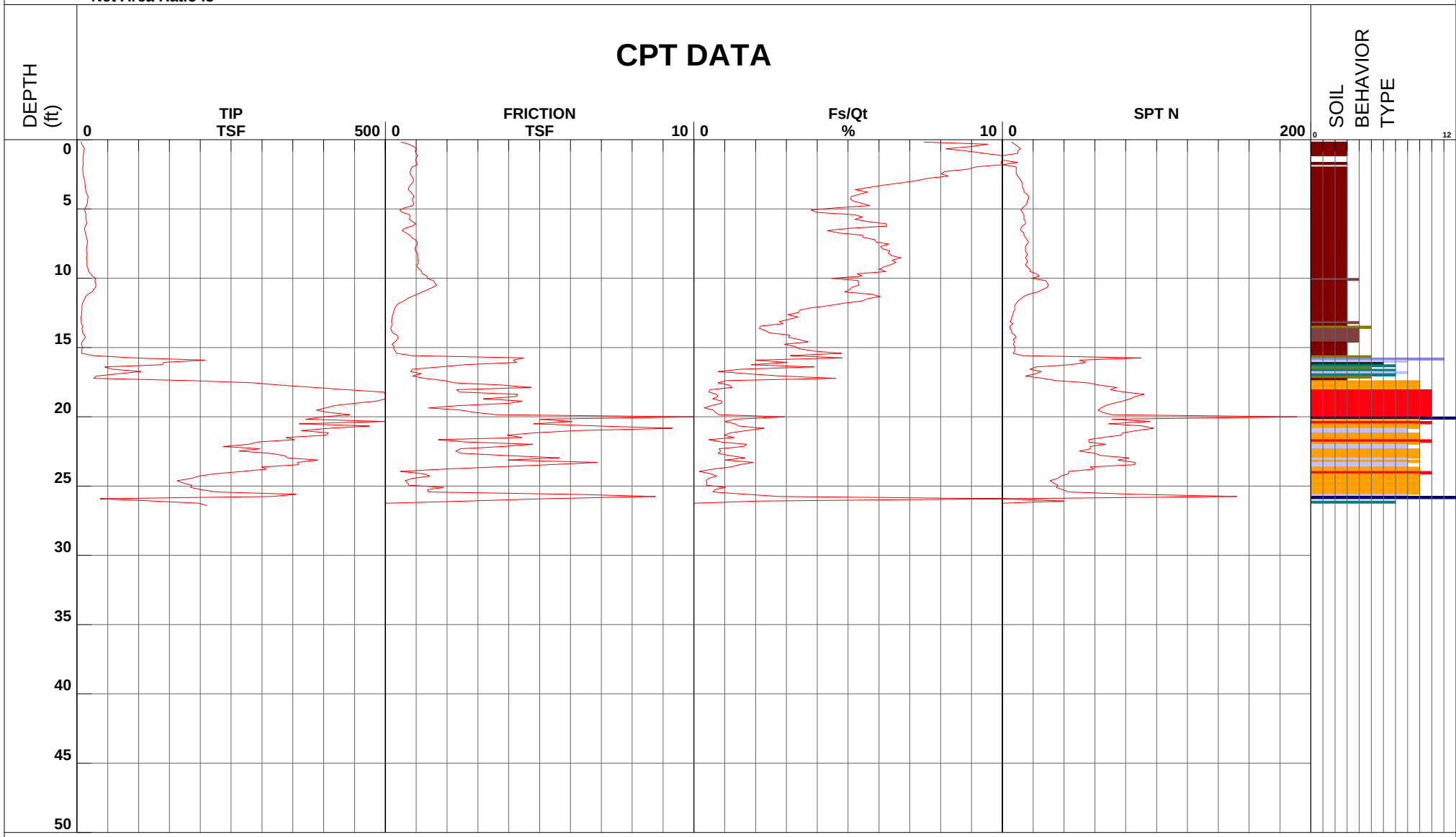


Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(208).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-03	Date and Time	3/16/2010 12:00:11 PM	Maximum Depth	26.41 ft
Water Table Depth	11.00 ft				

Net Area Ratio .8

CPT DATA



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

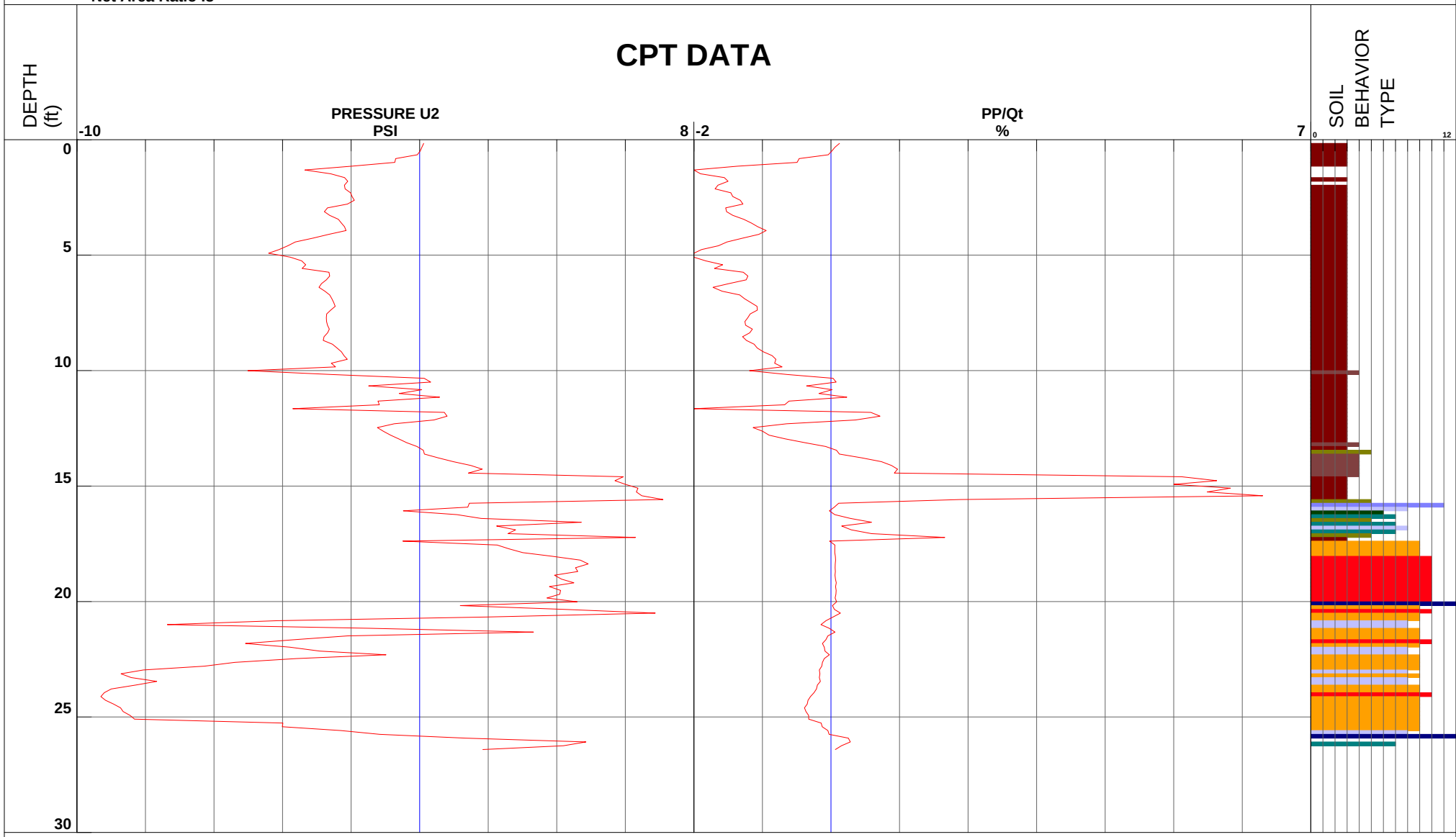
Cone Size 10cm squared

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

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(208).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-03	Date and Time	3/16/2010 12:00:11 PM	Maximum Depth	26.41 ft
Water Table Depth	11.00 ft				

Net Area Ratio .8



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

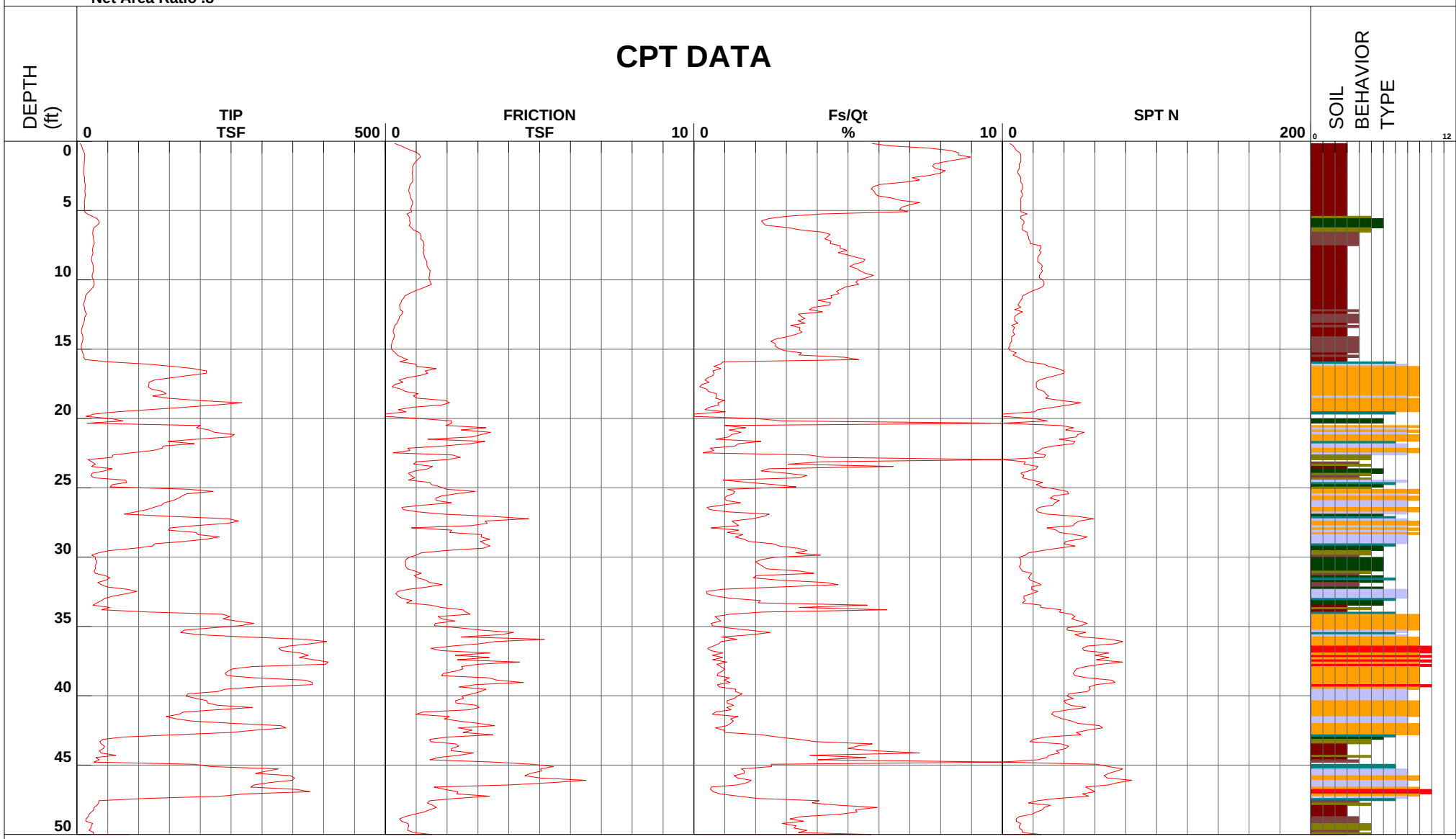
Cone Size 10cm squared

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

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(209).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-04	Date and Time	3/16/2010 12:44:49 PM	Maximum Depth	50.36 ft
Water Table Depth	11.00 ft				

Net Area Ratio .8



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

Cone Size 10cm squared

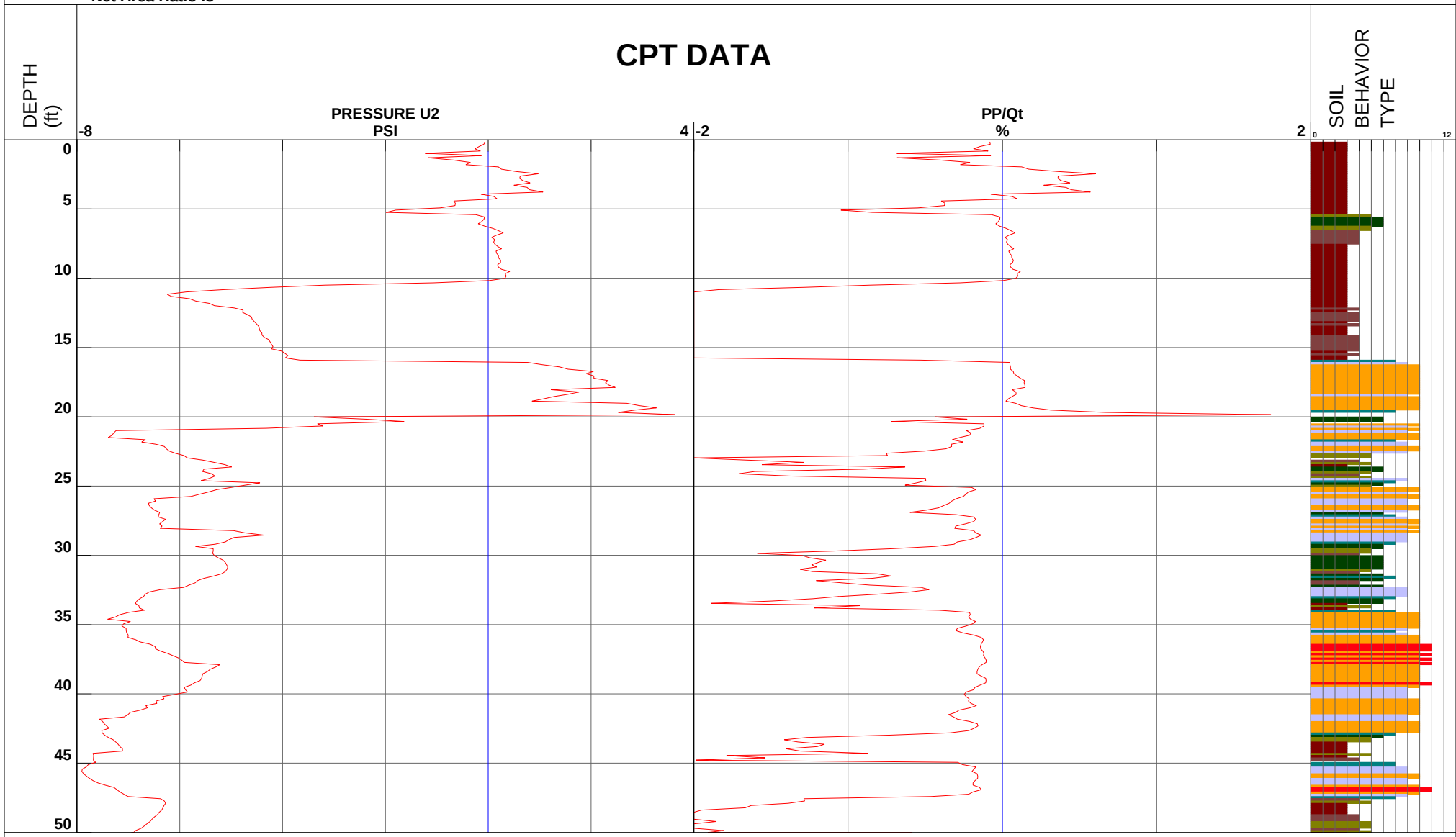
*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(209).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-04	Date and Time	3/16/2010 12:44:49 PM	Maximum Depth	50.36 ft
Water Table Depth	11.00 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
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6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

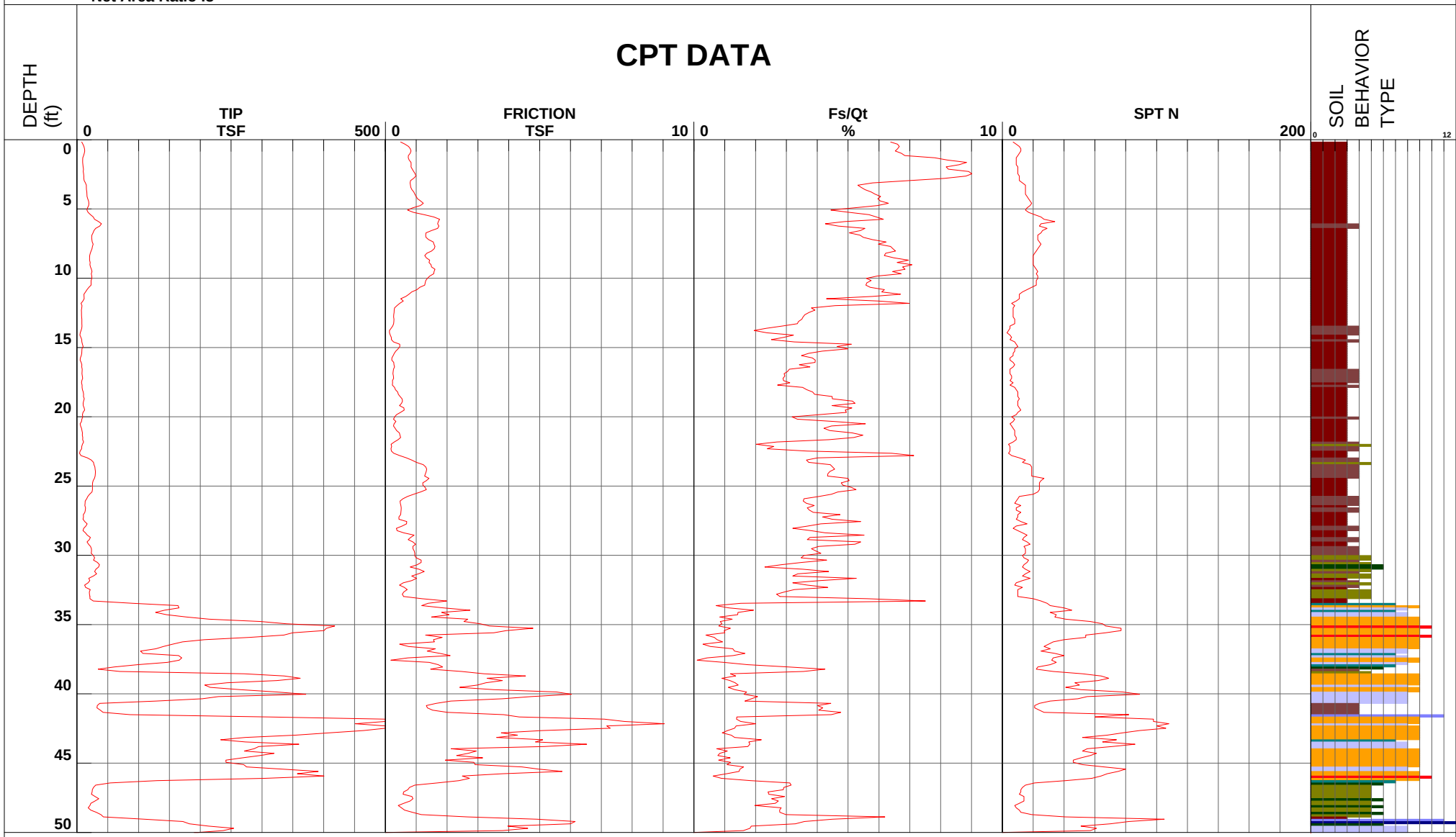
*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(210).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-05	Date and Time	3/16/2010 1:35:10 PM	Maximum Depth	50.20 ft
Water Table Depth	11.00 ft				

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

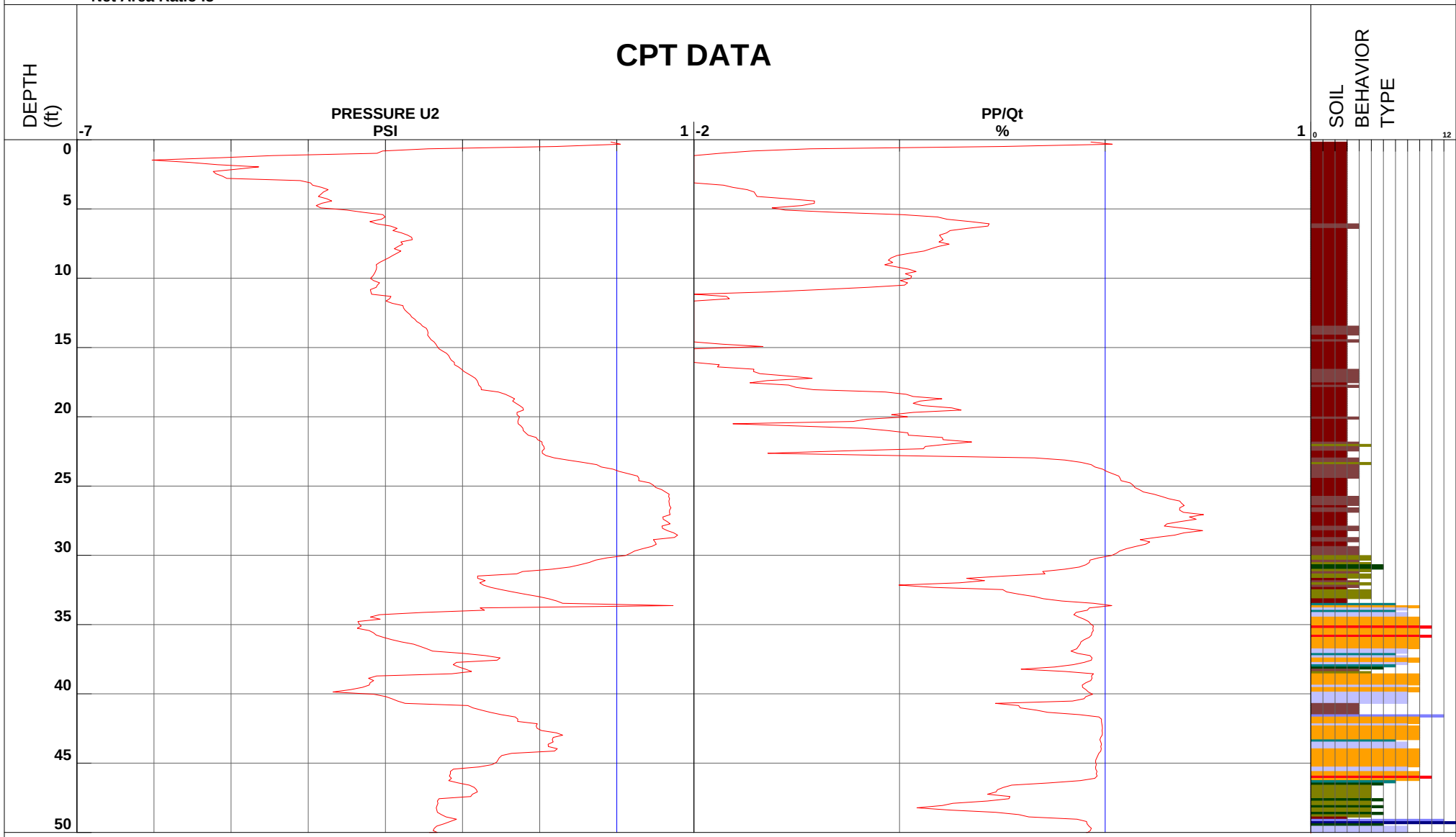
*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Lester Properties	Operator	BH-MA	Filename	SDF(210).cpt
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-05	Date and Time	3/16/2010 1:35:10 PM	Maximum Depth	50.20 ft
Water Table Depth	11.00 ft				

Net Area Ratio .8

CPT DATA



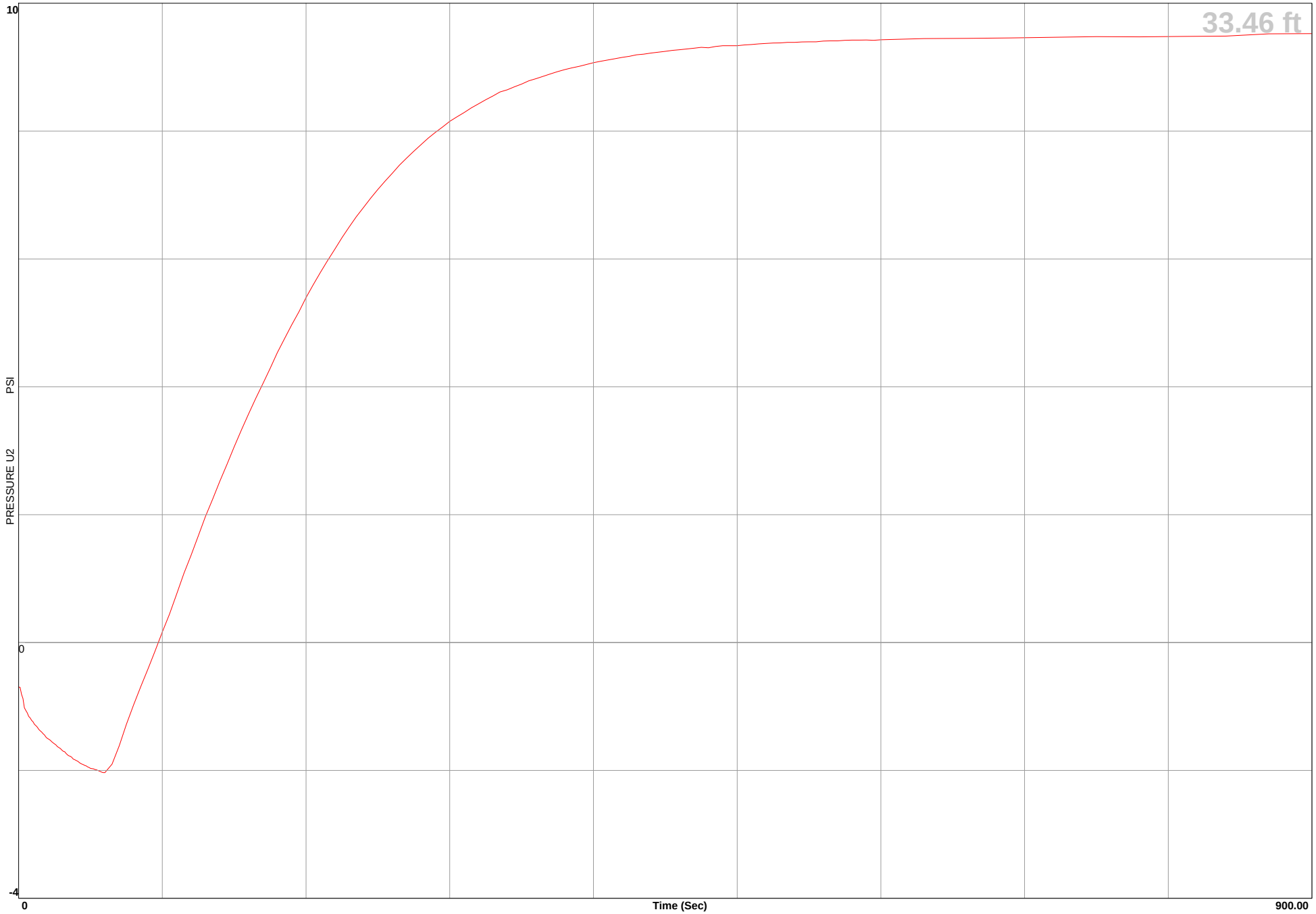
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

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

Cornerstone Earth Group

Location	Lester Properties	Operator	BH-MA		
Job Number	168-6-1	Cone Number	DSG1047	GPS	
Hole Number	CPT-05	Date and Time	3/16/2010 1:35:10 PM		
Equilized Pressure	9.4	Ground Water Depth	11.6		



APPENDIX B: LABORATORY TEST PROGRAM

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

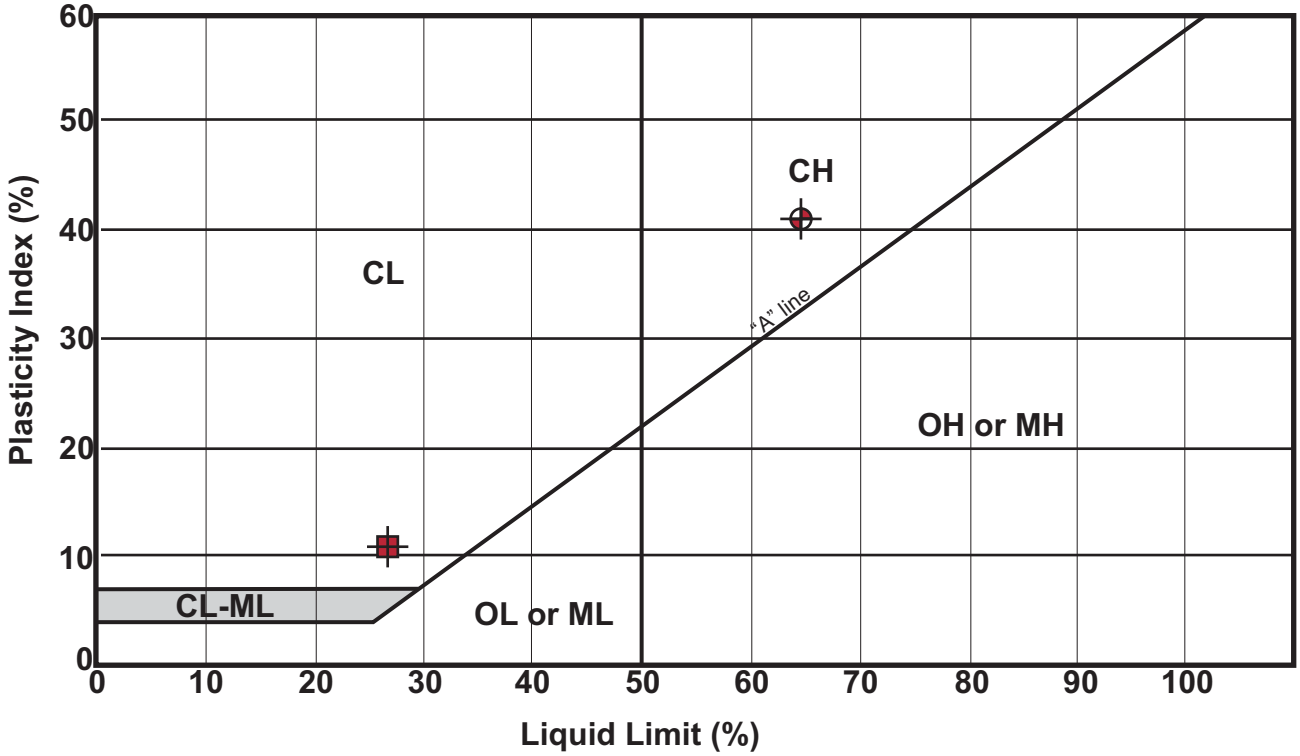
Moisture Content
The natural water content was determined (ASTM D2216) on 10 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

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

Atterberg Limits:
One Atterberg Limit tests (ASTM D4318) was performed on a sample of subsurface soils to measure the range of water contents over which this material exhibits plasticity. In addition, one Atterberg Limit tests was also performed on a granular soil at a depth of 19½ feet to aid in liquefaction evaluation. The Plasticity Indices were used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Test results are summarized on Figure B-1.

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

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-1	3.5	26	64	23	41	--	Fat Clay (CH)
■	EB-1	19.5	14	27	16	11	25	Clayey Sand (SC)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Lester Property Residential Development
San Jose, CA

Project Number: 168-6-1

Figure Number: Figure B1

Date: April 2010

Drawn By: FLL