PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED COYOTE CREEK TRAIL
MONTAGUE EXPRESSWAY TO WATSON PARK
SAN JOSE, CALIFORNIA

PROJECTS 2298E

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED COYOTE CREEK TRAIL MASTER PLAN
BETWEEN MONTAGUE EXPRESSWAY AND WATSON PARK
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1. INTRODUCTION

1.1 GENERAL

This report presents the results of our preliminary geotechnical investigation for a section of the proposed Coyote Creek Trail between Montague Expressway and Watson Park in San Jose, California. The limits of the project are shown on the Vicinity Map, Figure 1, of this report.

This report presents our preliminary conclusions and preliminary geotechnical recommendations for preparation of a master plan and an environmental document, and for preliminary design of the currently proposed trail and structures. The preliminary conclusions and recommendations are based on subsurface information we collected during this investigation and information provided by the City of San Jose from past investigations performed by other consultants in the vicinity of the project area. The preliminary conclusions and preliminary recommendations in this report should not be used for project final design and construction without review by a qualified geotechnical engineer, and should not be extrapolated to other areas or used for other projects without our review.

1.2 PROJECT DESCRIPTION

The proposed trail, between Montague Expressway in the north and Watson Park in the south, is about 4.1 miles in length. The project will include Class 1 trails, five pedestrian bridges, ten under-crossings and miscellaneous retaining walls. The project descriptions below are based on information provided to us. If the actual project differs from those described below, Pacific Geotechnical Engineering (PGE) should be contacted to review our preliminary conclusions and preliminary recommendations and present any necessary modifications to address the different project schemes.

Class 1 Trail: The proposed Class 1 trail will consist of a 12-foot wide paved path with 2-foot wide gravel shoulders on both sides, for a total width of 16 feet. Locally, a 10-foot trail width is proposed. The majority of the trail will be paved with asphalt concrete. Portland cement concrete will be used for the under-crossing ramps except at O'Toole Avenue, Highway 880 and Ridder Park Drive where the trail/under-crossings ramps will have asphalt concrete surface.

At its north end, the proposed trail will connect to an existing unpaved path on the western creek bank at Montague Expressway. South of Montague Expressway, the trail will cross over a proposed pedestrian bridge to the eastern creek bank. The trail will continue along the eastern creek bank, crossing under Charcot Avenue, O'Toole Avenue, Highway 880, Brokaw Road, Ridder Park Drive, and the existing Union Pacific Railroad (UPRR) Bridge, to just west of Oakland Road. At west of Oakland Road, the trail will cross over a new pedestrian bridge to the western/southern creek bank and continue under Oakland Road to an existing gravel path (service road) at the east end of Corie Court. The trail will follow the existing gravel path to a new pedestrian bridge at Hazlett Way and Notting Hill Drive. Over this bridge, the trail will continue on the eastern creek bank along Notting Hill Drive, through the San Jose Flea Market property, crossing under Berryessa Road, over Penitencia Creek, under Mabury Road, along

the City of San Jose Corporation Yard to just north of Highway 101. The trail will continue over a new pedestrian bridge to the western creek bank and terminate at Watson Park which is located southwest of Highway 101 and Coyote Creek.

The trail alignment will utilize existing Santa Clara Valley Water District (SCVWD) service roads, City property, existing streets and sidewalks, and private property.

Pedestrian Bridges: The five proposed pedestrian bridges are located at south of Montague Expressway, west of Oakland Road, Hazlett Way and Notting Hill Drive, Penitencia Creek and north of Highway 101. Each bridge will be a single span structure with a prefabricated steel truss superstructure and concrete deck. Information on the proposed bridges provided by the project Structural Engineer is summarized below.

Location	Length, feet	Estimated Total Dead Load per Abutment, Kips	Estimated Total Live Load per Abutment, Kips
South of Montague Expressway	180	200	97.5
West of Oakland Road	215	310	116.5
Hazlett Way & Notting Hill Drive	225	320	121.5
Penitencia Creek	60	95	32.5
North of Highway 101	160	185	86.5

Approach ramps are proposed for the pedestrian bridges except at Penitencia Creek. At south of Montague Expressway, the approach fills on both sides of the bridge will be up to about 2½ feet high (to roughly elevation 47½ feet) with retaining walls on both sides of the ramps.

At west of Oakland Road, approach fills with retaining walls on both sides of the fill ramps are proposed at both ends of the bridge. The proposed retaining walls will be up to about 13 feet high on the east side of the creek and up to about 8 feet high on the west side of the creek (to roughly elevation 68 feet at both ends of the bridge).

At Hazlett Way and Notting Hill Drive, approach fills with retaining walls on both sides of the fill ramps are proposed at both ends of the bridge. The proposed retaining walls will be up to about 9 feet high on the east side of the creek and up to about 7 feet high on the west side of the creek (to roughly elevation 75 feet at both ends of the bridge).

At north of Highway 101, approach fills with retaining walls on both sides of the fill ramps are proposed at both ends of the bridge. The proposed retaining walls will be up to about 11½ feet high on both the east and west sides of the creek (to roughly elevation 95½ feet at both ends of the bridge).

Under-crossings: The ten proposed under-crossings are located at Charcot Avenue, O'Toole Avenue, Highway 880, Brokaw Road, Ridder Park Drive, UPRR Bridge, Old Oakland Road, Berryessa Road, Mabury Road, and Highway 101.

> *Under-crossing at Charcot Avenue*: The concrete approach ramps will descend to an elevation of about 36.3 feet under the existing Charcot Avenue Bridge to provide a vertical clearance of about 8 feet. Retaining walls, up to about 10 feet high, are proposed on both sides of the ramps and on the land side of the proposed under-crossing.

Under-crossing at O'Toole Avenue: The asphalt concrete approach ramps will descend to an elevation of about 46 feet under the existing O'Toole Avenue Bridge to provide a vertical clearance of about 8 to 8½ feet. A retaining wall, up to about 8 feet high, is proposed on the land side of the proposed under-crossing adjacent to the existing O'Toole Avenue/I-880 off-ramp bridge abutment.

Under-crossing at Highway 880: The asphalt concrete trail under the existing Highway 880 Bridge will have a surface elevation of about 53 to 53½ feet, which we understand will be at approximate existing grades, to provide a vertical clearance of about 11¾ feet. A retaining wall, up to about 2 feet high, is proposed on uphill side of the trail just north of the Highway 880 Bridge.

Under-crossing at Brokaw Road: The concrete approach ramps will descend to an elevation of about 47 to 48½ feet under the existing Brokaw Road Bridge to provide a vertical clearance of about 8 feet. Retaining walls, up to about 4 feet high, are proposed on both sides of the approach ramp on the north side of the under-crossing.

Under-crossing at Ridder Park Drive: The asphalt concrete approach ramps will descend to an elevation of about 53 to 54 feet under the existing Ridder Park Drive Bridge to provide a vertical clearance of about 8½ feet. No retaining wall is proposed for the approach ramps.

Under-crossing at UPRR Bridge: The concrete approach ramps will descend to an elevation of about 52 feet under the existing UPRR Bridge to provide a vertical clearance of about 9 feet. No retaining wall is proposed for the ramps. An 8-foot high, 75-foot long protection cage with solid cover is proposed under the railroad bridge.

Under-crossing at Oakland Road: The concrete trail will have a surface elevation of about 56½ feet under the existing Oakland Road Bridge, about 3½ to 4½ feet above existing grades, to provide a vertical clearance of about 8½ feet. A retaining wall, up to about 8 feet high, is proposed on the creek side of the ramps and under-crossing. A retaining wall, up to about 4 feet high, is proposed on the land side of the ramp north of Oakland Road.

Under-crossing at Berryessa Road: The concrete approach ramps will descend to an elevation of about 71 to 72 feet under the existing Berryessa Road Bridge. A retaining wall, up to about 5 feet high, is proposed on the creek side of the under-crossing.

Under-crossing at Mabury Road: The concrete approach ramps will descend to an elevation of about 76 feet under the existing Mabury Road Bridge to provide a vertical clearance of about 8 feet. Retaining walls, up to about 6 feet high, are proposed on both sides of the approach ramps and under-crossing.

Under-crossing at Highway 101: The concrete trail will have a surface elevation of about 76 to 80 feet under the existing Highway 101 Bridge to provide a vertical clearance of about 8 feet. A retaining wall, up to about 3 feet high, is proposed on the creek side of the under-crossing.

Retaining Walls: Proposed retaining walls associated with the under-crossings are discussed above. Other site retaining walls are planned along the land side of the trail leading to an access node on the south side of Ridder Park Drive, at the south end of the proposed bridge over Penitencia Creek, and along the east side of the trail adjacent to the entrance road into the

City of San Jose Corporation Yard south of Mabury Road. These walls are about 2½ to 3 feet in height.

1.3 OBJECTIVE AND SCOPE OF INVESTIGATION

The objective of this preliminary geotechnical investigation was to collect subsurface information at selected locations and to provide feasibility-level information for preparation of a master plan and an environmental document and for preliminary design of the proposed trail and structures.

1. Reconnoitering of the project areas to observe surface conditions and mark locations of our subsurface exploration.

2. Obtaining encroachment and exploratory boring permits from Santa Clara Valley Water District and environmental clearance from City of San Jose for our field exploration.

3. Coordination of our drilling with Underground Service Alert, City of San Jose, and Santa Clara Valley Water District.

4. Review of geotechnical information in our files and geotechnical reports provided by the City of San Jose that are pertinent to the project.

5. Exploration, sampling and classification of subsurface soils at selected locations by means of ten exploratory drill holes and three Cone Penetrometer Test (CPT) probes.

6. Laboratory testing of selected soil samples recovered from our drill holes.

7. Engineering analysis of the field and laboratory data to formulate preliminary conclusions and recommendations for the project.

8. Preparation of this report summarizing our findings, preliminary conclusions and preliminary recommendations.

1.4 INFORMATION PROVIDED

For this investigation, the following was provided to us.

- Trail Alignment Plan, Sheets 1 through 5, latest version dated February 24, 2010, prepared and provided by Callander Associates

- Sections, Sheets 1 through 4, latest version dated February 24, 2010, prepared and provided by Callander Associates

- Enlargement Plans, Sheets 1 through 6, latest version dated February 24, 2010, prepared and provided by Callander Associates

- Information on the proposed bridges provided by Biggs Cardosa Associates, project structural engineer

- Geotechnical Reports prepared by various consultants, provided by the City of San Jose

- Soil Investigation for the Proposed Markovits and Fox Scrap Metal and Salvage Yard, San Jose, California, prepared by Woodward-Clyde-Sherard & Associates, dated December 4, 1961

- Soil Investigation, Proposed Storage Development, 1395 Mabury Road, San Jose, California, prepared by Terrasearch, Inc., dated July 7, 1980

- Soil and Foundation Investigation, Proposed One and Two-story Buildings, Schallenberger Road, San Jose, California, prepared by Engeotech, Inc., dated October 1981

- Geotechnical Engineering Study for the Proposed R&D Buildings, Old Oakland Road and Brokaw Road, San Jose, California, prepared by Associated Geotechnical Engineers, Dated August 3, 1984

- Geotechnical Investigation, Yard Court Property, San Jose, California, prepared by Freeman-Kern Associates, Inc., dated February 25, 1986

- Geotechnical Investigation Report for the Proposed San Jose Cardlock Facility at 2750 Kruse Avenue in San Jose, California, prepared by Kleinfelder, dated April 18, 2000

- Geotechnical Investigation, Interstate 880 and East Brokaw Road, San Jose, California, prepared by Treadwell & Rollo, dated October 19, 2007

- Geotechnical Investigation, Watson Community Park Masterplan Improvements, San Jose, California, prepared by URS Corporation, dated July 31, 2008

- Supplemental Geotechnical Investigation, Retail Building at Highway 880 and East Brokaw Road, San Jose, California, prepared by Treadwell & Rollo, dated February 19, 2009

2. SITE INVESTIGATION

2.1 FIELD INVESTIGATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration program. The site reconnaissance was to observe existing surface conditions along the proposed trail alignment and at locations of the proposed bridges and under-crossings. The subsurface exploration program was to explore soil conditions at selected locations of bridge supports, under-crossings, and trail alignment. Subsurface exploration for this study was limited to selected locations because of site accessibility, right-of-way limitations and approved scope.

2.2 SUBSURFACE EXPLORATION

The subsurface exploration program included ten drill holes (DH-1 through DH-10) and three CPT probes (CPT-1 through CPT-3), advanced on the top of creek banks. The drill holes and CPT probes were located in the field by referencing to existing site features and pacing; therefore, their locations should be considered approximate. The approximate locations of the drill holes and CPT probes are shown on Figures 2 through 6. The drill holes and CPT probes were backfilled with cement grout as required by the Santa Clara Valley Water District. Soil cuttings were drummed, hauled off the site, and disposed at a local landfill.

2.2.1 Drill Holes

The drill holes were advanced on September 16 through 18, 2009, to depths between 10 and 70 feet below the ground surface using a Mobile B-56 drilling rig equipped with an 8-inch diameter hollow stem auger.

In the field, our personnel visually classified the materials encountered in the drill holes and maintained a log of each drill hole. Samples were obtained from the drill holes by driving a 2½-inch inside diameter split spoon or a 2-inch outside diameter (1⅜ inch inside diameter) Standard Penetration Test (SPT) sampler up to a depth of 18 inches into the earth material using a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the penetration interval indicated on the log where harder material was encountered, was shown as blows per foot on the drill hole logs. The hammer was driven by a hydraulic winch and pulley system.

Soil samples were collected from the drill holes at about 5-foot vertical intervals. Soil samples were sealed in the field and transported to our laboratory for further evaluation and testing. Visual classification of soils encountered in our drill holes was made in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). The laboratory test results were used to refine our field classifications. Two Keys to Soil Classification, one for fine grained soils and one for coarse grained soils, are included in Appendix A together with the logs of the drill holes.

In addition, ten bulk samples of near-surface soils were collected along the trail alignment. The approximate locations of the bulk samples are also shown on Figures 2 through 6.

2.2.2 Cone Penetrometer Tests

CPT-1 through CPT-3 were performed by John Sarmiento & Associates on September 14, 2009, and ranged between approximately 45 and 70 feet in depth below the top of creek bank. 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 B 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

2.3 LABORATORY TESTING

Laboratory tests were performed on selected soil samples. These tests included water content, dry density, Atterberg limits, unconfined compressive strength, grain size analysis, percent passing a No. 200 sieve, R-value and corrosion potential. The laboratory test results are presented on the drill hole logs at the corresponding sample depths. The results of the Atterberg Limits, unconfined compressive strength, grain size analysis, and R-value tests are presented as Figures C-1 through C-17 in Appendix C. The results of the corrosion potential tests performed by CERCO Analytical are presented on a separate sheet after Figure C-17.

3. FINDINGS

3.1 SITE GEOLOGY

The project site lies within the city of San Jose, which is situated at the northern end of the Santa Clara Valley. Geology of the Santa Clara Valley has been extensively studied and mapped by the United States Geological Survey (USGS), California Geological Survey (CGS) and other investigators. The proposed trail segment is mapped by several workers as underlain by geologically young sediments deposited by Coyote Creek. Knudsen and others (2000) and Witter and others (2006) mapped the project areas as underlain by Latest Holocene [Holocene extends back to about 11,000 years before present] stream terrace deposits (map unit Qhty) inset into Holocene alluvial fan levee deposits (map units Qhl) in the upstream third of the segment, and Latest Holocene alluvial fan levee deposits (map unit Qhly) in the downstream two-thirds. Wentworth and others (1999) map the segment as being underlain by Holocene stream terrace deposits (map unit Qht) inset into Holocene levee deposits (map unit Qhl) (Wentworth and others, 1999).

3.2 EARTHQUAKE FAULTS AND SEISMICITY

The greater 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, and selected faults are described in detail in Appendix D of this report. Map distances are derived from the USGS Quaternary Fault and Fold database (http://earthquake.usgs.gov/regional/qfaults/)

Fault	Approximate Distance		Orientation from Project Site to Fault
	Project North	Project South	
Hayward (southeast extension)	3½ mi (5½ km)	3 mi (4¾ km)	Northeast
Calaveras	6¾ mi (10¾ km)	6½ mi (10¼ km)	Northeast
Hayward	7¼ mi (11½ km)	10 mi (16 km)	Northeast
Monte Vista-Shannon	10 mi (16 km)	8½ mi (13¾ km)	Southwest
San Andreas	13½ mi (22 km)	13½ mi (22 km)	Southwest
Sargent	18 mi (28¾ km)	15¾ mi (25½ km)	South
Zayante-Vergeles	21 mi (34 km)	19 mi (31 km)	Southwest
Greenville	21 mi (34 km)	21 mi (34 km)	Northeast

According to the 2007 California Building Code (CBC), ground motion parameters for buildings and other structures are taken as the lesser value developed from probabilistic and deterministic approaches, which can be calculated using the USGS Java Ground Motion Parameter Calculator version 5.0.9a (available from the USGS website). Using the USGS Java Calculator, the latitudes and longitudes of the proposed pedestrian bridges and under-crossings, and a Site Class D, the calculated peak ground acceleration (PGA) value for the Design Earthquake (DE)

is 0.4g. This PGA value is equivalent to S_{DS}/2.5. A Site Class D was selected based on subsurface materials encountered in our borings and CPTs, and the criteria in Table 1613A.5.2 of the 2007 CBC. For sites in the San Francisco Bay Area, the DE roughly corresponds to a PGA with a 10% probability of exceedance in a 50-year period (WGCEP, 2008).

The combined probability of large earthquakes occurring on one of the active faults in the area has been estimated by the Working Group on California Earthquake Probabilities (1990, 1999, 2003, 2008), with probabilities refined through several iterations. The greatest changes have been in the treatment of fault segments, rupture scenarios, and in the progressive consideration of more potential seismic sources. Current estimates (WGCEP, 2003, 2008) suggest that there is a 62% probability of a large magnitude (6.7 or greater) earthquake in the San Francisco Bay Region as a whole in the 30-year period ending in 2031.

3.3 SURFACE CONDITIONS

The proposed trail will be constructed along the tops and banks of Coyote Creek, utilizing much of the existing SCVWD service roads. Between Montague Expressway and Charcot Avenue, the service road on top of the eastern creek bank is about 14 to 17 feet wide, and has a gravel surface except for the northern roughly 500 feet which has a soil surface. The road surface is about 4 to 8 feet higher than the ground surface on the land side. On the creek side, the bank has inclinations of roughly 1½:1 to 2½:1 (horizontal:vertical) and heights of roughly 15 to 30 feet. Surface vegetation consists of trees and grass. Several slope failures were observed on the eastern creek bank. These failures are generally 1 to 5 feet deep and 2 to 9 feet wide. Some of these failures extend 1 to 5 feet onto the service road.

South of Charcot Avenue, the existing service road on top of the eastern creek bank is roughly 16 feet wide and has a gravel surface for a distance of about 800 to 850 feet, and becomes wider (up to roughly 50 feet wide) with a soil surface until it intersects the O'Toole Avenue Bridge. On the creek side, the bank has an inclination of roughly 2:1 (h:v) and heights of roughly 20 to 25 feet. There are isolated trees along the existing service road. Two slope failures observed on the eastern creek bank are generally 1 to 2 feet deep and 1 to 20 feet wide. One of the failures extends 2 to 3 feet onto the service road.

Between O'Toole Avenue and Highway 880, the existing service road is about 6 to 8 feet wide and unpaved. Creek bank below the service road has an inclination of roughly 2:1 (h:v) and heights of roughly 15 to 20 feet, and is covered with grass.

Between Highway 880 and Brokaw Road, the service road on top of the eastern creek bank is about 12 to 14 feet wide and unpaved. The road surface is about 10 feet higher than the ground surface on the land side. On the creek side, the bank has inclinations of roughly 1½:1 to 1:1 (h:v) and heights of roughly 15 to 18 feet. The vertical clearance below the Highway 880 Bridge is roughly 10 to 11 feet.

Between Brokaw Road and Ridder Park Drive, the top of the eastern creek bank is roughly 20 to 60 feet wide and unpaved. The creek bank has inclinations of roughly 2:1 to 2½:1 (h:v) and heights of roughly 20 to 25 feet.

Between Ridder Park Drive and the UPRR Bridge, the top of the eastern creek bank is a broad flat area covered with grass and isolated trees. The creek bank below has inclinations of roughly 3:1 or flatter (h:v) and heights of roughly 10 to 25 feet.

Between the UPRR Bridge and the proposed pedestrian bridge west of Oakland Road, the proposed trail will cross a narrow flat area between the creek bank and a private property to the north. On the south side of this pedestrian bridge, the trail will be constructed on a broad, flat area along the northeastern side of Schallenberger Road. This area is covered with grass and trees. There is an overhead power line along the northeastern side of Schallenberger Road. The vertical clearance below the Oakland Road Bridge is about 11 feet.

East of Oakland Road, the trail will cross an open, unpaved area between the creek and Corie Court until it connects to an existing gravel path/service road at the east end of Corie Court. The creek bank below this unpaved area has an inclination of about 1½:1 (h:v) and a height of about 16 feet. From Corie Court, the gravel path/service road, about 14 feet wide and borders by relatively flat ground, runs along the northeastern side of a business park and the north side of the Golden Wheel Mobile Home Park. At the northeastern corner of the mobile home park, the trail turns and continues southeasterly on top of an existing levee between the creek and the mobile home park to the proposed pedestrian bridge near the southeastern portion of the mobile home park. The top of the levee is about 8 to 10 feet wide and about 10 to 15 feet higher in elevation than the mobile park home. The creek side bank has inclinations of 1½:1 to 2:1 (h:v) and heights of 20 to 30 feet. The proposed pedestrian bridge will cross over Coyote Creek to near the intersection of Notting Hill Drive and Hazlett Way.

From Hazlett Way, the trail will cross a sloping area along the southwestern side of Notting Hill Drive, and continue along the paved parking lot of the San Jose Flea Market to Berryessa Road. The sloping area has an inclination of about 2½:1 (h:v) and a height of about 20 feet. Along the San Jose Flea Market parking lot, the creek bank has inclinations of roughly 2:1 to 2½:1 (h:v) and heights of roughly 20 to 25 feet. The trail will follow the existing asphalt concrete paved undercrossing at Berryessa Road.

Between Berryessa Road and Mabury Road, the trail will cross the southwestern portion of the San Jose Flea Market and its asphalt concrete parking lot to the south. South of Mabury Road, the trail will cross a flat unpaved area on the west side of the entrance road to the City of San Jose Corporation Yard to a locked gate. From there, the trail will continue along an existing levee between the creek and the corporation yard to a proposed pedestrian bridge north of Highway 101. The levee is about 10 feet high and has side slopes of about 2:1 (h:v). The creek bank below the levee has an inclination of about 2:1 (h:v) and extends up to 12 to 15 feet below. There is an abundance of trees.

3.4 SUBSURFACE CONDITIONS

A brief description of the materials encountered in each boring and CPT probe is presented below. For a more detailed description of the soil conditions encountered in our drill holes and CPT probes, refer to the drill hole logs in Appendix A and the CPT results in Appendix B.

Proposed Pedestrian Bridge South of Montague Expressway: DH-1 and CPT-1 were advanced on the top of the eastern and western creek bank to depths of 70 and 45 feet, respectively. In DH-1, a roughly 6-inch thick layer of gravel base was encountered at the ground surface. The gravel base is underlain by hard, low plasticity silty clay to a depth of about 2 feet; loose to medium dense sandy silt to a depth of about 12 feet; very stiff, low plasticity lean clay to silty clay to a depth of about 17 feet; medium dense silty sand and medium dense clayey sand to a depth of about 31½ feet; stiff, intermediate plasticity clay to a depth of about 42 feet; dense to very dense well graded sand with gravel and silt to a depth of about 59½ feet; stiff to hard, low

plasticity lean clay to a depth of about 69½ feet; and very dense silty sand to the maximum explored depth of about 70 feet.

In CPT-1, the interpreted soil behavior types include cohesive soils to a depth about 36½ feet, underlain by cohesionless soils to the maximum explored depth of about 45 feet.

Proposed Under-crossing at Charcot Avenue: DH-3 was advanced on the top of the eastern creek bank to a depth of about 25 feet. The surface has a roughly 6-inch thick gravel base layer. Materials encountered below the gravel base include loose silty sand to a depth of about 5½ feet; loose to medium dense poorly graded sand with silt to a depth of about 13 feet; and stiff to very stiff, intermediate plasticity clay to high plasticity fat clay to the maximum explored depth of about 25 feet.

Proposed Under-crossing at O'Toole Avenue: DH-4 was advanced on the top of the eastern creek bank to a depth of about 25 feet. The surface has a roughly 6-inch thick gravel base layer. Materials encountered below the gravel base include firm, low plasticity lean clay to intermediate plasticity clay to a depth of about 2 feet; medium dense clayey sand with gravel to a depth of about 9½ feet; and layers of stiff to hard, intermediate plasticity sandy clay and clay and high plasticity fat clay to the maximum explored depth of about 25 feet.

Proposed Under-crossing at Ridder Park Drive: DH-5 was advanced on the top of the eastern creek bank to a depth of about 25 feet. The surface has a roughly 3 to 4-inch thick gravel base layer. Materials encountered below the gravel base include very stiff to hard, low plasticity lean clay with sand to a depth of about 9 feet below the ground surface; very stiff, intermediate plasticity clay with sand to a depth of about 16½ feet; and very stiff, intermediate plasticity clay to high plasticity fat clay to the maximum explored depth of about 25 feet.

Proposed Under-crossing at UPRR Bridge: DH-6 was advanced on the top of the eastern creek bank to a depth of about 25 feet. No gravel base was encountered in this drill hole. Materials encountered below the ground surface include very stiff to hard, low plasticity lean clay with sand to a depth of about 3 feet; medium dense silty sand to a depth of about 5 feet; and very stiff to hard, intermediate plasticity clay to the maximum explored depth of about 25 feet.

Proposed Bridge West of Oakland Road: CPT-2 was advanced on the top of the western creek bank to a depth of about 70 feet. In CPT-2, the interpreted soil behavior types include cohesionless soils to a depth of about 3 feet; cohesive soils to a depth about 48½ feet; cohesionless soils to a depth of about 68 feet, and cohesive soils to the maximum explored depth of about 70 feet.

Proposed Bridge at Notting Hill Drive: DH-8 and CPT-3 were advanced on the top of the western and eastern creek bank to depths of 70 and 45 feet, respectively. In DH-8, a roughly 6-inch thick layer of gravel base was encountered below the ground surface. Materials encountered below the gravel base include hard, low plasticity sandy lean clay fill to a depth of about 4 feet; medium dense clayey sand to a depth of about 12 feet; layers of firm to very stiff, low plasticity lean clay, intermediate plasticity clay and high plasticity fat clay to a depth of about 44 feet; medium dense to very dense clayey sand, well graded sand and poorly graded sand to a depth of about 63 feet; very stiff, intermediate plasticity clay to a depth of about 66½ feet; and dense clayey sand to the maximum explored depth of 70 feet.

In CPT-3, the interpreted soil behavior types include sandy soils to a depth of about 15 feet and clayey soils to the maximum explored depth of about 45 feet.

Proposed Under-crossing at Mabury Road: DH-9 was advanced on the top of the eastern creek bank. The surface has a roughly 6-inch thick gravel base layer. Materials encountered below the gravel base include very stiff to hard, low plasticity sandy lean clay to a depth of about 4 feet; medium dense silty sand to a depth of about 9½ feet; and stiff to very stiff, intermediate plasticity clay to high plasticity fat clay to the maximum explored depth of about 25 feet.

Proposed Bridge North of Highway 101: DH-10 was advanced on the top of the eastern creek bank. In this hole, we encountered about 10 feet of fill consisting of hard, low plasticity lean clay and very dense clayey sand. The fill is underlain by loose to medium dense silt with sand to a depth of about 16½ feet and layers of stiff to hard, intermediate plasticity clay, high plasticity fat clay and low plasticity lean clay to the maximum explored depth of 70 feet. The clay layers have variable amounts of sand.

Proposed Trail: DH-2 and DH-7 were advanced at selected locations along the proposed trail. Both borings encountered a 6-inch thick layer of gravel base below the ground surface. In DH-2, materials encountered below the gravel base include stiff to hard, intermediate plasticity sandy clay to the maximum explored depth of about 10 feet.

In DH-7, materials encountered below the gravel base include very stiff to hard, intermediate plasticity clay with sand to a depth of about 6 feet and medium dense silty sand with gravel to the maximum explored depth of about 10 feet.

3.5 GROUNDWATER

Groundwater was encountered in holes DH-1, CPT-1, CPT-2, DH-8, CPT-3 and DH-10 at the time of field exploration, at a depth of about 31, 18, 14.6, 17, 21.9 and 27 feet, respectively. No groundwater was encountered in the other drill holes advanced for this preliminary investigation probably because of their shallower depths.

Information on groundwater level from past geotechnical reports provided by the City is tabulated below. These reports are for sites in the vicinity of the subject creek alignment.

Site Location	Report Prepared By	Reported Depth to Groundwater
Watson Park, southwest of Hwy 101 and Coyote Creek	URS Corporation	25 to 30 feet (November 2007) 20 to 28 feet (2006, month unknown)
Southeast of Brokaw Road & Highway 880	Treadwell & Rollo	17 to 18 feet (March 2007)
2750 Kruse Drive	Kleinfelder	16 to 18 feet (March 2000)
East end of Yard Court	Freeman Kern	no groundwater in borings up to 50 feet deep (January 1986)
Southwest of Brokaw and Oakland Roads	Associated Geotechnical Engineers	13 to 22 feet (June 1984)
Southeast of Schallenberger Rd and railroad	Engeotech	32 feet (October 1981)
1395 Mabury Road	Terrasearch	27 feet (June 1980)

It should be noted that groundwater depth is subject to seasonal fluctuations depending on water level in the creek, rainfall, irrigation, pumping in wells, or other factors that may not be evident at the time of our investigation.

3.6 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions, as described in this report, are based on data obtained from subsurface exploration and laboratory testing for this study. Our preliminary conclusions and preliminary recommendations are based on these interpretations. Please realize the project area has undergone different phases of development and grading. Therefore, it is likely that undisclosed variations in subsurface conditions exist within the project area, 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.

3.7 CORROSION POTENTIAL

Four soil samples from borings advanced for three of the proposed pedestrian bridges were tested for corrosion potential by CERCO Analytical. The test results, included after Figure C-17 in Appendix C, are again tabulated below.

Drill Hole and Depth	pH	Sulfate (ppm)	Resistivity (ohm-cm)	Chloride (ppm)
DH-1 @ 24 feet	8.3	20	3100	N.D.
DH-8 @ 39 feet	8.3	48	1900	N.D.
DH-10 @ 29 feet	8.8	88	1200	41
DH-10 @ 59 feet	8.3	49	1700	18

According to Corrosion Guidelines Version 1.0, dated September 2003, prepared by Corrosion Technology Branch, Materials Engineering and Testing Services, Division of Engineering Services, California Department of Transportation, a site is considered to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site.

> *Chloride concentration is 500 ppm or greater, sulfate concentration is 2,000 ppm or greater, or the pH is 5.5 or less.*

Based on the above guidelines and laboratory test results, the samples tested may be considered as “non-corrosive.”

4. DISCUSSION AND CONCLUSIONS

4.1 GENERAL

General preliminary conclusions and preliminary design recommendations for the project are presented in the following sections of this report. For this preliminary investigation, subsurface exploration was limited to selected locations. For final design, a qualified geotechnical engineer should review the information in this report and determine the level of additional subsurface exploration, laboratory testing and engineering analysis necessary to support completion of project design and construction. Modifications to the preliminary recommendations in this report may be necessary.

4.2 SURFACE RUPTURE AND SEISMIC GROUND SHAKING

Because the project area is not located within a State of California Earthquake Fault Zone and no mapped active faults are known to cross the project area, the probability of ground surface rupture at the project area due to displacement along a fault is remote.

The project area is in an area of high seismicity. Based on general knowledge of the local seismicity, it should be anticipated that, during its useful life, the project area will be subject to at least one severe earthquake (magnitude 7 to 8+) that could cause considerable ground shaking. It is also anticipated that the project area will periodically experience small to moderate magnitude earthquakes. Proposed improvements should be designed accordingly.

4.3 LIQUEFACTION AND LATERAL SPREADING

Soil liquefaction is a phenomenon in which saturated, cohesionless soils (generally sands) lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and silts. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type and relative density; 3) overburden pressure; and 4) depth to ground water.

The project area is located in a Santa Clara County Liquefaction Hazard Zone. Holzer and others (2008) conclude that there is a liquefaction probability of 30% to 40% along the project segment in the event of their scenario earthquake on the San Andreas fault (M7.8). The likelihood of liquefaction for the corresponding Calaveras fault scenario event (M6.9) is 10% to 20%, and 5% to 10% for a Hayward fault scenario (M7).

Our literature review indicates that there were reports of sand boils, lateral spreading and creek bank instability within the project areas as a result of the 1906 San Francisco earthquake.

Site-specific geotechnical information allows a more refined evaluation of liquefaction potential. Our liquefaction assessment was based on a PGA value of 0.4g (see Section 3.2), earthquake moment magnitude of 7.1, and a groundwater depth of 3 feet below ground surface which is the anticipated high water level in the creek. This assumption is conservative because the likelihood of a design earthquake occurring during high flood water in the creek is low.

The results of our liquefaction analysis suggest that some of the sand layers encountered in our drill holes and CPT probes could liquefy when subject to the design peak ground acceleration. These sand layers and their estimated liquefaction-induced ground settlements are presented below. Studies have shown that actual liquefaction-induced settlements could be 50 to 200 percent of the estimated values.

Drill Hole or CPT	Approximate Depth of Potentially Liquefiable Soils, feet	Estimated Liquefaction-induced Settlement, inches
DH-1	17 to 31½	3½
CPT-1	3 to 10½; 39 to 40	2½
DH-3	5½ to 13	2¾
DH-4	2 to 9½	½
CPT-2	8 to 10; 48 to 49	½
CPT-3	15 to 16; 32½ to 34	½
DH-8	4 to 12; 52 to 57	1½
DH-9	4 to 10	1

Lateral spreading is horizontal movement of soil toward a free face, such as a creek bank, typically associated with liquefaction. Liquefaction-induced lateral spreading can also occur on mild slopes (flatter than 5%) underlain by loose sands and a shallow groundwater table. If liquefaction occurs, the unsaturated overburden soil can slide as intact blocks over the lower, liquefied deposit, creating fissures and scarps. The potential for liquefaction-induced lateral spreading tends to mirror the potential for liquefaction for the area.

We have assessed the potential liquefaction-induced lateral displacements for a 20-foot high, 2:1 (horizontal:vertical) creek bank based on the procedures proposed by Youd, Hansen and Barlett (2002). We have assumed a design earthquake with a moment magnitude of 7.1 and a seismic source located 3 miles (4.8 km) from the site. The liquefiable soil layers are assumed to have a total thickness of 8 feet, an average fines content of 15%, and an average median diameter (D_{50}) of 3 mm. With these parameters, an estimated liquefaction-induced lateral displacement of about 4 to 5 feet was calculated. The potentially liquefiable soil layers below the invert of the creek channel are generally anticipated to have minor effect on lateral displacements of the creek bank.

Methods for mitigation of the potential impact of soil liquefaction include deep foundations and ground improvements. Deep foundations may include drilled piers or various types of piles, extending below the zone of liquefaction. Ground improvements may include stone columns, grouting, etc.

During the final design phase, a supplemental evaluation of liquefaction potential should be performed, preferably with CPT probes, to refine the findings of this preliminary study. Where liquefaction is found to be a concern, analysis to evaluate liquefaction-induced lateral spreading should be performed.

4.4 APPROACH FILLS AND BRIDGE LOADS

Approach fills will be required for the proposed pedestrian bridges except for the bridge over Penitencia Creek near Berryessa Road. These fills will be up to about 13 feet high and will be

confined by retaining structures. Depending on their final design, the fills will impose additional loads on the existing creek banks which may result in settlement of the underlying soils and a reduction of stability of the creek banks.

Similarly, the bridge loads will also impose additional stresses on the supporting soils which may result in a reduction of stability of the creek banks.

To evaluate the potential reduction in factor of safety of the creek banks as a result of the approach fill weight and bridge loads, we have performed a qualitative slope stability assessment at each proposed bridge location except for the bridge at Penitencia Creek. For this assessment, we first calculated a base factor of safety for the existing conditions using the limited subsurface information from this preliminary geotechnical investigation. The base factor of safety is an index, not the actual factor of safety of the creek bank. We then applied the anticipated approach fill weight at the top of the creek bank and calculated a new factor of safety. This second factor of safety is compared to the base factor of safety to evaluate the potential reduction in factor of safety. We then added the bridge loads to the fill weight and calculated the factor of safety under this condition for comparison with the base factor of safety. A summary of our assessment is tabulated below. Please note this assessment is qualitative for comparison purposes.

Proposed Bridge Location	**Approximate Fill Height (feet)**	**Preliminary Bridge Loads per Abutment (Kips)**	**Potential Reduction in Creek Bank Factor of Safety**	
			Fill Weight Only	**Fill Weight + Bridge Loads**
South of Montague Expressway	2½	200DL + 97.5 LL	10% - 15%	30% - 45%
West of Oakland Road	8 – 13	310DL + 116.5LL	10%	30%
Notting Hill & Hazlett Way	7 – 9	320DL + 121.5LL	25%	50%
North of Highway 101	11½	185DL + 86.5LL	5% - 10%	30%

During final design, a supplemental geotechnical investigation should be performed to include a more refine determination of the magnitude of settlement that may occur under the weight of the new fills and foundations, and the stability of the creek banks under the added stresses of the new fills and bridge loads.

Mitigation measures should be developed accordingly. Possible mitigation alternatives include use of light weight materials as approach fills, reinforcing the slopes for added stability and constructing the approaches as bridge structures instead of earth ramps. Light weight materials may include light weight fill, cellular foam concrete, and expanded polystyrene (EPS) geofoam. Typical unit weight for cellular foam concrete is about 25 to 55 pounds per cubic foot. EPS geofoam is a light weight, rigid foam plastic that weights about 1 to 2 pounds per cubic foot. Additional information for geofoam is available at the Federal Highway Administration website (http://www.fhwa.dot.gov/crt/lifecycle/geofoam.cfm) and the Syracuse University Geofoam Research Center website (http://geofoam.syr.edu).

4.5 CREEK BANK STABILITY

Failures of the existing creek bank were observed in several areas between Montague Expressway and O'Toole Avenue. Evaluation of creek bank stability is not included in the scope of this investigation. The SCVWD should be consulted regarding these observed failures and their repair.

4.6 SOIL EXPANSION

The near-surface soils consist of lean clay with low plasticity which generally corresponds to a low expansion potential.

5. PRELIMINARY RECOMMENDATIONS

5.1 EARTHWORK

Earthwork construction should conform to the City of San Jose Standard Specifications, latest edition, and the requirements of Santa Clara Valley Water District. General guidelines are presented below.

5.1.1 Clearing and Grubbing

Clearing and grubbing should conform to Section 16 of the City of San Jose Standard Specifications and should include clearing of existing structures, utility lines to be abandoned, deleterious materials, debris, obstructions, and stumps and primary roots of trees and brush (roots over 1 inch in diameter or longer than about 3 feet in length). Depressions, voids and holes that extend below the proposed finish grade should be cleaned and backfilled with engineered fill.

Surface vegetation and organic laden soils should be stripped. Organic laden soils are defined as soils with more than 3 percent by weight of organic content. The required stripping depth should be determined in the field by the Engineer at the time of construction. Stripped material may be stockpiled for use in landscape areas if approved by the project landscape architect, or otherwise removed from the site.

5.1.2 Excavations

Excavations for this project are anticipated to generally include cuts to achieve design grades, trenching for underground utilities, and excavations for foundations. Construction, shoring and bracing of excavations should comply 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.

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.

5.1.3 Subgrade Preparation

Subgrade preparation should be performed after the construction areas have been properly cleared and stripped, and after excavations to design subgrade elevations in cut areas. Subgrade preparation should conform to Section 21 of the City of San Jose Standard Specifications.

Soil with moisture content above optimum value should be anticipated during and shortly after rainy seasons, or in areas with irrigation. Where unstable, wet or soft soil is encountered, the soil will require processing before compaction can be achieved. When construction schedule does not allow air-drying, other means such as lime treatment of the soil or excavation and replacement 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.

5.1.4 Material 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 general engineered fill to achieve project grades, except when special material is required.

In general, 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 15 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a low expansion potential as indicated by Plasticity Index of 12 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.

5.1.5 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 requirements in Section 19 of the City of San Jose Standard Specifications.

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.

Except where 95 percent relative compaction is required, engineered fills consisting of on-site or imported soils should be compacted to a minimum of 90 percent relative compaction. The moisture content of the material should be brought to between 1 and 3 percent above the laboratory optimum value before compaction is performed.

5.1.6 Utility Trench Excavation and Backfill

Trench excavation, bedding and backfill should conform to Section 1301 of the City of San Jose Standard Specifications. Construction, shoring and bracing of excavations should comply 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.

5.1.7 Wet Weather Construction

If site grading and 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. The contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of 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 submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

5.2 BRIDGE FOUNDATIONS

Preliminary foundation design recommendations for the proposed pedestrian bridges are presented below. These recommendations are based on subsurface exploration near selected bridge support locations. For final design, additional subsurface exploration, laboratory testing and engineering analysis will be necessary to validate and/or supplement these design recommendations. Modifications to the preliminary design recommendations may be necessary.

5.2.1 Drilled Piers

The proposed bridges may be supported on drilled, cast-in-place, reinforced concrete piers deriving their vertical support capacity by adhesion between the pier shafts and the surrounding soils. Piers should have a minimum diameter of 24 inches. Piers should have a minimum length of 10 feet below finish grade or pier cap, whichever provides a deeper embedment. Center to center spacing of the piers should be a minimum of 3 pier diameters. Reinforcement in the piers should be determined by the structural engineer.

For dead plus live loading, a net allowable adhesion value of 450 pounds per square foot may be assumed along the pier shafts. This value may be increased by one-third when including transient loads, such as wind or seismic.

End bearing capacity should be ignored for friction piers. For piers located near or on a slope, the upper portion of each pier which is closer than 5 feet horizontally from the face of slope should be ignored in the calculation of load capacity, including compression, tension and lateral loads. If liquefiable soils are encountered, capacity from the liquefiable soil zones should be neglected and downdrag forces due to soil liquefaction should be included in the pier design. This should be determined during the supplemental geotechnical investigation.

Lateral deflections, shears and moments in the piers should be evaluated based on loading information provided by the structural engineer during the final design phase.

Settlements are expected to be primarily elastic with the majority of the settlement occurring immediately upon application of load. Post construction settlement of the proposed pier foundation system is anticipated to be less than ½ inch.

Granular soils were encountered in our drill holes and CPT probes. Steel casing should be anticipated to prevent caving in the pier holes. If water is encountered in the pier holes, concrete should be placed by the tremie method to displace the water out of the holes.

5.3 RETAINING WALLS

Retaining walls up to about 13 feet high will be constructed along the approach ramps for the proposed bridges and under-crossings. Other retaining structures include bridge abutment and wing walls and site retaining walls.

5.3.1 Lateral Pressures

Lateral pressures on the walls will be induced by the weight of retained soil, adjacent external loads, seismic surcharge force, and hydrostatic pressure. Lateral pressures will depend on the type of walls, whether wall movements are allowed or desired, backfill type, backfill slope gradient, magnitude of external loads, design water elevation, and subsurface drainage provisions. Our preliminary recommendations for design of cantilever retaining walls are presented below.

Soil Pressure	Normal Condition[1]	Rapid Drawdown[2]	High Flow[3]
Level Backfill Slope			
At-rest[6]	60 pcf	95 pcf	35 pcf
Active[4]	40 pcf	85 pcf	25 pcf
Seismic[7]	24 pcf	---	---
2:1 (horizontal:vertical) Backfill Slope			
At-rest[6]	92 pcf	95 pcf	45 pcf
Active[4]	60 pcf	110 pcf	30 pcf
Seismic[7]	24 pcf	---	---
Passive[5]	350 pcf	350 pcf	200 pcf

Notes:

1. Normal condition assumes water is below wall base or footing on both creek and backfill sides of wall.
2. Rapid drawdown condition assumes water on creek side is below wall base or footing and water on backfill side is at high flood water level.
3. High flow condition assumes water level is at high flood water level on both creek and backfill sides.
4. To develop active soil pressures, wall movements of about 0.005H to 0.01H would be necessary for cohesive soils and 0.005H for cohesionless soils.
5. To develop passive soil pressures against wall footings, movements of up to about $0.04D_f$ would be necessary for cohesive soils and $0.005D_f$ for cohesionless soils, where D_f is the footing embedment depth. Wall footings should be set back horizontally a minimum of $2D_f$ from the top of downslope creek bank.
6. Walls that can tolerate very 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.
7. The project structural engineer should decide whether to include seismic forces in the design of the retaining walls. Seismic surcharge is not included for rapid drawdown and high flow conditions because the probability of earthquake occurring simultaneously during rapid drawdown and high flow is generally considered low.
8. Slopes below the retaining walls must be protected from erosion and scour.
9. Additional surcharges such as potential lateral force from the adjacent bridge abutment to be determined by the project structural engineer.

We understand that mechanically stabilized embankment (MSE) systems may be used for the approach fills. It is likely that back-to-back MSE systems will be used in many locations because of the width to height ratio. For preliminary planning purposes, the minimum width to height ratio of the back-to-back MSE systems should be about 0.8. For preliminary design

purposes, the internal friction angle of the MSE backfill should be a minimum of 32 degrees. If cellular foam concrete or geofoam is to be considered as MSE backfill, special considerations should be developed during the final design phase.

5.3.2 Footing Foundations for Retaining Walls

Footings may be used for retaining walls located on relatively flat ground and if the entire wall footing is set back a minimum of 5 feet horizontally from the crest or face of creek bank at the footing bearing level. For walls that are located closer than 5 feet from the crest of creek bank or on the creek bank or sloping ground, drilled piers should be used to support the walls.

Footings may be designed using a net allowable soil bearing value of 2,500 pounds per square foot for dead plus live loads, with a one-third increase when including short-term wind or seismic loads. Footings should have a minimum width of 2 feet. The bottom of footings should be a minimum of 18 inches below finish grade and should have a minimum horizontal setback of 5 feet from the face of the adjacent creek bank or slope at the bottom of footing level.

Lateral loads may be resisted by a combination of friction between the footing bottom and the supporting soil and by passive resistance acting against the vertical side of the footings. For footings founded on undisturbed native soils, an ultimate coefficient of friction of 0.3 may be used. For footings poured neat against undisturbed native soils and located horizontally at least 2 times their embedment depth from the crest of the adjacent downslope creek bank (e.g. 3 feet horizontal setback for 1.5 feet footing embedment depth), the ultimate passive resistance given in the table in Section 5.3.1 above may be used. For footings located closer than the recommended minimum horizontal setback to the adjacent creek bank or slope, the passive resistance should be reduced by 50%.

5.3.3 Drilled Pier Foundations for Retaining Walls

Drilled piers may be used to support retaining walls that are located within 5 feet of crest of creek bank (or slope) or on creek bank (or slope). The parameters given in Section 5.2.1 “Drilled Piers” may be used for preliminary design of the foundation piers except that the minimum diameter of the drilled piers may be reduced to 18 inches.

Lateral loads against drilled piers supporting retaining walls may be evaluated using a passive soil pressure of 300 psf/ft acting on 1.5 pier diameters provided there is a minimum of 5 feet horizontal setback between the pier and the adjacent creek bank or slope. If the piers are closer to the creek bank or slope, the passive resistance would be reduced. The reduction should be evaluated during project final when locations of the piers and walls are finalized.

5.4 EXTERIOR FLATWORK

Preparation of subgrade soil and placement and compaction of engineered fill should be as outlined in the “Earthwork” section of this report. Soil subgrades should be maintained in a moist condition prior to pouring concrete slabs.

Concrete slabs should be constructed on a minimum 4-inch thick layer of Class 2 Aggregate Base compacted to a minimum of 90 percent relative compaction.

5.5 TRAIL PAVEMENTS

The proposed trail pavements will be subject predominantly to pedestrian and bicycle loading, with occasional maintenance vehicles and rare emergency vehicles. Soil subgrades for the trail pavements should be prepared to the recommendations presented in the “Earthwork” section of this report. Soil subgrades should be maintained in a moist condition prior to construction of the trail pavements.

We recommend a minimum pavement section of 2.5 inches of asphalt concrete over 4 inches of Class 2 Aggregate Base for the proposed trail. The aggregate base should be compacted to a minimum of 95 percent relative compaction based on ASTM Method D-1557, latest edition. This section requires periodic maintenance and repair, especially if subject to loading from a heavy vehicle.

During the final design phase, the existing gravel base along much of the trail alignment should be tested to evaluate if the material meets Caltrans requirements for Class 2 Aggregate Base. If the material meets Caltrans requirements, project design may consider re-using the existing gravel base material instead of importing new aggregate base.

5.6 SURFACE DRAINAGE

Engineering design of grading and drainage is the responsibility of the project Civil Engineer. Sufficient surface drainage should be provided to direct runoff away from structures, concrete flatwork 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, especially near structures, concrete slabs and pavements, could result in saturation of the soil and subsequent distress to site improvements. Trees should be planted away from structures, concrete slabs, pavements, etc. because tree roots could cause distress to those improvements. A qualified engineer and/or landscape architect should be consulted.

6. SUPPLEMENTAL GEOTECHNICAL INVESTIGATION

The preliminary recommendations in this report, based on information provided to us and our current understanding of the project, are for project feasibility-level design. Our subsurface exploration program was for selected locations and did not cover every proposed structure. We recommend that a supplemental geotechnical investigation be performed during the final design phase. This supplemental investigation should, as a minimum, address the following.

- Additional borings and/or CPT probes to evaluate foundation conditions at each bridge support, under-crossing and retaining wall location.
- Additional exploration, preferably with CPT probes, to assess liquefaction potential at the proposed structure locations.
- Perform liquefaction-induced lateral spreading analysis where potentially liquefiable soils are found.
- Finalize foundation type and foundation design parameters for the proposed structures, especially in areas with potentially liquefiable soils to determine downdrag loads on foundations and zones where foundation support should be neglected.
- Evaluate stability of the creek banks in high fill areas, such as in areas where thick approach fills are proposed.
- Evaluate potential settlement of underlying soils under the weight of the new approach fills.

7. LIMITATIONS

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

The preliminary 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 preliminary 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. For Pacific Geotechnical Engineering to remain the geotechnical consultant of record for the proposed project, we must provide supplemental geotechnical services during final design phase, plan review and construction observation services, as outlined above under the Plan Review, Earthwork and Foundation Observation 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 assumed in this report be encountered during project development, additional exploration, testing, and analysis may be required.

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 for project performance that the preliminary recommendations given in this report are made known to the design professionals involved with the project, that they be incorporated into project drawings and documents, and that the preliminary recommendations be validated and/or supplemented a design level geotechnical investigation.

Report prepared by,

PACIFIC GEOTECHNICAL ENGINEERING

Chalerm (Beeson) Liang
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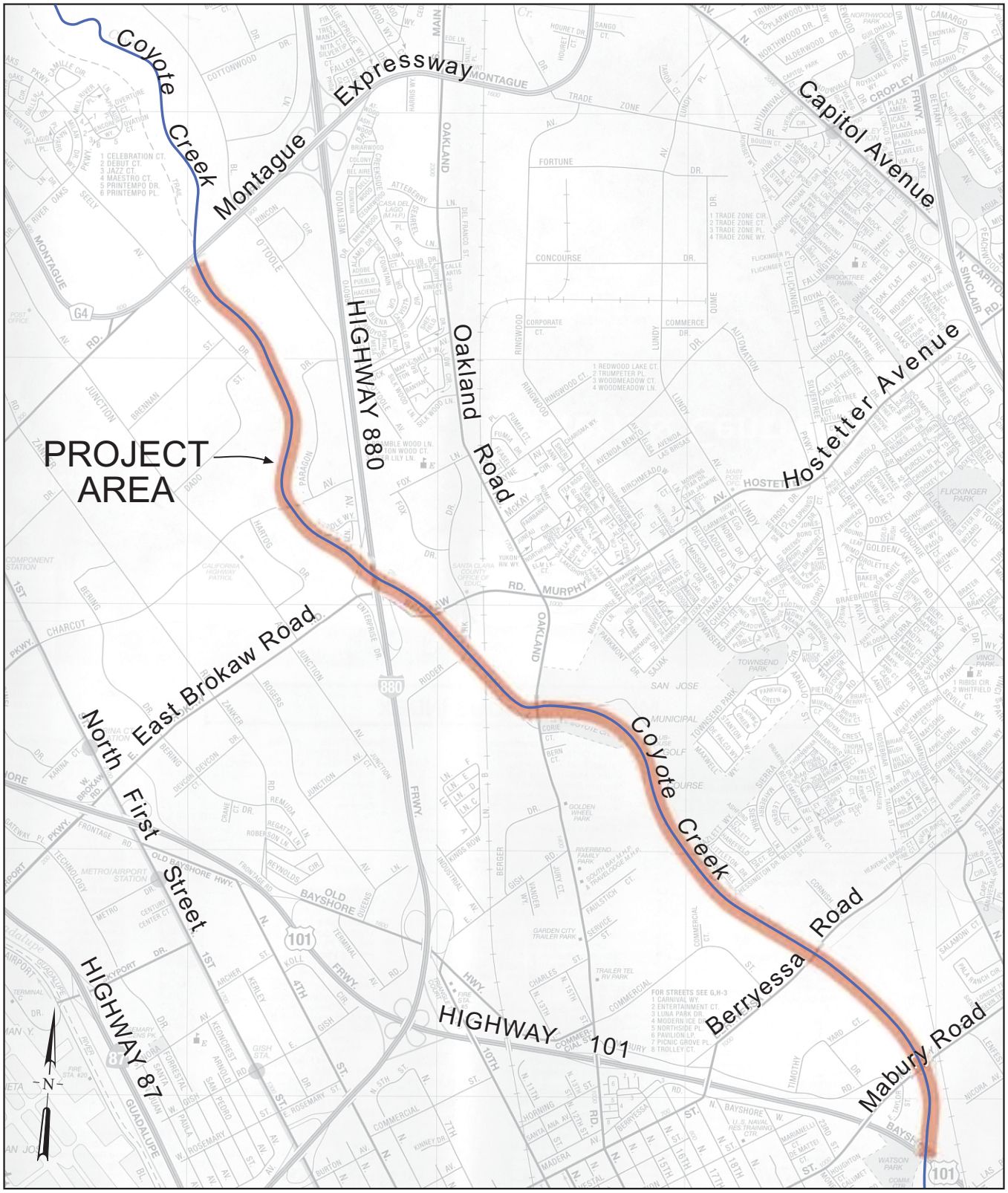
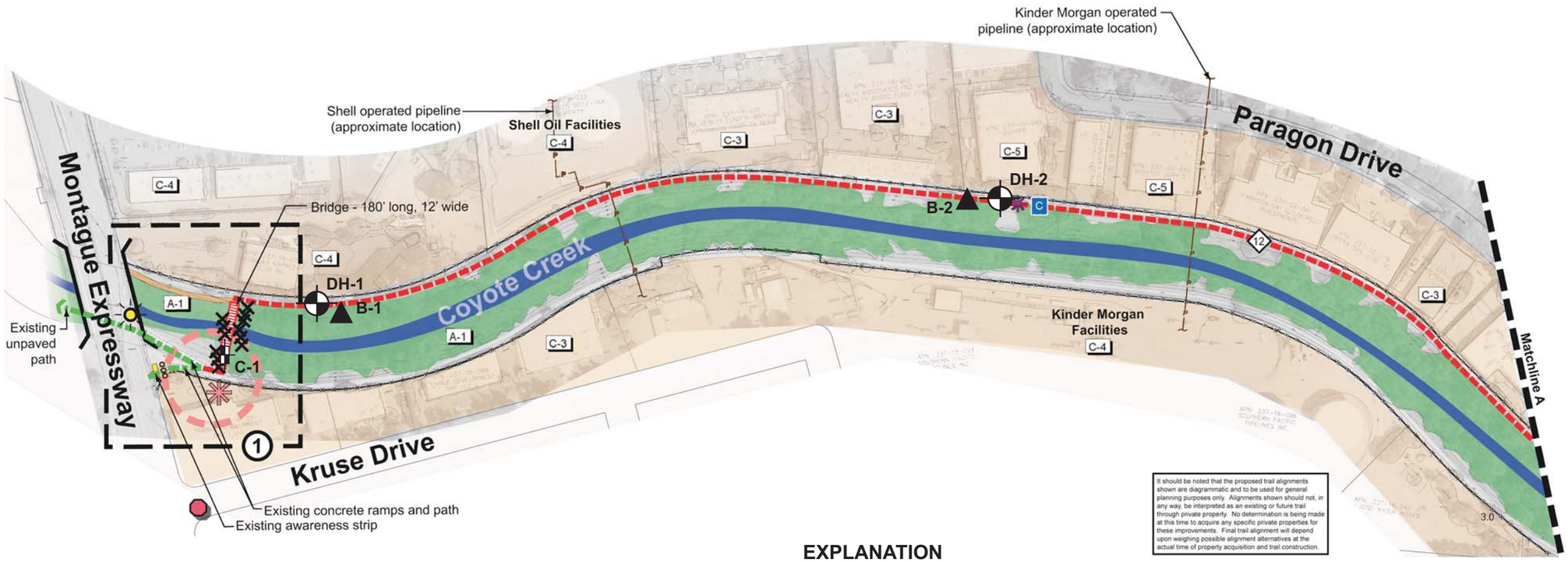
Copies: Marie Mai, Callander & Associates (4 hard copies + 1 electronic copy)



PACIFIC GEOTECHNICAL ENGINEERING	DATE MARCH 2010	**VICINITY MAP** **COYOTE CREEK TRAIL** **SAN JOSE, CALIFORNIA**	FIGURE 1 PROJECT 2298E



It should be noted that the proposed trail alignments shown are diagrammatic and to be used for general planning purposes only. Alignments shown should not, in any way, be interpreted as an existing or future trail through private property. No determination is being made at this time to acquire any specific private properties for these improvements. Final trail alignment will depend upon weighing possible alignment alternatives at the actual time of property acquisition and trail construction.

EXPLANATION

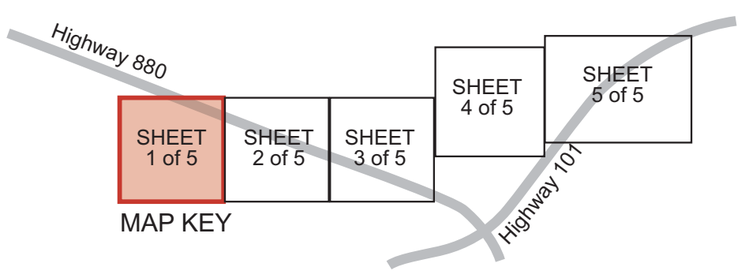
DH-10 Exploratory drill hole

C-3 CPT

B-10 Bulk sample

N

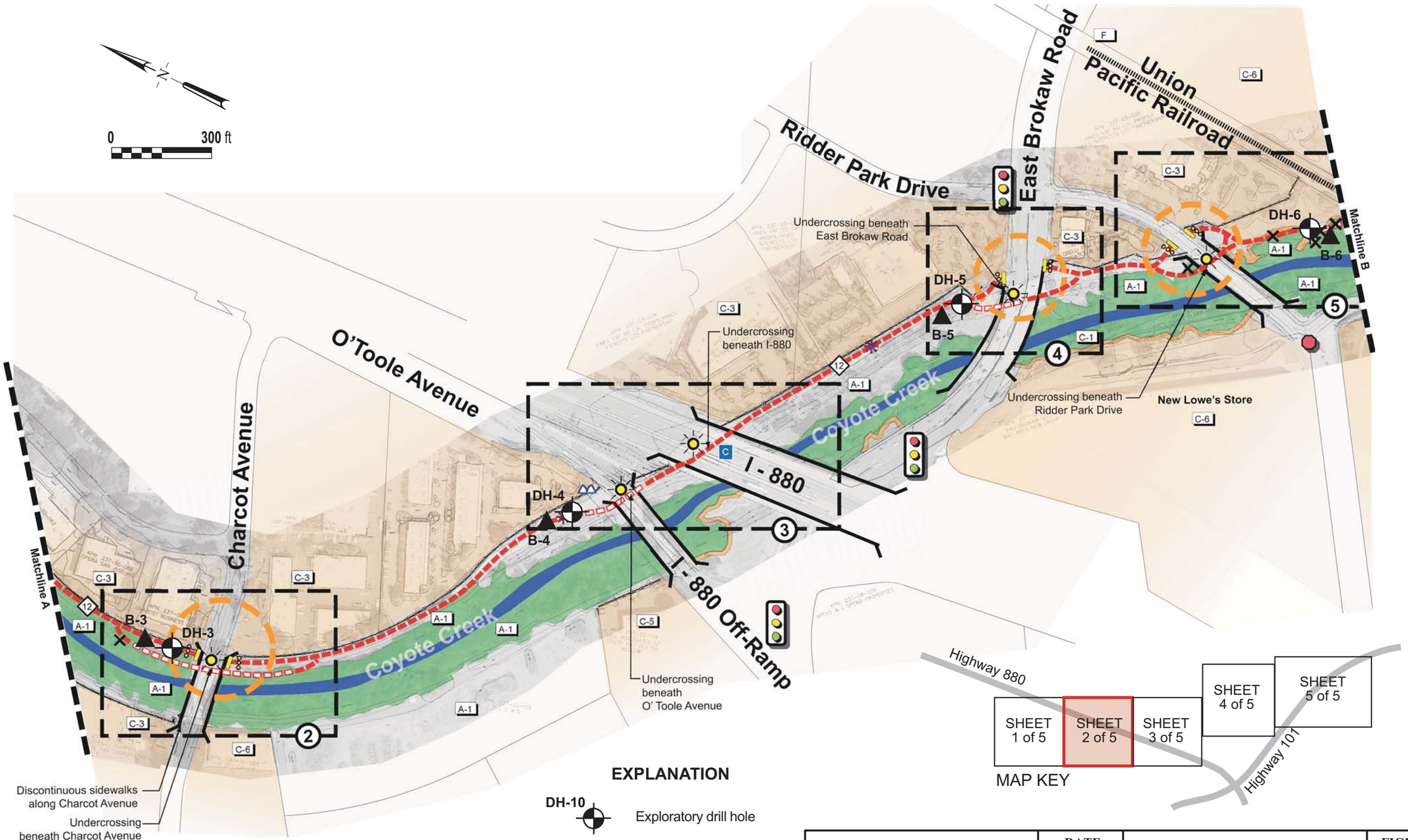
0 300 ft



MAP KEY

BASE: "Trail Alignment Plan, Coyote Creek Trail, Montague Expressway to Lower Silver Creek," by Callander Associates, dated July 1, 2009 (sheet 1 of 5).

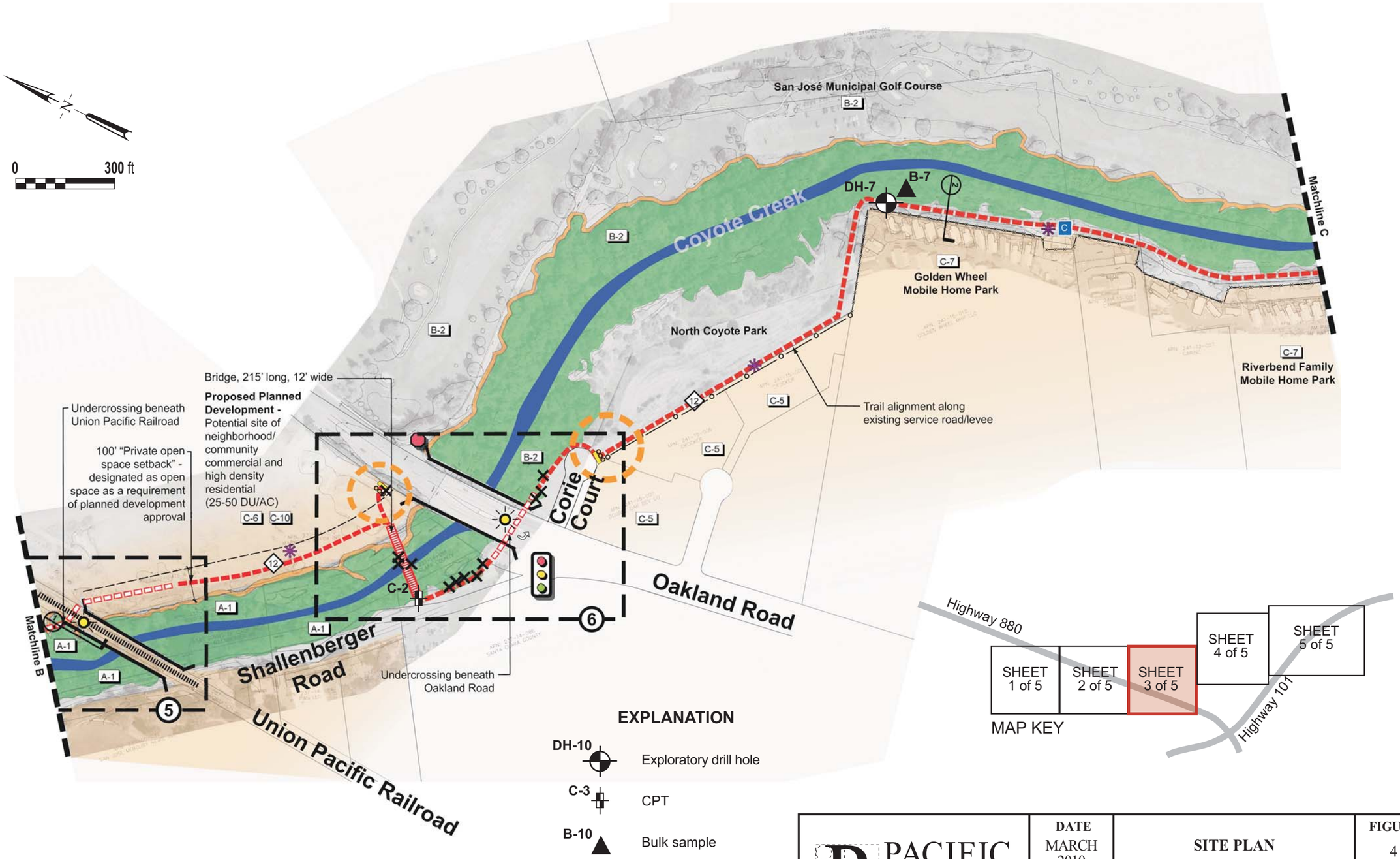
PACIFIC GEOTECHNICAL ENGINEERING	DATE MARCH 2010	SITE PLAN SHEET 1 of 5 COYOTE CREEK TRAIL SAN JOSE, CALIFORNIA	FIGURE 2
			PROJECT 2298E



BASE: "Trail Alignment Plan, Coyote Creek Trail, Montague Expressway to Lower Silver Creek," by Callander Associates, dated July 1, 2009 (sheet 2 of 5).

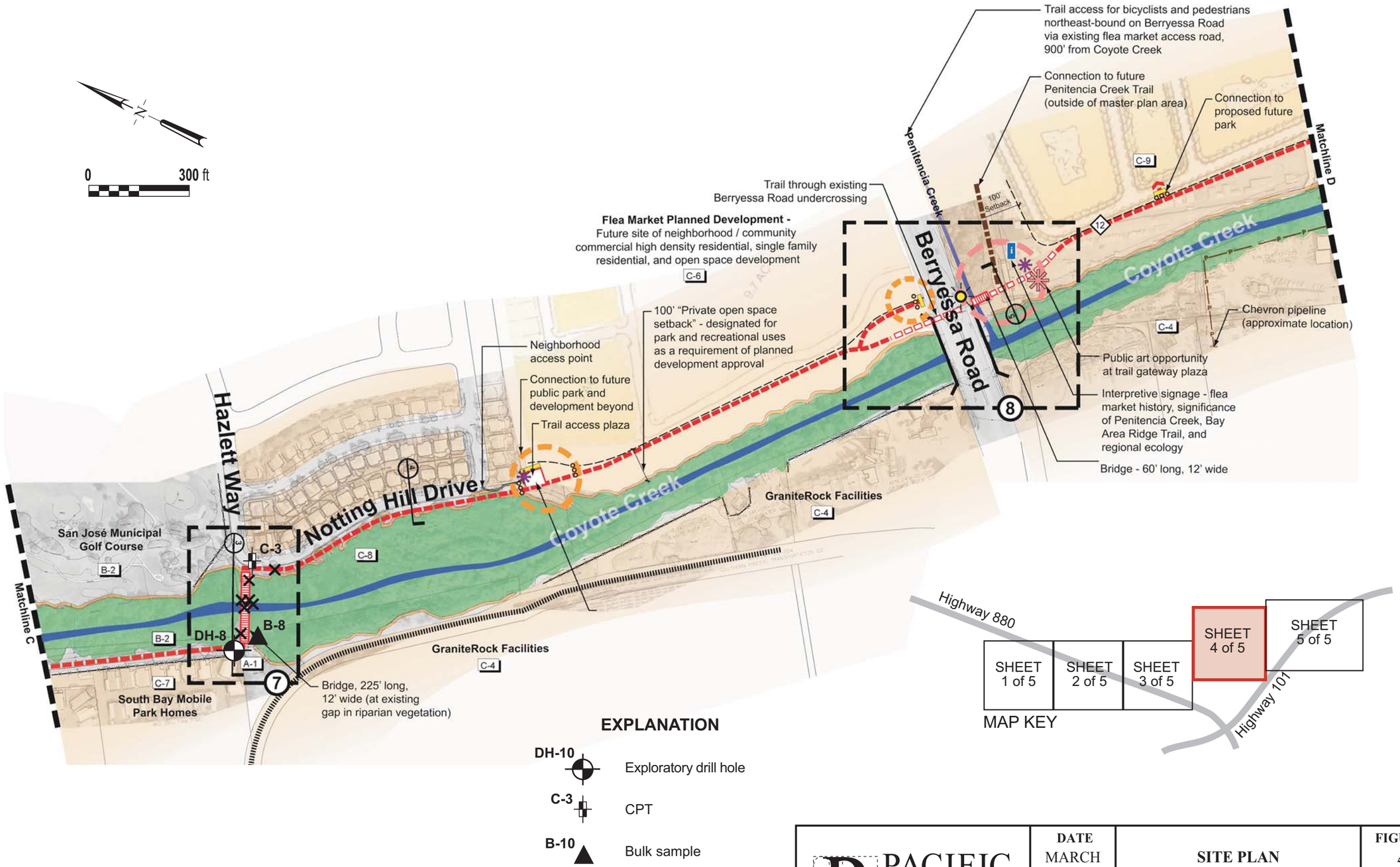
PACIFIC GEOTECHNICAL ENGINEERING

DATE	SITE PLAN	FIGURE
MARCH 2010	SHEET 2 of 5	3
	COYOTE CREEK TRAIL SAN JOSE, CALIFORNIA	PROJECT 2298E



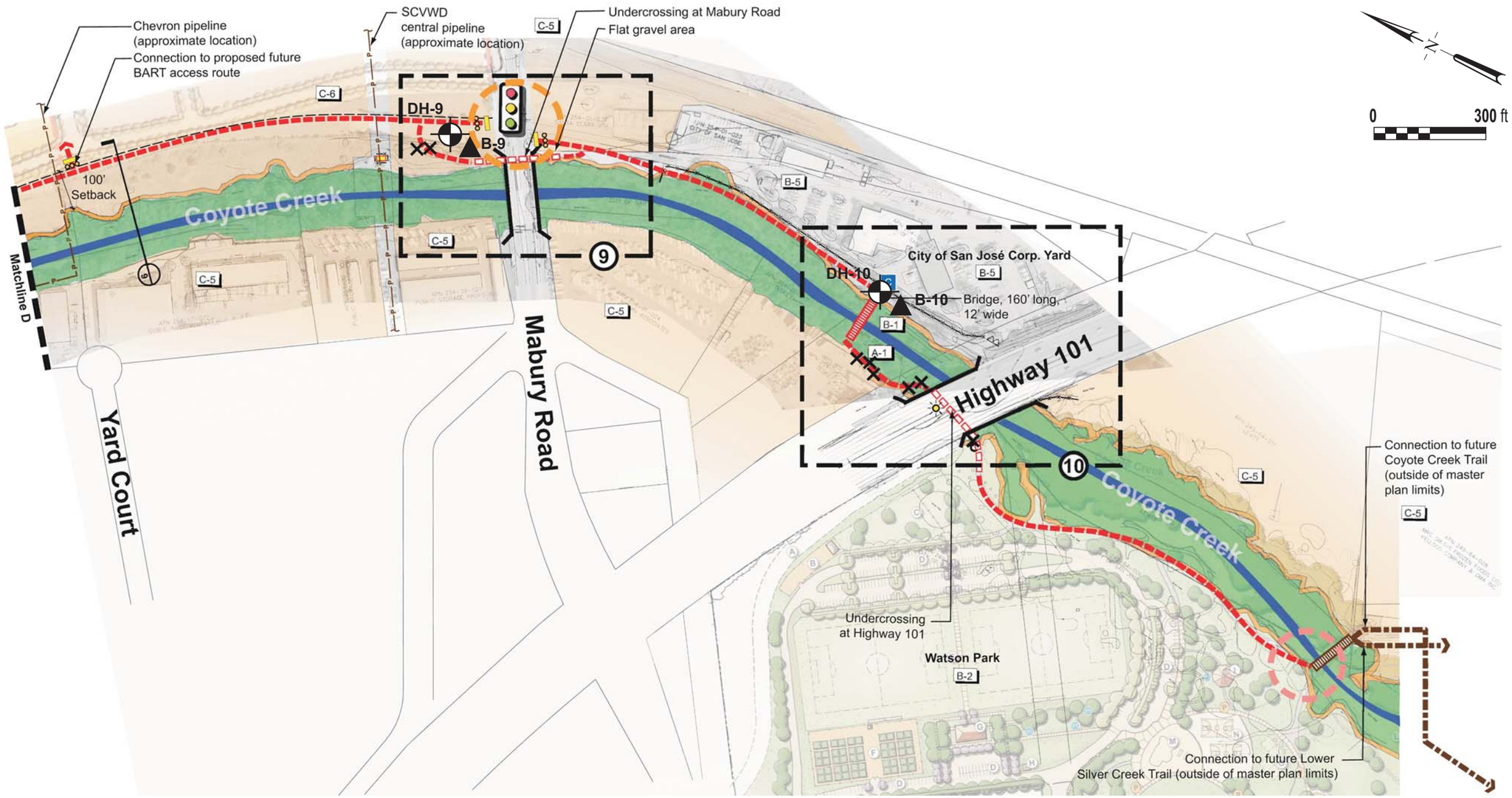
BASE: "Trail Alignment Plan, Coyote Creek Trail, Montague Expressway to Lower Silver Creek," by Callander Associates, dated July 1, 2009 (sheet 3 of 5).

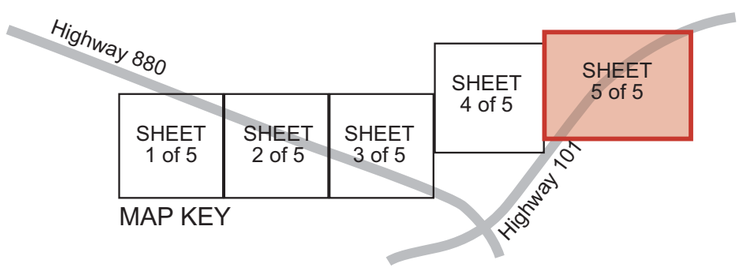
PACIFIC GEOTECHNICAL ENGINEERING	DATE MARCH 2010	**SITE PLAN** **SHEET 3 of 5** **COYOTE CREEK TRAIL** **SAN JOSE, CALIFORNIA**	**FIGURE** 4 **PROJECT** 2298E



BASE: "Trail Alignment Plan, Coyote Creek Trail, Montague Expressway to Lower Silver Creek," by Callander Associates, dated July 1, 2009 (sheet 4 of 5).

PACIFIC GEOTECHNICAL ENGINEERING	DATE MARCH 2010	SITE PLAN SHEET 4 of 5 COYOTE CREEK TRAIL SAN JOSE, CALIFORNIA	FIGURE 5 PROJECT 2298E





EXPLANATION

DH-10 — Exploratory drill hole

C-3 — CPT

B-10 — Bulk sample

BASE: "Trail Alignment Plan, Coyote Creek Trail, Montague Expressway to Lower Silver Creek," by Callander Associates, dated July 1, 2009 (sheet 5 of 5).

PACIFIC GEOTECHNICAL ENGINEERING	DATE MARCH 2010	SITE PLAN SHEET 5 of 5 COYOTE CREEK TRAIL SAN JOSE, CALIFORNIA	FIGURE 6
			PROJECT 2298E

APPENDIX A

KEYS TO SOIL CLASSIFICATION

AND

DRILL HOLE LOGS

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS

(50% OR MORE IS SMALLER THAN NO. 200 SIEVE SIZE)

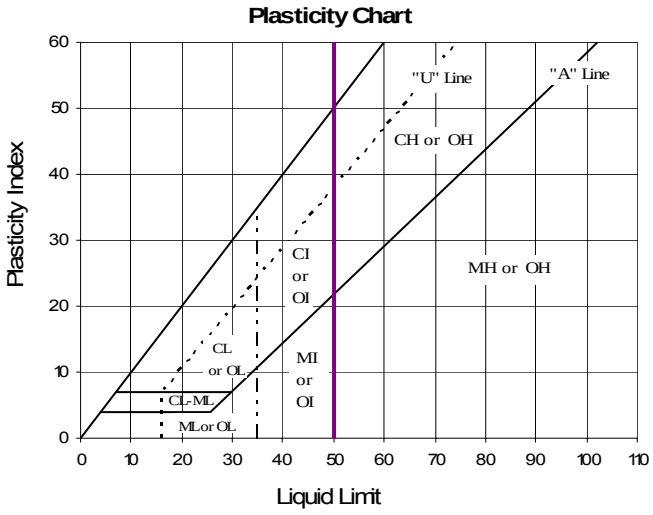
(modified from ASTM D2487 to include fine grained soils with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES
SILTS AND CLAYS (Liquid Limit less than 35) Low Plasticity	Inorganic	PI < 4 or plots below "A" line	ML	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CL	Lean Clay, Lean Clay with Sand or Gravel, Sandy or Gravelly Lean Clay, Sandy or Gravelly Lean Clay with Sand or Gravel
	Inorganic	PI between 4 and 7	CL-ML	Silty Clay, Silty Clay with Sand or Gravel, Sandy or Gravelly Silty Clay, Sandy or Gravelly Silty Clay with Sand or Gravel
	Organic	See footnote 3	OL	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (35 ≤ Liquid Limit < 50) Intermediate Plasticity	Inorganic	PI < 4 or plots below "A" line	MI	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CI	Clay, Clay with Sand or Gravel, Sandy or Gravelly Clay, Sandy or Gravelly Clay with Sand or Gravel
	Organic	See footnote 3	OI	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (Liquid Limit 50 or greater) High Plasticity	Inorganic	PI plots below "A" line	MH	Elastic Silt, Elastic Silt with Sand or Gravel, Sandy or Gravelly Elastic Silt, Sandy or Gravelly Elastic Silt with Sand or Gravel
	Inorganic	PI plots on or above "A" line	CH	Fat Clay, Fat Clay with Sand or Gravel, Sandy or Gravelly Fat Clay, Sandy or Gravelly Fat Clay with Sand or Gravel
	Organic	See note 3 below	OH	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)

1. If soil contains 15% to 29% plus No. 200 material, include "with sand" or "with gravel" to group name, whichever is predominant.
2. If soil contains ≥30% plus No. 200 material, include "sandy" or "gravelly" to group name, whichever is predominant. If soil contains ≥15% of sand or gravel sized material, add "with sand" or "with gravel" to group name.
3. Ratio of liquid limit of oven dried sample to liquid limit of not dried sample is less than 0.75.

CONSISTENCY	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 – 0.5	2 – 4
FIRM	0.5 – 1.0	5 – 8
STIFF	1.0 – 2.0	9 – 15
VERY STIFF	2.0 – 4.0	16 – 30
HARD	> 4.0	> 30

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below the water table



PACIFIC GEOTECHNICAL ENGINEERING

KEY TO SOIL CLASSIFICATION – COARSE GRAINED SOILS
(MORE THAN 50% IS LARGER THAN NO. 200 SIEVE SIZE)
(modified from ASTM D2487 to include fines with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES[1]
GRAVELS (more than 50% of coarse fraction is larger than No. 4 sieve size)	Gravels with less than 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	GW	Well Graded Gravel, Well Graded Gravel with Sand
		Cu < 4 and/or 1 > Cc > 3	GP	Poorly Graded Gravel, Poorly Graded Gravel with Sand
	Gravels with 5% to 12% fines	ML, MI or MH fines	GW-GM	Well Graded Gravel with Silt, Well Graded Gravel with Silt and Sand
			GP-GM	Poorly Graded Gravel with Silt, Poorly Graded Gravel with Silt and Sand
		CL, CI or CH fines	GW-GC	Well Graded Gravel with Clay, Well Graded Gravel with Clay and Sand
			GP-GC	Poorly Graded Gravel with Clay, Poorly Graded Gravel with Clay and Sand
	Gravels with more than 12% fines	ML, MI or MH fines	GM	Silty Gravel, Silty Gravel with Sand
		CL, CI or CH fines	GC	Clayey Gravel, Clayey Gravel with Sand
		CL-ML fines	GC-GM	Silty Clayey Gravel; Silty, Clayey Gravel with Sand
SANDS (50% or more of coarse fraction is smaller than No. 4 sieve size)	Sands with less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	SW	Well Graded Sand, Well Graded Sand with Gravel
		Cu < 6 and/or 1 > Cc > 3	SP	Poorly Graded Sand, Poorly Graded Sand with Gravel
	Sands with 5% to 12% fines	ML, MI or MH fines	SW-SM	Well Graded Sand with Silt, Well Graded Sand with Silt and Gravel
			SP-SM	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel
		CL, CI or CH fines	SW-SC	Well Graded Sand with Clay, Well Graded Sand with Clay and Gravel
			SP-SC	Poorly Graded Sand with Clay, Poorly Graded Sand with Clay and Gravel
	Sands with more than 12% fines	ML, MI or MH fines	SM	Silty Sand, Silty Sand with Gravel
		CL, CI or CH fines	SC	Clayey Sand, Clayey Sand with Gravel
		CL-ML fines	SC-SM	Silty, Clayey Sand; Silty, Clayey Sand with Gravel

US STANDARD SIEVES	3 Inch	¾ Inch	No. 4	No. 10	No. 40	No. 200
	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES & BOULDERS	GRAVELS		SANDS			SILTS AND CLAYS

RELATIVE DENSITY (SANDS AND GRAVELS)	STANDARD PENETRATION (BLOWS/FOOT)
Very Loose	0 - 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 - 50
Very Dense	50+

1. Add "with sand" to group name if material contains 15% or greater of sand-sized particle. Add "with gravel" to group name if material contains 15% or greater of gravel-sized particle.

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soi is below the water table

PACIFIC GEOTECHNICAL ENGINEERING

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 31 ft (20 min. after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 6 inches)												
SILTY CLAY: very dark grayish brown (10YR 3/2), dry to moist, hard	CL-ML	1	S D D	57	4.5+ 4.5+							
		2										
ALLUVIUM: SANDY SILT: dark grayish brown (10YR 4/2), dry to moist, loose to medium dense, with fine grained sand	ML	3										
		4										
		5	S D D	5	0.5 1.75	55		7	NP	82		
pores observed in samples; "NP" denotes non-plastic		6										
		7										
		8										
		9	S D D	12	1.0							
pores observed in samples		10										
		11										
		12										
LEAN CLAY to SILTY CLAY: mottled dark brown (10YR 3/3) and dark gray (2.5Y 4/1), moist, very stiff	CL/ CL-ML	13										
		14	S D D	22	3.0			18		106		
		15										
		16										
		17										
SILTY SAND: olive brown (2.5Y 4/3) and dark gray (2.5Y 4/1), moist, medium dense, fine grained sand	SM	18										
		19	S D D	15		34		16		102		
		20										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 31 ft (20 min. after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND (continued)	SM											
		21										
		22										
		23										
		24	S D D	18								
dark grayish brown (10YR 3/2), moist		25										
		26										
		27										
		28										
CLAYEY SAND: dark grayish brown (10YR 3/2), wet, loose to medium dense, fine grained sand	SC	29	S D D	13		37		22		104		
		30										
		31										
CLAY: brown (10YR 4/3), moist, stiff	CI	32										
		33										
		34	S I I	9	1.5							
		35										
		36										
		37										
		38										
		39	S D D	15	1.25 1.25			29		97	15	2096
		40										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
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I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 31 ft (20 min. after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued)	CI											
		41										
		42										
WELL GRADED SAND with GRAVEL and SILT: very dark grayish brown (10YR 3/2), wet, dense, fine to coarse grained sand and gravel	SW-SM	43										
		44	S I I	48								
		45										
		46										
		47										
		48										
very dense		49	S I I	93+		11		12				
		50										
		51										
		52										
		53										
		54	S I I	83		10		14				
		55										
		56										
		57										
		58										
		59	S I I	25								
see next page		60										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 31 ft (20 min. after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
LEAN CLAY: olive brown (2.5Y 4/3), moist, stiff to hard	CL											
		61										
		62										
		63										
dark gray (5Y 4/1)		64	S									
			D	58	2.2			22		106		
		65	D		4.5							
		66										
		67										
		68										
SILTY SAND: dark gray (10YR 4/1), wet, very dense; fine grained sand		69	S									
			D	30/6"								
	SM	70	D	50/4"								
BOTTOM OF HOLE = 70 Feet		71										
		72										
		73										
		74										
		75										
		76										
		77										
		78										
		79										
		80										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 2

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 4 inches)												
ALLUVIUM: SANDY CLAY: brown (10YR 5/3), dry, very stiff to hard; with fine grained sand and abundant roots and rootlets	CI	1	S D D	40	4.5+			14		105		
		2										
		3										
		4										
stiff		5	S D D	7	2.0							
		6										
		7										
		8										
dark olive brown (2.5Y 3/3), stiff to very stiff		9	S D D	19	2.5 4.5			19		101		
		10										
BOTTOM OF HOLE = 10 Feet No Groundwater Encountered		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 3

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 6 inches)												
ALLUVIUM: SILTY SAND: vey dark grayish brown (10YR 3/2), dry to moist, loose; fine to medium grained sand	SM	1										
		2	SDD	8				10		98		
		3										
		4										
		5	SDD									
POORLY GRADED SAND with SILT: very dark gray (10YR 3/2), dry, loose to medium dense; mostly fine to medium grained sand, with fine grained gravel	SP	6		9		5		5		97		
		7										
		8										
		9	SDD	12								
		10										
		11										
		12										
CLAY to FAT CLAY: very dark grayish brown (10YR 3/2), moist, stiff to very stiff	CI/ CH	13										
		14	SDD	13	2.0		49	26	25	91		
		15										
		16										
		17										
		18										
		19	SDD	21	2.5 3.0							
		20										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 3

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAYEY to FAT CLAY (continued)	CI/ CH	21										
		22										
		23										
		24	S									
blueish gray (Gley 2.5/1) mottled olive brown (2.5Y 4/3), stiff to very stiff			D D	10	1.0 2.5			27		100		
		25										
BOTTOM OF HOLE = 25 Feet No Groundwater Encountered		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 4

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 6 inches)												
LEAN CLAY to CLAY: very dark grayish brown (10YR 3/2), moist, firm	CL/ CI	1	S D D	32	0.5							
ALLUVIUM: CLAYEY SAND with GRAVEL: brown (10YR 4/3), dry to moist, medium dense; subangular to angular fine to coarse grained sand and gravel, minor cobbles	SC	2										
		3										
		4										
		5	S D D	29		21		12		108		
		6										
		7										
		8										
		9	S D D	36	4.5+	63		12		124		
SANDY CLAY: very dark grayish brown (10YR 3/2), moist, very stiff to hard; with mostly fine to medium grained sand	CI	10										
		11										
CLAY: brown (10YR 4/3), moist, very stiff	CI	12										
		13										
		14	S D D	22	3.4			22		106		
		15										
		16										
CLAY to FAT CLAY: very dark gray (10YR 3/1), moist, very stiff	CI/ CH	17										
		18										
		19	S D D	23	3.0 3.5			24		102		
		20										

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 4

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY to FAT CLAY (continued)	CI/ CH	21										
		22										
		23										
		24	S									
mottled blueish gray (Gley 2.5/1) and dark yellow brown (10YR 4/4), stiff to very stiff			D	20	2.5			21		113		
		25	D		2.5							
BOTTOM OF HOLE = 25 Feet No Groundwater Encountered		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 3 to 4 inches)												
ALLUVIUM: LEAN CLAY WITH SAND: brown (10 YR 4/3), dry to moist, very stiff to hard; with fine grained sand	CL	1	S									
		2	D	22	4.5+			14		115		
		3	D									
		4										
stiff		5	S									
		6	D	8								
		7	D									
		8										
		9	S									
CLAY with SAND: very dark grayish brown (10YR 3/2), moist, very stiff; with fine grained sand	CI	10	D	15		81		30 20		110		
		11	D									
		12										
		13										
moist to wet		14	S									
		15	D	16	1.5 1.5							
		16	D									
CLAY to FAT CLAY: dark yellowish brown (10YR 4/6) mottled olive brown (2.5Y 4/3), moist, very stiff	CI/ CH	17										
		18										
		19	S									
		20	D	17	2.75 2.5			27		99		

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY to FAT CLAY (continued)	CI/ CH	21										
		22										
		23										
stiff to very stiff		24	S D D	16	1.25 1.25			33		92		
		25										
BOTTOM OF HOLE = 25 Feet No Groundwater Encountered		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 9/18/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 6

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
ALLUVIUM: LEAN CLAY with SAND: brown (10 YR 4/3), dry, very stiff to hard; with mostly fine grained sand	CL	1										
		2	S D D	22	3.5 4.5	61		7		99		
		3										
SILTY SAND: Brown (10YR 4/3), moist, medium dense; fine grained sand	SM	4										
		5										
CLAY: olive brown (2.5Y 4/4), dry to moist, very stiff to hard	CI	6	S D D	31	4.5+			14		109		
		7										
		8										
		9	S D D	16	4.5+ 4.5+		37	17	15	90		
		10										
		11										
		12										
		13										
		14	S I I	11								
		15										
		16										
		17										
		18										
mottled dark gray (10YR 4/1) and dark yellowish brown (10YR 3/6), moist, very stiff		19	S D D	24	3.0 3.25			26		100		
		20										

DATE: 9/18/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 6

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued)	CI											
		21										
		22										
		23										
stiff to very stiff		24	S I I	9	4							
		25										
BOTTOM OF HOLE = 25 Feet No Groundwater Encountered		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 7

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 6 inches)												
ALLUVIUM: CLAY WITH SAND: dark grayish brown (10YR 4/2), moist, very stiff to hard; with fine grained sand	CI	1										
		2	S D D	30	4.5+ 4.5+	71		16		101		
		3										
		4										
		5										
		6	S D D	50	3.0							
SILTY SAND with GRAVEL: brown (10YR 4/3), moist, medium dense, fine to coarse grained sand; with fine grained gravel	SM	7										
		8										
		9	S D D	21		44		7		107		
BOTTOM OF HOLE = 10 Feet No Groundwater Encountered		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 8

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 17 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 6 inches)												
FILL, SANDY LEAN CLAY: brown (10YR 4/3), dry to moist, hard; with fine to coarse grained sand	CL	1										
			S									
		2	D	80	4.5+							
			D		4.5+	55		13		121		
		3										
		4										
CLAYEY SAND: brown (10 YR 4/3), dry to moist, medium dense; mostly fine grained sand	SC											
		5	S									
		6	D	20				10		108		
			D									
		7										
		8										
		9	S									
			D	19		46		10		116		
		10	D									
		11										
		12										
LEAN CLAY: dark grayish brown (10YR 4/2), moist, firm to stiff	CL											
		13										
		14	S									
			D	9				24		97		
		15	D									
		16										
		17										
FAT CLAY: very dark gray (10YR 3/1), moist to wet, very stiff	CH											
		18										
		19	S									
			D	24	3.75							
		20	D		3.75		57	26	32	97		

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 8

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER: D = 3" OD, 2½" ID Split-spoon; X = 2½" OD, 2" ID Split-spoon; I = Standard Penetrometer (2" OD SPT); S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: 17 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
FAT CLAY (continued)	CH											
		21										
		22										
		23										
LEAN CLAY to CLAY: dark grayish brown (2.5Y 4/2), moist to wet, firm	CL/ CI	24	S D D	8	1.5 0.5			27		99	4	1008
		25										
		26										
		27										
		28										
SANDY LEAN CLAY to SANDY CLAY: dark grayish brown (2.5Y 4/2), moist to wet, stiff	CL/ CI	29	S D D	13	1.5							
		30										
		31										
		32										
		33										
CLAY to FAT CLAY: black (2.5Y 2.5/1), moist, stiff	CI/ CH	34	S D D	20								
		35										
		36										
		37										
		38										
		39	S D D	41	1.5			24		104	15	1787
firm to stiff		40										

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 8

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER: D = 3" OD, 2½" ID Split-spoon; X = 2½" OD, 2" ID Split-spoon; I = Standard Penetrometer (2" OD SPT); S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: 17 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY to FAT CLAY (continued)	CI/ CH	41										
		42										
		43										
		44	S I I	58		29		15		124		
CLAYEY SAND: dark grayish brown (10YR 4/3), wet, dense; fine to coarse grained sand	SC	45										
		46										
		47										
		48										
WELL GRADED SAND with GRAVEL and CLAY: dark brown (10YR 3/3), wet, very dense; fine to coarse grained sand and gravel	SW-SC	49	S D	50/6"		10		10		128		
		50										
		51										
		52										
POORLY GRADED SAND with CLAY: dark grayish brown (10YR 4/2), wet, medium dense to dense; fine to medium grained sand	SP-SC	53										
		54	S I I	32		11		22				
		55										
		56										
		57										
		58										
very dense		59	S I	50/5"								
		60										

DATE: 9/17/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 8

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 17 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
POORLY GRADED SAND with CLAY (continued)	SP-SC	61										
		62										
		63										
CLAY: brown (10YR 5/3), moist, very stiff	CI	64	S I I	46	2.0 2.3							
		65										
		66										
CLAYEY SAND: very dark grayish brown (10YR 3/2), wet, dense; fine grained sand	SC	67										
		68										
		69	S I I	48				16				
		70										
BOTTOM OF HOLE = 70 Feet		71										
		72										
		73										
		74										
		75										
		76										
		77										
		78										
		79										
		80										

DATE: 9/18/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 9

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** ----- **Final:** -----

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
GRAVEL BASE (approx. 6 inches)												
ALLUVIUM: SANDY LEAN CLAY: brown (10YR 5/3), dry, very stiff to hard; with fine grained sand	CL	1	S									
		2	D	31	4.25	68		9		107		
		3	D		4.5+							
		4										
SILTY SAND: brown (10YR 4/3), dry, medium dense; fine grained sand	SM	5	S									
		6	D	25		32		6		103		
		7	D									
		8										
		9	S									
		10	D	11	3.5			24		96		
CLAY to FAT CLAY: very dark gray (2.5Y 3/1), moist, stiff to very stiff	CI/ CH	11										
		12										
		13										
		14	S									
		15	I	11	3.1							
		16	I									
		17										
		18										
		19	S									
		20	D	11	3.25			34		88	6	2666
			D		3.5							

DATE: 9/18/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 9

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** ----- **Final:** -----

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY to FAT CLAY (continued)	CI/ CH	21										
		22										
		23										
		24	S D D	18	1.5 2.0			23				
		25										
BOTTOM OF HOLE = 25 Feet No Groundwater Encountered		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 10

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 27 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
FILL, LEAN CLAY: very dark gray (10YR 3/1), dry, hard	CL	1	S D D	45								
FILL, CLAYEY SAND: very dark grayish brown (10YR 3/2), dry to moist, very dense; fine to medium grained sand	SC	2										
		3										
		4										
		5	S D D	71		46		7		129		
		6										
		7										
		8										
		9	S	50/6" (no recovery)								
ALLUVIUM, SILT with SAND: yellowish brown (10YR 5/4), dry, loose to medium dense; with fine grained sand	ML	10										
		11										
		12										
		13										
		14	S I I	10			29	8	5			
		15										
		16										
CLAY: olive brown (2.5Y 4/3) mottled with dark yellow brown (10YR 4/6), moist, very stiff to hard	CI	17										
		18										
		19	S D D	33	4.5+ 4.5+	98		25		106		
		20										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 10

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 27 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued)	CI											
		21										
		22										
		23										
stiff		24	S I I	11								
		25										
		26										
		27										
		28										
		29	S D D	11	1.0 1.0			19		110	15	2065
CLAY to FAT CLAY: black (2.5Y 2.5/1), moist, stiff, with minor fine to coarse grained sand	CI/ CH	30										
		31										
		32										
		33										
black (10YR 2/1)		34	S D D	16				22		102		
		35										
		36										
		37										
		38										
		39	S D D	13	1.5							
		40										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 10

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER: D = 3" OD, 2½" ID Split-spoon; X = 2½" OD, 2" ID Split-spoon; I = Standard Penetrometer (2" OD SPT); S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: 27 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY to FAT CLAY (continued)	CI/ CH	41										
		42										
		43										
		44	S D D	19	3.0 3.25		45	26	25	100	6	2610
CLAY: mottled greenish gray (Gley 1 5/1) and dark gray (Gley 1 4/1), moist, stiff to very stiff, with minor fine to coarse grained sand	CI	45										
		46										
		47										
		48										
		49	S D D	12	1.75 2.0							
SANDY CLAY: very dark greenish gray (Gley 1 3/1) and dark gray (Gley 1 4/1), moist, stiff, with fine grained sand	CI	50										
		51										
		52										
		53										
		54	S I I	35								
encountered approx. 1 foot thick layer of POORLY GRADED SAND; dark olive brown (2.5Y 3/3), moist to wet, medium dense, mostly fine to medium grained sand		55										
		56										
		57										
		58										
CLAY: dark greenish brown (2.5Y 4/2), moist, stiff to very stiff, with fine grained sand	CI	59	S D D	47				20		110	15	2817
		60										

DATE: 9/16/2009	LOG OF EXPLORATORY DRILL HOLE	DH- 10

PROJECT NAME: Coyote Creek Trail **PROJECT NUMBER:** 2298E

DRILL RIG: Mobile B56, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 27 ft (after drilling)

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued):	CI											
		61										
		62										
		63										
		64	S									
			D	58	4.5							
		65	D		2.0							
		66										
	CL	67										
		68										
		69	S									
			D	56				27		104		
		70	D		1.75							
BOTTOM OF HOLE = 70 Feet No Groundwater Encountered		71										
		72										
		73										
		74										
		75										
		76										
		77										
		78										
		79										
		80										

APPENDIX B

CONE PENETROMETER TEST RESULTS

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)
Terminated at 45.0 feet

CPT NO.: CPT-1
DATE : 09-14-2009
TIME : 10:08:07
Groundwater estimated at 18.0 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)
0.56	331.0	529.66	2.41	0.7	55	88	0.06	48	----	Gravelly SAND to SAND	110-120
1.02	728.1	1164.9	4.60	0.6	121	194	0.12	>48	----	"	"
1.53	480.5	768.80	3.28	0.7	80	128	0.18	>48	----	"	"
2.01	93.5	149.52	0.93	1.0	23	37	0.24	40	----	SAND to Silty SAND	120-130
2.54	24.1	38.61	1.13	4.7	24	39	0.31	----	3.20	CLAY	130-140
3.04	33.1	52.93	1.57	4.7	22	35	0.37	----	4.39	Silty CLAY to CLAY	"
3.57	25.3	40.53	1.12	4.4	17	27	0.45	----	3.35	"	"
4.06	24.4	39.06	1.26	5.2	24	39	0.51	----	3.22	CLAY	"
4.57	25.8	41.23	1.29	5.0	26	41	0.58	----	3.40	"	"
5.01	32.4	51.78	1.04	3.2	16	26	0.64	----	4.27	Clayey SILT to Silty CLAY	"
5.50	29.5	47.18	0.86	2.9	15	24	0.71	----	3.88	"	"
6.00	33.1	52.90	0.49	1.5	11	18	0.77	34	----	Silty SAND to Sandy SILT	120-130
6.51	23.1	36.12	0.36	1.6	9	14	0.83	----	3.02	Sandy SILT to Clayey SILT	"
7.01	18.2	27.40	0.30	1.7	7	11	0.89	----	2.36	"	110-120
7.51	19.2	27.89	0.26	1.4	8	11	0.95	----	2.50	"	"
8.01	21.2	29.60	0.26	1.2	8	12	1.01	----	2.76	"	"
8.53	19.9	27.14	0.26	1.3	8	11	1.07	----	2.58	"	"
9.04	16.5	21.99	0.27	1.6	7	9	1.12	----	2.12	"	"
9.54	16.8	21.97	0.23	1.4	7	9	1.18	----	2.17	"	"
10.04	15.6	19.86	0.27	1.7	6	8	1.24	----	1.99	"	"
10.54	18.0	22.41	0.51	2.8	9	11	1.30	----	2.31	Clayey SILT to Silty CLAY	120-130
11.04	18.9	22.90	0.48	2.5	9	11	1.36	----	2.42	"	"
11.57	19.3	22.86	0.55	2.8	10	11	1.43	----	2.48	"	"
12.07	19.3	22.23	0.36	1.8	8	9	1.49	----	2.47	Sandy SILT to Clayey SILT	"
12.56	17.4	19.68	0.41	2.4	9	10	1.55	----	2.22	Clayey SILT to Silty CLAY	"
13.07	18.1	20.01	0.64	3.5	12	13	1.62	----	2.30	Silty CLAY to CLAY	"
13.57	13.3	14.39	0.42	3.2	9	10	1.68	----	1.66	"	"
14.07	14.3	15.16	0.60	4.2	14	15	1.74	----	1.79	CLAY	"
14.57	26.2	27.47	0.67	2.6	13	14	1.81	----	3.38	Clayey SILT to Silty CLAY	"
15.06	21.6	22.32	0.76	3.5	11	11	1.87	----	2.76	"	"
15.57	12.5	12.66	0.52	4.2	12	13	1.93	----	1.53	CLAY	"
16.08	11.6	11.57	0.44	3.8	12	12	1.99	----	1.76	"	"
16.57	12.5	12.52	0.51	4.1	13	13	2.06	----	1.53	"	"
17.08	12.9	12.89	0.50	3.9	13	13	2.12	----	1.58	"	"
17.58	12.7	12.60	0.53	4.2	13	13	2.18	----	1.54	"	"
18.08	13.6	13.49	0.54	4.0	14	13	2.21	----	1.66	"	"
18.54	14.6	14.50	0.63	4.3	15	14	2.24	----	1.79	"	"
19.03	13.0	12.97	0.58	4.4	13	13	2.27	----	1.58	"	"
19.53	14.5	14.41	0.60	4.2	14	14	2.30	----	1.77	"	"
20.02	17.5	17.38	0.74	4.2	17	17	2.33	----	2.17	"	"
20.51	18.2	18.10	0.98	5.4	18	18	2.37	----	2.26	"	130-140
21.02	20.9	20.70	1.26	6.0	21	21	2.41	----	2.61	"	"
21.58	19.8	19.64	1.02	5.1	20	19	2.45	----	2.46	"	"
22.07	21.2	21.03	1.18	5.5	21	21	2.48	----	2.65	"	"
22.56	22.2	21.79	1.17	5.3	22	22	2.52	----	2.76	"	"
23.04	21.4	20.84	1.32	6.1	21	21	2.55	----	2.67	"	"
23.53	23.1	22.17	1.49	6.5	23	22	2.59	----	2.88	"	"
24.00	25.8	24.46	1.60	6.2	26	24	2.62	----	3.23	"	"
24.50	27.6	25.88	1.74	6.3	27	26	2.66	----	3.48	"	"
25.06	28.2	26.06	1.80	6.4	28	26	2.70	----	3.55	"	"
25.55	29.4	26.78	1.77	6.0	29	27	2.74	----	3.70	"	"
26.03	28.6	25.79	1.56	5.4	28	26	2.77	----	3.60	"	"
26.58	24.6	21.87	1.43	5.8	24	22	2.81	----	3.06	"	"
27.05	28.4	24.88	1.63	5.7	28	25	2.85	----	3.56	"	"
27.52	25.5	22.04	1.43	5.6	25	22	2.88	----	3.17	"	"

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)
Terminated at 45.0 feet

CPT NO.: CPT-1
DATE : 09-14-2009
TIME : 10:08:07
Groundwater estimated at 18.0 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)
28.03	25.4	21.64	1.54	6.1	25	21	2.92	----	3.15	"	"
28.58	28.0	23.52	1.63	5.8	28	23	2.96	----	3.49	"	"
29.04	21.4	17.69	1.26	5.9	21	18	2.99	----	2.60	"	"
29.52	19.4	15.98	1.02	5.2	19	16	3.02	----	2.34	"	"
30.06	17.6	14.40	0.83	4.7	18	14	3.06	----	2.09	"	120-130
30.51	15.2	12.40	0.70	4.6	15	12	3.09	----	1.77	"	"
31.05	13.9	11.29	0.63	4.5	14	11	3.12	----	1.59	"	"
31.56	13.6	10.96	0.53	3.9	13	11	3.15	----	1.54	"	"
32.03	18.6	14.99	0.73	3.9	12	10	3.18	----	2.21	Silty CLAY to CLAY	"
32.56	16.6	13.25	0.54	3.3	11	9	3.21	----	1.93	"	"
33.03	18.7	14.93	0.75	4.0	12	10	3.24	----	2.22	"	"
33.57	24.3	19.22	0.82	3.4	12	9	3.28	----	2.95	Clayey SILT to Silty CLAY	130-140
34.02	24.0	18.91	0.94	3.9	16	12	3.32	----	2.91	Silty CLAY to CLAY	"
34.54	45.4	35.58	1.76	3.9	23	18	3.35	----	5.76	Clayey SILT to Silty CLAY	"
35.08	31.7	24.72	1.07	3.4	16	12	3.39	----	3.93	"	"
35.56	28.9	22.41	0.88	3.0	14	11	3.43	----	3.55	"	"
36.01	22.5	17.39	1.00	4.4	22	17	3.46	----	2.70	CLAY	"
36.53	94.2	72.32	2.49	2.6	38	29	3.50	----	12.25	Sandy SILT to Clayey SILT	"
37.03	327.7	250.40	3.17	1.0	66	50	3.53	43	----	SAND	120-130
37.51	330.0	251.41	1.64	0.5	55	42	3.55	43	----	Gravelly SAND to SAND	100-110
38.07	260.7	197.73	1.46	0.6	52	40	3.58	42	----	SAND	110-120
38.55	270.5	204.35	1.73	0.6	54	41	3.60	42	----	"	"
39.05	241.7	181.89	1.58	0.7	48	36	3.63	41	----	"	"
39.55	158.7	118.99	0.59	0.4	32	24	3.65	39	----	"	100-110
40.05	131.4	97.96	2.23	1.7	33	24	3.69	38	----	SAND to Silty SAND	130-140
40.53	274.1	203.79	1.30	0.5	46	34	3.71	42	----	Gravelly SAND to SAND	100-110
41.04	284.9	210.96	1.89	0.7	57	42	3.74	42	----	SAND	110-120
41.52	316.1	233.28	1.59	0.5	53	39	3.76	43	----	Gravelly SAND to SAND	100-110
42.00	339.4	249.67	1.31	0.4	57	42	3.78	43	----	"	"
42.54	315.8	231.50	1.38	0.4	53	39	3.80	43	----	"	"
43.02	330.8	241.69	1.36	0.4	55	40	3.82	43	----	"	"
43.50	243.9	177.61	0.94	0.4	41	30	3.84	41	----	"	"
44.06	309.1	224.12	1.87	0.6	52	37	3.87	43	----	"	110-120
44.54	295.7	213.83	0.99	0.3	49	36	3.89	42	----	"	90-100
45.02	253.5	182.58	1.43	0.6	51	37	3.91	41	----	SAND	110-120

DEPTH = Sampling interval (.1 feet)
Qc = Tip bearing resistance TotStr = Total Stress using est. density**
Fs = Sleeve friction resistance Phi = Soil friction angle*
Rf = Tip/Sleeve ratio Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf)
SPT = Equivalent Standard Penetration Test* (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)
References: * Robertson and Campanella, 1988
** Olsen, 1989 *** Durgunoglu & Mitchell, 1975

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)
Terminated at 70.0 feet

CPT NO.: CPT-2
DATE : 09-14-2009
TIME : 12:58:07
Groundwater estimated at 14.6 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	0.1	0.22	0.08	12.0	0	0	0.06	----	0.02	Organic Material	90-100
1.03	102.0	163.25	1.87	1.8	34	54	0.13	41	----	Silty SAND to Sandy SILT	130-140
1.53	172.8	276.42	4.95	2.9	58	92	0.19	44	----	"	"
2.07	113.7	181.94	2.79	2.5	38	61	0.27	41	----	"	"
2.57	173.5	277.57	3.28	1.9	43	69	0.34	44	----	SAND to Silty SAND	"
3.08	129.5	207.20	3.43	2.6	43	69	0.40	42	----	Silty SAND to Sandy SILT	"
3.57	127.3	203.66	4.71	3.7	51	81	0.47	----	16.94	Sandy SILT to Clayey SILT	"
4.05	113.5	181.55	3.22	2.8	45	73	0.54	----	15.09	"	"
4.55	86.9	138.98	2.89	3.3	35	56	0.60	----	11.54	"	"
5.03	49.0	78.42	2.15	4.4	33	52	0.67	----	6.49	Silty CLAY to CLAY	"
5.55	34.9	55.90	1.75	5.0	35	56	0.74	----	4.61	CLAY	"
6.05	36.4	58.01	1.51	4.1	24	39	0.80	----	4.79	Silty CLAY to CLAY	"
6.55	32.2	49.14	1.22	3.8	16	25	0.87	----	4.23	Clayey SILT to Silty CLAY	"
7.04	32.0	46.79	1.48	4.6	21	31	0.94	----	4.21	Silty CLAY to CLAY	"
7.55	36.2	50.49	1.63	4.5	24	34	1.01	----	4.75	"	"
8.05	41.5	56.52	1.45	3.5	21	28	1.07	----	5.46	Clayey SILT to Silty CLAY	"
8.55	10.2	13.51	0.41	4.1	10	14	1.13	----	1.60	CLAY	110-120
9.01	35.8	46.50	0.83	2.3	14	19	1.19	----	4.69	Sandy SILT to Clayey SILT	130-140
9.52	40.1	50.72	1.35	3.4	20	25	1.26	----	5.27	Clayey SILT to Silty CLAY	"
10.03	36.5	44.93	1.69	4.6	24	30	1.33	----	4.78	Silty CLAY to CLAY	"
10.55	36.0	43.15	1.95	5.4	36	43	1.40	----	4.71	CLAY	"
11.05	41.7	48.53	2.15	5.2	42	49	1.47	----	5.46	"	"
11.55	53.1	60.36	2.17	4.1	27	30	1.54	----	6.98	Clayey SILT to Silty CLAY	"
12.06	65.3	72.67	2.56	3.9	33	36	1.61	----	8.60	"	"
12.58	65.1	70.70	2.60	4.0	33	35	1.68	----	8.56	"	"
13.09	76.5	81.20	2.66	3.5	31	32	1.74	----	10.08	Sandy SILT to Clayey SILT	"
13.58	81.1	84.78	3.37	4.2	41	42	1.81	----	10.69	Clayey SILT to Silty CLAY	"
14.06	87.9	90.48	3.62	4.1	44	45	1.88	----	11.59	"	"
14.55	65.8	66.74	3.69	5.6	66	67	1.94	----	8.65	Very Stiff Fine Grained *	"
15.03	72.1	72.51	3.10	4.3	36	36	1.98	----	9.48	Clayey SILT to Silty CLAY	"
15.51	71.1	71.11	2.64	3.7	36	36	2.01	----	9.35	"	"
16.08	88.5	88.40	3.40	3.8	44	44	2.05	----	11.66	"	"
16.56	57.6	57.52	2.64	4.6	38	38	2.09	----	7.54	Silty CLAY to CLAY	"
17.05	61.3	61.15	2.71	4.4	31	31	2.12	----	8.02	Clayey SILT to Silty CLAY	"
17.51	74.0	73.77	3.82	5.2	74	74	2.16	----	9.71	Very Stiff Fine Grained *	"
18.08	49.2	49.05	2.82	5.7	49	49	2.20	----	6.40	CLAY	"
18.56	36.0	35.86	2.25	6.2	36	36	2.23	----	4.64	"	"
19.04	35.7	35.48	2.37	6.7	36	35	2.27	----	4.59	"	"
19.53	35.8	35.62	2.13	5.9	36	36	2.30	----	4.60	"	"
20.02	37.1	36.83	2.57	6.9	37	37	2.34	----	4.77	"	"
20.58	35.7	35.47	2.37	6.6	36	35	2.38	----	4.58	"	"
21.06	30.1	29.84	2.11	7.0	30	30	2.41	----	3.82	"	"
21.54	29.4	29.12	2.12	7.2	29	29	2.45	----	3.72	"	"
22.03	28.0	27.74	2.04	7.3	28	28	2.48	----	3.54	"	"
22.53	25.5	25.02	1.76	6.9	25	25	2.52	----	3.19	"	"
23.01	25.5	24.78	1.81	7.1	25	25	2.56	----	3.20	"	"
23.57	21.5	20.57	1.50	7.0	21	21	2.60	----	2.65	"	"
24.04	19.4	18.32	1.48	7.6	19	18	2.63	----	2.37	"	"
24.50	17.2	16.05	1.11	6.4	17	16	2.66	----	2.07	"	"
25.05	15.3	14.10	0.89	5.8	15	14	2.70	----	1.81	"	120-130
25.52	15.2	13.89	0.95	6.2	15	14	2.73	----	1.80	"	"
26.06	17.7	15.92	0.95	5.4	18	16	2.77	----	2.12	"	130-140
26.53	15.5	13.78	0.90	5.8	15	14	2.80	----	1.82	"	120-130
27.07	16.3	14.32	0.83	5.1	16	14	2.83	----	1.93	"	"
27.54	16.6	14.47	1.01	6.1	17	14	2.86	----	1.97	"	130-140

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)
Terminated at 70.0 feet

CPT NO.: CPT-2
DATE : 09-14-2009
TIME : 12:58:07
Groundwater estimated at 14.6 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)
28.00	15.8	13.58	0.99	6.3	16	14	2.89	----	1.86	CLAY	120-130
28.54	15.9	13.47	0.90	5.7	16	13	2.93	----	1.86	"	"
29.03	14.6	12.23	0.84	5.7	15	12	2.96	----	1.69	"	"
29.57	15.7	13.00	0.86	5.5	16	13	2.99	----	1.83	"	"
30.04	14.4	11.84	0.80	5.6	14	12	3.02	----	1.65	"	"
30.51	16.3	13.37	1.03	6.3	16	13	3.05	----	1.91	"	130-140
31.04	16.5	13.44	0.90	5.5	16	13	3.09	----	1.93	"	120-130
31.51	17.7	14.31	0.96	5.4	18	14	3.12	----	2.07	"	130-140
32.03	12.6	10.15	0.77	6.1	13	10	3.15	----	1.39	"	120-130
32.56	16.9	13.60	0.87	5.2	17	14	3.19	----	1.97	"	"
33.03	14.8	11.83	0.89	6.0	15	12	3.22	----	1.68	"	"
33.55	16.0	12.74	0.90	5.6	16	13	3.25	----	1.84	"	"
34.07	14.6	11.53	0.81	5.6	15	11	3.28	----	1.64	"	"
34.52	13.4	10.56	0.69	5.1	13	11	3.31	----	1.48	"	"
35.05	15.6	12.28	0.91	5.8	16	12	3.34	----	1.78	"	"
35.57	15.8	12.34	0.94	6.0	16	12	3.38	----	1.79	"	"
36.02	16.5	12.83	0.99	6.0	16	13	3.40	----	1.88	"	"
36.53	17.5	13.56	1.15	6.6	17	14	3.44	----	2.01	"	130-140
37.04	17.1	13.17	0.97	5.7	17	13	3.48	----	1.96	"	"
37.56	17.4	13.30	1.05	6.0	17	13	3.52	----	1.99	"	"
38.04	16.5	12.58	0.91	5.5	16	13	3.55	----	1.87	"	120-130
38.56	15.4	11.71	0.72	4.6	15	12	3.58	----	1.72	"	"
39.06	17.5	13.21	1.10	6.3	17	13	3.62	----	1.99	"	130-140
39.57	15.6	11.69	0.84	5.4	16	12	3.65	----	1.73	"	120-130
40.00	15.4	11.53	0.90	5.8	15	11	3.67	----	1.71	"	"
40.57	12.5	9.32	0.59	4.7	12	9	3.71	----	1.32	"	"
41.05	14.8	10.93	0.80	5.4	15	11	3.74	----	1.61	"	"
41.55	17.5	12.85	0.92	5.3	17	13	3.77	----	1.96	"	"
42.06	16.5	12.09	0.94	5.7	16	12	3.80	----	1.83	"	"
42.56	16.9	12.29	0.96	5.7	17	12	3.83	----	1.88	"	"
43.05	14.3	10.37	0.66	4.6	14	10	3.87	----	1.53	"	"
43.53	15.8	11.37	0.66	4.2	16	11	3.90	----	1.72	"	"
44.03	14.2	10.23	0.51	3.6	9	7	3.93	----	1.51	Silty CLAY to CLAY	"
44.52	16.1	11.50	0.49	3.0	8	6	3.96	----	1.76	Clayey SILT to Silty CLAY	"
45.00	18.5	13.18	0.49	2.7	9	7	3.99	----	2.08	"	"
45.56	20.1	14.23	0.64	3.2	10	7	4.02	----	2.28	"	"
46.05	29.2	20.59	0.80	2.8	14	10	4.06	----	3.49	"	130-140
46.53	30.0	21.03	1.02	3.4	15	10	4.09	----	3.59	"	"
47.01	87.2	60.99	1.34	1.5	29	20	4.13	35	----	Silty SAND to Sandy SILT	"
47.53	39.1	27.24	1.43	3.6	19	14	4.17	----	4.80	Clayey SILT to Silty CLAY	"
48.03	50.0	34.66	1.09	2.2	20	14	4.20	----	6.25	Sandy SILT to Clayey SILT	"
48.56	80.7	55.64	2.68	3.3	32	22	4.24	----	10.33	"	"
49.05	156.8	107.73	1.79	1.1	31	22	4.27	38	----	SAND	120-130
49.51	158.9	108.70	3.01	1.9	40	27	4.30	38	----	SAND to Silty SAND	130-140
50.02	183.3	124.85	3.48	1.9	46	31	4.34	39	----	"	"
50.51	356.6	241.89	3.06	0.9	71	48	4.37	43	----	SAND	120-130
51.03	426.2	288.33	1.75	0.4	71	48	4.39	44	----	Gravelly SAND to SAND	100-110
51.51	479.5	323.36	3.05	0.6	80	54	4.42	45	----	"	110-120
52.02	436.1	293.27	2.21	0.5	73	49	4.44	44	----	"	100-110
52.54	345.6	231.64	2.30	0.7	58	39	4.47	43	----	"	110-120
53.04	347.0	231.94	1.73	0.5	58	39	4.49	43	----	"	100-110
53.53	426.3	284.01	2.64	0.6	71	47	4.52	44	----	"	110-120
54.01	501.2	333.04	2.42	0.5	84	56	4.54	45	----	"	100-110
54.50	497.0	328.92	4.32	0.9	83	55	4.57	45	----	"	120-130
55.03	442.4	291.72	2.45	0.6	74	49	4.60	44	----	Gravelly SAND to SAND	110-120

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)
Terminated at 70.0 feet

CPT NO.: CPT-2
DATE : 09-14-2009
TIME : 12:58:07
Groundwater estimated at 14.6 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)
55.55	349.8	229.71	2.83	0.8	70	46	4.63	43	----	SAND	120-130
56.02	363.1	237.52	2.88	0.8	61	40	4.66	43	----	Gravelly SAND to SAND	"
56.50	493.4	321.90	2.36	0.5	82	54	4.68	45	----	"	100-110
57.00	428.8	278.62	3.21	0.7	71	46	4.71	44	----	"	120-130
57.54	336.1	217.41	2.78	0.8	67	43	4.74	42	----	SAND	"
58.03	173.6	111.92	1.27	0.7	35	22	4.77	39	----	"	110-120
58.52	41.6	26.71	2.40	5.8	42	27	4.80	----	5.05	CLAY	130-140
59.05	446.0	285.29	2.12	0.5	74	48	4.83	44	----	Gravelly SAND to SAND	100-110
59.54	313.2	199.53	2.43	0.8	63	40	4.86	42	----	SAND	120-130
60.04	275.5	174.89	1.96	0.7	55	35	4.88	41	----	"	110-120
60.55	279.3	176.82	1.24	0.4	47	29	4.91	41	----	Gravelly SAND to SAND	100-110
61.01	246.4	155.69	0.67	0.3	41	26	4.92	41	----	"	90-100
61.52	245.6	154.79	0.75	0.3	41	26	4.94	41	----	"	"
62.00	223.8	140.76	0.63	0.3	37	23	4.95	40	----	"	"
62.55	224.9	141.05	1.02	0.5	45	28	4.98	40	----	SAND	100-110
63.02	229.7	143.66	1.00	0.4	46	29	5.00	40	----	"	"
63.50	209.2	130.61	0.45	0.2	35	22	5.01	40	----	Gravelly SAND to SAND	85-90
64.04	207.6	129.38	0.46	0.2	42	26	5.03	39	----	SAND	90-100
64.51	274.7	170.86	0.97	0.4	46	28	5.05	41	----	Gravelly SAND to SAND	100-110
65.07	262.5	162.95	0.80	0.3	44	27	5.06	41	----	"	90-100
65.56	228.5	141.63	0.68	0.3	38	24	5.08	40	----	"	"
66.03	287.7	177.75	2.50	0.9	58	36	5.11	41	----	SAND	120-130
66.54	335.0	206.39	2.14	0.6	56	34	5.14	42	----	Gravelly SAND to SAND	110-120
67.06	267.3	164.14	2.86	1.1	53	33	5.17	41	----	SAND	120-130
67.54	434.9	266.33	2.82	0.6	72	44	5.19	44	----	Gravelly SAND to SAND	110-120
68.04	363.5	222.10	1.42	0.4	61	37	5.22	43	----	"	100-110
68.51	79.3	48.28	2.05	2.6	32	19	5.25	----	10.00	Sandy SILT to Clayey SILT	130-140
69.00	27.3	16.54	1.07	3.9	18	11	5.28	----	3.06	Silty CLAY to CLAY	"
69.55	24.2	14.59	0.95	3.9	16	10	5.33	----	2.64	"	"
70.04	23.1	13.90	0.78	3.4	11	7	5.36	----	2.49	Clayey SILT to Silty CLAY	"

DEPTH = Sampling interval (.1 feet)
Qc = Tip bearing resistance TotStr = Total Stress using est. density**
Fs = Sleeve friction resistance Phi = Soil friction angle*
Rf = Tip/Sleeve ratio Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf)
SPT = Equivalent Standard Penetration Test* (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)
References: * Robertson and Campanella, 1988
** Olsen, 1989 *** Durgunoglu & Mitchell, 1975

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)
Terminated at 45.0 feet

CPT NO.: CPT-3
DATE : 09-14-2009
TIME : 11:38:07
Groundwater estimated at 11.9 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	162.5	260.00	3.12	1.9	54	87	0.06	44	----	Silty SAND to Sandy SILT	130-140
1.01	122.6	196.18	3.21	2.6	41	65	0.13	42	----	"	"
1.55	85.1	136.14	3.19	3.7	43	68	0.20	----	11.33	Clayey SILT to Silty CLAY	"
2.01	89.7	143.57	3.03	3.4	36	57	0.26	----	11.95	Sandy SILT to Clayey SILT	"
2.55	140.8	225.25	4.95	3.5	56	90	0.33	----	18.75	"	"
3.04	143.4	229.44	5.83	4.1	143	230	0.40	----	19.09	Very Stiff Fine Grained *	>140
3.52	203.0	324.80	7.01	3.5	102	162	0.47	45	----	SAND to Clayey SAND *	"
4.06	162.1	259.42	4.36	2.7	54	86	0.54	43	----	Silty SAND to Sandy SILT	130-140
4.53	129.5	207.18	4.36	3.4	52	83	0.61	----	17.22	Sandy SILT to Clayey SILT	"
5.03	184.6	295.34	4.47	2.4	62	98	0.67	44	----	Silty SAND to Sandy SILT	"
5.50	194.6	311.41	5.33	2.7	65	104	0.74	45	----	"	"
6.02	107.6	171.33	3.10	2.9	43	69	0.81	----	14.29	Sandy SILT to Clayey SILT	"
6.51	91.9	140.32	2.93	3.2	37	56	0.87	----	12.20	"	"
7.01	78.7	114.86	3.35	4.3	39	57	0.94	----	10.43	Clayey SILT to Silty CLAY	"
7.51	67.9	94.73	3.07	4.5	34	47	1.01	----	8.98	"	"
8.02	90.3	122.75	2.19	2.4	30	41	1.08	39	----	Silty SAND to Sandy SILT	"
8.57	95.8	126.59	2.15	2.2	32	42	1.15	39	----	"	"
9.02	129.9	167.53	2.10	1.6	32	42	1.21	41	----	SAND to Silty SAND	"
9.57	38.5	48.23	1.95	5.0	39	48	1.29	----	5.05	CLAY	"
10.03	278.0	339.93	4.46	1.6	56	68	1.35	45	----	SAND	"
10.57	188.2	223.44	5.86	3.1	63	74	1.42	43	----	Silty SAND to Sandy SILT	"
11.03	304.9	353.61	2.94	1.0	61	71	1.48	45	----	SAND	120-130
11.53	207.7	235.73	2.28	1.1	42	47	1.54	43	----	"	"
12.01	245.1	272.39	3.11	1.3	49	54	1.61	44	----	"	130-140
12.55	323.6	351.23	4.48	1.4	65	70	1.68	45	----	"	"
13.01	283.5	301.32	4.23	1.5	57	60	1.74	44	----	"	"
13.51	229.1	240.15	1.52	0.7	46	48	1.80	43	----	"	110-120
14.02	162.4	168.15	0.76	0.5	32	34	1.85	41	----	"	100-110
14.52	242.4	247.86	1.59	0.7	48	50	1.91	43	----	"	110-120
15.06	191.3	192.99	0.74	0.4	38	39	1.96	42	----	"	100-110
15.54	61.2	61.15	1.56	2.5	25	24	2.03	----	8.02	Sandy SILT to Clayey SILT	130-140
16.03	22.8	22.79	0.68	3.0	11	11	2.09	----	2.90	Clayey SILT to Silty CLAY	120-130
16.52	25.4	25.28	0.79	3.1	13	13	2.15	----	3.24	"	130-140
17.01	14.2	14.16	0.34	2.4	7	7	2.21	----	1.75	"	110-120
17.51	9.4	9.32	0.38	4.1	9	9	2.27	----	1.37	CLAY	"
18.07	14.7	14.55	0.75	5.1	15	15	2.34	----	1.80	"	120-130
18.56	16.9	16.79	0.77	4.5	17	17	2.40	----	2.10	"	"
19.05	16.2	16.04	0.87	5.3	16	16	2.46	----	1.99	"	"
19.54	20.8	20.36	0.85	4.1	14	14	2.53	----	2.60	Silty CLAY to CLAY	130-140
20.03	20.9	20.02	0.90	4.3	14	13	2.59	----	2.61	"	"
20.52	18.1	17.00	0.77	4.3	18	17	2.65	----	2.24	CLAY	120-130
21.07	13.9	12.69	0.63	4.5	14	13	2.72	----	1.67	"	"
21.57	13.1	11.73	0.40	3.1	9	8	2.78	----	1.56	Silty CLAY to CLAY	"
22.06	14.3	12.67	0.44	3.1	10	8	2.82	----	1.72	"	"
22.56	15.8	13.81	0.60	3.8	10	9	2.85	----	1.91	"	"
23.07	11.8	10.18	0.51	4.3	12	10	2.88	----	1.71	CLAY	"
23.58	8.2	7.01	0.33	4.0	8	7	2.91	----	1.33	"	110-120
24.04	9.1	7.74	0.26	2.8	6	5	2.93	----	1.26	Silty CLAY to CLAY	100-110
24.55	9.2	7.77	0.25	2.7	6	5	2.95	----	1.28	"	"
25.06	10.1	8.44	0.41	4.0	10	8	2.97	----	1.42	CLAY	110-120
25.57	12.8	10.58	0.43	3.3	9	7	3.01	----	1.49	Silty CLAY to CLAY	120-130
26.07	7.5	6.12	0.34	4.5	7	6	3.03	----	1.16	CLAY	110-120
26.57	7.9	6.43	0.26	3.3	8	6	3.05	----	1.23	"	100-110
27.08	9.1	7.44	0.29	3.2	6	5	3.08	----	1.23	Silty CLAY to CLAY	110-120
27.57	9.5	7.68	0.37	4.0	9	8	3.11	----	1.28	CLAY	"

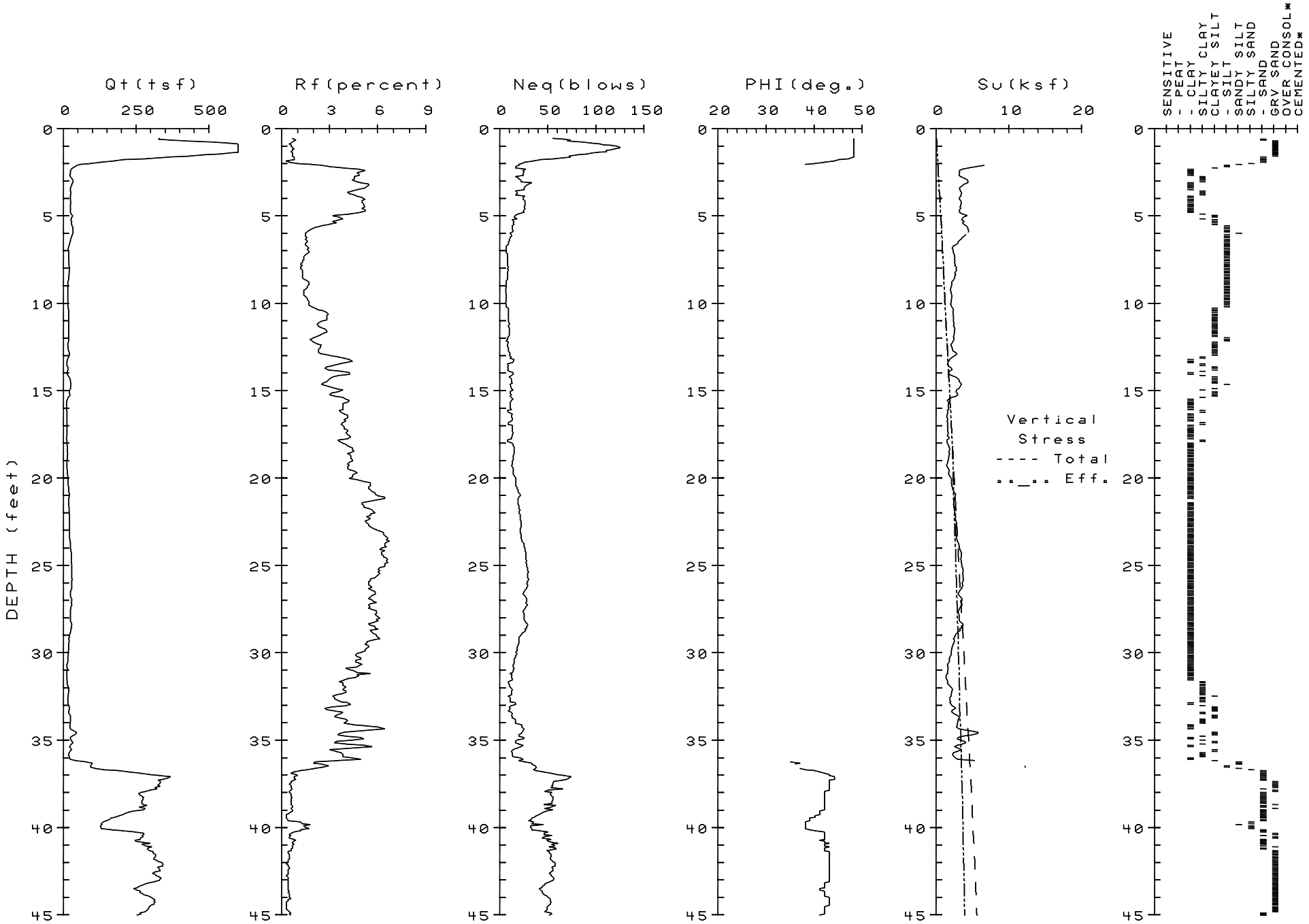
PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)
Terminated at 45.0 feet

CPT NO.: CPT-3
DATE : 09-14-2009
TIME : 11:38:07
Groundwater estimated at 11.9 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)
28.08	8.2	6.63	0.43	5.2	8	7	3.13	----	1.28	CLAY	110-120
28.50	17.4	14.00	0.53	3.1	9	7	3.16	----	2.08	Clayey SILT to Silty CLAY	120-130
29.08	43.0	34.51	1.18	2.7	17	14	3.20	----	5.49	Sandy SILT to Clayey SILT	130-140
29.50	11.8	9.43	0.45	3.8	12	9	3.23	----	1.66	CLAY	120-130
30.00	12.7	10.08	0.42	3.3	8	7	3.26	----	1.44	Silty CLAY to CLAY	"
30.53	10.9	8.61	0.38	3.5	7	6	3.29	----	1.49	"	110-120
31.04	9.7	7.67	0.32	3.3	6	5	3.31	----	1.29	"	"
31.54	10.3	8.10	0.35	3.4	7	5	3.34	----	1.39	"	"
32.05	11.9	9.30	0.51	4.3	12	9	3.37	----	1.65	CLAY	120-130
32.55	23.2	18.07	1.15	4.9	23	18	3.41	----	2.82	"	130-140
33.04	119.3	92.32	1.72	1.4	30	23	3.44	38	----	SAND to Silty SAND	"
33.53	125.0	96.31	1.33	1.1	31	24	3.47	38	----	"	120-130
34.04	96.0	73.70	0.62	0.6	24	18	3.50	36	----	"	110-120
34.54	17.3	13.22	0.48	2.7	9	7	3.53	----	2.02	Clayey SILT to Silty CLAY	120-130
35.04	11.2	8.52	0.30	2.7	7	6	3.56	----	1.50	Silty CLAY to CLAY	110-120
35.53	14.8	11.21	0.67	4.5	15	11	3.59	----	1.68	CLAY	120-130
36.04	13.0	9.77	0.47	3.6	9	6	3.62	----	1.43	Silty CLAY to CLAY	"
36.53	13.2	9.91	0.56	4.2	13	10	3.65	----	1.46	CLAY	"
37.00	13.8	10.32	0.50	3.6	9	7	3.68	----	1.53	Silty CLAY to CLAY	"
37.51	13.9	10.31	0.41	3.0	7	5	3.71	----	1.54	Clayey SILT to Silty CLAY	"
38.02	15.6	11.54	0.42	2.7	8	6	3.74	----	1.76	"	"
38.50	13.8	10.13	0.43	3.1	9	7	3.78	----	1.51	Silty CLAY to CLAY	"
39.01	10.8	7.91	0.25	2.3	5	4	3.80	----	1.39	Clayey SILT to Silty CLAY	100-110
39.51	16.2	11.85	0.64	3.9	11	8	3.83	----	1.84	Silty CLAY to CLAY	120-130
40.04	16.4	11.93	0.49	3.0	8	6	3.86	----	1.86	Clayey SILT to Silty CLAY	"
40.54	17.3	12.50	0.56	3.2	8	6	3.89	----	1.97	"	"
41.05	13.3	9.56	0.55	4.2	13	9	3.92	----	1.43	CLAY	"
41.54	10.7	7.62	0.43	4.1	10	7	3.95	----	1.34	"	110-120
42.04	12.2	8.71	0.40	3.3	8	6	3.98	----	1.28	Silty CLAY to CLAY	"
42.52	13.6	9.62	0.44	3.2	9	6	4.01	----	1.45	"	120-130
43.01	11.4	8.03	0.31	2.8	7	5	4.03	----	1.44	"	110-120
43.55	10.9	7.72	0.23	2.1	5	4	4.06	----	1.37	Clayey SILT to Silty CLAY	100-110
44.04	15.0	10.56	0.42	2.8	7	5	4.09	----	1.64	"	120-130
44.54	14.1	9.87	0.48	3.4	9	6	4.12	----	1.51	Silty CLAY to CLAY	"
45.03	14.0	9.79	0.40	2.9	7	5	4.14	----	1.50	Clayey SILT to Silty CLAY	110-120

DEPTH = Sampling interval (.1 feet)
Qc = Tip bearing resistance TotStr = Total Stress using est. density**
Fs = Sleeve friction resistance Phi = Soil friction angle*
Rf = Tip/Sleeve ratio Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf)
SPT = Equivalent Standard Penetration Test* (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)
References: * Robertson and Campanella, 1988
** Olsen, 1989 *** Durgunoglu & Mitchell, 1975



Terminated at 45.0 feet

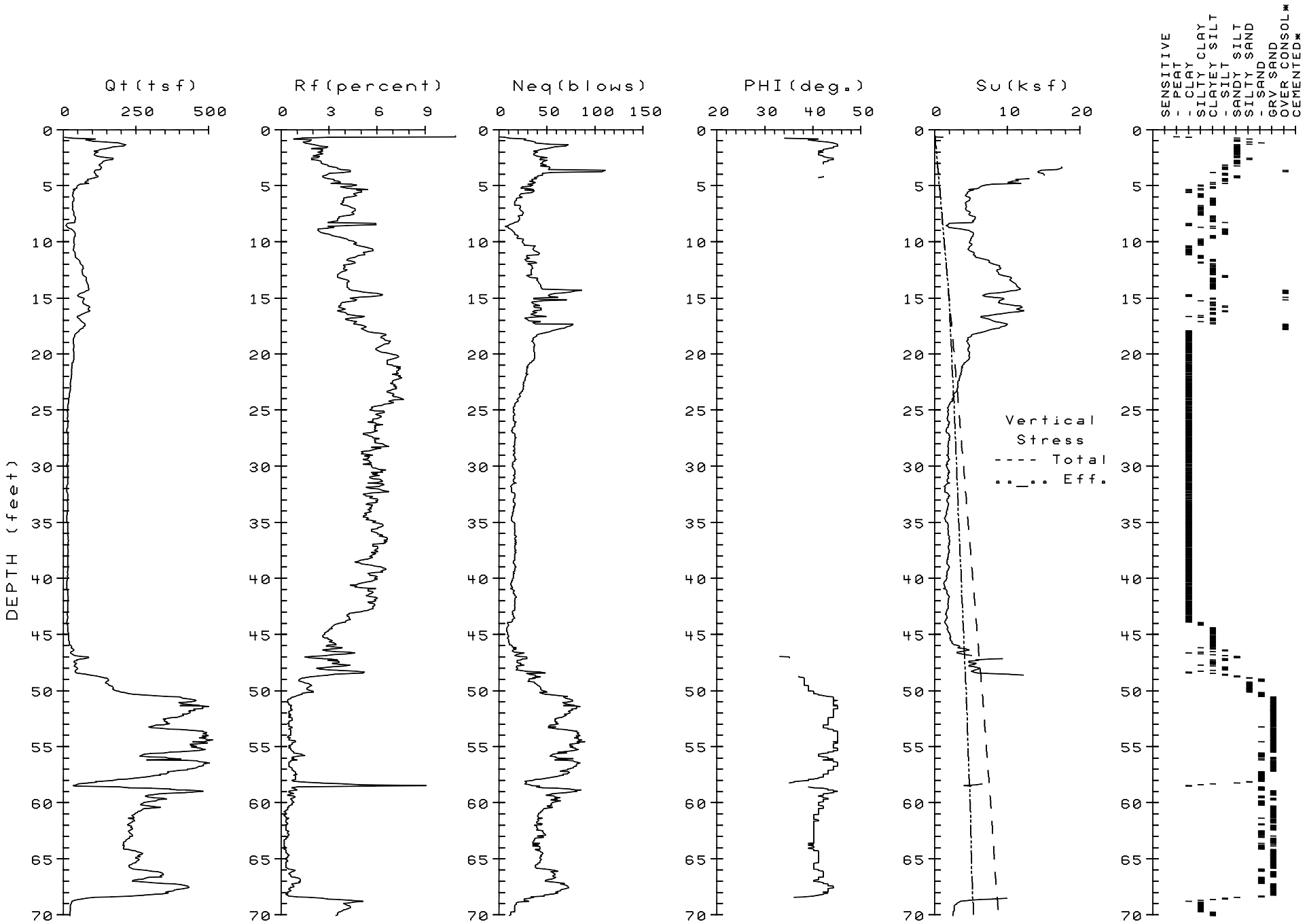
Groundwater estimated at 18.0 feet

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)

CPT NO.: CPT-1
DATE : 09-14-2009

PACIFIC GEOTECHNICAL

cpts by John Sarmiento & Associates



Terminated at 70.0 feet

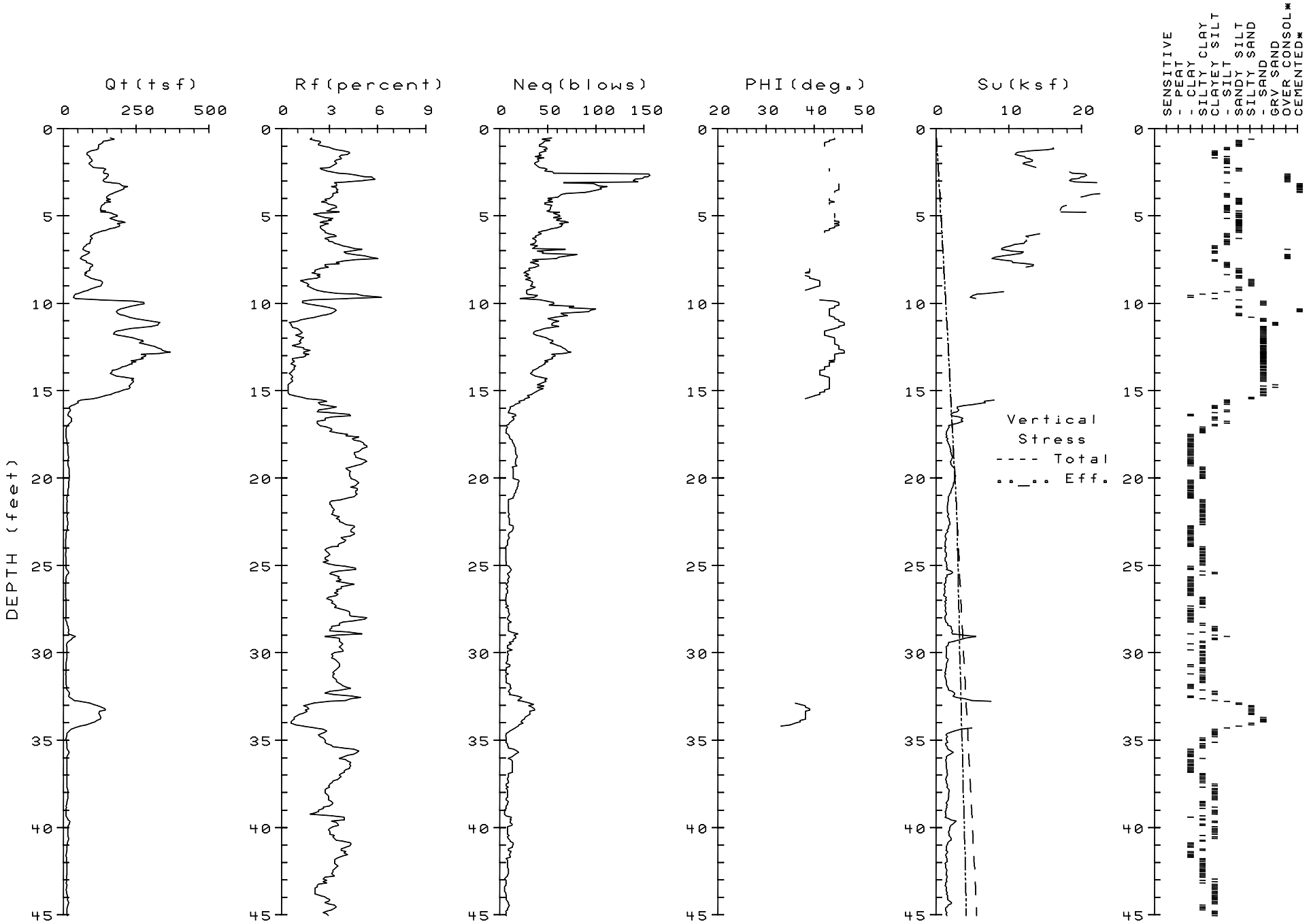
Groundwater measured at 14.6 feet

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)

CPT NO.: CPT-2
DATE : 09-14-2009

PACIFIC GEOTECHNICAL

cpts by John Sarmiento & Associates



Terminated at 45.0 feet

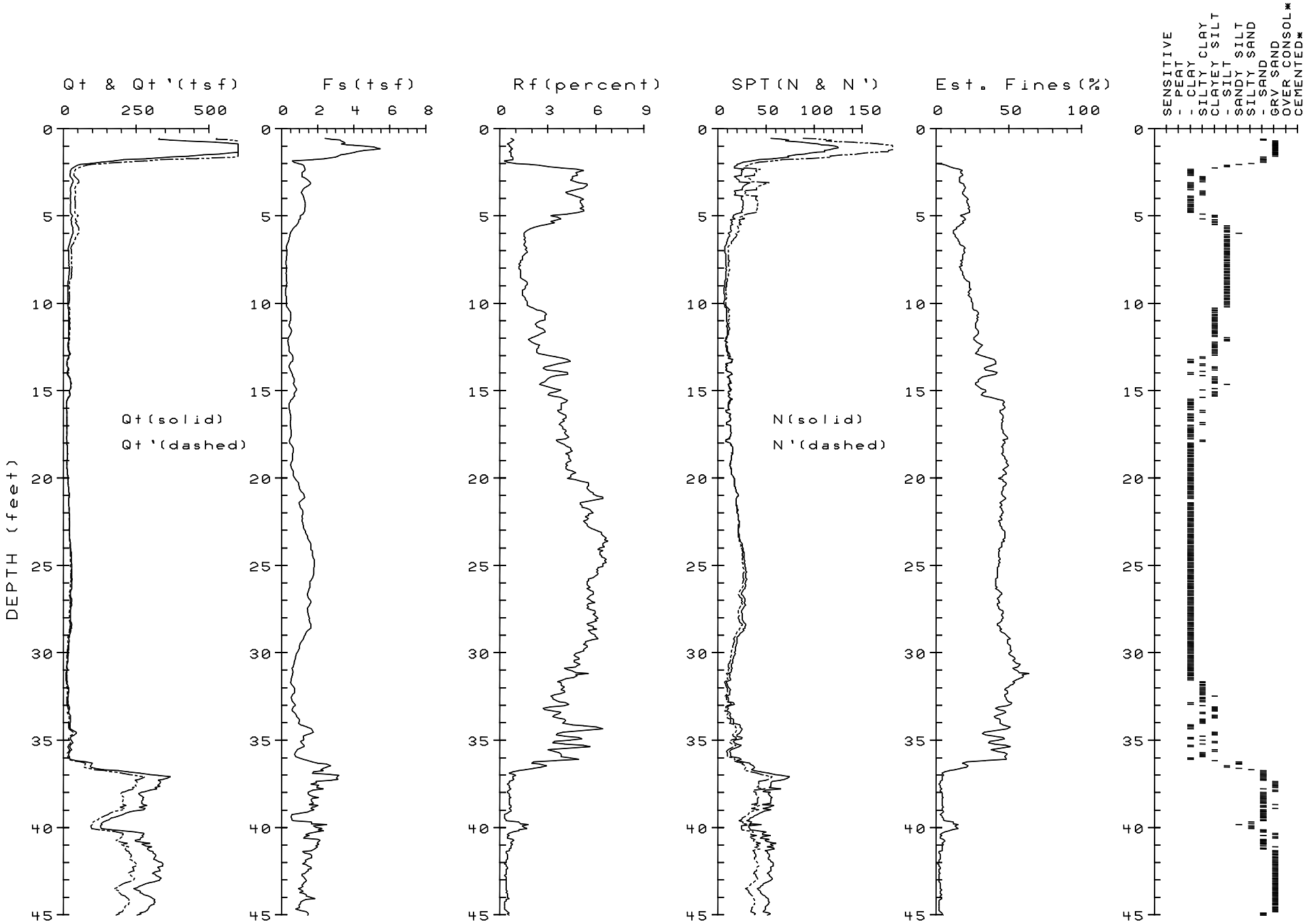
Groundwater measured at 21.9 feet

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)

CPT NO.: CPT-3
DATE : 09-14-2009

PACIFIC GEOTECHNICAL

cpts by John Sarmiento & Associates



Terminated at 45.0 feet

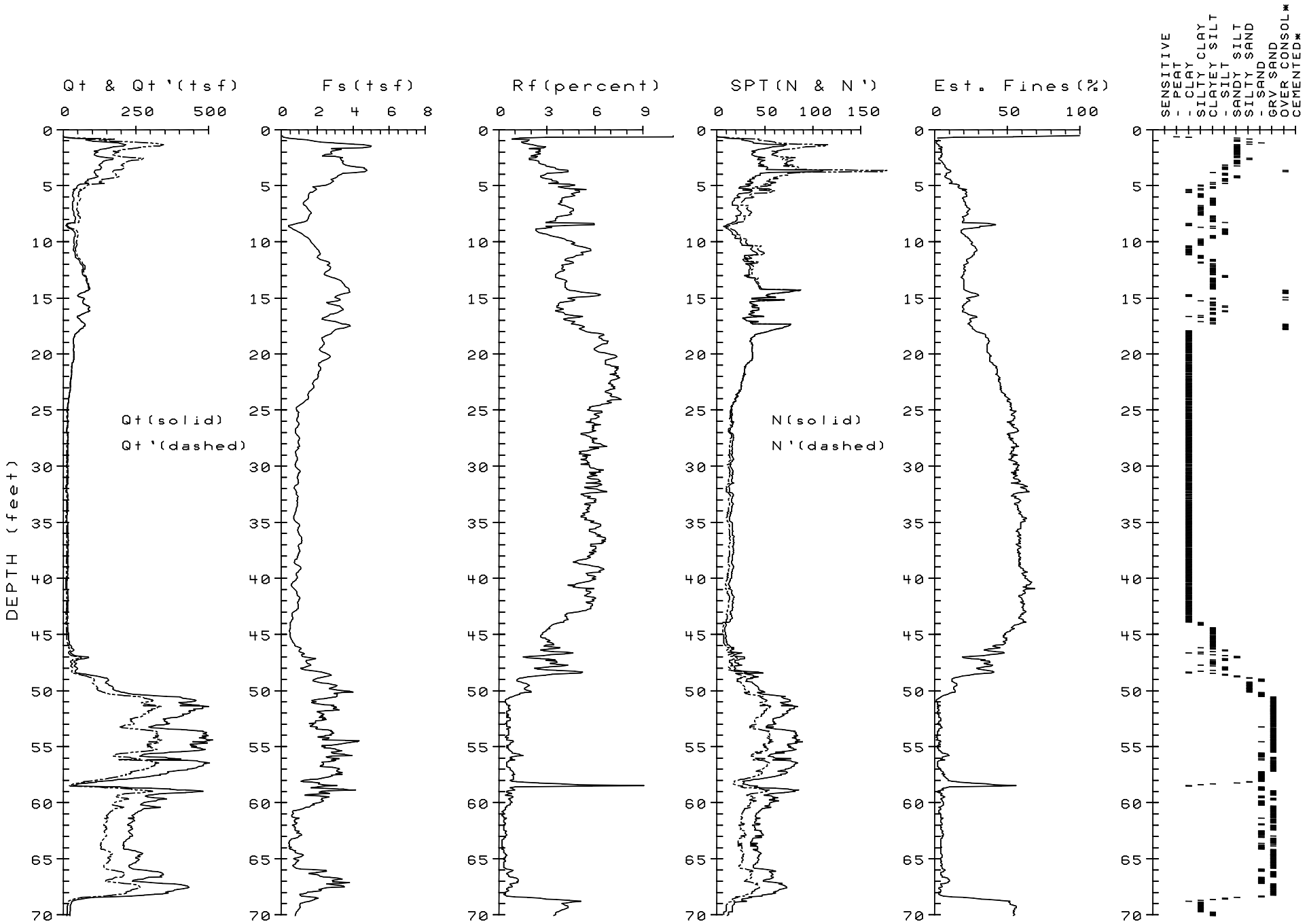
Groundwater estimated at 18.0 feet

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)

CPT NO.: CPT-1
DATE : 09-14-2009

PACIFIC GEOTECHNICAL

cpts by John Sarmiento & Associates



Terminated at 70.0 feet

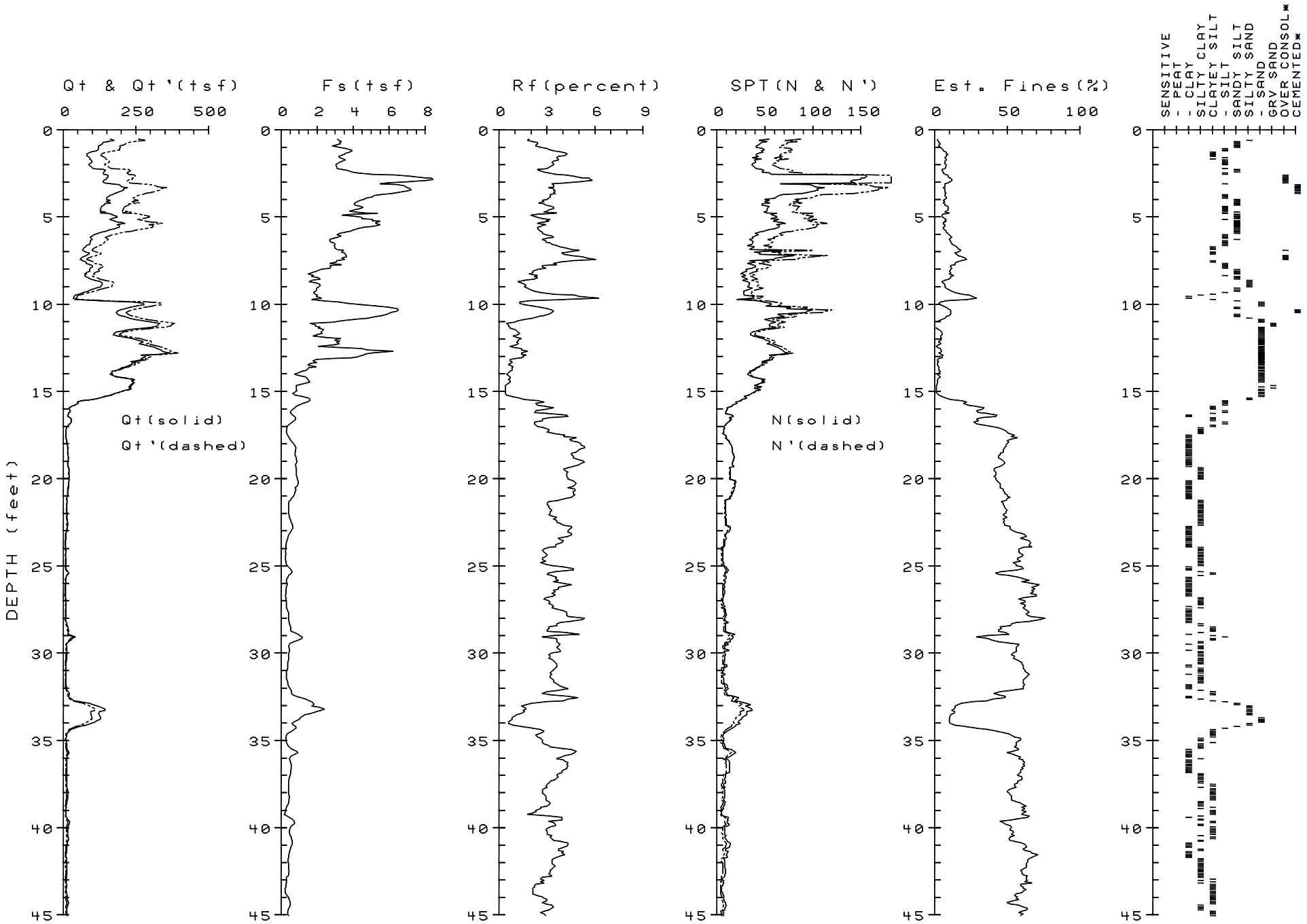
Groundwater measured at 14.6 feet

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)

CPT NO.: CPT-2
DATE : 09-14-2009

PACIFIC GEOTECHNICAL

cpts by John Sarmiento & Associates



Terminated at 45.0 feet

Groundwater measured at 21.9 feet

PROJECT: COYOTE CREEK TRAIL
LOCATION: San Jose CA
PROJ. NO.: 2298E(PGE-07)

CPT NO.: CPT-3
DATE : 09-14-2009

PACIFIC GEOTECHNICAL

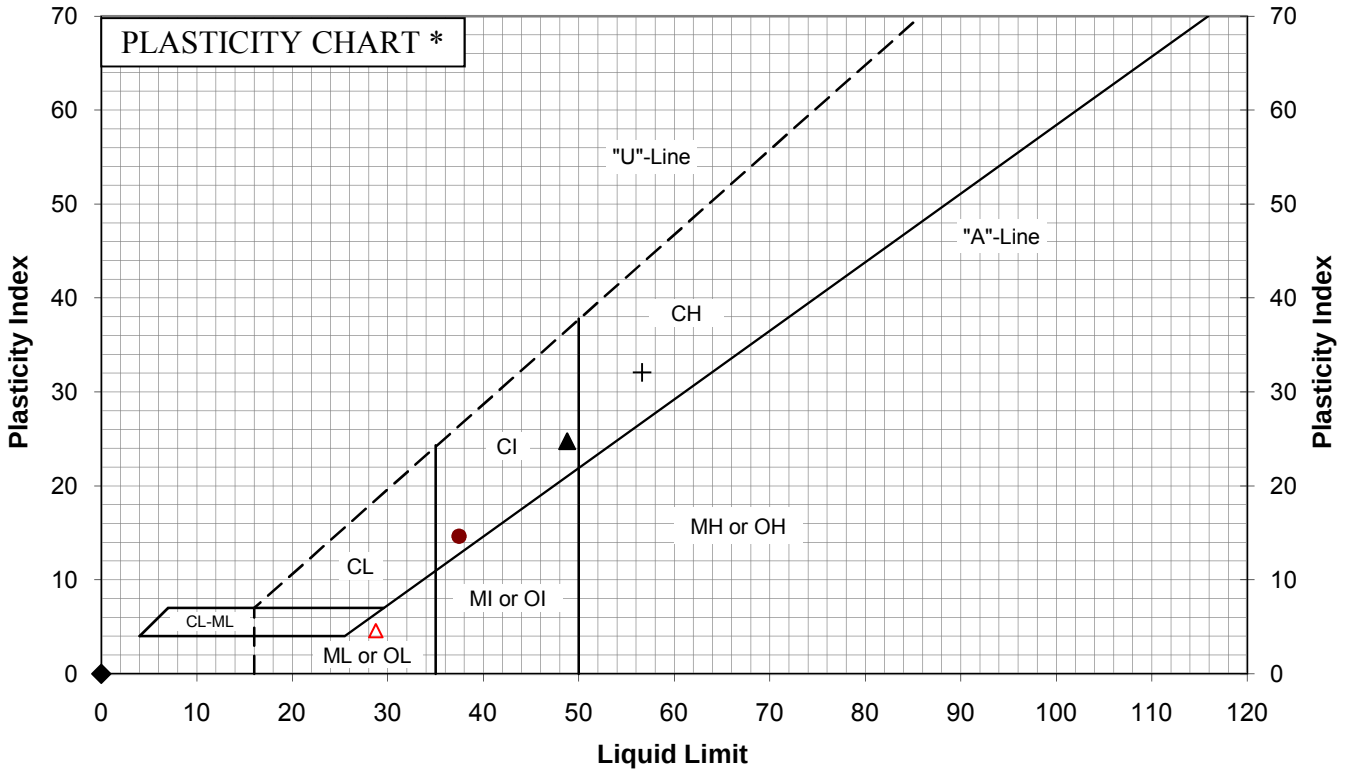
cpts by John Sarmiento & Associates

APPENDIX C

LABORATORY TEST RESULTS

ATTERBERG LIMITS TEST RESULTS

PROJECT NAME	Coyote Creek Trail			PROJECT No.	2298 E
DATE OF TEST	10/22/2009	10/22/2009	10/22/2009	10/22/2009	10/22/2009
KEY SYMBOL	♦	▲	●	+	Δ
DRILL HOLE No.	1	3	6	8	10
DEPTH (ft)	6	14.5	9.5	19.5	14-15
NATURAL WATER CONTENT (%)	7	26	17	26	8
% Retained No. 40 SIEVE (Est.)	0	0	0	0	0
% PASSING No. 200 SIEVE	55	N/A	N/A	N/A	N/A
LIQUID LIMIT	NP	49	37	57	29
PLASTIC LIMIT	NP	24	23	25	24
PLASTICITY INDEX	NP	25	15	32	5
CLASSIFICATION SYMBOL	Non Plastic	CI	CI	CH	ML



* Based on the Unified Soil Classification System modified to incorporate the "intermediate" classifications CI, MI, and OI for soils with liquid limits between 35 and 50. In the unmodified Unified Soil Classification System, such soils would be classified as CL, ML and OL, respectively.

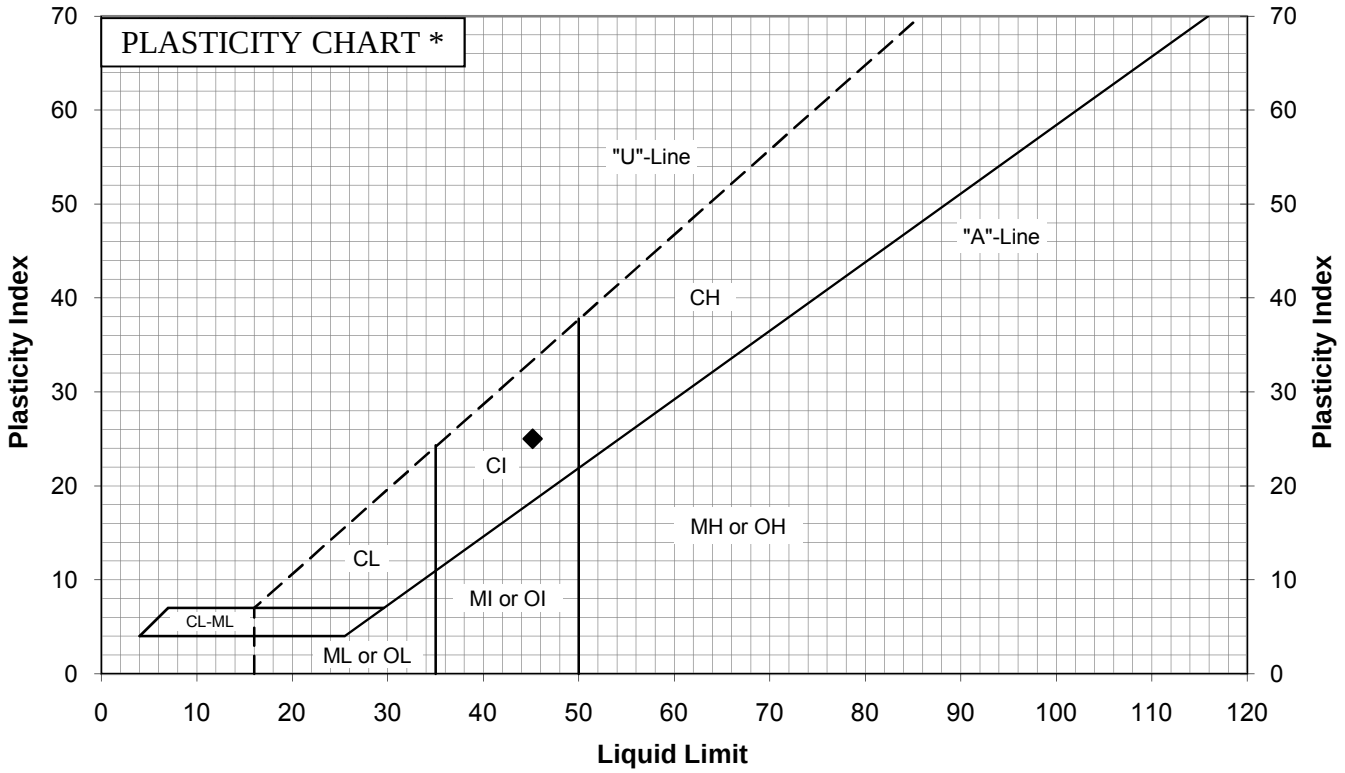
PACIFIC GEOTECHNICAL ENGINEERING

Figure C-1

Rev030106

ATTERBERG LIMITS TEST RESULTS

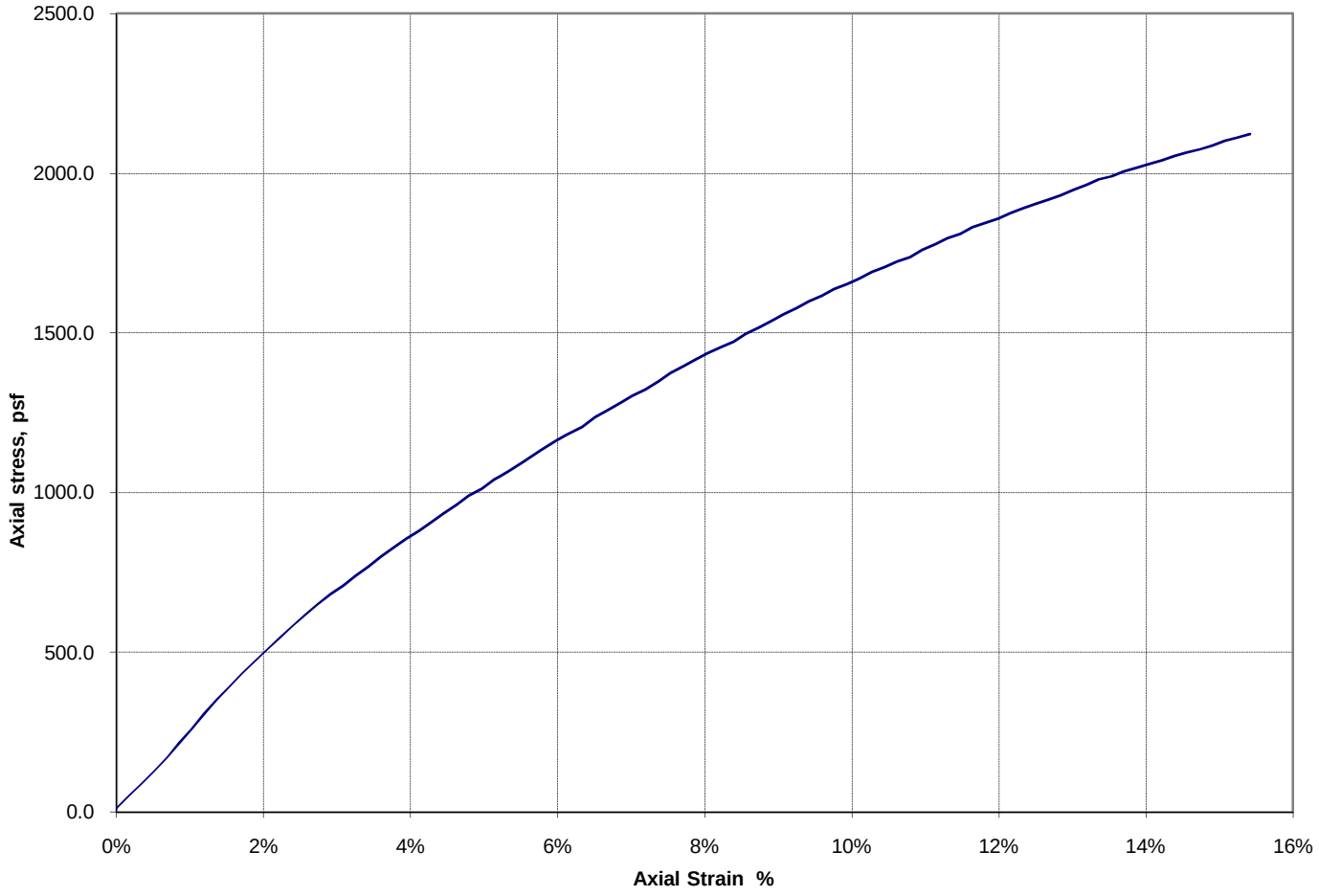
PROJECT NAME	Coyote Creek Trail			PROJECT No.	2298 E
DATE OF TEST	11/5/2009				
KEY SYMBOL	♦				
DRILL HOLE No.	10				
DEPTH (ft)	44.5				
NATURAL WATER CONTENT (%)	26				
% Retained No. 40 SIEVE (Est.)	0				
% PASSING No. 200 SIEVE	N/A				
LIQUID LIMIT	45				
PLASTIC LIMIT	20				
PLASTICITY INDEX	25				
CLASSIFICATION SYMBOL	CI				



* Based on the Unified Soil Classification System modified to incorporate the "intermediate" classifications CI, MI, and OI for soils with liquid limits between 35 and 50. In the unmodified Unified Soil Classification System, such soils would be classified as CL, ML and OL, respectively.

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	1	**DEPTH (ft)**	39.5	**SAMPLE**	**DATE OF TEST** 10/12/2009
DESCRIPTION OF SOIL	**CLAY:** brown (10YR 4/3), moist				
LENGTH OF SAMPLE:	5.84 **in.**			**WATER CONTENT:**	29%
DIAMETER OF SAMPLE:	2.39 **in.**			**DRY DENSITY:**	97 **pcf**
HEIGHT TO DIA. RATIO:	2.4			**MAX COMPRESSIVE STRENGTH:**	2096 **psf**
STRAIN RATE:	1.24% **per minute**			**STRAIN AT MAX STRENGTH:**	15.0%



REMARKS: Maximum compressive strength taken at 15% strain

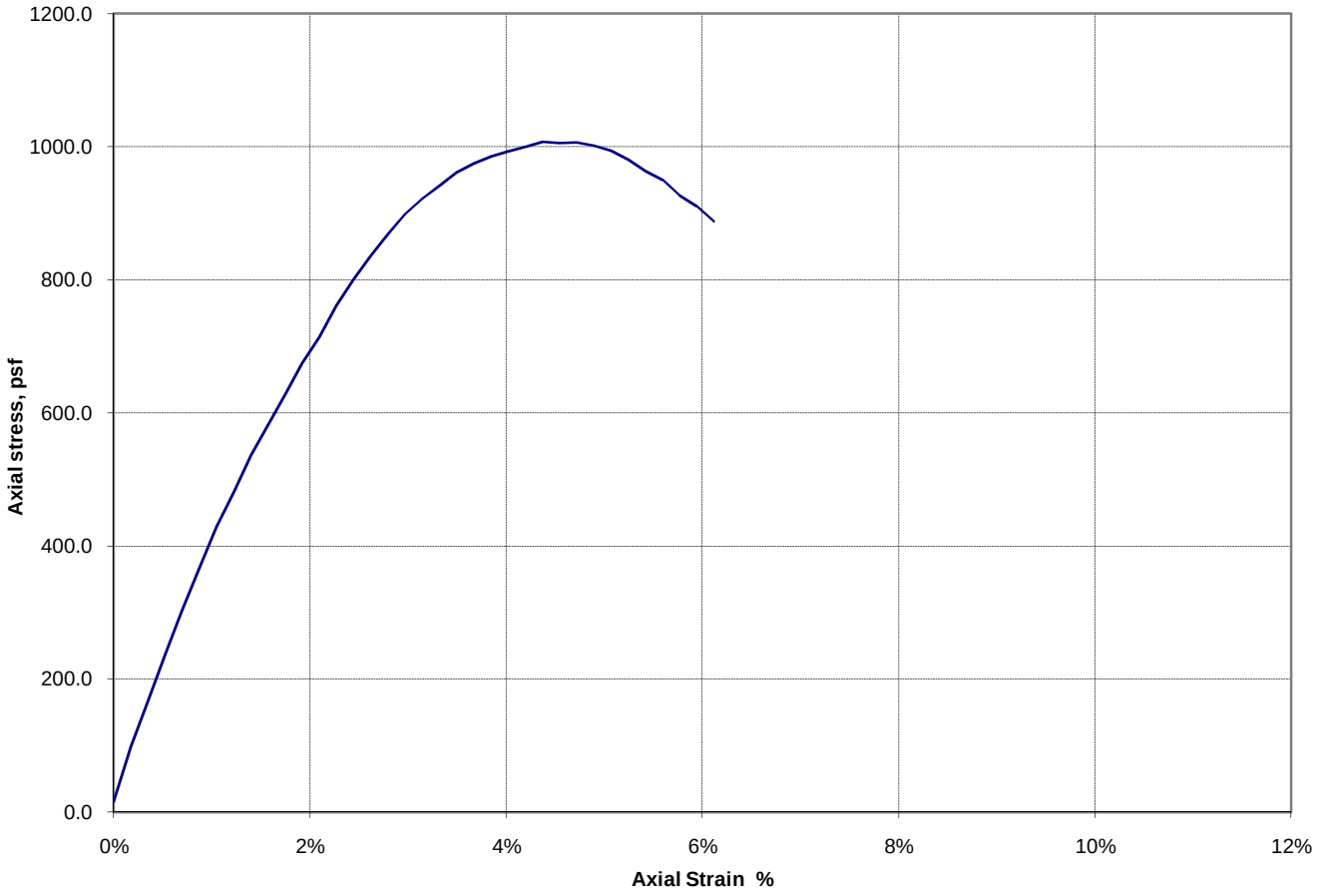
Rev 032806

PACIFIC GEOTECHINCAL ENGINEERING

Figure C-3

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	8	**DEPTH (ft)**	24.0	**SAMPLE**	**DATE OF TEST** 10/19/2009
DESCRIPTION OF SOIL	**LEAN CLAY to CLAY:** dark grayish brown (2.5Y 4/2), moist to wet				
LENGTH OF SAMPLE:	5.72 **in.**			**WATER CONTENT:**	27%
DIAMETER OF SAMPLE:	2.40 **in.**			**DRY DENSITY:**	99 **pcf**
HEIGHT TO DIA. RATIO:	2.4			**MAX COMPRESSIVE STRENGTH:**	1008 **psf**
STRAIN RATE:	1.34% **per minute**			**STRAIN AT MAX STRENGTH:**	4.4%



REMARKS:

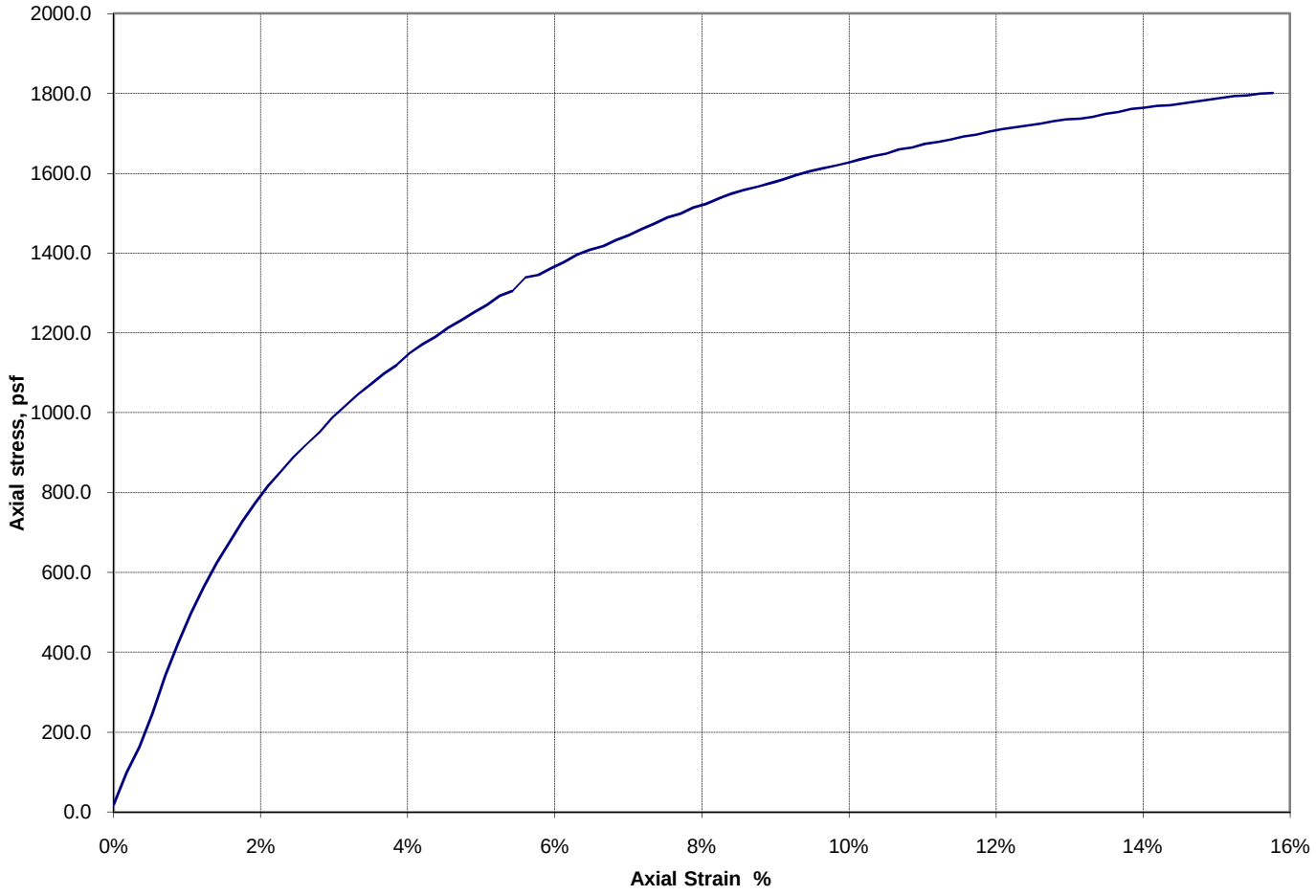
PACIFIC GEOTECHINCAL ENGINEERING

Rev 032806

Figure C-4

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	8	**DEPTH (ft)**	39.5	**SAMPLE**	**DATE OF TEST** 10/2/2009
DESCRIPTION OF SOIL	**CLAY to FAT CLAY:** Black (2.5Y 2.5/1), moist				
LENGTH OF SAMPLE:	5.71 **in.**			**WATER CONTENT:**	24%
DIAMETER OF SAMPLE:	2.39 **in.**			**DRY DENSITY:**	104 **pcf**
HEIGHT TO DIA. RATIO:	2.4			**MAX COMPRESSIVE STRENGTH:**	1787 **psf**
STRAIN RATE:	1.30% **per minute**			**STRAIN AT MAX STRENGTH:**	15.0%



REMARKS: Maximum compressive strength taken at 15% strain

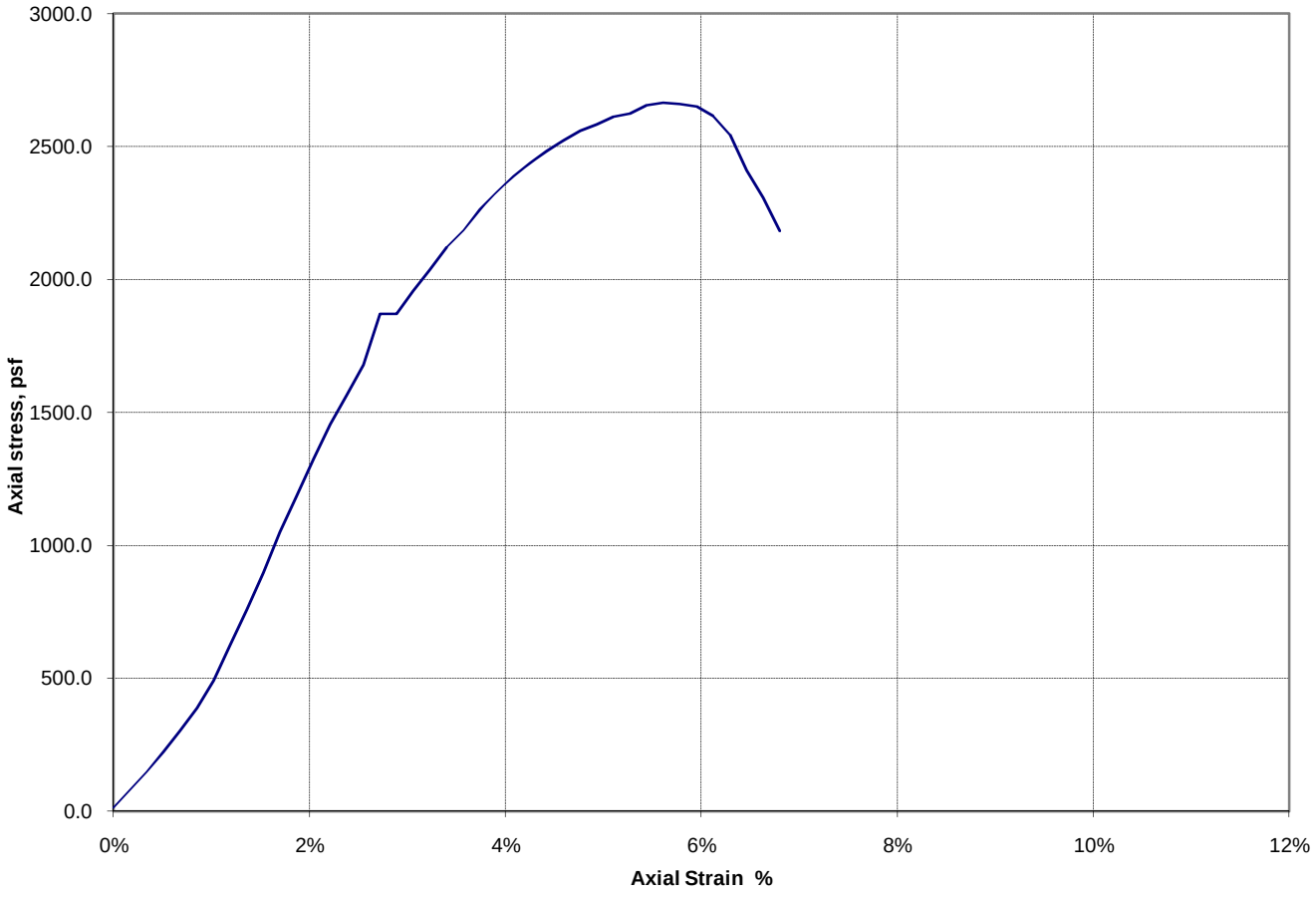
Rev 032806

PACIFIC GEOTECHINCAL ENGINEERING

Figure C-5

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	9	**DEPTH (ft)**	19.5	**SAMPLE**	**DATE OF TEST** 10/19/2009
DESCRIPTION OF SOIL	**CLAY to FAT CLAY:** Very dark gray (2.5Y 3/1), moist				
LENGTH OF SAMPLE:	5.88 **in.**			**WATER CONTENT:**	34%
DIAMETER OF SAMPLE:	2.39 **in.**			**DRY DENSITY:**	88 **pcf**
HEIGHT TO DIA. RATIO:	2.5			**MAX COMPRESSIVE STRENGTH:**	2666 **psf**
STRAIN RATE:	1.30% **per minute**			**STRAIN AT MAX STRENGTH:**	5.6%



REMARKS:

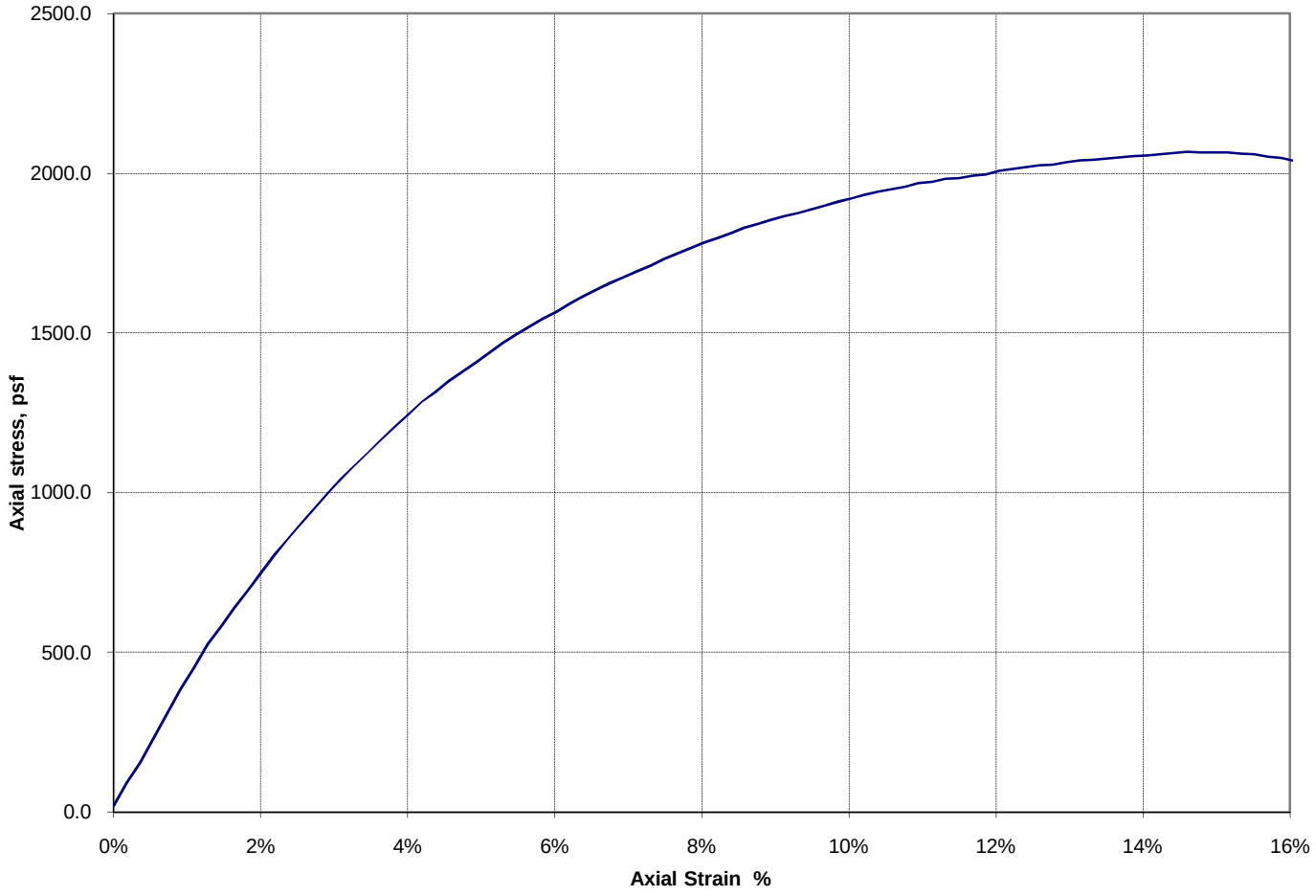
PACIFIC GEOTECHINCAL ENGINEERING

Rev 032806

Figure C-6

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	10	**DEPTH (ft)**	29.5	**SAMPLE**	**DATE OF TEST** 10/3/2009
DESCRIPTION OF SOIL	**CLAY to FAT CLAY:** Black (2.5Y 2.5/1), moist				
LENGTH OF SAMPLE:	5.48 **in.**			**WATER CONTENT:**	19%
DIAMETER OF SAMPLE:	2.38 **in.**			**DRY DENSITY:**	110 **pcf**
HEIGHT TO DIA. RATIO:	2.3			**MAX COMPRESSIVE STRENGTH:**	2065 **psf**
STRAIN RATE:	1.39% **per minute**			**STRAIN AT MAX STRENGTH:**	15.0%



REMARKS: Maximum compressive strength taken at 15% strain

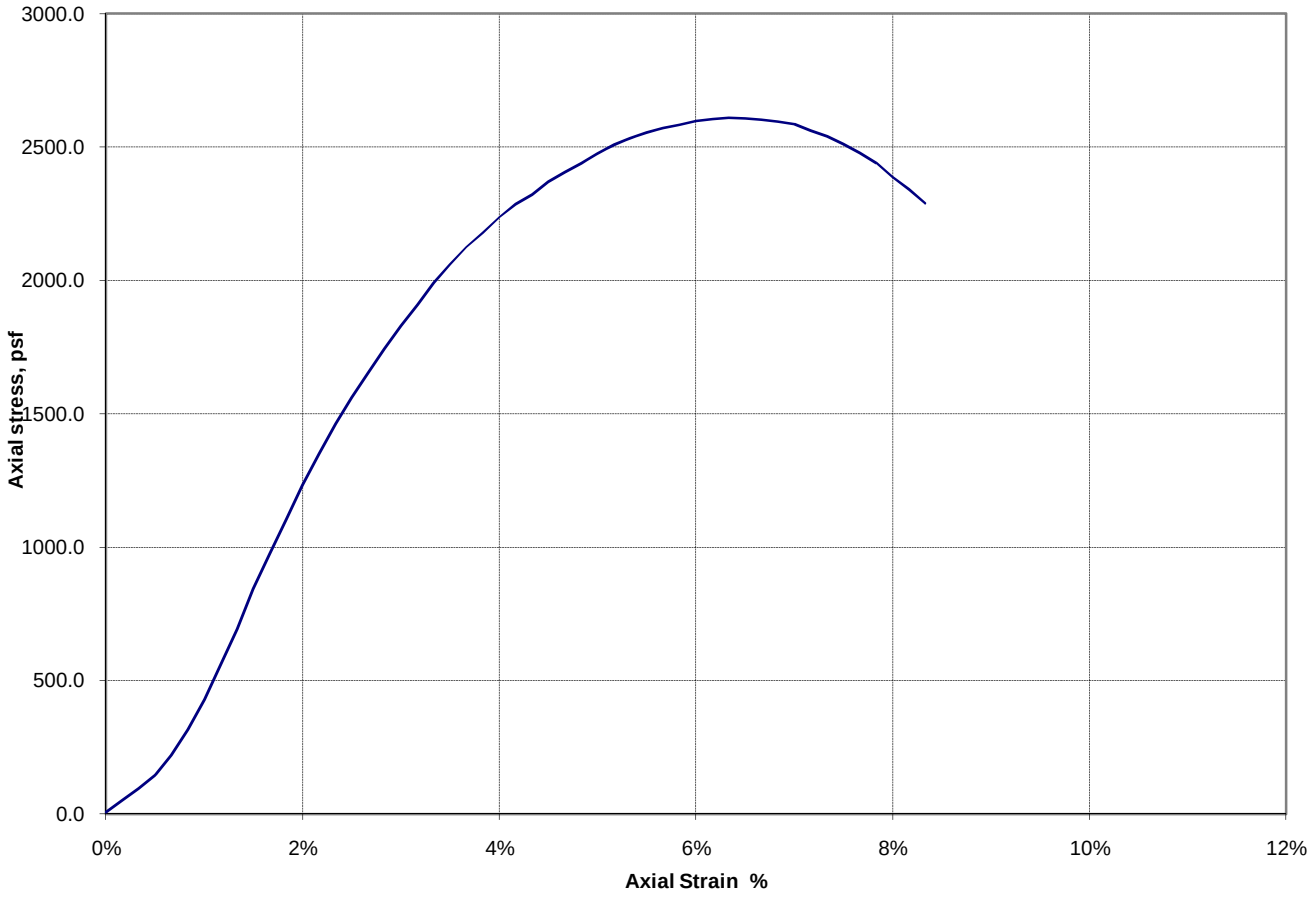
Rev 032806

PACIFIC GEOTECHINCAL ENGINEERING

Figure C-7

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	10	**DEPTH (ft)**	44.5	**SAMPLE**	**DATE OF TEST** 10/19/2009
DESCRIPTION OF SOIL	**CLAY:** Greenish gray (Gley 1 5/1) mottled dark gray (Gley 1 4/1), moist				
LENGTH OF SAMPLE:	6.00 **in.**			**WATER CONTENT:**	26%
DIAMETER OF SAMPLE:	2.39 **in.**			**DRY DENSITY:**	100 **pcf**
HEIGHT TO DIA. RATIO:	2.5			**MAX COMPRESSIVE STRENGTH:**	2610 **psf**
STRAIN RATE:	1.27% **per minute**			**STRAIN AT MAX STRENGTH:**	6.3%



REMARKS:

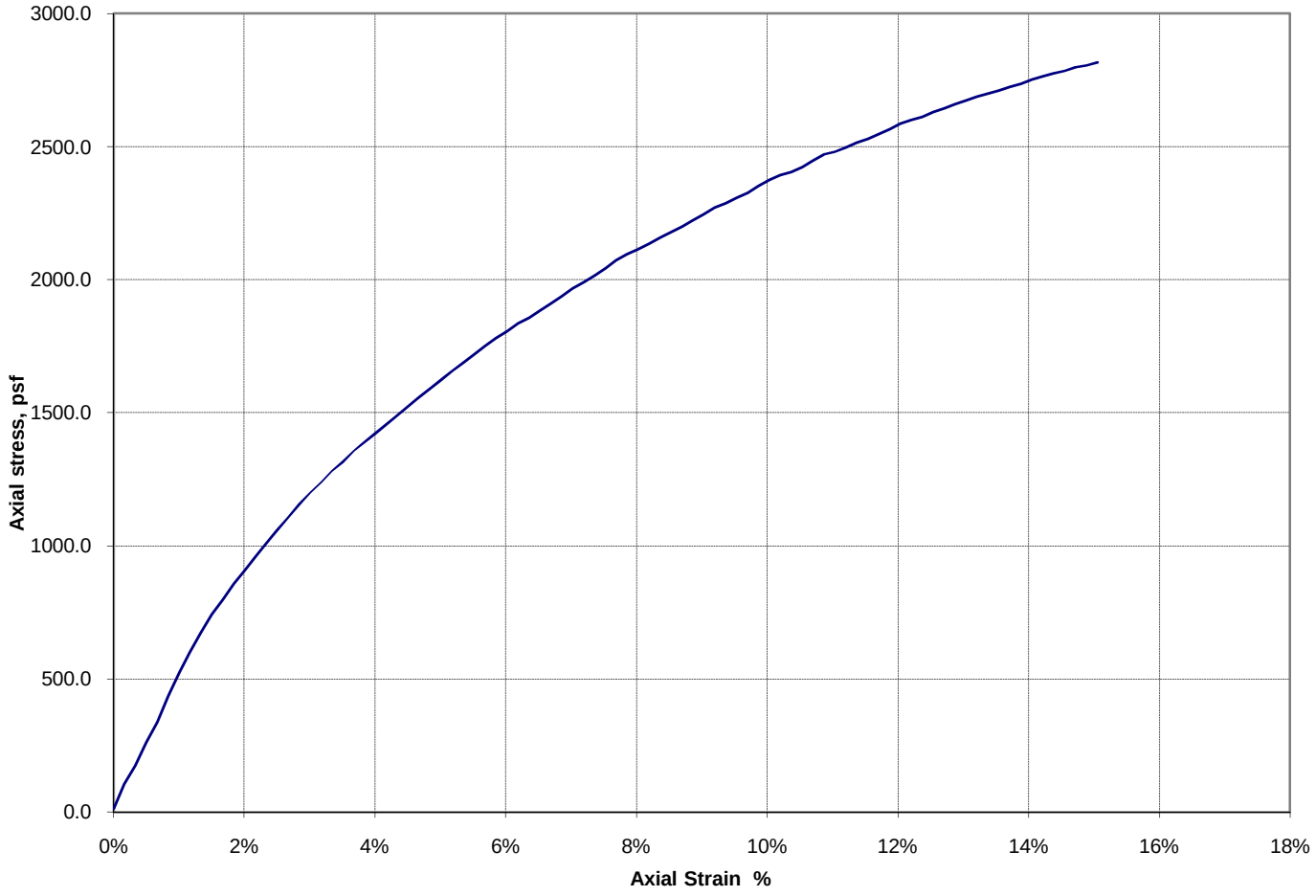
PACIFIC GEOTECHINCAL ENGINEERING

Rev 032806

Figure C-8

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	10	**DEPTH (ft)**	59.5	**SAMPLE**	**DATE OF TEST** 10/3/2009
DESCRIPTION OF SOIL	**LEAN CLAY to CLAY:** Dark greenish brown (2.5Y 4/2), moist				
LENGTH OF SAMPLE:	5.98 **in.**			**WATER CONTENT:**	20%
DIAMETER OF SAMPLE:	2.39 **in.**			**DRY DENSITY:**	110 **pcf**
HEIGHT TO DIA. RATIO:	2.5			**MAX COMPRESSIVE STRENGTH:**	2817 **psf**
STRAIN RATE:	1.34% **per minute**			**STRAIN AT MAX STRENGTH:**	15%



REMARKS: Maximum compressive strength taken at 15% strain

Rev 032806

PACIFIC GEOTECHINCAL ENGINEERING

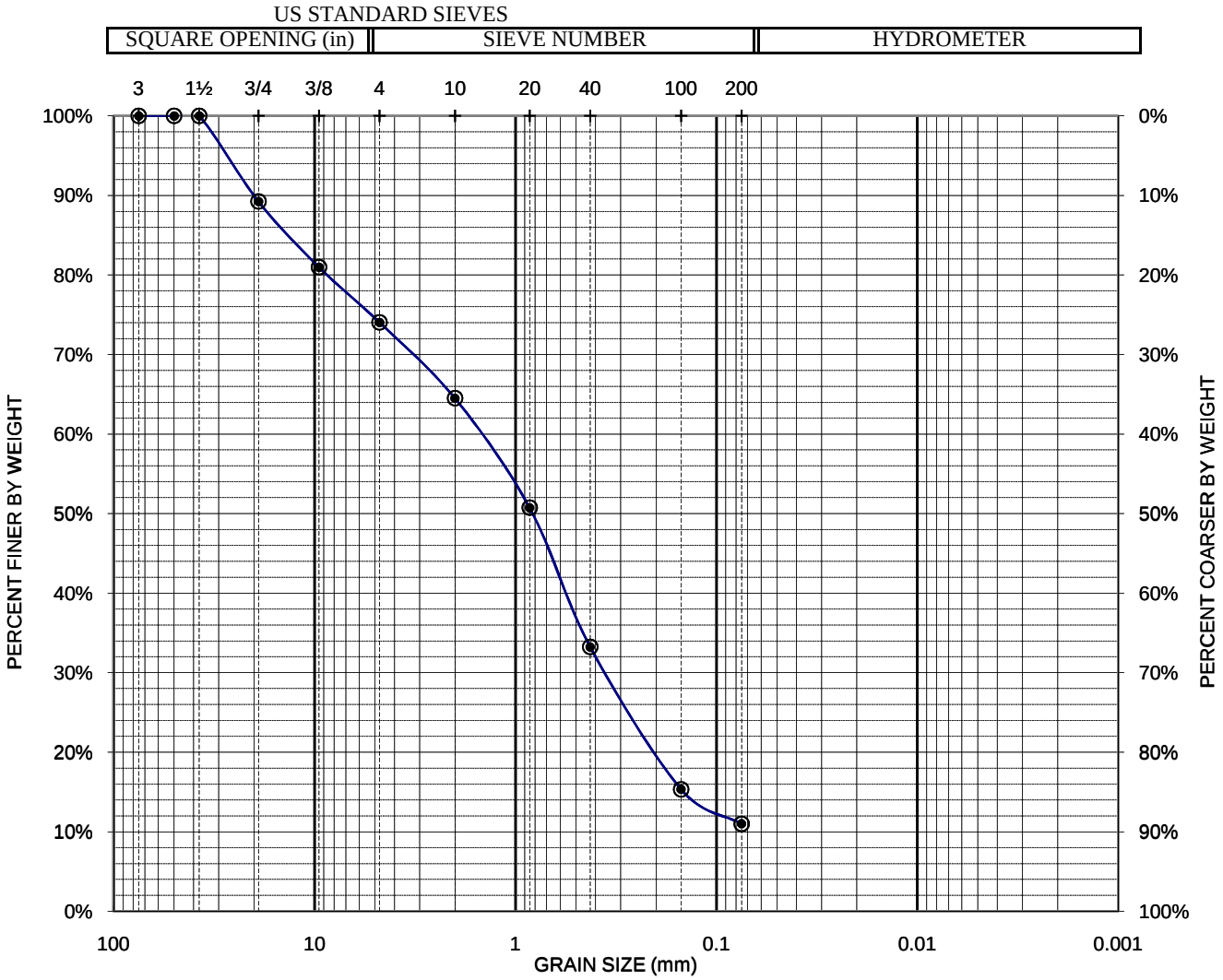
Figure C-9

GRAIN SIZE TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	1	**DEPTH (ft)** 49-50	**SAMPLE** 0	**DATE OF TEST**	10/22/2009

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: WELL GRADED SAND with GRAVEL and SILT: Very dark grayish brown (10YR 3/2), wet



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	26.0%		63.0%			11.0%

REMARKS:

PACIFIC GEOTECHNICAL ENGINEERING

RevOct20070

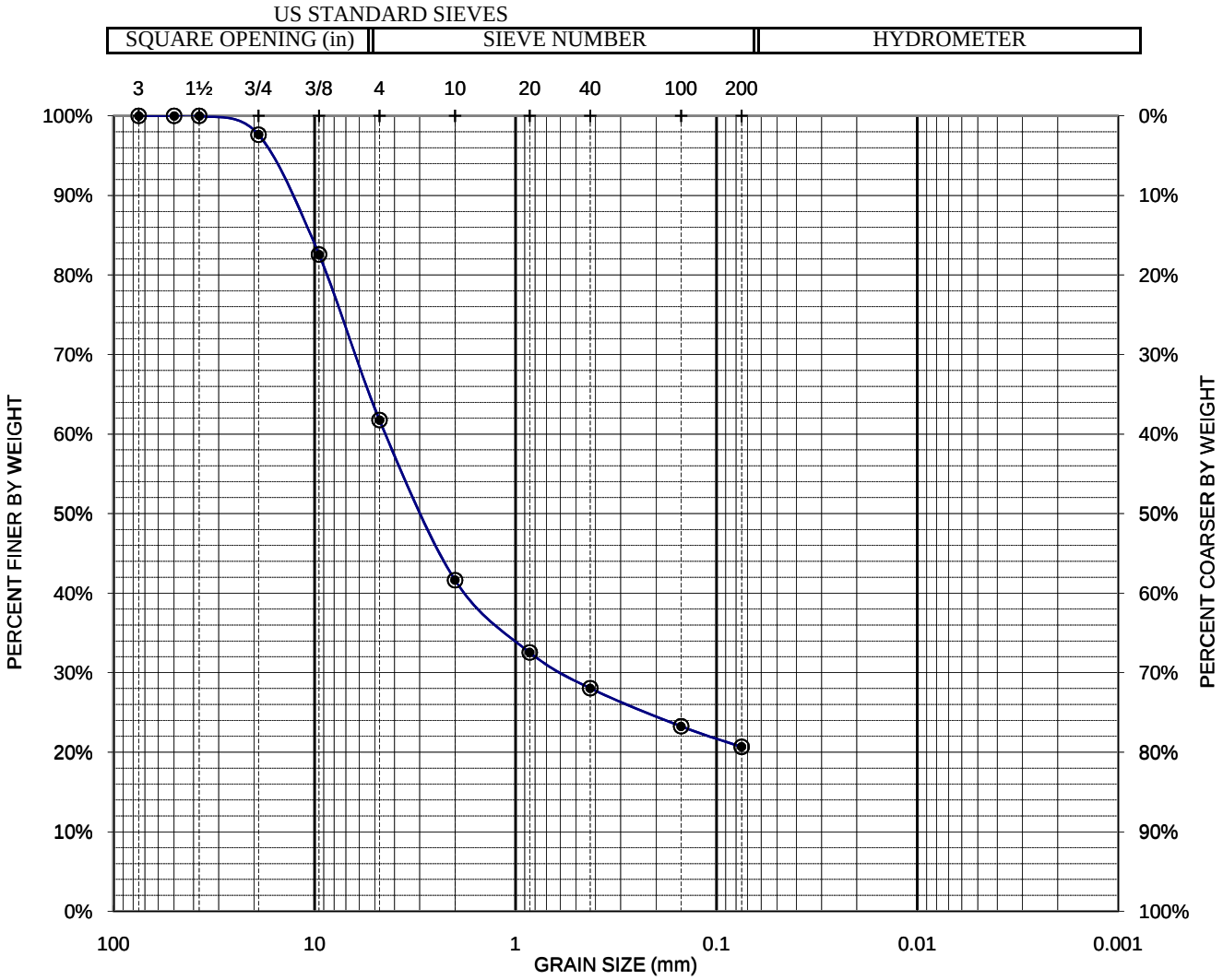
Figure C-10

GRAIN SIZE TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	4	**DEPTH (ft)** 5.5	**SAMPLE** 0	**DATE OF TEST**	10/22/2009

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: CLAYEY SAND with GRAVEL: Brown (10YR 4/3), dry to moist



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	38.2%		41.1%			20.7%

REMARKS:

PACIFIC GEOTECHNICAL ENGINEERING

Figure C-11

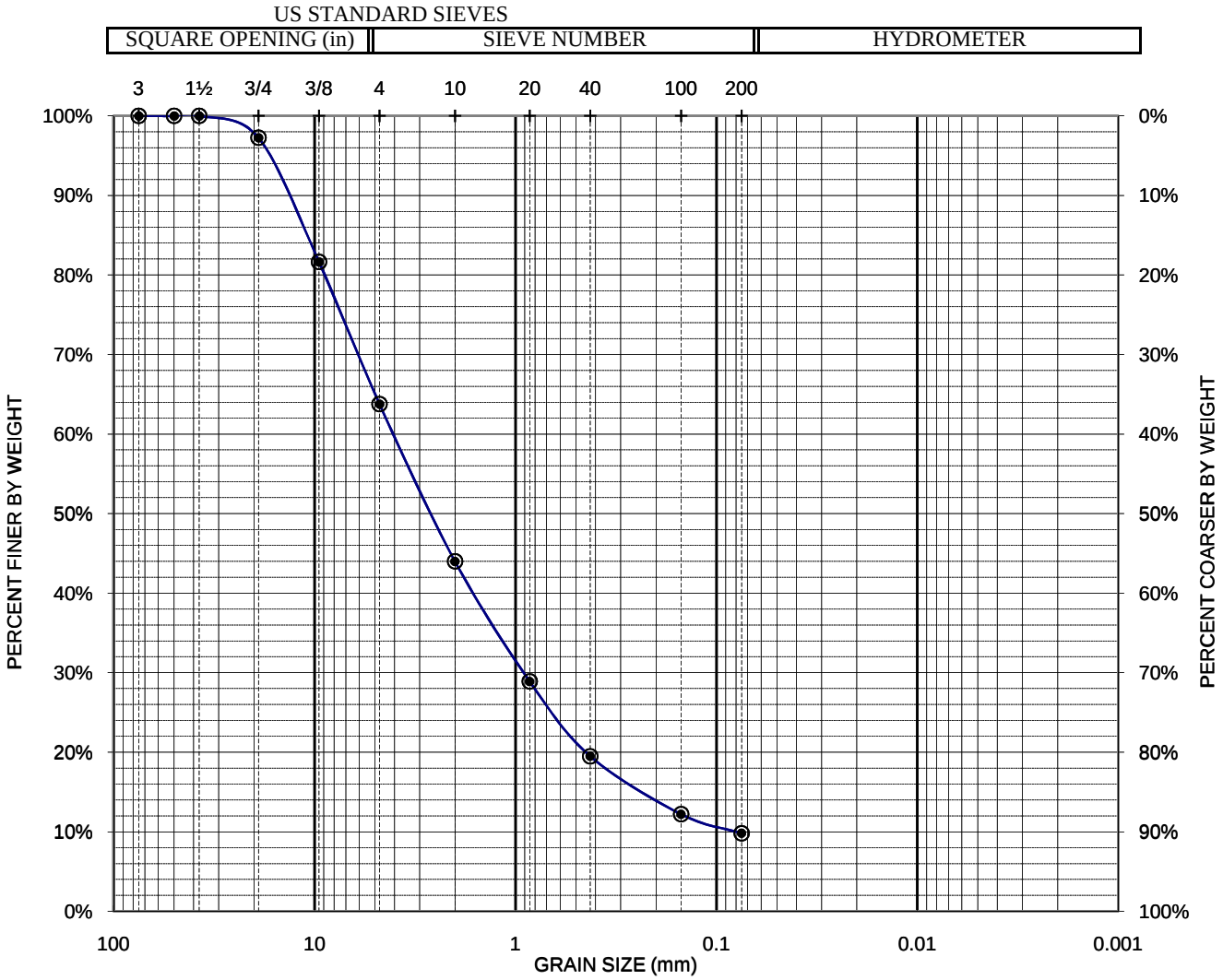
RevOct20070

GRAIN SIZE TEST RESULTS

PROJECT NAME	Coyote Creek Trail			**PROJECT No.**	2298 E
DRILL HOLE No.	8	**DEPTH (ft)** 49	**SAMPLE** 0	**DATE OF TEST**	10/22/2009

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: WELL GRADED SAND with GRAVEL and CLAY: Dark brown (10YR 3/3), wet



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	36.2%		53.9%			9.8%

REMARKS:

PACIFIC GEOTECHNICAL ENGINEERING

RevOct20070

Figure C-12

R-value Test Report (Caltrans 301)

Job No.:	226-184	**Date:**	10/08/09	**Initial Moisture,**	12.0%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	**20**
Project:	Coyote Creek Trail - 2298E	**Reduced**	RU		
Sample	B-1/DH-1	**Checked**	DC	**Expansion Pressure**	**65** psf
Soil Type:	Brown Sandy CLAY				

Specimen Number	A	B	C	D	**Remarks:**
Exudation Pressure, psi	216	385	324		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	60	40	50		
Weight of Soil & Mold, grams	3184	3150	3149		
Weight of Mold, grams	2080	2100	2099		
Height After Compaction, in.	2.56	2.47	2.43		
Moisture Content, %	17.6	15.8	16.7		
Dry Density, pcf	111.0	111.2	112.1		
Expansion Pressure, psf	0.0	120.4	81.7		
Stabilometer @ 1000					
Stabilometer @ 2000	124	106	114		
Turns Displacement	3.61	3.2	3.44		
R-value	17	28	22		

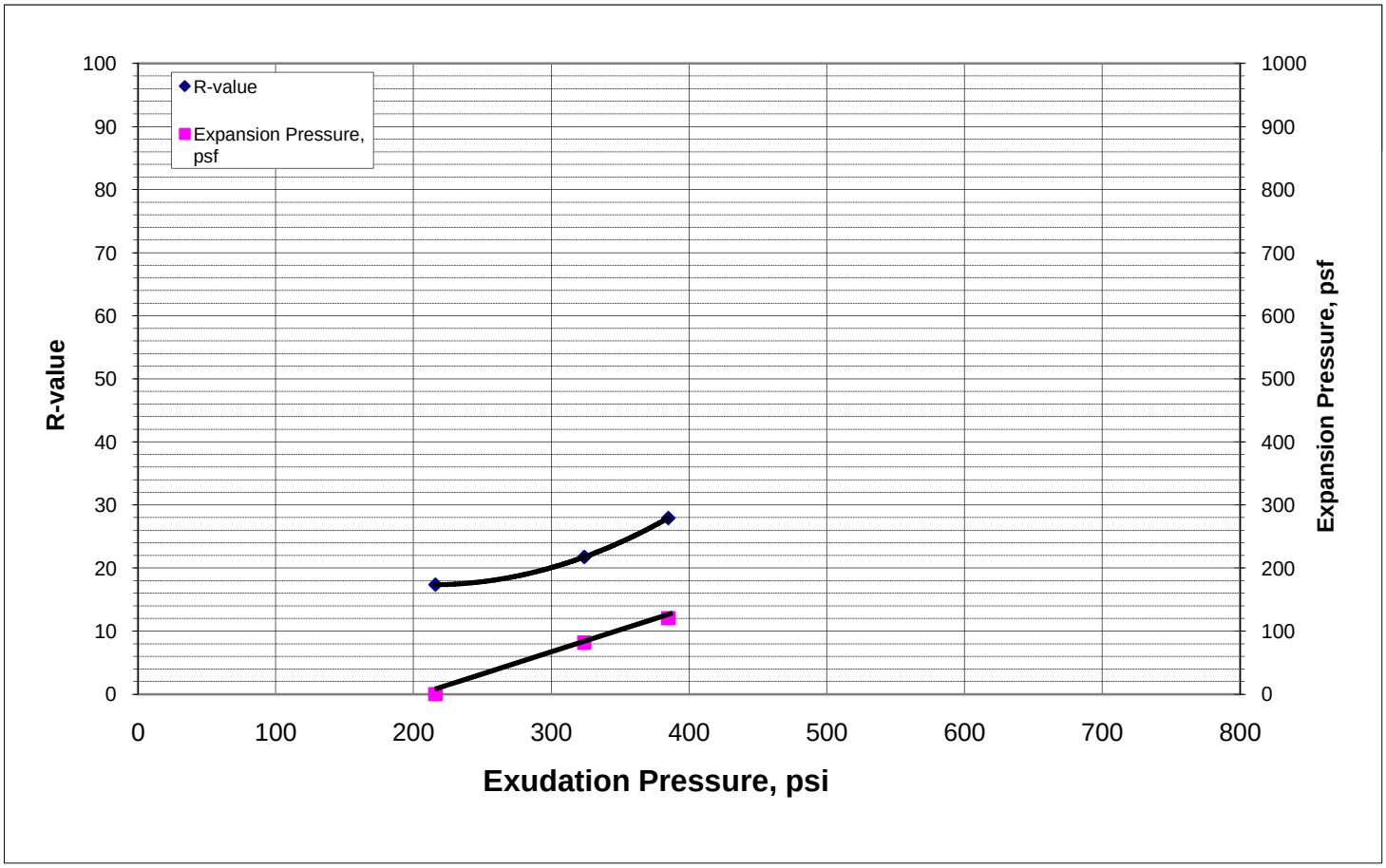


Figure C-13

R-value Test Report (Caltrans 301)

Job No.:	226-184	**Date:**	10/08/09	**Initial Moisture,**	10.0%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	**40**
Project:	Coyote Creek Trail - 2298E	**Reduced**	RU		
Sample	B-3/DH-3	**Checked**	DC	**Expansion Pressure**	**70** psf
Soil Type:	Brown Clayey SAND				

Specimen Number	A	B	C	D	**Remarks:**
Exudation Pressure, psi	134	223	445		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	50	30	10		
Weight of Soil & Mold, grams	3195	3248	3185		
Weight of Mold, grams	2088	2107	2078		
Height After Compaction, in.	2.5	2.54	2.48		
Moisture Content, %	14.5	12.7	10.9		
Dry Density, pcf	117.1	120.7	121.9		
Expansion Pressure, psf	12.9	34.4	176.3		
Stabilometer @ 1000					
Stabilometer @ 2000	122	104	38		
Turns Displacement	4.21	3.72	3.82		
R-value	16	27	68		

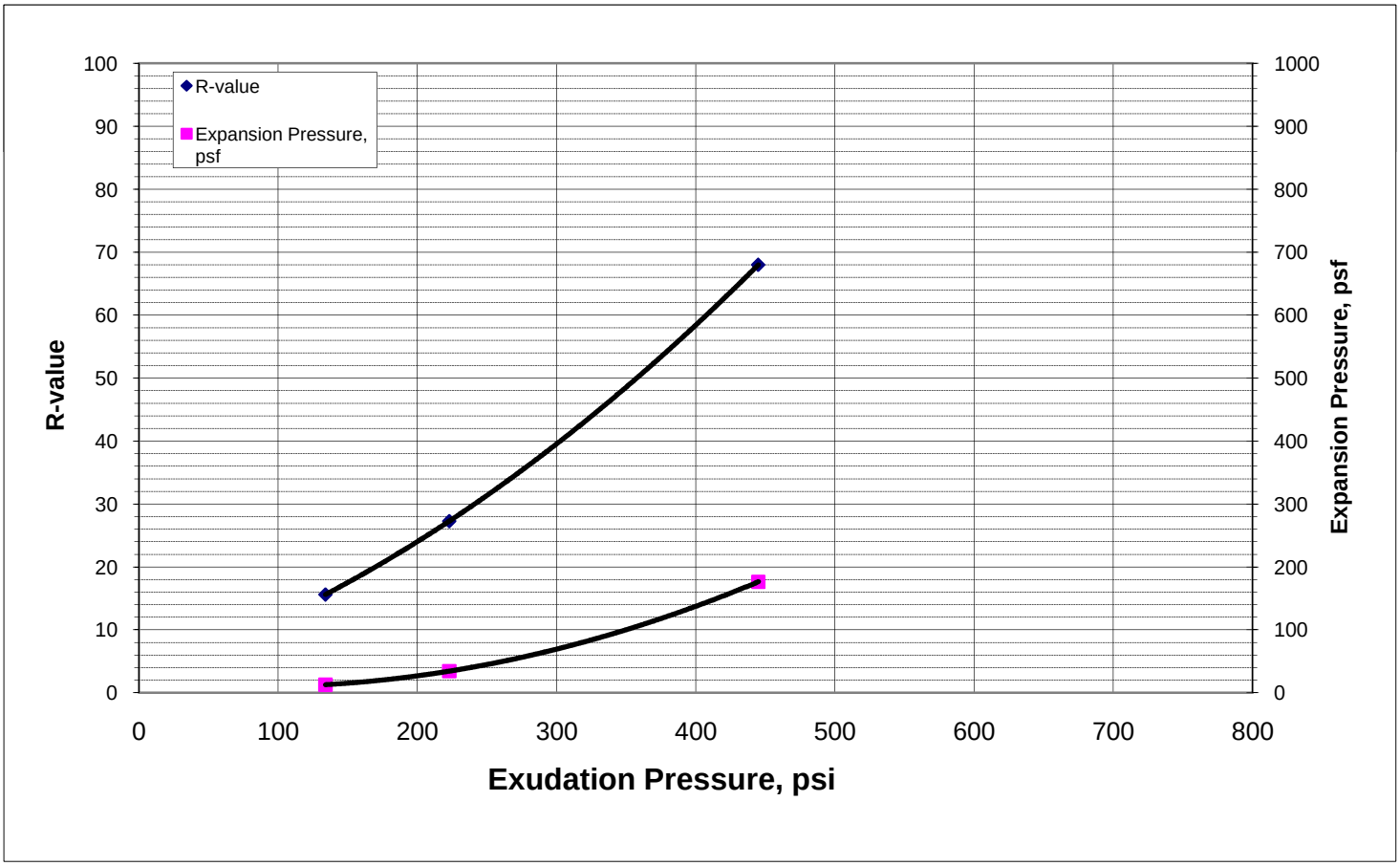


Figure C-14

R-value Test Report (Caltrans 301)

Job No.:	226-184	**Date:**	10/05/09	**Initial Moisture,**	11.6%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	**14**
Project:	Coyote Creek Trail - 2298E	**Reduced**	RU		
Sample	B-5/DH-5	**Checked**	DC	**Expansion Pressure**	**0** psf
Soil Type:	Brown Clayey SAND				

Specimen Number	A	B	C	D	**Remarks:**
Exudation Pressure, psi	194	347	436		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	55	35	20		
Weight of Soil & Mold, grams	3204	3164	3209		
Weight of Mold, grams	2105	2091	2109		
Height After Compaction, in.	2.56	2.5	2.5		
Moisture Content, %	16.7	14.8	13.4		
Dry Density, pcf	111.4	113.2	117.5		
Expansion Pressure, psf	0.0	8.6	141.9		
Stabilometer @ 1000					
Stabilometer @ 2000	145	127	116		
Turns Displacement	3.5	3.12	2.82		
R-value	7	17	25		

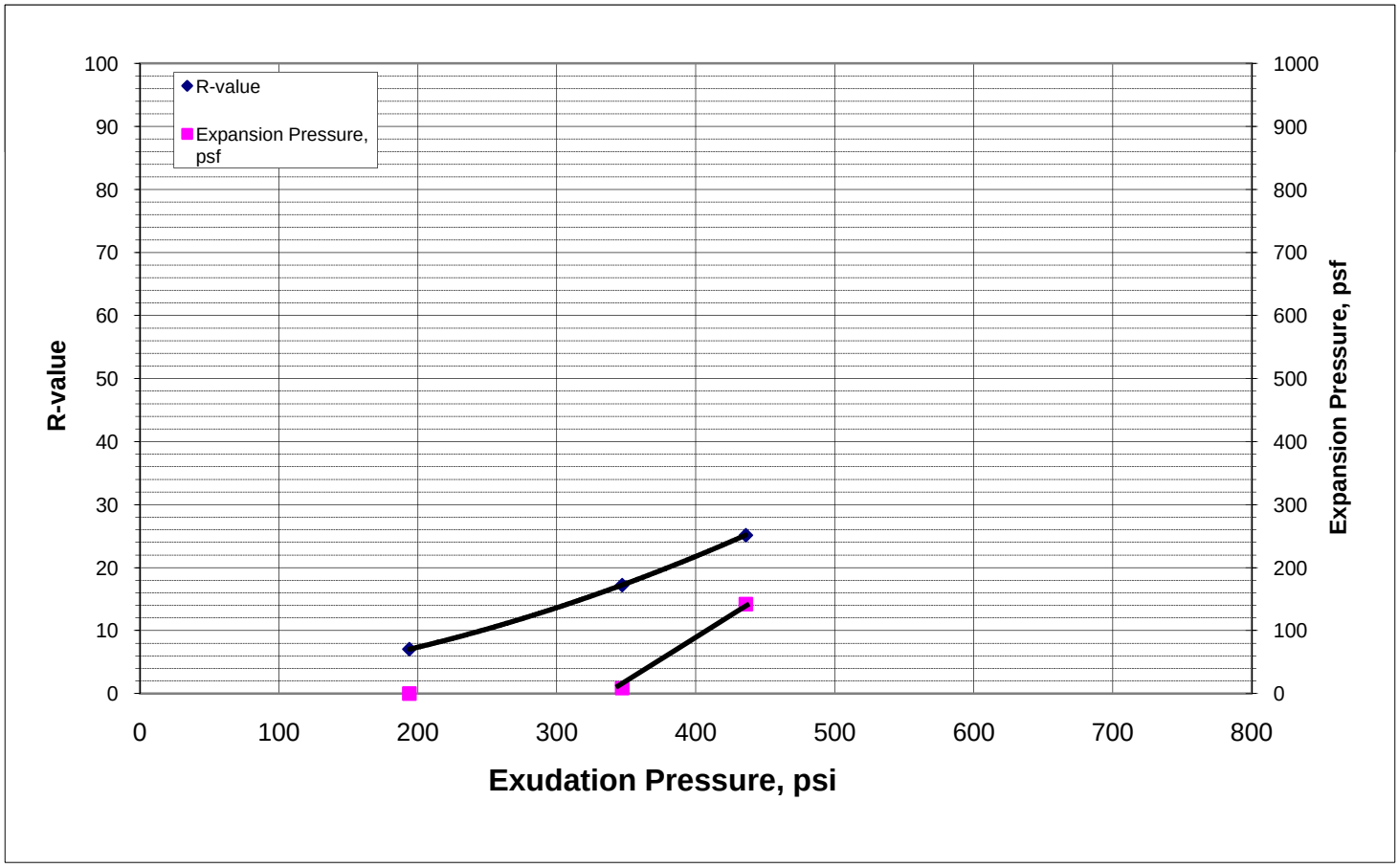


Figure C-15

R-value Test Report (Caltrans 301)

Job No.:	226-184	**Date:**	10/08/09	**Initial Moisture,**	10.8%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	**14**
Project:	Coyote Creek Trail - 2298E	**Reduced**	RU		
Sample	B-8/DH-8	**Checked**	DC	**Expansion Pressure**	**0** psf
Soil Type:	Brown Sandy CLAY				

Specimen Number	A	B	C	D	**Remarks:**
Exudation Pressure, psi	513	303	215		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	41	65	86		
Weight of Soil & Mold, grams	3169	3201	3211		
Weight of Mold, grams	2085	2091	2079		
Height After Compaction, in.	2.4	2.58	2.7		
Moisture Content, %	14.6	16.8	18.8		
Dry Density, pcf	119.3	111.5	106.9		
Expansion Pressure, psf	34.4	0.0	0.0		
Stabilometer @ 1000					
Stabilometer @ 2000	120	131	143		
Turns Displacement	3.3	3.52	4.31		
R-value	19	14	7		

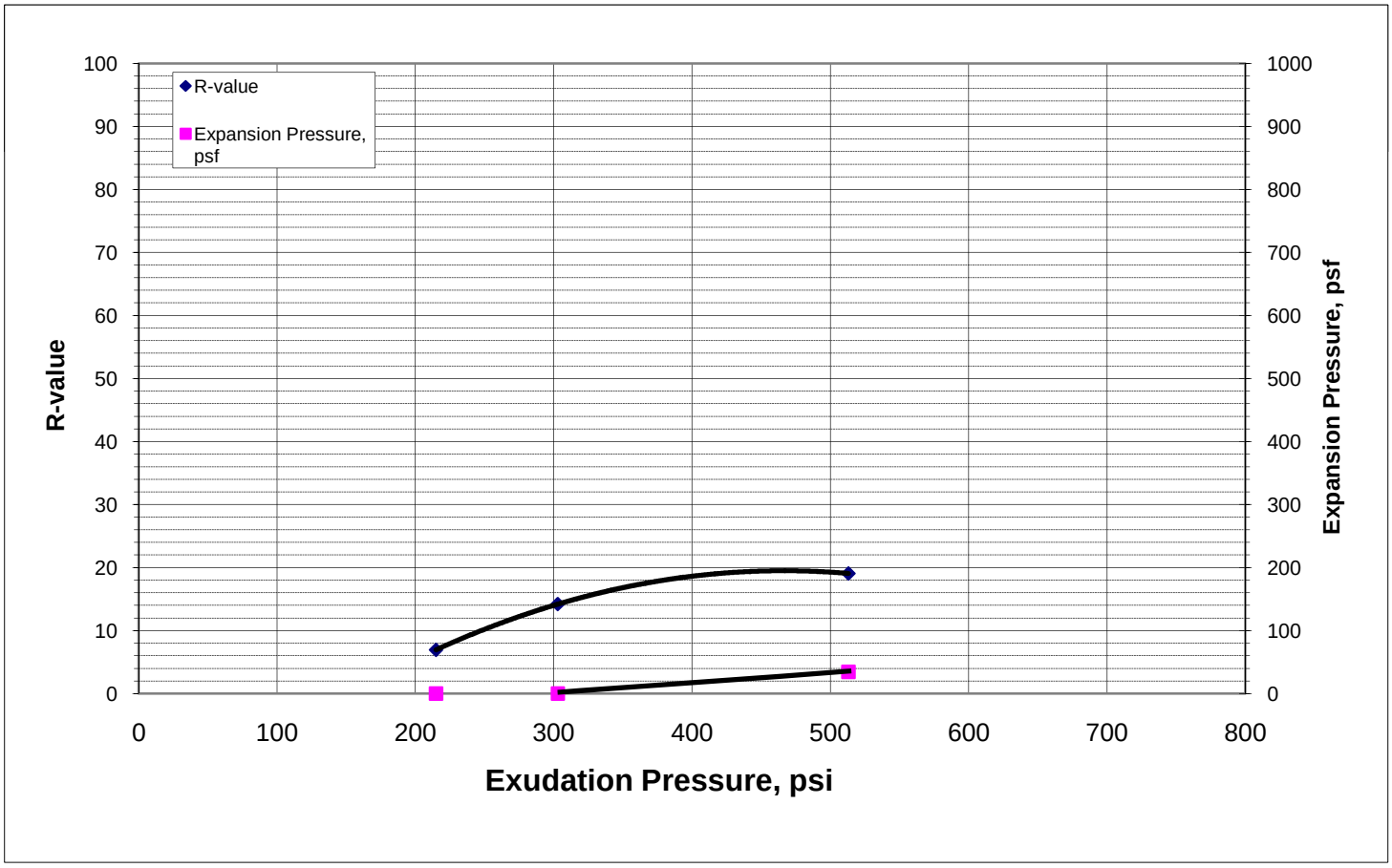


Figure C-16

R-value Test Report (Caltrans 301)

Job No.:	226-184	**Date:**	10/08/09	**Initial Moisture,**	10.7%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	**8**
Project:	Coyote Creek Trail - 2298E	**Reduced**	RU		
Sample	B-10/DH-10	**Checked**	DC	**Expansion Pressure**	**10** psf
Soil Type:	Brown Sandy CLAY				

Specimen Number	A	B	C	D	**Remarks:**
Exudation Pressure, psi	364	298	210		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	60	80	100		
Weight of Soil & Mold, grams	3188	3154	3134		
Weight of Mold, grams	2096	2105	2109		
Height After Compaction, in.	2.51	2.49	2.54		
Moisture Content, %	16.2	18.1	19.9		
Dry Density, pcf	113.3	108.0	101.9		
Expansion Pressure, psf	25.8	8.6	0.0		
Stabilometer @ 1000					
Stabilometer @ 2000	128	141	147		
Turns Displacement	3.31	3.95	4.34		
R-value	16	8	5		

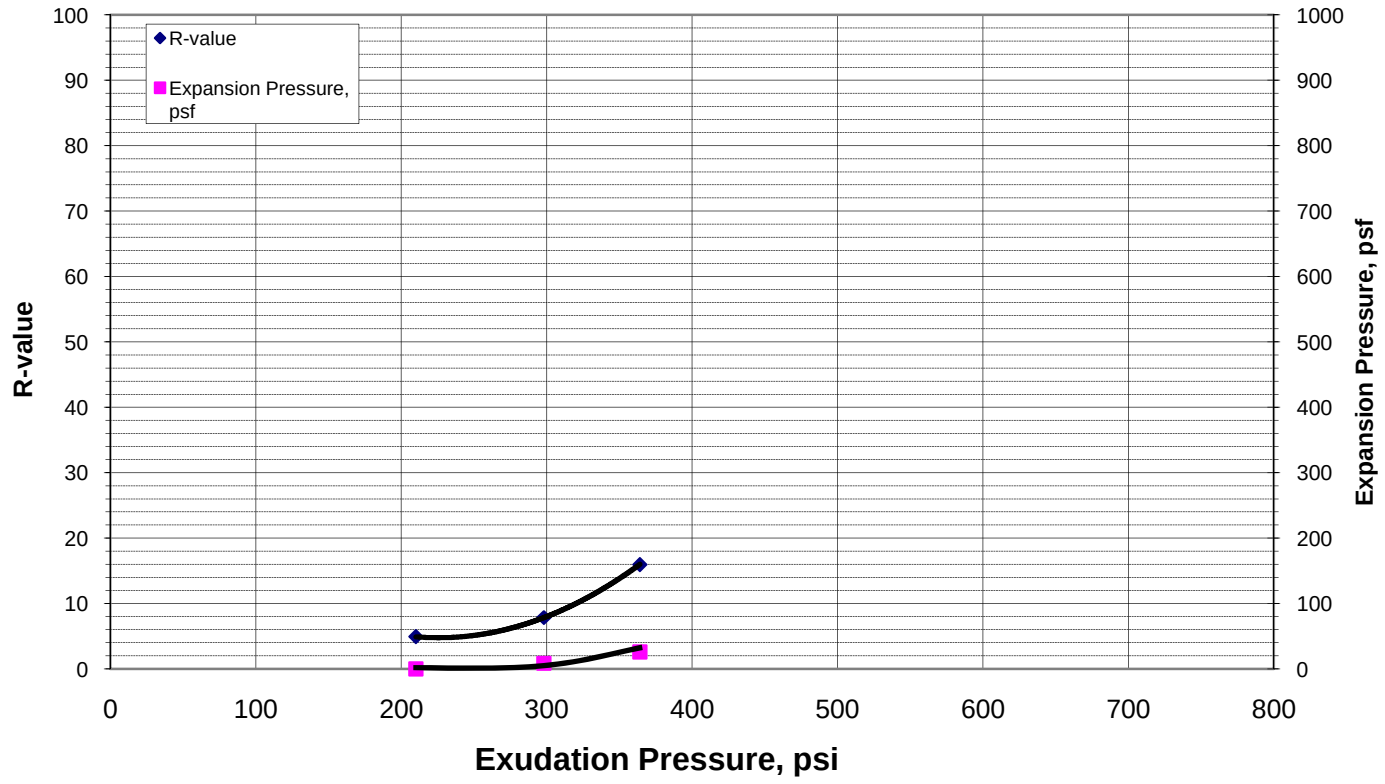


Figure C-17

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax: 925 462 2775
www.cercoanalytical.com

Client: Pacific Geotechnical Engineering
Client's Project No.: 2298E
Client's Project Name: Coyote Creek Trail
Date Sampled: Not Indicated
Date Received: 8-Oct-09
Matrix: Soil
Authorization: Laboratory Program Sheet dated 10/02/09

Date of Report: 28-Oct-2009

Job/Sample No.	Sample I.D.	Moisture (%)	pH	Min.Resistivity** (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0910052-001	DH-1 @ 24'	-	8.3	3,100	-	N.D.	20
0910052-002	DH-2 @ 39'	-	8.3	1,900	-	N.D.	48
0910052-003	DH-10 @ 59'	-	8.3	1,700	-	18	49
0910052-004	DH-10 @ 29'	-	8.8	1,200	-	41	88

Method:	226	CT 532/643	CT 532/643	-	CT 422/ASTM D4327	CT 417/ASTM D4327
Detection Limit:	-	-	-	50	15	15
Date Analyzed:	-	27-Oct-2009	27-Oct-2009	-	27-Oct-2009	27-Oct-2009

Cheryl McMillen
Laboratory Director

* Results Reported on an "As Received" Basis
** Due to insufficient quantity of soil, Miller Box was used
N.D. - None Detected

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

APPENDIX D

SELECTED REGIONAL FAULT DATA

SELECTED REGIONAL FAULT DATA

Calaveras fault – The Calaveras fault passes through the lower foothills of the Diablo Range and roughly marks the eastern margin of the southern Santa Clara Valley. The creeping southern segment of this fault merges with the San Andreas fault near Hollister. The northern and central segments of the Calaveras fault have generated 10 earthquakes greater than moment magnitude (M_w) 5.0 during historic time alone: 1861, 1897, 1899, 1911, 1943, 1949, 1955, 1979, and 1984 (Witter and others, 2003). All but the first of these occurred on the central segment. The magnitudes for many of these earthquakes were fairly close, suggesting a characteristic earthquake of M_w6.2 for the central segment. Current research (CGS, 1996, 2003; WGCEP, 2003) indicates that the maximum earthquakes for the northern, central and southern segments of the Calaveras fault are M_w6.8, 6.2, and 5.8 respectively. The maximum earthquake for a combined central- and southern-segment rupture would likely be M_w6.4.

Hayward fault – The Hayward fault forms the eastern margin of the San Francisco Bay basin, and is linked by a step-over at its northern end to the Rodgers Creek fault north of San Pablo Bay, although the behavior of the two faults appears independent (WGCEP, 2003). The last major earthquake on the Hayward fault occurred in 1868 along the southern segment of the fault, and had an estimated magnitude of M_w6.8. Until recently, it was thought that a similar-magnitude earthquake in 1836 occurred on a northern segment of the fault. However, recent research suggests that the 1836 earthquake occurred somewhere in the vicinity of San Juan Bautista, and not on either the Hayward or San Andreas faults (Toppozada and Borchardt, 1998; Bakun, 1999). The maximum earthquake for a combined northern- and southern-segment rupture of the Hayward fault is estimated to be M_w6.9 (CGS, 1996; WGCEP, 2003). Seismicity data indicate that the southern end of the Hayward fault joins with the Calaveras fault at depth; the portion of the Hayward fault nearest this junction ("Hayward southeast extension") is not considered a separate seismic source by the WGCEP (2003).

Monte Vista/Shannon fault – The Monte Vista/Shannon seismic source essentially composites several separately mapped frontal thrust faults along the northeastern margin of the Santa Cruz Mountains. While some of these west-dipping faults are not considered seismically capable, this aggregate seismic source is considered capable of a M_w6.7 earthquake (CGS, 2003).

Monterey Bay/Tularcitos fault – This relatively short fault lies essentially within Monterey Bay, and it accommodates some of the right-lateral slip carried solely by the San Gregorio fault in areas farther north, in addition to reverse motion. This fault is considered capable of a M_w7.3 earthquake (CGS, 1996, 2003), and it is included in the CGS statewide probabilistic seismic hazards model (CGS, 2003).

San Andreas fault – The San Andreas fault system extends about 700 miles through California, passing through the greater Bay Area from beyond Pt. Reyes to the north, down the San Francisco Peninsula, and extending on beyond Hollister to the south. This fault has generated at least four large, damaging earthquakes during historic time: 1838, 1857, 1906 and 1989. The earthquake of 1838 probably occurred on the Peninsula segment of the fault; it had an estimated magnitude of M_w6.8 (Bakun, 1999) to M_w7.5 (Toppozada and Borchardt, 1998). The earthquake of 1857 occurred in San Luis Obispo County; it had an estimated magnitude of approximately M_w7.9. The 1906 earthquake was probably centered just offshore of the Golden Gate of San Francisco Bay, and had an estimated magnitude of M_w7.9.

The 1989 Loma Prieta earthquake was epicentered in the Santa Cruz Mountains. This M_w6.9 earthquake caused 64 deaths, about 4,000 injuries and about 6 billion dollars of damage in the Bay Area.

The maximum earthquake for the Peninsula segment of the San Andreas fault is currently considered to be M_w7.1 (CGS, 2003; WGCEP, 2003). The maximum earthquake for the Santa Cruz Mountains segment is considered to be M_w7.0. Both segments were considered by the CGS (1996) to have the same 400-year return intervals for the maximum earthquake, although more recent work suggests a shorter return interval (e.g. Hall and others, 1999). A "1906"-style rupture involving several segments would likely be an M_w7.9 event (WGCEP, 2003), and is considered more likely than a multiple-segment event involving the Peninsula and Santa Cruz Mountains segments (WCCEP, 2003).

San Gregorio fault – The San Gregorio fault is a part of the San Andreas fault system located offshore of Monterey Bay. The fault comes onshore near Año Nuevo near the San Mateo/Santa Clara County line, and it is transitional northward to the Seal Cove fault. The northern segment of the fault has a relatively high slip rate of 7mm/yr (+/- 3mm) and is considered to have a maximum earthquake of M_w7.2, while the southern portion is capable of a M_w7.0 (CGS, 2003; WGCEP, 2003). A combined northern- and southern-segment rupture would likely be a maximum of M_w7.4.

Sargent fault - The Sargent fault is considered part of the San Andreas fault system as it splays off from the San Andreas fault north of the City of Santa Cruz. Like other thrust faults east of the San Andreas fault, the Sargent fault is thought to be tectonically coupled with the San Andreas fault at depth. However, Nolan and others (1995) suggest that the Sargent fault may not be tectonically coupled with the San Andreas, and that movement may be associated with distributed shear across the region. The WGCEP (2003) has deleted this fault as a seismic source from their probabilistic model; formerly (WGCEP, 1996) it was considered capable of a M_w6.8 earthquake.

Zayante-Vergeles fault – The Zayante-Vergeles fault accommodates strike-slip and reverse motion. It lies largely parallel to and west of the San Andreas fault in northern San Benito, Monterey, and southern Santa Cruz Counties. The CGS (2003) considers it to be capable of a M_w7.0 earthquake.