GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
PARK AVENUE AND DELMAS AVENUE
SAN JOSE, CALIFORNIA

PROJECT 2014.0039

For

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1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical investigation for the proposed residential development at the southwest corner of Park Avenue and Delmas Avenue in San Jose, California. The project area is referred to as "site" or "project site" in this report. The approximate location of the project site is shown on the Vicinity Map included with Figure 1 of this report. Figure 1 shows a layout of the proposed development. Figure 2 shows a layout of the existing and previously existing site surface features.

This report presents our conclusions and geotechnical recommendations for project design and construction. These conclusions and recommendations are based on subsurface information collected during this investigation and a 2006 geotechnical investigation by Donald E. Banta & Associates (DBA). The conclusions and recommendations in this report should not be extrapolated to other areas or used for other projects without our review.

1.2 PROJECT DESCRIPTION

The approximately 1.60-acre site will be developed with multi-family residential units above a single-level podium underground parking garage. The structures are anticipated to be four- and five-story buildings above the parking garage. Ancillary improvements will include exterior flatwork, underground utilities and landscaping. Retaining walls will include the subterranean parking structure walls and exterior short landscaping walls.

For preparation of our recommendations, we have anticipated the building loads to be typical of the above-described residential structures. We have also anticipated site grading will involve cuts up to be about 12 feet in depth to accommodate the underground parking garage, and cuts and fills of about 1 to 3 feet across the remainder of the site.

The above project descriptions are based on information provided to us. If the actual project differs from those described above, Pacific Geotechnical Engineering (PGE) should be contacted to review our conclusions and recommendations and present any necessary modifications to address the different project development schemes.

1.3 INFORMATION PROVIDED

For this investigation, Park Delmas Investors, LLC provided us with the following.

- Preliminary project development information
- A geotechnical report prepared by Donald E. Banta & Associates, Inc. for the site, dated December 19, 2006

- ALTA/ACSM Land Title Survey, Delmas Avenue, Sheets 1 and 2, prepared by Civil Engineering Associates, dated November 11, 2005

- Preliminary architectural design drawings, prepared by Steinberg Architects, dated February 21, 2014

1.4 PREVIOUS GEOTECHNICAL INVESTIGATION

In 2006, Donald E. Banta & Associates (DBA) performed a geotechnical investigation on the project site and prepared a report titled "Geotechnical Report, Park/Delmas Residential, Park Avenue at Delmas Avenue, San Jose, California," dated December 19, 2006. The DBA investigation included five exploratory borings and three Cone Penetrometer Test (CPT) probes, and laboratory testing on selected soil samples collected from the borings. Information from the DBA investigation was considered during our analysis.

1.5 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to perform supplemental subsurface exploration at the site and to develop geotechnical recommendations for design and construction of the proposed project. The following work was performed.

1. Reconnoitering of the site to observe existing site conditions and to mark locations of our exploration.

2. Notifying Underground Service Alert (USA) and our client of the drilling schedule.

3. Subsurface exploration by means of two CPT probes.

4. Review of the 2006 DBA geotechnical report.

5. Engineering analysis of the collected data.

6. Preparation of this report.

2. SITE INVESTIGATION AND LABORATORY TESTING

Our field investigation consisted of a site reconnaissance and a subsurface exploration program. Observations from our site reconnaissance are described in Section 3.1 of this report. Subsurface conditions are described in Section 3.2 of this report.

2.1 SUBSURFACE EXPLORATION

Our subsurface exploration program consisted of two Cone Penetrometer Test probes (CPT-1 and CPT-2). The CPT probes were located in the field by referencing to existing site features and pacing; therefore, their locations are approximate. The approximate locations of the CPT probes are shown on Figures 1 and 2. The CPT probes were backfilled with cement grout.

2.1.1 Drill Holes

No drill holes were advanced for this investigation. Logs of the five borings from the 2006 DBA report are included in Appendix B of this report.

2.1.2 Cone Penetrometer Tests

For this investigation, CPT-1 and CPT-2 were performed by John Sarmiento & Associates on February 14, 2014, to a depth of about 45 feet bgs. CPT involves pushing a small diameter (10 cm^2 cross-sectional area) steel probe into the ground using a hydraulic jack attached to a truck mounted rig. The tip of the probe is instrumented and takes almost continuous measurements (roughly every 1 inch) of tip resistance, side friction resistance, and pore pressure. The CPT data and typical interpreted soil properties, presented at about 6-inch depth intervals, are included in Appendix A and include the following:

Symbol	Explanation
Qc	Tip bearing resistance
Qc'	Tip bearing resistance normalized for overburden
Fs	Sleeve friction resistance
Rf	Tip/sleeve friction Ratio
SPT (N)	Equivalent standard penetration blow count
SPT' (N')	Corrected equivalent standard penetration blow count
EffVtStr	Estimated effective overburden stress
PHI	Interpreted internal friction angle
Su	Interpreted undrained shear strength
Soil Behavior type	Interpreted soil behavior type
Density Range	Estimated range of total soil density

Data of the three CPTs from the 2006 DBA report are included in Appendix B of this report.

2.2 LABORATORY TESTING

No Laboratory testing was performed for this investigation. Laboratory test data from the 2006 DBA report are included in Appendix C of this report.

3. FINDINGS

3.1 SURFACE CONDITIONS

The project site is bordered by Delmas Avenue on the northeast, Park Avenue on the northwest, Sonoma Avenue on the southwest, and existing developments and West San Carlos Street on the southeast. Ground surface across the site is relatively flat. A light-rail track runs parallel to and across Delmas Avenue to the northeast of the site.

Existing surface features on the site include a one-story commercial building and associated paved parking lot on the corner of Park Avenue and Sonoma Avenue. We understand several buildings once occupied the northern and northeastern portions of the site until March-April, 2010. These structures have been demolished. Remnants of the paved parking lot still remain. There are several small to large trees, mainly in the southern portion of the site.

3.2 SUBSURFACE CONDITIONS

DBA reported loose fills in all of their five borings, consisting of fat clay, sandy fat clay, and clayey sand to depths of about 2 to 4 feet below ground surface (bgs). Native soils below the fills, as reported by DBA, consist of stiff to very stiff, high plasticity fat clay to depths of about 6 to 7 feet bgs. The fat clay is underlain by stiff to very stiff clay of intermediate plasticity to depths of 13 to 15 feet bgs. These clays are underlain by interbedded layers of medium dense clayey sands, silty sands, sandy gravel and gravelly sand, and firm to stiff clays.

Our review of the logs of the three DBA 2006 CPT probes suggests cohesive soils to a depth of about 10 feet bgs, and interbedded layers of fine and coarse grained materials to the maximum explored depth of about 80 feet bgs.

Our two CPT probes advanced for this investigation suggests predominantly cohesive soils below ground surface to a depth of about 8 feet, dense granular/stiff cohesive soils to a depth of about 14 feet, and interbedded layers of fine and coarse grained soils to the maximum explored depth of about 45 feet bgs.

For a more detailed description of the soils interpreted in our two CPT probes, refer to the CPT data sheets included in Appendix A. For logs of the borings and CPT probes performed by DBA, refer to Appendix B.

3.3 GROUNDWATER

Groundwater was measured in our CPT-1 and CPT-2 at a depth of about 18 feet below ground surface after completion of testing. Groundwater was measured by DBA in their borings between depths of 17 and 18 feet. These groundwater levels were based on direct measurement in the borings and CPT holes. DBA estimated groundwater depths of roughly 13 to 17 feet in their CPT probes based on pore pressure dissipation measurements.

Historical high groundwater at the site was estimated to be about 22 feet bgs based on our review of Plate 1.2, “Depth to historically high ground water, historical liquefaction sites and locations of boreholes, San Jose West 7.5-Minute Quadrangle, California,” Seismic Hazard Zone Report 058, prepared by California Division of mines and Geology, Department of Conservation, 2002.

Fluctuations in the groundwater level may occur due to seasonal variations in rainfall and temperature, nearby water courses, pumping from wells, regional groundwater recharge program, irrigation or other factors that were not evident at the time of this investigation.

3.4 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions, as described in this report, are based on data obtained from our subsurface exploration and the 2006 investigation performed by Donald E. Banta & Associates. Our conclusions and geotechnical recommendations are based on these interpretations. The project site has undergone different phases of development and grading; therefore, it is likely that undisclosed variations in subsurface conditions exist at the site, such as old foundations, abandoned utilities and localized areas of deep and loose fill.

Careful observations should be made during construction to verify our interpretations. Should variations from our interpretations be found, we should be notified to evaluate whether any revisions should be made to our recommendations.

4. SEISMIC CONSIDERATIONS

4.1 EARTHQUAKE FAULTS AND SEISMICITY

The San Francisco Bay Area is seismically dominated by the active San Andreas Fault system, the tectonic boundary between the northward moving Pacific Plate (west of the fault) and the North American Plate (east of the fault). This movement is distributed across a complex system of generally strike-slip, right-lateral, subparallel faults. Regional faults that have a potential to generate large magnitude earthquakes and significant ground shaking at the site are listed below. Map distances are derived from the USGS Quaternary Fault and Fold database (accessed at http://earthquake.usgs.gov/regional/qfaults/).

Fault Name	Approximate Distance	Orientation from Site
Hayward (Southeast Extension)	8¼ km	East
Monte Vista-Shannon	10¾ km	Southwest
Calaveras (Central Segment)	13¼ km	East
San Andreas (Santa Cruz Mountains)	17¾ km	Southwest
Sargent	21¾ km	South
Verona	32 km	Northeast
Greenville	36½ km	Northeast

According to the 2013 CBC and ASCE 7-10, the spectral response acceleration at any period can be taken as the lesser of the spectral response accelerations from the probabilistic and deterministic ground motion approaches. We used the US Seismic Design Maps Application at the USGS website for this purpose to retrieve seismic design parameter values for design of buildings at the subject site. Two levels of ground motions are considered in the Application: Risk-targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE), with both probabilistic and deterministic values defined in terms of maximum-direction rather than geometric-mean, horizontal spectral acceleration. The probabilistic MCE_R spectral response accelerations are represented by a 5 percent damped acceleration response spectrum having a 1 percent probability of collapse within a 50-year period and in the direction of the maximum horizontal response. The probabilistic Design Earthquake (DE) S_a value at any period can be taken as two-thirds of the MCE_R S_a value at the same period.

Using the latitude and longitude of the site (latitude 37.3281, longitude -121.8967) and a Site Class D, the calculated geometric mean peak ground acceleration adjusted for site class effects (PGA_M) is 0.5g for the MCE_G (Geometric Mean Maximum Considered Earthquake). PGA_M is for use in evaluation of soil liquefaction, lateral spreading, seismic settlements and other soil issues per ASCE 7-10.

Estimation of probabilities of major earthquakes by the Working Group on California Earthquake Probabilities (WGCEP) is now in their fourth iteration, with the greatest changes in approach being the treatment of major faults as segmented, unsegmented or capable of different rupture scenarios; in the progressive consideration of more potential seismic sources, and in use of time-independent versus time-dependent models. Current estimates (WGCEP, 2003, 2008) are most detailed for the greater San Francisco Bay Area; WGCEP (2008) estimated a 63% probability of a large (magnitude 6.7 or greater) earthquake in the San Francisco Bay area as a whole over a 30-year period; this overall probability differed only slightly from the previous (WGCEP, 2003) probability of 62%. The estimate for the Calaveras fault alone is 7% (revised down from the 11% presented by WGCEP, 2003); for the (northern) San Andreas fault alone, 21%; and for the Hayward fault, 31% (revised upward from the WGCEP (2003) value of 27%).

4.2 LIQUEFACTION

Soil liquefaction is a phenomenon in which saturated granular soils, and certain fine-grained soils, lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as from earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and non-plastic silts. Certain gravels, plastic silts, and clays are also susceptible to liquefaction. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type; 3) relative density of granular soils; 4) moisture content and plasticity of fine-grained soils; 5) overburden pressure; and 6) depth to groundwater.

The project site is located in a liquefaction hazard zone based on the USGS Liquefaction Susceptibility Map (Knudson et al, 2000), State of California Seismic Hazard Zones map for the San Jose West Quadrangle (dated February 7, 2002), and the County of Santa Clara Liquefaction Hazard zone map.

Geotechnical data from our CPT-1 and CPT-2 were used in our liquefaction analysis using the computer code CLiq version 1.7.5.27. The analysis was based on a peak ground acceleration value of 0.5g, groundwater levels of 13 and 18 feet bgs, and an earthquake moment magnitude of 7. Our analysis indicates some of the sand layers may liquefy when subject to the design earthquake. Liquefaction-induced settlement was estimated to be about 1 to 1¼ inches for groundwater at 13 feet, and about ¾ to 1 inch for groundwater at 18 feet. Case histories have shown that actual settlements could vary between 50% and 200% of the estimated settlements. The results of our liquefaction analysis are presented in Appendix D.

Potential liquefaction-induced ground settlements estimated by DBA, as reported in their 2006 report, range between roughly 0.48 and 1 inch.

4.3 SEISMIC DESIGN PARAMETERS

The following seismic design parameters were developed based on the 2013 California Building Code, ASCE 7-10, subsurface information collected during this investigation, and longitudes and latitudes of the project site. Code parameters were calculated using the US Seismic Design Maps Application Version 3.0.1 available at the USGS website.

Parameter	ASCE 7-10 Value
Site Class	D*
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5
S_s	1.5g
S_1	0.6g
S_{Ms}	1.5g
S_{M1}	0.9g
S_{Ds}	1.0g
S_{D1}	0.6g

Note: * The site would normally be Site Class F because it is underlain by potentially liquefiable soils. Because the fundamental period of vibration of the proposed structures is anticipated to be less than 0.5 second, the site class can be determined by assuming there is no liquefaction (ASCE 7-10 Section 20.3.1). Therefore, Site Class D was selected. A site-specific analysis would be required if the structure period is greater than 0.5 second.

5. DISCUSSION AND CONCLUSIONS

5.1 GENERAL

Based on the results of this study, it is our opinion the project site may be developed as discussed in this report provided the recommendations presented in this report are incorporated into the project design and construction.

Our opinions, conclusions and recommendations are based on our understanding of the proposed development, literature and data review, properties of soils encountered in subsurface exploration, laboratory test results, and engineering analyses. The geotechnical issues we have considered for this project are discussed below. Detailed recommendations for design and construction of the project are presented in the “RECOMMENDATIONS” section of this report.

5.2 SURFACE FAULT RUPTURE

The site is not located in an Alquist-Priolo Earthquake Fault Zone or in a County of Santa Clara Earthquake Fault Zone. Because no active or potentially active faults are known to cross the site, it is reasonable to conclude the risk of fault rupture across the site is low.

5.3 SEISMIC GROUND SHAKING

The site is in an area of high seismicity. Based on general knowledge of site seismicity, it should be anticipated that, during the design life of the improvements, the site will be subject to high intensity ground shaking. The proposed improvements should be designed accordingly using applicable building codes and experience of the design professionals.

5.4 LIQUEFACTION

The site is in a County of Santa Clara and State of California Liquefaction Hazard zone. The results of our liquefaction analysis indicate some of the underlying sands may liquefy when subject to the design earthquake with the groundwater at a level of 18 feet bgs. The estimated liquefaction-induced ground settlement is about ¾ to 1 inch. This potential settlement is in addition to static settlement under the building loads.

5.5 EXPANSION POTENTIAL OF SITE SOILS

The Atterberg Limits test data in the 2006 DBA report indicate the fat clay in the upper roughly 6 to 7 feet has a high plasticity which generally corresponds to a high expansion potential. The clays between depths of roughly 7 and 14 feet have an intermediate plasticity which generally corresponds to a moderate expansion potential. The proposed subterranean garage slab is anticipated to be constructed on the moderate expansion potential clays. Exterior flatwork at grade is anticipated to be constructed on the high expansion potential clay.

Expansive soils have the ability to undergo volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, perched groundwater, drought or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs or pavements supported on the expansive soil.

Potential mitigations for expansive soils include: 1) moisture conditioning and controlled compaction of the soils; 2) support structures on special foundations such as post-tensioned slabs or drilled piers and grade beams; 3) support concrete slabs-on-grade on a layer of “non-expansive” fill; and 4) lime treat expansive soils to reduce their expansive potential (although this may not be desirable in and around landscaping areas).

For this project, we have anticipated the subterranean garage slab to consist of either a structural mat slab or conventional concrete slab-on-grade (with conventional footings). To reduce the potential impact of expansive soil, concrete slabs (garage slab and exterior concrete slabs) should be constructed on a minimum 12-inch thick layer of “non-expansive” fill over a section of properly moisture conditioned and compacted on-site soil. For the garage slab, the combined thickness of “non-expansive” fill and moisture-conditioned subgrade soil should be a minimum of 18 inches below the bottom of the slab. For at-grade concrete slabs, the minimum combined thickness of “non-expansive” fill and moisture-conditioned subgrade soil should be a minimum of 24 inches below the bottom of the slabs. Refer to the “Earthwork” section of this report for recommendations.

5.6 EXISTING FILL

Fills consisting of fat clay, sandy fat clay and clayey sand were encountered to depths of about 2 to 4 feet in the DBA borings. Most of the fills will be removed for construction of the subterranean parking garage. Where fills still remain, the fills should be removed and re-compacted prior to construction of surface structures or improvements, such as flatwork or pavements. Refer to the “Earthwork” section of this report for recommendations.

5.7 GROUNDWATER

As discussed in Section 3.3 of this report, groundwater was measured at a depth of roughly 18 feet in our two CPT probes for this study. Historical highest groundwater has been reported at a depth of about 22 feet in the site vicinity. In their 2006 report, DBA reported groundwater depths of roughly 13 to 18 feet in their borings and CPT probes.

Design and construction of the project, including the subterranean parking garage and other underground improvements, should consider the groundwater depth, especially the 13-foot depth reported by DBA. If groundwater is encountered during construction, dewatering and special soil preparation may be necessary to allow construction in a dry condition and on a stable subgrade. We recommend boring(s) be performed before the start of construction to evaluate depth to groundwater at that time. Modification to the project design may be necessary depending on the encountered groundwater depth.

5.8 EXISTING IMPROVEMENTS

The project site has been developed with existing and previously existing improvements. We understand several structures have been demolished and the demolition excavations have been backfilled. For construction of the subterranean parking garage, an excavation about 10 feet in depth will be required across most of the site. This excavation will remove existing fill, backfill and underground improvements within its limits.

During design and construction of the project, the presence of existing improvements outside of the subterranean parking garage limits should be considered. Prior to the start of construction, those existing improvements should be removed and the resulting excavations should be properly backfilled.

5.9 SOIL CORROSIVITY

Two selected soil samples were tested by CERCO Analytical for general soil corrosivity during the DBA 2006 investigation. The test results and a brief report from CERCO Analytical are included in Appendix C. The project design engineers should review the information for their designs. Additional testing may be necessary if soil corrosivity at specific locations is required.

The test results may be used in conjunction with ACI 318 in the selection of concrete for use at this site, especially for concrete that will be in direct contact with soil. If necessary, a corrosion engineer may be consulted for additional recommendations on mitigation of soil corrosion.

6. RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 Clearing and Stripping

Site clearing should include removal of designated improvements, deleterious materials, debris and obstructions, including existing buildings, foundations, concrete slabs, pavements, stumps and primary roots of trees and brush. Roots about 1 inch or larger in diameter or about 3 feet or longer in length should be removed. Depressions, voids and holes that extend below proposed finish grade should be cleaned and backfilled with engineered fill compacted to the recommendations in this report.

Where excavations for removal of previously-existed structures and improvements have been backfilled, documentation proofing the backfill has been properly compacted in lifts should be provided to the geotechnical engineer for review, unless the backfill is within the zone of excavation for construction of the subterranean parking garage. Backfill that is not properly backfilled should be removed and re-compacted in lifts to the recommendations in this report.

In areas outside of the subterranean parking garage and where improvements will be constructed, surface vegetation should be stripped to sufficient depth to remove the vegetation and organic-laden topsoil. Organic laden soils are defined as soils with more than 3 percent by weight of organic content. Stripped material may be stockpiled for use in future landscape areas if approved by the project landscape architect; otherwise, it should be removed from the site. For planning purposes, average stripping depth may be assumed to be about 3 inches. The actual stripping depth should be determined by the Geotechnical Engineer at the time of construction.

6.1.2 Excavations, Temporary Construction Slopes, Shoring and Dewatering

An excavation of roughly 10 to 12 feet below ground surface is anticipated for construction of the subterranean parking garage. Excavations are also anticipated for removal of underground obstructions and for construction of the new underground utilities and foundations. The excavations should be readily accomplished with conventional earth-moving equipment, depending on the equipment wear and tear the contractor is willing to accept. The planned excavations should be constructed in accordance with the current Cal-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor. For excavations with no groundwater or seepage, the on-site clayey soils may be considered as Type B soil in OSHA 29 CFR Part 1926, Appendix A to Subpart P.

The contractor is responsible for the design, installation, maintenance and removal of temporary shoring and bracing systems. The presence of nearby existing structures, pavements, and underground utilities must be incorporated in the design of the shoring and bracing systems. If drilled piers are used as soldier piles, the presence of relatively clean sandy soils and groundwater should be taken into consideration in the design and construction of the piers. The pier holes may have to be cased to avoid caving of the pier holes.

The presence of groundwater should be considered in the design and construction of excavations. Excavations extending below groundwater will require dewatering. Dewatering should lower the groundwater level to a minimum of 2 feet below the bottom of the excavations.

The design, installation, permitting, maintenance and removal of dewatering system are the responsibility of the contractor.

Trench excavations adjacent to existing or proposed foundations should be above an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations.

6.1.3 Over-excavation and Re-compaction of Existing Fills

Fills have been reported on the project site, to depths of about 2 to 4 feet bgs. Most of the fills will be removed during construction of the subterranean parking garage. Where fills will remain, the fills should be removed and re-compacted to the requirements for engineered fill in this report. Removal and re-compaction of existing fills should extend horizontally a minimum of 3 feet beyond the outermost limits of the proposed improvements unless it is restricted by existing improvements or property line.

Soil surfaces exposed by removal of existing fills should be scarified, moisture conditioned and compacted to the recommendations under “Subgrade Preparation” before raising the areas to design grades with engineered fills.

6.1.4 Subgrade Preparation

Subgrade soil in areas to receive engineered fills, mat slab foundation, concrete slabs-on-grade and pavements should be scarified to a minimum depth of 8 inches, moisture conditioned and compacted to the recommendations given under “Engineered Fill Placement and Compaction.” Prepared soil subgrades should be non-yielding.

Subgrade preparation should extend a minimum of 3 feet beyond the outermost limits of the proposed improvements, unless it is restricted by existing improvements or property line. After the subgrades have been prepared, the areas may be raised to design grades by placement of engineered fill.

Wet soils should be anticipated during and shortly after rainy months. Where encountered, unstable, wet or soft soil will require processing before compaction can be achieved. If construction schedule does not allow for air-drying, other means such as lime or cement treatment of the soil or excavation and replacement with suitable material may be considered. Geotextile fabrics may also be used to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

6.1.5 “Non-expansive” Fill

The DBA report indicates the surficial soil has a high expansion potential and the subgrade soil for the subterranean garage slab has a moderate expansion potential. Therefore, interior and exterior concrete slabs-on-grade, including the garage mat slab foundation, should be constructed on a 12-inch minimum thick layer of “non-expansive” fill meeting the requirements in the section of “Materials for Engineered fill.” For exterior slabs, the “non-expansive” fill should extend a minimum of 1 foot horizontally beyond the limits of the slabs.

6.1.6 Materials for Engineered Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as engineered fill to achieve project grades, except when special material (such as capillary break material and "non-expansive" fill) is required. The on-site high expansion potential fat clay should not be used as engineered fill.

Engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill, including "non-expansive" fill, should have a low expansion potential as indicated by Plasticity Index of 15 or less, or Expansion Index of less than 20.

All import fills should be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

6.1.7 Engineered Fill Placement and Compaction

Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness, moisture conditioned to the required moisture content, and mechanically compacted to the recommendations below. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage. Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Soil subgrades consisting of highly or moderately expansive clays should be compacted to between 87 and 92 percent relative compaction at moisture content between 3 and 5 percent above the laboratory optimum value. Engineered fill consisting of moderately expansive on-site clays should be compacted to between 87 and 92 percent relative compaction at moisture content between 3 and 5 percent above the laboratory optimum value. Engineered fills consisting of soils of low expansion potential (including the "non-expansive" fill) should be compacted to a minimum of 90 percent relative compaction with moisture content between about 1 and 3 percent above the laboratory optimum value.

In pavement areas, the upper 8 inches of subgrade soil should be compacted to a minimum of 95 percent relative compaction. Aggregate base in vehicle pavement areas should be compacted at slightly above the optimum moisture content to a minimum of 95 percent relative compaction.

6.1.8 Utility Trench Excavation and Backfill

Pipe zone backfill, extending from the bottom of the trench to about 1 foot above the top of pipe, may consist of free-draining sand (less than 5% passing a No. 200 sieve), lean concrete or sand cement slurry. Sand, if used as bedding, should be compacted to a minimum of 90 percent relative compaction.

Above the pipe zone, utility trenches may be backfilled with on-site soil or imported soil. Trench backfill above the bedding material should be compacted to the requirements given in the section of "Engineered Fill Placement and Compaction." Trench backfill should be capped with at least 12 inches of compacted, on-site soil similar to that of the adjoining subgrade. The upper 8 inches of trench backfill in areas to be paved should be compacted to a minimum of 95 percent relative compaction. The backfill material should be placed in lifts not exceeding about 6 inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill should not be permitted.

6.1.9 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill should be compacted at moisture content meeting our recommendations. Consideration should be given to reducing the potential for water infiltration from the exterior to under the buildings through utility lines crossing the building perimeter. In utility lines crossing beneath perimeter foundations, permeable backfill should be terminated at least 1 foot outside of the perimeter foundation. Impermeable material, such as concrete or clay soil, should be used for the entire trench depth to act as a seepage cutoff.

Where concrete slabs or pavements abut against landscaped areas, the base rock layer and subgrade soil should be protected against saturation. Water if allowed to seep into the subgrade soil or pavement section could reduce the service life of the improvements. Methods that may be considered to reduce infiltration of water include: 1) subdrains installed behind curbs and slabs in landscape areas; 2) vertical cut-offs, such as a deepened curb section, or equivalent, extending at least 2 inches into the subgrade soil; and 3) use of a drip or controlled irrigation system for landscape watering.

6.1.10 Wet Weather Construction

If earthwork construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors who should be responsible to protect their work to avoid damage by rainwater. Standing water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submits a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

6.2 FOUNDATIONS

Foundations for the proposed subterranean parking garage may consist of conventional footings with a conventional concrete slab-on-grade floor, provided the estimated liquefaction-induced settlement is acceptable. Foundations for short landscaping retaining walls may consist of conventional footings. General recommendations for foundation design are presented below. The geotechnical engineer should review the foundation plans and details before construction, and observe the foundation excavations during construction to determine if the excavations extend into suitable bearing material.

Foundation excavations should be clean of loose soil and should not be allowed to dry before placement of concrete. If visible cracks appear in the foundation excavations, the excavations should be thoroughly moisture conditioned beginning at least two days prior to placement of concrete to close all cracks. It is also important that the base of the foundation excavations not be allowed to become excessively wet, resulting in soft soils. Water should not be allowed to pond in the bottom of the excavations. Areas, which become water damaged, should be over-excavated to a firm base. The over-excavated areas may be backfilled with engineered fill or lean concrete.

To maintain the desired support, the bottom of foundations adjacent to utility trenches should be below an imaginary plane having an inclination of 1.5 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent utility trenches.

6.2.1 Conventional Footings

The proposed subterranean parking garage may be supported on conventional continuous and isolated footings. Footings may also be considered for landscaping retaining walls which are expected to be 3 feet or less in height. Footings should bear on undisturbed native soil and/or properly compacted engineered fill. Footings should extend a minimum of 18 inches below pad grade or lowest adjacent finish grade, whichever provides a deeper embedment. Footings should be a minimum of 18 inches wide.

For dead plus live loads, footings may be designed using a net allowable soil bearing pressure of 2,800 pounds per square foot. This value may be increased by one-third when considering short-term loads such as wind and seismic forces. Reinforcement for the foundations should be determined by the project structural engineer.

6.2.2 Resistance to Lateral Loads

Lateral loads may be resisted by a combination of friction between the bottom of foundations and the supporting subgrade and by passive resistance acting against the vertical sides of the foundations. For foundations supported on properly compacted engineered fills or undisturbed native soils, an ultimate coefficient of friction of 0.3 may be used. For foundations poured neat against the excavation sides, an ultimate passive resistance calculated using an equivalent fluid weight of 300 pcf may be assumed for foundations above the groundwater table. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas and for the garage slab. In unpaved areas, the passive pressure can be assumed to act starting at a depth of 1 foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is placed directly against undisturbed soil or engineered fills. Voids created by the use of forms should be backfilled with property compacted engineered fill or with concrete.

6.3 CONCRETE SLABS-ON-GRADE

Concrete slabs-on-grade are expected to include the subterranean parking garage slab (with conventional footings) and exterior at-grade flatwork. Concrete slabs-on-grade should be constructed on a layer of “non-expansive” fill on properly moisture conditioned and compacted soil subgrade, as recommended in the “Earthwork” section of this report. Soil subgrades MUST be maintained in a moist condition prior to placement of concrete for the slabs. Design of reinforcement, joint spacing, etc. is the responsibility of the design engineer.

Interior concrete slabs-on-grade that will be covered with floor coverings or where vapor transmission through the slabs is undesirable should be underlain by at least 4 inches of capillary break material such as free draining, clean drain rock or 3/8 inch pea gravel. A visqueen should be placed over the capillary break material. The visqueen should be a high quality polymer at least 15 mils thick that is resistant to puncture during slab construction. Typically, the membrane and the slab are separated by 2 inches of sand; but the use of sand should be determined by the project structural engineer and/or the project architect. For the subterranean garage slab, the 6-inch thick section of sand and capillary break material may be considered as the upper 6 inches of the recommended "non-expansive" fill section.

A lower water-cement ratio (0.45 to 0.50) will help reduce the permeability of the floor slab. It should be understood that the recommended plastic membrane is not intended to waterproof the concrete slab floor. For waterproofing, the project designers and/or a flooring expert should be contacted.

Exterior concrete slabs-on-grade should be cast free from adjacent foundations or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure.

6.4 RETAINING WALLS

Retaining walls for this project include the perimeter walls of the subterranean parking garage and landscaping retaining walls. The walls of the parking garage are expected to be 10 to 12 feet high and landscaping retaining walls are expected to be 5 feet or less in height. Retaining walls will be subject to lateral pressures due to the weight of retained soil, external loads adjacent to the walls, surcharge force from earthquake shaking, and hydrostatic pressure. Lateral pressures will depend on the degree of movement the walls are allowed (or desired), the type of backfill and the method of its placement, the magnitude of external loads, and subsurface drainage provisions. Our recommendations for design of retaining walls are presented below.

Soil Pressure	Drained Backfill	Undrained Backfill
At-rest (1)	60 pcf	95 pcf
Active (2)	40 pcf	85 pcf
Seismic surcharge (3)	23 pcf	
Passive (4)	300 pcf	200 pcf

Notes:

1. Walls that can tolerate little or no movement, or walls where movement and settlement of the backfill associated with active soil condition is not desirable, should be designed using at-rest soil pressure.
2. To develop active soil pressures, wall movements of about 0.005H to 0.01H may be necessary for cohesive soils, with up to 0.005H for cohesionless soils.
3. Consider seismic surcharge as an inverted equivalent fluid pressure (inverted triangle) and apply the resultant force at 0.6H above the base of the wall (H is the total height of the wall).
4. To develop passive soil pressures, movements of up to about 0.005H may be necessary for cohesionless soils, with up to about 0.04H for cohesive soils.
5. Wall backfill should consist of granular soil or approved on-site soils of low expansion potential. Clays of high expansion potential should not be used as wall backfill.
6. Over-compaction of wall backfill should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended above. Backfill within 3 feet of the walls should be compacted with hand-operated equipment.

Pressures due to static external loads, including surface loads and loads from adjacent foundations, should be added to the soil pressures recommended above in design of the retaining walls. For a uniform vertical load at the ground surface, the additional lateral pressure on the walls should be calculated as a rectangular pressure distribution equal to the magnitude of the vertical load multiplied by a factor of 0.33 for active soil condition and 0.5 for at-rest soil condition.

To achieve a drained backfill condition, a subsurface drain should be installed behind each retaining wall extending from the wall bottom to about 1 to 2 feet below finished grade. The drain should consist of a 12-inch minimum wide blanket of drainage material consisting of either Class 2 Permeable material (Caltrans Standard Specifications, Section 68) or clean, 1/2 to 3/4-inch maximum size crushed rock or gravel. If crushed rock or gravel is used, it should be encapsulated in a geotextile filter fabric, such as Mirafi 140N or equivalent. Filter fabric is optional if Class 2 Permeable material is used. The top 2 feet below finish grade should be backfilled with compacted clayey soil to reduce infiltration of surface water. Alternatively, pre-fabricated drainage panel, such as Mirafi G100W or equivalent, may be considered.

A 4-inch minimum diameter, perforated, schedule 40 PVC (or equivalent) pipe should be installed (with perforations facing down) along the base of each wall on a 2-inch thick bed of drain rock. The pipes should be sloped to drain by gravity to a proper collection system and be discharged at a proper outlet as designed by the project Civil Engineer.

Lateral soil pressures for undrained backfill should be used if subsurface drainage is not provided behind the retaining walls or if the walls are below design groundwater level.

6.5 SURFACE AND SUBSURFACE DRAINAGE

Engineering design of grading and drainage at the site is the responsibility of the project Civil Engineer. We suggest the following for consideration by the project Civil Engineer, as appropriate.

Sufficient surface drainage should be provided to direct water away from buildings, foundations, concrete slabs-on-grade and pavements, and towards suitable collection and discharge facilities. Ponding of surface water should be avoided by establishing positive drainage away from all improvements.

Over-watering could result in soil saturation and subsequent distress to site improvements. Trees should be planted away from structures, foundations, concrete slabs, utilities, pavements, etc. because tree roots could cause distress to those improvements. A qualified engineer and/or landscape architect should be consulted.

7. PLAN REVIEW, EARTHWORK AND FOUNDATION OBSERVATION

Post-report geotechnical services by Pacific Geotechnical Engineering (PGE), typically consisting of pre-construction design consultations and reviews, construction observation and testing services, are necessary for PGE to confirm the recommendations contained in this report. This report is based on limited sampling and investigation, and by those constraints may not have discovered local anomalies or other varying conditions that may exist on the project site. Therefore, this report is only preliminary until PGE can confirm that actual conditions in the ground conform to those anticipated in the report. Accordingly, as an integral part of this report, PGE recommends post-report geotechnical services to assist the project team during design and construction of the project. PGE requires that it perform these services if it is to remain as the project geotechnical engineer-of-record.

During design, PGE can provide consultation and supplemental recommendations to assist the project team in design and value engineering, especially if the project design has been modified after completion of our report. It is impossible for us to anticipate every design scenario and use of construction materials during preparation of our report. Therefore, retaining PGE to provide post-report consultation will help address design changes, answer questions and evaluate alternatives proposed by the project designers and contractors.

Prior to issuing project plans and specifications for construction bidding purposes, PGE should review the grading, drainage and foundation plans and the project specifications to determine if the intent of our recommendations has been incorporated in these documents. We have found that such a review process will help reduce the likelihood of misinterpretation of our recommendations which may cause construction delay and additional cost.

Construction phase services can include, among other things, the observation and testing during site clearing, stripping, excavation, mass grading, subgrade preparation, fill placement and compaction, backfill compaction, foundation construction and pavement construction activities.

Pacific Geotechnical Engineering would be pleased to provide cost proposals for follow-up geotechnical services. Post-report geotechnical services may include additional field and laboratory services.

8. LIMITATIONS

In preparing the findings and professional opinions presented in this report, we have endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were performed. No warranty, express or implied, is provided.

The conclusions and recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept or general location and type of structures are modified, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations. To remain as the project geotechnical engineer-of-record, PGE must be retained to provide geotechnical services as discussed under the Post-report Geotechnical Services section of this report.

Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those described in this report be encountered during project development, PGE should be consulted to review the conditions and determine whether our recommendations are still valid. Additional exploration, testing, and analysis may be required for such evaluation.

Should persons concerned with this project observe geotechnical features or conditions at the site or surrounding areas which are different from those described in this report, those observations should be reported immediately to Pacific Geotechnical Engineering for evaluation.

It is important that the information in this report be made known to the design professionals involved with the project, that our recommendations be incorporated into project drawings and documents, and that the recommendations be carried out during construction by the contractor and subcontractors. It is not the responsibility of Pacific Geotechnical Engineering to notify the design professionals and the project contractors and subcontractors.

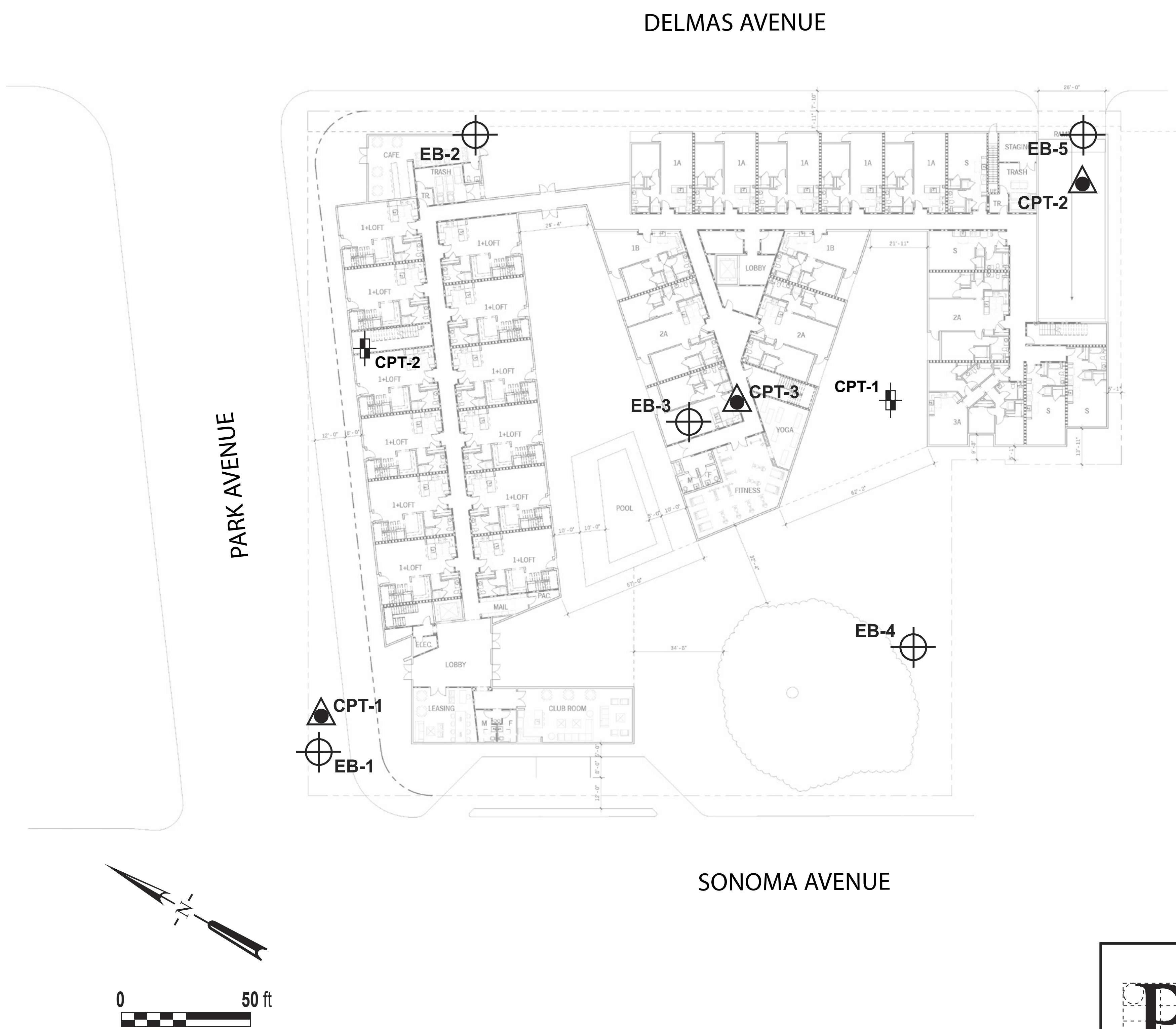
The findings, conclusions and recommendations presented in this report are applicable only to the specific project development on this specific site. These data should not be used for other projects, sites or purposes unless they are reviewed by PGE or a qualified geotechnical professional.

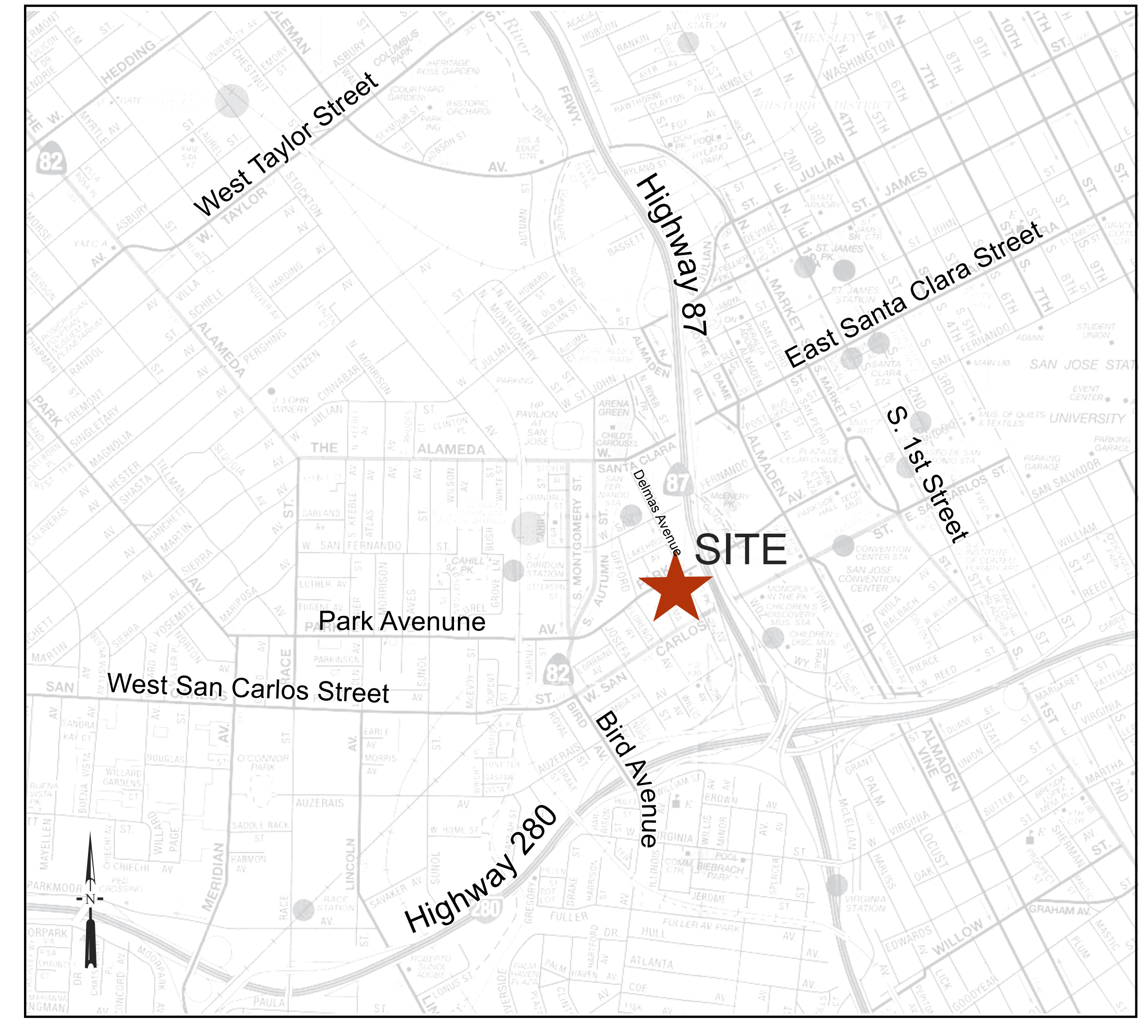
Report prepared by,

Pacific Geotechnical Engineering

Chalerm (Beeson) Liang
GE 2031

Distribution: Park Delmas Investors, LLC, Mr. Dominic Boitano (6)





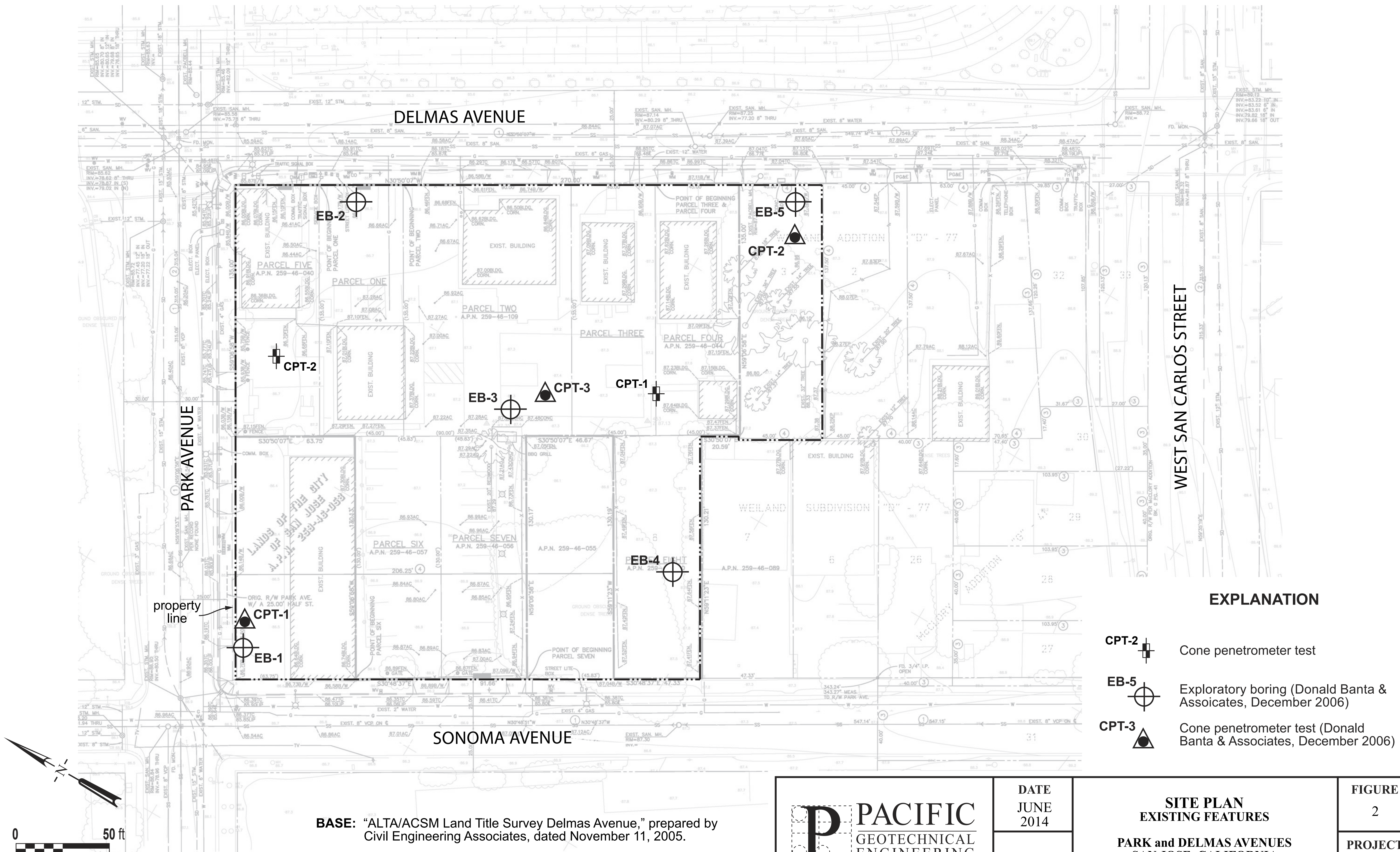
VICINITY MAP - no scale

EXPLANATION

CPT-2	Cone penetrometer test
EB-5	Exploratory boring (Donald Banta & Assoicates, December 2006)
CPT-3	Cone penetrometer test (Donald Banta & Associates, December 2006)

BASE: "Level 1 - Podium Plan," prepared by Steinburg Architects, dated February 22, 2014.

PACIFIC GEOTECHNICAL ENGINEERING	DATE JUNE 2014	SITE PLAN PROPOSED IMPROVEMENTS PARK and DELMAS AVENUES SAN JOSE, CALIFORNIA	FIGURE 1 PROJECT 2014.0039



BASE: "ALTA/ACSM Land Title Survey Delmas Avenue," prepared by Civil Engineering Associates, dated November 11, 2005.

PACIFIC GEOTECHNICAL ENGINEERING	DATE: JUNE 2014	SITE PLAN EXISTING FEATURES PARK and DELMAS AVENUES SAN JOSE, CALIFORNIA	FIGURE 2 PROJECT 2014.0039

APPENDIX A

CPT DATA

PROJECT: PARK AND DELMAS SITE
LOCATION: San Jose CA
PROJ. NO.: 2014.0039(PGE-26)
Terminated at 45.0 feet

CPT NO.: CPT-1
DATE: 02-13-2014
TIME: 10:47:00
Groundwater estimated at 18.5 feet

PACIFIC GEOTECHNICAL
cpts by John Sarmiento & Associates

DEPTH (feet)	Qc (tsf)	Qc' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0.55	24.0	38.40	0.62	2.6	12	19	0.06	----	3.20	Clayey SILT to Silty CLAY	120-130
1.04	30.3	48.4	0.90	3.0	15	24	0.13	----	4.02	"	130-140
1.53	28.4	45.49	0.99	3.5	14	23	0.20	----	3.78	"	"
2.03	19.5	31.20	0.73	3.8	13	21	0.26	----	2.58	Silty CLAY to CLAY	120-130
2.55	26.8	42.83	1.14	4.3	18	29	0.33	----	3.55	"	130-140
3.07	28.9	46.19	1.65	5.7	29	46	0.40	----	3.82	CLAY	"
3.57	27.2	43.58	1.58	5.8	27	44	0.47	----	3.60	"	"
4.01	22.2	35.47	1.33	6.0	22	35	0.53	----	2.92	"	"
4.52	22.0	35.17	1.55	7.0	22	35	0.59	----	2.89	"	"
5.03	19.1	30.58	1.53	8.0	19	31	0.66	----	2.50	"	"
5.54	24.0	38.38	1.61	6.7	24	38	0.73	----	3.15	"	"
6.03	28.3	45.28	1.80	6.4	28	45	0.80	----	3.72	"	"
6.51	34.5	53.06	2.27	6.6	35	53	0.86	----	4.55	"	"
7.05	36.7	53.65	2.50	6.8	37	54	0.94	----	4.82	"	"
7.51	39.2	54.91	2.57	6.6	39	55	1.00	----	5.16	"	"
8.06	52.4	71.41	2.79	5.3	52	71	1.07	----	6.92	"	"
8.53	59.0	78.43	2.84	4.8	39	52	1.14	----	7.79	Silty CLAY to CLAY	"
9.07	61.5	79.41	3.43	5.6	61	79	1.21	----	8.12	Very Stiff Fine Grained *	"
9.52	51.6	65.11	3.10	6.0	52	65	1.27	----	6.80	CLAY	"
10.04	46.7	57.31	2.84	6.1	47	57	1.34	----	6.14	"	"
10.55	51.2	61.12	2.73	5.3	51	61	1.41	----	6.73	"	"
11.07	62.4	72.41	3.04	4.9	62	72	1.48	----	8.22	Very Stiff Fine Grained *	"
11.51	76.8	87.22	2.97	3.9	38	44	1.54	----	10.13	Clayey SILT to Silty CLAY	"
12.02	78.0	86.71	4.50	5.8	78	87	1.61	----	10.29	Very Stiff Fine Grained *	"
12.53	135.3	146.97	4.69	3.5	54	59	1.68	----	17.93	Sandy SILT to Clayey SILT	"
13.03	152.4	161.83	5.37	3.5	76	81	1.74	41	----	SAND to Clayey SAND *	"
13.51	126.3	132.04	5.00	4.0	126	132	1.81	----	16.72	Very Stiff Fine Grained *	>140
14.06	81.5	83.69	3.68	4.5	81	84	1.89	----	10.73	"	130-140
14.50	43.4	44.00	1.67	3.9	22	22	1.94	----	5.66	Clayey SILT to Silty CLAY	"
15.05	16.9	16.93	1.23	7.2	17	17	2.02	----	2.12	CLAY	"
15.52	48.6	48.52	1.38	2.8	19	19	2.08	----	6.34	Sandy SILT to Clayey SILT	"
16.07	97.4	97.10	2.17	2.2	32	32	2.16	38	----	Silty SAND to Sandy SILT	"
16.53	140.0	139.37	1.91	1.4	35	35	2.22	40	----	SAND to Silty SAND	"
17.00	135.0	134.24	2.31	1.7	34	34	2.28	40	----	"	"
17.54	75.9	75.32	2.41	3.2	30	30	2.36	----	9.96	Sandy SILT to Clayey SILT	"
18.02	44.8	44.42	1.80	4.0	22	22	2.42	----	5.81	Clayey SILT to Silty CLAY	"
18.51	129.6	128.37	1.93	1.5	32	32	2.46	39	----	SAND to Silty SAND	"
19.07	61.2	60.57	2.57	4.2	31	30	2.50	----	7.99	Clayey SILT to Silty CLAY	"
19.54	73.0	71.57	3.38	4.6	73	72	2.53	----	9.56	Very Stiff Fine Grained *	"
20.01	44.9	43.50	2.28	5.1	45	44	2.56	----	5.81	CLAY	"
20.58	11.4	10.94	0.68	6.0	11	11	2.60	----	1.68	"	120-130
21.06	12.8	12.09	0.71	5.5	13	12	2.63	----	1.51	"	"
21.54	12.2	11.47	0.82	6.7	12	11	2.66	----	1.44	"	"
22.02	74.4	68.86	1.76	2.4	25	23	2.69	36	----	Silty SAND to Sandy SILT	130-140
22.51	104.7	95.68	1.91	1.8	35	32	2.73	38	----	"	"
23.08	137.0	123.30	1.87	1.4	34	31	2.77	39	----	SAND to Silty SAND	"
23.52	212.4	189.61	1.42	0.7	42	38	2.80	42	----	SAND	110-120
24.09	83.8	73.64	1.72	2.1	28	25	2.84	36	----	Silty SAND to Sandy SILT	130-140
24.57	20.4	17.67	1.29	6.3	20	18	2.87	----	2.50	CLAY	"
25.04	8.0	6.90	0.43	5.3	8	7	2.90	----	1.27	"	110-120
25.54	7.9	6.76	0.39	4.9	8	7	2.92	----	1.25	"	"
26.03	9.3	7.84	0.44	4.8	9	8	2.95	----	1.26	"	"
26.53	10.5	8.70	0.50	4.8	10	9	2.98	----	1.45	"	120-130
27.03	9.9	8.15	0.47	4.8	10	8	3.01	----	1.35	"	110-120

PROJECT: PARK AND DELMAS SITE
LOCATION: San Jose CA
PROJ. NO.: 2014.0039(PGE-26)
Terminated at 45.0 feet

CPT NO.: CPT-1
DATE: 02-13-2014
TIME: 10:47:00
Groundwater estimated at 18.5 feet

PACIFIC GEOTECHNICAL
cpts by John Sarmiento & Associates

DEPTH (feet)	Qc (tsf)	Qc' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
27.53	10.3	8.43	0.46	4.5	10	8	3.03	----	1.41	CLAY	110-120
28.06	8.9	7.31	0.38	4.3	9	7	3.06	----	1.42	"	"
28.55	7.7	6.27	0.30	3.9	8	6	3.09	----	1.16	"	"
29.05	14.1	11.45	0.59	4.2	14	11	3.12	----	1.63	"	120-130
29.55	13.6	11.01	0.55	4.0	14	11	3.15	----	1.56	"	"
30.05	10.8	8.67	0.34	3.1	7	6	3.17	----	1.47	Silty CLAY to CLAY	110-120
30.55	10.4	8.33	0.37	3.6	10	8	3.20	----	1.40	CLAY	"
31.05	9.5	7.61	0.80	8.4	10	8	3.23	----	1.25	"	120-130
31.54	45.1	35.81	1.76	3.9	23	18	3.27	----	5.74	Clayey SILT to Silty CLAY	130-140
32.03	15.0	11.84	0.57	3.8	10	8	3.30	----	1.72	Silty CLAY to CLAY	120-130
32.54	20.7	16.30	1.00	4.8	21	16	3.33	----	2.48	CLAY	130-140
33.03	9.6	7.55	0.38	3.9	10	8	3.36	----	1.25	"	110-120
33.54	10.3	8.05	0.33	3.2	7	5	3.39	----	1.36	Silty CLAY to CLAY	"
34.04	16.4	12.74	0.72	4.4	16	13	3.42	----	1.89	CLAY	120-130
34.52	19.1	14.76	1.01	5.3	19	15	3.45	----	2.25	"	130-140
35.02	17.7	13.62	1.02	5.8	18	14	3.49	----	2.06	"	"
35.52	14.7	11.27	0.91	6.2	15	11	3.52	----	1.66	"	120-130
36.01	20.1	15.29	0.96	4.8	20	15	3.56	----	2.37	"	130-140
36.51	26.1	19.73	1.36	5.2	26	20	3.59	----	3.16	"	"
37.01	22.0	16.58	0.92	4.2	15	11	3.63	----	2.62	Silty CLAY to CLAY	"
37.50	9.0	6.72	0.36	4.0	9	7	3.66	----	1.30	CLAY	110-120
38.03	7.9	5.89	0.19	2.4	5	4	3.68	----	1.08	Silty CLAY to CLAY	100-110
38.52	49.0	36.38	1.76	3.6	24	18	3.71	----	6.20	Clayey SILT to Silty CLAY	130-140
39.04	236.9	174.98	4.30	1.8	47	35	3.75	41	----	SAND	"
39.56	224.8	165.08	3.26	1.5	45	33	3.79	41	----	"	"
40.01	208.5	152.45	2.42	1.2	42	30	3.82	40	----	"	120-130
40.56	190.6	138.62	2.07	1.1	38	28	3.85	40	----	"	"
41.00	208.6	150.92	3.04	1.5	42	30	3.88	40	----	"	130-140
41.53	245.1	176.22	2.98	1.2	49	35	3.92	41	----	"	"
42.03	333.5	238.37	3.86	1.2	67	48	3.96	43	----	"	"
42.55	328.6	233.69	3.41	1.0	66	47	3.99	43	----	"	120-130
43.06	297.1	210.52	1.84	0.6	50	35	4.02	42	----	Gravelly SAND to SAND	110-120
43.54	311.2	219.69	2.76	0.9	62	44	4.05	43	----	SAND	120-130
44.06	200.4	140.85	2.87	1.4	40	28	4.09	40	----	"	130-140
44.55	26.3	18.37	1.82	7.0	26	18	4.12	----	3.11	CLAY	"
45.04	56.1	39.05	1.59	2.8	22	16	4.16	----	7.08	Sandy SILT to Clayey SILT	"

DEPTH = Sampling interval (~0.1 feet)
Qc = Tip bearing uncorrected   Qt = Tip bearing corrected   Fs = Sleeve friction resistance   Rf = Qt / Fs
SPT = Equivalent Standard Penetration Test   Qt' and SPT' = Qt and SPT corrected for overburden
EffVtStr = Effective Vertical Stress using est. density**   Phi = Soil friction angle*
Su = Undrained Soil Strength* (see classification chart)
References: * Robertson and Campanella, 1988   **Olsen, 1989   *** Durgunoglu & Mitchell, 1975

PROJECT: PARK AND DELMAS SITE
LOCATION: San Jose CA
PROJ. NO.: 2014.0039(PGE-26)
Terminated at 45.0 feet

CPT NO.: CPT-2
DATE: 02-13-2014
TIME: 10:07:00
Groundwater measured at 18.4 feet

PACIFIC GEOTECHNICAL
cpts by John Sarmiento & Associates

DEPTH (feet)	Qc (tsf)	Qc' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0.55	19.7	31.55	0.05	0.2	7	11	0.06	31	----	Silty SAND to Sandy SILT	85-90
1.06	22.7	36.4	0.43	1.9	9	15	0.13	----	3.02	Sandy SILT to Clayey SILT	120-130
1.53	29.0	46.37	0.74	2.5	12	19	0.19	----	3.85	"	130-140
2.04	26.3	42.00	0.65	2.5	10	17	0.25	----	3.48	"	120-130
2.56	20.3	32.50	0.46	2.3	10	16	0.32	----	2.69	Clayey SILT to Silty CLAY	"
3.06	25.3	40.46	0.76	3.0	13	20	0.39	----	3.35	"	130-140
3.57	27.5	43.97	0.79	2.9	14	22	0.45	----	3.63	"	"
4.00	29.5	47.14	1.05	3.6	15	24	0.51	----	3.89	"	"
4.51	32.6	52.18	1.94	6.0	33	52	0.58	----	4.31	CLAY	"
5.02	28.0	44.83	1.98	7.1	28	45	0.65	----	3.69	"	"
5.52	25.5	40.86	1.68	6.6	26	41	0.72	----	3.36	"	"
6.03	36.0	57.58	1.97	5.5	36	58	0.79	----	4.75	"	"
6.50	38.1	58.96	2.42	6.4	38	59	0.85	----	5.02	"	"
7.07	37.3	54.95	2.70	7.2	37	55	0.93	----	4.91	"	"
7.57	41.8	58.82	2.60	6.2	42	59	0.99	----	5.51	"	"
8.06	55.2	75.53	2.58	4.7	37	50	1.06	----	7.29	Silty CLAY to CLAY	"
8.53	57.9	77.35	3.02	5.2	58	77	1.12	----	7.65	Very Stiff Fine Grained *	"
9.01	52.7	68.58	3.23	6.1	53	69	1.19	----	6.95	CLAY	"
9.57	67.7	85.46	3.57	5.3	68	85	1.26	----	8.94	Very Stiff Fine Grained *	"
10.04	89.9	110.81	4.35	4.8	90	111	1.33	----	11.90	"	"
10.51	122.4	146.94	5.53	4.5	122	147	1.39	----	16.22	"	>140
11.04	142.9	166.59	6.26	4.4	143	167	1.47	----	18.96	"	"
11.56	106.4	120.86	4.70	4.4	106	121	1.54	----	14.08	"	130-140
12.00	102.0	113.75	4.48	4.4	102	114	1.60	----	13.50	"	"
12.54	138.7	150.92	3.70	2.7	46	50	1.67	40	----	Silty SAND to Sandy SILT	"
13.05	239.4	254.70	2.94	1.2	48	51	1.74	43	----	SAND	"
13.57	216.0	225.90	3.18	1.5	43	45	1.81	43	----	"	"
14.01	307.2	316.89	3.68	1.2	61	63	1.87	45	----	"	"
14.53	338.2	343.16	5.78	1.7	68	69	1.94	45	----	"	"
15.03	263.8	263.75	4.42	1.7	53	53	2.01	44	----	"	"
15.55	299.1	298.68	3.96	1.3	60	60	2.08	44	----	"	"
16.04	257.0	256.27	3.93	1.5	51	51	2.14	43	----	"	"
16.56	239.9	238.83	3.81	1.6	48	48	2.21	43	----	"	"
17.02	193.5	192.47	2.70	1.4	39	38	2.28	42	----	"	"
17.54	247.0	245.33	1.81	0.7	49	49	2.33	43	----	"	110-120
18.02	159.5	158.19	2.01	1.3	32	32	2.40	41	----	"	130-140
18.58	54.0	53.48	2.34	4.3	27	27	2.44	----	7.03	Clayey SILT to Silty CLAY	"
19.08	15.9	15.79	0.75	4.7	16	16	2.47	----	1.96	CLAY	120-130
19.50	65.3	64.63	1.39	2.1	22	22	2.50	35	----	Silty SAND to Sandy SILT	130-140
20.00	68.6	67.04	2.81	4.1	34	34	2.54	----	8.97	Clayey SILT to Silty CLAY	"
20.51	58.6	56.60	1.78	3.0	23	23	2.58	----	7.64	Sandy SILT to Clayey SILT	"
21.09	12.0	11.40	1.18	9.9	12	11	2.61	----	1.76	CLAY	120-130
21.51	71.0	66.97	2.01	2.8	28	27	2.64	----	9.28	Sandy SILT to Clayey SILT	130-140
22.02	33.7	31.40	1.52	4.5	22	21	2.68	----	4.30	Silty CLAY to CLAY	"
22.52	84.2	77.34	1.44	1.7	28	26	2.72	37	----	Silty SAND to Sandy SILT	"
23.06	180.2	163.84	1.14	0.6	36	33	2.74	41	----	SAND	110-120
23.56	121.1	108.91	1.51	1.2	30	27	2.78	38	----	SAND to Silty SAND	120-130
24.07	75.1	66.63	2.87	3.8	38	33	2.81	----	9.80	Clayey SILT to Silty CLAY	130-140
24.51	41.7	36.52	1.74	4.2	21	18	2.84	----	5.34	"	"
25.02	15.2	13.15	1.29	8.5	15	13	2.88	----	1.81	CLAY	"
25.52	8.5	7.31	0.44	5.2	9	7	2.91	----	1.37	"	110-120
26.04	8.9	7.54	0.57	6.4	9	8	2.94	----	1.44	"	120-130
26.56	8.7	7.28	0.56	6.4	9	7	2.97	----	1.39	"	110-120
27.00	11.3	9.33	0.72	6.4	11	9	3.00	----	1.59	"	120-130

PROJECT: PARK AND DELMAS SITE
LOCATION: San Jose CA
PROJ. NO.: 2014.0039(PGE-26)
Terminated at 45.0 feet

CPT NO.: CPT-2
DATE: 02-13-2014
TIME: 10:07:00
Groundwater measured at 18.4 feet

PACIFIC GEOTECHNICAL
cpts by John Sarmiento & Associates

DEPTH (feet)	Qc (tsf)	Qc' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
27.53	10.5	8.60	0.80	7.7	10	9	3.03	----	1.44	CLAY	120-130
28.06	7.2	5.89	0.61	8.4	7	6	3.06	----	1.07	"	110-120
28.53	8.1	6.58	0.56	6.9	8	7	3.08	----	1.24	"	"
29.05	12.2	9.91	0.69	5.7	12	10	3.11	----	1.37	"	120-130
29.57	11.9	9.64	0.62	5.2	12	10	3.15	----	1.67	"	"
30.08	9.3	7.46	0.50	5.4	9	7	3.17	----	1.22	"	110-120
30.50	7.0	5.59	0.40	5.7	7	6	3.19	----	1.00	"	"
31.01	35.0	27.94	2.34	6.7	35	28	3.23	----	4.40	"	130-140
31.50	9.2	7.34	0.94	10.2	9	7	3.26	----	1.20	Organic Material	120-130
32.07	8.0	6.34	0.49	6.1	8	6	3.29	----	1.19	CLAY	110-120
32.58	7.7	6.10	0.35	4.6	8	6	3.32	----	1.13	"	"
33.01	7.8	6.09	0.35	4.5	8	6	3.34	----	1.12	"	"
33.52	12.2	9.52	0.76	6.2	12	9	3.37	----	1.33	"	120-130
34.03	14.0	10.90	0.79	5.7	14	11	3.41	----	1.57	"	"
34.58	20.1	15.54	1.17	5.8	20	15	3.45	----	2.38	"	130-140
35.07	215.0	165.55	2.44	1.1	43	33	3.48	41	----	SAND	120-130
35.53	270.0	206.88	3.85	1.4	54	41	3.51	42	----	"	130-140
36.07	272.9	207.92	3.79	1.4	55	42	3.55	42	----		"
36.53	182.7	138.46	2.90	1.6	46	35	3.58	40	----	SAND to Silty SAND	"
37.06	149.3	112.48	2.52	1.7	37	28	3.62	39	----	"	"
37.57	178.5	133.71	2.97	1.7	45	33	3.66	40	----	"	"
38.07	318.3	237.16	4.29	1.4	64	47	3.69	43	----	SAND	"
38.53	317.3	235.21	4.49	1.4	63	47	3.73	43	----	"	"
39.03	241.1	177.74	2.88	1.2	48	36	3.76	41	----	"	"
39.53	159.6	117.10	1.36	0.9	32	23	3.80	39	----	"	120-130
40.05	213.9	155.98	2.91	1.4	43	31	3.83	41	----	"	130-140
40.55	258.9	188.02	1.86	0.7	52	38	3.86	42	----	"	110-120
41.06	276.0	199.59	1.51	0.6	46	33	3.89	42	----	Gravelly SAND to SAND	"
41.53	265.0	190.72	2.57	1.0	53	38	3.92	42	----	SAND	120-130
42.04	264.0	189.22	1.96	0.7	53	38	3.94	42	----	"	110-120
42.57	255.3	182.16	1.85	0.7	51	36	3.97	41	----	"	"
43.08	297.9	211.33	4.07	1.4	60	42	4.01	42	----	"	130-140
43.50	269.1	190.29	2.65	1.0	54	38	4.03	42	----	"	120-130
44.03	253.9	178.72	3.20	1.3	51	36	4.07	41	----	"	130-140
44.52	219.1	153.68	1.95	0.9	44	31	4.10	40	----	"	120-130
45.04	323.7	226.00	5.02	1.6	65	45	4.14	43	----	"	130-140

DEPTH = Sampling interval (~0.1 feet)
Qc = Tip bearing uncorrected   Qt = Tip bearing corrected   Fs = Sleeve friction resistance   Rf = Qt / Fs
SPT = Equivalent Standard Penetration Test   Qt' and SPT' = Qt and SPT corrected for overburden
EffVtStr = Effective Vertical Stress using est. density**   Phi = Soil friction angle*
Su = Undrained Soil Strength* (see classification chart)
References: * Robertson and Campanella, 1988   **Olsen, 1989   *** Durgunoglu & Mitchell, 1975

APPENDIX B

LOGS OF BORINGS AND CPT PROBES FROM 2006 DONALD E BANTA & ASSOCIATES GEOTECHNICAL INVESTIGATION

Unified Soil Classification System

PRIMARY DIVISIONS				GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS LARGER THAN # 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN # 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
			GRAVELS WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
				GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
		SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN # 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 # 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 and silty clays of high plasticity, fat clays
				OH	Organic clays and silts of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				Pt	Peat and other highly organic soils

DEFINITION OF TERMS

CLEAR SQUARE SIEVE OPENINGS

75 µm | 425 µm | 2 mm | 4.75 mm | 3/4" | 3" | 12"

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

#200 | #40 | #10 | #4 ← American Standard Sieve Sizes

GRAIN SIZES

SANDS	BLOWS / FOOT†
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS	STRENGTH*	BLOWS / FOOT†
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	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" I.D.) split spoon sampler.
*Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by pocket penetrometer, torvane, or visual observation.

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Consulting Geotechnical Engineers

KEY TO EXPLORATORY BORING LOGS

PARK/DELMAS RESIDENTIAL

San Jose, California

575-40 | December 2006 | Figure A-1

Drill Rig Hollow Flight Auger	Surface Elevation ~ 86.5 feet	Logged By GC
Groundwater Depth ~ 17 feet	Boring Diameter 8 inches	Date Drilled 10/12/06

DESCRIPTION AND CLASSIFICATION / SAMPLE DATA

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	Depth (Feet)	SAMPLER	Blows Per Foot	Percent Moisture	Dry Density (Pcf)	Plasticity Index / Liquid Limit (%)	Percent Passing #200/#4 Sieve	Shear* Strength (Ksf)
2.5 inches of Asphalt											
SANDY CLAY — Possible Fill	dark brown	firm	CL	1							
SILTY CLAY	black with white mottling	stiff to very stiff	CH	2	X	9	32			74/--	3.0(p)
				4–5		18	25	94		81/--	3.0(p)
				6		27	22			82/--	2.2(p)
SILTY CLAY, with fine sand	light brown and gray	stiff	CL	9–10		14	23	98	19/37	79/--	2.0(p)
CLAYEY SAND, with gravel	gray-brown	medium-dense	SC	14–15		29	13	124		28/--	
— ? — ? —				17			▽ (ATD)				
SILTY CLAY - CLAYEY SILT	brown	stiff	CL/ML	18							
SILTY SAND	brown	medium-dense	SM	20	X	19	20			30/--	
				24	X		27			24/--	
SILTY CLAY	black	stiff	CH	24–25	X	8	34			92/--	1.2(t)

Bottom of Boring = 25.0 feet

Note: "(t)" indicates shear strength by Torvane.

"(p)" indicates shear strength by pocket penetrometer.

Depth scale: 1–30 feet

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EXPLORATORY BORING LOG 1

PARK/DELMAS RESIDENTIAL
San Jose, California

575-40 December 2006 Sheet 1 of 1

Drill Rig Hollow Flight Auger	Surface Elevation ~ 86.5 feet	Logged By GC
Groundwater Depth ~ 17 feet	Boring Diameter 8 inches	Date Drilled 10/12/06

DESCRIPTION AND CLASSIFICATION				Depth (Feet)	SAMPLER	SAMPLE DATA					
DESCRIPTION AND REMARKS	COLOR	CONSIS-TENCY	SOIL TYPE			Blows Per Foot	Percent Moisture	Dry Density (Pcf)	Plasticity Index / Liquid Limit (%)	Percent Passing #200/#4 Sieve	Shear* Strength (Ksf)
1.75 inches of Asphaltic Concrete over 4 inches of Aggregate Base											
SILTY CLAY, mixed with gravel and wood FILL	dark brown	loose	CL/CH	1–2	x	4	19			50/--	
SILTY CLAY	dark brown to black	stiff	CH	3–6		14	38	82	50/72	83/--	3.0(p)
— — ?— — — — — — —? — — (boundary)				7							
SILTY CLAY - CLAYEY SILT	light brown to gray	stiff	CL/ML	7–10		12	24	103		80/--	
				11–15	x	Push	16	117		40/--	
CLAYEY SANDS, with gravel	gray brown	medium-dense	SC	16–17			▽ (ATD)				
— — ?— — — — — — —? — — (boundary)				18							
SILTY SAND	light brown	medium-dense	SM	19	x		21			27/--	
					x	11	27			84/--	1.1(t)
SILTY CLAY - CLAYEY SILT	light brown and gray	stiff	CL/ML	20–22							
SILTY SAND	light brown and gray	medium-dense	SM	23–25		31	22	105			
Bottom of Boring = 25.0 feet				26–30							

Note: "(t)" indicates shear strength by Torvane.

"(p)" indicates shear strength by pocket penetrometer.

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EXPLORATORY BORING LOG 2

PARK/DELMAS RESIDENTIAL
San Jose, California

575-40 December 2006 Sheet 1 of 1

Drill Rig Hollow Flight Auger	Surface Elevation ~ 87.4 feet	Logged By GC
Groundwater Depth Not Established	Boring Diameter 8 inches	Date Drilled 10/12/06

DESCRIPTION AND CLASSIFICATION / SAMPLE DATA

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	Depth (Feet)	SAMPLER	Blows Per Foot	Percent Moisture	Dry Density (Pcf)	Plasticity Index / Liquid Limit (%)	Percent Passing #200/#4 Sieve	Shear* Strength (Ksf)
2 inches of Asphaltic Concrete over 4 inches of Aggregate Base											
SANDY CLAY, with scattered debris and organics FILL	brown	loose	CL/SC	1–3	X	3	30			54/88	
SANDY CLAY	dark brown	stiff	CL	4–5		11	26	91		76/--	2.2(p)
— — — ?– — — — — — — —? — — —				6							
SILTY CLAY	light brown and gray	stiff	CL	7–10		16	21	105	21/38	80/--	3.5(p)
				11–13	X	Push	27	96		92/--	1.4(p)
CLAYEY SAND	brown	medium-dense	SC	14–16	X	13	14			32/92	
— — — ?– — — — — — — —? — — —				17							
SILTY SAND, with scattered gravel	brown	medium-dense	SM	18–19		29	18	109		32/100	
— — — ?– — — — — — — —? — — —				20							
SILTY CLAY - CLAYEY SILT	brown	stiff	CL/ML	21–23							
SILTY SAND	brown	medium-dense	SM	24–26	X	25	21			17/99	
— — — ?– — — — — — — —? — — —				27							
SILTY CLAY, with very fine sand	gray	firm	CL	28–30	X	5	28		16/34	83/--	0.9(t)

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EXPLORATORY BORING LOG 3

PARK/DELMAS RESIDENTIAL
San Jose, California

575-40 December 2006 Sheet 1 of 2

Drill Rig Hollow Flight Auger	Surface Elevation ~ 87.4 feet	Logged By GC
Groundwater Depth Not Established	Boring Diameter 8 inches	Date Drilled 10/12/06

DESCRIPTION AND REMARK	COLOR	CONSISTENCY	SOIL TYPE	Depth (Feet)	SAMPLER	Blows Per Foot	Percent Moisture	Dry Density (Pcf)	Plasticity Index / Liquid Limit (%)	Percent Passing #200/#4 Sieve	Shear* Strength (Ksf)
SILTY CLAY (continued)	gray and brown	firm	CL	31							
				32							
				33							
				34							
				35	x	13	30			77/--	.08(T) 1.1(P)
				36							
— — — ?— — — — — — —? — — —				37							
SILTY SAND	brown	medium-dense	SM	38							
				39							
				40	x	24	17			18/--	
				41							
— — — ?— — — — — — —? — — —				42							
SANDY SILT - CLAYEY SILT	brown	dense	ML	43							
				44							
				45	x	42	30			73/--	
Bottom of Boring = 45.0 feet				46							
Note: "(t)" indicates shear strength by Torvane.				47							
"(p)" indicates shear strength by pocket penetrometer.				48							
				49							
				50							
				51							
				52							
				53							
				54							
				55							
				56							
				57							
				58							
				59							
				60							

Drill Rig Hollow Flight Auger	Surface Elevation ~ 87.5 feet	Logged By GC
Groundwater Depth ~ 18 feet	Boring Diameter 8 inches	Date Drilled 10/12/06

DESCRIPTION AND CLASSIFICATION / SAMPLE DATA

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	Depth (Feet)	Blows Per Foot	Percent Moisture	Dry Density (Pcf)	Plasticity Index / Liquid Limit (%)	Percent Passing #200/#4 Sieve	Shear* Strength (Ksf)
Sod										
CLAYEY SAND	brown	medium-dense	SC	2	12	13			35/--	
Possible Fill				3						
SILTY CLAY	black	very stiff	CH	4–5	18	32	87		75/--	4.5+(p)
— — — ?— — — — — — —? — — —				7						
SILTY CLAY	light gray	very stiff	CL	9–10	12	22	102		73/--	3.5(p)
— — — ?— — — — — — —? — — —				13						
CLAYEY SAND, with gravel	gray-brown	medium-dense	SC	14–15	28	10	123		21/77	
— — — ?— — — — — — —? — — —				16–17						
SANDY GRAVEL	gray-brown	medium-dense	GP/GW	18		∇ (ATD)				
				19	18	73			6/55	
SILTY SAND	brown	medium-dense	SM	20		22			35/--	
SANDY GRAVEL	brown	medium-dense	GM	24	45	14	121		11/65	
SILTY SAND	brown	medium-dense	SM	25		22	104		33/--	

Bottom of Boring = 25.0 feet

Note: "(p)" indicates shear strength by pocket penetrometer.

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EXPLORATORY BORING LOG 4

PARK/DELMAS RESIDENTIAL
San Jose, California

575-40 | December 2006 | Sheet 1 of 1

Drill Rig Hollow Flight Auger	Surface Elevation ~ 87.1 feet	Logged By GC
Groundwater Depth ~ 17 feet	Boring Diameter 8 inches	Date Drilled 10/12/06

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	Depth (Feet)	SAMPLER	Blows Per Foot	Percent Moisture	Dry Density (Pcf)	Plasticity Index / Liquid Limit (%)	Percent Passing #200/#4 Sieve	Shear* Strength (Ksf)
SANDY CLAY	black	very stiff	CL	1							
FILL				2	x	11	22			61/--	
SILTY CLAY	gray with brown	very stiff	CH	3							
				4	■	14	28	86		60/--	4.5(p)
				5							
				6							
?- - - - - - - -?											
SANDY CLAY - SANDY SILT	brown	medium-dense	CL/ML	7							
				8							
				9	■	28	9	106		55/--	4.0(p)
				10							
				11							
?- - - - - - - -?				12							
GRAVELLY SAND, with trace clay	brown	medium-dense	SW	13							
				14	■	45	4	116		5/56	
				15							
				16							
?- - - - - - - -?				17			▽ (ATD)				
SILTY SAND - SANDY SILT	brown	medium-dense	SM/ML	18							
				19	■	36	18			47/--	4.0(p)
				20							
				21							
SAND	brown	medium-dense	SP/SW	22							
				23							
				24	x	17	18			4/85	
				25							
Bottom of Boring = 25.0 feet				26							
Note: "(p)" indicates shear strength by pocket penetrometer.				27							
				28							
				29							
				30							

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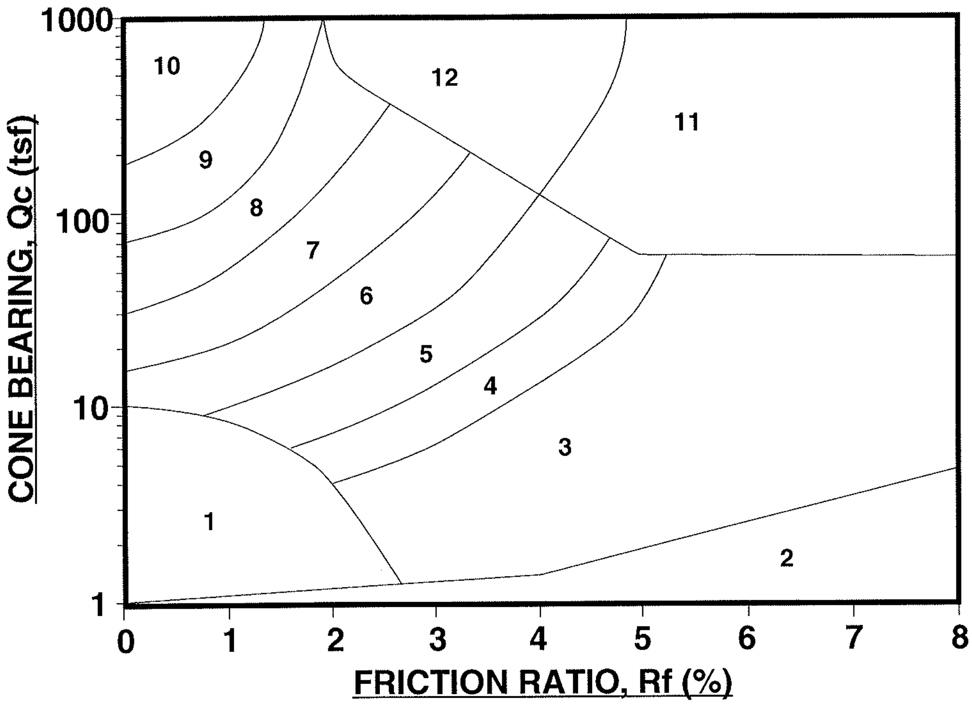
EXPLORATORY BORING LOG 5

PARK/DELMAS RESIDENTIAL
San Jose, California

575-40 December 2006 Sheet 1 of 1

SIMPLIFIED SOIL CLASSIFICATION CHART FOR STANDARD ELECTRONIC CONE PENETROMETER



ZONE	Qc/N [1]	SOIL BEHAVIOR TYPE
1	2	Sensitive Fine Grained
2	1	Organic Material
3	1	CLAY
4	1.5	Silty CLAY to CLAY
5	2	Clayey SILT to Silty CLAY
6	2.5	Sandy SILT to Clayey SILT
7	3	Silty SAND to Sandy SILT
8	4	SAND to Silty SAND
9	5	SAND
10	6	Gravelly SAND to SAND
11	1	Very Stiff Fine Grained (*)
12	2	SAND to Clayey SAND (*)

(*) Overconsolidated or Cemented

Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc*100 = Friction Ratio

Base from chart provided by Gregg In Situ, Inc.

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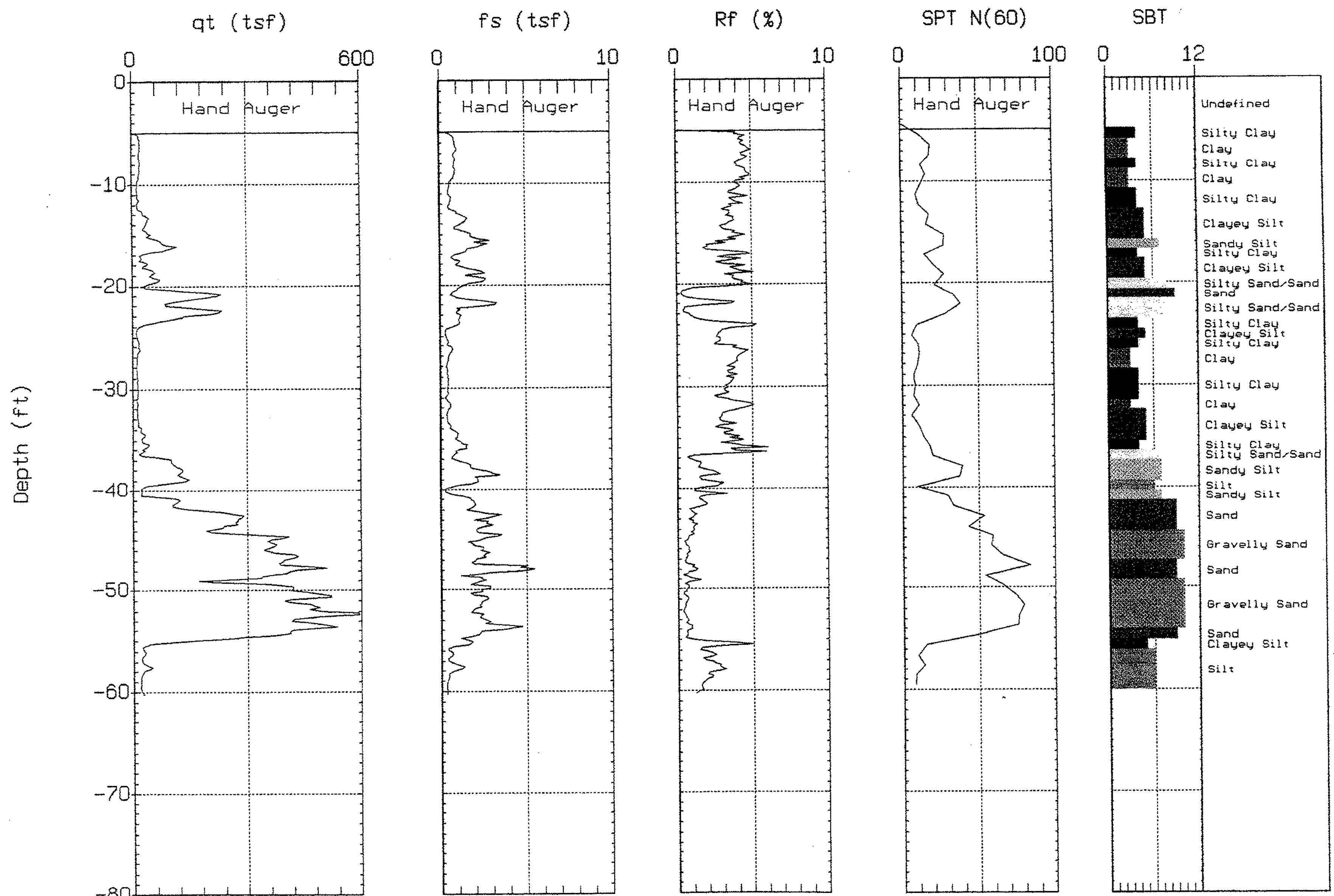
KEY TO CONE PENETROMETER SOUNDINGS

PARK/DELMAS RESIDENTIAL

San Jose, California

575-40 | December 2006 | Figure A-2



Max. Depth: 60.20 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

The above data was obtained and reduced by Gregg In Situ on November 6, 2006.

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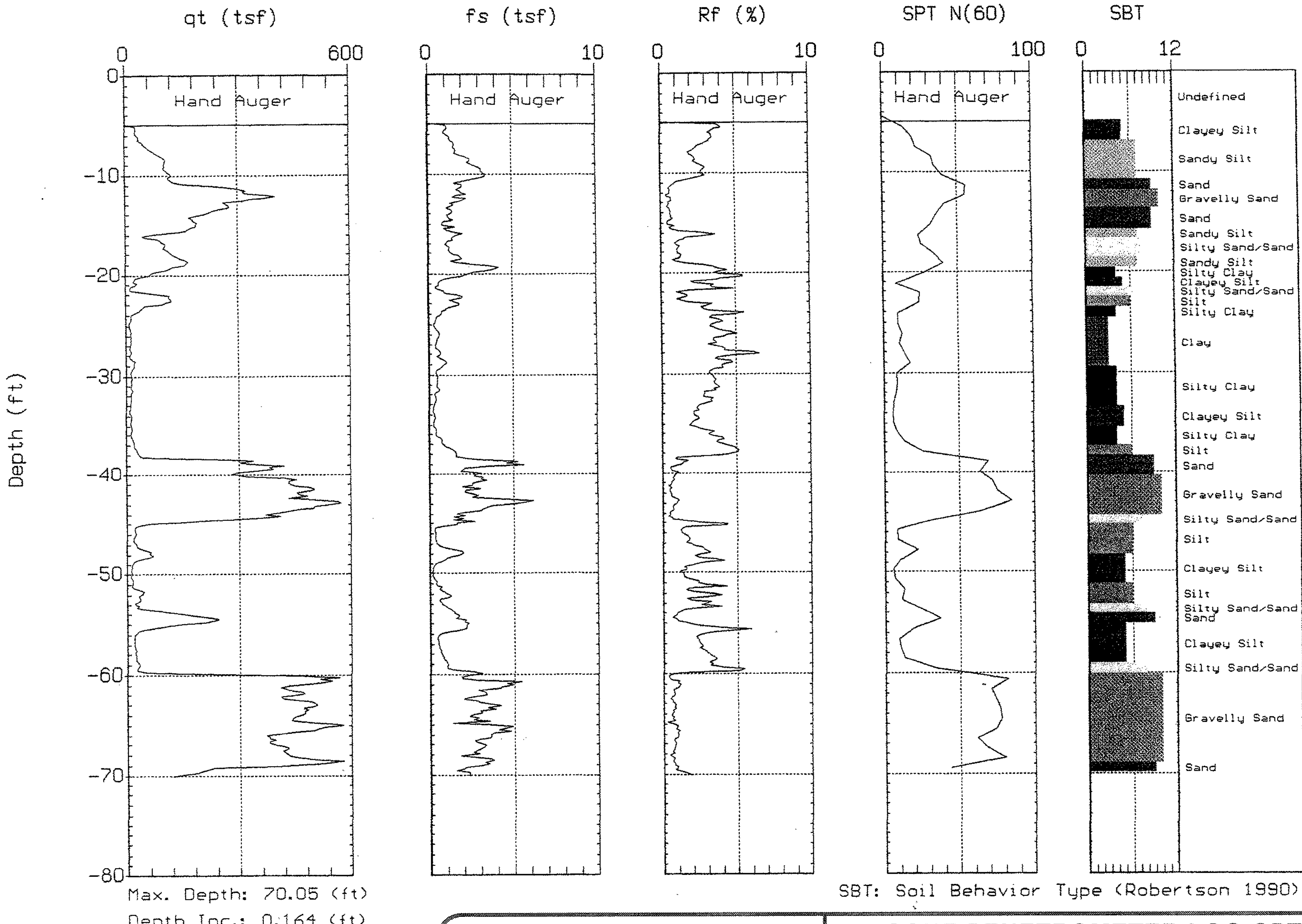
CONE PENETROMETER LOG CPT-1

PARK/DELMAS RESIDENTIAL
San Jose, California

575-40 **December 2006** **Sheet 1 of 1**



The above data was obtained and reduced by Gregg In Situ on November 6, 2006.

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CONE PENETROMETER LOG CPT-2

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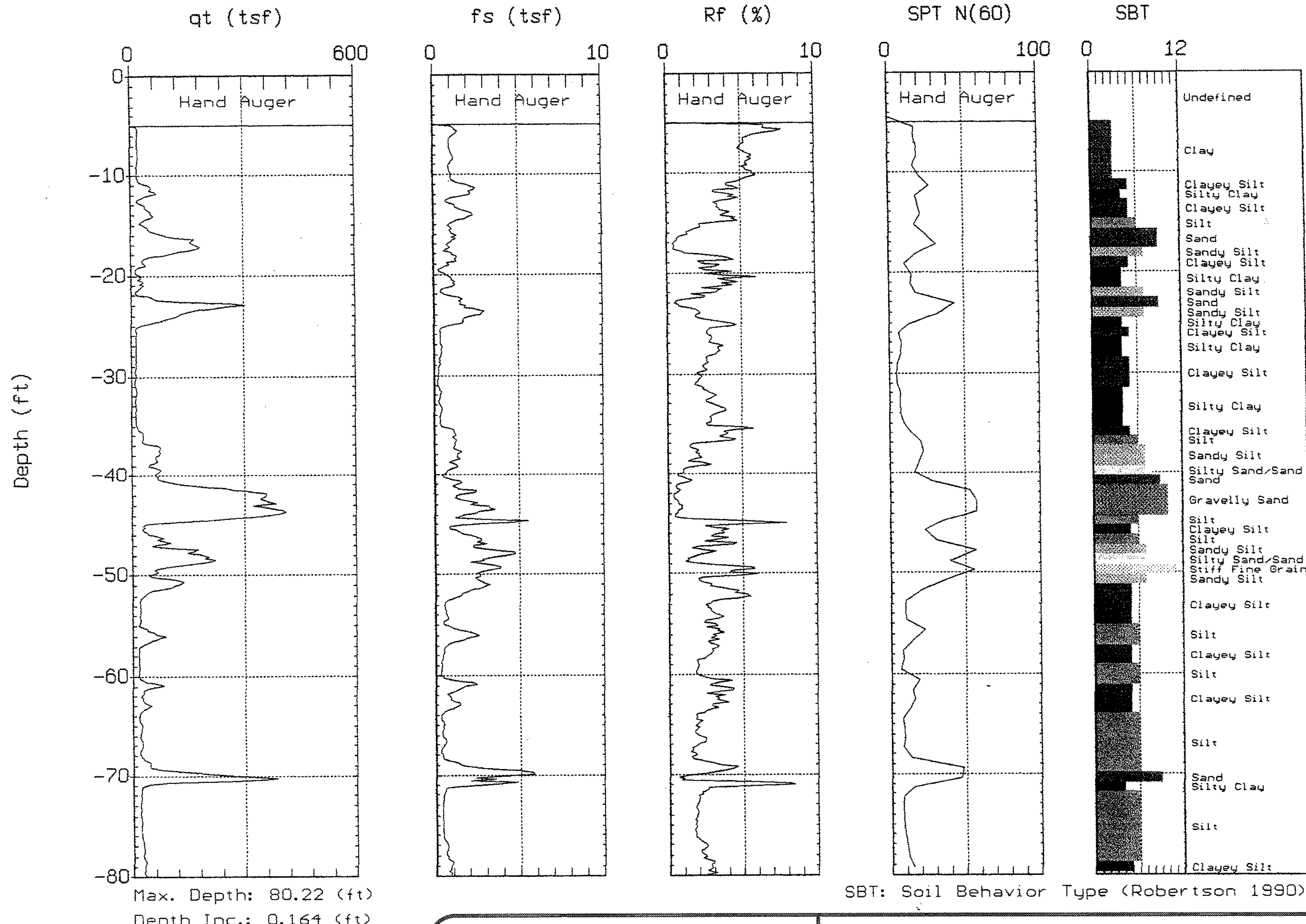
San Jose, California

575-40 | **December 2006** | **Sheet 1 of 1**



The above data was obtained and reduced by Gregg In Situ on November 6, 2006.

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CONE PENETROMETER LOG CPT-3

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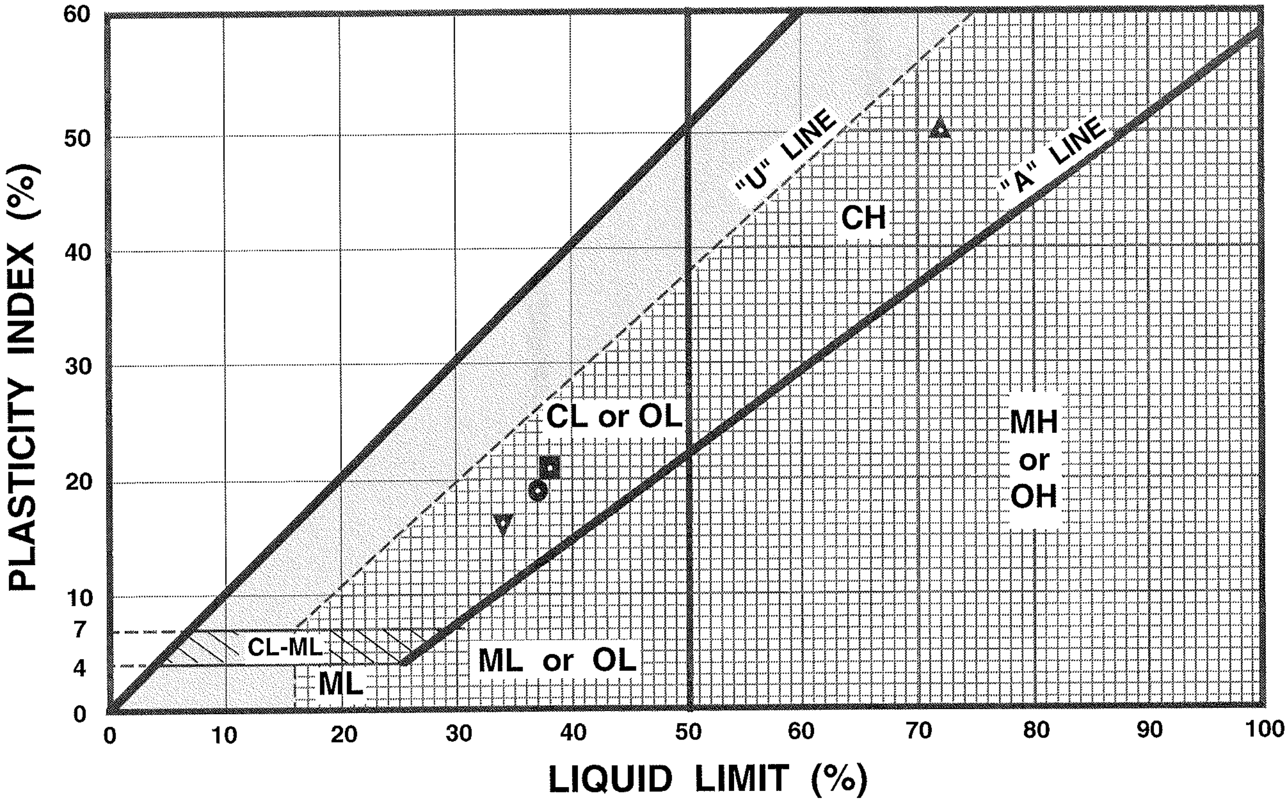
575-40 | **December 2006** | **Sheet 1 of 1**

APPENDIX C

LABORATORY TEST DATA FROM
2006 DONALD E BANTA & ASSOCIATES
GEOTECHNICAL INVESTIGATION



KEY SYMBOL	BORING NUMBER	SAMPLE DEPTH (Feet)	NATURAL WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING No. 200 SIEVE (%)	LIQUIDITY INDEX	UNIFIED SOIL CLASS-IFICATION SYMBOL
⊙	EB-1	9.5	23	37	19	79	--	CL
▲	EB-2	4.5	38	72	50	83	--	CH
■	EB-3	9.5	21	38	21	80	--	CL
▼	EB-3	29.5	28	34	16	83	--	CL

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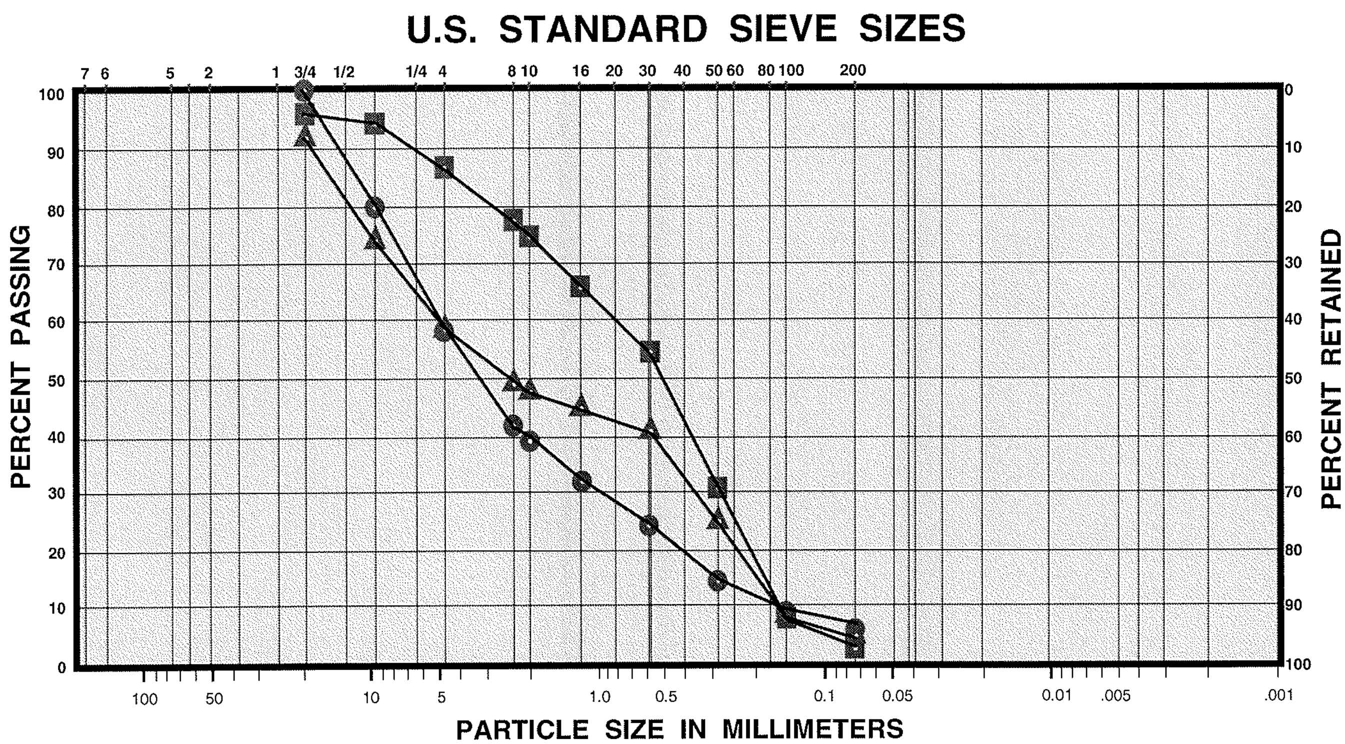
Consulting Geotechnical Engineers

PLASTICITY CHART AND DATA

PARK/DELMAS RESIDENTIAL

San Jose, California

575-40 | December 2006 | Figure B-1



COBBLE	GRAVEL		SAND			SILT and CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NUMBER	SAMPLE DEPTH (Feet)	ELEVATION (Feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
●	EB-4	18.5-19.5	--	SW	Brown Gravelly Sand
▲	EB-5	13.5-15	--	SW	Brown Gravelly Sand
■	EB-5	23.5-25	--	SW	Brown Sand with scattered gravel

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Consulting Geotechnical Engineers

GRADATION CHART AND DATA

PARK/DELMAS RESIDENTIAL

San Jose, California

575-40 December 2006 Figure B-2

California State Certified Laboratory No.2153

CERCO analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

3 November, 2006

Job No.0610154
Cust. No.10731

Mr. Gary Carpenter
Banta & Associates
415 Meridian Avenue
San Jose, CA 95126

Subject: Project No.: 575-40
Project Name: Delmas @ Park, S
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Carpenter:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on October 16, 2006. Based on the analytical data, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified as "corrosive" and Sample No.002 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a detection limit of 15 mg/kg.

The sulfate ion concentrations range from 33 to 34 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 7.6 to 7.9 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential for both samples is 430-mV, which is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call JDH Corrosion Consultants, Inc. at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO
analytical, inc.
3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

Client: Donald E. Banta & Associates
Client's Project No.: 575-40
Client's Project Name: Delmas @ Park, S
Date Sampled: 12-Oct-06
Date Received: 16-Oct-06
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 3-Nov-2006

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0610154-001	EB-5 @ 4'	430	7.6	-	1,700	-	N.D.	33
0610154-002	EB-5 @ 9'	430	7.9	-	3,200	-	N.D.	34

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	24-Oct-2006	24-Oct-2006	-	2-Nov-2006	-	24-Oct-2006	24-Oct-2006

* Results Reported on "As Received" Basis

N.D. - None Detected

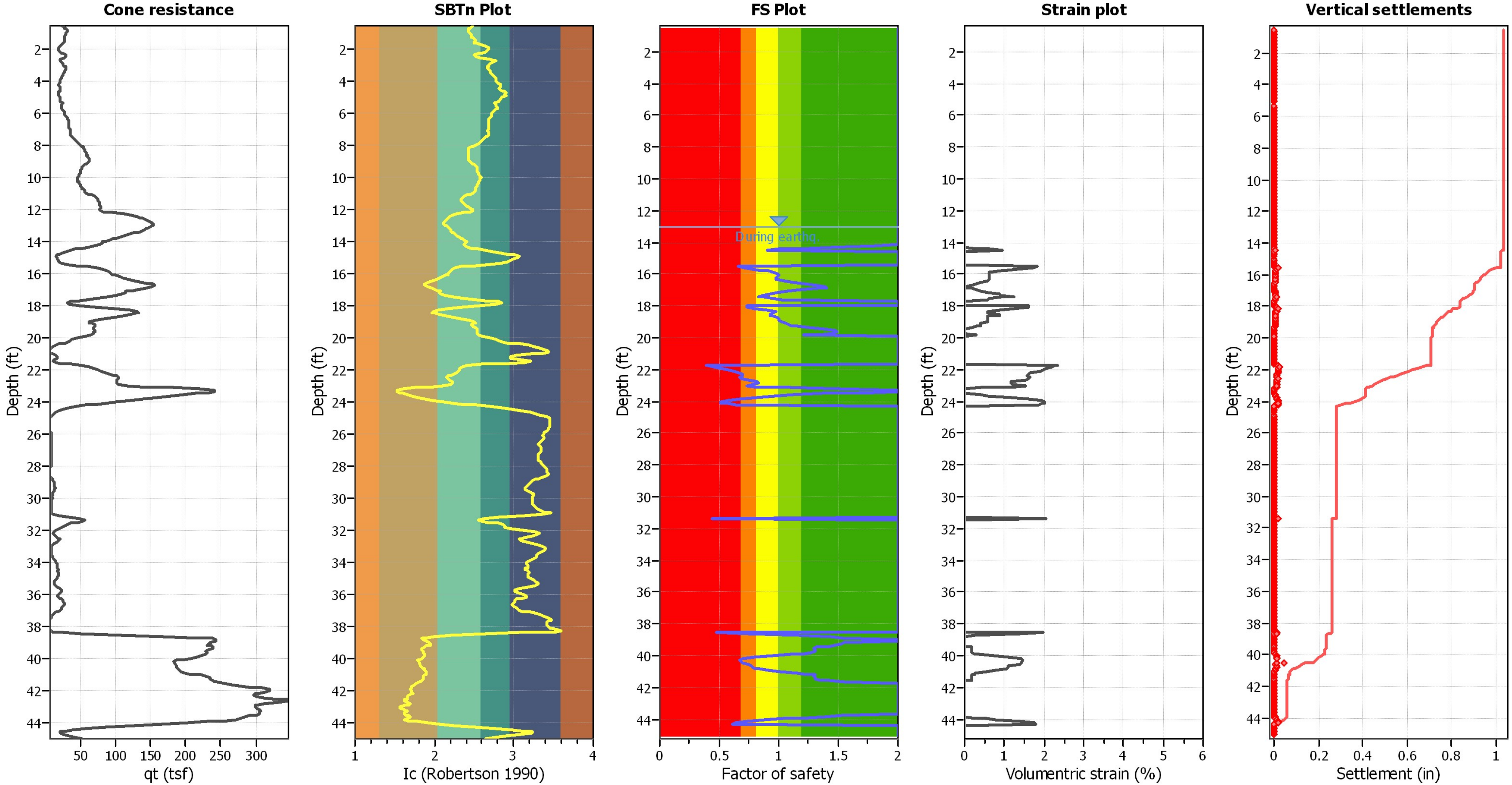
Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX D

LIQUEFACTION ANALYSIS RESULTS

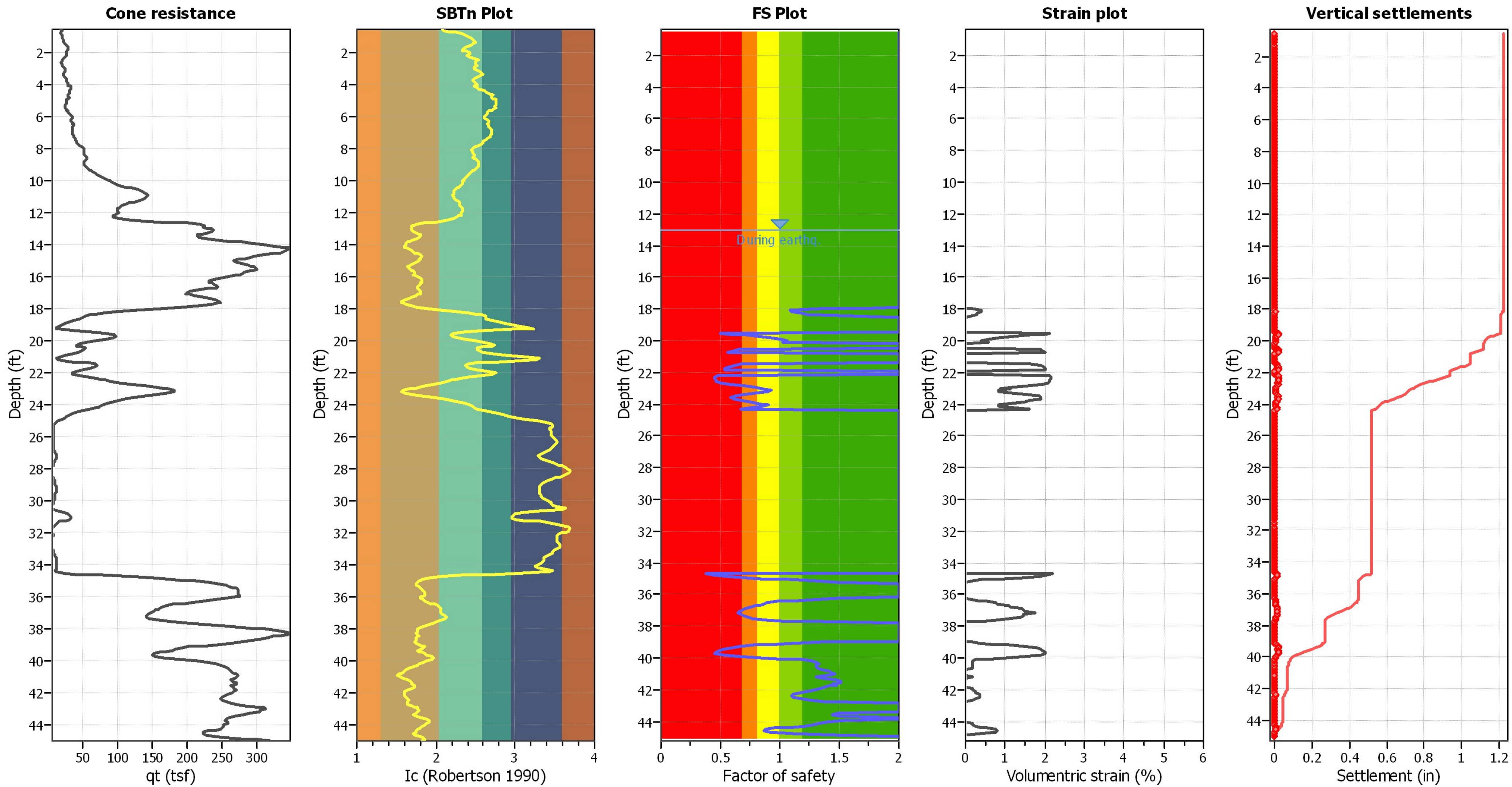
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



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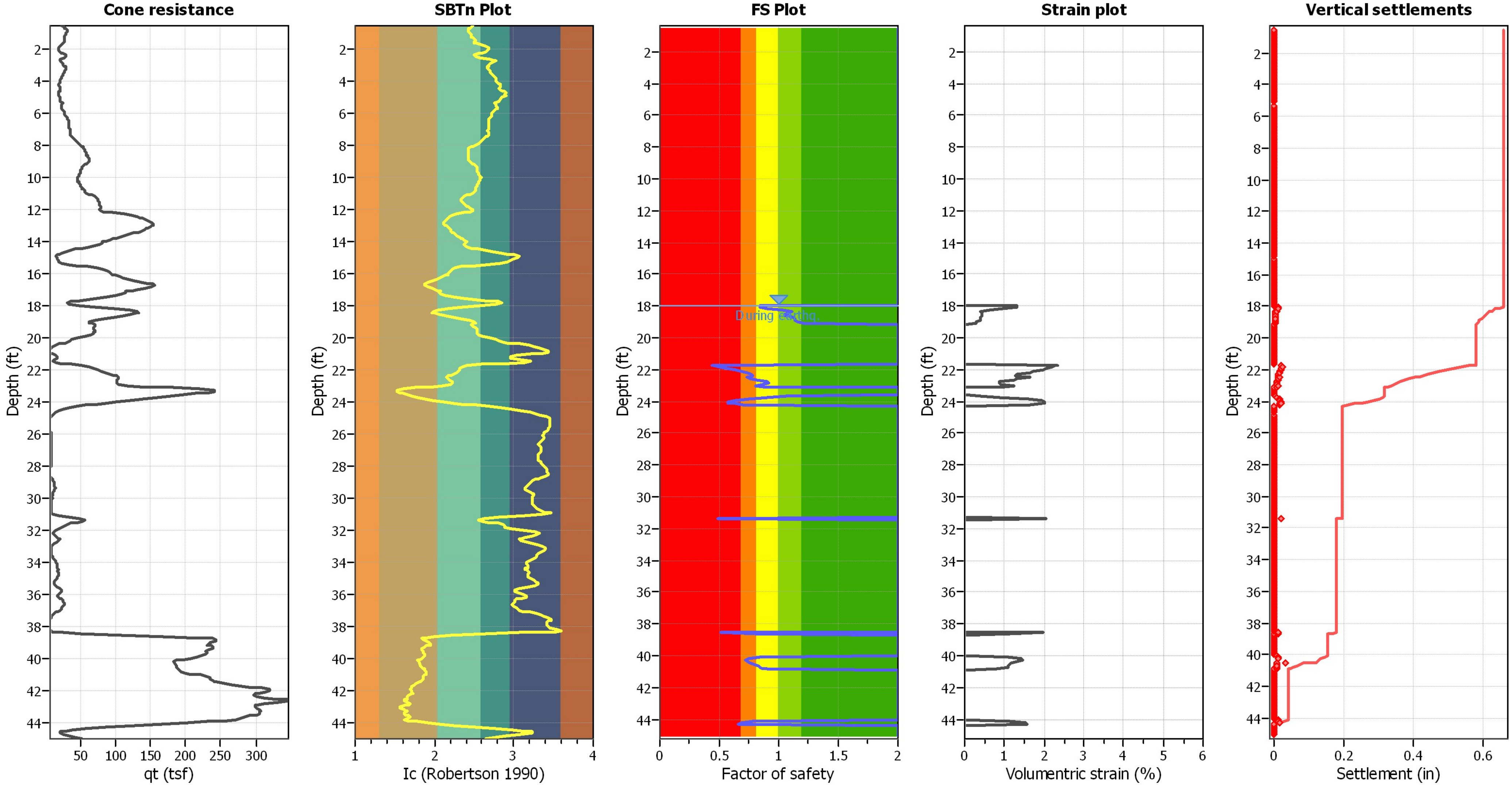
Estimation of post-earthquake settlements



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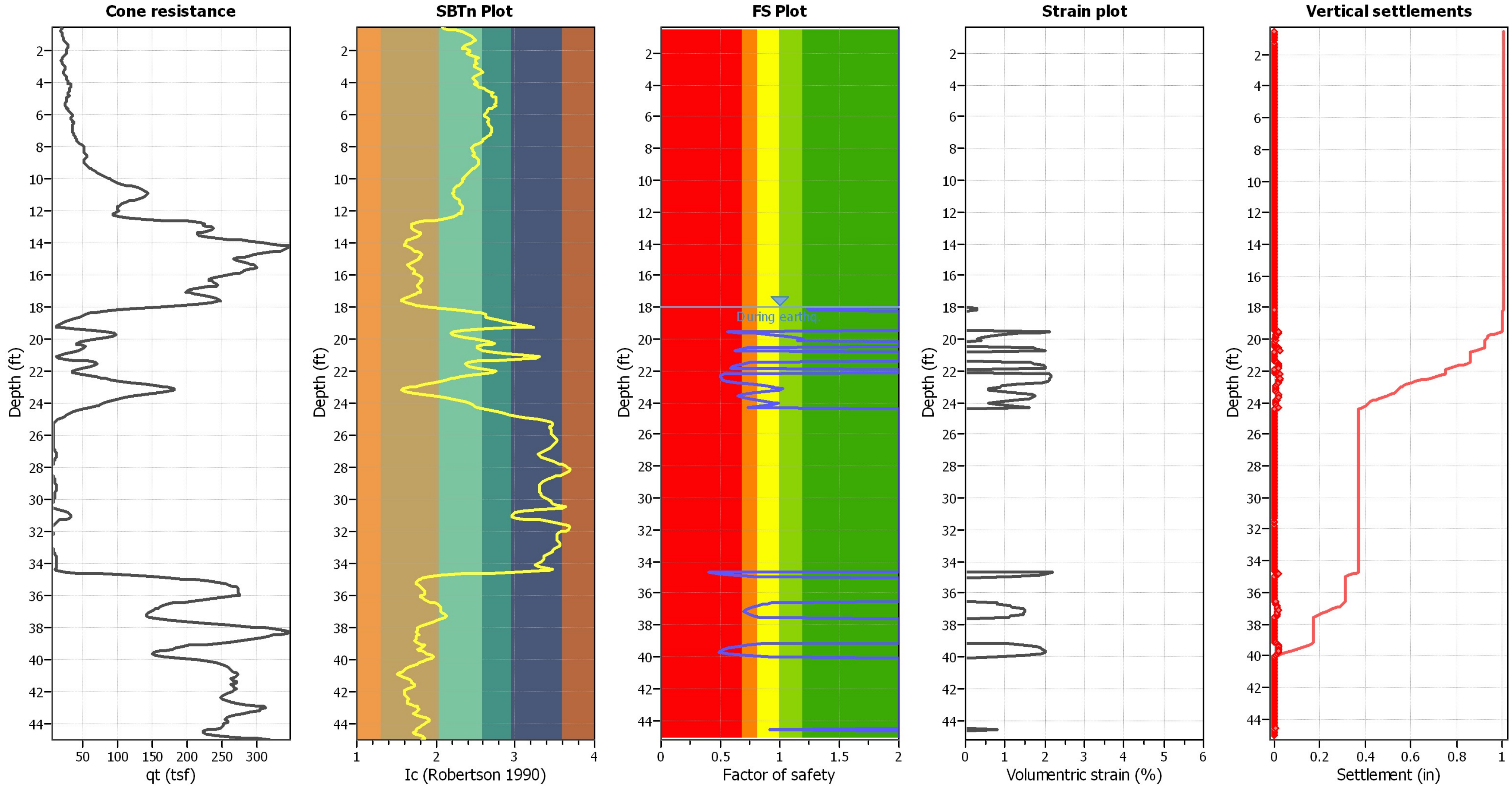
Estimation of post-earthquake settlements



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