APPENDIX D

Geotechnical Report

TRC Lowney

Geotechnical Investigation

Residential Development

at Cadence Campus

San Jose, California

Report No. 881-9 has been prepared for:

Essex Property Trust, Inc.

Palo Alto, California

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GEOTECHNICAL INVESTIGATION
RESIDENTIAL DEVELOPMENT
AT CADENCE CAMPUS
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geotechnical investigation for the proposed residential development to be located at the Cadence Campus in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design of the proposed development.

For our use we received a Site Plan, Composite Building Floor Plan and Typical Unit Floor Plans prepared by KTGY Group, Inc. dated May 1, 2006.

1.1 Project Description

As presently planned, the project consists of construction of 6 multi-story residential structures on the approximately 14-acre site. Each of the structures will have 2 levels of parking with the bottom level about 1½ stories below existing grade. The buildings located near the north corner of the site will be a combination of wood frame and concrete construction over 2 parking levels. The other four buildings are planned to consist of wood-framed residential over 2 levels of concrete parking. Structural loads were not determined at the time of the preparation of this report. For our analysis we estimated that the average building areal loads will be approximately 800 to 1,200 and 1,200 to 1,500 pounds per square foot (psf) for dead plus live loads for the 4-story and the 7-story buildings, respectively. We have estimated that maximum building column loads will be about 800 kips for the 4-story building and about 1,300 kips for the 7-story building dead plus live loads.

A small 1-story leasing building may also be constructed at-grade.

1.2 Scope of Services

Our scope of services was presented in detail in our agreement with you dated September 8, 2006. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by advancing 8 Cone Penetration Tests (CPTs), drilling 4 hollow stem auger borings and retrieving soil samples for observation and laboratory testing.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analysis to evaluate site earthwork, building foundations, slabs-on-grade, retaining walls and pavements.

- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

Environmental services were not included as part of this study.

2.0 SITE CONDITIONS

2.1 Exploration Program

Subsurface exploration was performed on October 4 and 5, 2006 using truck-mounted Cone Penetration Test (CPT) equipment and on October 9, 2006 using conventional, truck-mounted hollow stem auger drilling equipment to investigate, sample, and log subsurface soils. The CPTs were advanced to depths ranging from 50 to 120 feet. The 4 exploratory borings were drilled to depths ranging from about 50 to 80 feet. CPTs and borings were permitted and backfilled in accordance with Santa Clara Valley Water District guidelines. A bulk sample of the surface soils from the parking area was obtained for pavement design purposes. The approximate locations of the borings are shown on the Site Plan, Figure 2. Logs of our CPTs and borings and details regarding our field investigation are included in Appendix A. Our laboratory tests are discussed in Appendix B.

2.2 Surface

We also performed a brief surface reconnaissance during our site exploration. The site consists of a 14-acre parcel located east of the intersection of River Oaks Parkway and Seely Avenue. Office buildings are located on the site and to the southeast of the project site. Residential properties border the site to the northwest. A non-operating tree orchard is located to the east and De Las Estros Coyote Creek is located to the northeast.

The relatively level site is gently sloping and is estimated to have approximately 5 feet of topographic relief. The bottom of the creek to the northeast is on the order of about 7 feet lower that the subject site. At the time of our investigation, there was about 1 foot of water in the channel.

2.3 Subsurface

The soil profile at the site appears to be relatively uniform, consisting of approximately 6 to 12½ feet of stiff to hard interbedded sand, clay and silt over stiff clays to depths of about 25 to 38 feet. Below the stiff clay, our explorations encountered generally dense sands to the terminal depths of our explorations, with the exception of CPT-1, which encountered silts and clays to a depth of about 92 feet.

2.4 Ground Water

Free ground water was encountered in the borings at the time of drilling at a depth as shallow as 17 feet. Because the borings were grouted immediately after drilling and due to the clayey nature of the upper profile, this may not represent a stable water table. Based on pore pressure dissipation records obtained during the CPTs, the depth to groundwater is estimated to be on the order of 12 to 13 feet below grade. Information from the California Geologic Survey indicates that the ground water levels in the vicinity are known to be at depths as shallow as about 7 feet. We judge that a groundwater level of 7 feet should be considered in design. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time our measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone) or a City of San Jose Potential Hazard Zone. As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to virtually all developments in the San Francisco Bay Area. The "Seismicity" section that follows summarizes potential levels of ground shaking at the site.

3.3 Liquefaction

3.3.1 General Background

The site is located within a State of California Seismic Hazard Zone for liquefaction (CGS, 2004 - Milpitas Quadrangle). Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are loose to moderately dense, saturated granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment.

When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause increased hydrostatic pressure that induces liquefaction. Liquefaction can cause softening, and large cyclic deformations can result. In loose granular soils, softening can also be accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground (NCEER/NSF, 1998).

Loose granular soil can also settle (compact) during liquefaction and as pore pressures dissipate following an earthquake. Very limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

3.3.2 Subsurface Conditions Encountered

The granular soils encountered in the current explorations were generally medium dense to dense. As hollow-stem drilling methods are not appropriate for determining blow counts, the following discussion of our liquefaction analyses includes only the data collected from our rotary-wash boring and CPTs. As shown below, we encountered several layers of silts and sands below the design ground water depth of 7 feet that have potential for liquefaction. No liquefaction analyses were performed on layers above the design ground water depth.

3.3.3 Methods of Analysis and Results

Our liquefaction analyses followed the methods presented by the 1998 NCEER Workshops (Youd, et al., 2001) in accordance with guidelines set forth in CDMG Special Publication 117 (CDMG, 1997). The NCEER methods for SPT and CPT analyses update simplified procedures presented by Seed and Idriss (1971). The analysis method compares the cyclic resistance ratio (CRR) with the earthquake-induced cyclic stress ratio (CSR) at different depths due to the estimated earthquake ground motions. The relationship for CSR is presented as follows:

$$CSR = 0.65\ (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. CRR is a function of the soil density and grain characteristics.

The factor of safety (FS) against liquefaction is expressed as the ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). If the FS is less than 1.0, the soil is considered to be liquefiable during seismic shaking.

$$FS = CRR/CSR$$

We evaluated the liquefaction potential of the medium dense sand and silt strata encountered using both a pseudo-peak horizontal ground acceleration of 0.51g.

Our CPT tip pressures were corrected for overburden and soil behavior. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor "n" applied to the Normalized Cone Resistance "Q" to evaluate how likely a layer is to contain significant plastic fines and have a low liquefaction potential.

Cyclic Resistance Ratios (CRR) were calculated for the CPT method using normalized CPT tip pressures corrected to clean sand values and the CPT clean sand base curves presented in the NCEER method. The CRRs were then corrected for the design ground water level and magnitude scaling factors. The factor of safety against liquefaction is the ratio of the CRR to the CSR (cyclic stress ratio) or seismic demand on a soil layer based on the Seed and Idriss (1971) equation. Estimates of volumetric change and settlement were determined by the Ishihara and Yoshimine (1990) method. As discussed in the SCEC report, differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement. The results of our analyses are presented below.

Table 2. Results of Liquefaction Analyses – CPT Method

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	*q_{c1N} (tsf)	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)
CPT-1	26.8	4.0	54.8	0.4	Likely	1.1
	31.3	1.5	43.9	0.3	Likely	0.5
	33.2	3.0	96.0	0.5	Likely	0.7
					Total =	**2¼**
CPT-2	35.7	1.5	37.3	0.3	Likely	0.5
					Total =	**¼**
CPT-3	31.3	0.5	27.7	0.2	Likely	0.2
					Total =	**¼**
CPT-4	35.7	3.5	93.7	0.6	Likely	0.7
					Total =	**¾**
CPT-5	19.4	1.5	54.1	0.3	Likely	0.5
	31.0	0.5	32.4	0.3	Likely	0.2
	36.4	0.5	35.7	0.3	Likely	0.2
	37.2	1.0	59.9	0.5	Likely	0.3
	47.5	2.5	102.3	0.6	Likely	0.6
					Total =	**1 ¾**
CPT-6	31.0	0.5	39.5	0.3	Likely	0.3
	33.7	1.0	65.6	0.4	Likely	0.3
	39.6	2.0	1212.0	0.7	Likely	0.4
					Total =	**1**
CPT-7	30.3	4.0	117.1	0.5	Likely	0.9
					Total =	**1**
CPT-8	30.3	1.0	57.7	0.4	Likely	0.3
	33.7	1.0	88.6	0.5	Likely	0.3
	38.6	2.5	123.8	0.6	Likely	0.5
	42.1	1.0	101.6	0.6	Likely	0.2
	43.6	1.5	109.9	0.6	Likely	0.3
					Total =	**1½**

Our analyses indicate that the silt and sand layers theoretically can liquefy, resulting in as much as about 2½ inches of total settlement. As discussed in the SCEC (1999) report, anticipated differential settlements for level sites with deep sediments will be on the order of half of the total estimated settlements, resulting in differential settlement estimates of about 1¼ inch between individual foundation elements.

As the methods of analysis used to determine estimated total settlement do not take into account the possibility of surface ground rupture, we considered the effects of a capping layer. In order for liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must exert a large enough force to break through the surface layer. Based on work by Youd and Garris (1995), it is our opinion that the liquefiable layers are unlikely to vent or boil due to their relatively thin nature and relatively non-liquefiable capping layers.

3.3.4 Summary of Results

To summarize the results of our liquefaction analyses, some sand and silt layers encountered, especially in the upper 50 feet, are theoretically liquefiable. There appears to be enough of a cap to contain the silty sand seams from causing ground surface rupture. Theoretical total liquefaction-induced settlements are estimated to be as much as 2½ inches. Liquefaction-induced differential settlements are estimated to be about 1¼ inch.

3.4 Differential Compaction

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of the near-surface soils. Because the subsurface soils encountered at the site are generally stiff fine grained soils and do not appear to change in thickness or consistency abruptly over short distances, we judge the probability of significant differential compaction at the site to be low.

3.5 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

Although De Las Estros Coyote Creek is located relatively close to the site, because liquefiable layers near the bottom channel elevation were not encountered, and because the proposed structure will have bearing elevations below the bottom of the creek channel, it is our opinion the lateral spreading risk to the proposed structures is low.

4.0 SEISMICITY

4.1 Regional Active Faults

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The San Andreas Fault, which generated the great San Francisco earthquake of 1906, passes about 14 miles southwest of the site. Two other major active faults in the area are the main trace of the Hayward Fault, located about 10 miles northeast of the site, and southeast extension of the Hayward Fault, located about 4 miles northeast.

4.2 Maximum Estimated Ground Shaking

Maps published in the CGS seismic hazards report for the Milpitas Quadrangle (2001) indicate that pseudo-peak horizontal ground acceleration of 0.51g has a 10 percent probability of exceedance in 50 years. Pseudo-peak ground accelerations have been normalized to a 7.5 Mw seismic event, weighted to account for regional fault distances and seismic activities.

4.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2002), referred to as WG02, determined there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2003 and 2032. This result is an important outcome of WG02's work, because any major earthquake can cause damage throughout the region.

This potential was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

4.4 UBC Site Coefficient

Based on our borings and alluvium thickness maps of Santa Clara County (Rogers and Williams 1974), the site is underlain by stiff soils extending to depths in excess of 500 feet. The California Division of Mines and Geology (CDMG) has issued maps locating "Active Fault Near-Source Zones" to be used with the 2001 California Building Code ("Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," CDMG/ICBO February 1998). Faults are classified as either "A," "B," or "C" as shown below. Only faults classified as "A" or "B" are mapped since faults classified as "C" do not increase the near-source factor.

Table 3. Seismic Source Definitions

		Seismic Source Definition*	
Seismic Source Type	**Seismic Source Description**	**Maximum Moment Magnitude, M**	**Slip Rate, SR (mm/yr)**
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C.	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	$M < 6.5$	$SR \leq 2$

*Note: Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining seismic source type.

The following table lists Type A and Type B faults within 25 kilometers of the site:

Table 4. Approximate Distance to Seismic Sources

Fault	Seismic Source Type	Distance (kilometers)
**Hayward (Southeast Extension)	B	6.1
*Hayward (Total Length)	A	10.3
Calaveras (south)	B	11.7
Monte Vista - Shannon	B	16.0
San Andreas (1906)	A	22.0

*Nearest Type A fault
**Nearest Type B fault

Based on this information, the site may be characterized for design based on Chapter 16 of the 2001 CBC using the information in Table 5 below.

Table 5. 2001 CBC Site Categorization and Site Coefficients

Categorization/Coefficient	Design Value
Soil Profile Type (Table 16-J)	S_D
Seismic Zone (Figure 16-2)	4
Seismic Zone Factor (Table 16-I)	0.4
Seismic Source Name	Hayward (main trace)
Seismic Source Type (Table 16-U)	A
Distance to Seismic Source (kilometers)	10.3
*Near Source Factor N_a (Table 16-S)	1.00
Near Source Factor N_v (Table 16-T)	1.19
Seismic Coefficient C_a (Table 16-Q)	0.44
Seismic Coefficient C_v (Table 16-R)	0.76

*Note: For Seismic Zone 4, the near-source factor Na used to determine Ca need not exceed 1.1 for structures complying with all the conditions within CBC Section 1629.4.2.

5.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

5.1 Conclusions

From a geotechnical engineering viewpoint the proposed development may be constructed as planned, provided design and construction is performed in accordance with the recommendations presented in this report.

The primary geologic and geotechnical concerns at the site are:

- Potential for liquefaction
- Shallow ground water
- Deep excavations

We have prepared a brief description of these concerns and presented typical approaches to be considered during project planning. The following conclusions and recommendations are provided for project planning and design. Project plans should be reviewed by TRC Lowney for compliance with our report and so that supplemental recommendations can be prepared, if needed. Additional review may be needed after a contractor is selected so that comments can be presented regarding the contractor's proposed approaches to various aspects of the project.

5.1.1 Potential for Liquefaction

As discussed in the "Liquefaction" section of this report, some of the sand and silt layers are theoretically capable of liquefying during a design earthquake. We estimated that liquefaction-induced total settlements for the buildings will be up to about 2½ inches, with post-liquefaction differential settlement of about 1¼ inch. Shallow foundations or mat foundations should be designed to accommodate the seismic differential settlement. Detailed recommendations are presented in the "Foundation" section of this report.

5.1.2 Shallow Ground Water

As discussed in the "Ground Water" section of this report, we recommend a design ground water level of about 7 feet below existing site grades. Structures constructed below grade will need to be designed to resist hydrostatic pressure or will need to be designed with a permanent dewatering system. The governing jurisdiction should be contacted before proceeding with a dewatering design. Additionally, basement waterproofing will be required. Excavations extending below the ground water will require dewatering.

5.1.3 Deep Excavations

Excavations on the order of 18 to 20 feet will be required for construction of below grade portions of the project. The bottom of the excavation may be up to 13 feet below the ground water level. Temporary dewatering and shoring will be required for the project. Details are discussed in the "Temporary Dewatering" section of this report.

5.2 Plans, Specifications, and Construction Review

Because subsurface conditions may vary from those predicted by relatively small diameter, widely spaced borings, and to check that our recommendations have been properly implemented, we recommend we be retained to 1) review final construction plans and specifications and 2) observe the earthwork and foundation construction. Also, geotechnical conditions can be affected by the construction process. For the above reasons our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

6.0 EARTHWORK

6.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that the backfilling be carried out under our observation.

After clearing, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. At the time of our field investigation, we estimated that a stripping depth of approximately 3 inches would be required in vegetated areas. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement. Alternatively, the loose backfill locations should be carefully documented during demolition for excavation and re-compaction during site grading.

6.2 Removal of Existing Fill

All fills should be removed down to native soil. If the fill material meets the requirements in the "Material for Fill" section below, it may be reused as engineered fill. Side slopes of fill excavations in building and pavement areas should be sloped at inclinations no greater than 3:1 (horizontal to vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

6.3 Abandoned Utilities

Abandoned utilities within the proposed building area should be removed in their entirety. Utilities within the proposed building area would only be considered for in-place abandonment provided they do not conflict with new improvements, that the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structure.
Utilities outside the building area should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged. The potential risks are relatively low for small diameter pipes (4 inches or less) abandoned in-place and increasingly higher with increasing diameter.

6.4 Subgrade Preparation

After the site has been properly cleared, stripped, and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment. If the subgrade is wet or unstable at the bottom of the basement excavation, stabilization consisting of 18 to 24 inches of crushed rock over a stabilization fabric may be required as a working surface. A good dewatering program for excavations will help provide a stable excavation base.

6.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with no more than 15 percent larger than 2½ inches. Imported and non-expansive fill material should be inorganic and should have a Plasticity Index of 15 or less. Imported fill should have sufficient binder to prevent caving of the foundation and utility trenches. Proposed imported fill should be approved by a member of our staff at least four days prior to delivery to the site. Compliance testing for aggregate base may take up to 10 days to complete.

Consideration should also be given to the environmental characteristics as well as the corrosion potential of imported fill. Laboratory testing, including pH, soluble sulfates, chlorides, and resistivity will provide information regarding corrosion potential. Import soils should not be more corrosive than the native materials.

6.6 Compaction

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be uniformly compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness at a uniform moisture content near the laboratory optimum. Each successive lift should be relatively firm and non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

6.7 Wet Weather Conditions

Earthwork contractors should be made aware of the moisture sensitivity of clayey soils and potential compaction difficulties. If construction is undertaken during wet weather conditions, the surficial soils may become saturated, soft and unworkable. Subgrade stabilization techniques might include the use of engineering fabrics and/or crushed rock or chemical treatment. Therefore, we recommend that consideration be given to construction during summer months. Recommendations for stabilizing subgrade soils at the bottom of the excavation are present in the "Subgrade Preparation" section of this report.

6.8 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of low permeability soil be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas and coming into contact with expansive subgrade material.

6.9 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

On a preliminary basis, and based on the soils encountered in the upper 20 feet during our site exploration, site soils can be classified as Type C based on soil classification proposed by OSHA. A representative of TRC Lowney should be retained to verify soil conditions in the field at the time of excavation to finalize our classification of soil type.

6.10 Temporary Shoring Support System

As previously discussed, excavations on the order of 18 to 20 feet are planned to construct the two levels of underground parking. The excavations could potentially be temporarily supported by several methods including tiebacks, soil nailing, braced shoring, temporary slopes if space is adequate, or potentially other methods. Where shoring is required, restrained shoring will most likely be necessary to limit deflections and disruption to nearby improvements. It has been our experience that cantilever shoring might be feasible for temporary shoring to a height of only about 10 feet where allowable deflections are limited. The choice of shoring method should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice of support.

The temporary shoring should be designed for additional surcharges due to adjacent loads such as from construction vehicles, street traffic, and adjacent buildings. To prevent excessive surcharging of the walls, we recommend that heavy loads such as construction equipment and stockpiles of materials be kept at least 20 feet from the top of the excavations. If this is not possible, the shoring must be designed to resist the additional anticipated lateral loads. Shoring systems should be designed with sufficient rigidity to prevent detrimental lateral displacements. Minimum parameters/loads for design of a temporary shoring system are given in Table 6.

Table 6. Temporary Shoring System Design Parameter

Design Parameter	Design Value (psf)
Minimum Lateral Wall Surcharge[1]	120 psf
Earth Pressure[2] From ground surface to H/4 (ft)	Increase from 0 to 25H psf
Earth Pressure Below H/4 (ft)	Uniform pressure of 25H psf
Passive Pressure[3]	800 psf

Note:
1 For the upper 5 feet (minimum for incidental loading)
2 Where H equals height of excavation
3 Can assume to act over 2 times the diameter of soldier piles, neglecting the upper foot

In order to limit potential movements of the shoring system, the shoring designer and contractor should consider several design and construction issues. For the movements of shoring to be reduced, the designer will have to provide for a uniform and timely mobilization of the soil pressures. Tiebacks or interior bracing should be loaded to the design loads prior to excavation of the adjacent soil so that load induced strains in the retaining system will not result in the system moving toward the excavation. In addition, a relatively stiff shoring system should be designed to limit deflections under loading. In general, we recommend designing a shoring system to deflect less than about ½-inch.

In addition, ground subsidence and deflections can be caused by other factors, such as voids created behind the shoring system by over-excavation, soil sloughing, erosion of sand or silt layers due to perched water, etc. All voids behind the shoring system should be filled by grouting to minimize potential problems as soon as feasible during installation of the shoring system.

Our borings were drilled with a hollow-stem auger, so we were not able to evaluate the potential for caving of site soils, which may become a factor during soldier pile and/or tie-back installation. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes using proposed production drilling equipment may be prudent, to evaluate possible excavation difficulties such as caving soils, cobbles, boulders and/or other excavation difficulties.

In conjunction with the shoring installation, as previously discussed, a monitoring program should be set up and carried out by the contractor to determine the effects of the construction on adjacent buildings and other improvements such as streets, sidewalks, utilities and parking areas. As a minimum, we recommend horizontal and vertical surveying of reference points on the shoring and on adjacent streets and buildings, in addition to an initial crack survey. We also recommend that all supported and/or sensitive utilities be located and monitored by the contractor. Reference points should be set up and read prior to the start of construction activities. Points should also be set on the shoring as soon as initial installations are made. Alternatively, inclinometers could be installed by the contractor at critical locations for a more detailed monitoring of shoring deflections. Surveys should be made at least once a week, and more frequently during critical construction activities, or if significant deflections are noted. TRC Lowney can provide inclinometer materials and has the equipment and software to read and analyze the data quickly.

This report is intended for use by the design team. The contractor shall perform additional subsurface exploration and/or geotechnical studies as they deem necessary for the chosen shoring system. The contractor is also responsible for site safety and the means and methods of construction, including temporary shoring. Temporary shoring must be designed by a licensed California Civil or Structural Engineer. Prior to construction, we recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review.

6.11 Temporary Dewatering

As previously discussed, measured ground water elevations and historic high ground water levels are above the planned excavation depths; therefore, temporary dewatering will be necessary during construction. Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions in the San Jose area, they should be aware that modifications to the dewatering system, such as adding well points, may be required during construction depending on the conditions encountered.

We recommend that any dewatering of the site be carried out in such a manner as to maintain the ground water a minimum of 5 feet below the bottom of the mass excavation. The contractor should design a system to achieve this criteria. Additionally, the ground water should be maintained a minimum of 2 feet below all local excavations for deepened foundations, utilities or other structures. Should dewatering be temporarily shut down, it could have considerable detrimental affects on the excavations, including flooding, destabilization of the bottom of the excavation, shoring failures, etc. Therefore, we recommend that consideration be given to having the dewatering contractor provide backup power in case of loss of power or other redundancies, as deemed necessary.

Special considerations may be required prior to discharge of ground water from dewatering activities depending on the environmental impacts at the site or at nearby locations. These requirements may include storage and testing under permit prior to discharge. Impacted ground water may require discharge at an offsite facility.

Dewatering may cause subsidence of ground around the project area. Excessive drawing down of the ground water should not be permitted. We estimate that if the ground water is drawn down to about 23 feet, about 1-inch of subsidence could occur adjacent to the excavation. It is essential that we review the contractor's dewatering plan prior to construction.

6.12 Surface Drainage

Positive surface water drainage gradients (2% minimum) should be provided within 5 feet of the buildings adjacent to the structures to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be carried at least 5 feet away from foundations and slabs in closed conduits and directed to suitable discharge facilities. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

6.13 Construction Observation

All grading and earthwork should be performed under the observation of our representative to check that the site is properly prepared, that selected fill materials are satisfactory, and that placement and compaction of fills is performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. The project plans and specifications should incorporate all recommendations contained in this report.

7.0 FOUNDATIONS

For the proposed multi-story buildings, shallow foundations were not judged to be practical, due to excessive settlement. Based on assumed structural loads, we estimate shallow foundation settlements will exceed two inches under static loading. In addition, about 1¼ inch of liquefaction induced differential settlement is anticipated.

It is our opinion that conventionally reinforced mat foundations designed to support the structure and resist hydrostatic ground water pressures will be the most suitable type of foundation type for the project. If settlements are not tolerable from a structural engineering view point, deep foundations and basement floors designed to resist hydrostatic ground water pressures may be required.

Structural loads were not available at the time this report was prepared. We anticipate that maximum dead plus live column loads for 4-stories of wood-frame residential over two levels of concrete-frame parking will be on the order of 800 kips; we estimate the average areal bearing pressure to be approximately 800 to 1,200 psf. For the 7 story wood and concrete-frame residential over 2-levels of concrete-frame parking, we estimate that maximum dead plus live column loads will be on the order of 1,300 kips, with maximum average areal pressures of about 1,200 to 1,500 psf. Details regarding building designs were not available at the time this report was prepared. Differences in loading and bearing elevation assumptions may change our analysis significantly and could require deep foundations or other foundation alternatives.

At-grade wood-frame single story structures, such as leasing buildings, may be supported on conventional reinforced mat foundations designed in accordance with section 1815 of the CBC provided the estimated settlements are tolerable from a structural engineering viewpoint.

7.1 Reinforced Mat Foundations for Multi-Story Residential over Parking

The proposed multi-story buildings may be supported on conventionally reinforced mat foundations, provided the estimate a total and differential settlements discussed below are tolerable from a structural engineering viewpoint. The estimates provided are preliminary and based on assumed structural loads. We should be retained to revise our estimates ones structural loads are available.

All mats should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the mat foundation pads prior to placement of reinforcing steel. Details regarding design of the mat foundations for the 4-story and 7-story buildings over the 2-levels of parking are presented below.

7.1.1 4-Story Residential over 2-Levels of Parking

We estimate that the proposed structures will have an average areal bearing pressure of approximately 800 to 1,200 pounds per square foot (psf). Under concentrated loading areas we recommend that the mat foundation be designed with a maximum allowable bearing pressure of 3,750 for dead plus live loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

On a preliminary basis, for mats bearing approximately 18 to 20 feet below the existing site grades, we estimate that the center of the mat will undergo approximately 1½ to 1¾ inches of total settlement. At the edges of the mat approximately 1-inch of total settlement can be expected under dead plus live loading. We anticipate that mat foundations will be designed using finite element modeling. During foundation design, we should work iteratively with the structural engineer to converge on appropriate modulus of subgrade reaction values, which will vary across the mat, and are not only dependant on soil properties, but also the loading magnitudes and geometry. For an initial iteration, we recommend a modulus of subgrade reaction of 5 pounds per cubic inch (pci) and 10 pci at the mat edges. Once an initial finite element analysis is complete, we should be provided with the mat contact pressure and deflection output to revise our modulus of subgrade reaction estimates. In addition, the mat foundations should be capable of resistively approximately 1¼ inch of differential settlement between the center and edges of the mat due to seismically induced liquefaction settlement.

Mats should also provide resistance to hydrostatic groundwater pressures. Details are discussed in the "Hydrostatic Pressures" section of this report.

7.1.2 7-Story Residential over 2-Levels of Parking

We estimate that the proposed structures will have an average areal bearing pressure of approximately 1,200 to 1,500 pounds per square foot (psf). Under concentrated loading areas we recommend that the mat foundation be designed with a maximum allowable bearing pressure of 3,750 for dead plus live loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

On an initial basis, for mats bearing approximately 18 to 20 feet below the existing site grades, we estimate that the center of the mat will undergo approximately 1¾ to 2¼ inches of total settlement. At the edges of the mat approximately 1-inch of total settlement can be expected under dead plus live loading. We anticipate that mat foundations will be designed using finite element modeling. During foundation design, we should work iteratively with the structural engineer to converge on appropriate modulus of subgrade reaction values, which will vary across the mat, and are not only dependant on soil properties, but also the loading magnitudes and geometry. For an initial iteration, we recommend a modulus of subgrade reaction of 5 pounds per cubic inch (pci) and 10 pci at the mat edges. Once an initial finite element analysis is complete, we should be provided with the mat contact pressure and deflection output to revise our modulus of subgrade reaction estimates. In addition, the mat foundations should be capable of resisting approximately 1¼ inch of differential settlement between the center and edges of the mat due to seismically induced liquefaction settlement.

Mats should also provide resistance to hydrostatic groundwater pressures. Details are discussed in the "Hydrostatic Pressures" section of this report.

7.2 Mat Foundations for Light at-grade Structures

The proposed single story at-grade buildings may be supported on conventionally reinforced mat foundations. Mat foundations may be designed in accordance with the 2001 Uniform Building Code Section 1815, using an effective weighted plasticity index of 15 and a minimum cantilever length of 2 feet. All mats should be designed with a thickened edge at least 12 inches wide and 12 inches thick. The thickened edge should be considered from top to bottom of mat.

Mats are anticipated to have bearing pressure of less than 300 pounds per square foot (psf) for dead plus live loads. We recommend maximum localized allowable bearing pressures of 2,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

All mats should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the mat foundation pads prior to placement of reinforcing steel.

We estimate that total post-construction differential movement should be approximately 1/2-inch across the proposed buildings under static loads. The structures should also be checked for about 1-inch of differential settlement across the mats due to post-earthquake liquefaction induced settlement. If foundations designed in accordance with the above recommendations are not capable of resisting the settlements described above, additional reinforcing may be required.

If desired to minimize floor wetness in habitable areas, we recommend that a moisture barrier system be constructed, as described below.

7.3 Hydrostatic Ground Water Pressures and Waterproofing

Below grade structures should be designed to be permanently dewatered or should be designed to resist hydrostatic ground water pressures. We recommend a ground water level of about 7 feet below the existing site grade be used for design. Suitable factors of safety (or freeboard) should be applied to the ground water pressure design.

If permanent dewatering is desired, local jurisdictions should be contacted to verify that permanent dewatering is permitted.

Basement walls and floors should be waterproofed to prevent unwanted moisture from seeping into the basement. We recommend an experienced waterproofing consultant provide detailed design recommendations for dewatering.

7.4 Lateral Loads

Lateral loads may be resisted by friction between the bottom of concrete mats and the supporting subgrade. A maximum allowable frictional resistance of 0.25 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against deepened mats poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

7.5 At-Grade Moisture Protection Considerations

Since the long-term performance of concrete slabs-on-grade depends on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete floor of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with slab-on-grade floor construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 10-mil thick vapor barrier should be placed directly below the slab-on-grade floors. The vapor barrier should extend to the edge of the slab-on-grade floors. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break. The vapor barrier should be sealed at all seams and penetrations.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels should not be permitted.

- When using Type I cement: all concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 7 days. When using Type II cement, all concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 14 days.

- Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

- Water vapor emission levels and pH should be determined as required by the manufacturer's of the floor covering materials before floor installation. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted, the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

8.0 RETAINING WALLS

8.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top, such as below grade garage walls, be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

For basement walls constructed below the ground water table, the walls should be designed for undrained loading unless a permanent dewatering system is provided.

8.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 2 feet out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as TCMirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively impervious compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall.

8.3 Backfill

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

8.4 Foundation

Below-grade garage walls will likely be supported on mat foundations designed for the structures; at-grade retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Footings" section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the "Lateral Loads" section.

9.0 PAVEMENTS

9.1 Asphalt Concrete

We obtained a representative bulk sample of the surface soil from the parking area and performed an R-value test to provide data for pavement design. The results of the test are included in Appendix B and indicate an R-value of 20. Because surface soil vary across the site we recommend a design R-value of 10. Using estimated traffic indices for various pavement-loading requirements, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 7.

Table 7. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R-Value = 10

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	7.0	9.5
	4.5	2.5	8.5	11.0
Automobile Parking Channel	5.0	3.0	9.0	12.0
	5.5	3.0	11.0	14.0
Truck Access & Parking Areas	6.0	3.5	11.5	15.0
	6.5	4.0	13.0	17.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

9.2 Portland Cement Concrete Pavements

Recommendations for Portland Cement Concrete (PCC) pavements are presented below in Table 8. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 8. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
10	6
25	6.5
300	7

R-value testing resulted in an R-value of 20. Because surface soil may vary across the site we recommend a design R-value of 10. The table above is based on an R-value of 10 and a modulus of rupture of 500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the Portland Cement Concrete pavements to control the cracking inherent in this construction.

9.3 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

9.4 Exterior Sidewalks

We recommend that exterior concrete sidewalks be at least 4 inches thick and underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Portland Cement Concrete Pavements" section of this report.

10.0 LIMITATIONS

This report has been prepared for the sole use of Essex Property Trust, Inc., specifically for design of the Cadence Campus in San Jose, California. The opinions presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discreet locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we 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, TRC Lowney cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC Lowney' report by others. Furthermore, TRC Lowney will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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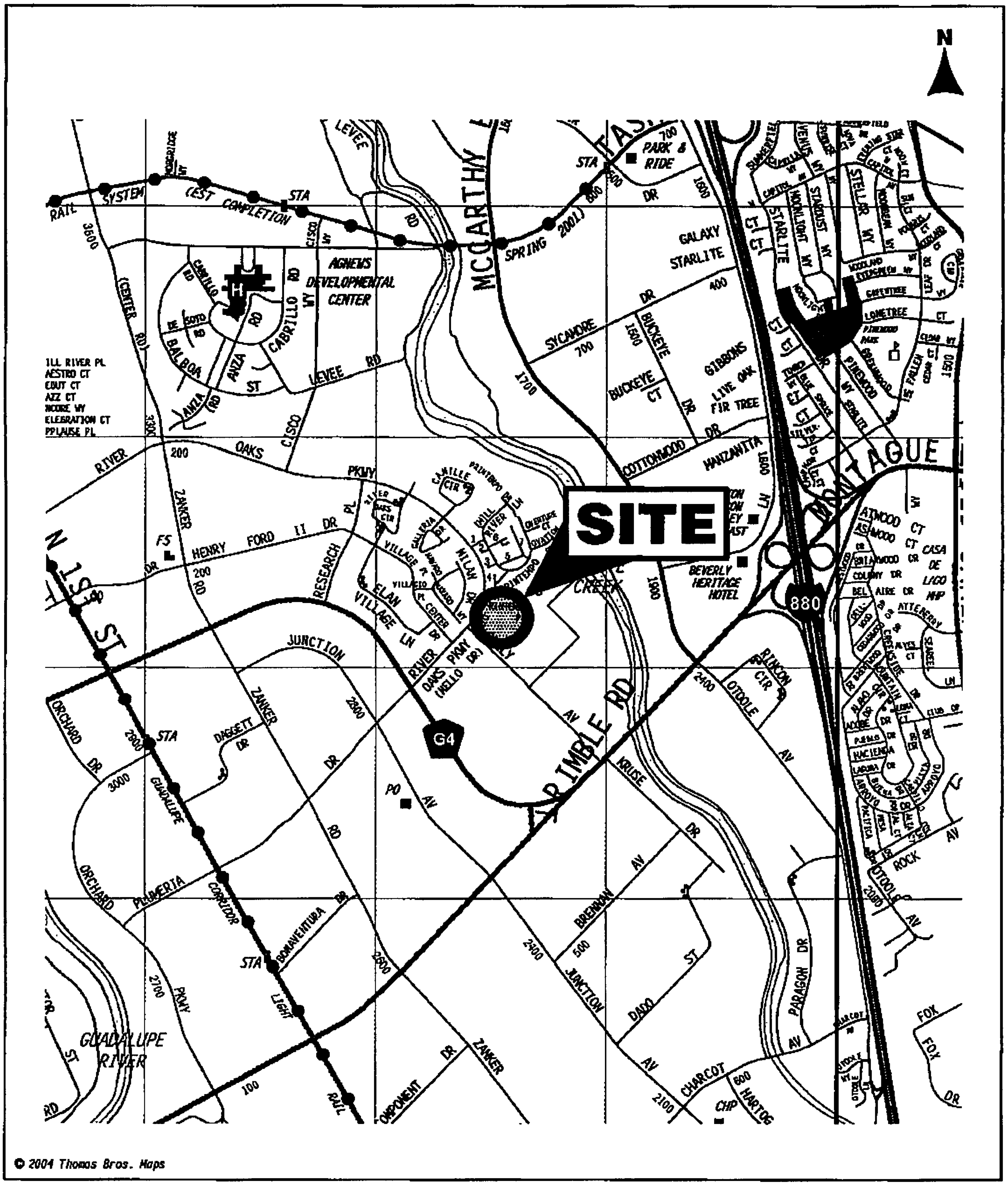


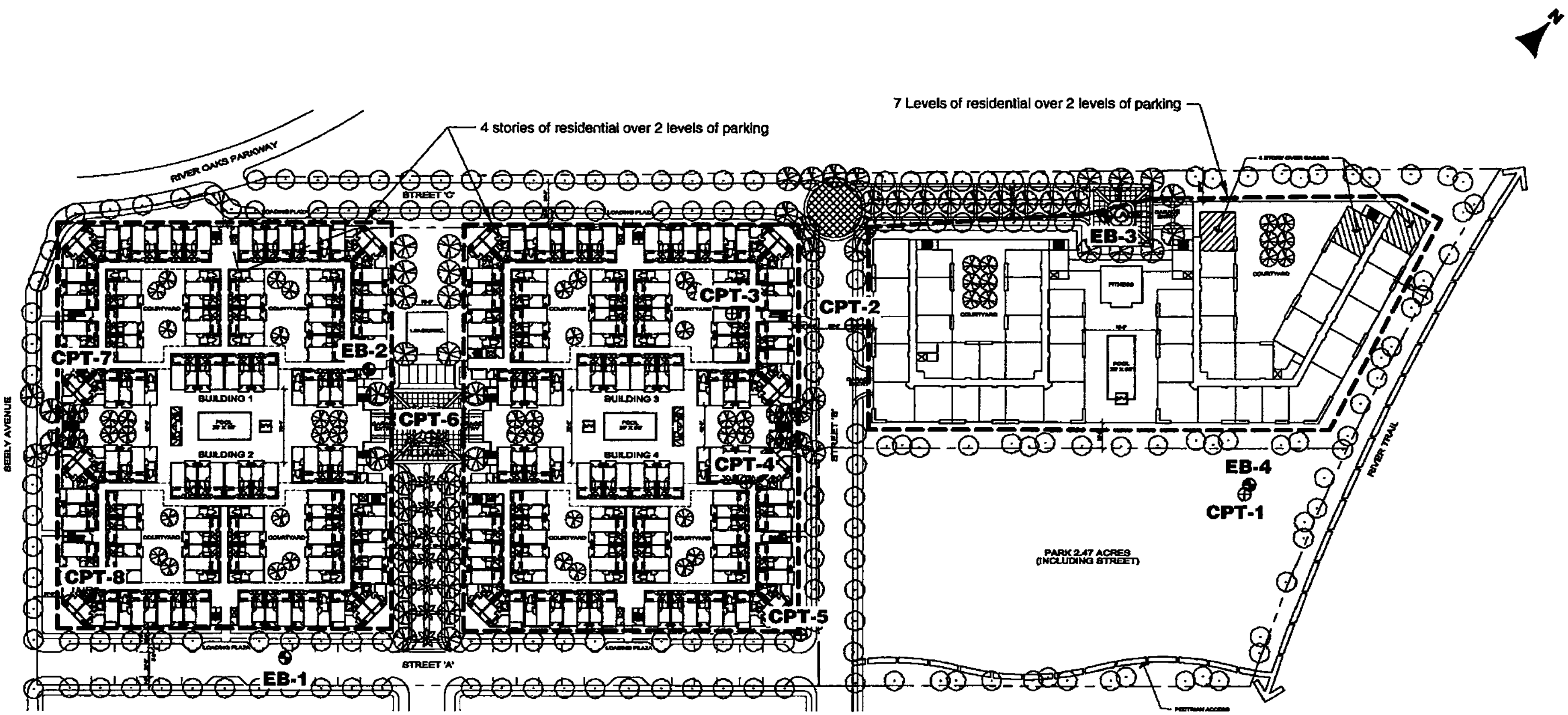
11/05 EB

VICINITY MAP

RESIDENTIAL DEVELOPMENT AT CADENCE CAMPUS
San Jose, California

TRC Lowney

FIGURE 1
881-9



LEGEND

- Approximate location of exploratory boring
- Approximate location of cone penetration test

Base by KTGY Group, dated 5/1/06

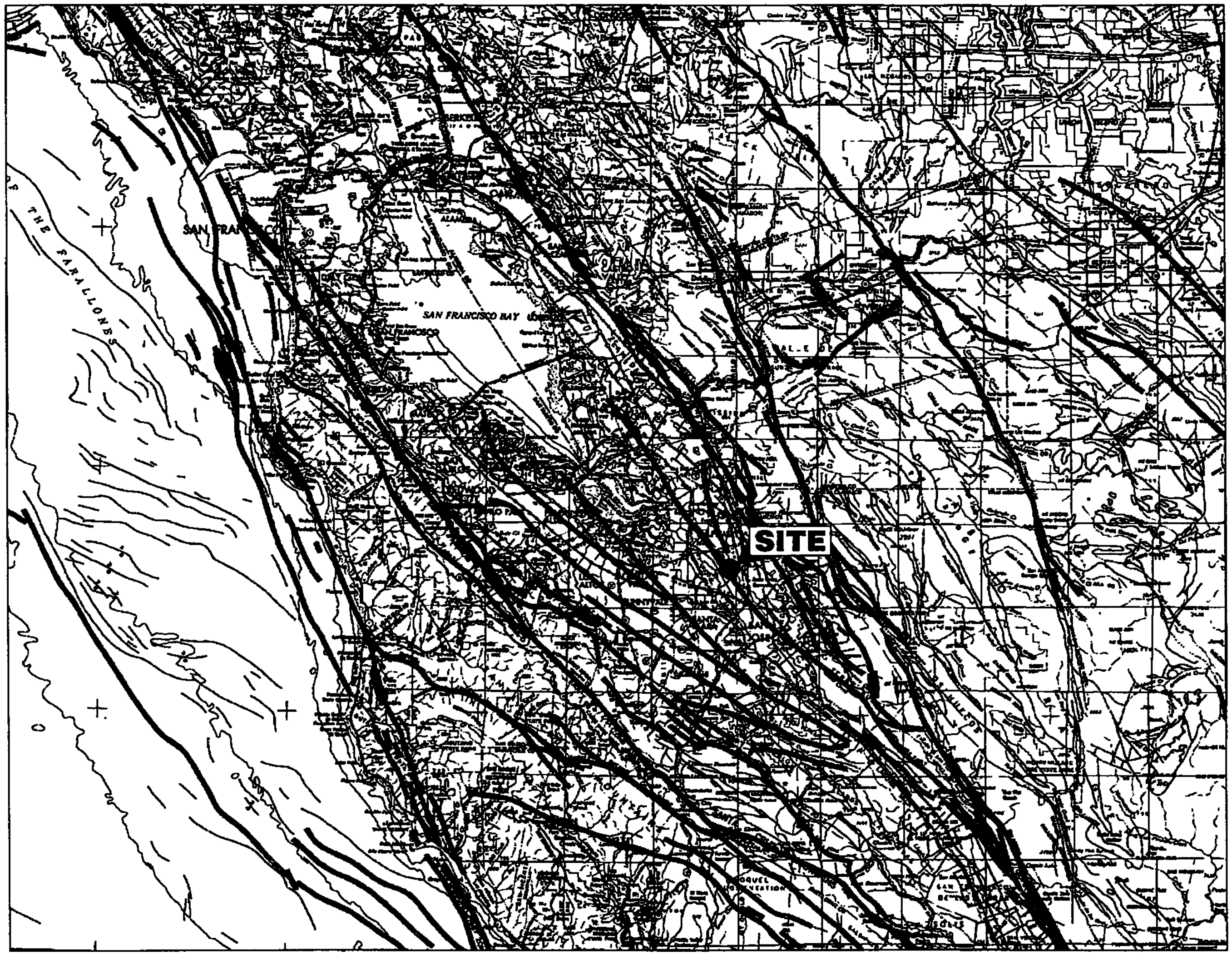
SITE PLAN
RESIDENTIAL DEVELOPMENT
AT CADENCE CAMPUS
San Jose, California

TRC Lowney

FIGURE 2

881-9

11/06'EB



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

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]
		Pleistocene	10,000			Faults showing evidence of displacement during late Quaternary time[3]
	Early Quaternary		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.

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bartugno & others (1991)

0 10 20 Miles

Scale 1:500,000

REGIONAL FAULT MAP

RESIDENTIAL DEVELOPMENT
AT CADENCE CAMPUS
San Jose, California

TRC Lowney

FIGURE 3

881-9

Division of Mines and Geology, James F. Davis, State Geologist

11/06/EB

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted Cone Penetration Test (CPT) equipment and truck-mounted hollow-stem auger drilling equipment. Eight 2-inch-diameter CPTs were drilled on October 4 and 5, 2006, to a maximum depth of 120 feet. Four 8-inch-diameter exploratory borings were drilled on October 9, 2006 to a maximum depth of 80 feet. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the CPTs and the borings, as well as a key to the classification of the soil, are included as part of this appendix.

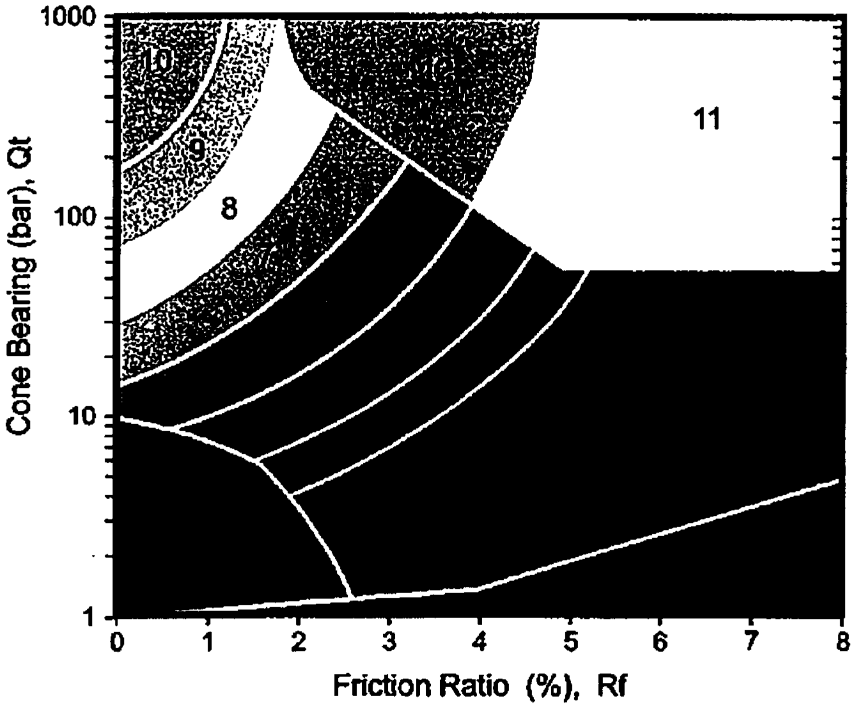
The locations of CPTs and the borings were approximately determined by pacing from site features. The locations of the CPTs and the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 2.5-inch I.D. samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the last two 6-inch increments is the uncorrected Standard Penetration Test measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring 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 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.

* * * * * * * * * * * * *



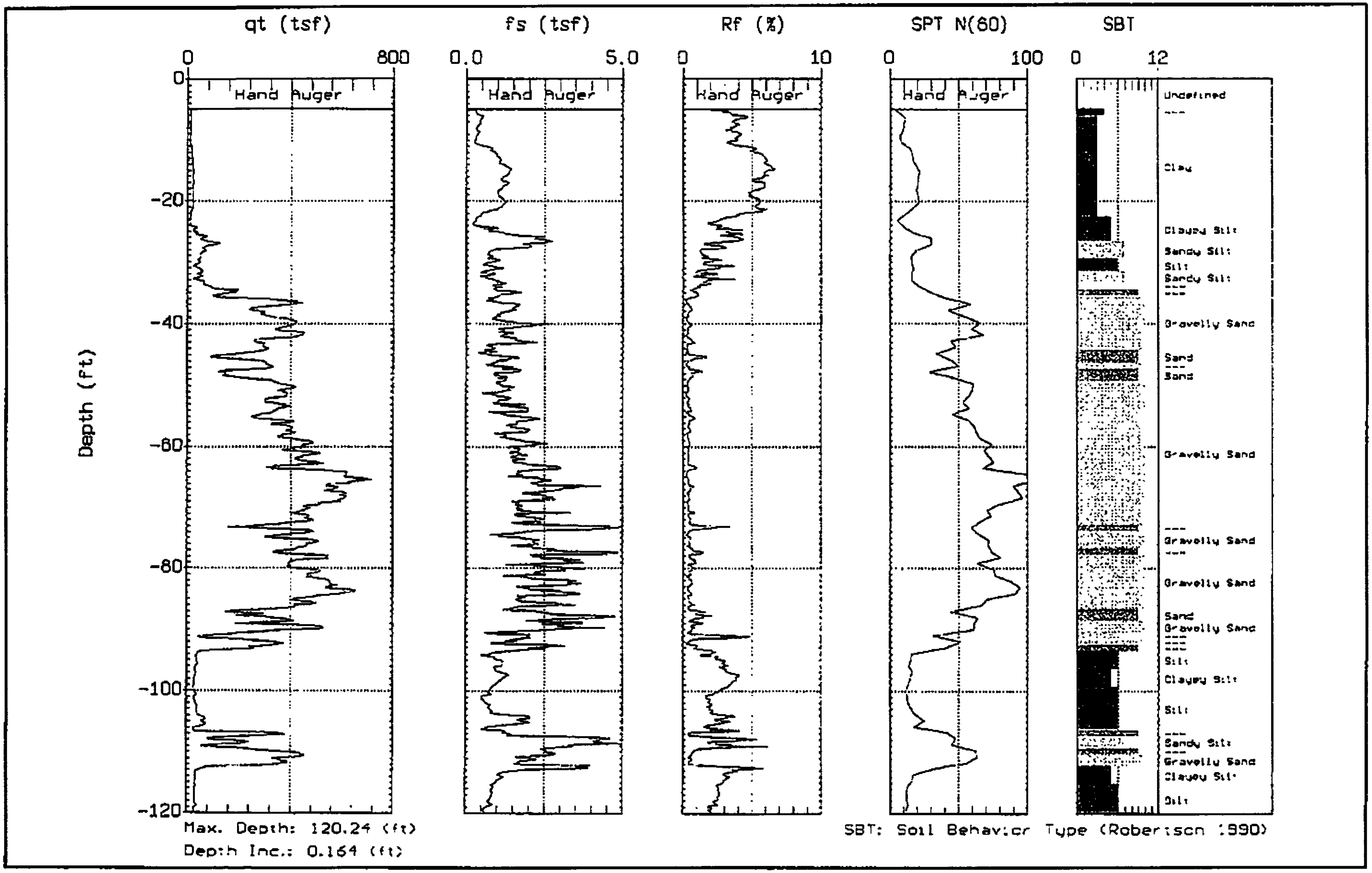
Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

Robertson (1990)

KEY TO CONE PENETROMETER TEST

TRC Lowney



11/06'EB

CONE PENETRATION TEST - CPT-1

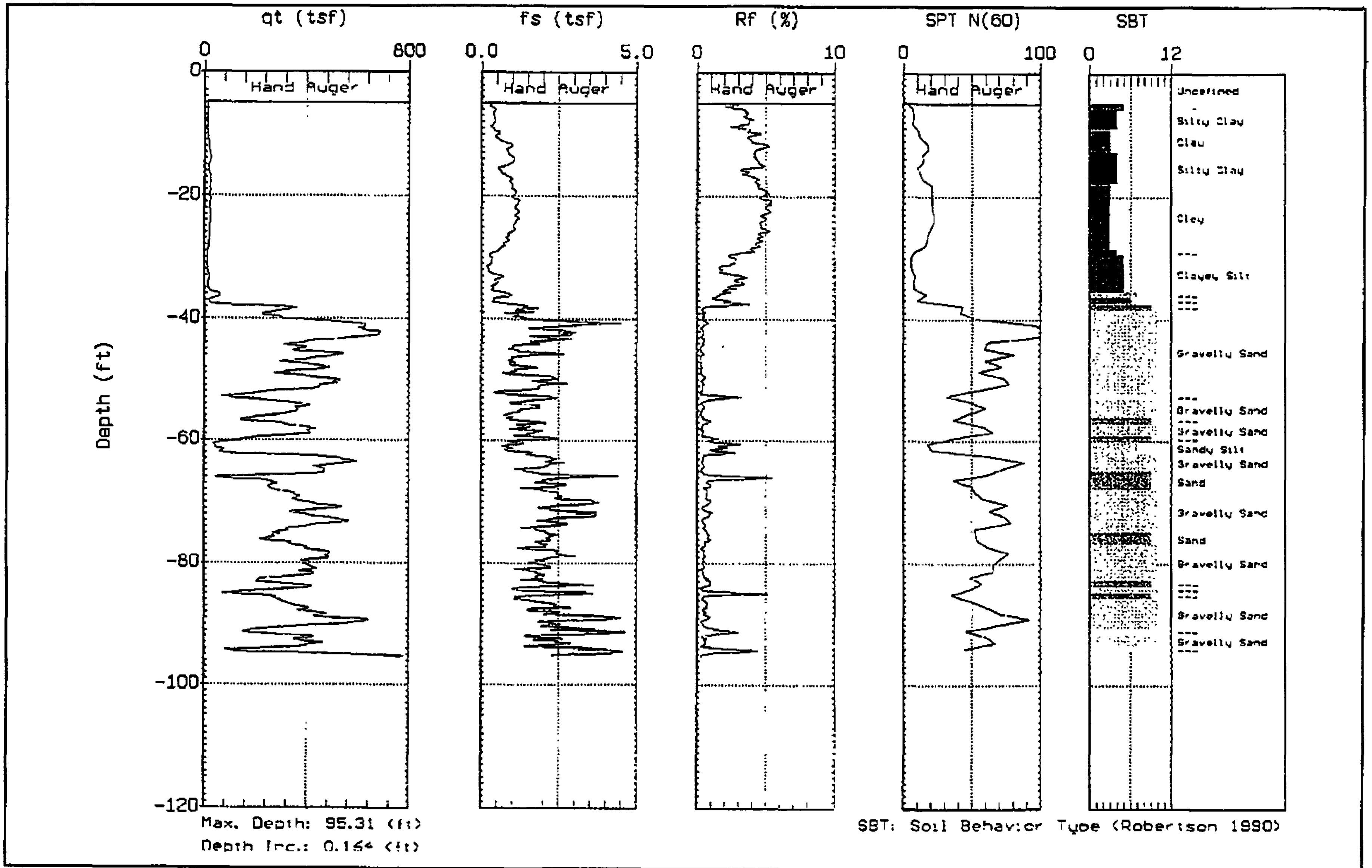
CADENCE CAMPUS

San Jose, California

TRC Lowney

CPT-1

881-9



11/06*EB

CONE PENETRATION TEST - CPT-2

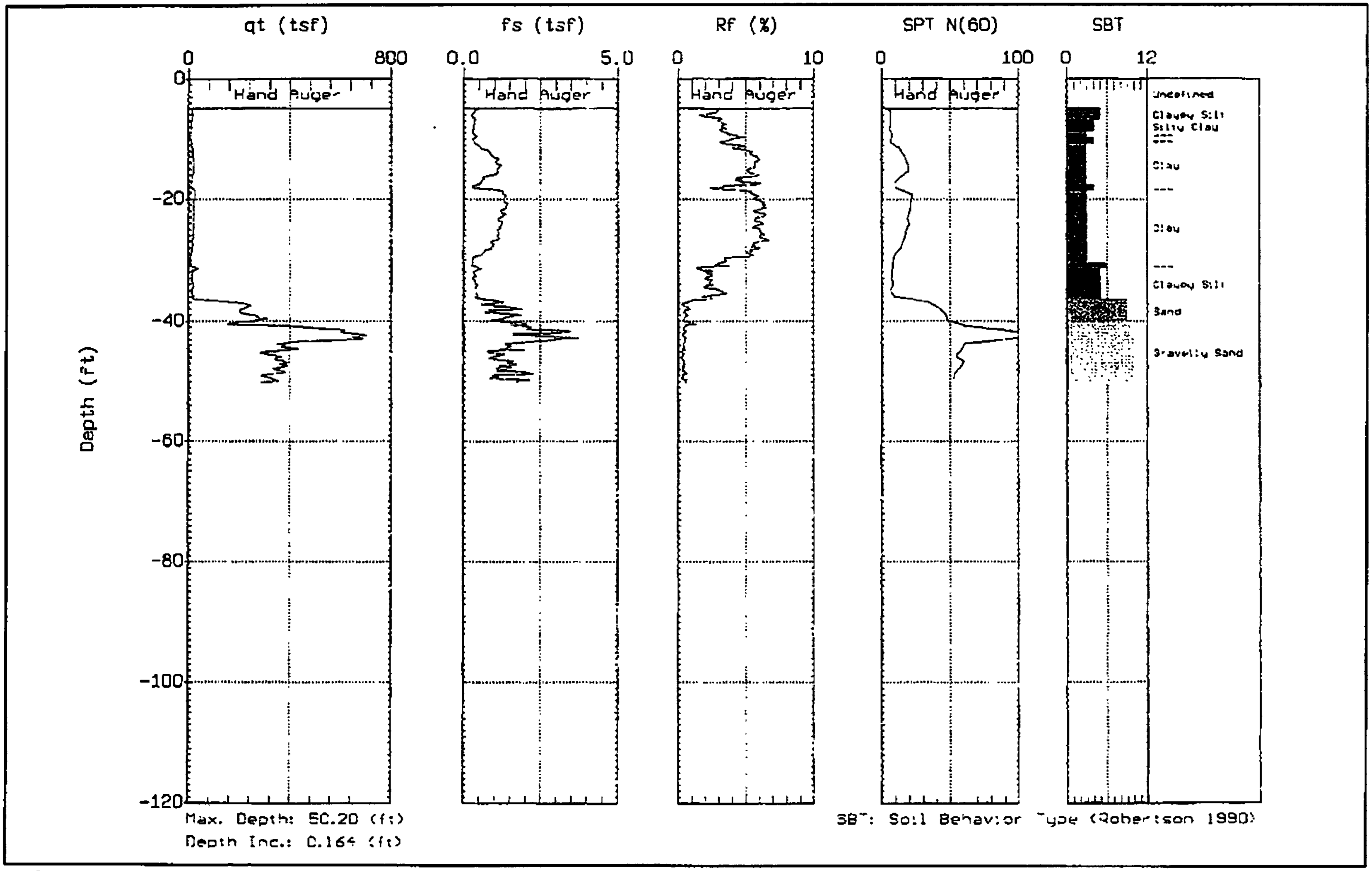
CADENCE CAMPUS

San Jose, California

TRC Lowney

CPT-2

881-9



11/05'EB

CONE PENETRATION TEST - CPT-3

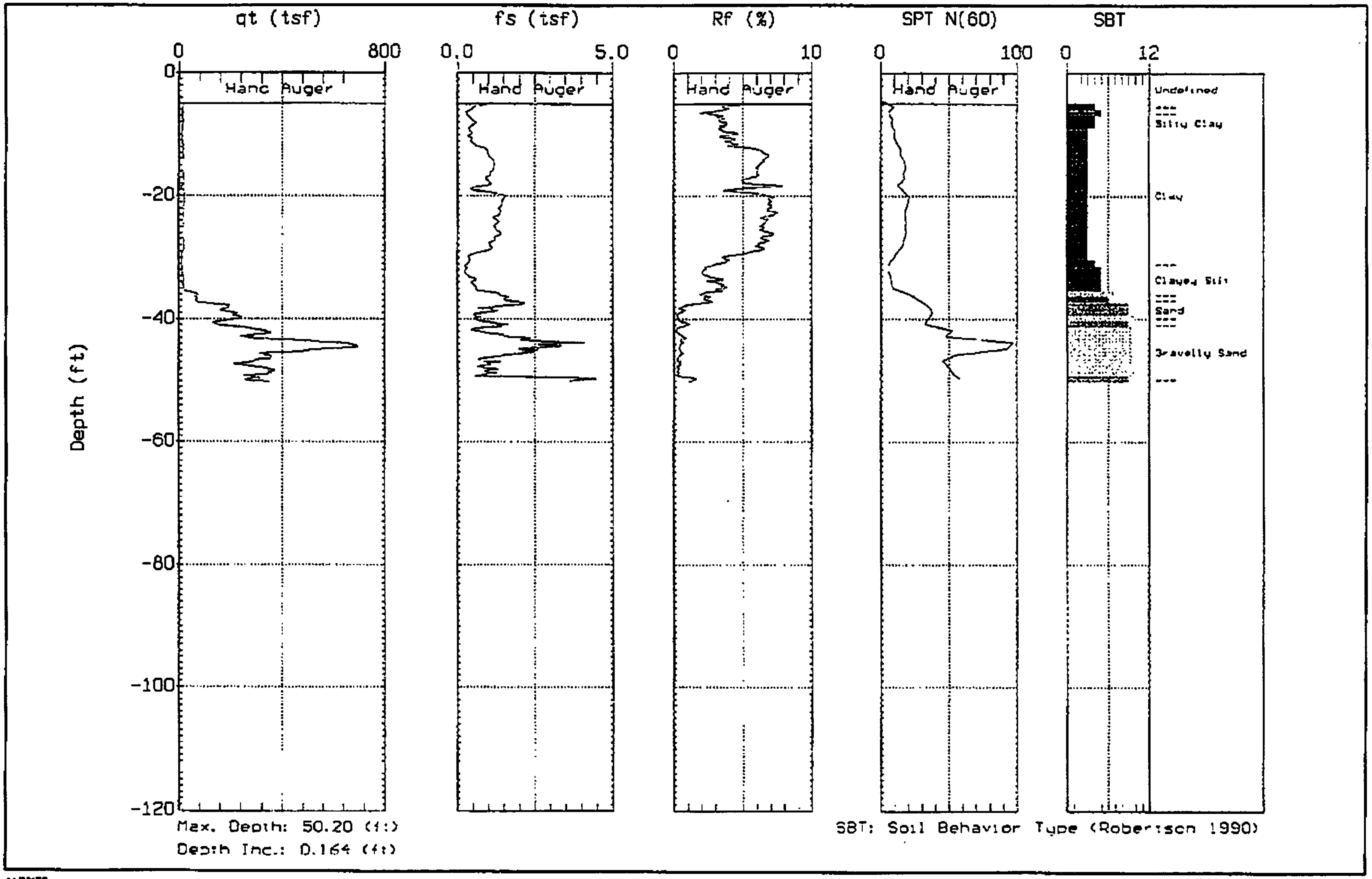
CADENCE CAMPUS

San Jose, California

TRC Lowney

CPT-3

881-9



11/06*EB

CONE PENETRATION TEST - CPT-4

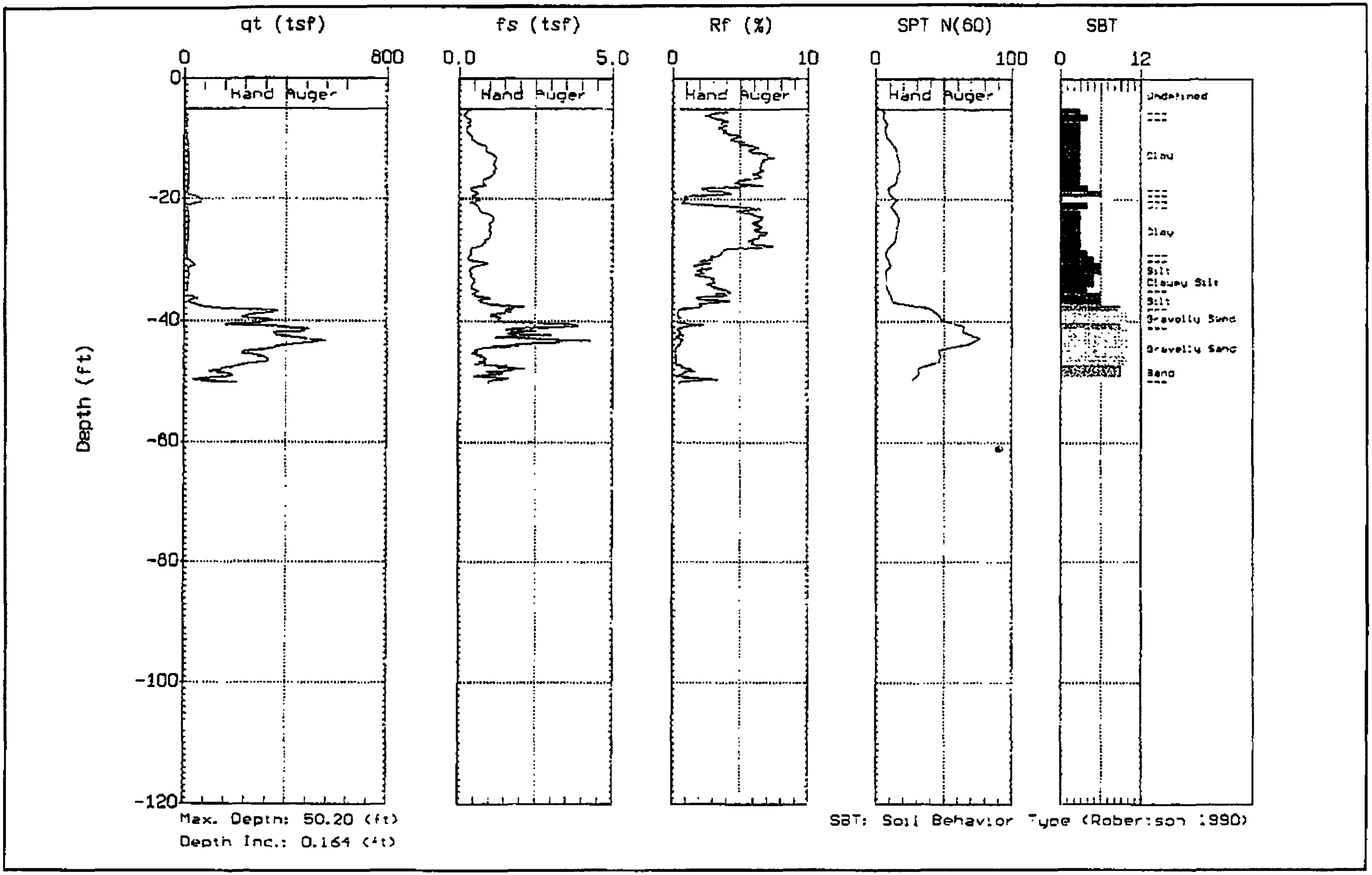
CADENCE CAMPUS

San Jose, California

TRC Lowney

CPT-4

881-9



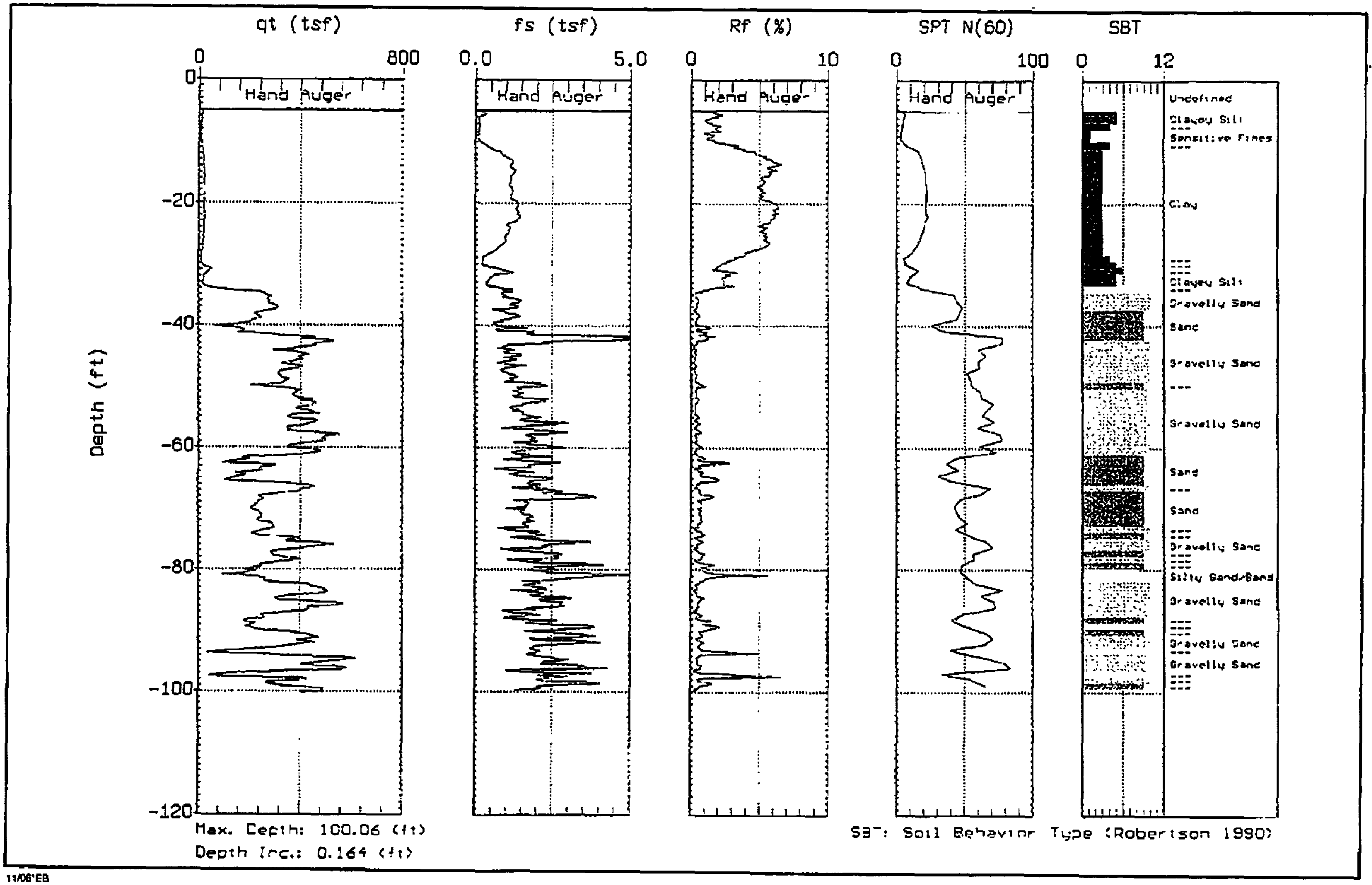
11/06/'88

CONE PENETRATION TEST - CPT-5

CADENCE CAMPUS
San Jose, California

TRC Lowney

CPT-5
881-9



11/06'EB

CONE PENETRATION TEST - CPT-6

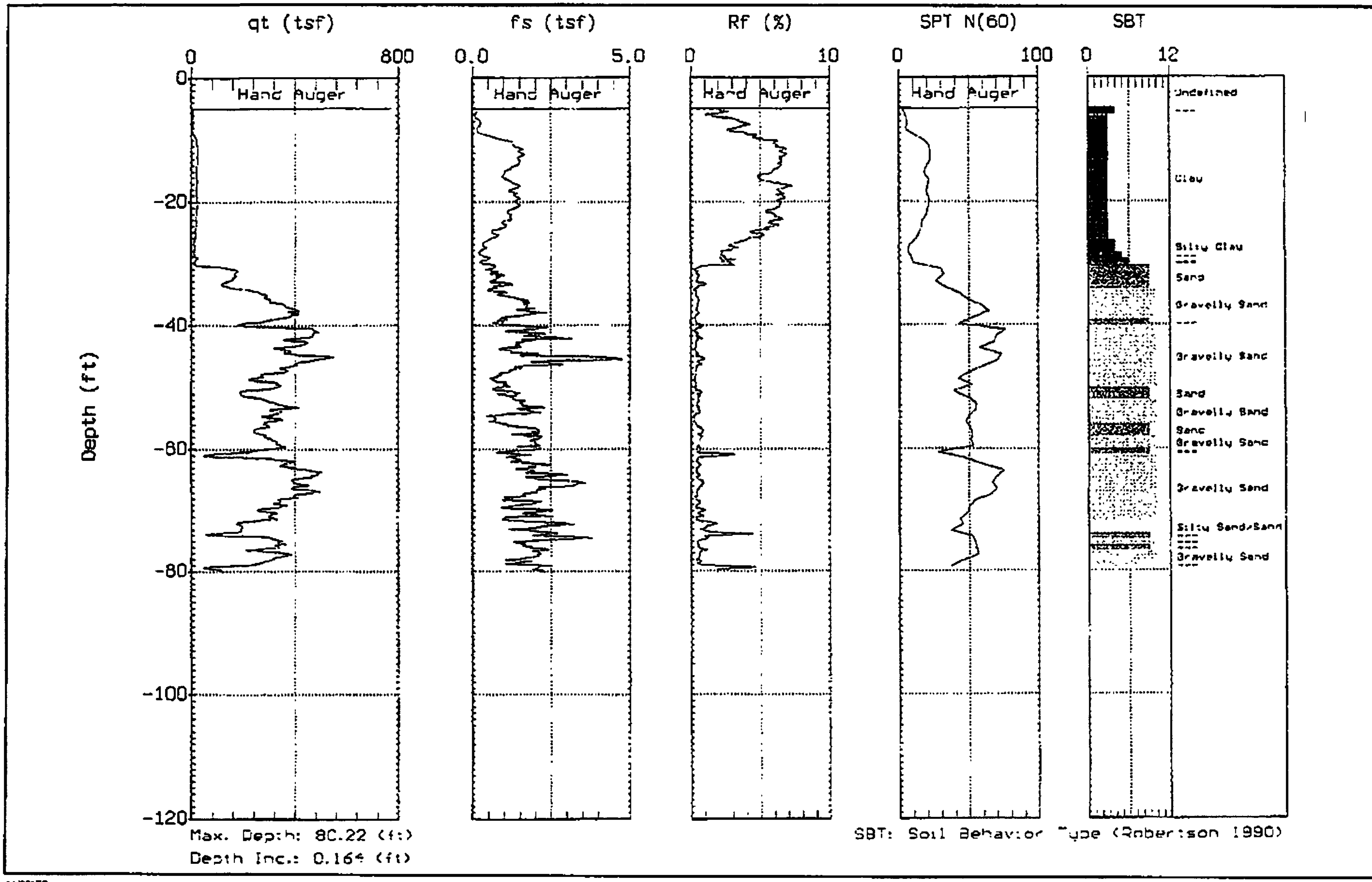
CADENCE CAMPUS

San Jose, California

TRC Lowney

CPT-6

881-9



11/06'EB

CONE PENETRATION TEST - CPT-7

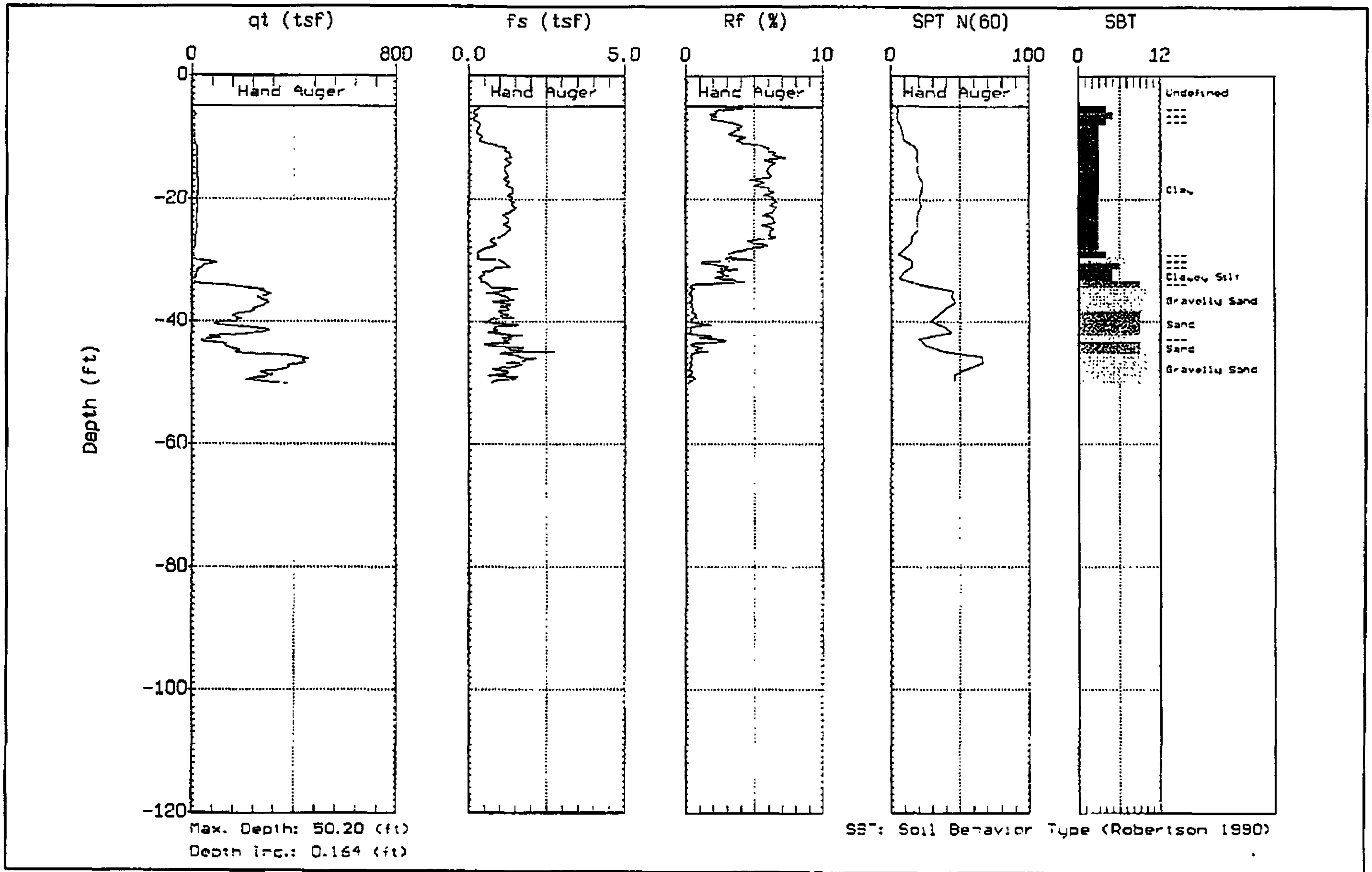
CADENCE CAMPUS

San Jose, California

TRC Lowney

CPT-7

881-9



11/06'EB

CONE PENETRATION TEST - CPT-8

CADENCE CAMPUS
San Jose, California

TRC Lowney

CPT-8
881-9

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW		Well graded gravels, gravel-sand mixtures, little or no fines
			GP		Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM		Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC		Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW		Well graded sands, gravelly sands, little or no fines
			SP		Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM		Silty sands, sand-silt-mixtures, non-plastic fines
			SC		Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50 %		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL		Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50 %		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH		Inorganic clays of high plasticity, fat clays
			OH		Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT		Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE	200		40	10	4	3/4"	3"	12"
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		
mm	0.08	0.4	2	5	19	76mm	

CLEAR SQUARE SIEVE OPENINGS

GRAIN SIZES

TERZAGHI SPLIT SPOON STANDARD PENETRATION — MODIFIED CALIFORNIA — ROCK CORE — PITCHER TUBE — NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

TRC Lowney

FIGURE A-1

EXPLORATORY BORING: EB-1

DRILL RIG: MOBILE B-61	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-9-06 FINISH DATE: 10-9-06	COMPLETION DEPTH: 80.0 FT.

SURFACE ELEVATION:

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
0	3½ inches asphalt concrete over 6 inches aggregate					
	SILT WITH SAND (ML) hard, moist, brown, fine sand, low plasticity	ML	11	11	91	
			12	8	93	50
5	**SANDY SILT (ML)** very stiff, moist, brown, fine sand, low plasticity	ML	7	10		
	LEAN CLAY (CL) very stiff, moist, brown, some fine sand, low to moderate pasticity	CL	21	18	100	
10						
15			29	24	94	
20			26	24	102	
			500psi			
25			24	27	95	
30	medium stiff		12	26	96	

Continued Next Page

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

LA_CORP.GDT 11/14/06 MV* FLL

EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: MOBILE B-61	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-9-06 FINISH DATE: 10-9-06	COMPLETION DEPTH: 80.0 FT.

This log is a part of a report by TRC Lowney, 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.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30	**LEAN CLAY (CL)** very stiff, moist, brown, some fine sand, low to moderate pasticity	CL	125psi			
		POORLY GRADED SAND WITH SILT (SP-SM) dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM	56			
	35						
		medium dense		35			
	40						
				44			
	45						
				46			
	50						
				36			
	55						
				45			
	60						

Continued Next Page

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

LA_CORP.GDT 11/14/06 MV' FLL

EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: MOBILE B-61	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-9-06 FINISH DATE: 10-9-06	COMPLETION DEPTH: 80.0 FT.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	60		**POORLY GRADED SAND WITH SILT (SP-SM)** very dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel							
					54	X				
	65									
					50/6"	X				
	70			SP-SM						
					54	X				
	75									
					67	X				
	80		Bottom of Boring at 80 feet							
	85									
	90									

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

LA CORP.GDT 11/14/06 MV· FLL

TRC Lowney

EXPLORATORY BORING: EB-2

DRILL RIG: MOBILE B-53	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-10-06 FINISH DATE: 10-10-06	COMPLETION DEPTH: 51.5 FT.

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Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION:					
	0	2½ inches asphalt concrete over 6½ inches aggregate					
		LEAN CLAY (CL) hard, moist, dark gray, some fine sand, high plasticity Plasticity Index = 10, Liquid Limit = 27	CL	41	14	114	
		SILTY CLAY WITH SAND (CL-ML) stiff, moist, brown, fine sand, low plasticity	CL-ML	15	21	103	
	5	**SILTY SAND (SM)** loose, wet, brown, fine sand	SM	7	26	96	
		SILTY CLAY WITH SAND (CL-ML) hard, dry, brown, fine sand, low plasticity	CL-ML				
		LEAN CLAY (CL) stiff to very stiff, moist, brown, some fine sand, low to moderate pasticity		22	25	99	
	10						
	15			30	23	101	
			CL				
	20			100psi			
				27	29	89	
	25						
	30			100psi			

Continued Next Page

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 30.0 FEET

LA CORP.GDT 11/14/06 MV FLL

TRC Lowney

EXPLORATORY BORING: EB-2 Cont'd

DRILL RIG: MOBILE B-53	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-10-06 FINISH DATE: 10-10-06	COMPLETION DEPTH: 51.5 FT.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30		CL				
		POORLY GRADED SAND WITH SILT (SP-SM) very dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM				
	~34			25	13		6
	~39	very dense		50/4"			
	~44	dense		44			
	~47	**SILTY SAND (SM)** medium dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SM	19	18		14
	~50.5			18			
	51.5	Bottom of Boring at 51½ feet					

GROUND WATER OBSERVATIONS:

∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 30.0 FEET

TRC Lowney

EXPLORATORY BORING: EB-3

DRILL RIG: MOBILE B-61	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-9-06 FINISH DATE: 10-9-06	COMPLETION DEPTH: 80.0 FT.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION:					
	0	3 inches asphalt concrete over 7 inches aggregate					
		SILTY CLAY WITH SAND (CL-ML) hard, moist, brown, fine sand, low plasticity; Plasticity Index = 5, Liquid Limit = 23	CL-ML	18	7	104	
		SANDY SILT (ML) stiff, moist, brown, fine sand, low plasticity	ML	5	12	80	
	5	**SILTY CLAY (CL-ML)** hard to very stiff, moist, brown, some fine sand, low pasticity	CL-ML	13	13	94	
				12	17	91	
	10						
		LEAN CLAY (CL) stiff to very stiff, moist, brown, some fine sand, moderate pasticity	CL				
				26	25	100	
	15						
				31	23	99	
	20						
				100psi			
	25						
		SILTY SAND (SM) medium dense, wet, gray and brown mottled, fine sand	SM	30			
	30						

Continued Next Page

GROUND WATER OBSERVATIONS:

∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 28.0 FEET

LA CORP.GDT 11/14/06 MV' FLL

TRC Lowney

EXPLORATORY BORING: EB-3 Cont'd

DRILL RIG: MOBILE B-61	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-9-06 FINISH DATE: 10-9-06	COMPLETION DEPTH: 80.0 FT.

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Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30	**SILTY SAND (SM)** medium dense, wet, gray and brown mottled, fine sand	SM				
	~34			11	24		31
	~37	**POORLY GRADED SAND WITH SILT (SP-SM)** medium dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM				
	~39			30			
	~44			23	21		23
	44.5	**CLAYEY SAND (SC)** medium dense, wet, gray, fine sand	SC				
	~47.5	**POORLY GRADED SAND WITH SILT (SP-SM)** dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM				
	~49			34			
	~54	very dense		58	12		12
	~59			36			

Continued Next Page

GROUND WATER OBSERVATIONS:
∇: FREE GROUND WATER MEASURED DURING DRILLING AT 28.0 FEET

LA CORP.GDT 11/14/06 MV FLL

TRC Lowney

EXPLORATORY BORING: EB-3 Cont'd

DRILL RIG: MOBILE B-61	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-9-06 FINISH DATE: 10-9-06	COMPLETION DEPTH: 80.0 FT.

This log is a part of a report by TRC Lowney, 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.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	60	**POORLY GRADED SAND WITH SILT (SP-SM)** dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM				
		SILTY SAND (SM) medium dense, wet, gray, fine sand	SM	27	28		47
	65						
		POORLY GRADED SAND WITH SILT (SP-SM) very dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM	66			
	70						
		CLAYEY SAND (SC) very dense, moist, gray, fine to coarse sand	SC	63			
	75	**POORLY GRADED SAND WITH SILT (SP-SM)** very dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM	67			
	80	Bottom of Boring at 80 feet					
	85						
	90						

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 28.0 FEET

LA CORP.GDT 11/14/06 MV* FLL

TRC Lowney

EXPLORATORY BORING: EB-4

DRILL RIG: MOBILE B-53	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-10-06 FINISH DATE: 10-10-06	COMPLETION DEPTH: 49.5 FT.

This log is a part of a report by TRC Lowney, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	3 inches asphalt concrete over 4 inches aggregate					
		SILTY CLAY WITH SAND (CL-ML) stiff, moist, brown, fine sand, low plasticity	CL-ML	9	21	98	
				7	23	95	
	5			14	22	101	
				10	22	98	
	10						
		LEAN CLAY (CL) very stiff, moist, brown, some fine sand, moderate pasticity	CL				
	15			100psi			
				21	22	108	
	20						
				56psi			
	25	**SANDY LEAN CLAY (CL)** stiff, moist, brown, fine sand, low plasticity	CL				
		SILTY SAND (SM) medium dense, wet, brown, fine sand	SM	12	24		32
	30						

Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 11/14/06 MV* FLL

TRC Lowney

EXPLORATORY BORING: EB-4 Cont'd

DRILL RIG: MOBILE B-53	PROJECT NO: 881-9
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: CADENCE CAMPUS
LOGGED BY: ELS	LOCATION: SAN JOSE, CA
START DATE: 10-10-06 FINISH DATE: 10-10-06	COMPLETION DEPTH: 49.5 FT.

This log is a part of a report by TRC Lowney, 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.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression; 1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30		**SILTY SAND (SM)** medium dense, wet, brown, fine sand	SM					
			dense, decreasing silt		40	X			
	35								
			POORLY GRADED SAND WITH SILT (SP-SM) very dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel	SP-SM	59	X			
	40								
					78	X			
	45								
					50/6"	X			
	50		Bottom of Boring at 49½ feet						
	55								
	60								

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 11/14/06 MV· FLL

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APPENDIX B
LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 31 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 24 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.

Plasticity Index: Plasticity Index determinations (ASTM D4318) were performed on 2 samples of the subsurface soils to measure the range of water contents over which these materials exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on Figure B-1 and on the logs of the borings at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 8 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

R-Value: An R-value resistance test (California Test Method No. 301) was performed on a representative sample of the surface soils at the site to provide data for the pavement design. The test indicated an R-value of 20 at an exudation pressure of 300 pounds per square inch. The results of the test are presented on Table B-2.

Consolidation: Consolidation tests (ASTM D2435) were performed on 3 undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility properties of these soils. Results of the consolidation tests are presented graphically on Figures B-3, 4 & 5.

Triaxial Consolidated Undrained Tests (ASTM D4767) were performed on 5 representative samples of the subsurface soils at the site to assist in evaluating the strength properties of these soils. Results of the Triaxial Consolidated Undrained tests are presented graphically on Figure B-6 and B-7

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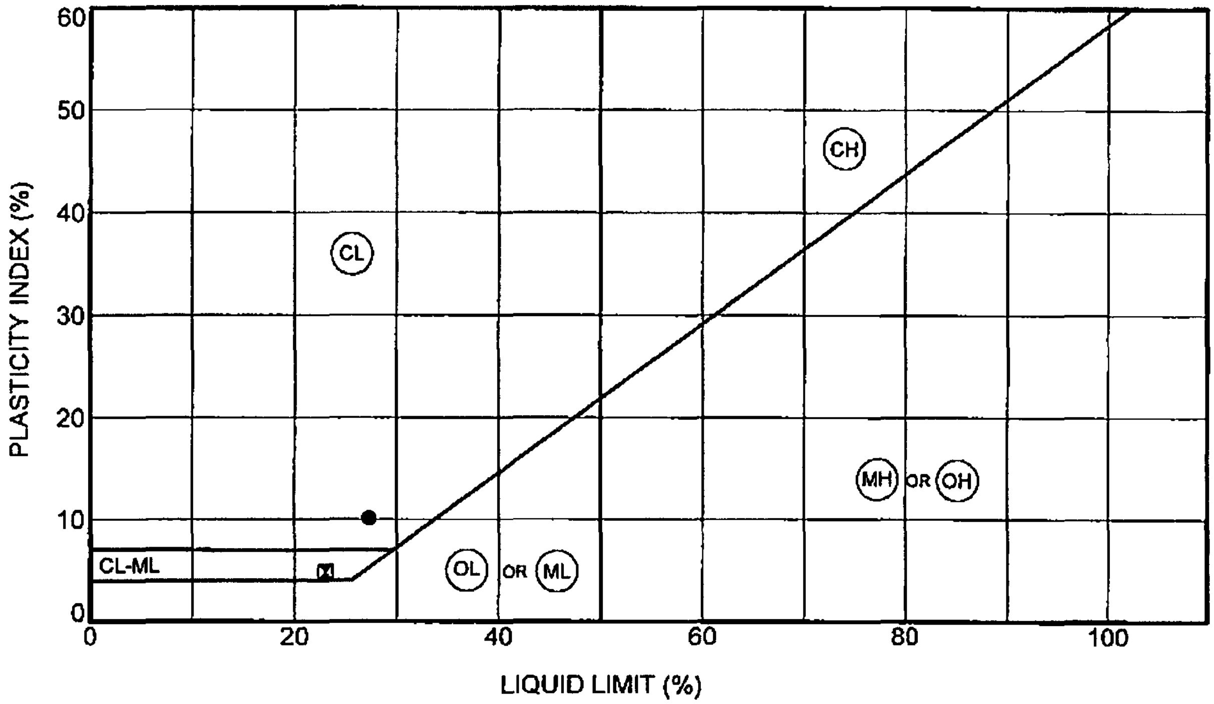


Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	EB-2	2.0	14	27	17	10		LEAN CLAY (CL)
⊠	EB-3	2.0	7	23	18	5		SILTY CLAY WITH SAND (CL-ML)

LA CORP.GDT 11/14/06 MV' FLL

TRC Lowney

PLASTICITY CHART AND DATA

Project: CADENCE CAMPUS

Location: SAN JOSE, CA

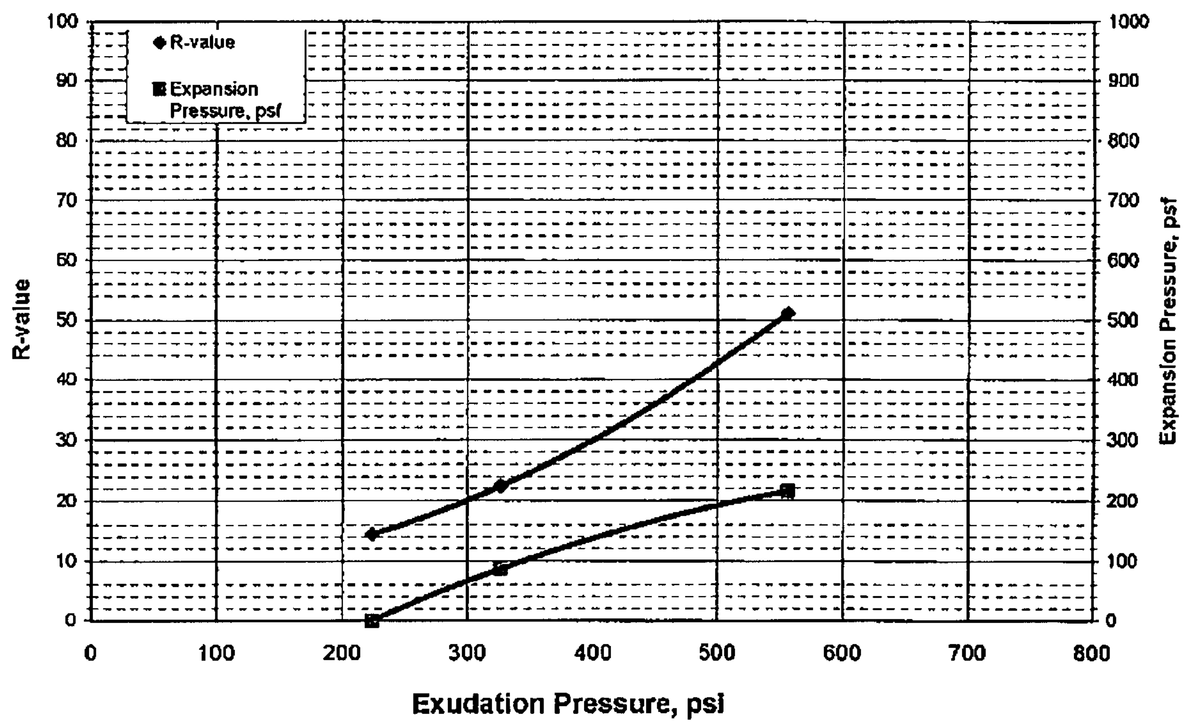
Project No.: 881-9

FIGURE B-1

Job No.: 881-9	Date: 10/24/06	Initial Moisture, 11.7%
Client:	Tested MD	R-value by Stabilometer 20
Project: Cadence Campus - 881-9	Reduced RU	
Sample EB-1	Checked DC	Expansion Pressure 65 psf
Soil Type: Brown Clayey SAND (Silty)		

Specimen Number	A	B	C	D
Exudation Pressure, psi	326	224	556	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	17	39	4	
Weight of Soil & Mold, grams	3125	3095	3134	
Weight of Mold, grams	2089	2104	2081	
Height After Compaction, in.	2.37	2.29	2.31	
Moisture Content, %	13.2	15.3	12.0	
Dry Density, pcf	116.9	113.7	123.2	
Expansion Pressure, psf	86.0	0.0	215.0	
Stabilometer @ 1000				
Stabilometer @ 2000	112	126	60	
Turns Displacement	3.35	3.55	3.25	
R-value	22	14	51	

Remarks:



R-VALUE TEST

CADENCE CAMPUS

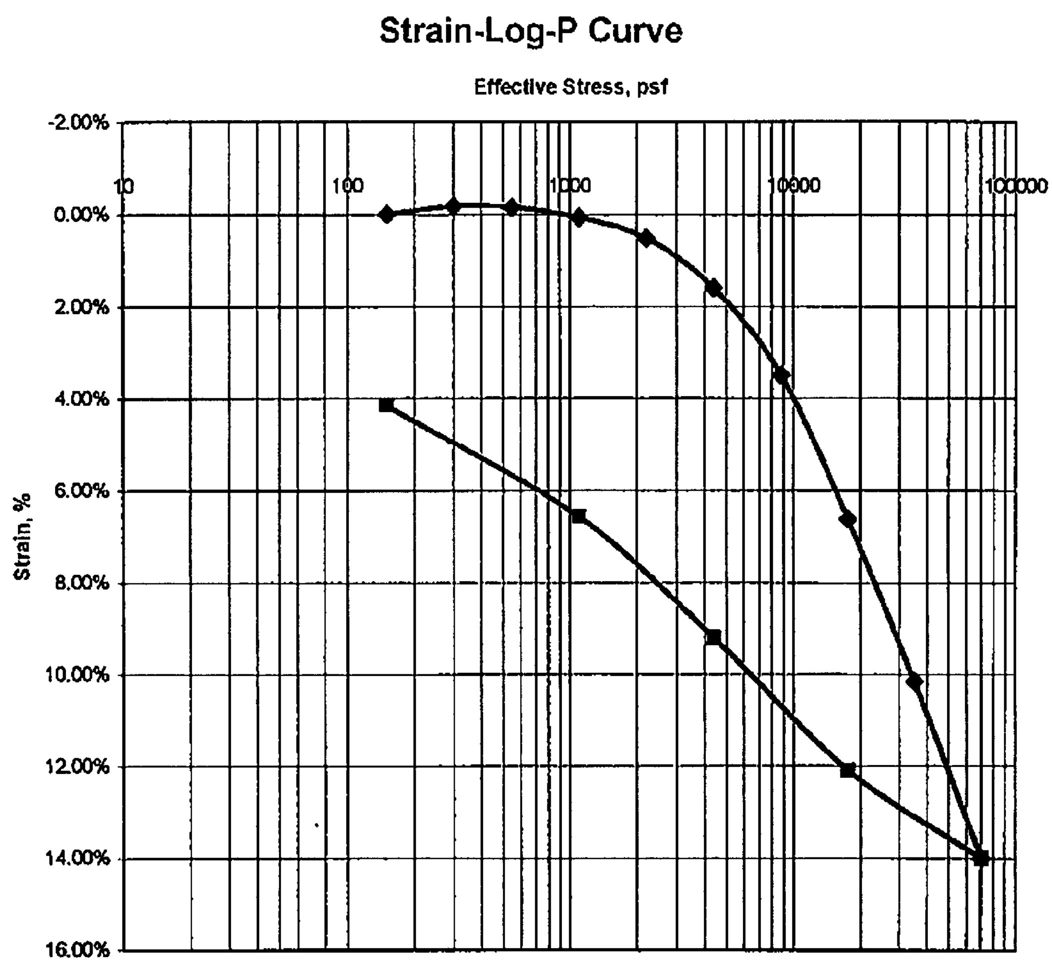
San Jose, California

TRC Lowney

FIGURE B-2

881-9

Job No.:	881-9	Boring:	EB-2	Run By:	MD
Client:		Sample:	6A	Reduced:	PJ
Project:	Cadence Campus - 881-9	Depth, ft.:	18.5	Checked:	PJ/DC
Soil Type:	Brown CLAY			Date:	10/31/2006



Ass. Gs = 2.7	Initial	Final
Moisture %:	25.3	24.8
Dry Density, pcf:	99.8	101.0
Void Ratio:	0.689	0.669
% Saturation:	99.2	100

Remarks:

CONSOLIDATION TEST

CADENCE CAMPUS

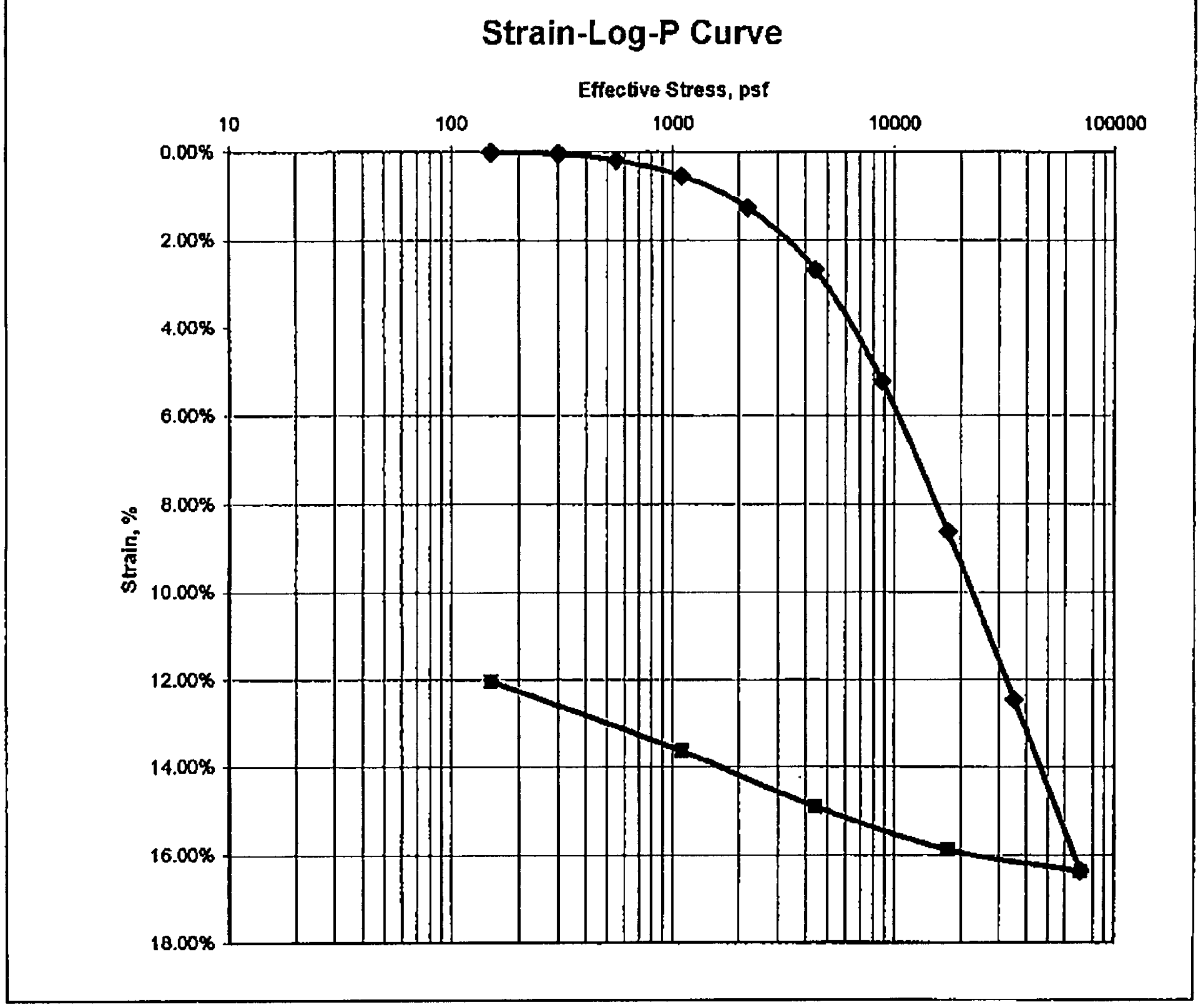
San Jose, California

TRC Lowney

FIGURE B-3

881-9

Job No.:	881-9	Boring:	EB-3	Run By:	MD
Client:		Sample:	7A	Reduced:	PJ
Project:	Cadence Campus - 881-9	Depth, ft.:	22.5	Checked:	PJ/DC
Soil Type:	Brown & Gray CLAY w/ Sand (Silty), trace nodules & Root holes			Date:	10/31/2006



Ass. Gs = 2.7	Initial	Final
Moisture %:	24.3	18.9
Dry Density, pcf:	99.8	111.7
Void Ratio:	0.689	0.509
% Saturation:	95.1	100

Remarks:

CONSOLIDATION TEST

CADENCE CAMPUS

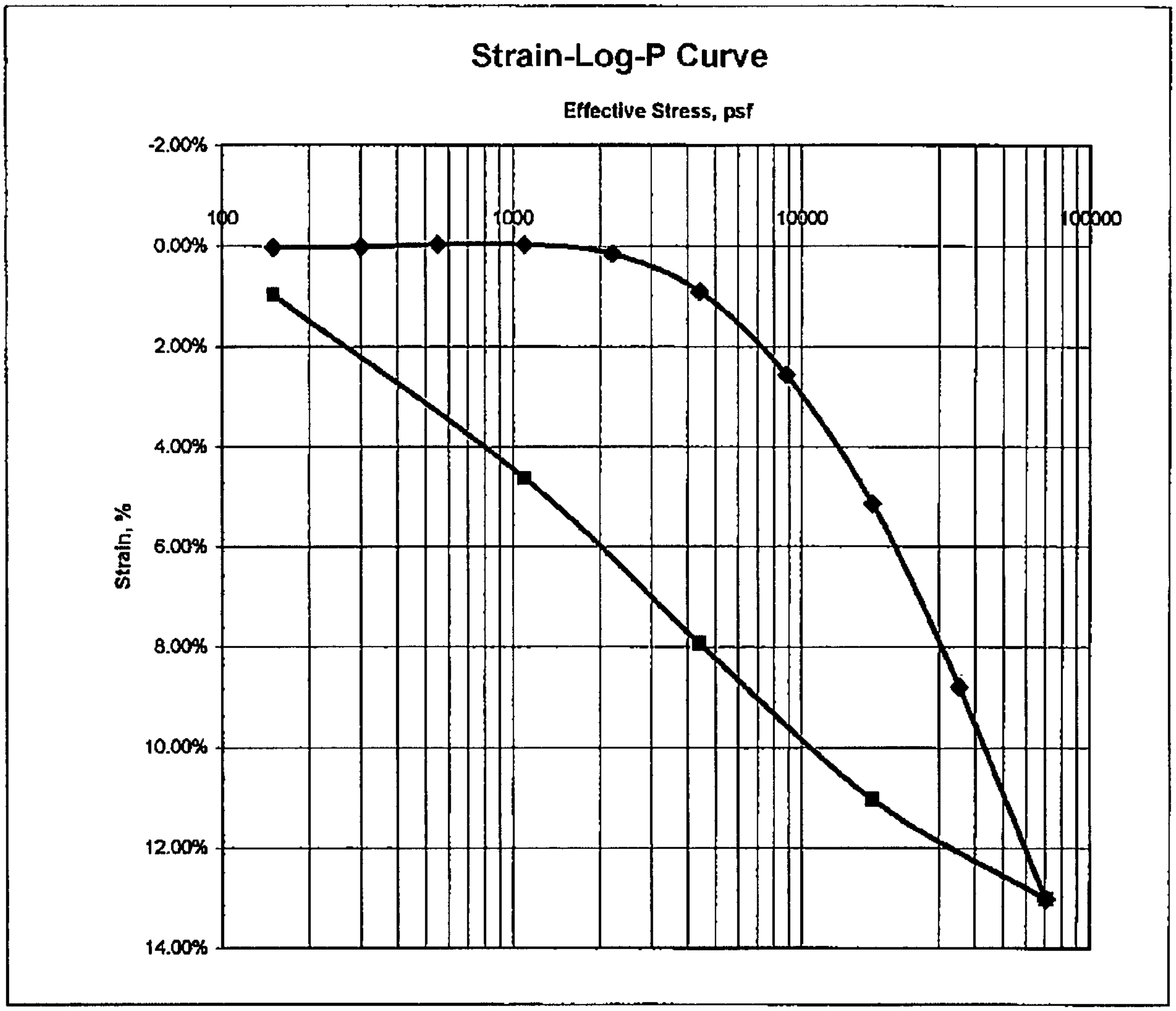
San Jose, California

TRC Lowney

FIGURE B-4

881-9

Job No.:	881-9	Boring:	EB-4	Run By:	MD
Client:		Sample:	5A	Reduced:	PJ
Project:	Cadence Campus - 881-9	Depth, ft.:	14.0	Checked:	PJ/DC
Soil Type:	Brown CLAY			Date:	10/31/2006



Ass. Gs = 2.7	Initial	Final
Moisture %:	24.0	25.0
Dry Density, pcf:	102.1	100.7
Void Ratio:	0.651	0.674
% Saturation:	99.5	100

Remarks:

CONSOLIDATION TEST

CADENCE CAMPUS

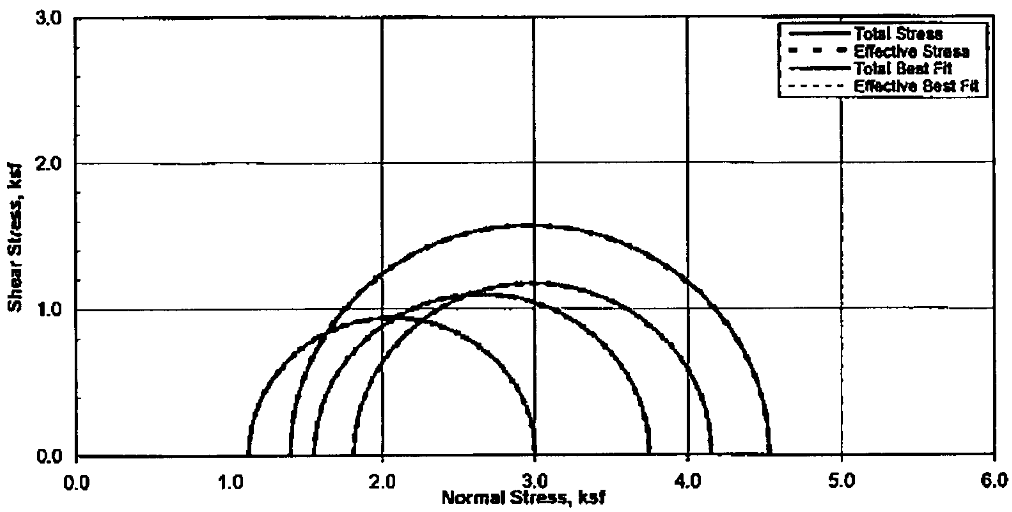
San Jose, California

TRC Lowney

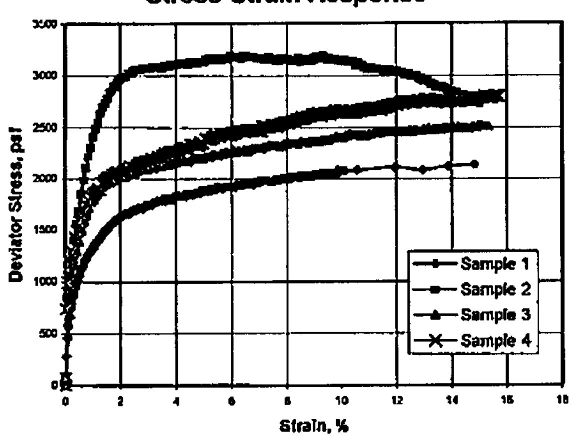
FIGURE B-5

881-9

Triaxial Consolidated Undrained
(ASTM D4767)



Stress-Strain Response



Sample:	1	2	3	4
MC, %	23.8	24.7	24.3	25.4
Dry Dens, pcf	102.6	102.5	99.4	101.1
Sat. %	97.2	100.5	92.0	100.0
Void Ratio	0.673	0.675	0.726	0.697
Diameter In	2.87	2.87	2.87	2.86
Height, in	6.08	6.08	6.00	5.92
	Final			
MC, %	25.3	25.9	25.4	24.1
Dry Dens, pcf	101.2	100.2	101.1	101.4
Sat. %	100.0	100.0	100.0	95.9
Void Ratio	0.696	0.713	0.697	0.692
Diameter, in	2.89	2.91	2.86	2.87
Height, in	6.07	6.04	5.92	6.00
Cell, psi	36.3	48.3	49.4	51.4
BP, psi	28.5	38.6	38.6	38.8
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	5.0
Deviator ksf	1.880	3.137	2.195	2.342
Excess PP				
Sigma 1	3.003	4.533	3.750	4.157
Sigma 3	1.123	1.397	1.555	1.814
P, ksf	2.063	2.965	2.653	2.986
Q, ksf	0.940	1.568	1.098	1.171
Stress Ratio	2.674	3.246	2.411	2.291
Rate in/min	0.025	0.025	0.025	0.025
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

Job No.:	881-9	Date:	11/1/2006
Client:			BY:MD/DC
Project:			
Sample 1)	B4-5A @ 14'	Brown CLAY	
Sample 2)	B2-6A @ 18.5'	Brown CLAY	
Sample 3)	B3-7A @ 22.5'	Brown CLAY	
Sample 4)	B1-10A @ 30'	Brown Silty SAND / Sandy SILT	

REMARKS: Strengths picked at 5% strain.

TRIAXIAL CONSOLIDATED UNDRAINED

CADENCE CAMPUS

San Jose, California

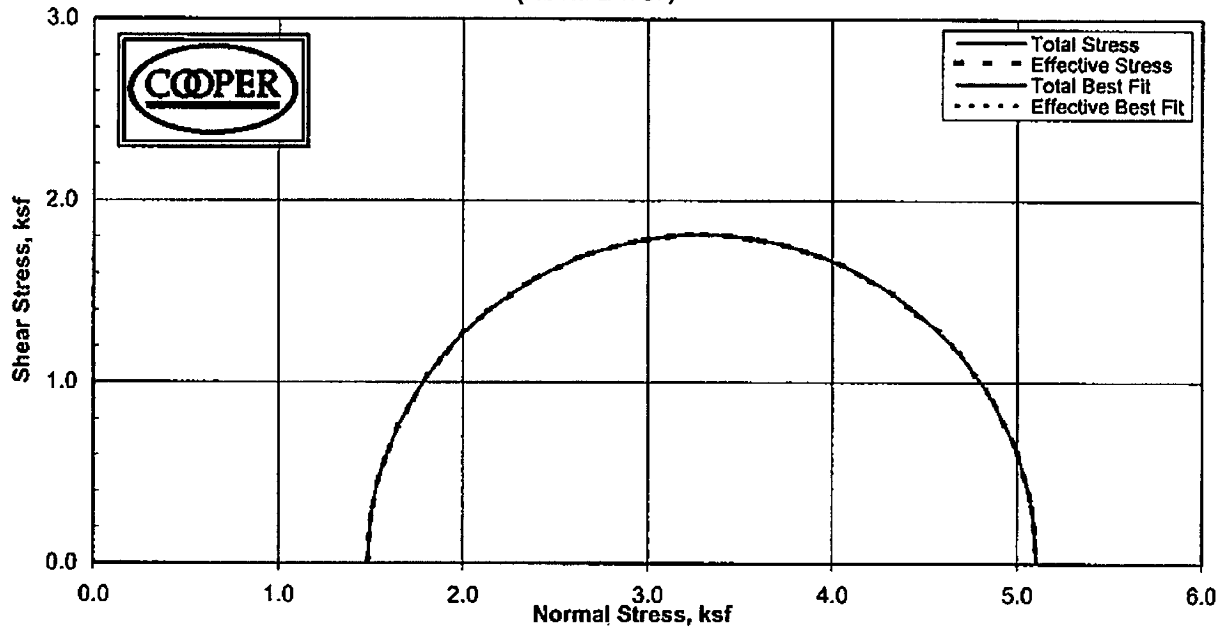
TRC Lowney

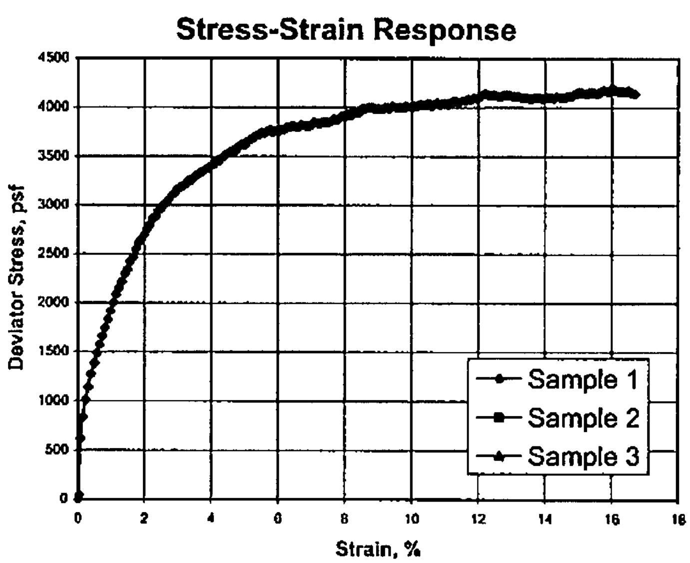
FIGURE B-6

881-9

Triaxial Consolidated Undrained

(ASTM D4767)





Sample:	1	2	3	4
MC, %	24.6			
Dry Dens, pcf	100.4			
Sat. %	98.2			
Void Ratio	0.678			
Diameter in	2.87			
Height, in	6.00			
	Final			
MC, %	25.7			
Dry Dens, pcf	99.5			
Sat. %	100.0			
Void Ratio	0.694			
Diameter, in	2.89			
Height, in	5.99			
Cell, psi	48.7			
BP, psi	38.4			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	3.623			
Excess PP	0.000			
Sigma 1	5.106			
Sigma 3	1.483			
P, ksf	3.294			
Q, ksf	1.811			
Stress Ratio	3.442			
Rate in/min	0.002			
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

7:07

B-7

Job No.:	028-1913	Date:	11/6/2006
Client:	TRC Lowney		BY:MD/DC
Project:	881-9		
Sample 1)	EB1-7A @ 20'	Brown Mottled Gray CLAY w/Sand	
Sample 2)			
Sample 3)			
Sample 4)			

REMARKS: Strengths picked at 5% strain.