CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Santana Row Lot 9 Mixed-Use Development
LOCATION	Olsen Drive and Santana Row San Jose, California
CLIENT	Federal Realty Investment Trust
PROJECT NUMBER	371-1-10
DATE	March 1, 2013

CORNERSTONE EARTH GROUP

Type of Services	**Updated Geotechnical Investigation**
Project Name	**Santana Row Lot 9 Mixed-Use Development**
Location	**Olsen Drive and Santana Row** **San Jose, California**
Client	**Federal Realty Investment Trust**
Client Address	**3055 Olin Avenue, Suite 2100** **San Jose, California**
Project Number	**371-1-10**
Date	**March 1, 2013**

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Type of Services	**Updated Geotechnical Investigation**
Project Name	**Santana Row Lot 9 Mixed-Use Development**
Location	**Olsen Drive and Santana Row** **San Jose, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Federal Realty Investment Trust for the Santana Row Lot 9 Mixed-Use Development project in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A proposed development plan titled "Option A-OR-2, Santana Row Design Studies," prepared by BAR Architects, dated October 2, 2012.
- A geotechnical report titled "Geotechnical Investigation, Proposed Mixed-Use Buildings on Lot 9/10, Santana Row, San Jose, California," prepared by Kleinfelder, Inc., dated December 12, 2005. In addition to field and laboratory programs performed by Kleinfelder during its own investigation, the report included the following:
    - Field and laboratory programs by Lowney Associates, dated May 2 and 3, and December 12, 2005.
    - Data from a Cone Penetration Test (CPT-2) performed by Lowney Associates in 1999.

1.1 PROJECT DESCRIPTION

The project will consist of constructing a four-story subterranean parking garage, which will be supporting five to eight stories of commercial space over retail space for the western two-thirds of the site; the eastern third of the garage will be supporting a single-level podium under four to five stories of residential space. The subterranean garage will be of concrete construction, while the commercial/retail and residential spaces will be of steel and wood-frame construction, respectively. It is our understanding that the subterranean garage will encompass the entire site, with various landscaping and flatwork features located near existing grade.

The proposed four-level subterranean parking garage will require an excavation of approximately 45 feet below existing site grades. Based on preliminary information provided by Peoples Associates Structural Engineers (PASE), we understand that maximum interior column loads will be on the order of 1,250 kips.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our email, dated February 12, 2013. Our scope of work consisted of reviewing the field and laboratory programs performed during the previous investigations referenced above, engineering analysis, development of recommendations for site work, grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this updated report. Brief descriptions of the previous exploration and laboratory programs are presented below.

1.3 SUBSURFACE EXPLORATION

We reviewed previous exploratory boring and Cone Penetration Test (CPT) logs performed at the site during previous geotechnical investigations. Previous subsurface explorations in the Lot 9 area include:

- One exploratory boring (KB-1) to a depth of 46 feet by Kleinfelder, Inc. in 2005;
- Four exploratory borings (EB-1, EB-2. EB-3, and EB-4) to depths of 30 to 89½ feet by Lowney Associates in 2005;
- Five CPTs (CPT-6, CPT-7, CPT-8, CPT-9, and CPT-10) to depths of 36½ to 60 feet by Lowney Associates in 2005;
- One CPT (CPT-2) to a depth of 42 feet by Lowney Associates in 1999;

Approximate exploratory boring and CPT locations are shown on the Site Plan, Figure 2. Boring and CPT logs are provided in Appendix A.

1.4 LABORATORY TESTING

Geotechnical laboratory test results from the previous investigations are included in Appendix B. Testing included moisture contents, dry densities, grain size analyses, washed sieve analyses, Plasticity Index tests, triaxial compression tests, a consolidation test, and corrosion (resistivity and pH) testing.

1.5 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Based on the alluvium thickness map of the Santa Clara Valley by Rogers and Williams (1974), the site is underlain by approximately 500 feet of alluvium.

The site is mapped as Holocene alluvial fan deposits (California Geological Survey, 2002; Witter et al, 2006). Holocene alluvial fan deposits are mapped on the broader, more gently sloping portions of the Santa Clara Valley. Fan deposits in this area are dominated by clay and silt, with interbedded lenses of coarser alluvium (sand and occasional gravel).

2.2 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 2007 estimates there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south of San Francisco, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

Faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the approximate distances to the State-considered active faults within 25 kilometers of the site.

A regional fault map is presented as Figure 5, illustrating the distances of the site to significant fault zones.

Table 1: Approximate Fault Distances within 25-Kilometers

Fault Name	Distance (miles)	Distance (kilometers)
Monte Vista-Shannon	4½	7½
San Andreas (1906)	8½	14
Hayward (Southeast Extension)	9	14½
Hayward (Total Length)	11½	18⅓
Calaveras (South)	11½	18⅓
Sargent	12½	20

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

As shown on the Vicinity Map (Figure 1), the existing Santana Row development, including Lot 9, is located south of Stevens Creek Boulevard and east of Winchester Boulevard in San Jose, California. Lot 9 is currently bounded by Olsen Drive to the north, additional at-grade parking lots to the south and east, and a retail/commercial building to the west.

During the previous investigations and up to the present, the approximately 3-acre site was and is currently occupied by an existing at-grade asphalt concrete paved parking lot. The rectangular site is relatively level. Landscaping islands and associated improvements are located throughout the site. Surface pavements generally consisted of 1 to 2 inches of asphalt concrete over 2½ to 4 inches of aggregate base.

3.2 SUBSURFACE CONDITIONS

Based on the previous borings performed, below the surface pavements Lot 9 is generally blanketed by up to 4½ feet of undocumented fill. The fill generally consisted of very stiff to hard sandy lean clay with gravel. Below the fill, the borings and CPTs generally encountered alluvial soils consisting of interbedded medium stiff to very stiff lean clays with various amounts of silt, sand, and gravel; and medium dense to dense sands with various amounts of clay, silt, and gravel to a depth of approximately 35 feet.

Below a depth of 35 feet, all explorations encountered dense to very dense sands and gravels with varying fines content to the maximum depth explored of 89½ feet below existing grades.

Exploratory boring and CPT logs in Appendix A provide more detailed descriptions of the subsurface conditions logged at each exploratory boring and CPT location. Our generalized interpretations of the distribution of subsurface materials are shown in Cross-Sections A-A' and B-B' (Figures 3 and 4, respectively).

3.2.1 Plasticity/Expansion Potential

Plasticity Index (PI) tests performed on samples at depths ranging from 2 to 29½ feet resulted in PIs of 7 to 18, indicating low to moderate expansion potential to wetting and drying cycles.

3.3 GROUND WATER

Published data (CGS, San Jose West Quadrangle, 2002) indicates that seasonal and/or historical high ground water levels in the vicinity of the site are at a depth of approximately 50 feet below the ground surface. Ground water was encountered in the exploratory borings (EB-1, EB-3, and EB-4) at depths ranging from approximately 45 to 49 feet below current site grades.

Free ground water was not encountered in boring KB-1 due to rotary wash circulation.

Free ground water was not encountered in our boring for Lot 11, to the west. Ground water was not observed in previous borings for the Lot 11 investigations (Lowney, 2005, Kleinfelder, 2006) due to the rotary wash drilling technique.

Ground water was encountered in previous borings performed for Lot 8B (CEG, 2010) and Lot 6B (CEG, 2011) at depths ranging from approximately 48½ to 50½ feet below the site grades.

Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, perched water, regional variations, and other factors. A discussion of design ground water levels and impacts on site development are presented in Section 5.1.2.

3.4 PRELIMINARY SOIL CORROSION SCREENING

Lowney Associates (2005) tested four samples from depths of 3½ to 13½ feet for resistivity and pH. The laboratory test results are summarized in Table 2.

Table 2: Summary of Corrosion Test Results

Sample/Test Location Number	Depth (feet)	Soil pH	Minimum Resistivity (1) (ohm-cm)
EB-1	3½	7.3	2,623
EB-2	5½	7.5	1,586
EB-3	13½	7.9	3,744
EB-4	9	7.8	1,243

Notes: (1) Laboratory resistivity measured at 100% saturation

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

Based on the resistivity test results, the on-site soils can be characterized as moderately to severely corrosive to buried metal.

Testing for soluble sulfate contents was not performed on samples from Lot 9. We have performed soluble sulfate tests during our investigations for Lot 6B (2010), Lot 8B (2011), and Lot 11 (2012). Soluble sulfate levels on those three sites were measured to be less than 0.01 percent by weight, indicating that there is not a significant corrosion potential to buried concrete improvements. Cementitious materials for sulfate exposure shall be in accordance with 2010 CBC Section 1904.5 and ACI 318-08, Tables 4.2.1 and 4.3.1.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed in Section 2, six significant faults are located within 25 kilometers of the site. The site is not located within a currently designated California Alquist-Priolo Earthquake Fault Zone, formerly known as a Special Studies Zone (CDMG, 1982), a City of San Jose Fault Hazard Zone (CSJ, 1983), or a Santa Clara County Fault Rupture Hazard Zone (SCC, 2012). No known surface expression of fault traces is known 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 earthquakes can cause strong ground shaking at the site. The California Geological Survey (CGS, 2008) recommends evaluating geologic hazards, such as soil liquefaction, for ground motions having a 10 percent probability of exceedance in 50 years. Based on the USGS *National Seismic Hazard Mapping Project*, the peak ground acceleration (PGA) at the site corresponding to 10% probability of exceedance in 50 years is 0.49g for stiff alluvial soil conditions (Peterson, et al., 2008).

The PGA was also estimated using a value equal to $S_{DS}/2.5$ as allowed in the California Building Code, resulting in a PGA of 0.40g. Seismic criteria for structural design are presented in Section 7.1.

4.3 LIQUEFACTION POTENTIAL

The site is not located within a State-designated Liquefaction Hazard Zone (CGS, San Jose West Quadrangle, 2002) or a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2012).

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 4 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.Our analysis followed the procedures in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) and in accordance with CGS Special Publication 117A guidelines (CGS, 2008) for quantitative analysis. As discussed in the “Subsurface” section above, the soils encountered below the design ground water depth of 45 feet consistest of very stiff cohesive soils and very dense granular soils. Our screening analysis indicates that the potential for liquefaction triggering at the site is low.

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.

There are no open faces within 200 feet of the site where lateral spreading could occur and no weak or potentially-liquefiable soil zones; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone D, an area of undetermined, but possible flood hazard. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if 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 Lexington Reservoir.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Effect of basement excavation on adjacent structures and improvements
- Ground water depth
- Presence of undocumented fill
- Differential movement at on-grade to on-structure transitions
- Soil corrosion potential

5.1.1 Effect of Basement Excavation on Adjacent Structures and Improvements

The planned 45-foot deep basement excavation will be constructed directly adjacent to existing at-grade structures. We recommend that the shoring system be designed to limit deflections and disruptions to adjacent structures and nearby improvements. To minimize the potential impact to adjacent structures, the shoring system for the garage excavation should be designed

to limit deflections at the ground surface to less than ½ inch. If limiting deflections to less than ½ inch is not feasible, underpinning of adjacent structures or improvements may be required.

Potential temporary shoring methods include tiebacks and braced shoring (if feasible). Additional recommendations including permanent and temporary shoring design parameters are provided in subsequent sections of this report.

We recommend that a monitoring program be developed to evaluate the effects of the excavation and shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. The monitoring frequency should be established and agreed to by the project team prior to the start of excavation and shoring construction. We also recommend performing a pre-construction photographic survey to document any existing distress to the neighboring buildings.

5.1.2 Ground Water Depth

As discussed in Section 3.3, ground water has been encountered in borings at depths of 45 to 60 feet below the ground surface across the Santana Row developments during the current and previous investigations.

In order to account for seasonal variations in the ground water level and regional rise in the ground water table during the life of the structure, we recommend designing the structure for long-term ground water levels of greater than those encountered at the site.

We recommend extending the wall drainage system to a depth of 42 feet below existing grades, and designing the floor slabs and the portions of the walls below a depth of 42 feet to resist hydrostatic pressure. Hydrostatic loading on the walls and floor slab are discussed in Sections 7.4 and 10.1, respectively.

At the owner's option, the wall drainage system, which will include a sump pump, could be lowered to decrease the hydrostatic load on the walls and floor slab. Lowering the drainage system would increase the potential for seasonal or full-time pumping during the life of the project. Additionally, if the wall drainage and sump is lowered to the floor slab elevation and the floor is not designed for hydrostatic uplift, an underslab drainage system would need to be installed, as discussed in Section 8.1.1.

Free ground water and/or wet soils may be encountered during construction in the garage excavation. If ground water or wet soils are encountered, localized de-watering or subgrade stabilization may be required. Subgrade stabilization recommendations are presented in Section 6.6.

5.1.3 Presence of Undocumented Fill

As discussed in the "Subsurface" section, the previous investigations performed indicated the site was blanketed by up to 4½ feet of fill. Based on the planned excavation depth, all undocumented fill will be removed from within the planned building footprints. Undocumented fill outside the building footprint, which is left in place, may pose a risk to surface improvements, such as sidewalks and at-grade pavements. Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 inches of fill below pavement subgrade is reworked and compacted. Detail recommendations are included in the "Earthwork" section of our report.

5.1.4 Differential Movement at On-Grade to On-Structure Transitions

Some surface improvements may transition from on-grade support to overlying the basement/parking structure. Where the depth of soil cover overlying the basement/parking structure roof is thin or where basement/parking structure walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the walls. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or pavers that can cantilever at least 3 feet beyond the walls. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see "Retaining Wall" section).

5.1.5 Soil Corrosion Potential

As discussed in Section 3.4, the preliminary soil corrosion test results indicated that the on-site soils are moderately to severely corrosive to buried metallic improvements, such as metal pipes and tie-back anchors. We recommend that a corrosion engineer be retained to review the corrosion test results and develop specific corrosion protection design recommendations. If permanent tie-back anchors will be used, we recommend that Class I corrosion protection is required for all permanent anchors (PTI, 2004).

The on-site soils were determined to have a negligible corrosion potential to Portland Cement Concrete. Consideration may be given to collecting and testing additional samples for sulfates and pH to confirm the classification of corrosive to mortar coated steel and concrete.

5.2 PLANS AND SPECIFICATIONS REVIEW

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

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION, CLEARING AND PREPARATION

6.1.1 Site Stripping

The proposed building areas should be stripped of all surface vegetation, and any existing surface and subsurface improvements. Demolition of existing improvements is discussed in detail below. Based on our observations, the majority of the site is currently covered by asphalt concrete pavement. In addition, excavations on the order of 45 feet are planned for the below-grade garage. Where applicable, surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight and removed from the site.

6.1.2 Tree and Shrub Removal

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

6.1.3 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas. A discussion of recycling existing improvements is provided later in this report.

6.1.4 Abandonment of Existing Utilities

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

6.2 REMOVAL OF EXISTING FILLS

As discussed earlier, virtually all the fill that blankets the site will be removed during the mass parking garage excavation. Any undocumented fill encountered, however, should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below.

6.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 45 feet at the site may be classified as OSHA Site C materials. A Cornerstone representative should be retained to confirm the preliminary site classification. Recommended soil parameters for temporary shoring are provided in the “Temporary Shoring” section of this report.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1:1 inclination unless the OSHA soil classification indicates differently.

6.4 BELOW-GRADE EXCAVATIONS

Temporary shoring may support the planned cuts up to 45 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.4.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, braced excavations, soil nailing, or potentially other methods. Restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 3: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Restrained Wall – Uniform Earth Pressure	25H*
Passive Pressure	350 pcf up to 2,000 psf maximum uniform pressure

* H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

The restrained earth pressure may also be distributed as described in Figure 24 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the “Wall Drainage” section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

The borings were performed with hollow-stem auger and mud-rotary wash drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, tie-back, or soil nail installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below ground water) or difficult drilling or cobble conditions were encountered during the explorations, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. For multi-level excavations, the installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agreed to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

6.4.2 Construction Dewatering

Ground water levels are expected to be at or above the bottom of the below-grade garage excavation; therefore temporary dewatering will be necessary during construction and possibly during the design life of the structure. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

The dewatering design should maintain ground water at least 5 feet below the bottom of the mass excavation, and at least 2 feet below localized excavations such as deepened footings, elevator shafts, and utilities. If the dewatering system was to shut down for an extended period of time, destabilization and/or heave of the excavation bottom requiring over-excavation and stabilization, flooding and softening, and/or shoring failures could occur; therefore, we recommend that a backup power source be considered.

Depending on the ground water quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm or sanitary, or trucked to an off-site facility.

6.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive

additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

Due to the sandy soils likely to be encountered at the subgrade elevation, we recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

6.6 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.6.3 Below-Grade Excavation Stabilization

As the planned basement excavation will extend near/below the current ground water level, we recommend that the contractor plan to excavate an additional 12 to 18 inches below subgrade, place a layer of stabilization fabric (Mirafi 500X, or equivalent) at the bottom, and backfill with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric.

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

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

6.7.2 Re-Use of On-Site Site Improvements

We anticipate that significant quantities of asphalt concrete (AC) grindings and aggregate base (AB) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections. AC/AB grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications. If existing alligator cracking of the AC pavements is present, it is likely that the grinding operation will leave significant oversize chunks and won't meet the Class 2 AB gradation requirements but may meet Caltrans subbase requirements. Depending on the quantities of oversized material, the grindings may still be used within the structural section; however, the pavement design will need to be modified to account for the difference, typically resulting in the addition of about 1 inch to the structural section.

6.8 COMPACTION REQUIREMENTS

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

Table 4: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base3	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base3	95	Optimum
Asphalt Concrete	Asphalt Concrete	95	NA

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

6.9 TRENCH BACKFILL

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

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

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

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

6.10 SITE DRAINAGE

Surface water should not be allowed to collect or pond adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope away from foundations at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent. Roof runoff should be directed at least 5 feet away from building areas in landscape areas.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

In our opinion, we recommend the structure, including the below-grade parking garage, be supported on spread footings or a mat foundation provided the recommendations in the "Earthwork" section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2010 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot. Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Earthquake Ground Motion Parameters*, Version 5.1.0, revision date February 10, 2011, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 5: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.31883°
Site Longitude	-121.94750°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.506g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.506g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.900g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.004g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.600g

[1]For Site Class B, 5 percent damped.

7.3 SPREAD FOOTINGS

The proposed building may be supported on conventional isolated and/or continuous spread footings bearing entirely on undisturbed soil, or entirely on a uniform section of engineered fill. Footings should be at least 24 inches wide and extend at least 24 inches below lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following:

- bottom of the adjacent interior slab-on-grade, or
- finished exterior grade, excluding landscaping topsoil.

Spread footings bearing entirely on undisturbed soil or engineered fill and constructed in accordance with above recommendations would be capable of supporting a maximum allowable bearing pressure of 3,000 psf for dead loads, 4,500 psf for combined dead plus live loads, and 6,000 psf for all loads including wind and seismic. The above pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected.

Based on a maximum interior column load provided by Peoples Associates Structural Engineers of approximately 1,250 kips (dead plus live loads), we estimate footings will undergo total settlements of approximately 1½ inches with post-construction differential settlement of approximately ¾-inch between adjacent foundation elements under static loads.

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing and mat dead loads, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the above ultimate values.

7.3.1 Spread Footing Construction Considerations

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

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

7.4 REINFORCED CONCRETE MAT FOUNDATIONS

As an alternative to spread footings with a conventional slab-on-grade floor, the structures could be supported on a mat foundation bearing on native soil prepared in accordance with the "Earthwork" section of this report, and designed in accordance with the recommendations below.

7.4.1 Mat Foundation Bearing Pressures

The parking garage mat may be designed for an average allowable bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads, with maximum localized allowable bearing pressures of 4,500 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes. All mats should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the mat foundation pads prior to placement of reinforcing steel.

If desired to minimize floor moisture in moisture-sensitive areas, we recommend that a moisture barrier system be constructed beneath the mat area. Guidelines are provided below in the

"Moisture Protection Considerations" section of this report. It appears the mat will extend below the design ground water level of 42 feet as discussed in Section 5.1.2; therefore, consideration should be given to waterproofing the mat. Hydrostatic uplift and waterproofing recommendations are presented in the “Hydrostatic Uplift and Waterproofing” section of this report.

7.4.2 Mat Foundation Settlement

Based on the maximum dead-plus-live column loading of 1,250 kips provided by Peoples Associates Structural Engineers and the allowable bearing pressures presented above, we estimate that the total static mat settlement is estimated to range from about ½-inch to ¾-inch, with post-construction static differential settlement of less than ½-inch over a distance of about 30 feet.

7.4.3 Mat Modulus of Soil Subgrade Reaction

The modulus value of soil subgrade reaction is a model element that represents the response to a specific loading condition, including the magnitude, rate, and shape of loading, given the subsurface conditions at that location. Based on assumed loads and contact pressures our analyses indicate a preliminary modulus of subgrade reaction of 30 pounds per cubic inch to be appropriate for initial mat design. Once soil foundation pressures are finalized, we can provide revised soil subgrade modulus reaction figures.

7.4.4 Mat Foundation Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above.

7.4.5 Mat Foundation Construction Considerations

The mat will be constructed near the current ground water level and even if temporary dewatering is included, the soil above the water table will be at near saturated conditions. Subgrade stabilization may be required as discussed in the “Earthwork” section above.

7.5 HYDROSTATIC UPLIFT AND WATERPROOFING

Because of the proximity of the ground water table to the proposed basement floor, the mat foundation should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design ground water should be waterproofed and designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design ground water level, a drainage system may be added as discussed in the “Retaining Wall” section, if desired; otherwise the walls should be designed as undrained for the full height.

In addition, the portions of the structures extending below design ground water should be waterproofed to limit moisture infiltration, including mat foundation/thickened slab areas, all construction joints, and any retaining walls. We recommend that a waterproofing specialist design the waterproofing system.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 PARKING STRUCTURE SLABS-ON-GRADE

Garage slabs-on-grade should be at least 5 inches thick and if constructed with minimal reinforcement intended for shrinkage control only, should have a minimum compressive strength of 3,000 psi. If the slab will have heavier reinforcing because the slab will also serve as a structural diaphragm, the minimum compressive strength may be reduced to 2,500 psi at the structural engineer's discretion. The garage slab should also be supported on at least 4 inches of Class 2 aggregate base compacted in accordance with the "Compaction" section of this report.

If there will be areas within the garage that are moisture sensitive, such as equipment and elevator rooms, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness for unreinforced slabs, and at greater spacing for reinforced slabs.

8.1.1 Underslab Drainage

In addition to the perimeter building retaining wall subdrain, and if it is planned to drain all walls and floors, a network of subdrain trenches embedded in a layer of gravel beneath the underlying capillary break also can be used for water drainage. Trenches should be a minimum of 8-inches-square and should consist of a 4-inch minimum diameter perforated pipe surrounded with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. Alternatively, ½-inch to ¾-inch diameter crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. Trenches should be spaced approximately 20 to 25 feet apart. The trench/rock may need to be thickened to allow the pipe to gravity feed to the drainage pump system.

As an alternative to subdrain trenches, TenCate Mirafi G200N drainage composite, Contech C-100 strip-drain (filter geotextile bonded to both sides), or an approved equivalent drainage system also can be used. The drainage system should be placed directly beneath the capillary break and spaced approximately 15 to 20 feet apart. The drainage system (drainage panels) should be connected to a drainage pump system.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor.

These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

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

- Polishing the concrete surface with metal trowels is not recommended.

- Where floor coverings are planned, all concrete surfaces should be properly cured.

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

8.3 EXTERIOR FLATWORK

8.3.1 Pedestrian Concrete Flatwork

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

8.3.2 Pedestrian Pavers

Concrete unit pavers subject to pedestrian and/or occasional light auto loading should be supported on at least 4 inches of Caltrans Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Pavers that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 10. The design R-value was chosen based on engineering judgment considering the variable surface conditions.

Table 6: Asphalt Concrete Pavement Recommendations, Design R-value = 10

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.0	9.5
4.5	2.5	8.5	11.0
5.0	3.0	9.0	12.0
5.5	3.0	11.0	14.0
6.0	3.5	11.5	15.0
6.5	4.0	12.5	16.5

* Caltrans Class 2 aggregate base; minimum R-value of 78

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

9.2 PORTLAND CEMENT CONCRETE

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

Table 7: PCC Pavement Recommendations, Design R-value = 10

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

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

9.3 VEHICULAR CONCRETE UNIT PAVERS

Where vehicular concrete unit pavers are desired in standard traffic areas, we recommend that the pavers be underlain by a 6-inch-thick concrete sub-slab designed as discussed above, including the aggregate base section. Pavers should be placed on a bituminous or mortar setting bed over the concrete sub-slab. Where the pavers will be used as emergency vehicle access (EVA), the pavers should be placed over at least 12 inches of Class 2 aggregate base and prepared subgrade as recommended in the "Earthwork" section. A maximum 1 inch thick sand setting bed may be used to level the pavers on the aggregate base.

SECTION 10: BASEMENT AND RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any at-grade site retaining walls should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Basement walls should be designed as restrained walls. If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired. If walls are not drained, we recommend waterproofing the walls below the design ground water level of 45 feet below the existing ground surface and designing walls and slabs to resist hydrostatic pressures on walls and slabs.

Table 8: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2010 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We reviewed the seismic earth pressures for the proposed basement using procedures generally based on the Mononobe-Okabe method. The seismic increment when added to the recommended active earth pressure are less than the recommended restrained earth pressures. Based on current recommendations for seismic earth pressures (Lew, et al., 2010), an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above.

10.3 WALL DRAINAGE

10.3.1 At-Grade Site Walls

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated

pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

10.3.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and closed or through-wall system such as the TotalDrain system from AmerDrain.

Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path. In addition, where drainage panels will connect from a horizontal application for plaza areas to vertical basement wall drainage panels, the drainage path must be maintained. We are not aware of manufactured corner protection suitable for this situation; therefore, we recommend that a section of crushed rock be placed at the transitions. The crushed rock should be at least 3 inches thick, extend at least 12 inches horizontally over the top of the basement roof and 12 inches down from the top of the basement wall, and have a layer of filter fabric covering the crushed rock.

Drainage panels should terminate 18 to 24 inches from final exterior grade unless capped by hardscape. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. If the shoring system will be offset behind the back of permanent wall, the drainage systems discussed in the "At-Grade Site Walls" section may also be used.

10.4 BACKFILL

Backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

10.5 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Federal Realty Investment Trust specifically to support the design of the Santana Row Lot 9 Mixed-Use Development project in San Jose, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

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

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

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

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

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

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

SECTION 12: REFERENCES

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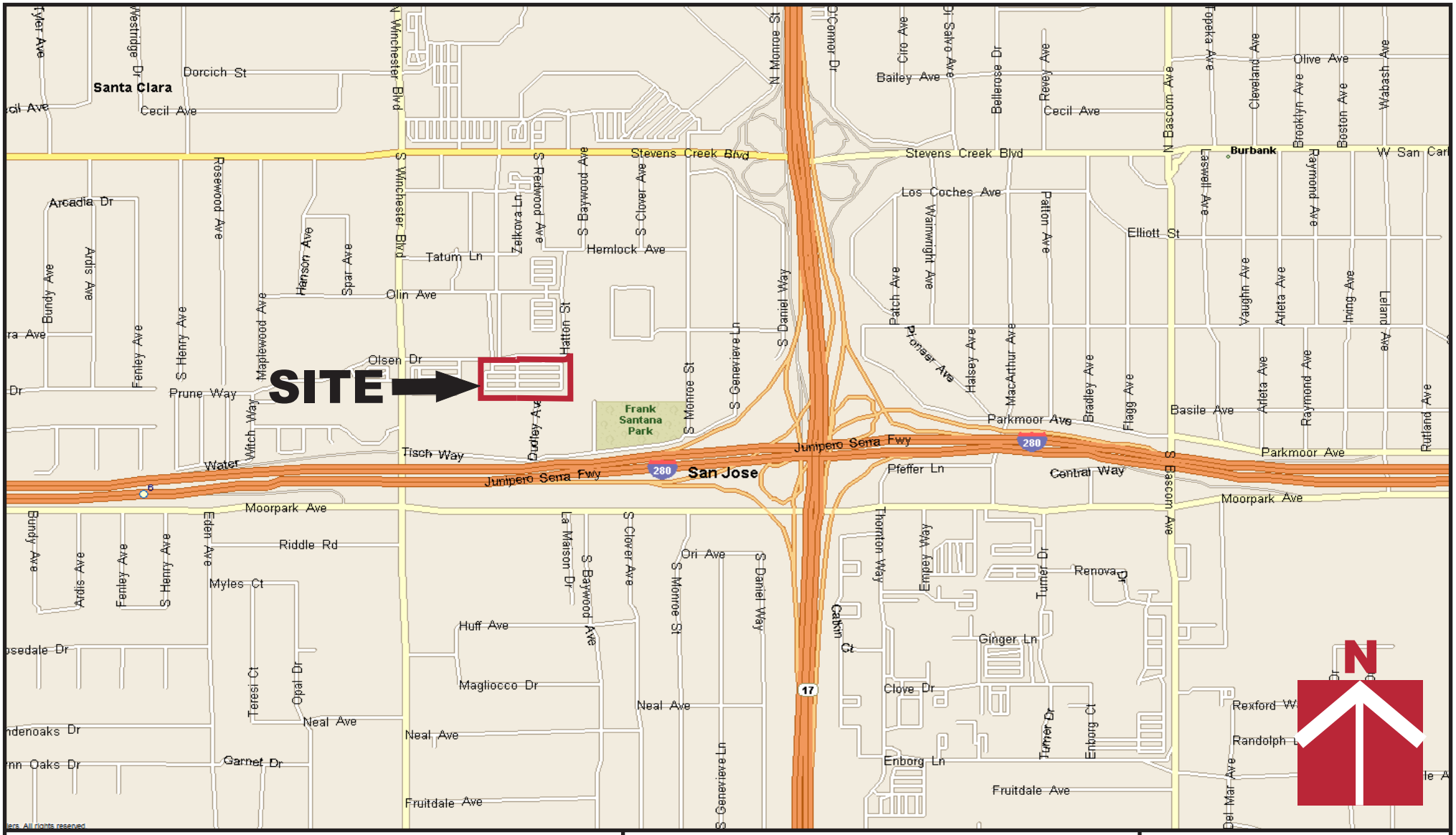
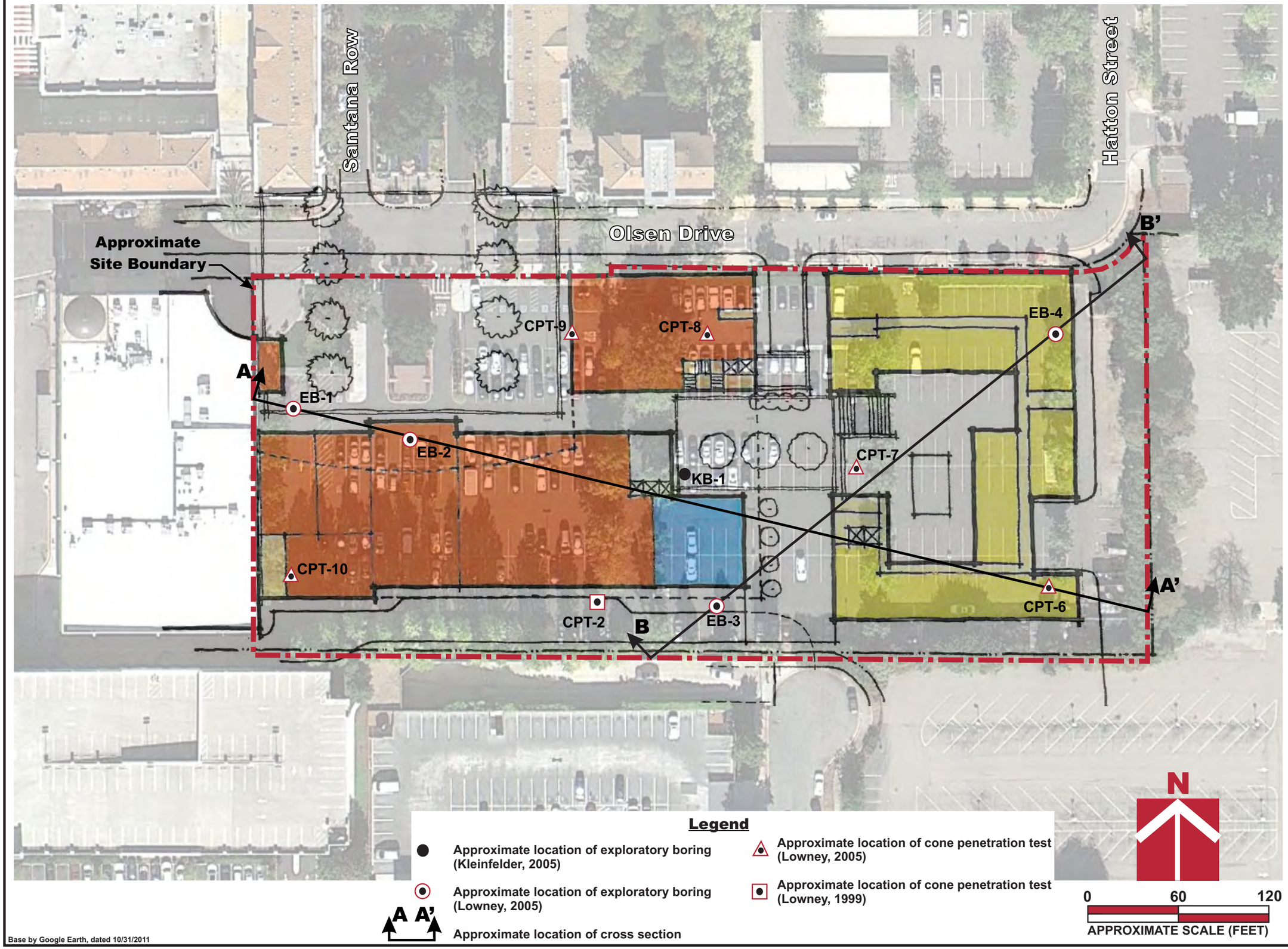


Vicinity Map

Santana Row Lot 9 Mixed-Use Development
San Jose, CA

CORNERSTONE EARTH GROUP

Project Number	371-1-10
Figure Number	Figure 1
Date	February 2013
Drawn By	RRN



Project Number 371-1-10

Figure Number Figure 2

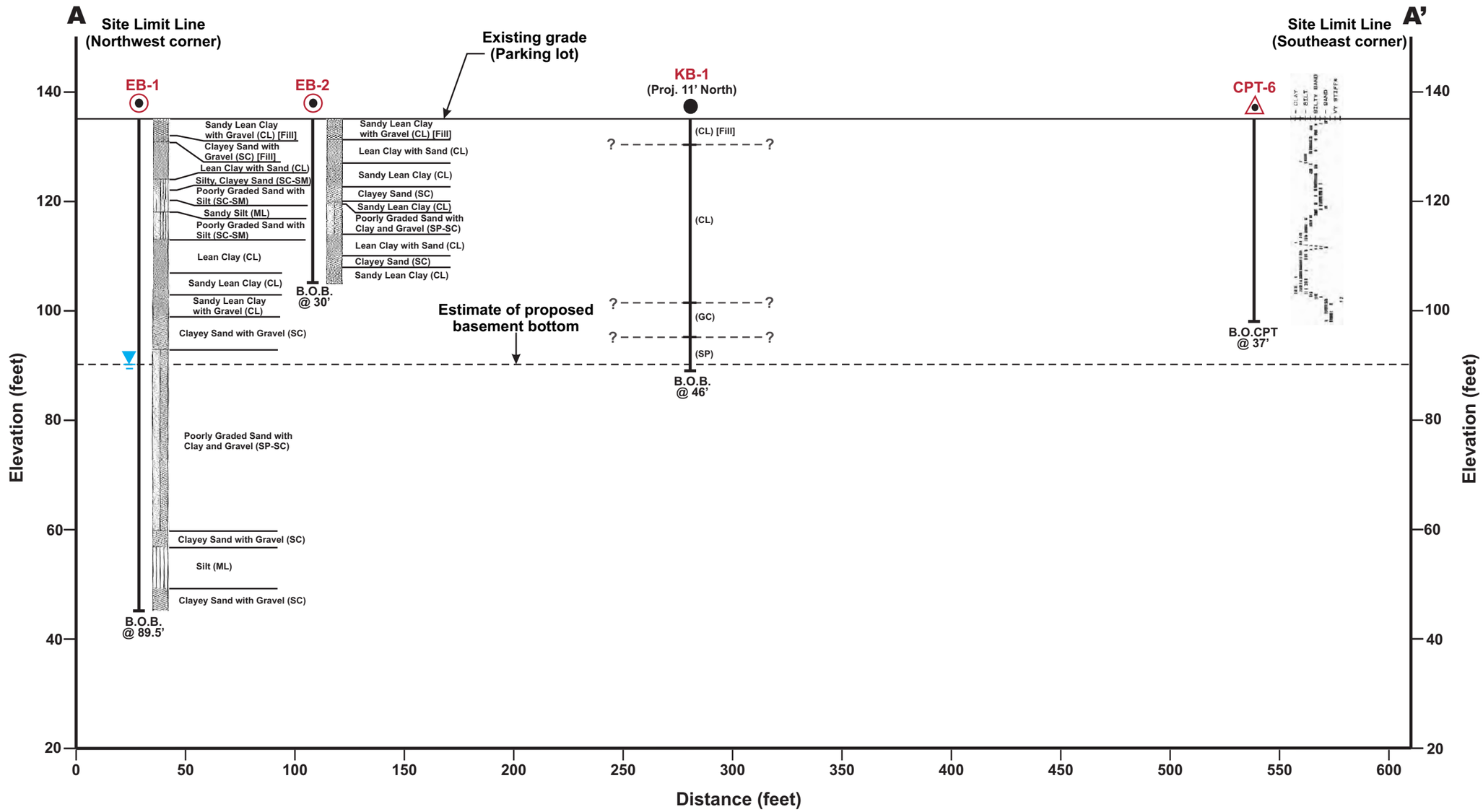
Date February 2013

Drawn By RRN

Site Plan

Santana Row Lot 9 Mixed-Use Development
San Jose, CA

CORNERSTONE EARTH GROUP



Section A-A'

(View Looking North)
1"=50' Horizontal
1"=20' Vertical

Symbols

- Approximate location of exploratory boring (Kleinfelder, 2005)
- Approximate location of exploratory boring (Lowney Associates, 2005)
- Approximate location of cone penetration test (Lowney Associates, 2005)
- Free groundwater encountered

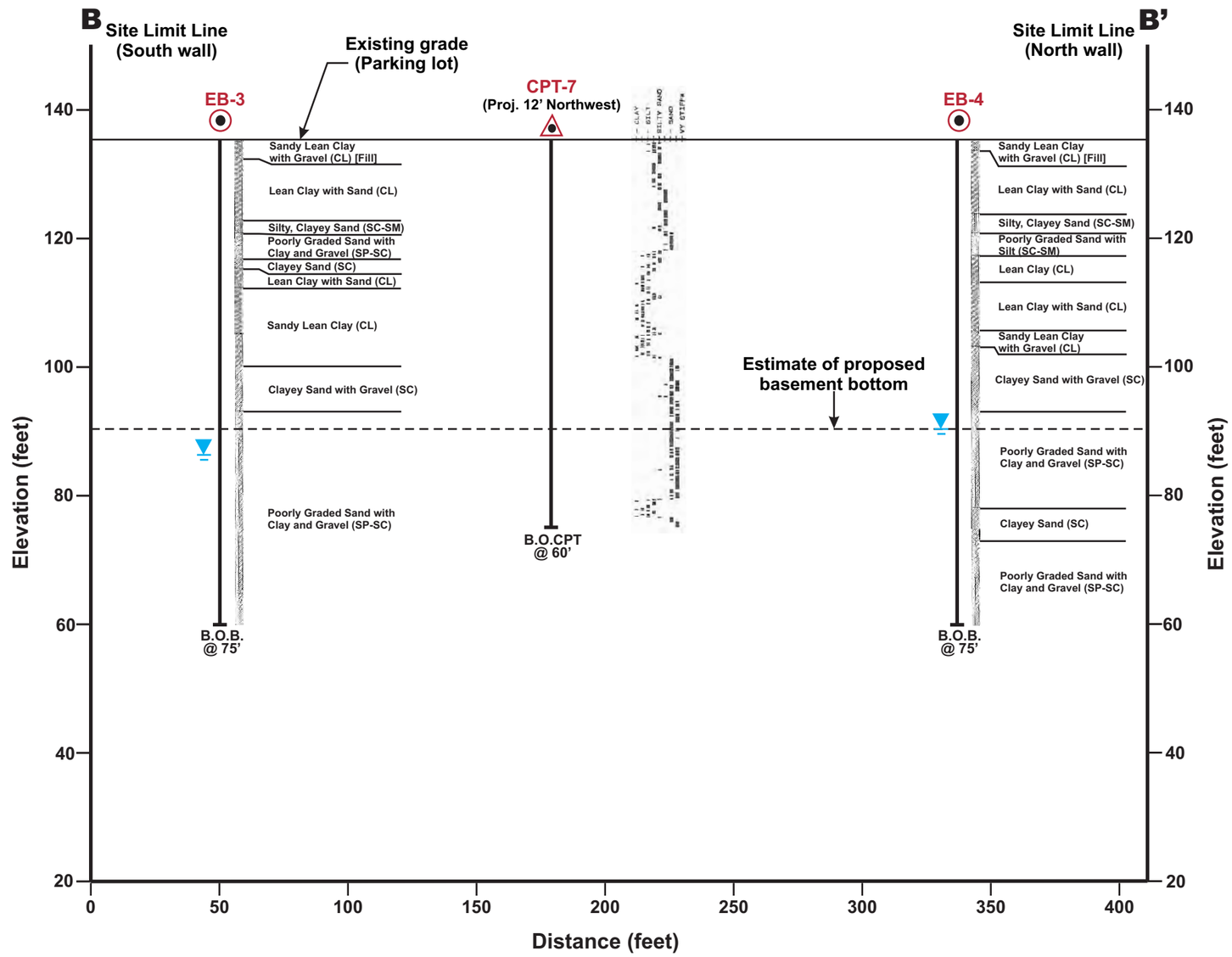
Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

Project Number 371-1-10
Figure Number Figure 3
Drawn By RRN
Date February 2013

Geologic Cross-Section A-A'

Santana Row Lot 9 Mixed-Use Development
San Jose, CA

CORNERSTONE EARTH GROUP



Section B-B'

(View Looking Northwest)
1"=50' Horizontal
1"=20' Vertical

Symbols

- Approximate location of exploratory boring (Lowney Associates, 2005)
- Approximate location of cone penetration test (Lowney Associates, 2005)
- Free groundwater encountered

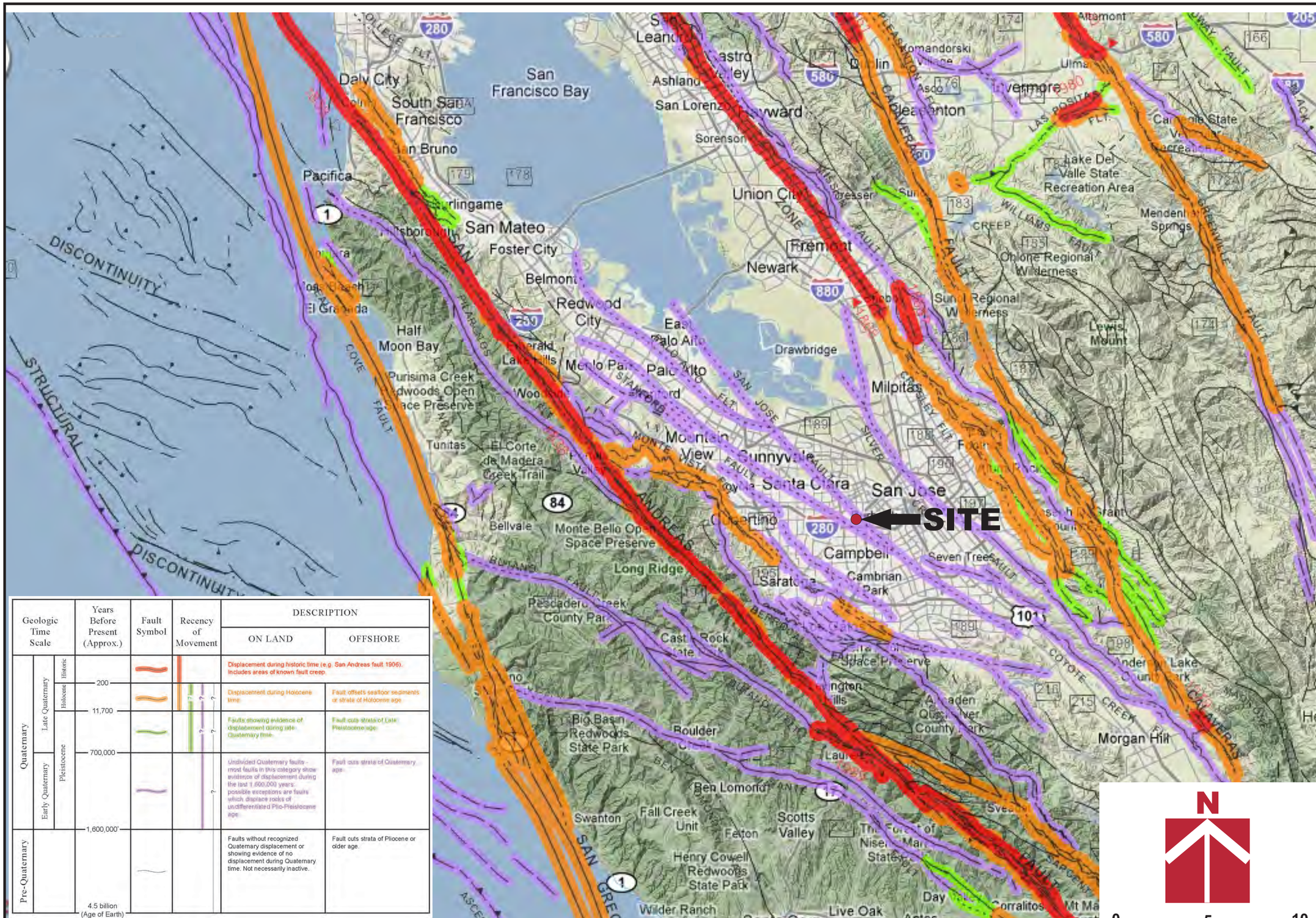
Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

Project Number 371-1-10
Figure Number Figure 4
Drawn By RRN
Date February 2013

Geologic Cross-Section B-B'

Santana Row Lot 9 Mixed-Use Development
San Jose, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700		? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary	Pleistocene	700,000		?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.
			4.5 billion (Age of Earth)				

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number: 371-1-10

Figure Number: Figure 5

Drawn By: RRN

Date: February 2013

Regional Fault Map

Santana Row Lot 9 Mixed-Use Development
San Jose, CA

CORNERSTONE EARTH GROUP

APPENDIX A: PREVIOUS FIELD INVESTIGATIONS AND LABORATORY TESTING PROGRAMS

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture.
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

Standard Penetration Split Spoon Sampler 2.0 inch O.D., 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.

0745, 5/31 — Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

0800, 5/31 — Approximate water level observed in boring following drilling

PEN — Pocket Penetrometer reading, in tsf

TV:Su — Torvane shear strength, in ksf

LL	LIQUID LIMIT	TX	TRIAXIAL SHEAR
PI	PLASTICITY INDEX	CONSOL	CONSOLIDATION
%-#200	SIEVE ANALYSIS (#200 SCREEN)	R-Value	RESISTANCE VALUE
DS	DIRECT SHEAR	SE	SAND EQUIVALENT
C	COHESION (PSF)	EI	EXPANSION INDEX
PHI	FRICTION ANGLE	FS	FREE SWELL (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

KLEINFELDER

PROJECT NO. 59959/9-10

BORING LOG LEGEND

SANTANA ROW MIXED-USE BUILDINGS
LOTS 9 AND 10
SAN JOSE, CALIFORNIA

PLATE

A-1

8/29/05 8:32:39 AM

Date Completed:	7/26/05	*Drilling method:*	Mud Rotary
Logged By:	F. Mwape	*Hammer Wt:*	140 lbs., 30" drop
Total Depth:	46.0 ft	*Notes:*	Rope and Cathead

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: **Estimated 135 feet (MSLD)**
								ASPHALT - approximately 1.5 inches thick
								AGGREGATE BASE - approximately 4 inches thick
		24					3.0	**SANDY LEAN CLAY with GRAVEL (CL)** - dark brown, moist, very stiff, with brick fragments (FILL)
								- sand is fine to coarse, gravel is fine to coarse
5							2.0	**LEAN CLAY (CL)** - brown, moist, very stiff, with trace fine sand and silt pockets
		21					3.7	
10								- with sand below 9.5 feet
		19	103.0	16.3			1.3 4.0	
15				29.7				- silty clay with fine sand, 15 to 17 feet
		6				LL=29; PI=7		
20								- stiff, 20 to 25 feet - brown and light gray below 20 feet
							1.3	- fine clayey sand layer, 22 to 22.5 feet
25			93.1	19.7		Consol LL=31; PI=15	0.7	- firm at 25 feet, stiff below 25 feet
							1.2	
				11.5				**SANDY LEAN CLAY (CL)** - brown, moist, stiff
30								- sandy fraction is fine to coarse

KLEINFELDER

PROJECT NO. *59959/9-10*

LOG OF BORING NO. KB-1

SANTANA ROW MIXED-USE BUILDINGS
LOTS 9 AND 10
SAN JOSE, CALIFORNIA

PLATE

A-2

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Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
							1.3	**SANDY LEAN CLAY (CL)** - continued - with fine to coarse gravel below 31.5 feet
35		44				Passing -#200=14%		**CLAYEY GRAVEL with SAND (GC)** - brown, moist, dense, fine to coarse gravel and sand - sandy fraction is fine to coarse
40		50/6"						**POORLY GRADED SAND with GRAVEL (SP)** - brown, moist, very dense, with trace clay - gravel fraction is fine to coarse
45		50/6"						
								Boring terminated at 46 feet below ground surface Boring backfilled with grout. Groundwater masked due to rotary wash methods. Blow counts shown are uncorrected.
50								
55								
60								

KLEINFELDER

PROJECT NO. 59959/9-10

LOG OF BORING NO. KB-1

SANTANA ROW MIXED-USE BUILDINGS
LOTS 9 AND 10
SAN JOSE, CALIFORNIA

PLATE

A-2
(cont'd)

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

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08, 0.4, 2, 5, 19, 76mm

GRAIN SIZES

TERZAGHI SPLIT SPOON STANDARD PENETRATION

MODIFIED CALIFORNIA

D&M UNDERWATER SAMPLER

SHELBY TUBE

NO RECOVERY

SAMPLERS

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

RELATIVE DENSITY

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

CONSISTENCY

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

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D–2487)

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE A–1

EXPLORATORY BORING: EB-1

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-3-05 FINISH DATE: 5-3-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 89.5 FT.

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

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

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION:						
	0	1 inch asphalt concrete over 4 inches aggregate base						
		SANDY LEAN CLAY WITH GRAVEL (CL) [FILL] hard, moist, dark brown and brown mottled, fine to coarse sand, fine to coarse gravel, low plasticity Plasticity Index = 8, Liquid Limit = 25	CL, FILL	34	▲	8	108	
		CLAYEY SAND WITH GRAVEL (SC) [FILL] loose, moist, brown, fine sand, fine to coarse gravel, low plasticity	SC, FILL	9	▲	7	110	
	5	**LEAN CLAY WITH SAND (CL)** stiff, moist, brown, fine sand, low plasticity	CL	14	▲	15	91	
	10	very stiff		15	▲	18	107	
		SILTY, CLAYEY SAND (SC-SM) loose to medium dense, moist, brown, fine sand	SM-SC	15	▲	14	93	43
		POORLY GRADED SAND WITH SILT (SP-SM) medium dense, moist, gray and brown, fine to medium sand	SP-SM	21	▲			
	15	**SANDY SILT (ML)** medium stiff, moist, brown, fine sand, low plasticity	ML	5	▲	31		
	20	**POORLY GRADED SAND WITH SILT (SP-SM)** medium dense, moist, gray and brown, fine to medium sand, some fine gravel	SP-SM	25	▲	4		
	25	**LEAN CLAY (CL)** medium stiff to stiff, moist, brown, some fine sand, low to moderate plasticity	CL	6	▲	25		
	30	**SANDY LEAN CLAY (CL)** stiff, moist, brown, fine sand, low plasticity	CL	19	▲	19	105	

Continued Next Page

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 45.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-3-05 FINISH DATE: 5-3-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 89.5 FT.

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

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

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
			CL					
		SANDY LEAN CLAY WITH GRAVEL (CL) hard, moist, brown, fine to medium sand, fine to coarse gravel, low plasticity	CL	44		9	122	
	35							
		CLAYEY SAND WITH GRAVEL (SC) dense, moist, brown and gray, fine to medium sand, fine to coarse gravel	SC	41				
	40							
		POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC) dense to very dense, wet, gray and brown, fine to medium sand, fine to coarse gravel		58		8	113	
∇	45							
				61				
	50		SP-SC					
				61		14		
	55							
				50/3"				
	60			48		9		

Continued Next Page

GROUND WATER OBSERVATIONS:
∇: FREE GROUND WATER MEASURED DURING DRILLING AT 45.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-3-05 FINISH DATE: 5-3-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 89.5 FT.

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ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	65		**POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC)** dense to very dense, wet, gray and brown, fine to medium sand, fine to coarse gravel	SP-SC	48		9		
	70								
	75		**CLAYEY SAND WITH GRAVEL (SC)** dense, wet, brown and gray, fine to coarse sand, fine to coarse gravel	SC	36				
	80		**SILT (ML)** very stiff, moist, gray to bluish gray, some fine sand, low plasticity	ML	40		28	97	88
	85		**CLAYEY SAND WITH GRAVEL (SC)** dense, wet, brown and gray, fine to coarse sand, fine to coarse gravel	SC	50/3"				
	90		Bottom of Boring at 89½ feet						

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

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 15.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-2

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-2-05 FINISH DATE: 5-2-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 30.0 FT.

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SURFACE ELEVATION:

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	1 inch asphalt concrete over 3 inches aggregate base					
		SANDY LEAN CLAY WITH GRAVEL (CL) [FILL] hard, moist, brown, fine to medium sand, fine gravel, low plasticity	CL, FILL	21	11	106	
		LEAN CLAY WITH SAND (CL) very stiff, moist, brown, fine sand, low plasticity		7			
	5		CL	12	15	95	
		SANDY LEAN CLAY (CL) very stiff, moist, brown, fine sand, low plasticity		17			
	10		CL	16	18	98	
		CLAYEY SAND (SC) medium dense, moist, brown, fine sand	SC	13			
	15	**SANDY LEAN CLAY (CL)** stiff, moist, brown, fine sand, low plasticity	CL	29	6		
		POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC) medium dense to dense, moist, gray and brown, fine to coarse sand, fine to coarse gravel	SP-SC	37			
	20			29	21		
		LEAN CLAY WITH SAND (CL) stiff, moist, brown, fine to medium sand, low plasticity	CL				
	25	**CLAYEY SAND (SC)** medium dense, moist, brown, fine sand	SC				
		SANDY LEAN CLAY (CL) stiff, moist, brown, fine sand, low plasticity Plasticity Index = 12, Liquid Limit = 28	CL	22	17		
	30	Bottom of Boring at 30 feet					

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

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 6/6/05 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-3

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-2-05 FINISH DATE: 5-2-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 75.0 FT.

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

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION:						
	0	1 inch asphalt concrete over 3½ inches aggregate base						
		SANDY LEAN CLAY WITH GRAVEL (CL) [FILL] hard, moist, brown, fine to medium sand, fine gravel, low plasticity	CL, FILL	37	X	8	120	
		LEAN CLAY WITH SAND (CL) very stiff, moist, brown, fine sand, low plasticity	CL	24	X	16	84	
	5			9	X	16	95	
				26	X	15	92	
	10			22	X	18	95	
		SILTY, CLAYEY SAND (SC-SM) loose to medium dense, moist, brown, fine sand	SM-SC	19	X	8	97	
	15	**POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC)** dense, moist, light brown, fine to medium sand, fine gravel	SP-SC	44	X	4		
		CLAYEY SAND (SC) loose, moist, brown, fine sand	SC	9	X	14		
	20	**LEAN CLAY WITH SAND (CL)** stiff, moist, brown, fine sand, trace fine to coarse gravel, low plasticity	CL	11	X			
		SANDY LEAN CLAY (CL) stiff, moist, brown, fine sand, some fine to coarse gravel, low plasticity	CLs	25	X	15	100	
	25							
		medium stiff		20	X	14	113	
	30							

Continued Next Page

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 49.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

EXPLORATORY BORING: EB-3 Cont'd

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-2-05 FINISH DATE: 5-2-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 75.0 FT.

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

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30		**SANDY LEAN CLAY WITH GRAVEL (CL)** very stiff, moist, brown, fine to medium sand, fine to coarse gravel, low plasticity	CL	31		10	132	
	35		**CLAYEY SAND WITH GRAVEL (SC)** dense to very dense, wet, reddish brown and gray, fine to medium sand, fine to coarse gravel	SC	38		8		
	40				50/5"				
			POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC) dense to very dense, wet, gray and brown, fine to coarse sand, fine to coarse gravel	SP-SC	41		7		
	45				45				
▽	50				31		13		7
	55				50/5"				
	60								

Continued Next Page

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 49.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

EXPLORATORY BORING: EB-3 Cont'd

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-2-05 FINISH DATE: 5-2-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 75.0 FT.

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

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

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	60		**POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC)** dense to very dense, wet, gray and brown, fine to coarse sand, fine to coarse gravel						
					48	X	11		
	65			SP-SC					
					50/4"	X			
	70								
					50/4"	X	11		
	75		Bottom of Boring at 75 feet						
	80								
	85								
	90								

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 49.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-4

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-3-05 FINISH DATE: 5-3-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 75.0 FT.

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

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
		SURFACE ELEVATION:				
	0	2 inch asphalt concrete over 2½ inches aggregate base				
		SANDY LEAN CLAY WITH GRAVEL (CL) [FILL] very stiff, moist, dark brown and brown mottled, fine to medium sand, fine gravel, low plasticity	CL, FILL	25	18	108
		LEAN CLAY WITH SAND (CL) very stiff, moist, brown, fine sand, low to moderate plasticity. Plasticity Index = 15, Liquid Limit = 37	CL	14	20	95
	5			18	15	96
	10			16	16	101
		SILTY, CLAYEY SAND (SC-SM) medium dense, moist, brown, fine sand	SM-SC	10	7	
				16		
	15	**POORLY GRADED SAND WITH SILT (SP-SM)** stiff, moist, brown, fine to medium sand, some fine gravel	SP-SM	14	8	
	20	**LEAN CLAY (CL)** very stiff, moist, brown, some fine to medium sand, low to moderate plasticity	CL	18	19	
	25	**LEAN CLAY WITH SAND (CL)** stiff, moist, brown with gray mottles, fine sand, low to moderate plasticity	CL	25	20	98
	30			35	16	116

Continued Next Page

GROUND WATER OBSERVATIONS:
∇: FREE GROUND WATER MEASURED DURING DRILLING AT 45.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

EXPLORATORY BORING: EB-4 Cont'd

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-3-05 FINISH DATE: 5-3-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 75.0 FT.

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

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30	**SANDY LEAN CLAY WITH GRAVEL (CL)** very stiff, moist, brown with gray mottles, fine to medium sand, fine to coarse gravel, low plasticity Plasticity Index = 14, Liquid Limit = 33						
		CLAYEY SAND WITH GRAVEL (SC) dense, moist, brown, fine to coarse sand, fine to coarse gravel		46	X	5		
	35		SC					
				58	X			
	40							
		POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC) very dense, wet, brown and gray, fine to coarse sand, fine to coarse gravel		52	X	8		5
	45							
	50		SP-SC	81	X			
	55			79	X	9		
		CLAYEY SAND (SC) medium dense to dense, moist, bluish-gray, fine sand	SC	28	X	20		32
	60	*Continued Next Page*						

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 45.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-4 Cont'd

DRILL RIG: MOBILE B-40
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BMM
START DATE: 5-3-05 FINISH DATE: 5-3-05

PROJECT NO: 1477-1P
PROJECT: SANTANA ROW LOTS 9/10
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 75.0 FT.

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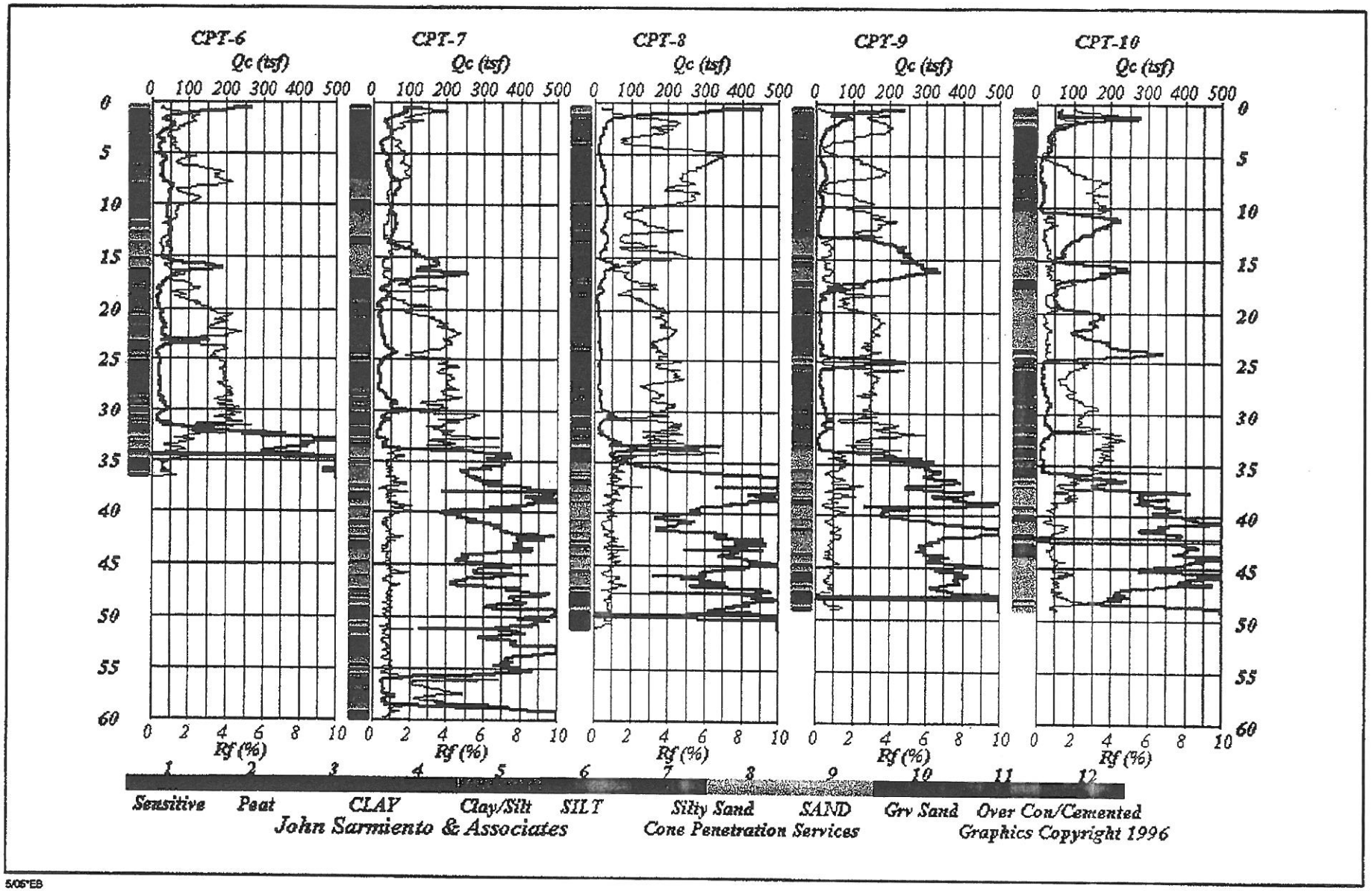
ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	60		**CLAYEY SAND (SC)** medium dense to dense, moist, bluish-gray, fine sand	SC					
	65		**POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC)** very dense, wet, brown and gray, fine to coarse sand, fine to coarse gravel	SP-SC	74	X	14		
	70				39	X	12		
	75				59	X	12		
			Bottom of Boring at 75 feet						
	80								
	85								
	90								

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

GROUND WATER OBSERVATIONS.
⊻: FREE GROUND WATER MEASURED DURING DRILLING AT 45.0 FEET

LA CORP.GDT 6/6/05 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

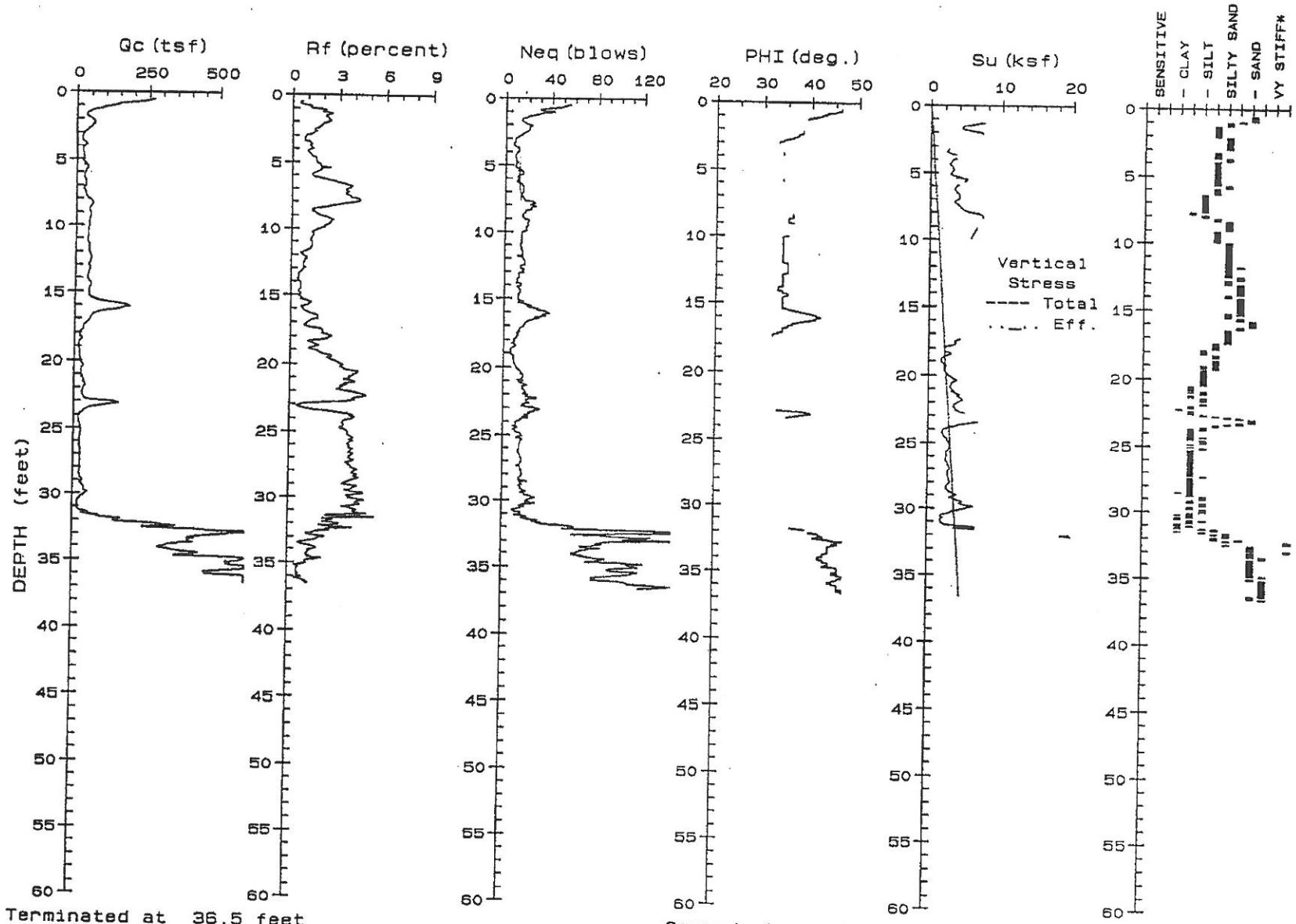


5/05"EB

CONE PENETRATION TEST - CPT-6, 7, 8, 9, 10

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

1477-1P



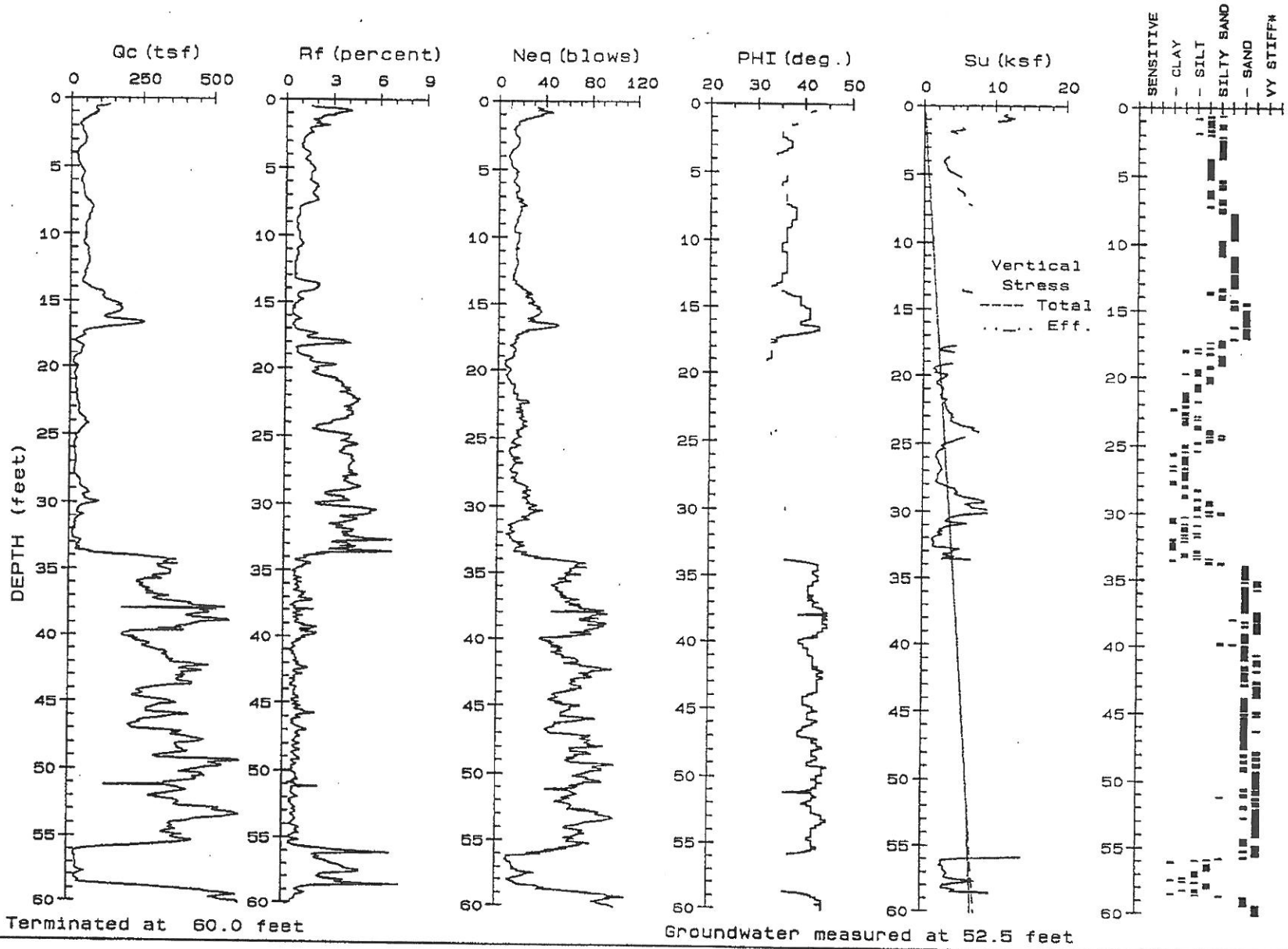
Terminated at 36.5 feet

Groundwater not encountered

PROJECT: Santana Row - Lots 6 & 8-10
LOCATION: San Jose CA
PROJ. NO.: 1477.1P (LNY-129)

CPT NO.: CPT-6
DATE : 05-03-2005

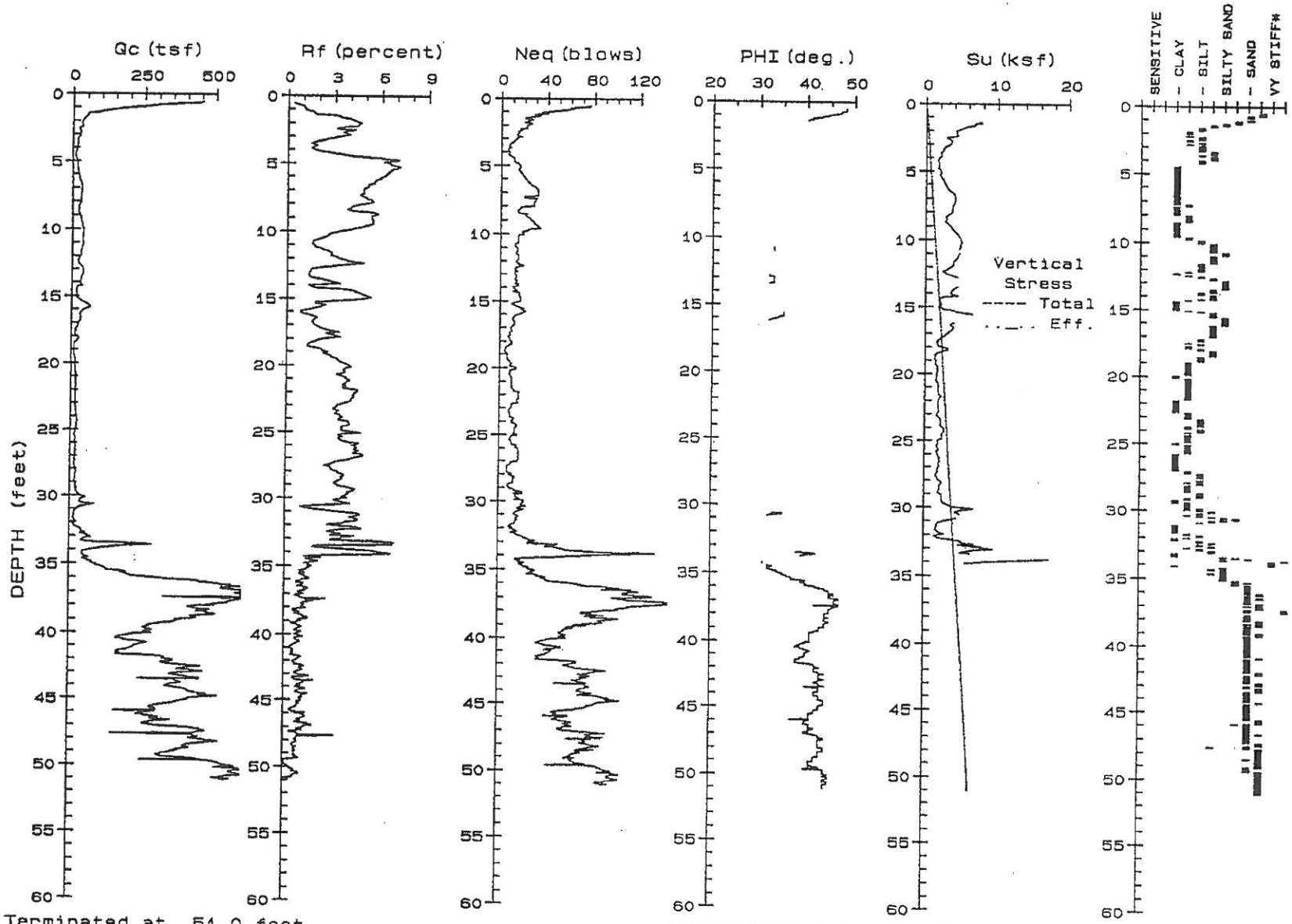
John Sarmiento & Associates
Cone Penetration Testing Service



PROJECT: Santana Row - Lots 6 & 8-10
LOCATION: San Jose CA
PROJ. NO.: 1477.1P (LNY-129)

CPT NO.: CPT-7
DATE : 05-03-2005

John Sarmiento & Associates
Cone Penetration Testing Service



Terminated at 51.0 feet

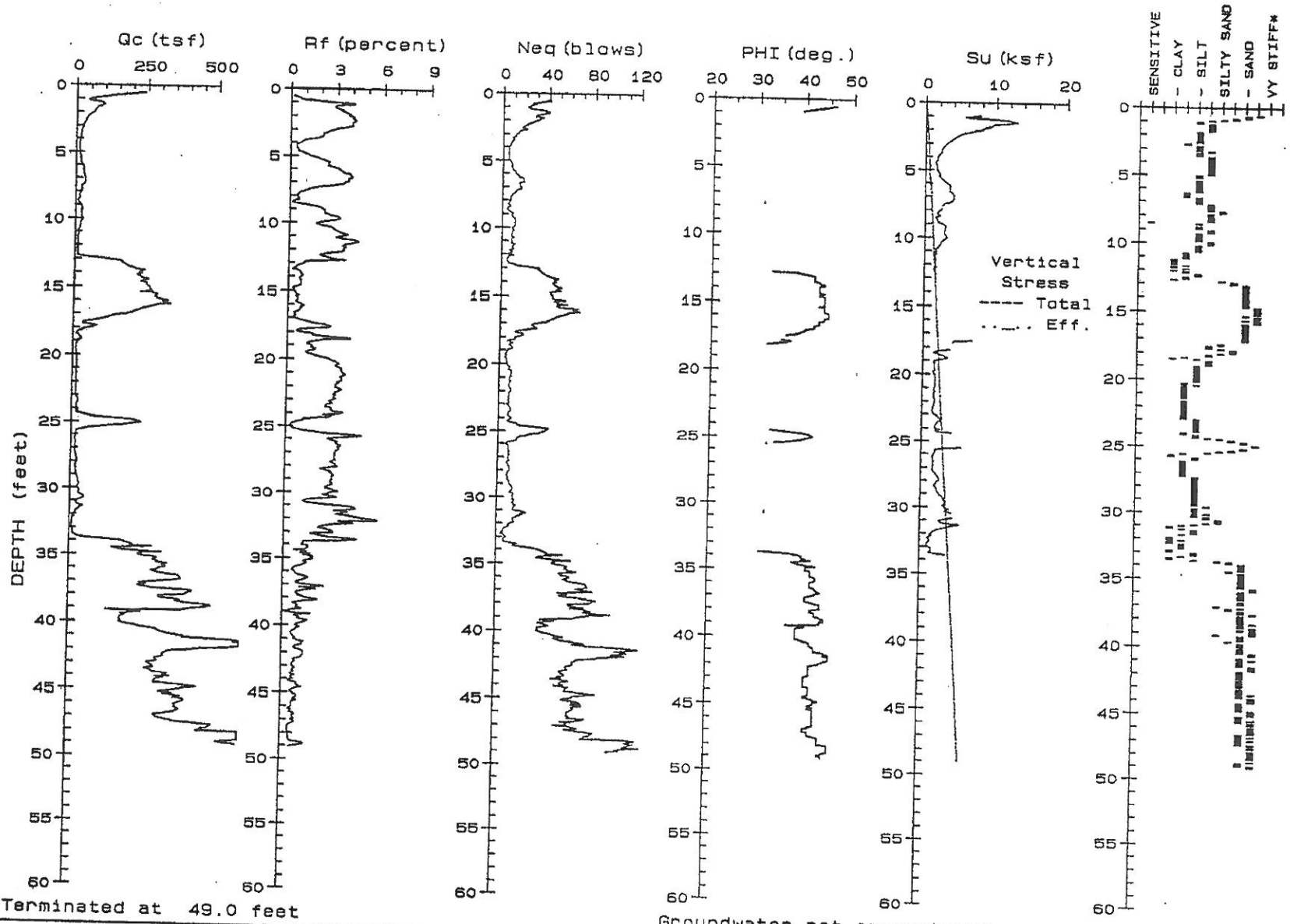
Groundwater not encountered

PROJECT: Santana Row - Lots 6 & 8-10
LOCATION: San Jose CA
PROJ. NO.: 1477.1P (LNY-129)

CPT NO.: CPT-8
DATE : 05-02-2005

John Sarmiento & Associates

Cone Penetration Testing Service



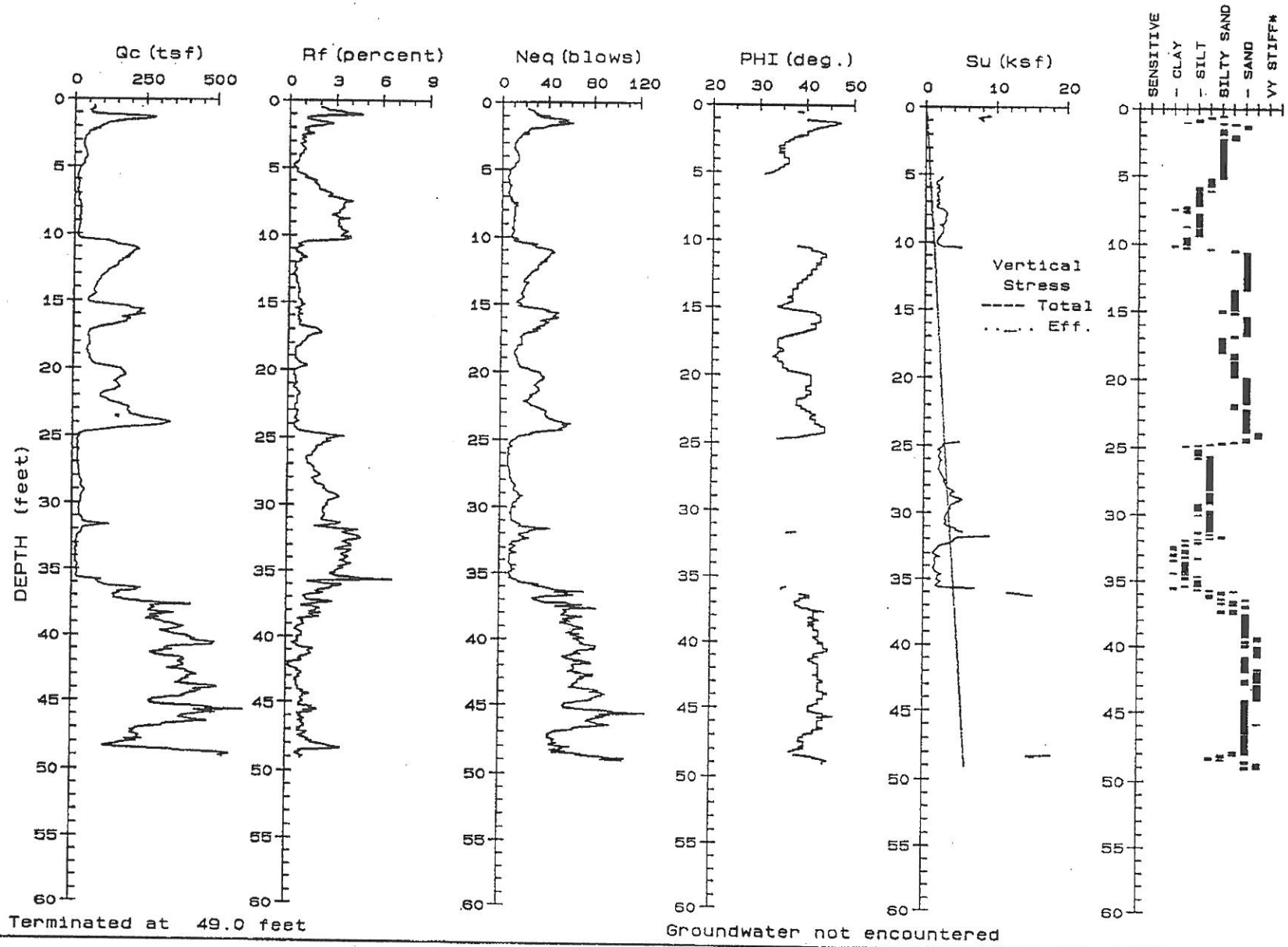
Terminated at 49.0 feet

Groundwater not encountered

PROJECT: Santana Row - Lots 6 & 8-10
LOCATION: San Jose CA
PROJ. NO.: 1477.1P (LNY-129)

CPT NO.: CPT-9
DATE : 05-02-2005

John Sarmiento & Associates
Cone Penetration Testing Service



PROJECT: Santana Row - Lots 6 & 8-10
LOCATION: San Jose CA
PROJ. NO.: 1477.1P (LNY-129)

CPT NO.: CPT-10
DATE : 05-03-2005

John Sarmiento & Associates
Cone Penetration Testing Service

PREVIOUS EXPLORATION LOGS

APPENDIX A

FIELD INVESTIGATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted hollow-stem auger drilling and cone penetration test (CPT) equipment. Four 8-inch-diameter exploratory borings were drilled on May 2 through 5, 2005 to a maximum depth of 90 feet. Six CPTs were advanced to a maximum depth of 60 feet on May 2 though 4, 2005. CPT data was obtained at 0.16 feet intervals, and consisted of cone tip resistance, sleeve friction and other parameters. The data obtained was correlated using the references cited, to determine the indicated soil type, shear strength, equivalent Standard Penetration Test (SPT), N-value (blows per foot), and other parameters. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Our boring and CPT logs, as well as a key to the classification of the soil, are included as part of this appendix.

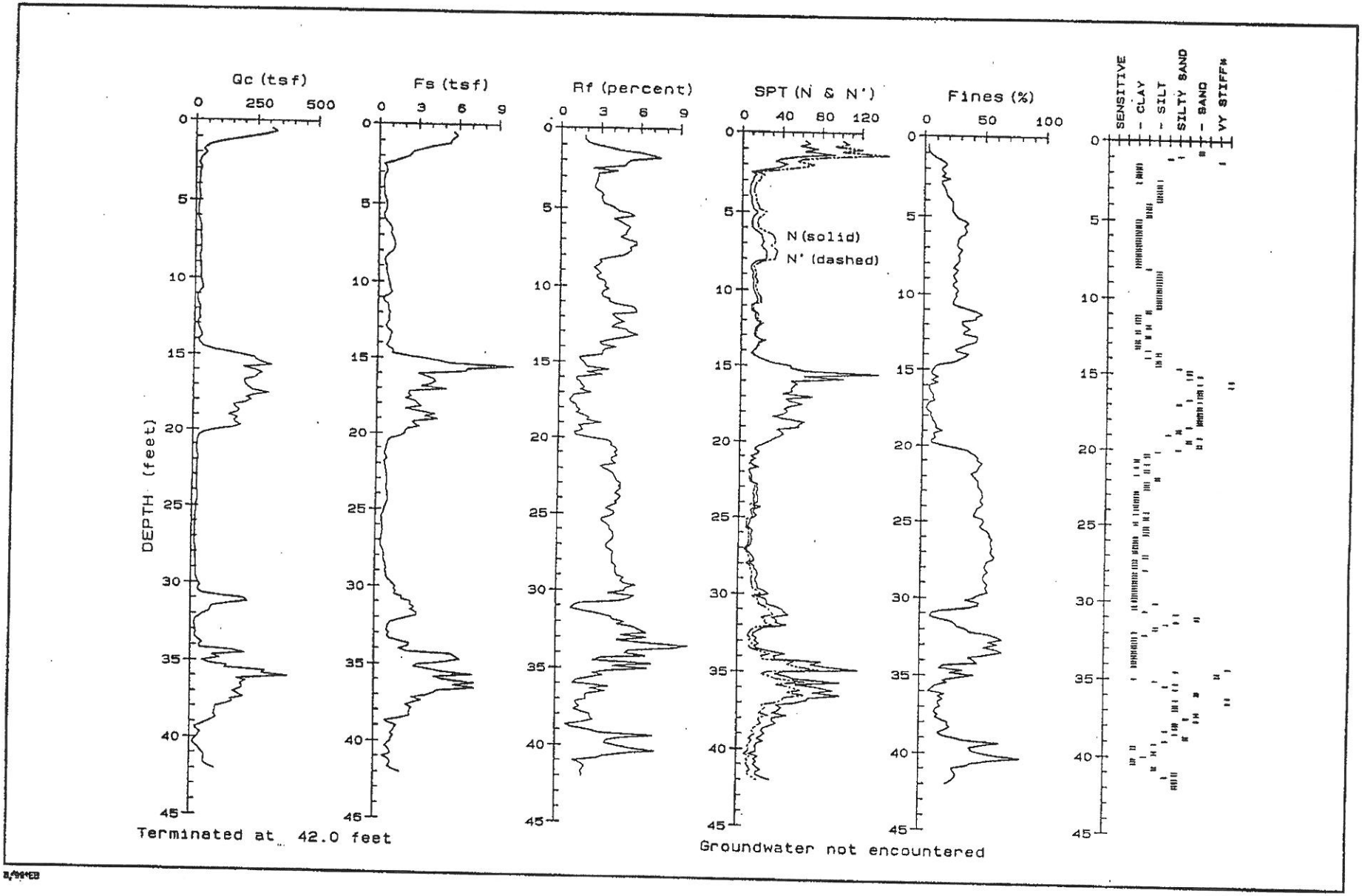
The locations of borings and CPTs were approximately determined by pacing from existing site boundaries and structures. The elevations of the borings were not determined. The locations of the borings and CPT should be considered accurate only to the degree implied by the method used.

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

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

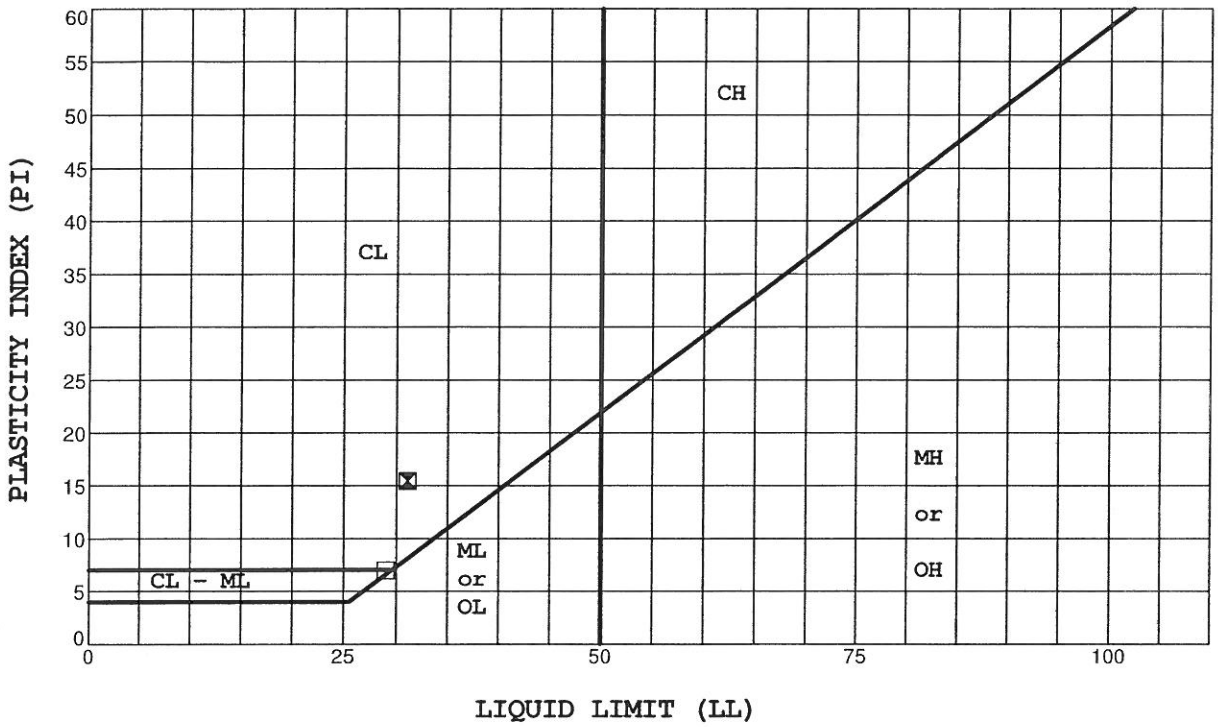
The attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered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

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



CONE PENETRATION TEST LOG - CPT-2

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services



Symbol	Boring	Depth	LL	PL	PI	Sample Description
☐	KB-1	16.0	29	22	7	Lean Clay
☒	KB-1	24.0	31	16	15	Lean Clay with Sand

Unified Soil Classification

Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine sands of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

KLEINFELDER

PROJECT NO. 59959/9-10

PLASTICITY CHART

SANTANA ROW MIXED-USE BUILDINGS
LOTS 9 AND 10
SAN JOSE, CALIFORNIA

PLATE

C-1

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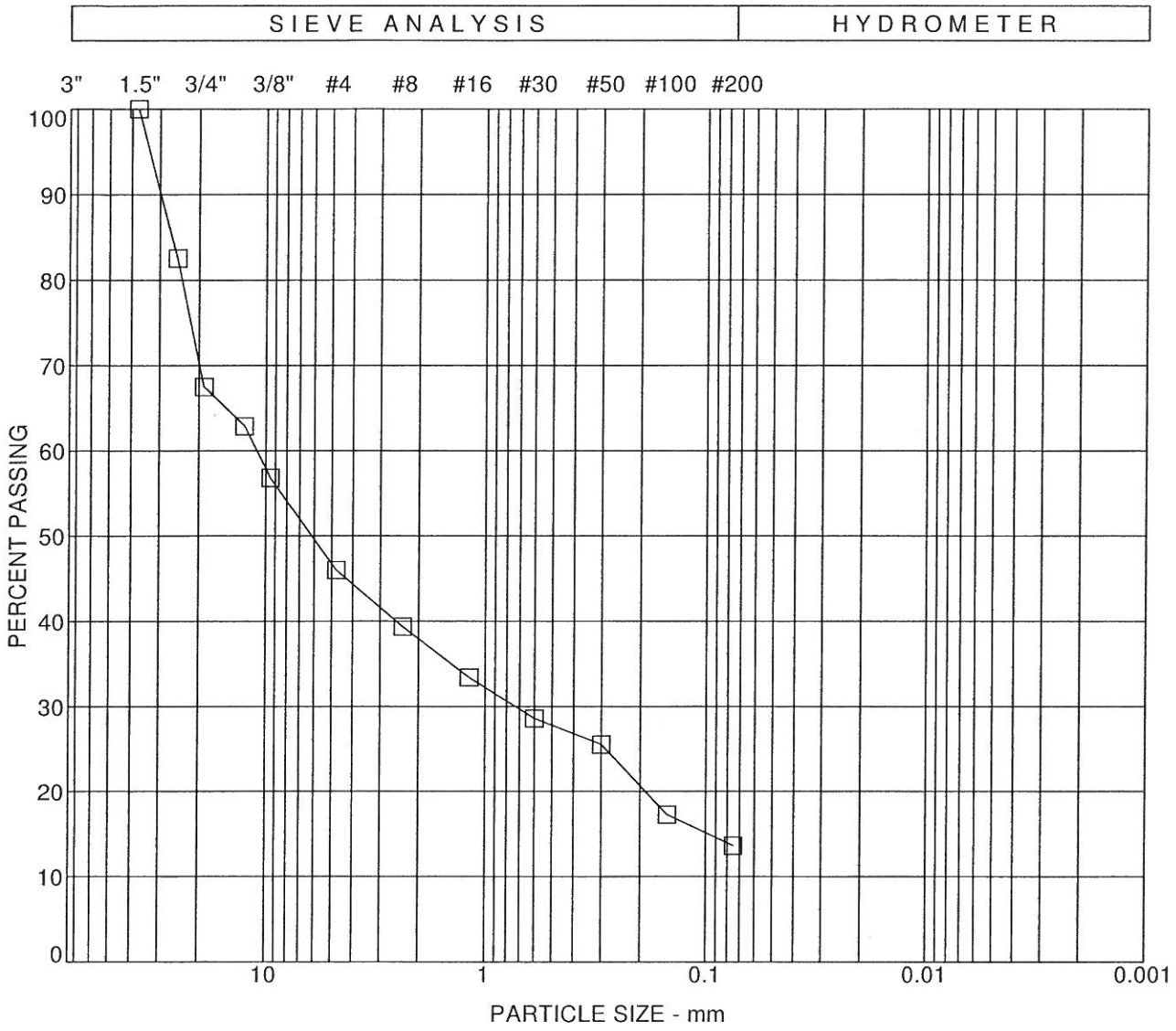

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH (ft)	CLASSIFICATION
□	KB-1	35.0	Clayey Gravel with Sand

KLEINFELDER

PROJECT NO. 59959/9-10

GRAIN SIZE DISTRIBUTION

SANTANA ROW MIXED-USE BUILDINGS
LOTS 9 AND 10
SAN JOSE, CALIFORNIA

PLATE

C-2

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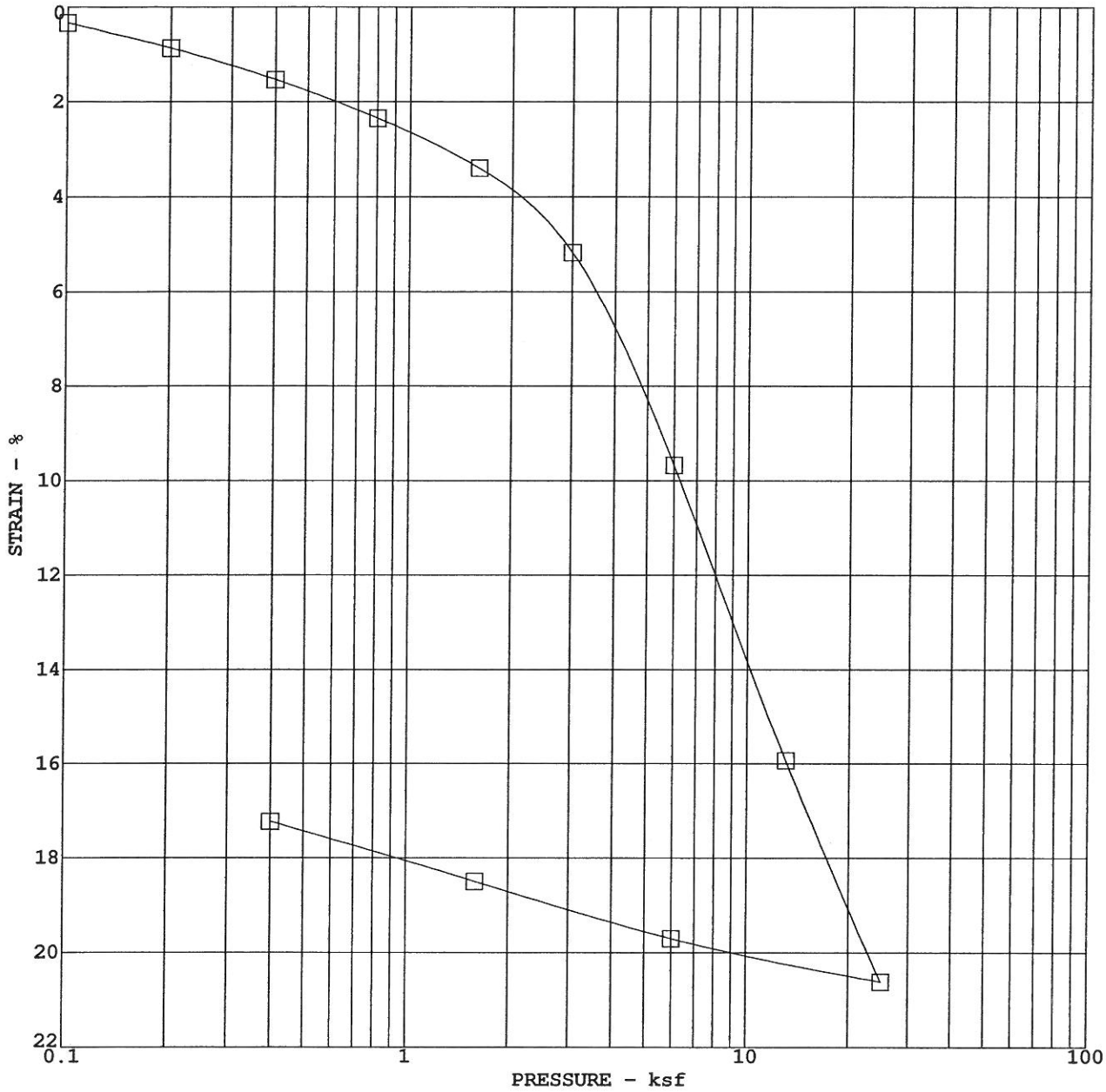


BORING NO. KB-1

DEPTH 24.0

DESCRIPTION Lean Clay with Sand

PRECONSOLIDATION PRESSURE 3.00 ksf

COMPRESSION RATIO= $C_c/1+e_o$ **0.180**

RECOMPRESSION RATIO= $C_r/1+e_o$ **0.043**

LL = **31** PL = **16**

	INITIAL	FINAL
DRY DENSITY, lb/ft^3	93.1	110.0
WATER CONTENT, %	19.7	19.9
SAMPLE HEIGHT, in.	.6780	.5612

KLEINFELDER

PROJECT NO. 59959/9-10

CONSOLIDATION TEST

SANTANA ROW MIXED-USE BUILDINGS
LOTS 9 AND 10
SAN JOSE, CALIFORNIA

PLATE

C-3

L:\2005\05PROJECTS\59959\59959_9-10.GPJ

12/12/05 10:06:03 AM

APPENDIX B

LABORATORY PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 53 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

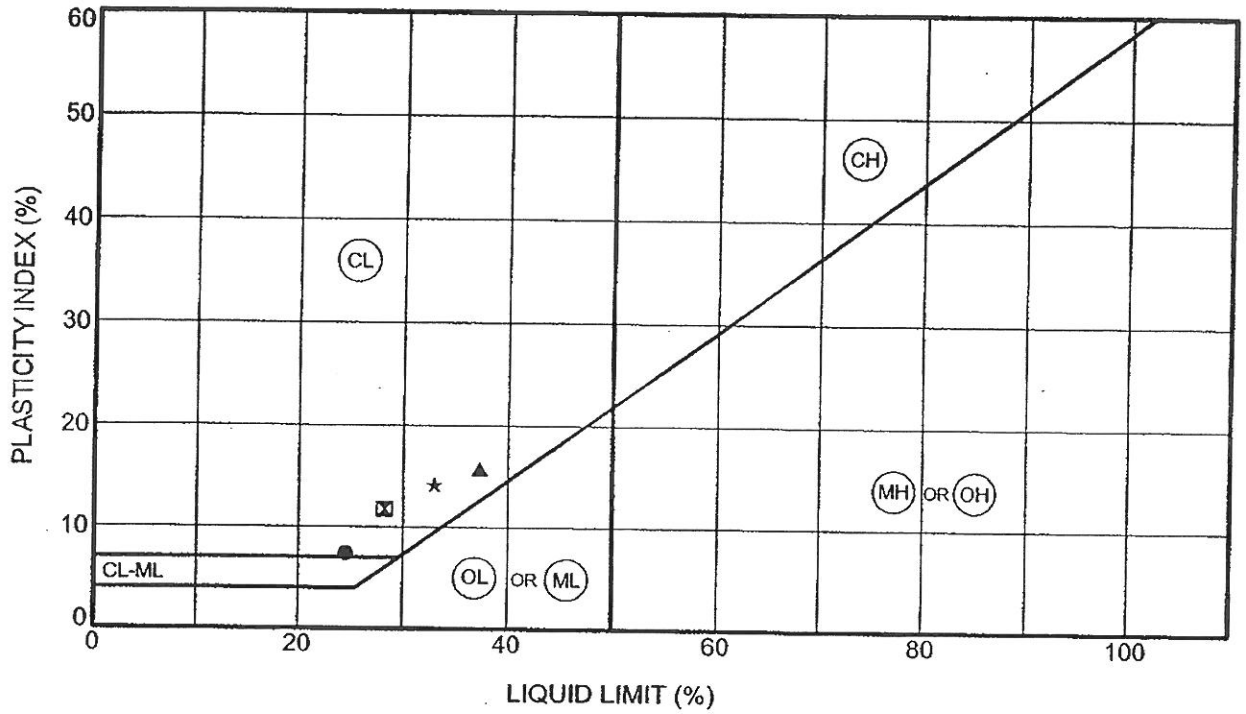
Dry Densities: In place dry density determinations (ASTM D2937) were performed on 27 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Plasticity Index determinations (ASTM D4318) were performed on 4 samples of the subsurface soils to measure the range of water contents over which these materials exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on Figure B-1 and on the logs of the borings at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 5 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.

Consolidated Undrained Shear Strength (CU): Four triaxial consolidated-undrained shear tests were performed to estimate the undrained shear strength. Results of these tests are included in this appendix.

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



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	EB-1	2.0	8	25	17	8		SANDY LEAN CLAY (CL) [FILL]
⊠	EB-2	29.5	17	28	16	12		SANDY LEAN CLAY (CL)
▲	EB-4	2.0	18	37	22	15		LEAN CLAY WITH SAND (CL)
★	EB-4	29.5	16	33	19	14		SANDY LEAN CLAY WITH GRAVEL (CL)

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

PLASTICITY CHART AND DATA

Project: SANTANA ROW

Location: SAN JOSE, CA

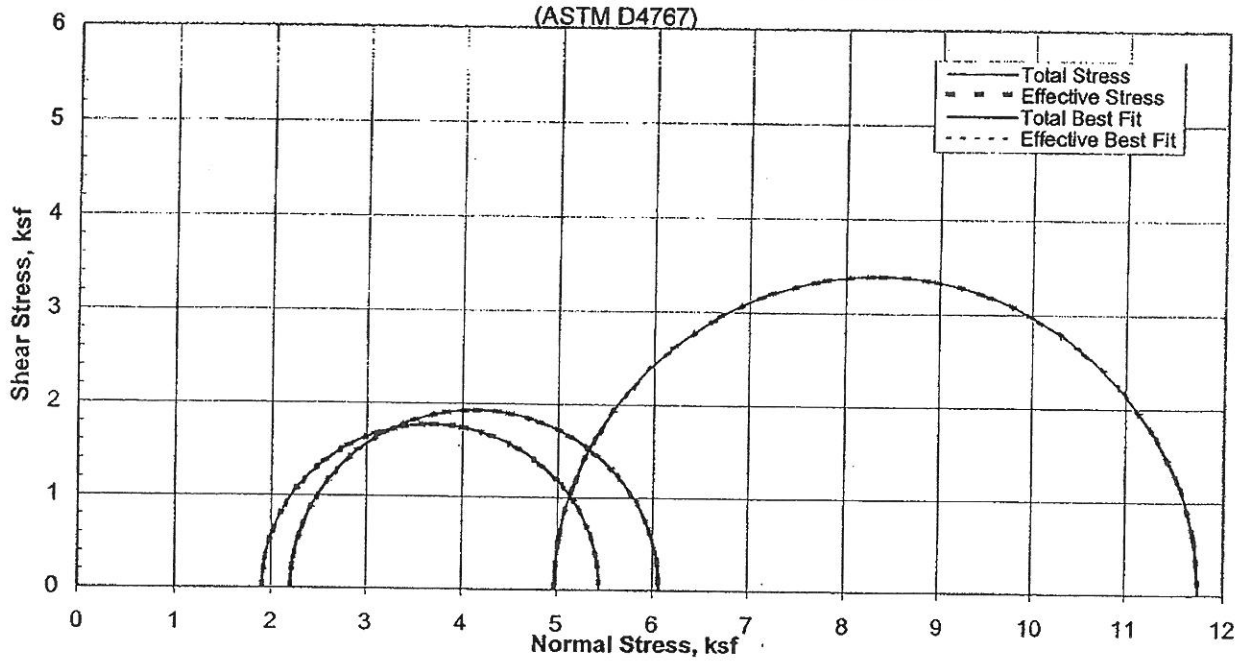
Project No.: 1477-1P

FIGURE B-1

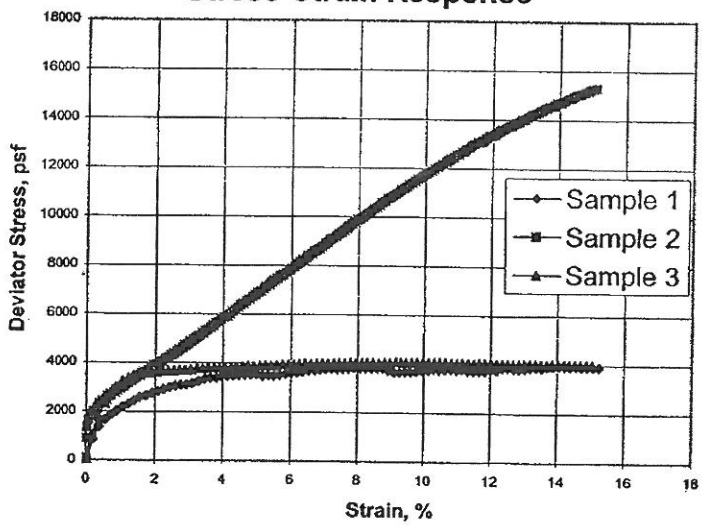
LA CORP.GDT 5/16/05 MV FLL

Triaxial Consolidated Undrained

(ASTM D4767)



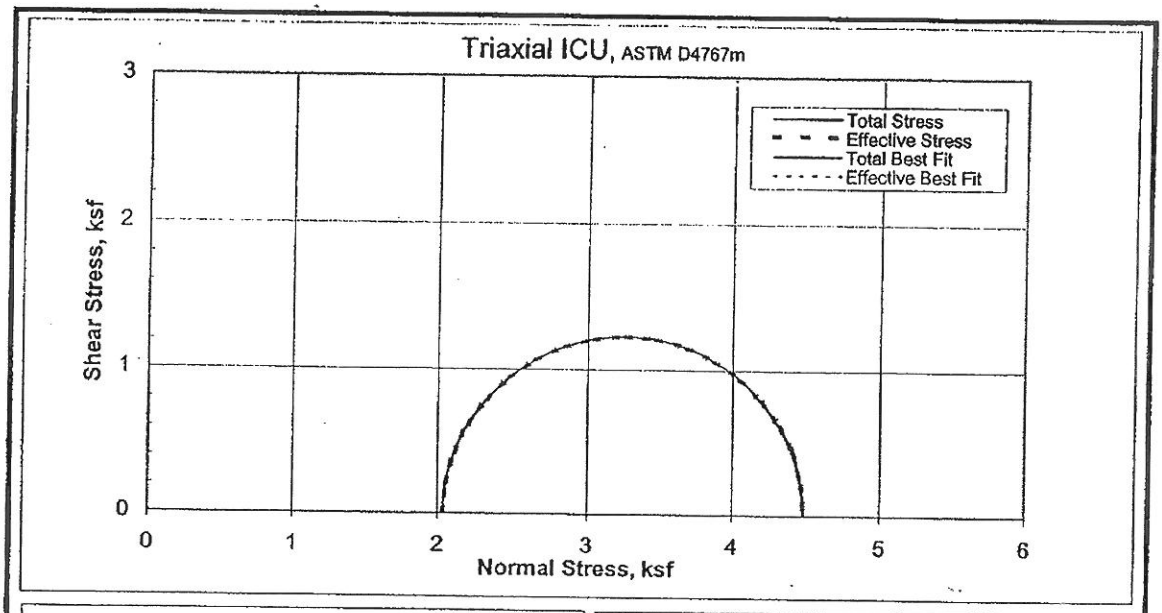
Stress-Strain Response

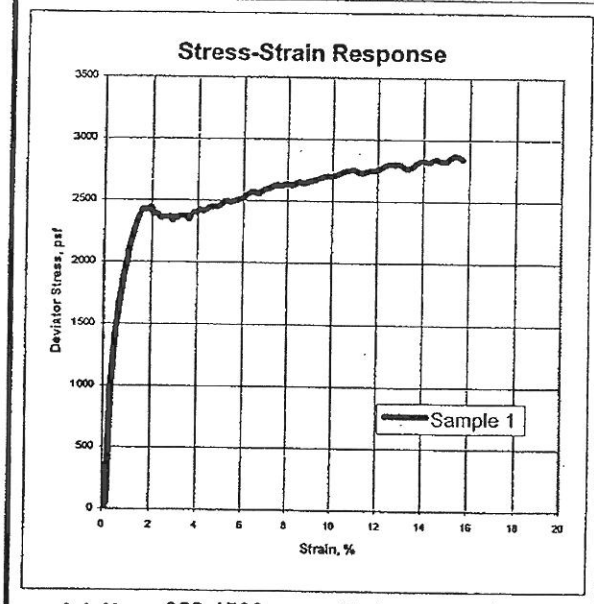


Sample:	1	2	3	4
MC, %	18.7	28.6	20.4	
Dry Dens, pcf	109.3	95.9	101.7	
Sat. %	93.3	101.8	83.9	
Void Ratio	0.541	0.757	0.657	
Diameter in	2.88	2.42	2.88	
Height, in	5.99	5.00	5.99	
	Final			
MC, %	19.1	24.8	22.7	
Dry Dens, pcf	111.1	100.9	104.5	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.517	0.669	0.613	
Diameter, in	2.87	2.35	2.84	
Height, in	5.91	5.02	6.00	
Cell, psi	52.4	73.1	51.6	
BP, psi	39.2	38.6	36.3	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	3.547	6.786	3.864	
Excess PP	0.000	0.000	0.000	
Sigma 1	5.448	11.754	6.067	
Sigma 3	1.901	4.968	2.203	
P, ksf	3.674	8.361	4.135	
Q, ksf	1.773	3.393	1.932	
Stress Ratio	2.866	2.366	2.754	
Rate in/min	0.040	0.040	0.040	
Total C				
Total Phi				
Eff. C				
Eff. Phi				

Job No.:	028-1599	Date:	5/24/2005
Client:	Lowney Associates		BY:MJ
Project:	Santana Row Lots 9/10 - 1477-1P		
Sample 1)	EB-1, 11A @ 25'	Brown Clayey SAND (silty)	
Sample 2)	EB-1, 22B @ 81'	Gray Sandy SILT	
Sample 3)	EB-2, 10A @ 23.5'	Brown Silty SAND	
Sample 4)			

REMARKS: Strengths picked at 5% strain.





Sample:	1	2	3	4
MC, %	17.5			
Dry Den, pcf.	109.4			
Sat. %	87.3			
Void Ratio	0.540			
Diameter in	2.88			
Height, in	5.99			
	Final			
MC, %	19.6			
Dry Den, pcf.	110.2			
Sat. %	100.0			
Void Ratio	0.529			
Diameter, in	2.88			
Height, in	5.93			
Cell, psi	62.9			
BP, psi	48.8			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	2.456			
Excess PP	0.000			
Sigma 1	4.487			
Sigma 3	2.030			
P, ksf	3.258			
Q, ksf	1.228			
Stress Ratio	2.210			
Rate in/min	0.002			
Total C				
Total Phi				
Eff. C				
Eff. Phi				

Job No.: 028-1599a **Date:** 5/26/2005

Client: Lowney Associates BY:DC

Project: Santana Row Lots 9/10 - 1477-1P

Sample 1) EB-4, 10A @ 25' Brown Sandy CLAY near Clayey SAND

Sample 2)

Sample 3)

Sample 4)

REMARKS: Values picked at 5% strain.

Corrosivity Test Summary

CTL # 028-1599 Date: 5/20/2005 Tested By: PJ Checked: PJ

Client: Lowney Associates Project: Santana Row Lots 9/10 Proj. No: 1477-1P

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	As Received	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	ASTM D2216	
EB-1	2A	3.5	-	-	2,623	-	-	-	7.3	-	-	Dark Brown Clayey SAND w/ Gravel
EB-2	3A	5.5	-	-	1,586	-	-	-	7.5	-	-	Brown CLAY
EB-3	6A	13.5	-	-	3,744	-	-	-	7.9	-	-	Brown Silty SAND
EB-4	4A	9.0	-	-	1,243	-	-	-	7.8	-	-	Brown CLAY