

**Appendix F**  
**Geotechnical Feasibility Evaluation**

Type of Services	Geotechnical Feasibility Evaluation
Project Name	Areas 3 & 4 EIR Study
Location	Stevenson Boulevard & Mowry Avenue Newark, California
Client	David J. Powers & Associates
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Project Number	118-3-3
Date	February 4, 2009

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<b>Type of Services</b>	<b>Geotechnical Feasibility Evaluation</b>
<b>Project Name</b>	<b>Areas 3 &amp; 4 EIR Study</b>
<b>Location</b>	<b>Stevenson Boulevard &amp; Mowry Avenue Newark, California</b>

## **SECTION 1: INTRODUCTION**

This geotechnical feasibility evaluation was prepared for the sole use of David J. Powers & Associates for preparation of the Environmental Impact Report for Study Areas 3 and 4 located in Newark, California. The general site location is shown on the Vicinity Map, Figure 1. The purpose of this study was review available geotechnical and geologic data, perform limited field exploration to evaluate the existing subsurface conditions, and determine potential geotechnical and geologic concerns that could impact future development within the study areas.

For our use, we were provided with the following documents:

- A report titled, "Phase 1 Environmental Site Assessment, 101-acre Heath Property, Mowry Avenue, Newark, CA," prepared by PES Environmental, Inc. dated September 11, 2006.
- A report titled, "Phase 1 Environmental Site Assessment, 115-acre Rogers Property, Stevenson Boulevard, Newark, CA," prepared by PES Environmental, Inc. dated September 11, 2006.
- A report titled, "Phase 1 Environmental Site Assessment, 80-acre Sobrato Site, Cherry Street and Stevenson Boulevard, Newark, CA," prepared by PES Environmental, Inc. dated August 11, 2006.
- An electronic copy of a site topographic plan of Areas 3 and 4 prepared by Kier & Wright, undated.
- A letter titled "Comments in Response to the Notice of Preparation of an Environmental Impact Report and Notice of Public Scoping Meeting for Newark Area 3 & 4 Specific Plan," prepared by Friends of Coyote Hills dated June 11, 2007.
- Electronic copies of the Conceptual Land Use Plan for Areas 3 and 4.

- A plan titled, "Newark Area 4", prepared by H.T. Harvey & Associates, dated January 2008, indicating areas of uplands and wetlands, and the relative quality of each.

## 1.1 PROJECT DESCRIPTION

The approximately 900-acre site depicted on the Site Plan and Topographic Map, Figure 2, is generally bounded by Mowry Avenue to the west and Stevenson Boulevard to the east. A Union Pacific (UP) railroad right-of-way roughly bisects the study area, and divides the sites currently designated as Areas 3 and 4. A brief discussion of the two study areas is presented below.

### Area 3

Area 3 is bounded by Mowry Avenue to the west, Cherry Street to the north, Stevenson Boulevard to the east, and the UP railroad tracks to the south. The area is developed with several existing commercial buildings, a fire station, the Ohlone College campus expansion, the George M. Sillman recreational complex, and two undeveloped parcels. Approximately 77 acres of undeveloped land within Area 3 are being considered for residential use and an elementary school. Portions of the site will need to be raised such that finished floors are located at Elevation 11.75 feet, or greater depending on local requirements. Preliminary import fill estimates are anticipated to be on the order of 56,000 cubic yards. The location of potential development within Area 3 is shown on the Conceptual Land Use Plan, Figure 2A.

### Area 4

Area 4 is bounded by the UP railroad tracks to the north and east, Mowry Avenue to the west, and Mowry Slough and the Newark/Fremont city limits to the south. Area 4 is to be developed for two potential uses, including an 18-hole championship golf course, likely in the northwestern areas, and residential development, likely in the central/southwestern area. For the residential development, site grades will need to be raised by up to approximately 12 feet such that finished floors are located at Elevation 11.75 feet, or greater depending on local requirements. Preliminary import fill estimates are anticipated to be on the order of 1.1 to 1.6 million cubic yards. We understand that appurtenant streets, a Stevenson Boulevard overpass crossing the UP tracks, parking, utilities, landscaping and other improvements necessary for site development are also planned.

Most of Area 4 is covered with low grasses and sparse bushes. Several drainage channels, including two Alameda County Flood Control District channels, bisect or border portions of Area 4. Based on project wetlands mapping by H.T. Harvey & Associates, significant portions of Area 4 are considered upland areas; however, not all areas are contiguous, forming areas of intermixed wetland-upland areas.

For the purpose of this report, which is to evaluate the feasibility of residential development, we have focused our efforts on the areas of proposed residential development; the 77-acre parcel in Area 3 and the core upland area of the approximately 150-acre residential development

portion of Area 4. The location of potential development within Area 4 is shown on the Conceptual Land Use Plan, Figure 2A.

## **1.2 SCOPE OF SERVICES**

Our scope of services was presented in our proposal dated April 6, 2007, and consisted of limited field and laboratory programs to evaluate physical and engineering properties of the subsurface soils within a portion of Area 4, review of concurrent geotechnical work for the residential development project, engineering analysis to prepare preliminary estimates of long-term settlement, determining the primary geotechnical and geologic impacts that could affect future development, and preparation of this feasibility-level report. Brief descriptions of our exploration and laboratory programs are presented below.

## **1.3 EXPLORATION PROGRAM**

Field exploration consisted of performing exploratory borings using conventional truck-mounted, hollow-stem auger drilling equipment. Four borings (EB-1 through EB-4) were drilled on July 23 and 24, 2007, to depths ranging from 40 to 46½ feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. The approximate boring locations are shown on the Site Plan and Topographic Map, Figure 2. Details regarding our exploration program are included in Appendix A.

To supplement the data obtained from the exploratory borings, we reviewed subsurface data collected from ten Cone Penetration Test (CPT) soundings and two supplemental borings drilled as part of the residential development study. The CPTs were performed using 20-ton truck- and track-mounted equipment. CPT-5 through CPT-7 were performed on the 77-acre parcel in Area 3; CPT-1 through CPT-4 and CPT-8 through CPT-10 were performed within the central portion of Area 4. The CPTs were advanced to depths of approximately 50 feet. The CPTs were also backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. The approximate locations of the CPT's are shown on the Site Plan and Topographic Map, Figure 2.

We also reviewed data from two additional exploratory borings (EB-5 and EB-6) that were drilled within Area 4 on April 25, 2008, to depths ranging from approximately 35 to 44 feet. The approximate locations of the supplemental borings are also shown on the Site Plan and Topographic Map, Figure 2. Logs from the residential development study are included in Appendix C.

## **1.4 LABORATORY TESTING PROGRAM**

In addition to visual classification of samples, the laboratory program focused on obtaining data for preliminary settlement analysis and seismic ground deformation estimates. Testing included moisture contents and dry densities, grain size analyses, and one consolidation test. Details regarding our laboratory program are included in Appendix B.

In addition, we reviewed laboratory data collected from the two supplemental borings drilled as part of the residential development study. This laboratory program focused on obtaining additional data for preliminary settlement analysis and seismic ground deformation estimates. Moisture contents and dry densities, grain size analyses, Atterberg Limit, and consolidation tests were performed for this investigation.

## **1.5 ENVIRONMENTAL SERVICES**

As requested, Cornerstone Earth Group also performed a screening level hazardous materials review for Areas 3 and 4, the results of which were presented in separate reports.

## **SECTION 2: REGIONAL SETTING**

### **2.1 GEOLOGIC SETTING**

San Francisco Bay is a northwesterly trending structural depression that lies along the boundary of the Pacific and North America tectonic plates. The Bay is within the Coast Ranges geomorphic province of California, which is characterized by a series of nearly parallel mountain ranges (Goldman, 1969). Active faults, including the San Andreas, Hayward, and Calaveras Faults, roughly parallel the western and eastern limits of the Bay. The Bay began forming during the Pleistocene Epoch, approximately 2 million years ago, the San Francisco-Marín block began to tilt eastward along the Hayward Fault. The eastern side of the block became a depression and filled with sediment and water.

Bedrock units exposed in the eastern portions of the Bay range from Jurassic-Cretaceous to Quaternary age (approximately 135 million years old to recent). The oldest bedrock units (Jurassic-Cretaceous age) include the Franciscan Formation, which consists of interbedded sandstone and shale, limestone, radiolarian chert, and metavolcanic rocks (Goldman 1969). The Franciscan Formation is west of the Hayward Fault, and is exposed in the hills along the Peninsula. East of the Hayward Fault, a thick sequence of Tertiary age sandstones and shales of the Great Valley Sequence overlies the Franciscan Formation. Along the eastern shoreline of the Bay, layers of Quaternary-age alluvial sediments mantle the Franciscan Formation. Since Cretaceous time, the Bay Area has undergone numerous episodes of faulting and folding. As such, rock units exposed along fault zones are typically sheared and highly weathered.

### **2.2 SITE GEOLOGY**

Areas 3 and 4 are located within a gentle southwest-sloping alluvial plane primarily within the Niles and Newark Quadrangles (CGS, 2003; Helley et al, 1997; Wagner et al, 1991), as shown on Figures 3 through 5. The area is mapped as being underlain by either Holocene or late Pleistocene alluvial fan and/or Bay Mud deposits, most of which have been deposited by the nearby Alameda Creek. Bay Mud deposits are generally mapped as being located in the flat-lying region that borders the San Francisco Bay and associated sloughs. Holocene alluvial fan deposits in the study area have been subdivided into several units, including San Francisco Bay Mud (Qhbm), fine-grained alluvial fan deposits (Qhff), latest Pleistocene to Holocene alluvial fan deposits (Qf), and latest Pleistocene to Holocene alluvial fan levee deposits (Ql). These

deposits generally consist of interbedded clays, silts, sands and gravels of varying thickness and composition (CGS, 2003).

Area 3 is primarily mapped as being underlain by Pleistocene to Holocene alluvial fan deposits (Qf and Ql) (CGS, 2003). Helley et al (1997) mapped Area 3 as being underlain primarily by Holocene basin (Qhb) and floodplain (Qfp) deposits.

Area 4 is mapped as being mostly underlain by Holocene Bay Mud deposits (Qhbm) (CGS, 2003). However, Helley et al (1997) mapped Area 4 as being underlain by Holocene basin (Qhb) and salt-affected basin (Qhbs) deposits on the northern two-thirds of the area, and Holocene Bay Mud beneath the southern one-third of the area, as shown on Figure 4.

Earlier mapping by Wagner et al (1991) indicated similar alluvial deposits in the area; however, an inferred lineament or trace of the Silver Creek Fault is mapped passing just west of Mowry Slough, as shown in Figure 5. Subsequent mapping by Helley et al (1997) and the CGS (2003) did not show the inferred lineament of the Silver Creek Fault.

Historic high ground water is generally mapped as being at depths ranging from 5 to 10 feet below existing ground surface elevations in Area 3, and generally less than 5 feet in Area 4 (CGS, 2003).

### **2.3 REGIONAL SEISMICITY**

The San Francisco Bay area is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities (2003) estimates there is a 62 percent chance of at least one magnitude 6.7 earthquake occurring in the Bay Area region between 2003 and 2032. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The tables below present the State-considered active faults within 25 kilometers of Study Areas 3 and 4. Since Areas 3 and 4 are relatively large and are almost one mile apart from end to end, distances to nearby faults have been approximated from the middle of each study area. A regional fault map is presented as Figure 6, illustrating the relative distances of the site to significant fault zones.

**Table 1: Approximate Fault Distances**

Fault Name	Distance from Area 3		Distance from Area 4	
	(miles)	(kilometers)	(miles)	(kilometers)
Hayward (Total Length)	3.8	6.2	4.8	7.8
Hayward (Southeast Extension)	5.8	9.4	5.2	8.3
Calaveras (North)	7.5	12.1	8.2	13.2
Calaveras (South)	12.1	19.5	11.6	18.6
Monte Vista-Shannon	12.5	20.2	12.1	19.5
San Andreas (Peninsula)	14.9	24.0	14.0	22.5

**SECTION 3: SITE CONDITIONS**

**3.1 SITE HISTORY**

Study Areas 3 and 4 lie just east of former tidal marshes of the San Francisco Bay. Based on historic topographic maps, the railroad tracks that divide the two study areas were constructed prior to 1899. The marshes west of the railroad tracks were subject to tidal influences until Mowry Slough and nearby areas were diked in the early 1900's to create salt ponds west of the slough. Prior to creating the dikes, numerous narrow, shallow, tidally influence channels meandered through Area 4.

**3.1.1 Area 3**

During the early to mid 1900's, Area 3 was used primarily for agricultural purposes or remained fallow. Based on available aerial photographs of the vicinity, several former residential or barn structures were observed on the 77-acre parcel during the 1960's and 1970's. In a 1975 photograph, numerous stockpiles and equipment storage were visible throughout the northeast quadrant of the 77-acre parcel. Interviews conducted as part of the Phase I site assessments (PES, 2006) indicated that the various stockpiles and equipment were possibly associated with horse stables and possible stockpiled soil and construction debris. In a 1982 photograph, the structures and equipment storage on the 77-acre portion of Area 3 were gone. The parcel appears to have remained undeveloped since the 1980's. The previous Phase I report also indicated that the portions of the 77-acre parcel may have used as a dairy in the 1950's.

**3.1.2 Area 4**

The northeast portion of the proposed development area within Area 4 was occupied by a salt storage facility and associated buildings in the 1930's and 1940's. By the late 1950's, salt storage operations appear to have stopped and portions of Area 4 were under cultivation (likely hay production). The small unnamed slough that extends northeast from Mowry Slough into Area 4 appears to have remained unchanged to the present time. In general, the development

area within the eastern portion of Area 4 has remained relatively unchanged since the 1960's, except for annual surface soil tilling.

Ace Auto Wreckers and Pick-N-Pull are two commercial properties located between the ACWD channel and Mowry Avenue. Since at least the late 1930s, the general site vicinity appears to have been mainly agricultural land with widely spaced residences. By 1958, a commercial building was constructed to the north of the Site (north of the rail road tracks) and salt evaporation ponds were developed on property to the west of Mowry Avenue. Automobile wrecking yards appear to have occupied the adjacent property to the northeast (the current Pick-N-Pull property) since the late 1960s, as well as the adjacent property to the southwest (the 10-acre Newark Partners parcel, located between Ace Auto Wreckers and Mowry Slough) since the early 1980s. By the 1990s, an increase in commercial and residential development in the general vicinity to the north of the Area 4 is apparent.

The 10-acre Newark Partners parcel (formerly Tolbertson property) was primarily vacant in 2006 but has historically been used since approximately 1980 for vehicle dismantling and storage activities. Fill reportedly was placed on the parcel between the late 1950s and early 1960s. Debris or soil mixed with debris was encountered on approximately 7 acres of the parcel, ranging in thickness from approximately 2 to 10 feet. It was reported that the landfilling/debris disposal may have been conducted by a former entity referred to as the East Bay Disposal Company (EBDC).

## **3.2 SURFACE DESCRIPTION**

### **3.2.1 Area 3**

Area 3 is bounded by Mowry Avenue to the northwest, Cherry Street to the north, Stevenson Boulevard to the east, and the UP railroad tracks to the southwest. The area is developed with several existing commercial buildings, a fire station, the Ohlone College campus expansion, the George M. Silliman recreational complex, and undeveloped parcels.

The 77-acre Sobrato parcel being considered for residential development is roughly located within the northwest quadrant of Area 3 and is bounded by Cherry Avenue to the north, Stevenson Boulevard to the east, an Alameda County Flood Control channel to the west, and existing commercial developments to the south. The Sobrato parcel is undeveloped, except for a 10- to 15-foot-wide irrigated landscaping easement along Cherry Avenue and Stevenson Boulevard. The remainder of the parcel is undeveloped and had recently been tilled; only minor sparse vegetation was observed in the tilled areas. A concrete-lined drainage ditch was observed along the southern property boundary. An existing sound wall also borders the commercial development to the south.

Available topographic information indicates that site grades generally range from approximately Elevation 15 to 18 feet (datum unknown). The site is relatively flat and appears to slope gently towards the southwest.

### **3.2.2 Area 4**

The irregularly-shaped parcels located within the approximately 550-acre Area 4 are roughly bounded by the Union Pacific (UP) railroad tracks to the northeast, the City of Fremont/Newark limits to the southeast, Mowry Slough and levee system to the southwest, and Mowry Avenue and a used-car part lot to the northwest. The primary access to Area 4 is at the southern end of Stevenson Boulevard. The parcels are generally undeveloped, with the exception of the auto dismantlers sites located in the northwestern portion of Area 4, and a single-family residence located on the south side of the UP railroad tracks where Stevenson Boulevard ends. Several buildings and automobile storage cover the auto dismantler's sites. In addition, two barns are located approximately two to three hundred yards to the west of the single-family; one additional barn is located approximately two hundred yards to the southeast of the existing house. Several unpaved access roads bisect or border the parcels within Area 4. The majority of Area 4 is used for dry farming (hay cultivation); the soil is reportedly tilled annually and planted prior to mowing and baling. The site had recently been mowed and baled at the time of our site investigation, and tilling was just beginning.

Available topographic information indicates that site grades generally range from approximately Elevation 0 to 10 feet (datum unknown). The site is relatively flat and appears to slope gently towards the southwest.

Mowry slough borders the southwest side of Area 4. The slough is flanked by man-made levees. The levees appear to be roughly 5 to 8 feet high, with side slopes ranging from 1:1 to 2:1 (horizontal:vertical). Alameda County Water District flood control channels also bisect the northwest and border the southeast portions of Area 4. A shallow remnant slough extends onto the southern portion of the site that is tidally influenced.

## **3.3 SUBSURFACE CONDITIONS**

### **3.3.1 Area 3**

Based on our review of available geologic maps, Area 3 is generally underlain by native alluvial fan deposits consisting of interbedded clay, silt and sand with varying amounts of gravel. Artificial fills are likely present within currently developed parcels as part of previous site grading. The near-surface clayey soils within Area 3 will likely exhibit moderate plasticity and shrink/swell potential when subject to wetting and drying cycles. In addition, near-surface clay soils are anticipated to be poorly drained (USDA, 1980).

Based on our review of the subsurface data collected for the residential development study, this 77-acre parcel within Area 3 is generally underlain by native alluvial fan deposits consisting of interbedded clay, silt and sand with varying amounts of gravel. Cone Penetration Tests CPT-5, CPT-6, and CPT-7 performed in this area encountered medium stiff to stiff silty/sandy clay and sandy/clayey silt to depths ranging from 24 to 34 feet. The upper clay and silt is underlain by interbedded loose to dense sand, silty sand, and clayey sand to the maximum depth explored at 50 feet. CPT logs are presented in Appendix C.

We also reviewed the results of one Plasticity Index (PI) test that was performed on a representative surface sample collected within the 77-acre parcel of Area 3. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests indicated a PI of 19, indicating moderate expansion potential to wetting and drying cycles.

### 3.3.2 Area 4

To begin evaluating the subsurface conditions within the proposed residential development portion of Area 4, we reviewed available geologic maps and subsurface data collected as part of the residential development study, and performed limited subsurface exploration for this evaluation.

Area 4 is generally underlain by native alluvial fan deposits consisting of interbedded silty clay, clayey and sandy silt, and localized sand layers. The upper 3 to 4 feet of the upper clay is desiccated due to previous drying; therefore, is generally medium stiff to stiff and is considered relatively incompressible. Below the desiccated zone, the borings and Cone Penetration Tests performed in Area 4 encountered soft to medium stiff, moderately compressible silty clay to depths on the order of 18 to 22 feet. The near-surface clayey soils within Area 4 will likely exhibit moderate to high plasticity and shrink/swell potential when subject to wetting and drying cycles. In addition, near-surface clay soils are anticipated to be poorly drained (USDA, 1980).

The upper silty clay is generally underlain by interbedded medium stiff to stiff silty clay and clayey silt to the maximum depth explored at 46½ feet, except in Boring EB-1, where interbedded loose to dense silty sand and sandy gravel was encountered between a depth of approximately 20 to 42 feet. Exploratory boring logs for EB-1 through EB-4 are presented in Appendix A.

As discussed, we reviewed the subsurface data collected by Cornerstone Earth Group from seven CPTs (CPT-1 through CPT-4 and CPT-8 through CPT-10) and two exploratory borings (EB-5 and EB-6) drilled as part of the residential development study for Area 4. The borings were drilled in Area 4 to determine the potential extent of soft, compressible Bay Mud and the characteristics of potentially liquefiable soils within the area likely to include residential development. Based on our review of these borings and laboratory data, the subsurface conditions were relatively consistent with those encountered in the CPT's; however, the upper alluvial soil was found to be over-consolidated and only moderately compressible when compared to typical young Bay Mud deposits. The moisture content of the upper alluvial clay ranged from approximately 20 to 40 percent at depths ranging from 4 to 25 feet. The dry density of these clays generally ranged from 88 to 115 pounds per cubic foot. Further discussion of the compressibility of the upper silty clay layer is presented later in this report. It should be noted that localized deposits of highly compressible Bay Mud may be present on the western portion of Area 4 and gradually increase in thickness towards the west-southwest. Exploratory boring and CPT logs reviewed from the residential development study are presented in Appendix C.

### **3.4 GROUND WATER**

Based on available published data, seasonal and/or historical high groundwater on the order of 5 to 10 feet below the ground surface could be expected for the Area 3 vicinity. In Area 4, historic high ground water is anticipated to be less than 5 feet below existing site grades (CGS, 2003). Free ground water was initially encountered in the initial Area 4 borings (EB-1 through EB-4) at the time of drilling at depths ranging from 15 to 20 feet. Stabilized ground water was measured in Boring EB-3 at a depth of approximately 4 feet below existing grade.

We also reviewed ground water data from explorations for the residential development portions of Areas 3 and 4. Pore pressure dissipation tests were performed at depths of 23 and 40 feet during CPT-3 and CPT- 7, respectively. Based on the maximum recorded pore pressure recorded during each test, the estimated depth to ground water was approximately 6 feet in CPT-7 (Area 3) and approximately 0 feet in CPT-3 (Area 4), which could indicate that the shallow ground water is under slight pressure in Area 4. Ground water was measured at the end of drilling at a depth of approximately 10 and 11½ feet in Borings EB-5 and EB-6, respectively. It should be noted that the measured ground water levels may not represent stabilized conditions.

Fluctuations in ground water levels occur due to many factors including the following: seasonally water levels can generally fluctuate 2 to 5 feet, underground drainage patterns can cause perched water conditions above steady ground water levels, regionally the average ground water level can fluctuate over long time periods, and other factors.

## **SECTION 4: GEOLOGIC HAZARDS**

### **4.1 FAULT RUPTURE**

As discussed above, several significant faults are located within 25 kilometers of Areas 3 and 4. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 6, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

### **4.2 ESTIMATED GROUND SHAKING**

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. The magnitude-weighted pseudo-peak acceleration for the site with a 10 percent chance of exceedance in 50 years is approximately 0.54g (CGS, Newark Quadrangle, 2003, Niles Quadrangle, 2004). Pseudo-peak ground accelerations have been normalized to a 7.5Mw seismic event, including weighting to account for regional seismic activity and fault distances.

### 4.3 LIQUEFACTION POTENTIAL

Areas 3 and 4 are located within a State-designated Liquefaction Hazard Zone (CGS, Newark Quadrangle, 2003, Niles Quadrangle, 2004). This potential impact was addressed on a preliminary basis by evaluating data from our field exploration and reviewing subsurface data obtained from the residential development investigation performed within portions of Areas 3 and 4. Further study will be required to further characterize the lateral extent, depth, and properties of the potentially liquefiable soils within Areas 3 and 4.

#### 4.3.1 Background

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

#### 4.3.2 Analysis and Results

As discussed in the "Subsurface" section above, several sand and silt layers were encountered below the design ground water depth of 4 feet for Area 3 and 3 feet for Area 4. Following the procedures in the 1998 NCEER Workshop Proceedings (Youd et. al., 2001) and in accordance with CDMG Special Publication 117 guidelines (CDMG, 1997), these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.0 are considered to be potentially liquefiable.

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

The soil's CRR is estimated from the in-situ density and strength obtained from field SPT blowcounts ("N" value) from rotary-wash borings and from CPT field tip pressures. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are unreliable in sands below ground water. The "N" values and tip pressures are both corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The "N" values are also corrected for fines content, hammer efficiency,

boring diameter, rod length, and sampler type (with or without liners). The CPT method utilizes the soil behavior type index ( $I_c$ ) to estimate the plasticity of the layers. Selected soil samples collected from the correlation borings (performed adjacent to CPT-1 through CPT-4 and CPT-8 through CPT-10) were tested to evaluate the plasticity index and in-situ moisture content, as well as visual observed for confirmation of CPT soil behavior types.

Soils with significant quantities of plastic fines (PI greater than 12) generally correlate with  $I_c$  values greater than 2.6; these soils and soils with "N" values of 30 or CPT tip pressures greater than 160 tsf are typically considered too plastic or too dense/stiff to liquefy. These soil layers have been screened out during our analyses and are not presented below. The results of our preliminary CPT-based liquefaction analyses are presented in the table below.

**Table 2: Results of Liquefaction Analysis**

Development Area	CPT No.	Estimated Total Liquefaction-Induced Settlement (inches)
Area 4 (Central Upland Area)	CPT-1	1½
	CPT-2	2
	CPT-3	3¾
	CPT-4	1½
	CPT-8	¼
	CPT-9	½
	CPT-10	¼
Area 3 (77-acre Parcel)	CPT-5	4¼
	CPT-6	2¼
	CPT-7	3

**4.3.3 Summary**

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement generally ranging from ½ to 4 inches based on the Ishihara and Yoshimine (1990) method. While we are not reporting liquefaction analyses based on the hollow-stem borings, by inspection of material types and blowcounts, the borings appear to indicate similar ranges of potential liquefaction settlements as analyzed from the CPTs.

In Area 3, liquefaction-induced settlement is estimated to range from approximately 1½ to 4 inches, while approximately ½ to 4 inches of settlement is estimated within Area 4. As discussed in the SCEC report, differential movement for level ground sites over deep soil sites will be about half of the total settlement. Differential settlements are anticipated to be on the order of ½ to 2 inches over a horizontal distance of 50 feet. Further study will be required to

characterize the lateral extent and magnitude of potential liquefaction-induced settlement for design of new structures or improvements within Areas 3 and 4. Further discussion of the potential impacts due to liquefaction is presented in the "Conclusions" section.

#### **4.3.4 Ground Rupture Potential**

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the existing non-liquefiable cap of clay blanketing Area 4 is sufficient to prevent ground rupture; therefore, the above preliminary settlement estimates are considered reasonable for feasibility-level planning.

#### **4.4 LATERAL SPREADING**

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

An Alameda County Water District (ACWD) drainage channel bisects the northern portion of Areas 3 and 4. The channel is approximately 5 to 8 feet deep, and shallow layers of potentially liquefiable soils may be present as shallow as 5 to 10 feet. If localized shallow layers of potentially liquefiable soils are present near the channel, the potential for localized lateral spreading towards the ACWD channel is considered moderate to high. Further study should be performed adjacent to the ACWD channel if proposed development is to be located within 200 feet of the channel.

Area 4 is bounded by Mowry Slough to the southwest. Although the bottom of the slough is less than 10 feet deep (below original site grades), there could be a potential for lateral spreading to occur if shallow, potentially liquefiable soils are encountered in the vicinity. Further study should be performed adjacent to Mowry Slough if proposed development is to be located within 200 feet of the slough, or if the levees are to be relied upon for flood protection.

#### **4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING**

Loose unsaturated sandy soils can settle during strong seismic shaking. The unsaturated soils encountered in the borings within Area 4 were predominantly stiff to very stiff clays; therefore, the potential for significant differential seismic settlement affecting proposed improvements is considered low. Due to the relatively shallow depth to ground water in Area 3, the potential for seismic settlement of unsaturated soils is considered low, unless localized layers of loose sands are encountered above the ground water table.

#### **4.6 SLOPE STABILITY**

Based on available USGS topographic maps (Newark & Niles Quadrangles), the Areas 3 and 4 are relatively flat with a general gradient sloping to the southwest towards the San Francisco Bay. The potential for landslides within existing native soils is considered low. Any proposed embankments or engineered fill slopes should be designed and constructed to achieve an appropriate long-term static and seismic stability.

#### **4.7 FLOODING**

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, Areas 3 and 4 are located within Zones X, AE and AO (FIRM Panel 060009-0005-F, February 9, 2000). In general, most of Area 3 is located within Zone X, defined as areas of 500-year flood; areas of 100-year flood with an average depth of less than 1 foot or with drainage areas less than one square mile; and areas protected by levees from 100-year flood. The southeast corner of Area 3 is located within Zone AE, determined to have a base flood elevation of 8 feet.

Area 4 is located primarily within Zone AE with a base flood elevation of 8 feet, except for the portion between Mowry Avenue and the Alameda County Water District channel, which is considered to be in Zone X. A small portion of this area adjacent to the ACWD channel is indicated as Zone AO, determined to have a flood depth of approximately 1 foot. The project civil engineer should be retained to confirm this information and verify the base flood elevations, as appropriate.

The Association of Bay Area Governments has compiled a database of Dam Failure Inundation Hazard Maps (ABAG, 1995). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is located within a dam failure inundation area for the Calaveras, Turner, and Del Valle Reservoirs.

#### **4.8 TSUNAMI OR SEICHE**

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would

have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The western edge of Area 4 is approximately 2 to 2½ miles inland from the San Francisco Bay shoreline, and is approximately 0 to 10 feet above mean sea level. The western edge of Area 4 is bounded by an existing levee, and future residential development within Area 4 will require fill placement to raise site grades to at least 8 feet above mean sea level. Hence, the potential for inundation due to tsunami or seiche is considered low.

#### **4.9 COMPRESSIBLE SOILS**

As discussed, Area 4 lies at the margin of young, relatively compressible alluvial deposits. Based on the exploratory borings performed in Area 4, and our review of additional borings and CPTs performed for the residential development study, the proposed development area is underlain by up to approximately 20 feet of soft to medium stiff, moderately compressible silty clay. Preliminary settlement analyses were performed to estimate future long-term settlement that would be expected to occur due to areal fill placement. Preliminary settlement estimates are based on the site history previously described and are intended to provide future settlement estimates for a period of 50 years after construction.

##### **4.9.1 Areal Fill Settlement**

Based on the site history and review of consolidation test data collected during this investigation and the residential development study, we estimated that the young alluvial clay is slightly over-consolidated under the weight of existing soils and prior to placement of new fills. A modified compression index (compression ratio),  $C_{cc}$ , ranging from 0.1 to 0.18 was used for our preliminary analysis of the alluvial clays to a depth of approximately 40 feet. An over-consolidation ratio (OCR) ranging from 1.1 to 2 was also used for the young alluvial clay and underlying older alluvial soils. A design ground water depth of 3 feet below existing site grades was assumed in our analysis.

Since conceptual grading plans have not been developed, we assumed that up to 8 feet of fill could potentially be placed across the Area 4 development. We assumed an average unit weight of 125 pounds per cubic foot for new engineered fill. The results of our analysis are presented in the following table.

**Table 3: Preliminary Areal Fill Settlement Estimates – Area 4**

<b>New Fill Thickness (feet)</b>	<b>Preliminary 50-year Settlement Estimates<sup>1</sup> (inches)</b>
2	1½ to 3
4	3 to 5
6	4½ to 7
8	6 to 9

<sup>1</sup>Settlement estimates for intermediate new fill thicknesses may be approximated by linear interpolation between the above values; does not include additional settlement due to future building loads.

As an example, if 6 feet of new engineered fill were placed within the proposed development area within Area 4, approximately 4½ to 7 inches of settlement would be anticipated for a period of 50 years after construction was completed. Due to the high clay content within the alluvial clay, the rate of settlement is estimated to be roughly 2 to 3 years for 50 percent consolidation, and 10 to 15 years for 90 percent of the consolidation settlement to occur.

The above preliminary settlement estimates should be considered during conceptual planning of surface drainage and gravity-flow utilities to reduce the potential for grade reversal and joint separation or leakage. Any underground utility pipes entering buildings should be designed to accommodate the expected differential settlement between the buildings and the adjacent ground.

It should be noted that these values apply only to the central portion of Area 4 where exploration has been performed. Portions of Area 4 to the west of this central area may settle more due to possible increases in the compressibility of the underlying alluvial deposits. Due to the inherent variability of typical young alluvial clay deposits and the proximity of the site to the former tidal shoreline, additional site exploration and detailed laboratory testing and analysis will be required to further evaluate long-term settlement of compressible soils. The values presented above are intended for conceptual planning only, and should not be relied upon for detailed design of future site improvements.

**4.9.2 Building Load Settlement**

Settlement due to buildings loads is not included in the above estimates. Depending on the actual buildings sizes, loads and foundation type, post-construction settlement due to building loads alone will be additive to the values presented above. For one- to three-story, wood-frame residential construction, post-construction settlement will likely be on the order of 1 to 3 inches near the center of each building. Detailed foundation settlement estimates should be performed during future phases of investigation.

## **SECTION 5: ENVIRONMENTAL IMPACTS AND MITIGATION MEASURES**

### **5.1 OVERVIEW**

The feasibility-level recommendations that follow are intended for use during preparation of an Environmental Impact Report for Areas 3 and 4. Preliminary and design-level geotechnical investigations should be performed once conceptual and site development plans are prepared indicating where proposed improvements and structures are planned. The findings from future investigation will be used to confirm the feasibility-level recommendations and develop detailed recommendations for mitigation, design and construction.

The primary geotechnical concerns that will impact development of Area 3 are significantly different than those of Area 4 due to their relative proximity to former tidal marshes and influences from the San Francisco Bay. Area 3 is at a higher elevation and underlain by older, stiffer alluvial deposits. Area 4 is at a lower elevation and underlain by younger, softer alluvial soils. Prior to the early 1900's, Area 4 was also subject to tidal influences and was drained by numerous shallow, meandering sloughs. As a result of these differences, the primary geotechnical concerns for Areas 3 and 4 have been separated, as discussed in the following sections.

### **5.2 SIGNIFICANCE CRITERIA**

For the purposes of this evaluation, the project would have a significant effect if it would:

- Be located on a site with geologic features that pose a substantial hazard to property and/or human life (e.g., and active fault, and active landslide); or
- Expose people or property to major geologic hazards that cannot be avoided or reduced through the use of standard engineering design and seismic safety techniques; or
- Cause substantial erosion or siltation.

### **5.3 IMPACT ANALYSIS AND MITIGATION**

#### **Area 3 Impacts**

- Total and differential settlement due to potentially liquefiable soils
- Moderately expansive near-surface soils
- Shallow ground water
- Corrosion potential to buried metal
- Potential undocumented fill in areas of prior site development

## Area 4 Impacts

- Total and differential settlement due to potentially liquefiable soils
- Long-term total and differential settlement of moderately compressible alluvial soil due to the weight of any fill or building loads
- Moderately to highly expansive near-surface soils
- Shallow ground water
- Corrosion potential to buried metal and concrete structures
- Potential undocumented fill in areas of prior site development
- Long-term static and seismic stability of the Mowry Slough levees
- Differential settlement of the Stevenson Boulevard overpass embankment

Descriptions of each geotechnical or geologic impact, the level of significance, and a general discussion of potential mitigation measures follow the listed impacts. A summary of the potential impacts and mitigation measures is tabulated at the end of this report section.

### 5.3.1 Liquefaction-Induced Settlement

Preliminary liquefaction analyses indicate that there is a high potential for liquefaction of localized sand and low plasticity silt and clay layers during a significant seismic event in both Areas 3 and 4. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is relatively low, the analysis indicates that liquefaction-induced settlement on the order of ½ to 4 inches could occur in localized areas, resulting in differential settlement up to 2 inches over a horizontal distance of 50 feet. The potential for liquefaction appears to be more wide-spread in Area 3, where the sand layers are more uniform and consistent, and more isolated in Area 4 where the sands are more likely associated with isolated ancient drainage channels.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

Mitigation: Structures will need to be supported on rigid foundations designed to tolerate the anticipated total and differential settlements. Otherwise, deep foundations may be required to support structures in firm soil below potentially liquefiable layers. If improvements cannot be designed to tolerate potential settlement, ground improvement techniques could be used to mitigate differential settlement.

### 5.3.2 Settlement Due to Compressible Soils – Area 4 Only

Long-term settlement will likely govern the grading methodology and the design of foundations for the portions of Area 4 underlain by moderately compressible alluvial soil. Preliminary settlement estimates indicate that long-term (50-year) consolidation settlement on the order of 1

inch will occur for each foot of new fill placed within the development portion of Area 4. Due to the high clay content within the alluvial soils, the rate of settlement is estimated to be roughly 2 to 3 years to achieve 50 percent consolidation, and 10 to 15 years to achieve 90 percent of the consolidation settlement. It should be noted that these values apply only to the central portion of Area 4 where exploration has been performed. Portions of Area 4 to the west of this central area may settle more due to possible increases in the compressibility of the underlying alluvial deposits.

Abrupt fill thickness transitions, such as landscaping berms or different building pad elevations, will cause differential settlement across the transition areas. This could impact gravity flow utilities, retaining walls or fences, and walkways by causing abrupt settlement, sags, or cracks. Building loads will cause additional long-term settlement, the magnitude of which will be influenced by the actual load and type of foundation.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

Mitigation: Settlement due to fill and buildings loads can likely be mitigated by supporting lightly loaded structures on rigid foundations designed to resist differential settlement. As an alternative, buildings could be supported on deep foundations.

Ground improvement techniques, such as surcharging, rammed aggregate piers, or soil/cement mixing, could also be considered as settlement mitigation alternatives, if needed. If surcharging is considered, this would likely include installing vertical wick drains and surcharging building areas with additional imported fill to allow the settlement to occur at an increased rate.

### **5.3.3 Presence of Moderately Expansive Soils**

Moderately expansive surficial soils generally blanket Areas 3 and 4, which is common throughout the San Francisco Bay Area. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

Mitigation: To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. Limit moisture changes in the expansive surficial soils by using positive drainage away from buildings and improvements, as well as limiting landscaping watering. Non-expansive fill could potentially be generated on-site by treating native soils with lime to reduce the soil expansion potential.

#### 5.3.4 Shallow Ground Water

Shallow ground water could impact grading and the design of underground improvements. These impacts typically consist of potentially wet and unstable foundation subgrade, difficulty achieving compaction, and difficult underground utility installation.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

Mitigation: Design underground improvements for potential hydrostatic uplift pressures. Dewatering and shoring of utility trenches may be required in areas where deep utilities are planned.

#### 5.3.5 Soil Corrosion Potential

Due to the clayey near-surface soils and shallow ground water conditions, the corrosion potential for buried metallic structures, such as metal pipes, will likely be corrosive to severely corrosive. In addition, alluvial soils near the Bay margin typically contain moderate to high levels of soluble sulfates, which can be potentially corrosive to concrete in contact with soils containing sulfates.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

Mitigation: Soil corrosion testing should be performed in Areas 3 and 4 during future phases of investigation. It will be necessary to consult with a corrosion engineer to determine appropriate mitigation measures for site improvements. Special requirements for corrosion protection could be considered to protect metal pipelines, such as cathodic protection or specially coated pipes. In addition, if near-surface soils contain moderate to high levels of soluble sulfates, then buried concrete structures in contact with these soils may require special concrete mix design, such as using Type II cement and a higher compressive strength or Type V cement, to mitigate impacts from sulfate attack.

#### 5.3.6 Undocumented Fills

Localized undocumented fills may be encountered within previously developed parcels, such as the auto dismantler sites. Poorly compacted fills could contribute to long-term settlement of new improvements or foundations that are constructed above them. The lateral extent and depth of potential undocumented fills are not known at this time and should be further evaluated during future phases of investigation where improvements are planned.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

Mitigation: Extent unknown. Undocumented fills would likely need to be over-excavated and re-compacted or removed and replaced with engineered fill material prior to site development. As

the golf course is planned for the western portion of the site, much of the undocumented fills could potentially remain.

### **5.3.7 Mowry Slough Levee Stability**

The existing levees bordering Mowry Slough were constructed in the early 1900's and were not likely constructed to modern compaction standards. In addition, the long-term stability of the levees has likely never been evaluated. We understand that portions of Area 4 will be filled about 3 to 8 feet to place planned improvements above flood elevation. If these levees are to be relied upon for flood protection, additional studies will need to be conducted to characterize the levee materials, analyze the existing static and seismic stability, and determine possible stabilization alternatives if mitigation is required.

Level of Significance: Less than significant

Mitigation: Not required unless the residential development will rely on the levees for flood control.

### **5.3.8 Stevenson Boulevard Overpass Embankment**

The proposed railroad overpass will connect Areas 3 and 4 at the end of Stevenson Boulevard. An existing fill embankment was placed on the east side of the railroad tracks that will reportedly be used for the east bridge abutment. The west abutment embankment has not yet been constructed. Due to the underlying moderately compressible soils in Area 4, and possibly beneath the existing east embankment, differential settlement will likely occur between the two abutments.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

Mitigation: Bridge foundations will likely need to be designed to account for potential differential settlement for the bridge foundations, as well as the approached slabs and asphalt pavement sections constructed on the embankments. A site specific investigation should be performed for the proposed Stevenson Boulevard railroad overpass.

### **5.3.9 Lateral Spreading Adjacent to Existing Channels**

Localized lateral spreading could occur adjacent to the existing Alameda County Water District flood control channels or Mowry Slough. If improvements are to be constructed within setback areas adjacent to the channels, additional studies will need to be conducted to characterize the potential for lateral spreading, analyze the seismic stability, and determine possible stabilization alternatives if mitigation is required.

Level of Significance: Less than significant provided the mitigation measures below are incorporated into the project design.

**Mitigation:** Extent unknown. If needed, provide adequate setbacks from existing channels. New structures to be constructed within setback areas would likely need to be supported on deep foundations designed to resist lateral movement. Ground improvement techniques could be used to improve soil strength properties to mitigate lateral spreading potential.

**Table 4: Summary of Geotechnical/Geologic Impacts and Mitigation**

<b>Impact</b>	<b>Mitigation Measure</b>
Liquefaction-induced settlement	Design foundations to tolerate or resist settlement; implement ground improvement techniques (e.g. ground densification) to reduce long-term impacts
Settlement due to compressible soils	Design foundations to tolerate or resist settlement; implement ground improvement techniques (e.g. preloading or surcharging) to reduce long-term impacts; design gravity flow improvements to account for long-term settlement
Expansive Soils	Design foundations and improvements to tolerate or resist seasonal movement due to shrink or swell of expansive soils; implement chemical treatment (e.g. lime treatment) to alter soil properties and reduce expansion potential of on-site soils
Shallow ground water	Design underground improvements to resist hydrostatic uplift pressure, as needed; develop dewatering and shoring plans to account for shallow ground water, if needed
Soil corrosion potential	Design at-grade and underground improvements with adequate corrosion protection to reduce long-term impacts
Potential undocumented fill in prior development areas	Over-excavated/re-compact undocumented fills and/or replace with engineered fill material in planned improvement areas prior to site development
Stability of existing levees	If needed, provide adequate setbacks from existing levees; perform ground improvement to increase soil strength properties and improve the static and seismic stability
Stevenson Boulevard overpass embankment	Design foundations and approach embankments to account for post-construction differential settlement; mitigate differential settlement using ground improvement techniques
Lateral spreading adjacent to existing channels	If needed, provide adequate setbacks from existing channels; support improvements within setback areas on deep foundations to resist lateral movement; perform ground improvement to improve soil properties

## **5.4 OTHER GEOTECHNICAL CONSIDERATIONS**

### **5.4.1 Import Fill Review**

Due to the need for significant quantities of imported soil to raise site grades in Area 4, consideration should be given to the time and cost of reviewing, testing and approving potential sources of imported soil.

### **5.4.2 Additional Site Bridges**

Any future bridges constructed within Areas 3 or 4, such as crossings for the Alameda County Water District (ACWD) drainage channels, will require site specific investigations be performed at the abutments to characterize the subsurface conditions, determine suitable foundation alternatives, and to provide mitigation recommendations for static and seismic settlement, potential lateral spreading, and erosion and scour protection recommendations.

### **5.4.3 Future Geotechnical Investigations**

The recommendations contained in this feasibility study were based on limited site development information, very limited site exploration, review of available subsurface information, and our experience in the area with similar projects. As site conditions may vary significantly between the small-diameter Cone Penetration Tests performed during this investigation, we recommend that we be retained to perform preliminary and/or design-level geotechnical investigations for specific site improvements such as residential developments, bridges, or school sites, once conceptual or detailed site development plans are available.

The geotechnical aspects of the project structural, civil, and landscape plans and specifications should also be reviewed, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction. Geotechnical observation and testing services will be required during earthwork and foundation construction.

## **SECTION 6: EARTHWORK**

### **6.1 GENERAL CONSIDERATIONS**

Conventional earth moving equipment can likely be used for both Areas 3 and 4. Depending on the time of year construction begins and the quantity of grading in Area 4, however, it may be necessary to limit the weight of rubber-tired equipment to reduce disturbance and softening of shallow clay soils below the desiccated crust. In addition, it may be necessary to stage the construction of abrupt fill slopes or embankments to reduce the potential for causing slope failures due to underlying weak soils.

Imported soil placed to raise site grades in Area 4 will cause the ground surface to settle significantly over a period of 30 to 50 years. The total settlement will need to be accounted for in the design of finished surface grades for roadways, utilities, and building pads. Therefore, the

total quantity of imported fill will be greater than anticipated to account for long-term ground subsidence and to maintain site elevations above flood levels.

## **6.2 PRELIMINARY EARTHWORK GUIDELINES**

All fill as well as scarified surface soils in those areas to receive fill or slabs-on-grade should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition. Fills greater than 5 feet thick should be compacted to at least 93 percent for the portion of fill below 5 feet. The upper 6 inches of subgrade in pavement areas and all aggregate base materials should be compacted to at least 95 percent relative compaction (ASTM D-1557, latest edition). All utility trenches should be compacted to at least 90 percent relative compaction (ASTM D-1557, latest edition) by mechanical means only. Permanent cut and fill slopes should have a maximum inclination of 2:1 (horizontal:vertical) for slopes up to 10 feet high, and 3:1 for slopes greater than 10 feet high.

## **SECTION 7: LIMITATIONS**

This report has been prepared for the sole use of David J. Powers & Associates, Inc. specifically for use during preparation of the Environmental Impact Report for Study Areas 3 and 4 in Newark, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No other warranty, expressed or implied, is made or should be inferred.

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

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

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

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

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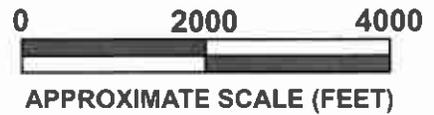
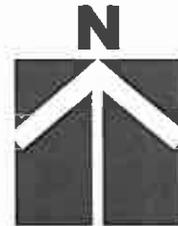
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Vicinity Map

Areas 3 and 4 EIR Study  
Newark, CA

Project Number  
118-3-3

Figure Number  
Figure 1

Date: June 2008  
Drawn by: MGV

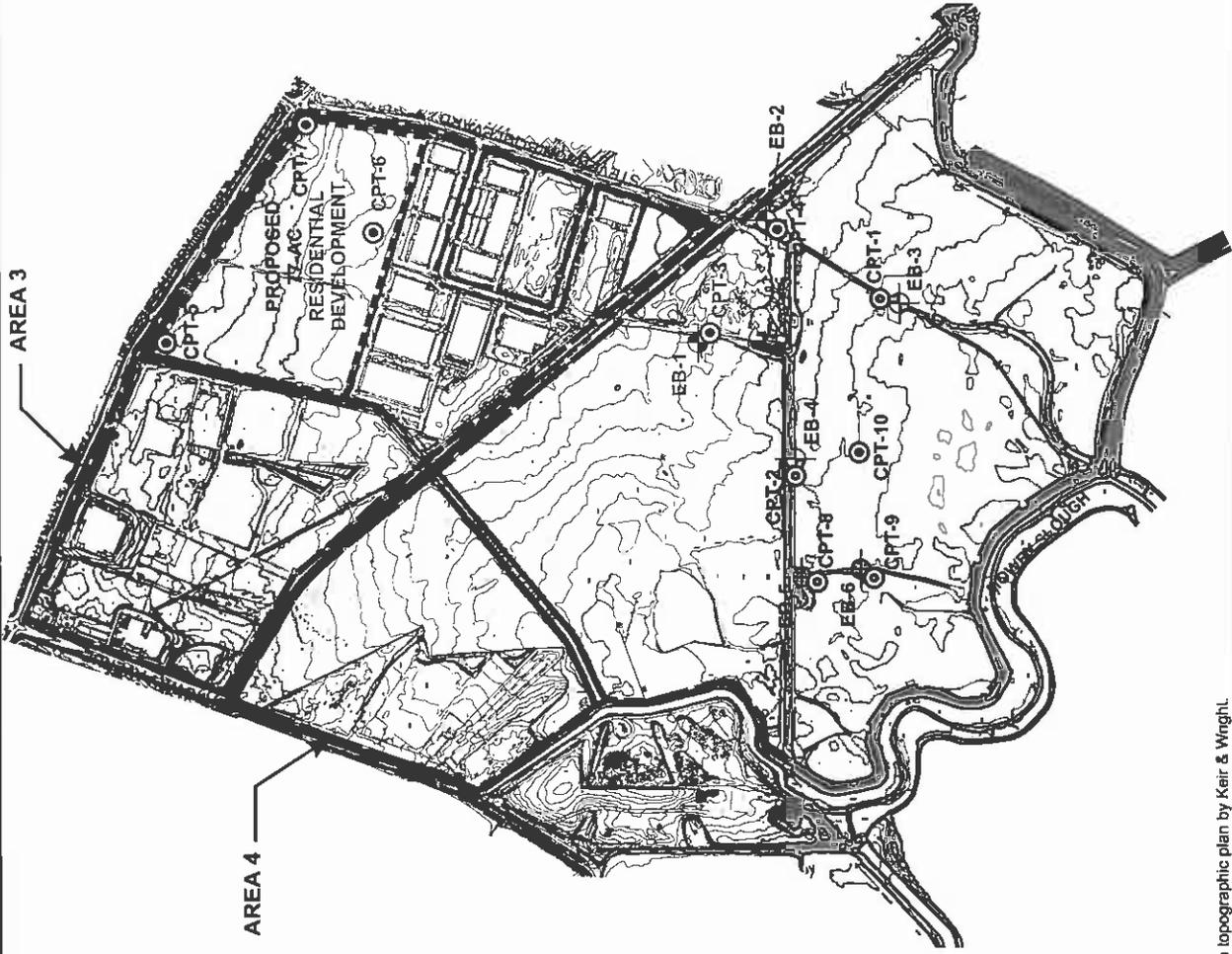
**LEGEND:**

--- Project Boundary

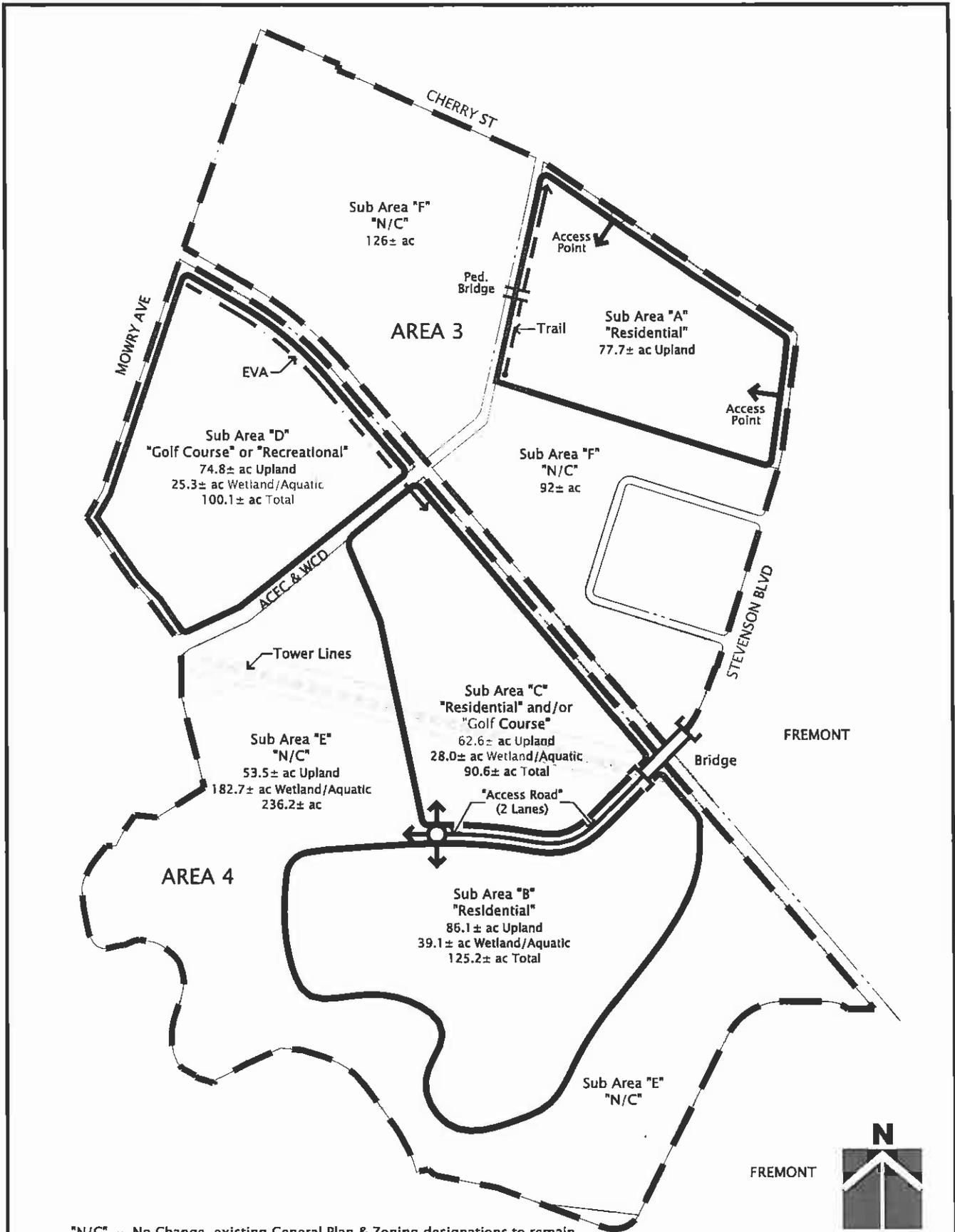
⊕ Approximate Location of  
Exploratory Boring for  
EIR Evaluation (2007)

⊕ Approximate Location of Supplemental  
Exploratory Boring for proposed  
residential development (2008)

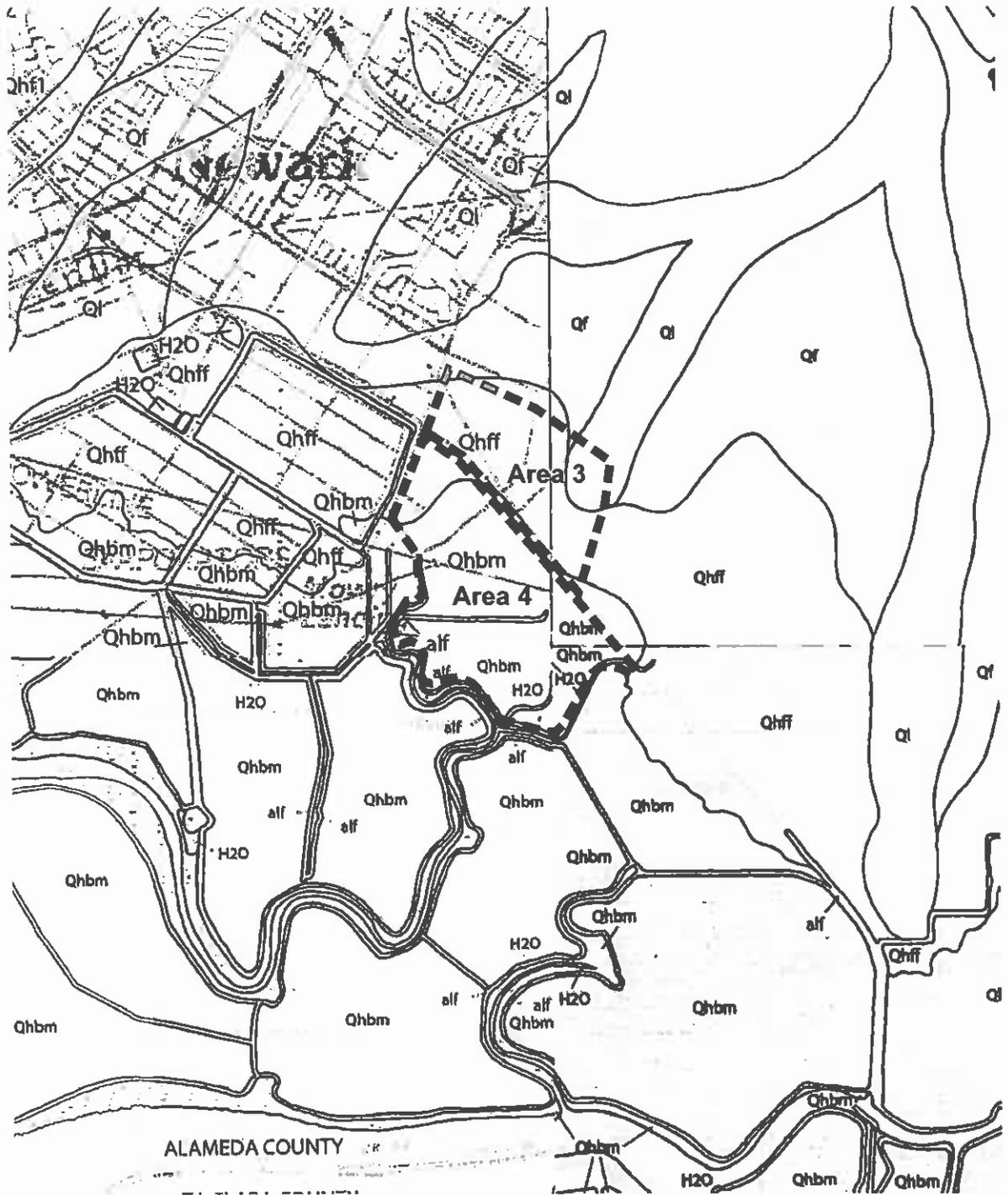
⊙ Approximate location of Cone  
Penetration Test (CPT) for proposed  
residential development (2007/2008)



Note: Base topographic plan by Keir & Wright.



 <b>CORNERSTONE EARTH GROUP</b>	<b>Conceptual Land Use Plan</b>	Project Number 118-3-3
	<b>Areas 3 and 4 EIR Study</b> <b>Fremont, CA</b>	Figure Number Figure 2A
	Date February 2009	Drawn By FLL



**Legend**

- Qhbm Holocene San Francisco Bay Mud
- Qhff Holocene alluvial fan deposits, fine-grained facies
- Qf Late Pleistocene to Holocene alluvial fan deposits
- Ql Late Pleistocene to Holocene alluvial fan levee deposits
- alf Artificial levee fill

Source: Seismic Hazard Zone maps of the Newark, Niles, Mountain View, Milpitas 7.5 Minute Quadrangle, California. California Geological Survey Report no. 90, 98, 51, and 60. (Dated: 2003).



**APPROXIMATE SCALE (FEET)**



**Regional Geologic Map  
(CGS, 2003)**

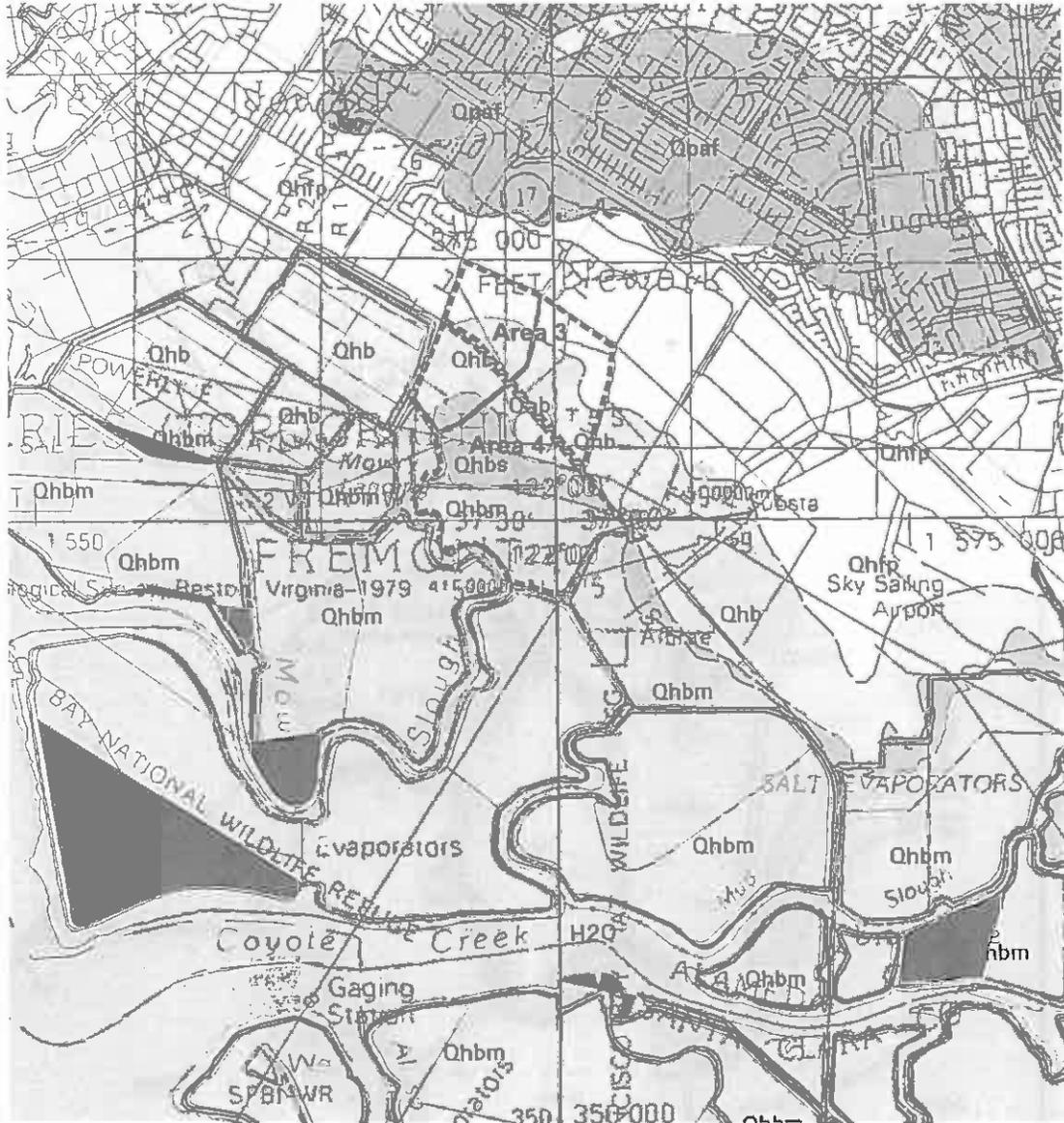
**Areas 3 and 4 EIR Study  
Newark, CA**

Project Number  
118-3-3

Figure Number  
Figure 3

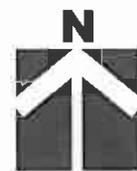
Date  
June 2008

Drawn By  
MGV



**Legend**

- Qhb - Basin deposits (Holocene)
- Qhbs - Salt affected Basin deposits (Holocene)
- Qhbm - Bay Mud deposits (Holocene)
- Qhfp - Floodplain deposits (Holocene)
- Qpaf - Alluvial Fan deposits (Pleistocene)



**APPROXIMATE SCALE (FEET)**

Source: Geologic map, Helley & Graymer, 1997.



Regional Geologic Map  
(Helley/Graymer - 1997)

Areas 3 and 4 EIR Study  
Newark, CA

Project Number  
118-3-3

Figure Number  
Figure 4

Date: June 2008 Drawn By: MGY



**Legend**

- Q Alluvium
- Qi Intertidal Deposits
- Qo Older Alluvium

Note. Base by Wagner, Burtugno, McJunkin (1991)



**APPROXIMATE SCALE (MILES)**



**Regional Geologic Map  
(Wagner et al - 1991)**

**Areas 3 and 4 EIR Study  
Newark, CA**

Project Number  
**118-3-3**

Figure Number  
**Figure 5**

Date  
June 2008

Drawn By  
MGV



CORNERSTONE  
EARTH GROUP

Areas 3 and 4 EIR Study  
Newark, CA

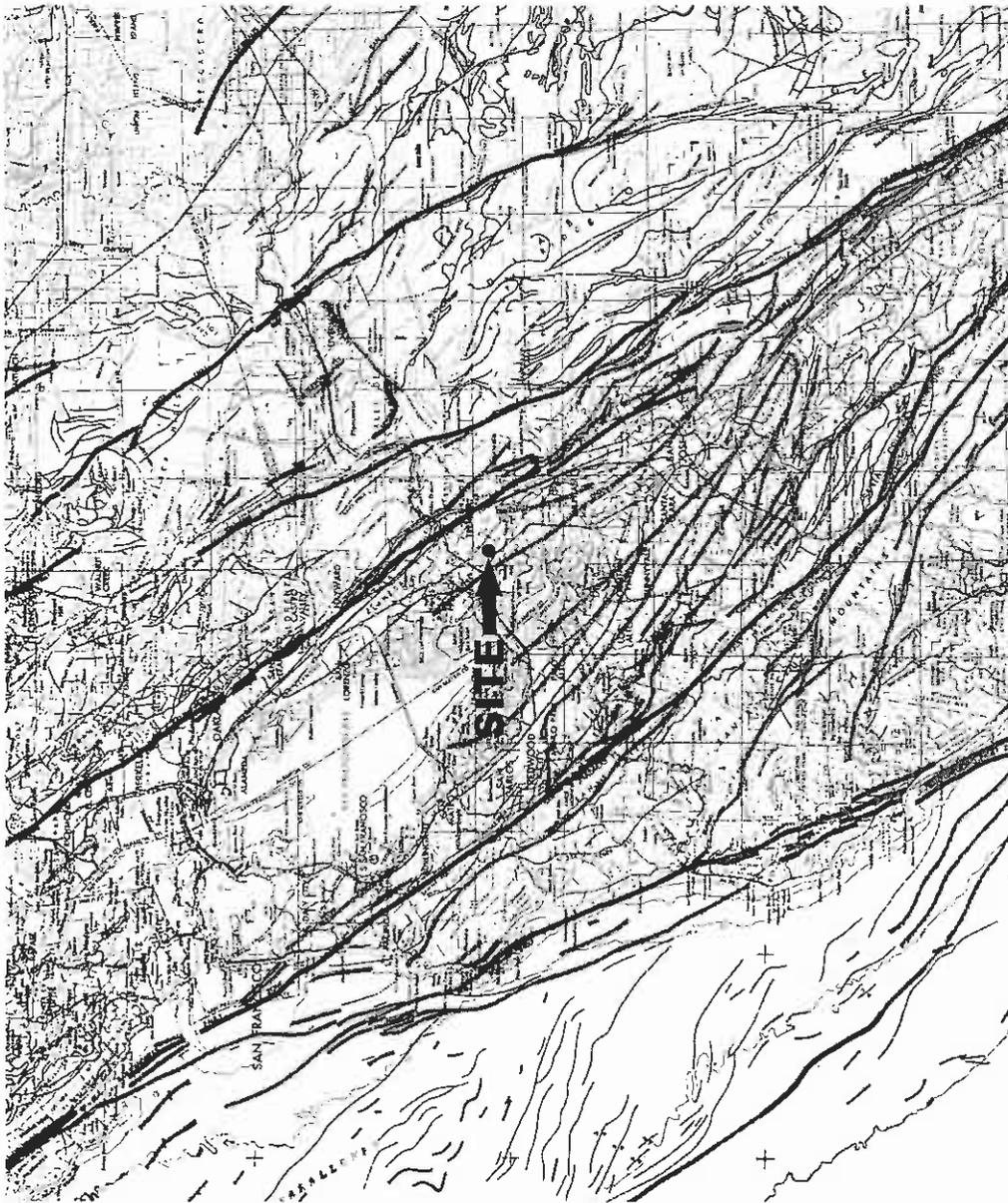
Regional Fault Map

Figure Number  
Figure 6

Project Number  
118-3-3

Date  
September 2007

Drawn By  
MGV



0 5 10 15 20  
Approximate Scale (Miles)

Geologic Time Scale	Major Fault System	Major Fault Name	Approximate Location	Approximate Length (Miles)	Approximate Width (Miles)	Approximate Depth (Miles)	Approximate Age (Millions of Years)	Description
Quaternary	San Francisco	San Francisco	San Francisco	16	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Early Quaternary	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.
Recent	Haystack	Haystack	Haystack	10	1	1	0.1	Continuation of the Hayward Fault, N. of the S.F. Bay Area. Active since 1800. The Hayward Fault is a normal fault.

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

## APPENDIX A – FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Four 8-inch-diameter exploratory borings were drilled on July 23, and 24, 2007, to depths of 40 to 46½ feet. The approximate locations of 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). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were approximated by interpolating existing topographic maps. The locations of 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. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. 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 log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Field tests included an evaluation of 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.

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.

# UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

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

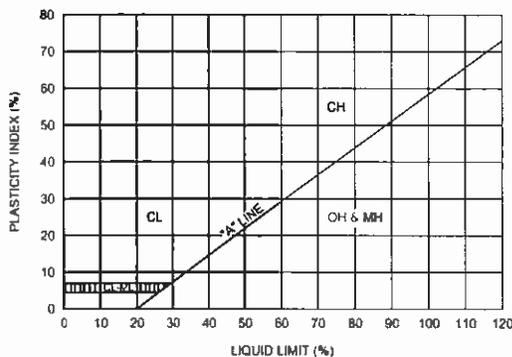
### SAMPLER TYPES

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

### ADDITIONAL TESTS

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

### PLASTICITY CHART



### PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.5
LOOSE	4 - 10	SOFT	2 - 4	0.5 - 1.0
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	1.0 - 2.0
DENSE	30 - 50	STIFF	8 - 15	2.0 - 4.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	4.0 - 8.0
		HARD	OVER 30	OVER 8.0

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

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



CORNERSTONE  
EARTH GROUP

LEGEND TO SOIL  
DESCRIPTIONS

Figure Number  
A-1



PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

GROUND ELEVATION 6 FT +/- BORING DEPTH 45 ft.

LATITUDE 37°30'17.838" LONGITUDE -123°59'54.564"

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING 20 ft.

▼ AT END OF DRILLING Not Encountered

DATE STARTED 7/23/07 DATE COMPLETED 7/23/07

DRILLING CONTRACTOR EGI

DRILLING METHOD 8" HSA

LOGGED BY JRD

NOTES \_\_\_\_\_

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. The 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.

CORNERSTONE EARTH GROUP - CORNERSTONE GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
6.0	0		<b>Silty Clay (CH)</b> very stiff, moist, dark gray, trace fine sand, some light gray mottling	21	MC-1	101	23			
			color change to mottled brown and gray	21	MC-2B	104	21			
1.0	5		<b>Silty Clay (CL)</b> stiff, moist, blue-gray and brown, trace roots, moderate plasticity	13	MC-3B	93	29			
				7	MC-4	80	40			
				9	MC-5A	85	33			
				13	MC-6A	95	27			
-14.0	20		<b>Silty Sand (SM)</b> loose to medium dense, wet, gray, fine to medium sand	8	SPT-7		25		48	
				26	SPT-8		25		31	
				16	NR					
			with occasional sandy silt seams, approximately 1 to 2 inches thick	11	SPT-9		23		25	
-26.0	30		<b>Silty Gravel (GM)</b> dense, wet, gray, subrounded	36	SPT-10		8		8	
-29.0	35									

Continued Next Page



# CORNERSTONE EARTH GROUP

**BORING NUMBER EB-1**

PAGE 2 OF 2

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
-29.0	35		<b>Silty Gravel (GM)</b> dense, wet, gray, subrounded							
	40			17	SPT-11		9		9	
	39				SPT-12					
-36.5	45		<b>Silty Clay (CL)</b> very stiff, moist, brown and gray mottled, moderate plasticity	41	MC-13					
-39.0	45		Bottom of Boring at 45.0 feet.							
	50									
	55									
	60									
	65									
	70									
	75									

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- UNDRAINED SHEAR STRENGTH, ksf
- HAND PENETROMETER
  - △ TORVANE
  - UNCONFINED COMPRESSION
  - ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
- 10 20 3.0 4.0 5.0



PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

DATE STARTED 7/24/07 DATE COMPLETED 7/24/07

GROUND ELEVATION 5 FT +/- BORING DEPTH 40 ft.

DRILLING CONTRACTOR EGI

LATITUDE 37°30'11.8584" LONGITUDE -122°0'6.9012"

DRILLING METHOD 8" HSA

GROUND WATER LEVELS:

LOGGED BY JRD

▽ AT TIME OF DRILLING 15 ft.

NOTES

▼ AT END OF DRILLING Not Encountered

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
5.0	0		<b>Silty Clay (CH)</b> stiff, moist, black, trace fine sand, high plasticity, upper 12 inches loose	25	MC-1					○
2.5			<b>Silty Clay (CH)</b> stiff, moist, dark gray, moderate to high plasticity, some carbonate nodules	14	MC-2A	100	25			○
0.0	5		<b>Silty Clay (CL)</b> medium stiff, moist, olive-brown, with gray mottles, trace fine sand, moderate plasticity	9	MC-3B	64	87			○
	10		color change to olive-gray, some brown mottles	8	MC-4A	87	32			△
	15		<b>Silty Clay (CH)</b> soft to medium stiff, moist, blue-gray, trace roots	9	MC-5	88	34			△
			<b>Sandy Silt (ML)</b> soft, wet, blue-gray, fine to medium sand	2	MC-6A	101	24	9	72	
	20			4	MC-7A	109	21	8	63	
	25		<b>Silty Clay (CH)</b> very stiff, wet, blue-gray with brown mottles, some fine sand, moderate plasticity	27	MC-8A	111	19			△
				25	MC-9A					○
	30		<b>Silty Clay (CL)</b> medium stiff to stiff, wet, brown, some fine sand	11	MC-10A	109	20			○
	35			19	MC-11					○

Continued Next Page

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# CORNERSTONE EARTH GROUP

**BORING NUMBER EB-2**

PAGE 2 OF 2

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf					
										○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0 5.0					
-30.0	35		<b>Silty Clay (CL)</b> medium stiff to stiff, wet, brown, some fine sand  (no recovery with MC, sampled with SPT to recover disturbed sample)	19	○ NR X SPT-12										
-35.0	40		Bottom of Boring at 40.0 feet.												
	45														
	50														
	55														
	60														
	65														
	70														
	75														

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ



PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

DATE STARTED 7/23/07 DATE COMPLETED 7/23/07

GROUND ELEVATION 2 FT +/- BORING DEPTH 40 ft.

DRILLING CONTRACTOR EGI

LATITUDE 37°29'58.7364" LONGITUDE -123°59'57.3108"

DRILLING METHOD 8" HSA

GROUND WATER LEVELS:

LOGGED BY JRD

▽ AT TIME OF DRILLING 15 ft.

NOTES \_\_\_\_\_

▼ AT END OF DRILLING 4 ft.

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

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
2.0	0		<b>Silty Clay (CL) [Fill]</b> stiff, moist, brown-gray, trace sand							
-1.0	5		<b>Silty Clay (CL)</b> stiff, moist, dark gray, high plasticity, some carbonate nodules	22	MC-1					
				14	MC-2A	83	30			
				10	MC-3A	122	25			
			color change to light gray with brown mottles							
-6.0	10		<b>Silty Clay (CH)</b> medium stiff to stiff, moist, blue-gray, some brown mottling, trace organics	6	MC-4A	94	29			
					ST-5					
			clayey sand seam from 12 1/2 feet to 13 feet	5	MC-6B	92	28			
					ST-7					
				15						
				15	MC-8A	105	23	18		
-19.5	20		<b>Silty Clay (CL)</b> stiff, moist, brown, trace fine sand, moderate plasticity							
				22	MC-9A					
				16	MC-10A	108	20			
				25	MC-11A	101	25			
			color change to olive-brown with some black staining							

Continued Next Page



# CORNERSTONE EARTH GROUP

**BORING NUMBER EB-3**

PAGE 2 OF 2

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
-33.0	35		<b>Silty Clay (CL)</b> stiff, moist, brown, trace fine sand, moderate plasticity							
			thin clayey sand seam at 38.5 feet to 38.8 feet	23	MC-12					
-38.0	40		Bottom of Boring at 40.0 feet.							
	45									
	50									
	55									
	60									
	65									
	70									
	75									

CORNERSTONE EARTH GROUP - CORNERSTONE GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ



PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

GROUND ELEVATION 3 FT +/- BORING DEPTH 46.5 ft.

LATITUDE 37°30'9.108" LONGITUDE -123°59'41.2836"

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING 15 ft.

▼ AT END OF DRILLING Not Encountered

DATE STARTED 7/23/07 DATE COMPLETED 7/23/07

DRILLING CONTRACTOR EGI

DRILLING METHOD 8" HSA

LOGGED BY JRD

NOTES \_\_\_\_\_

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
3.0	0		<b>Clayey Sand (SC) [Fill]</b> medium dense, moist, brown, fine to medium sand, some gravel	29	MC-1A	103	10			○
1.7			<b>Silty Clay (CH)</b> very stiff to hard, moist, black, trace fine sand, high plasticity	22	MC-2A	108	13			○
-1.5	5		<b>Silty Clay (CL)</b> stiff, moist, olive brown and gray mottled, moderate plasticity, some carbonate nodules	12	MC-3A	103	24			△
	10		becomes soft, some organics	6	MC-4A	83	37			⊗
					ST-5		37			⊗
-11.0	15		<b>Silty Clay (CH)</b> soft, wet, dark gray to olive gray, high plasticity, some sand	4	MC-7	96	28			○
-15.0	20		<b>Sandy Clay (CL)</b> medium stiff to stiff, wet, olive-brown, fine to medium sand	6	MC-8	103	23			△
	25			19	MC-9A	108	20	14		○
	30			13	MC-10B	108	21			○
-32.0	35									

Continued Next Page



# CORNERSTONE EARTH GROUP

**BORING NUMBER EB-4**

PAGE 2 OF 2

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf	TEST TYPES									
											○	△	●	▲	1.0	2.0	3.0	4.0	5.0	
-32.0	35																			
-32.5			<b>Silty Sand (SM)</b> loose, wet, brown, fine to medium sand	11	MC-11A	103	22		38											
-36.0	40		<b>Silty Clay (CL)</b> medium stiff to stiff, wet, brown, trace fine sand	13	MC-12															
	45		3 inch clayey sand seam at 44 1/2 feet	9	MC-13															
-43.5			Bottom of Boring at 46.5 feet.	8	SPT-14															
	50																			
	55																			
	60																			
	65																			
	70																			
	75																			

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ

## APPENDIX B – LABORATORY TEST PROGRAM

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

### **Moisture Content**

The natural water content was determined (ASTM D2216) on 37 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 31 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.

### **Washed Sieve Analyses**

The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on eight 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.

**Plasticity Index:** Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits 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 shown on the boring logs at the appropriate sample depths.

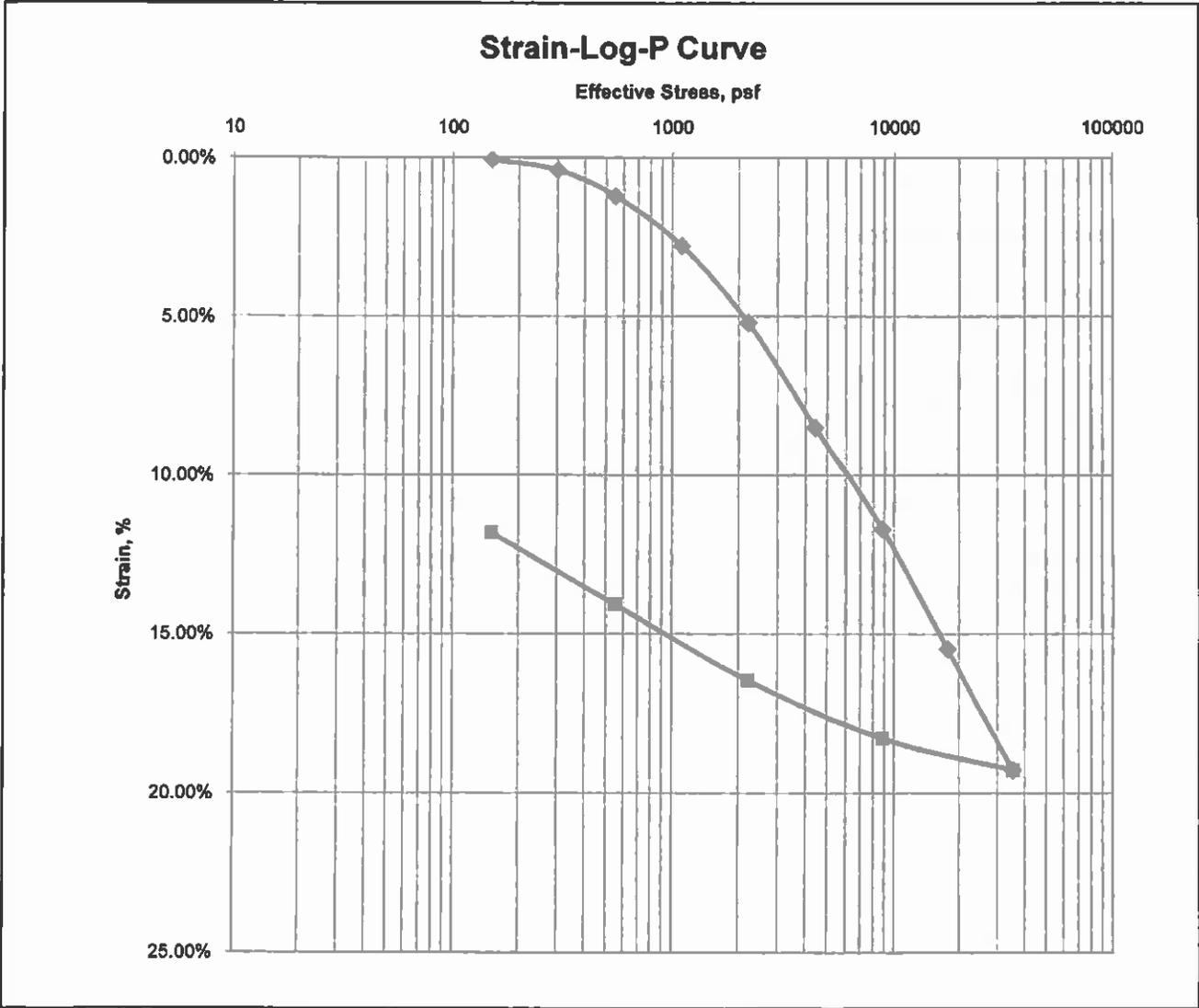
**Consolidation:** One consolidation test (ASTM D2435) was performed on a relatively undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation test are presented graphically in this appendix.



# Consolidation Test

## ASTM D2435

Job No.: <u>640-027</u>	Boring: <u>EB-4</u>	Run By: <u>MD</u>
Client: <u>Cornerstone Earth Group</u>	Sample: <u>6</u>	Reduced: <u>PJ</u>
Project: <u>Newark - 118-3-3</u>	Depth, ft.: <u>12.5-15</u>	Checked: <u>PJ/DC</u>
Soil Type: <u>Dark Gray CLAY, trace Sand</u>		Date: <u>8/7/2007</u>



Ass. Gs =	2.7	<b>Initial</b>	<b>Final</b>
Moisture %:		30.8	23.8
Dry Density, pcf:		91.8	102.7
Void Ratio:		0.836	0.641
% Saturation:		99.4	100

**Remarks:**

**APPENDIX C – EXPLORATORY BORING LOGS AND LABORATORY TEST RESULTS  
FROM RESIDENTIAL DEVELOPMENT STUDY (CORNERSTONE EARTH  
GROUP, 2008)**



PROJECT NAME Newark Feasibility Area 3 and 4

PROJECT NUMBER 102-4-1

PROJECT LOCATION Newark, CA

DATE STARTED 4/25/08 DATE COMPLETED 4/25/08

GROUND ELEVATION 6 FT +/- BORING DEPTH 35 ft.

DRILLING CONTRACTOR EGI

LATITUDE 37°30'8.5608" LONGITUDE -123°59'29.3676"

DRILLING METHOD 8" HSA

GROUND WATER LEVELS:

LOGGED BY Jacob Fink

▽ AT TIME OF DRILLING 13 ft.

NOTES

▼ AT END OF DRILLING 10 ft.

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
6.0	0		<b>Silty Clay (CL) [Fill]</b> stiff, moist, dark gray, moderate to high plasticity	20	MC-1	92	23			○
4.0			<b>Silty Clay (CL)</b> medium stiff to soft, wet, olive gray, moderate to high plasticity	6	MC-2B	65	62			○
0.0	5		<b>Silty Clay with Fine Sand (CH)</b> medium stiff, wet, olive gray, calcite nodules, fine to medium sand, moderate to high plasticity	16	MC-3B	98	27			○
	10			10	MC-4B	88	31			○
	15		color change to dark gray	10	MC-6B	102	24			○
	20		very stiff, color change to olive gray	37	MC-8B	114	18			○
-18.5	25		<b>Sandy Clay (CL)</b> stiff, wet, light brown, calcite nodules, fine sand, low to moderate plasticity	47	MC-9B	118	17			○
	30			30	MC-10B	102	24			○
-25.0			<b>Clayey Silt with Sand (ML)</b> soft, wet, olive gray with light brown, increase sand with depth	9	MC-11B				78	○
-29.0	35		Bottom of Boring at 35.0 feet.							

CORNERSTONE EARTH GROUP - CORNERSTONE GDT - 6/9/08 11:08 - P:\DRAFTING\GINT FILES\102-4-1 NEWARK AREA 3 AND 4.GPJ



PROJECT NAME Newark Feasibility Area 3 and 4

PROJECT NUMBER 102-4-1

PROJECT LOCATION Newark, CA

DATE STARTED 4/25/08 DATE COMPLETED 4/25/08

GROUND ELEVATION 5 FT +/- BORING DEPTH 44 ft.

DRILLING CONTRACTOR EGI

LATITUDE 37°30'3.3984" LONGITUDE -123°59'29.9724"

DRILLING METHOD 8" HSA

GROUND WATER LEVELS:

LOGGED BY Jacob Fink

▽ AT TIME OF DRILLING 13 ft.

NOTES \_\_\_\_\_

▼ AT END OF DRILLING 11.33 ft.

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
5.0	0		<b>Silty Clay (CL) [Fill]</b> stiff, moist, dark gray, moderate to high plasticity	27	MC-1	81	41			
2.0	5		<b>Silty Clay (CH)</b> medium stiff, wet, olive gray to light gray, moderate to high plasticity brown mottles, roots	12	MC-2B	86	35			
	15			15	MC-3B	95	30			
	10			9	MC-4	89	33			
	12		color change to dark gray	6	MC-6B	102	24			
	15				ST-7					
	20		stiff, moist	33	MC-8	107	22			
	25		color change to olive gray	48	MC-9B	118	17			
	25		<b>Sandy Clay (CL)</b> very stiff, moist, light brown, fine sand, low to moderate plasticity	14	MC-10B	106	22			
	30									
	30		<b>Clayey Silt with Sand (ML)</b> soft, wet, olive gray with brown mottles, minor amounts of coarse sand	9	SPT-11		28		88	
	35									

Continued Next Page



# CORNERSTONE EARTH GROUP

PROJECT NAME Newark Feasibility Area 3 and 4

PROJECT NUMBER 102-4-1

PROJECT LOCATION Newark, CA

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf	TEST RESULTS									
											○	△	●	▲	1.0 2.0 3.0 4.0 5.0					
-30.5	-31.0		<b>Sandy Clay (CL)</b> stiff, moist, olive gray with brown mottles, fine sand, moderate plasticity	22	SPT-12A		27	9	89											
				31	SPT-13		24													
	40			30	MC-14B	109	21													
-37.0			<b>Silty Sand (SM)</b> medium dense, wet, light brown, fine grained rounded sand, decrease silt content with depth	19	SPT-15A				77											
-38.5	-39.0			26	SPT-16A 16B		22		28											
	45		<b>Silty Clay (CL)</b> very stiff, moist, olive gray with minor amounts of brown mottles, high plasticity																	
			Bottom of Boring at 44.0 feet.																	
	50																			
	55																			
	60																			
	65																			
	70																			
	75																			

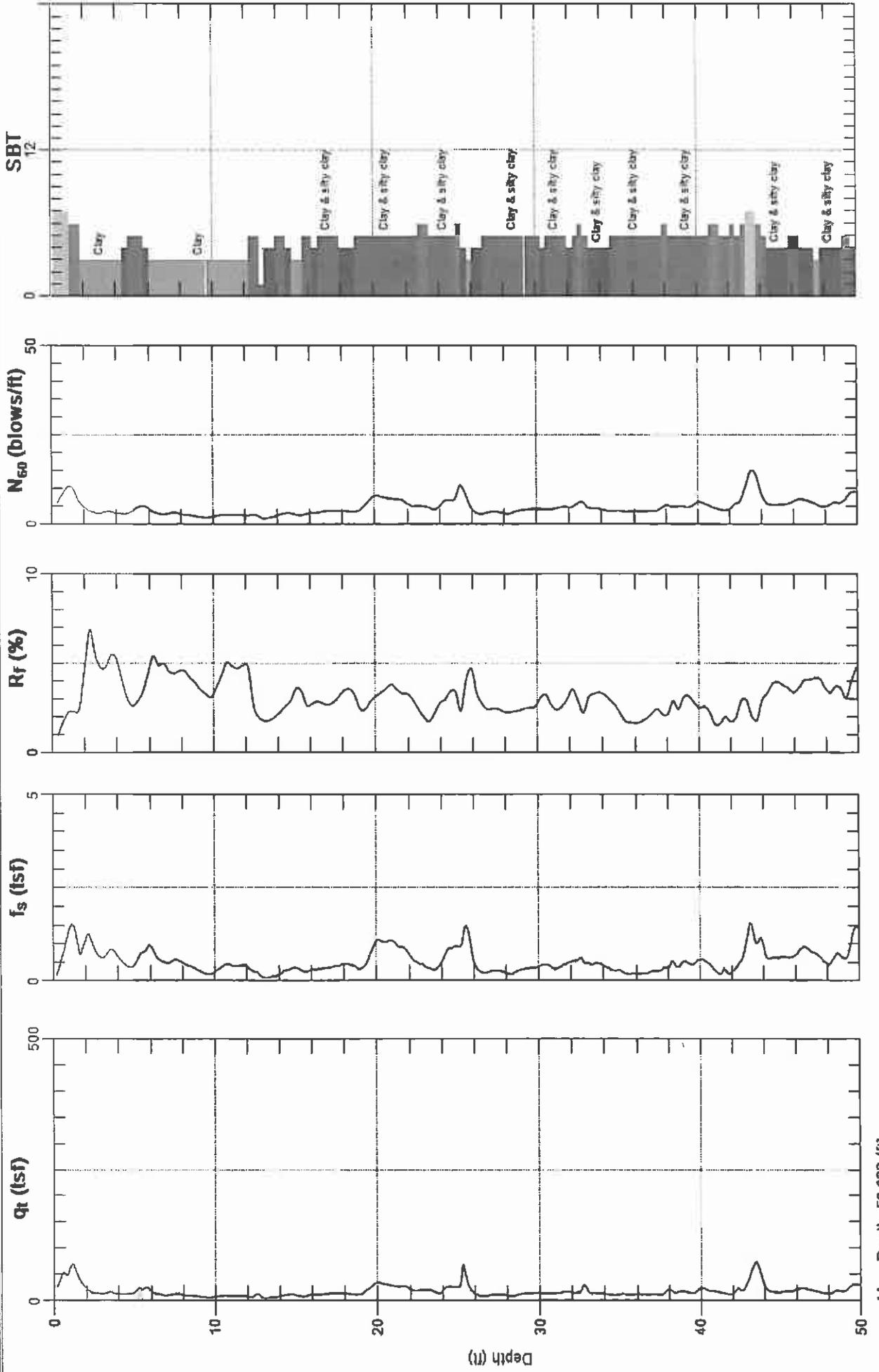
CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:05 - P:\DRAFTING\GINT FILES\102-4-1 NEWARK AREA 3 AND 4.GPJ



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-01

Engineer: J.DYE  
Date: 5/9/2007 07:33



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

SBT: Soil Behavior Type (Robertson 1990)



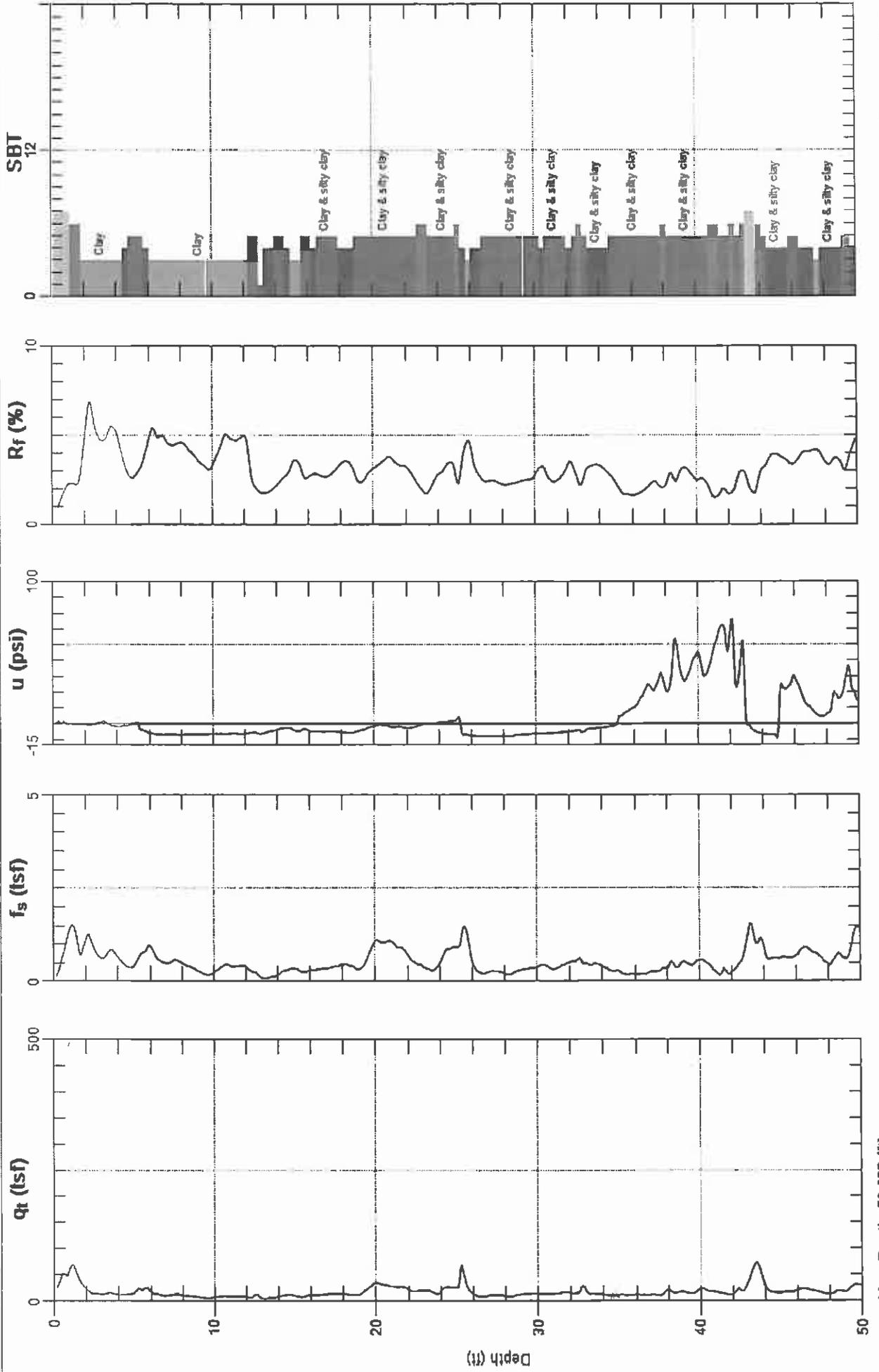
# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4

Engineer: J.DYE

Sounding: CPT-01

Date: 5/9/2007 07:33



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

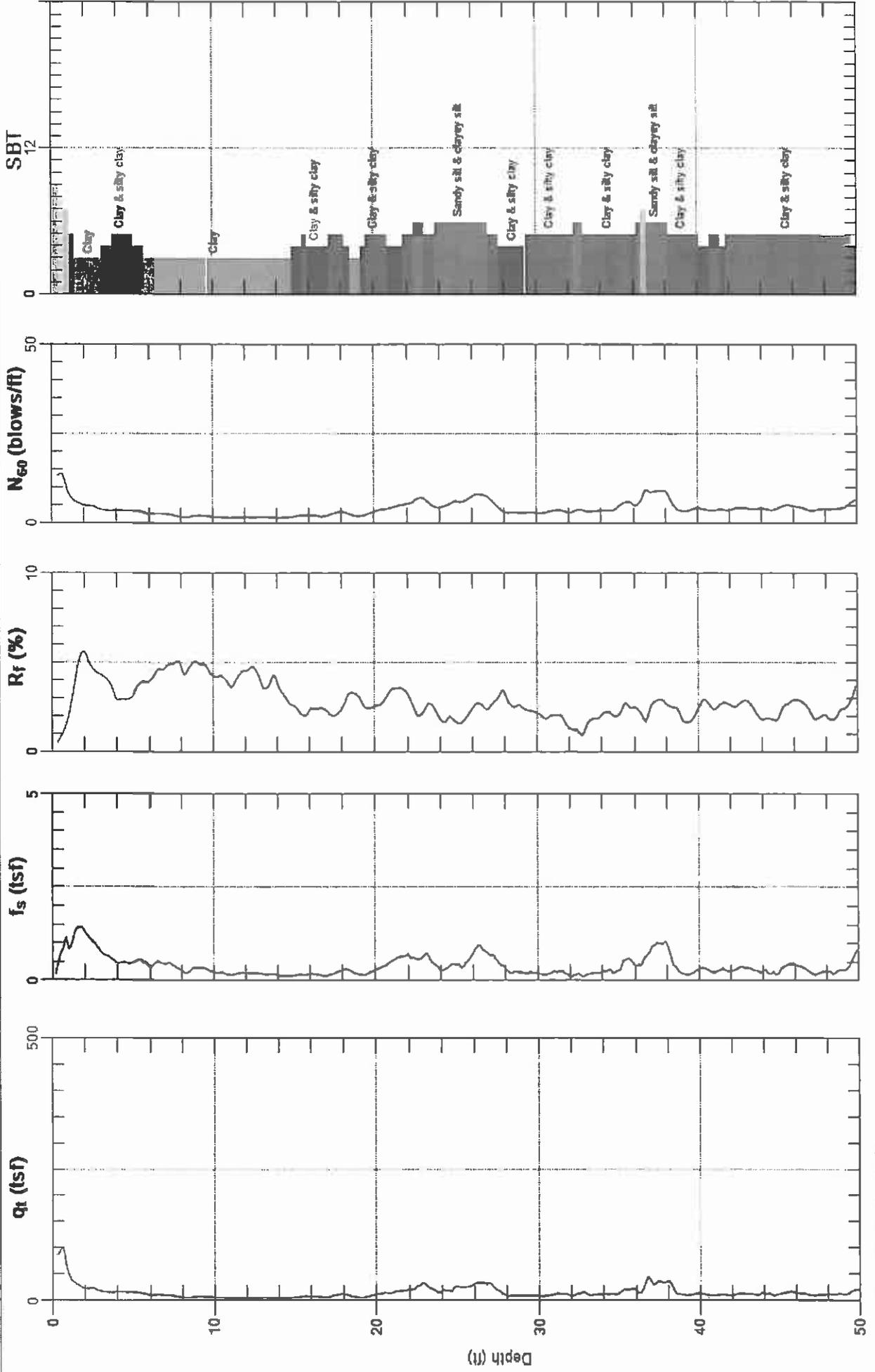
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-02

Engineer: J.DYE  
Date: 5/9/2007 08:52



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

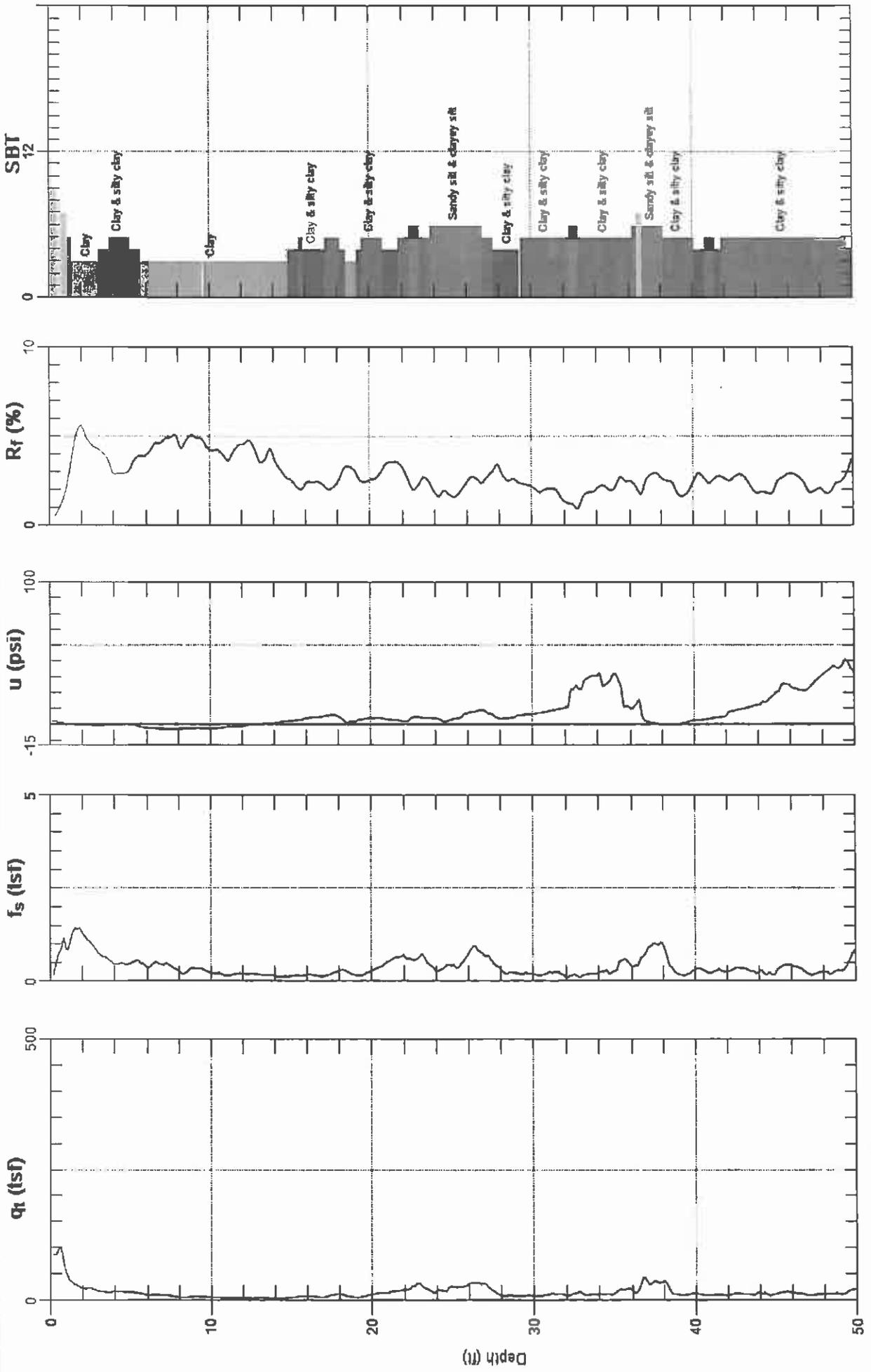
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-02

Engineer: J.DYE  
Date: 5/9/2007 08:52



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

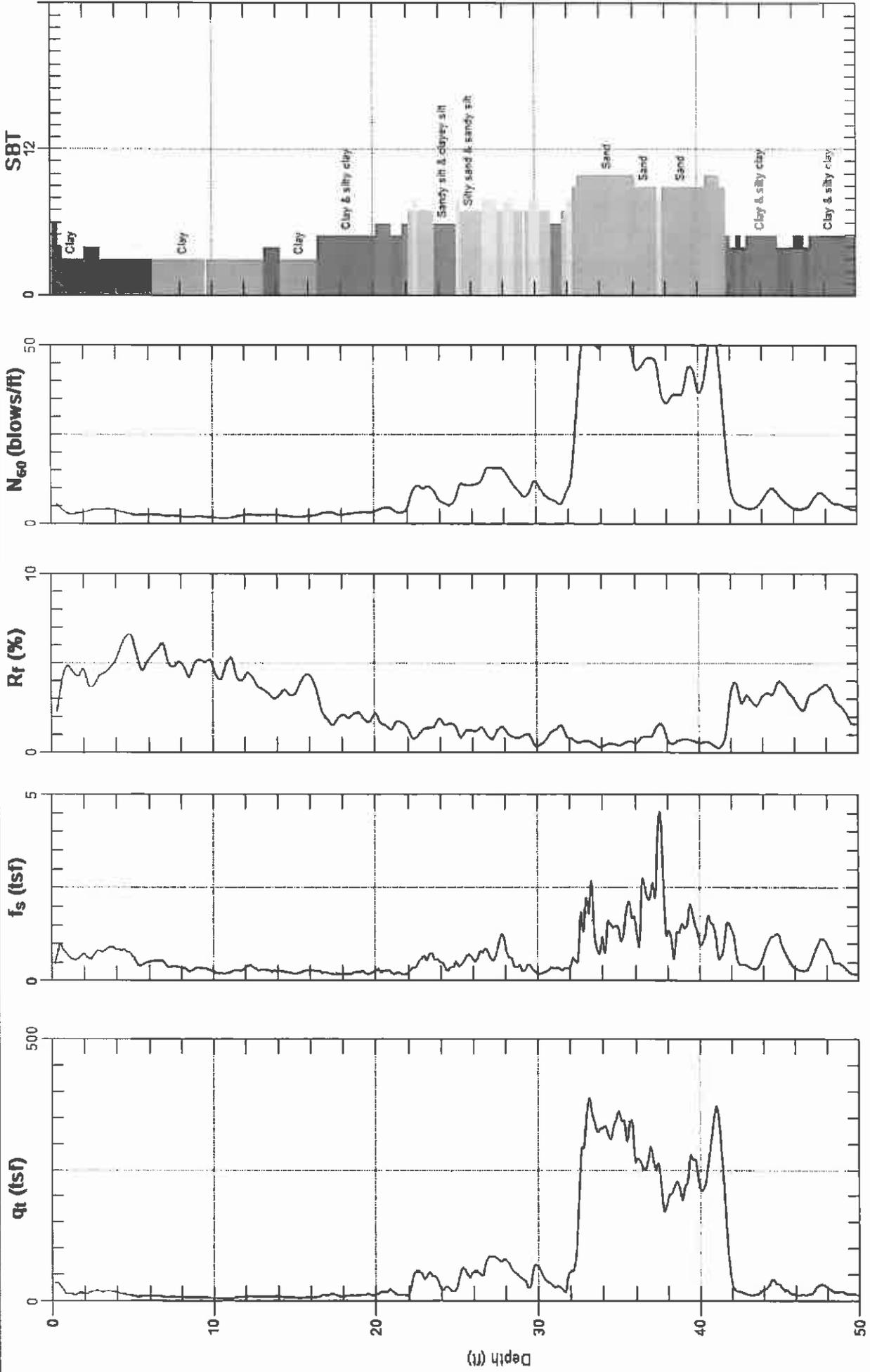
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-03

Engineer: J.DYE  
Date: 5/9/2007 10:42



Max Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

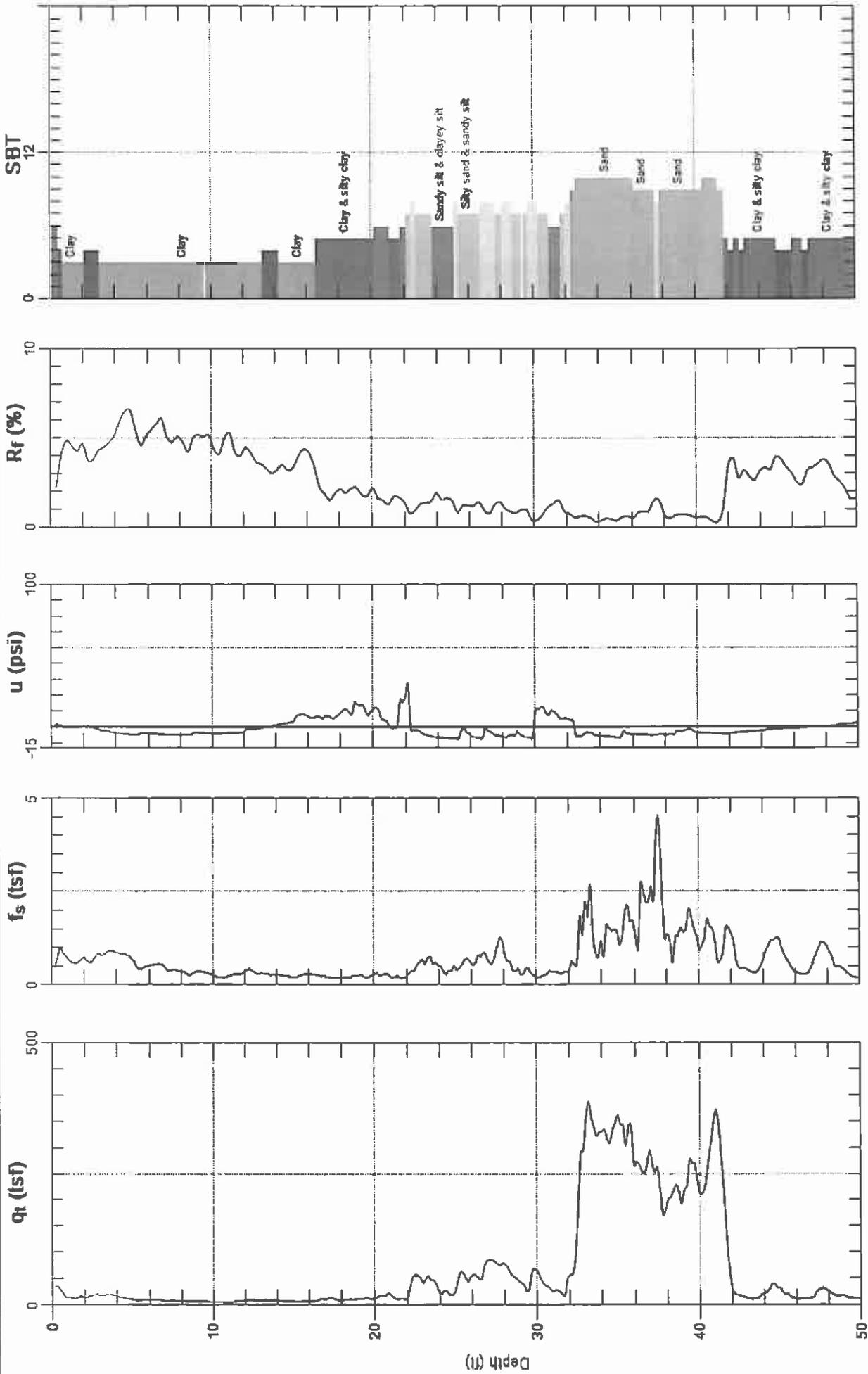
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-03

Engineer: J.DYE  
Date: 5/9/2007 10:42



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

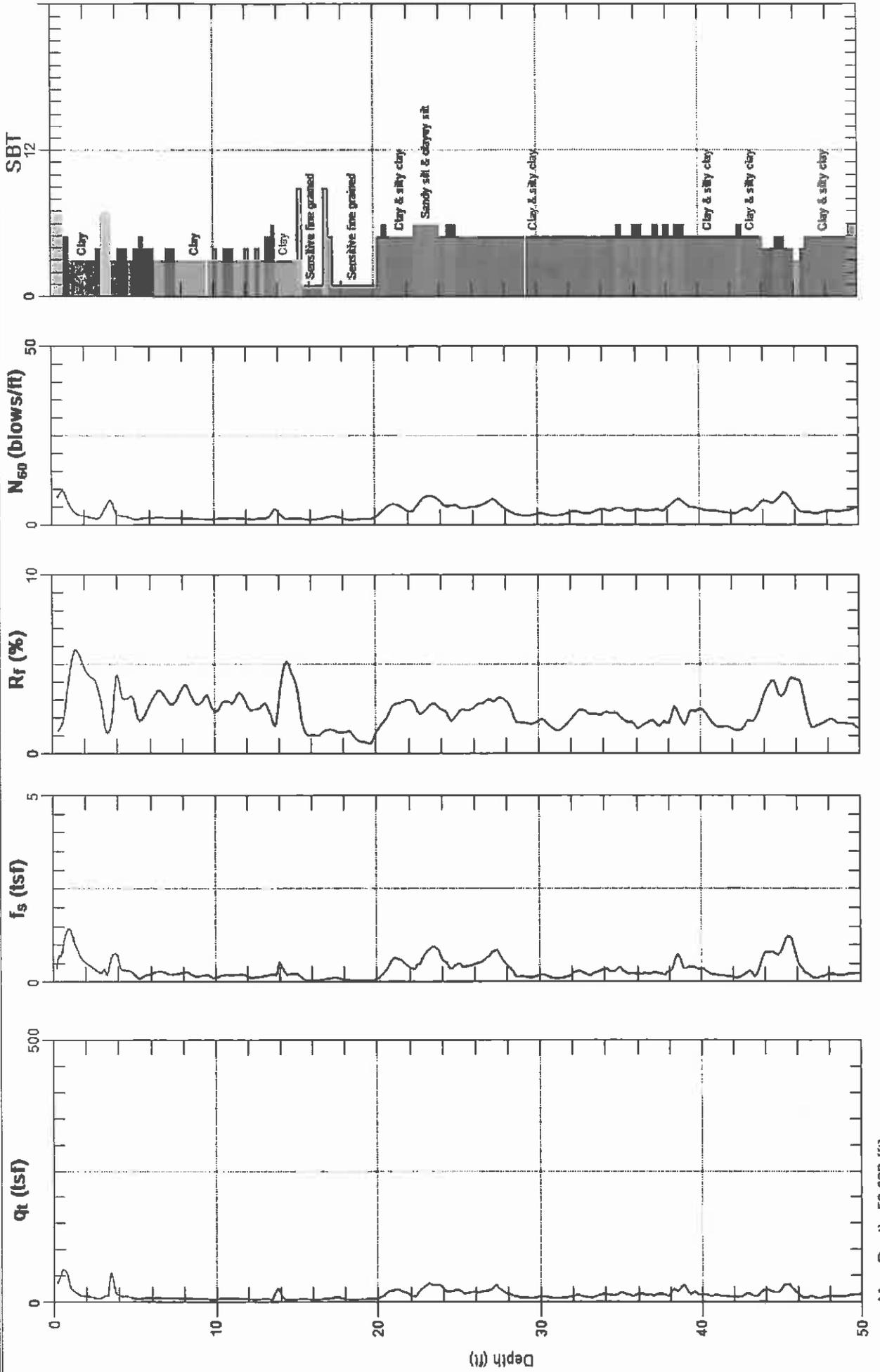
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-04

Engineer: J.DYE  
Date: 5/9/2007 06:12



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

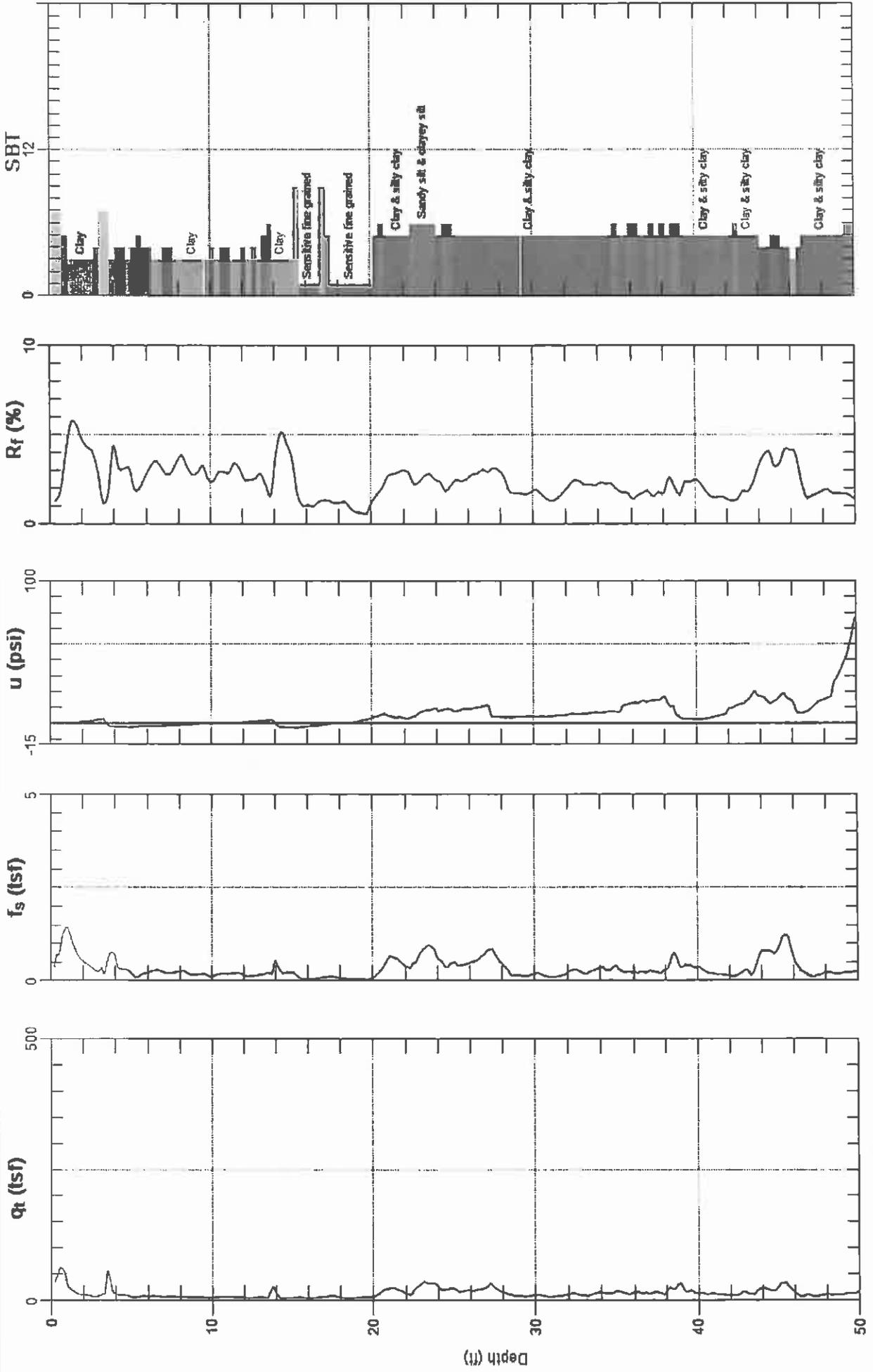
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-04

Engineer: J.DYE  
Date: 5/9/2007 06:12



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

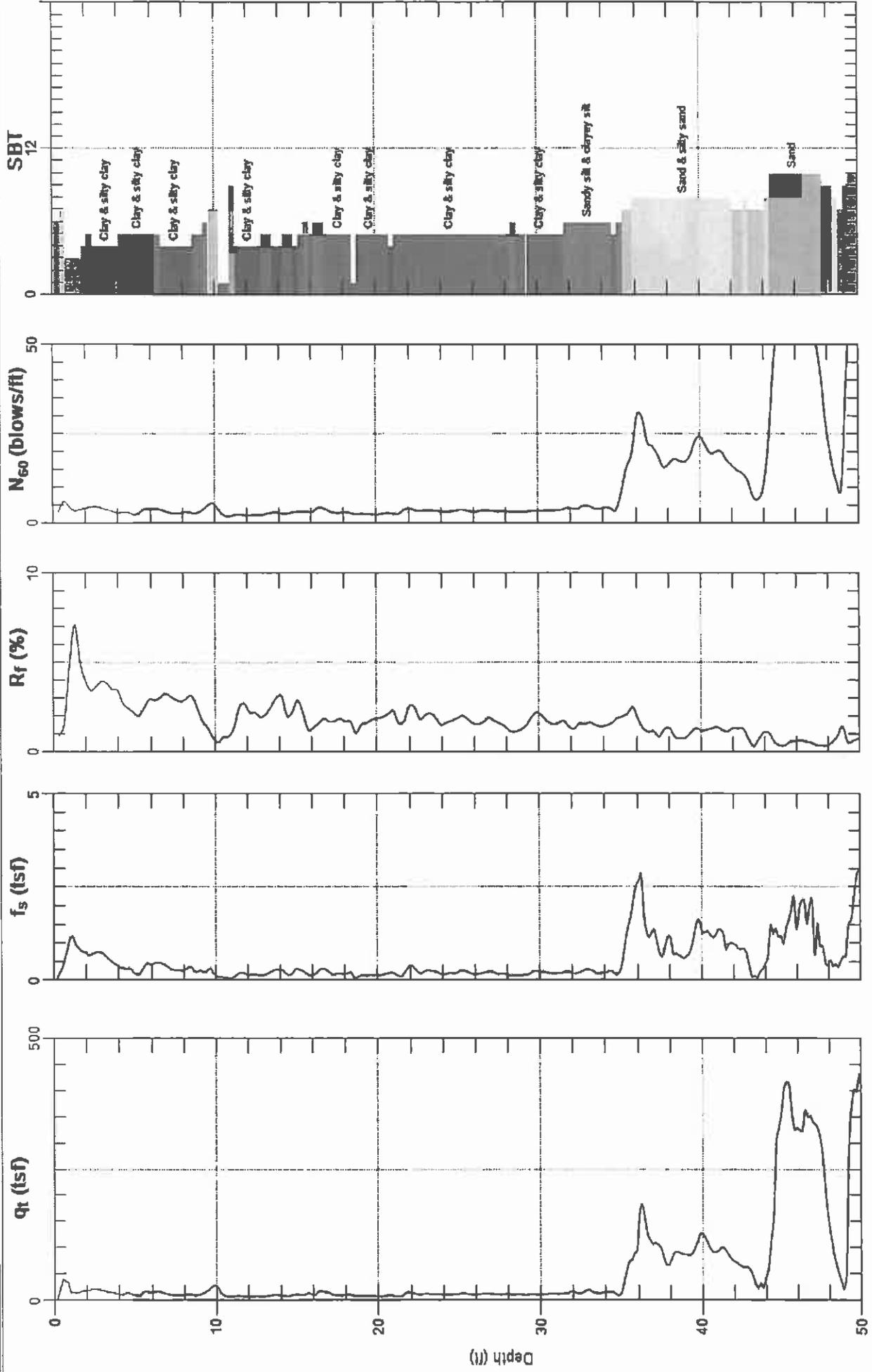
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-05

Engineer: J.DYE  
Date: 5/10/2007 05:37



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

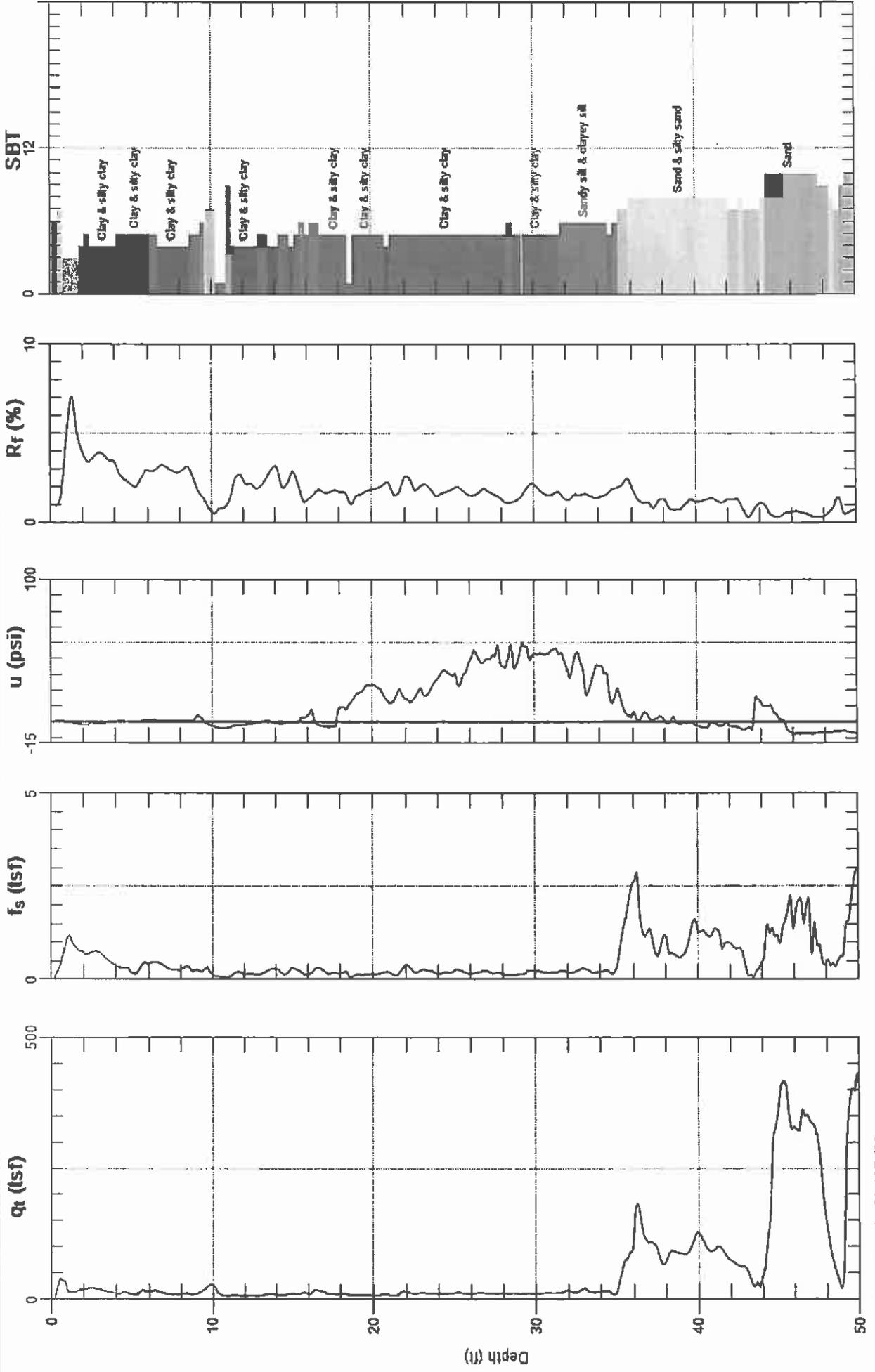
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-05

Engineer: J.DYE  
Date: 5/10/2007 05:37



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

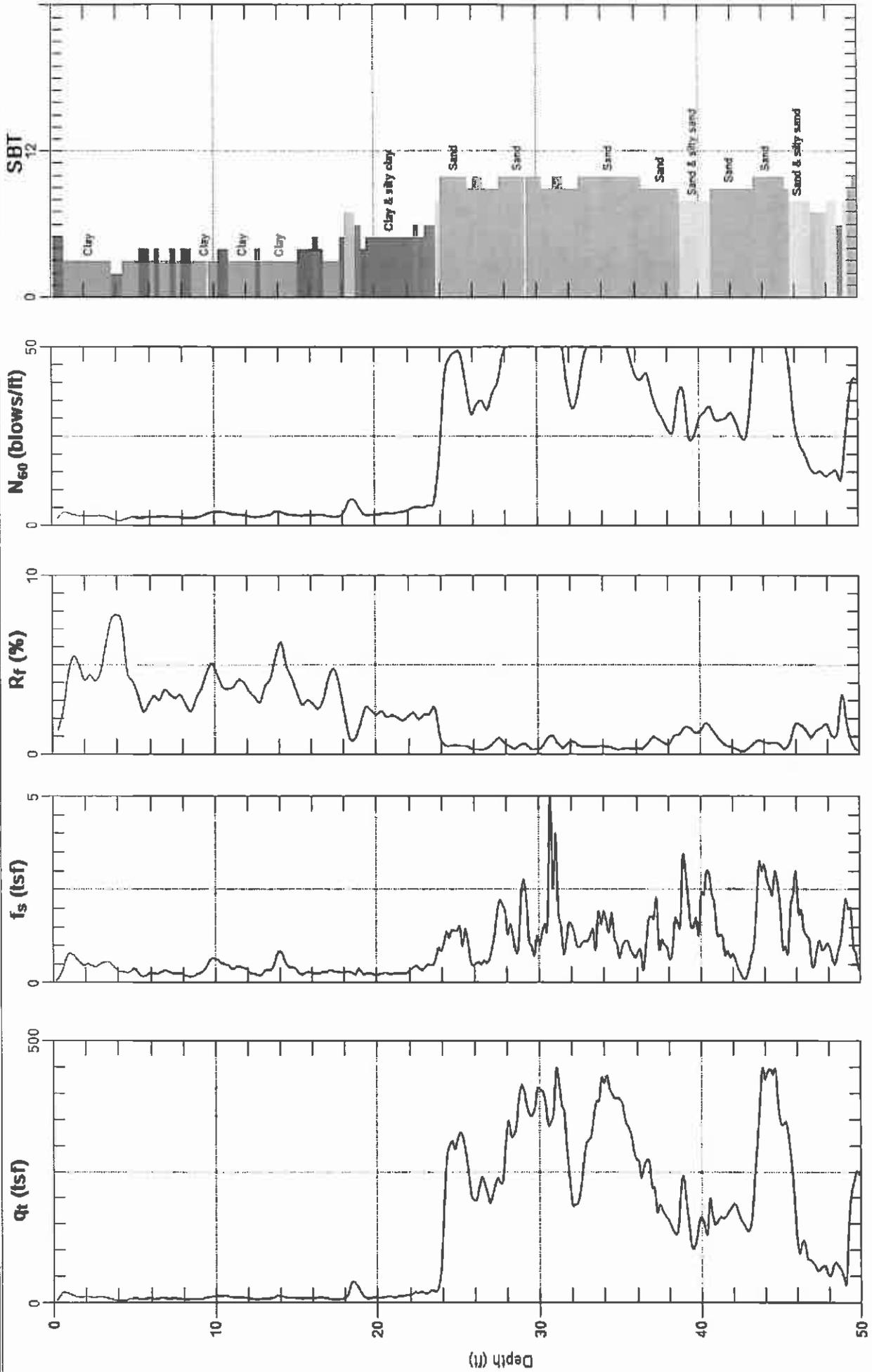
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-06

Engineer: J.DYE  
Date: 5/9/2007 01:40



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

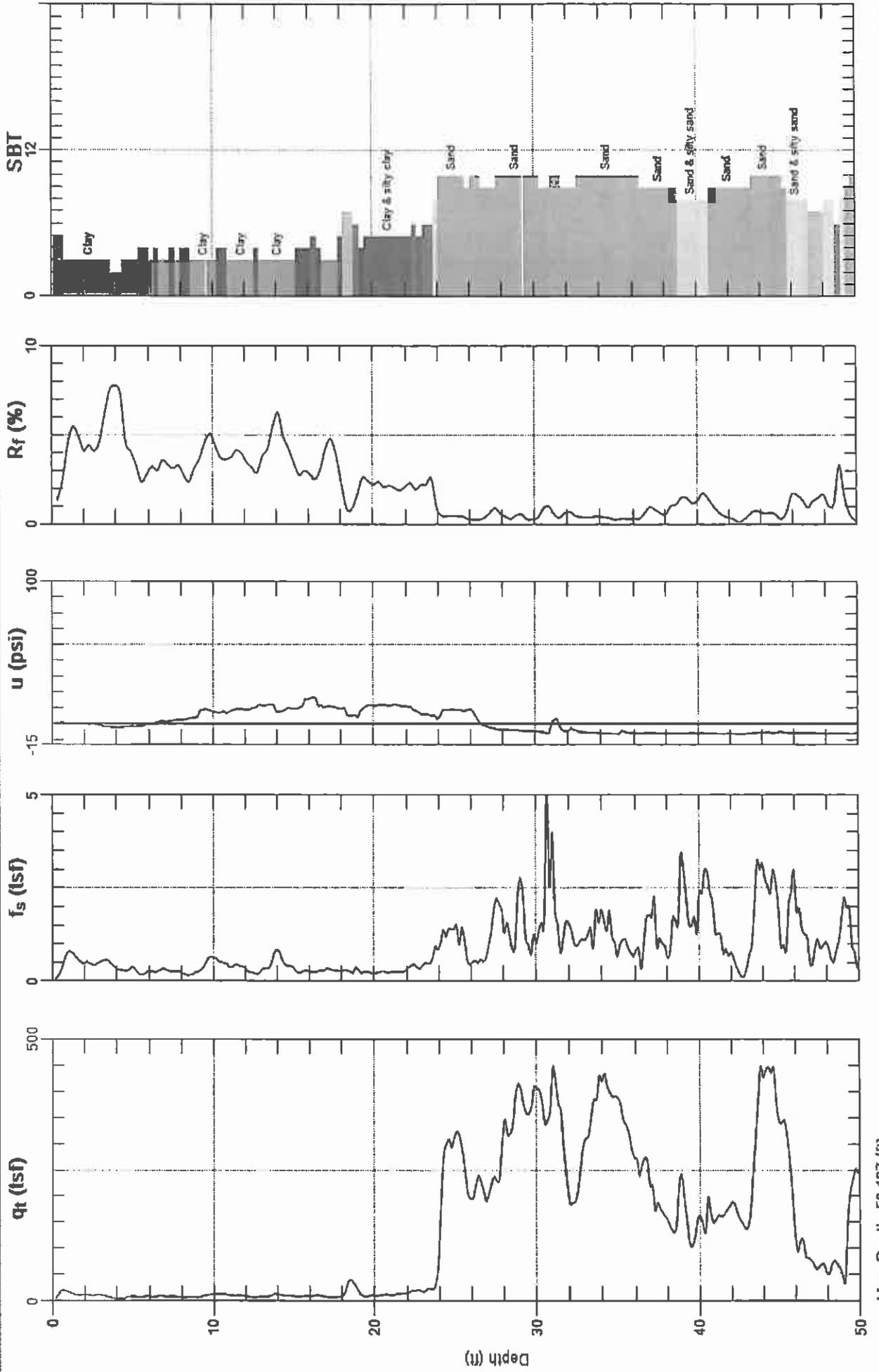
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-06

Engineer: J.DYE  
Date: 5/9/2007 01:40



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

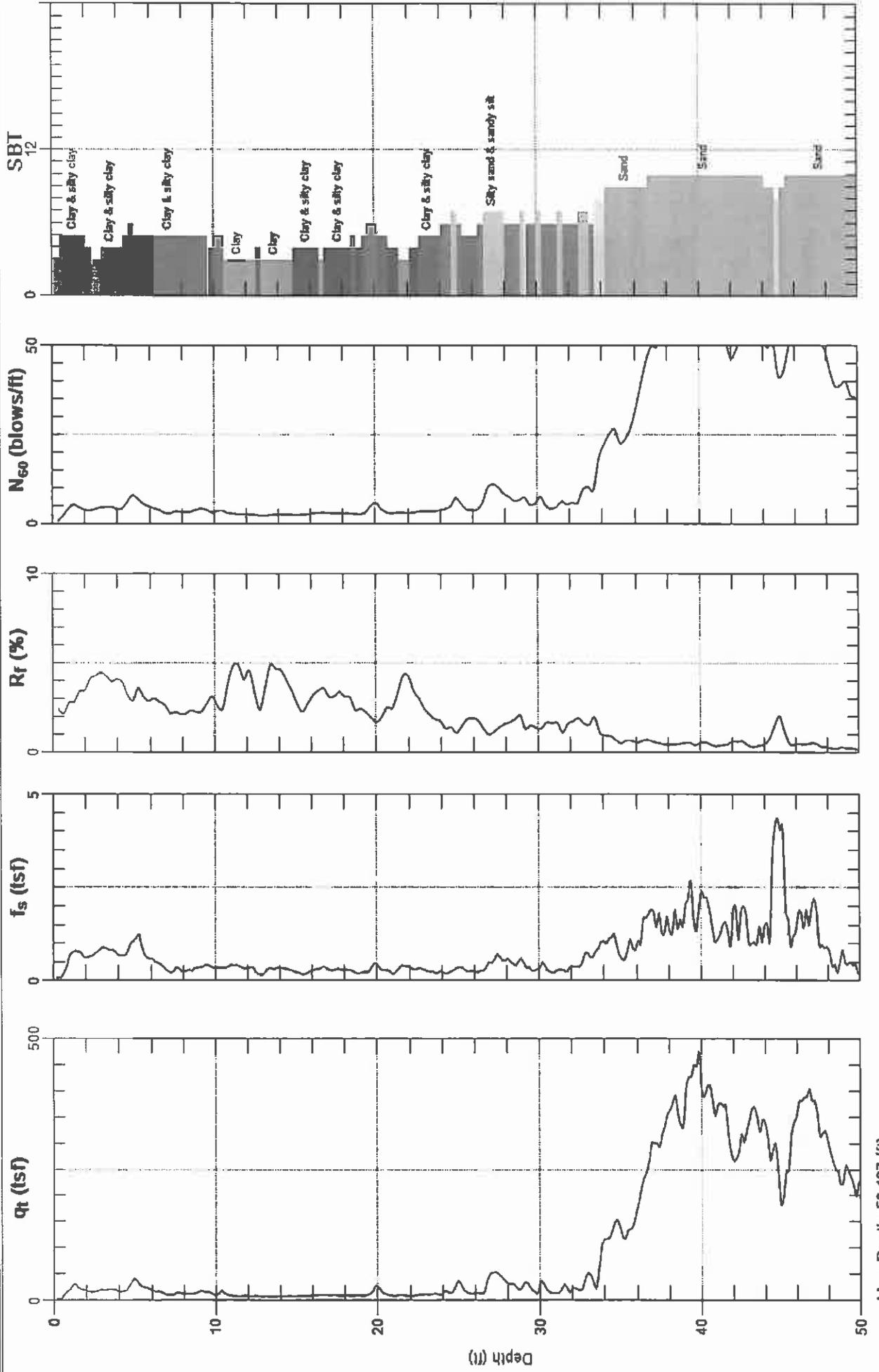
SBT: Soil Behavior Type (Robertson 1990)



# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4  
Sounding: CPT-07

Engineer: J.DYE  
Date: 5/9/2007 12:18



Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



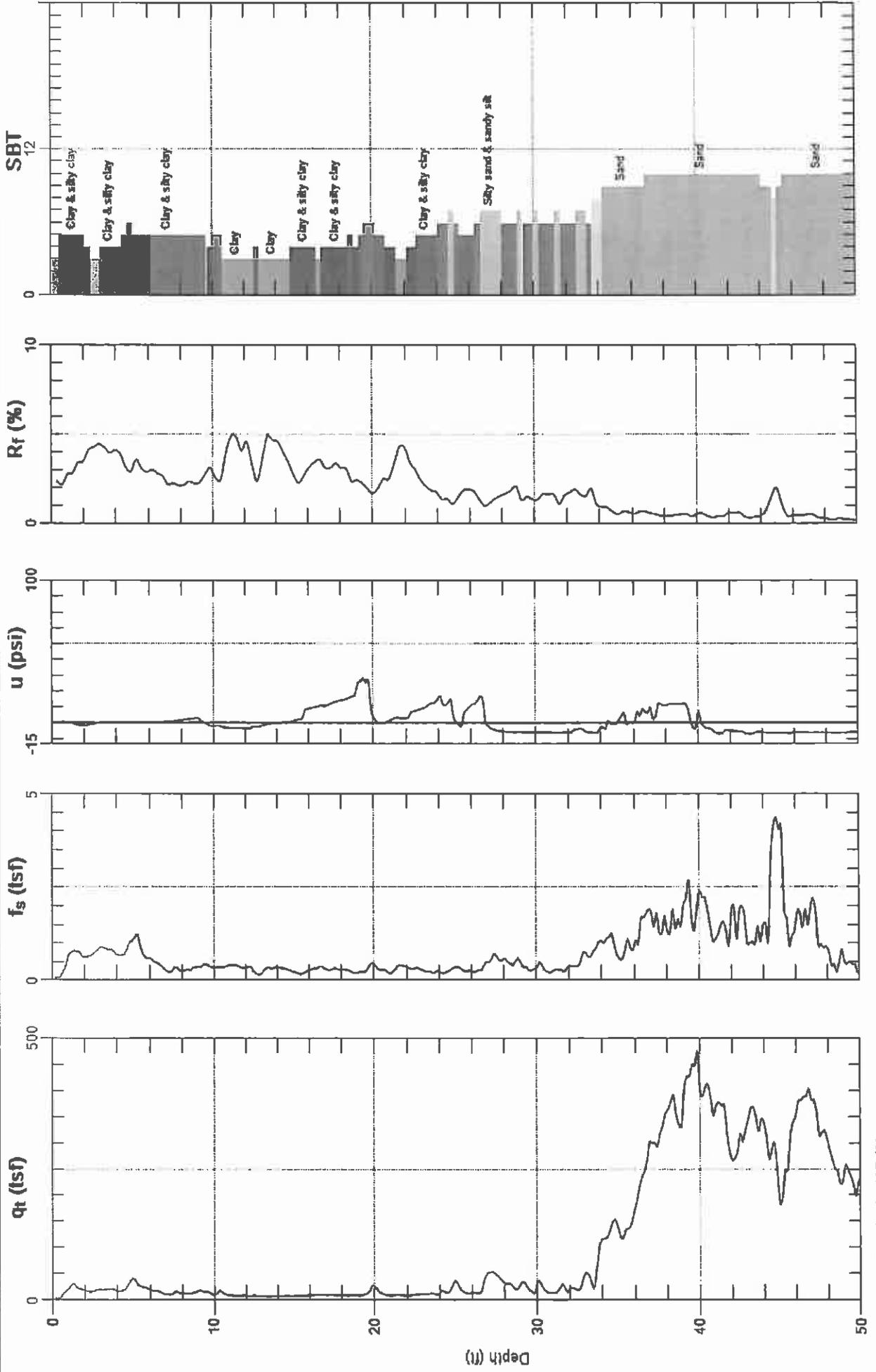
# CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4

Engineer: J.DYE

Sounding: CPT-07

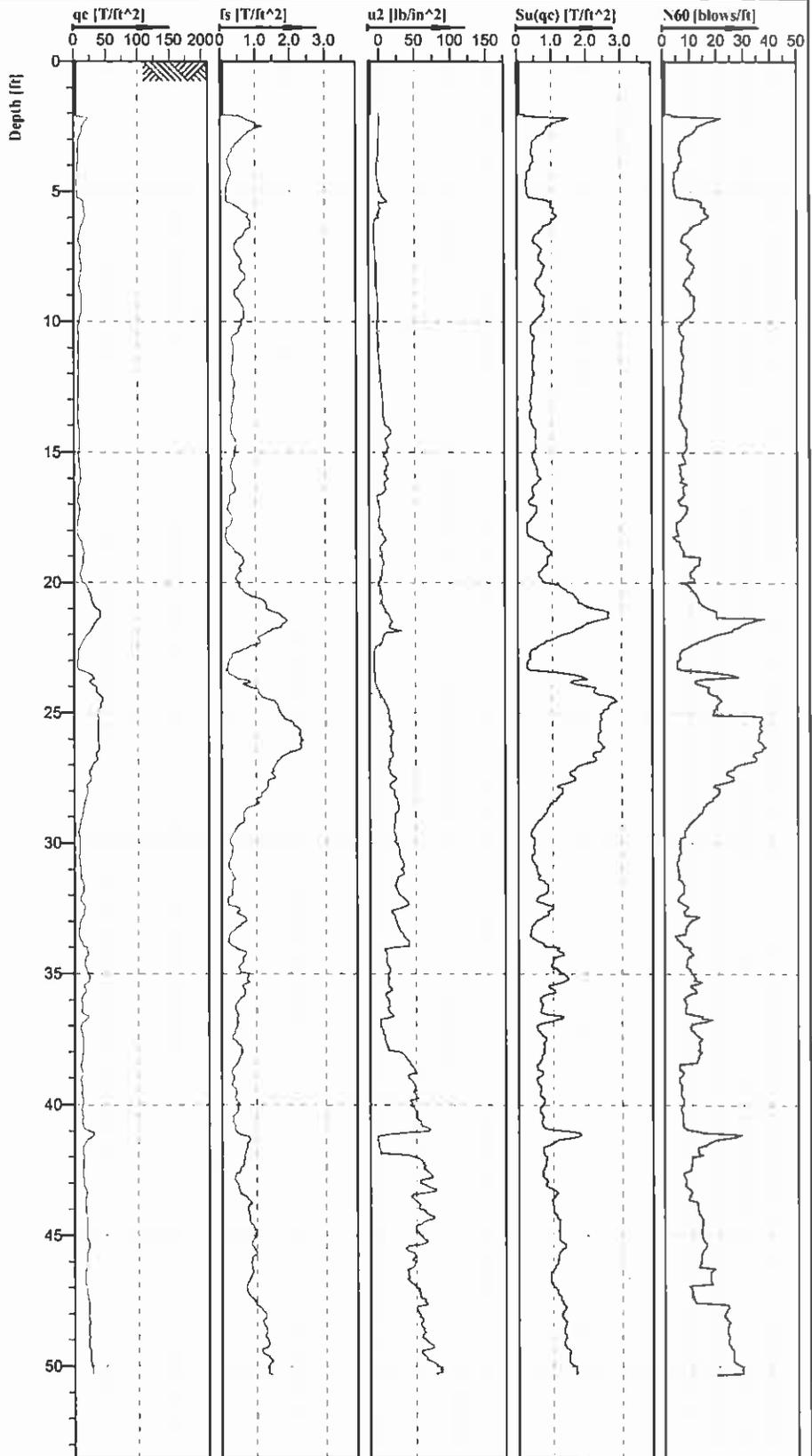
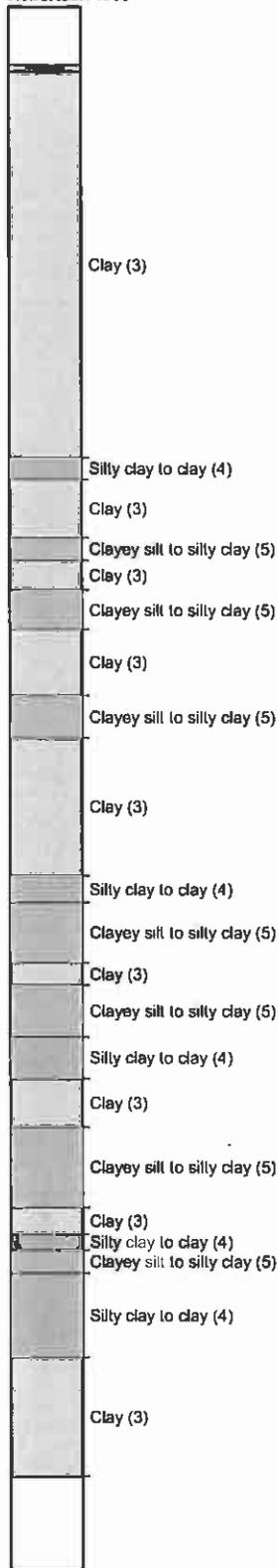
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Max. Depth: 50.197 (ft)  
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

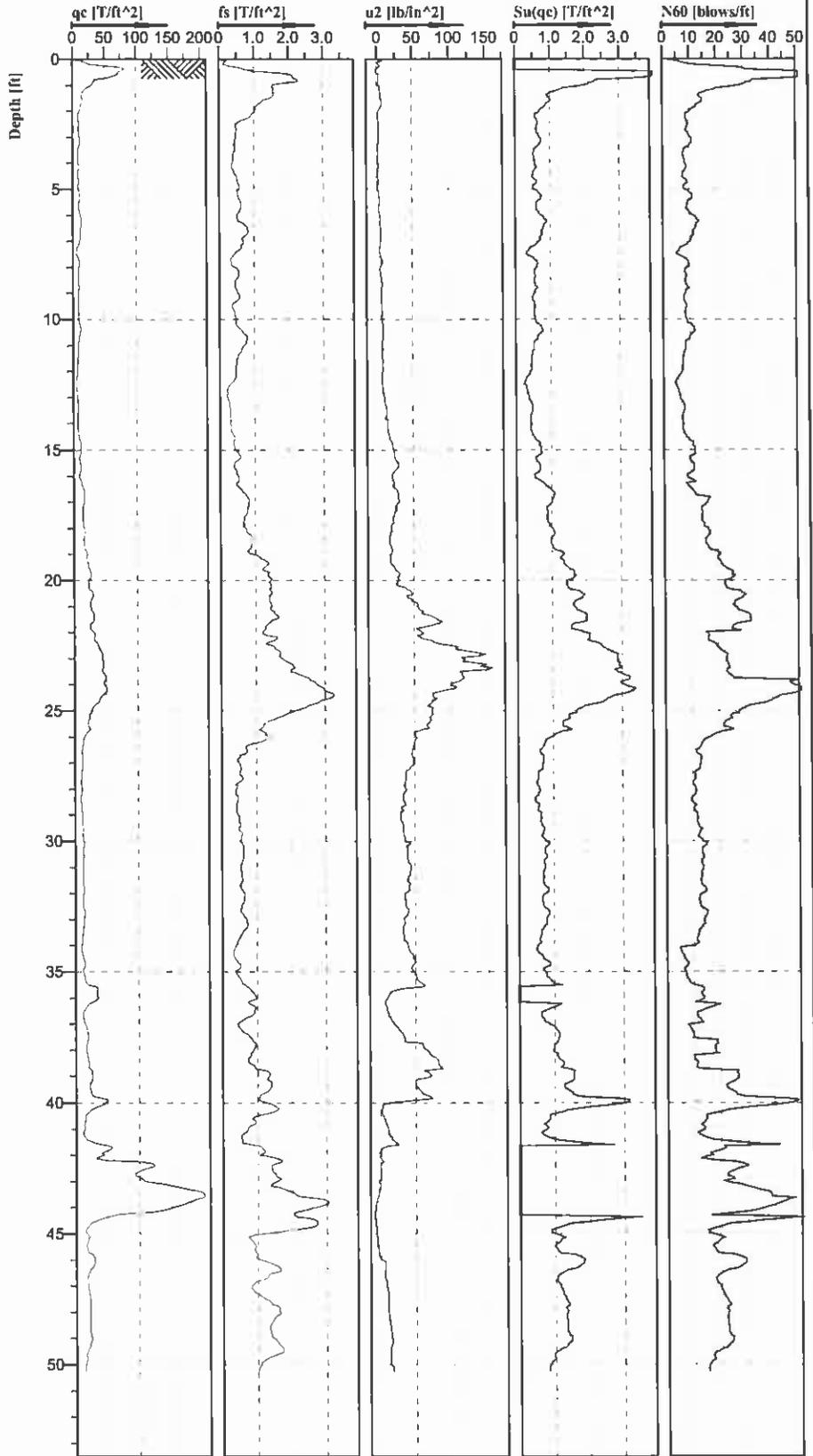
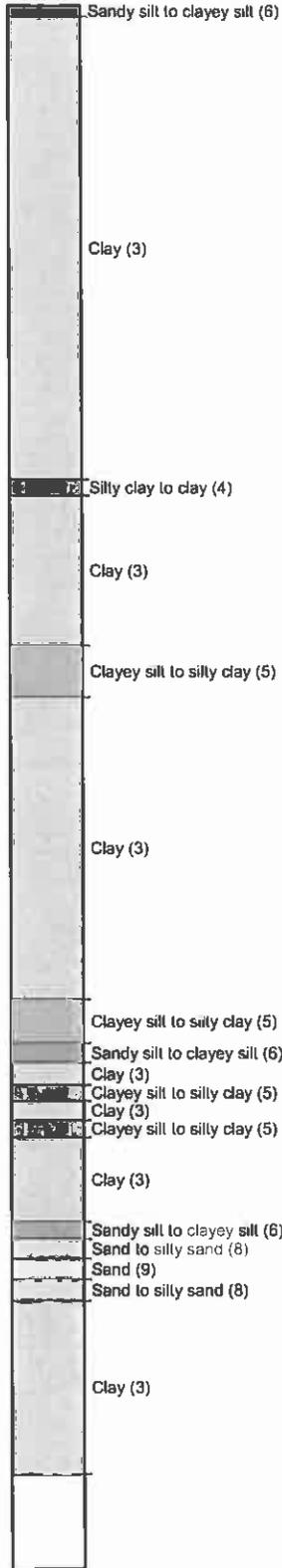
Classification by  
Robertson 1986



Cone No 3794  
Tip area [cm<sup>2</sup>] 10  
Sleeve area [cm<sup>2</sup>] 150

Location: Newark, California	Position:	Ground level:	Test no: CPT-8
Project ID: 102-4-1	Client: Cornerstone Earth Group	Date: 4/24/2008	Scale: 1 : 75
Project: Areas 3 and 4		Page: 1/1	Fig:
File: CPT-8.cpd			

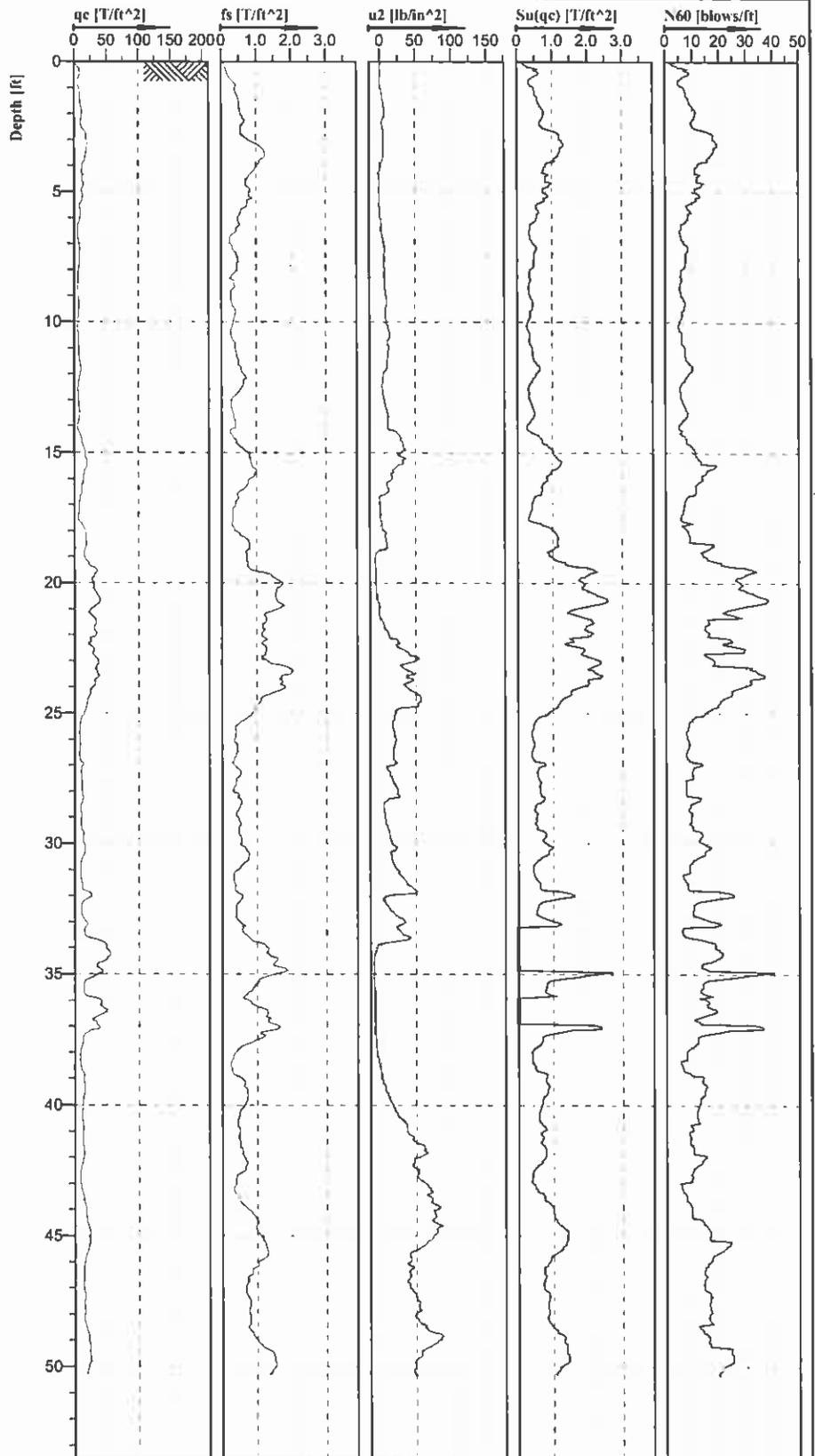
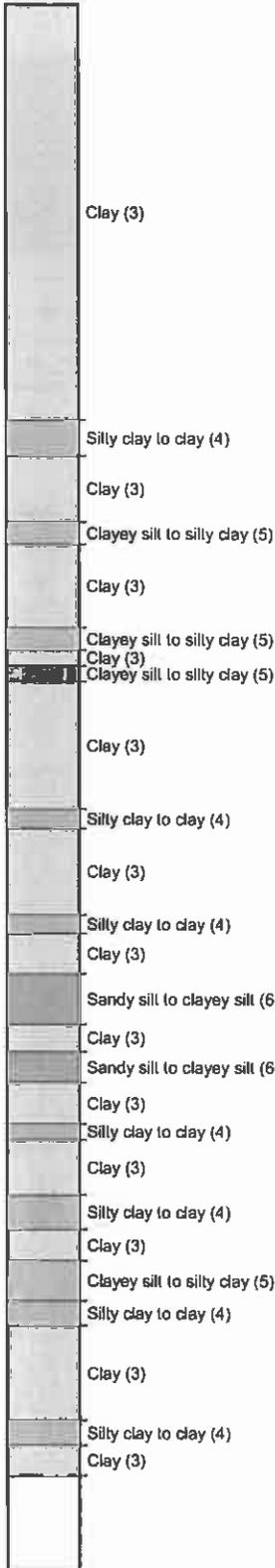
Classification by  
Robertson 1986



  
 Cone No 3794  
 Tip area [cm<sup>2</sup>] 10  
 Sleeve area [cm<sup>2</sup>] 150

Location: Newark, California	Position:	Ground level:	Test no CPT-9
Project ID 102-4-1	Client: Cornerstone Earth Group	Date: 4/24/2008	Scale: 1 : 75
Project: Areas 3 and 4		Page 1/1	Fig:
		File: CPT-9.cpd	

Classification by  
Robertson 1986



  
 Cone No 3794  
 Tip area [cm<sup>2</sup>] 10  
 Sleeve area [cm<sup>2</sup>] 150

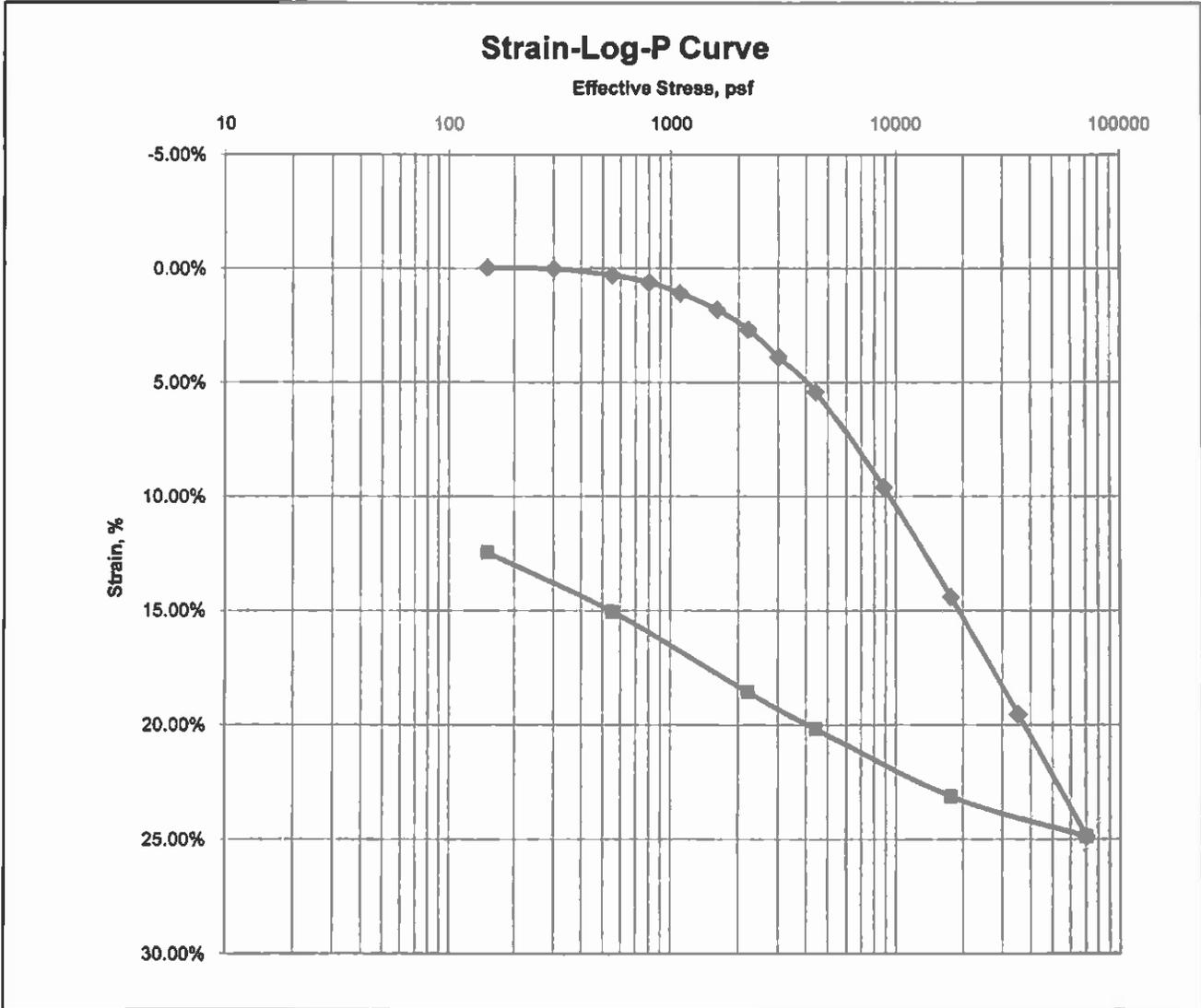
Location: Newark, California	Position:	Ground level:	Test no: CPT-10
Project ID: 102-4-1	Client: Cornerstone Earth Group	Date: 4/24/2008	Scale: 1 : 75
Project: Areas 3 and 4		Page: 1/1	Fig
File: CPT-10.cpd			



# Consolidation Test

## ASTM D2435

Job No.: <u>640-084</u>	Boring: <u>EB-5</u>	Run By: <u>MD</u>
Client: <u>Comerstone Earth Group</u>	Sample: <u>5</u>	Reduced: <u>PJ</u>
Project: <u>Newark Area 3 and 4 - 102-4-1</u>	Depth, ft.: <u>10-12</u>	Checked: <u>PJ/DC</u>
Soil Type: <u>Greenish Gray CLAY w/ Sand</u>	Date: <u>5/16/2008</u>	



Ass. Gs =	2.7	<b>Initial</b>	<b>Final</b>
Moisture %:		39.1	30.9
Dry Density, pcf:		81.4	91.9
Void Ratio:		1.072	0.834
% Saturation:		98.6	100

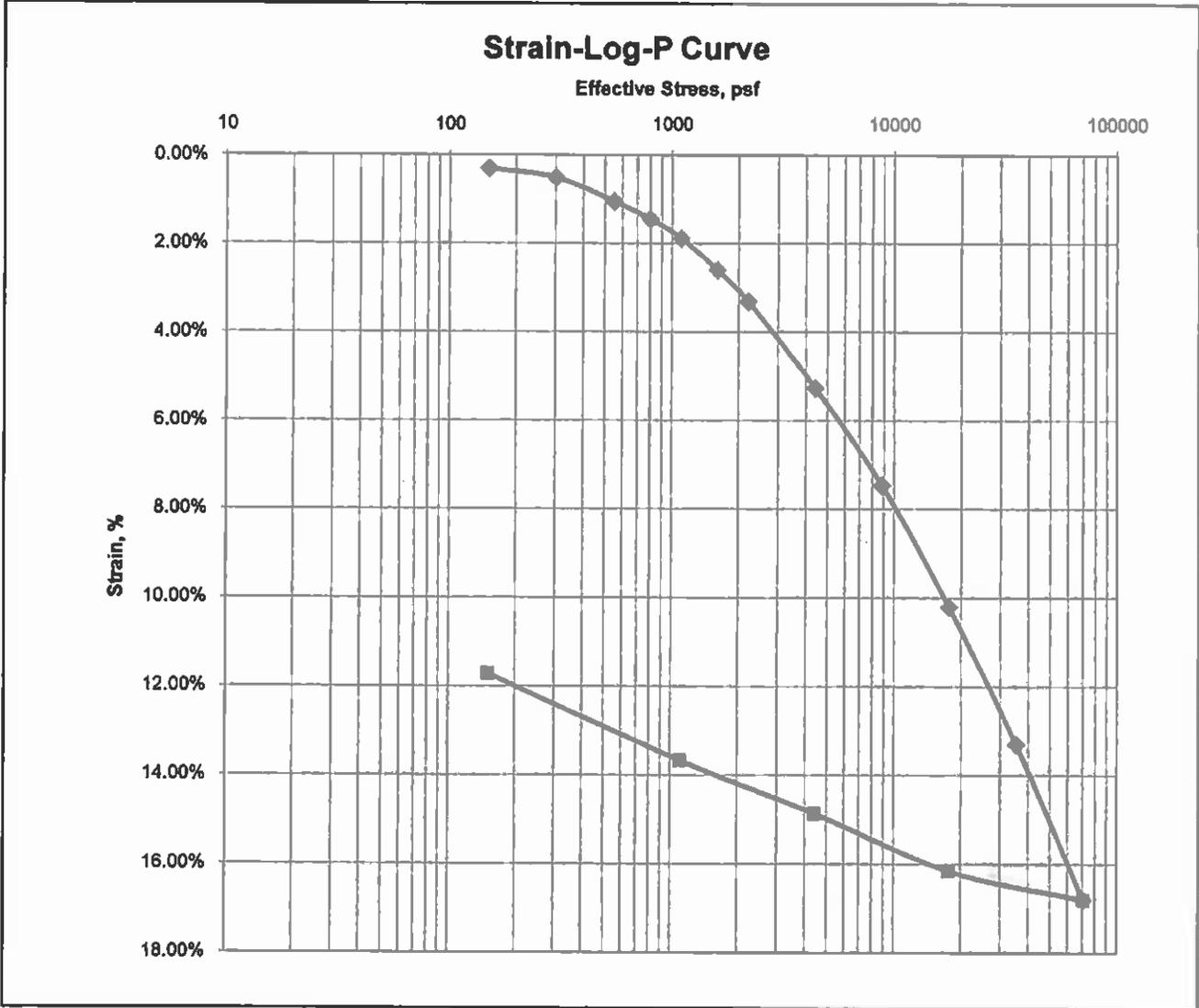
Remarks:



# Consolidation Test

## ASTM D2435

Job No.:	640-084	Boring:	EB-5	Run By:	MD
Client:	Cornerstone Earth Group	Sample:	7	Reduced:	PJ
Project:	Newark Area 3 and 4 - 102-4-1	Depth, ft.:	16	Checked:	PJ/DC
Soil Type:	Greenish Gray SILT	Date:	5/16/2008		



Ass. Gs =	2.7	<b>Initial</b>	<b>Final</b>
Moisture %:		24.6	18.5
Dry Density, pcf:		102.4	112.4
Void Ratio:		0.647	0.499
% Saturation:		102.7	100

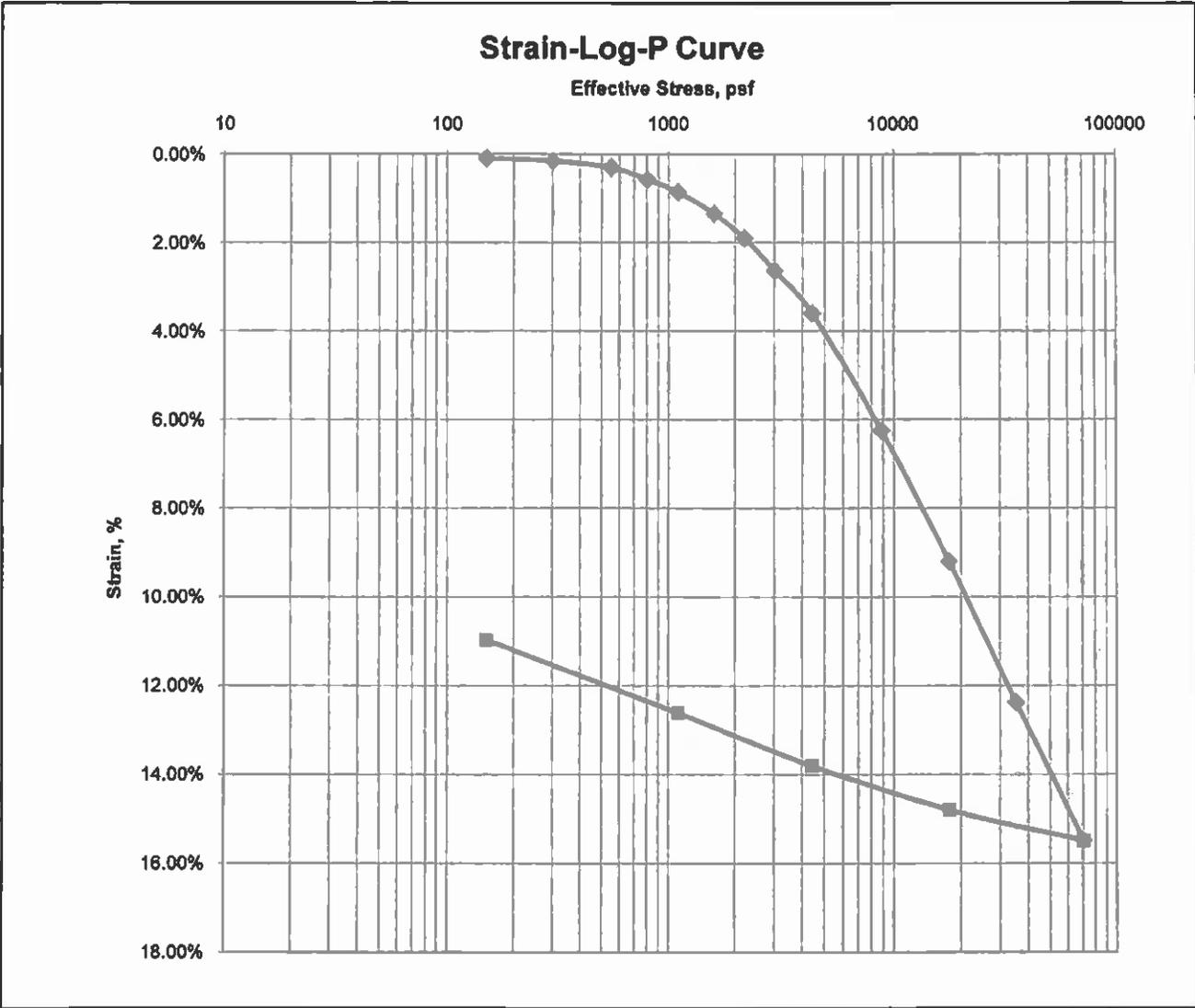
**Remarks:**



# Consolidation Test

## ASTM D2435

Job No.: <u>640-084</u>	Boring: <u>EB-6</u>	Run By: <u>MD</u>
Client: <u>Cornerstone Earth Group</u>	Sample: <u>7</u>	Reduced: <u>PJ</u>
Project: <u>Newark Area 3 and 4 - 102-4-1</u>	Depth, ft.: <u>14-16</u>	Checked: <u>PJ/DC</u>
Soil Type: <u>Greenish Gray CLAY</u>		Date: <u>5/16/2008</u>



Ass. Gs =	2.7	<b>Initial</b>	<b>Final</b>
Moisture %:		22.1	17.2
Dry Density, pcf:		103.4	115.1
Void Ratio:		0.631	0.464
% Saturation:		94.7	100

Remarks: