ATTACHMENT 4.6-A: PRELIMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

Prepared for

San Diego Gas & Electric Company
8316 Century Park Court, CP52G
San Diego, California 92123

PRELIMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION VINE SUBSTATION

SAN DIEGO, CALIFORNIA

Prepared by

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engineers | scientists | innovators

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Project Number SC0368-30

December 2013

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11 December 2013

Mr. Edwin Reese
San Diego Gas & Electric Company
8316 Century Park Court, CP52G
San Diego, California 92123

Subject: **Preliminary Geotechnical and Geologic Hazards Investigation**
Vine Substation
San Diego, California
Geosyntec Project: SC0368-30

Dear Mr. Reese:

Geosyntec Consultants, Inc. (Geosyntec) is pleased to provide the San Diego Gas & Electric Company (SDG&E) with the accompanying report presenting the results of our preliminary geotechnical and geologic hazards investigation for the Vine Substation project. Our engineering services were performed in accordance with our proposal dated 6 May 2013, our Agreement No. 6160015028 with the SDG&E Civil/Structural Engineering Group, and Release Order 5660028999. This report presents our conclusions and recommendations pertaining to the project and the results of the field exploration program and laboratory testing.

In our opinion, the site is suitable to develop the proposed substation, provided that the recommendations of this report are incorporated into planning, preliminary design, detailed design, and construction. However, interaction will be required during design development between SDG&E and Geosyntec, particularly with respect to re-evaluation and refinement of remedial grading and deep foundation design recommendations. Design iteration and additional geotechnical investigation will be required to optimize the substation geotechnical design.

We appreciate the opportunity to provide geotechnical support to the project. If you need further assistance, please contact us at (858) 674-6559.

Sincerely,

Jennifer L. Nevius

Jennifer L. Nevius, G.E. 2825
Senior Project Engineer

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LIST OF ACRONYMS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
Benton	Benton Engineering, Inc.
bgs	below (existing) ground surface
BMPs	Best Management Practices
Cal OSHA	California Occupational Safety and Health Administration
Caltrans	State of California Department of Transportation
CBC	California Building Code
CGS	California Geological Survey
CIDH	Cast-In-Drilled-Hole
CPT	Cone Penetration Test
CUFAD	Compression/Uplift Foundation Analysis and Design
DCV	design capture volume
E_{pmt}	pressuremeter modulus
EPRI	Electric Power Research Institute
FEMA	Federal Emergency Management Agency
Geosyntec	Geosyntec Consultants, Inc.
IEEE	Institute of Electrical and Electronics Engineers
km	kilometer
ksi	kips per square inch
KTE	Kehoe Testing and Engineering, Inc.
kV	kilovolt
LID	low impact development
M	moment magnitude
M_L	Local magnitude
m	meter
MFAD	Moment Foundation Analysis and Design
MS4	municipal separate storm sewers
NAVD 88	North American Vertical Datum of 1988
NPDES	National Pollutant Discharge Elimination System
ohm-cm	ohm-centimeters
PCC	portland cement concrete
pcf	pounds per cubic foot
pci	pounds per cubic inch

LIST OF ACRONYMS AND ABBREVIATIONS (cont.)

PID	photoionization detector
ppm	parts per million
psf	pounds per square foot
PVC	poly-vinyl chloride
Qaf	artificial fill
Qc	colluvium
Qop6	old paralic deposits
R-value	Resistance value for pavement design
RCFZ	Rose Canyon fault zone
SBT	Soil Behavior Type
SDG&E	San Diego Gas & Electric Company
USCS	Unified Soil Classification System
USGS	United States Geological Survey

1. INTRODUCTION

This report presents the results of the preliminary geotechnical and geologic hazards investigation performed by Geosyntec Consultants, Inc. (Geosyntec) for the San Diego Gas & Electric Company (SDG&E) Vine Substation in San Diego, California. This report was prepared for SDG&E and their consultants for project planning and preliminary design by Ms. Jennifer Nevius, G.E. and Mr. Alexander Greene, C.E.G. of Geosyntec. Mr. Steve Fitzwilliam, G.E. of Geosyntec provided senior review.

1.1 Project Description

We understand that SDG&E is proposing to develop a new 69/12 kilovolt (kV) substation on the approximately 1.5-acre site south of Vine Street and west of Kettner Boulevard in San Diego, California (Figure 1 and Figure 2). Geosyntec's understanding of the project is based on discussions with Mr. Edwin Reese, Mr. Chris Bolton, and Mr. Craig Riker of SDG&E. The new substation is anticipated to be energized by December 2016.

A preliminary concept for the grading and arrangement for substation facilities and equipment are shown on Figure 3. Elements of the new substation are anticipated to include typical substation equipment such as switchgears, capacitor banks, firewalls, circuit breakers, transformers, switch gear, switch rack, bus supports, a control shelter, and screen walls. Foundation types for such equipment typically include shallow spread, strip, and mat foundations, or deep drilled pier foundations. Detailed design information is not currently available. Preliminary deep foundation design information for selected structures foundations, including diameters and estimated structural loads from SDG&E standard designs or similar projects are summarized in the following table.

Typical or Preliminary Foundation Design Parameter	Foundation Type		
	Switch Rack	**12 kV Terminal Arrestor**	**Firewall**
Diameter (feet)	4	2.5	3
Depth (feet)	14	8	10
Center to Center Spacing (feet)	30	8	8
Center to Center Spacing (diameters, D)	7.5D	3.2D	2.7D
Unfactored Downward Axial Load (kips)	43	0	NA
Unfactored Uplift Axial Load (kips)	1	0	NA
Factored Shear X-direction (kips)	18.6	2.5	NA
Factored Shear Y-direction (kips)	38.8	2.5	NA
Factored Moment X-direction (kips-ft)	406	35	NA
Factored Moment Y-direction (kips-ft)	170	35	NA

a. Notes:
b. Preliminary foundation information provided by SDG&E.
c. Axial loads shown are unfactored; our understanding is that SDG&E typically uses a load factor of 2.0.
d. Factored shear and moment values shown include a factor of 2.0.
e. NA = Not available at this time.

1.2 Purpose and Scope of Investigation

The purpose of this preliminary geotechnical and geologic hazards investigation was to explore the subsurface conditions and provide geologic information and geotechnical engineering recommendations for project planning, design, and construction. The scope of our investigation included a review of previous geotechnical investigations, site reconnaissance, field explorations including geotechnical borings and Cone Penetration Test (CPT) soundings, geotechnical laboratory testing, engineering and geologic analyses and evaluations, and development of the conclusions and recommendations presented herein.

1.3 Report Organization

This report presents our findings and conclusions, and provides geologic information and geotechnical recommendations for the civil and structural design and construction of the project. Specifically, the results of the investigation were used to develop conclusions and recommendations regarding:

- General subsurface conditions;
- Geologic setting;
- Geologic and seismic hazards;
- Earthwork;

- Seismic design considerations, including building code and substation equipment qualification level;
- Allowable vertical and lateral capacities of shallow foundations;
- Parameters for design of axially and laterally loaded drilled pier foundations;
- Estimated foundation settlements;
- Earth pressures for retaining walls;
- Utility trenches;
- Slabs-on-grade;
- Flexible pavements;
- Corrosion potential; and
- Construction considerations.

The previous investigations, geotechnical borings, CPT soundings, geotechnical laboratory testing, and seismic evaluations are provided in the appendices of this report.

2. GEOTECHNICAL INVESTIGATION

This preliminary geotechnical and geologic hazards investigation included a review of published regional geologic information, review of previous site geotechnical information, a site reconnaissance, geotechnical borings, CPT soundings, and geotechnical laboratory testing.

2.1 <u>Review of Regional Information</u>

To develop our understanding of the geologic setting and site history, Geosyntec reviewed the following publically-available documents:

- United States Geological Survey (USGS) Historic Site Aerial Photograph [USGS, 1972];
- Historic Site Aerial Photographs [GoogleEarth, 1994-2013];
- "Maps of Known Active Fault Near-Source Zones, California and Adjacent Portions of Nevada" [California Department of Conservation, Division of Mines and Geology 1998];
- "City of San Diego Seismic Safety Study" [City of San Diego Development Services Department, 2008];
- "Earthquake Fault Zones – Point Loma Quadrangle" [California Geological Survey, 2003]; and
- "Geologic Map of the San Diego 30'x60' Quadrangle, California" [Kennedy and Tan, 2005].

Detailed references for these documents are provided in Section 8 of this report.

2.2 <u>Previous Geotechnical Investigations</u>

The previous site development is not well documented in the referenced information available for review. Our knowledge of previous site conditions is based largely on conditions reported in the previous geotechnical investigations provided by SDG&E. Geosyntec reviewed two site geotechnical investigation reports prepared by Benton Engineering, Inc. (Benton) [1974 and 1977].

The Benton [1974] report documented a preliminary geotechnical investigation for the site that advanced two large-diameter (24- and 36-inch diameter) borings, designated Boring 1 and Boring 2, to depths of 15 feet and 25 feet, respectively, and performed geotechnical laboratory testing for compaction and consolidation characteristics. No groundwater was observed in these borings.

The Benton [1974] investigation concluded that site fill indicated insufficient compaction, and therefore removal and recompaction of fill was recommended for structural support. This report also documents that at the time of the investigation, a retaining wall bounded approximately the northeast portion of the site. Benton [1974] reported that a gas station previously occupied the northeast portion of the site, and the fuel reservoir had been removed. However, no records are referenced within that report, or are currently available to document environmental closure of the gas station, the reservoir removal, or the environmental soil conditions in this area. Figure 3 presents a historical aerial photograph of the site area from the USGS archive [USGS, 1972] which shows two structures in the northeast portion of the site, and landscaping along the retaining wall alignment reported by Benton [1974]; the approximate locations of these features are presented on Figure 2. The Benton [1974] report recommended additional borings for fill and environmental characterization.

The Benton [1977] report documented the results of a supplemental geotechnical investigation to evaluate the removal depths of unsuitable soils. Twelve test pits (designated Test Pit 3 to Test Pit 14) were excavated at the site to depths between 4 feet and 13 feet below the existing ground surface (bgs). The report concluded that loose fill and/or porous alluvium existed to a depth of 0.5 feet in the upper area behind the existing retaining wall and from 2.7 feet to 5.7 feet in the lower area of the site. These unsuitable soils were recommended to be removed and recompacted. The Benton [1977] report indicated that the unsuitable soils in the lower area of the site were to be removed and recompacted in one phase of work, and that in a later phase of work, the unsuitable soils behind the retaining wall would be removed and the retaining wall removed by cutting off the wall 2 feet below the proposed finish grade. No records are available to document that the recommended remedial earthwork was completed or to what extent the retaining wall was removed.

Copies of these Benton investigation reports are presented in Appendix A of this report. The locations of the previous borings, test pits, retaining wall, and site structures are shown on Figure 2 as best approximated from the respective Benton [1974 and 1977] reports and the referenced USGS photograph.

2.3 Pre-Investigation Activities

Prior to commencing our field investigation program, a site- and project-specific health and safety plan was prepared, and Geosyntec contacted Underground Service Alert to coordinate clearance of the proposed exploration locations with respect to below ground utilities. A Geosyntec geologist performed a visual site reconnaissance on 5 July 2013 to evaluate surficial site conditions and to mark out the boring locations for utility clearance.

Due to the proposed depth and the potential to encounter groundwater within the borings and CPT soundings, Geosyntec obtained the required geotechnical construction boring permits from the County of San Diego Department of Environmental Health.

2.4 Geotechnical Borings

The current field investigation included drilling five borings (designated Boring B-1 through Boring B-5), generally located in the four corners and the center of the site, to collect representative geotechnical data. The borings were advanced by Tri-County Drilling, Inc. of San Diego, California using a CME 75 truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. The borings were advanced on 10 and 11 July 2013 to their target depths of approximately 41.5 feet or 51.5 feet bgs. These boring locations are shown on Figure 2, as located in the field by measured offsets from existing landmarks. A summary of the boring exploration information is presented in Table 1.

The borings were logged by a Geosyntec geologist, and the soil samples classified in accordance with the Unified Soil Classification System (USCS). A key to logs and the individual boring logs are presented in Appendix B of this report. Sampling information and other pertinent field data and observations are included on the boring logs. Due to the reported former presence of a gas station at the site, Geosyntec screened the soil samples in the field for potential indications of contamination (visual or olfactory) using a Photoionization Detector (PID). The PID did not measure volatile organic compounds or other soil vapors from the site soil samples; notations for the PID measurements are presented on the boring logs in Appendix B of this report.

Boring B-2 and Boring B-3 were left open for about one day during the field exploration program with a two-inch diameter slotted poly-vinyl chloride (PVC) pipe installed as a temporary piezometer for groundwater level observation purposes. On 11 July 2013, groundwater level measurements were recorded periodically, and the PVC pipe was subsequently removed prior to backfill of the borings.

After withdrawal of the augers, portions of the unsupported bore holes caved, generally near the observed groundwater level. The free-standing portions of the borings were backfilled with bentonite grout, and hydrated bentonite chips in some instances, and topped with quickset concrete to restore the pavement surface. Investigative derived waste (soil cuttings) from the borings were temporarily stored in a roll-off bin on site, and then transported for disposal at a municipal solid waste landfill.

2.5 Cone Penetration Test Soundings

CPTs are vertical soundings advanced through the soil with a truck-mounted rig providing thrust. Geosyntec engaged Kehoe Testing and Engineering, Inc. (KTE) of Huntington Beach, California to advance CPT soundings in alignments near the southern and northern property boundaries to support geologic and geotechnical site characterization.

The soundings were conducted using a 60-kilopound capacity cone with a tip area of 2.3 square inches. The CPT measures cone bearing, sleeve friction, and dynamic pore water pressure at 1-inch intervals during penetration to provide a nearly continuous geologic log. The CPT soundings were performed in accordance with American Society for Testing and Materials (ASTM) Standard Test Method D5778 [ASTM, 2012]. Measurements of CPT resistance were used to evaluate the variation of material types and engineering properties. Soil Behavior Type (SBT) and stratigraphic interpretation is based on relationships between cone bearing, sleeve friction, and pore water pressure. The friction ratio is a calculated parameter (defined as the ratio of the sleeve friction to cone bearing) and is used to infer SBT. The results of the CPT soundings and interpretive data provided by KTE are presented in Appendix C of this report.

Twenty-six CPT soundings (designated CPT C-1 through CPT C-22) were advanced between 30 July and 2 August 2013 to their target depths or CPT refusal. The CPT soundings were performed to depths ranging from approximately 43 to 100 feet bgs. CPT refusal was encountered in a few localized areas at depths less than 5 feet and between approximately 43 and 60 feet bgs. Where additional CPT sounding attempts were made, those explorations were designated with a sequential letter after the CPT identification numbers (i.e., C-12, C-12A, C-12B etc.). The locations of the CPT soundings are shown on Figure 2, as located in the field by measured offsets from existing landmarks. CPT soundings performed adjacent to geotechnical borings were compared to correlate and validate the SBT. A summary of the CPT exploration information is presented in Table 1.

Seismic shear wave velocity measurements were collected during CPTs C-1, C-8, C-10, C-15, C-16, and C-22 by measuring the travel time and distance between a geophone in the CPT cone and a seismic source (an air-actuated hammer located inside the front jack of the CPT rig) at the ground surface. The shear wave velocity measurements ranged from about 900 to 1,500 feet per second (ft/s) with an average of approximately 1,300 ft/s. A summary of the shear wave velocity measurements is presented in Appendix C of this report.

After withdrawal of the CPT rods, portions of the unsupported holes caved, generally near the groundwater level. The free-standing portions of the CPT holes were backfilled with bentonite grout, and pavements restored using asphaltic cold patch.

2.6 Geotechnical Laboratory Testing

The soil encountered in the borings was visually classified, and soil samples from the borings were tested by GForce of San Diego, California or their subcontractors to evaluate the physical and engineering properties of the material. The laboratory tests were performed in general accordance with ASTM [2012] or other standard test procedures. The geotechnical laboratory tests performed for this investigation are summarized below.

Geotechnical Laboratory Test	Test Designation
Moisture Content	ASTM D2216
Grain Size Analysis	ASTM D422 or D1140
Laboratory Compaction	ASTM D1557
Atterberg Limits	ASTM D4318
Expansion Index	ASTM D4829
Unit Weight	ASTM D2937
Specific Gravity of Soils	ASTM D854
Soil Corrosivity	ASTM D4972, CTM 417, CTM 422
R-value	CTM 301

A suite of tests for chloride and sulfate content, resistivity, and pH were performed on selected soil samples to evaluate their potential corrosivity. One R-value test was performed to evaluate subgrade resistance for pavement design. The laboratory tests and results are summarized at the corresponding sample locations on the boring logs in Appendix B of this report; detailed test results are presented in Appendix D of this report.

3. SITE CONDITIONS

Knowledge of the site conditions was developed from a review of regional geologic conditions, previous geotechnical investigations by others, and the current investigation.

3.1 Regional Geology

The San Diego area is locally underlain by a thick sequence (greater than 5 kilometers [km]) of Mesozoic volcanic flow rocks and volcaniclastic breccias, of which a large portion has undergone low-grade metamorphism. These late Jurassic to early Cretaceous-age rocks have been intruded by a basement of Cretaceous-age granitic rocks of the Peninsular Ranges Batholith. A series of sedimentary rocks from the Late Cretaceous, Eocene, and Oligocene periods overlie the basement and metamorphic rocks and are represented by the Rosario, La Jolla, and Poway Groups, respectively. The sedimentary deposits of the Rosario group unconformably rest upon the deeply weathered basement and rocks, followed by the La Jolla and Poway Group sediments which were deposited during several major marine transgressive-regressive cycles. The next major marine transgression did not occur until the Pliocene when the strata of the San Diego Formation were deposited. The San Diego coastal margin has undergone relatively steady uplift since the deposition of the San Diego Formation and continues today. A series of marine abrasion platforms have evolved during this time and are represented as terraces which contain Pleistocene-age paralic deposits locally [Kennedy and Tan, 2005].

The site lies within the coastal margin along the western flanks of the Peninsular Ranges of southern California. The general site area includes hilly, locally rugged terraced surfaces dissected by numerous drainages extending to the southwest. The site is at the northwest corner of San Diego Bay, which roughly defines a gently folded, fault-bounded, sedimentary basin referred to as the “San Diego Embayment”. A regional geologic map is presented on Figure 4.

3.2 Seismic Setting

The tectonic setting of the San Diego region is dominated by right-lateral, strike-slip faults with a general northwest by southeast trend. Faults of tectonic significance that have been mapped in the San Diego region and the historical earthquake epicenters in the region are presented on Figure 5. These regional faults include the Elsinore and San Jacinto faults to the east, the Coronado Bank and San Diego Trough faults in the offshore zone to the west, the San Miguel-Vallecitos and Aqua Blanca faults to the south, and the Rose Canyon fault zone (RCFZ) which extends through the general vicinity of the site. These faults and their respective distances from the site and design maximum moment magnitudes are presented in Table 2.

The site is located within the RCFZ, which dominates the seismic exposure in San Diego [Lindvall and Rockwell, 1995]. The RCFZ is the southern extension of the Newport-Inglewood - Rose Canyon fault zone and is associated with the San Andreas fault system which forms the tectonic boundary between the North American and Pacific plates. Together with the Newport-Inglewood fault zone, the RCFZ is considered a continuous zone of five fault segments with a total length of approximately 175 km. Studies in San Diego indicate an estimated slip rate of 1.5 millimeters/year along the RCFZ [Rockwell et al., 1991].

The on-shore portion of the RCFZ extends along the northeast flank of Mount Soledad and continues southward along the eastern margins of Mission Bay. The RCFZ splays out to the south as a number of strands between Mission Bay and San Diego Bay. These strands are known as the Silver Strand, Coronado, and Spanish Bight faults, respectively [Treiman, 2002].

San Diego has experienced strong seismic shaking and minor damage from local and distant earthquakes, but none have been very destructive. A large earthquake in 1862 may have been centered locally [Anderson et al., 1989], and some researchers have suggested the 1862 event could have been in or near San Diego Bay. More recently, San Diego Bay has been the location of several “swarms” of repeated small to moderate magnitude earthquakes. In 1985, a series of earthquakes (largest event was Moment Magnitude [**M**] 4.7) was generally centered just south of the San Diego-Coronado Bridge [Reichle et al., 1985]. The maximum credible earthquake on the Rose Canyon Fault is **M**7.2. In comparison, the **M** of this historic earthquake and the maximum credible earthquake on the RCFZ would correspond to respective local magnitudes (M_L) of approximately 4.7 and between 6.7 to 7.0 on the obsolete Richter Scale.

3.3 Surface Conditions

The site is currently occupied by an asphalt-paved commercial parking lot. The site is bounded respectively to the north, east, south, and west by Vine Street, Kettner Boulevard, the closed portion of Upas Street, and trolley and rail right-of-way to the west. Aerial imagery and topographic information referencing the North American Vertical Datum of 1988 (NAVD 88) is presented on Figure 2 [Project Design Consultants, 2013].

The site has been previously graded and is gently sloping to the southwest. Elevations in the general site area range from approximately +44 feet NAVD 88 near the eastern site boundary along Kettner Boulevard to an elevation of approximately +35 feet to +36 feet NAVD 88 near the western site boundary. From the parking area, a 3-foot to 4-foot high slope ascends to Kettner Boulevard, and an approximately 4-foot high slope descends to the San Diego Metropolitan Transit System right-of-way.

As seen on Figure 2, the low-height slopes are vegetated with a few trees and moderate height brush. A concrete-lined drainage ditch is present at the toe of the descending slope on the west side of the site.

3.4 Subsurface Conditions

The site subsurface conditions were observed and documented in the previous Benton borings and test pit explorations and the recent geotechnical borings and CPT soundings. These explorations indicate that undocumented fill typically overlies colluvial deposits and old paralic deposits (previously referred to as the Bay Point Formation) across the site area. Generalized geologic cross sections are presented on Figure 6 through Figure 9; the locations of these cross sections, previous explorations, and current explorations are presented on Figure 2. Additionally, detailed logs of these explorations are presented in Appendix A (previous investigations), Appendix B (geotechnical borings), and Appendix C (CPT soundings) of this report.

3.4.1 Undocumented Fill

Fill was reported by Benton [1974 and 1977] and observed in the borings and inferred from the CPT soundings performed for this investigation to a depth of approximately 0 to 5 feet bgs in a majority of the subsurface explorations. Benton Boring 2 and Test Pit 14, and Geosyntec Boring B-5 were the deepest reported or observed fill depths at 12.25 feet, 10.5 feet, and 7 feet bgs, respectively. These explorations are approximately located in the central portion of the site, in the area of previous site development. These deeper fill depths may be associated with underground storage tank installation or removal activities.

Records for fill placement and compaction are not available for review; therefore, the fill is considered undocumented fill. The Benton [1977] report indicates that removal and recompaction was planned, but confirming documentation is not available. The fill observed in the Geosyntec borings and reported by Benton consists primarily of silty fine sand to fine sandy silt. Localized gravels were observed in the fill in Geosyntec Boring B-2, Boring B-4, and Boring B-5. Metal debris was observed in the fill at a depth of approximately 6 feet in Boring B-5, and concrete debris was observed in shallow refusal attempts near CPT C-18C and reported in Benton Test Pit 14.

3.4.2 Colluvium

The native soil immediately below the fill is described as alluvium by Benton [1974 and 1977]; Geosyntec describes these same materials as colluvium. Alluvium refers to materials deposited by flowing streams, whereas colluvium refers to material that accumulates at the foot of a slope, such as the ascending slope to the east of the site, east of Kettner Boulevard. These Quaternary-age materials were not encountered in Boring B-3, but were encountered during the current investigation below the fill to a depth of approximately 3.5 to 11.5 feet bgs in other areas of the site. The consistency of the colluvium was variable in nature and the descriptions range from silty fine sand to silt and silty clay that is slightly porous to very porous.

3.4.3 Old Paralic Deposits

The Bay Point Formation is widespread and well exposed in the area adjacent to the present-day coastline [Kennedy, 1975]. Subsequent geologic mapping efforts have subdivided the Bay Point Formation into a series of paralic and older paralic deposits [Kennedy and Tan, 2005]. These deposits are composed mostly of marine and nonmarine, poorly consolidated, fine- and medium-grained, pale brown, fossiliferous sandstone. These old paralic deposits are considered to be nonmarine slope wash.

Recent geologic mapping (shown on Figure 4) describes the site area as underlain by “Old Paralic Deposits, Unit 6” (Qop$_6$) of middle to early Pleistocene age [Kennedy and Tan, 2005]. These old paralic deposits (late to middle Pleistocene) rest on the 22-23 meter (m) Nestor terrace; and primarily consist of poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate.

3.5 Groundwater

Groundwater was not encountered within the previous Benton explorations performed at the site to depths up to 25 feet bgs. However, groundwater was inferred at the time of drilling of the geotechnical borings performed for this investigation between depths of 24 feet and 25 feet bgs based on measured groundwater level. Groundwater level was also inferred based on other conditions such as soil sample saturation or borehole caving upon auger or CPT rod withdrawal. The observed groundwater levels mimic the site grades and geologic layering, sloping to the southwest. The groundwater was measured at elevations between +11 feet NAVD 88 and +19 feet NAVD 88.

Temporary piezometers were installed in Boring B-2 and Boring B-3, consisting of a 2-inch diameter slotted PVC pipe, installed after drilling and prior to auger withdrawal, to monitor the groundwater level. The groundwater level measured in these temporary piezometers was recorded between approximately 24 feet and 25 feet bgs and is considered to represent relatively stabilized levels, inasmuch as standing water levels were monitored up to about 6 to 18 hours after drilling. However, the subsurface materials at the site generally exhibited a relatively high percentage of silt and clay-sized materials, which reduce permeability. It is possible that the observed groundwater levels in the temporary piezometers may not represent fully stabilized groundwater conditions due to the time rate of travel of groundwater through these materials.

In addition, the field investigation was performed during a season characterized with relatively little rainfall. Seasonal rainfall can influence the position of the groundwater level, with wetter seasons increasing the elevation of groundwater.

Groundwater samples were not collected during this investigation, in accordance with the SDG&E environmental release for the project, because visual or olfactory indications of contamination were not observed in site soil.

3.6 Stratigraphic Correlations

3.6.1 Geologic Cross Section A-A’

Geologic Cross Section A-A’ (Figure 6) was created adjacent to the northern site boundary from the southwest to the northeast and includes two borings, one test pit, and eight CPT soundings. Section A-A’ suggests that the fill and colluvium within this portion of the site are of relatively uniform thickness and extend to depths of approximately 4 and 9 feet bgs, respectively. Within the underlying old paralic deposits, six stratigraphic packages of alternating silty sands and silts to clays were observed within the limits of our subsurface explorations at respective basal contact elevations between elevation +9 and -30 to -35 feet, NAVD 88. These stratigraphic units exhibit relatively uniform thickness, and range in dip direction from slight westerly in the upper portion of the unit to easterly with increasing depth.

3.6.2 Geologic Cross Section B-B’

Geologic Cross Section B-B’ (Figure 7) was created adjacent to the southern site boundary from the southwest to the northeast and includes two borings, three test pits, and 17 CPT soundings. Section B-B’ indicates that the fill pinches out in the vicinity of CPT C-13A near the northeastern site boundary; the fill basal contact increases with depth uniformly to the southwest to a maximum thickness of 3 to 4 feet bgs near the southwestern site boundary. The underlying colluvium appears to outcrop immediately beneath the asphalt surface northeast of CPT C-13A. Similar to the overlying fill, the basal contact of the colluvium increases with depth uniformly to the southwest to a maximum thickness of 10 to 11 feet bgs. Within the underlying old paralic deposits, seven stratigraphic packages of alternating silty sands, sands, and silts to clays were identified within the limits of our subsurface explorations. These stratigraphic units exhibit a gentle westerly dip, and the basal contacts undulate locally resulting in variable unit thicknesses from northeast to southwest.

3.6.3 Geologic Cross Section C-C’

Geologic Cross Section C-C’ (Figure 8) was located diagonally across the site from the northwest to the southeast and includes four borings, five test pits, and two CPT soundings. Section C-C’ indicates that the thickness of the fill across the site from northwest to southeast is variable, ranging in depth from approximately 3 to 12 feet bgs. The deepest fill appears to be located in the vicinity of the Benton Boring 2. It is possible that the area of the deepest fill was associated with a previous underground storage tank installation or removal; the excavation appears to extend to the underlying old paralic deposits. The colluvial deposits underlie the fill across the majority of Section C-C’ except in the vicinity of the deeper fill noted above and within the southeastern portion of the site where it pinches out and fill was observed to overlie the old paralic deposits directly. The basal contact of the colluvium exhibits a uniform westerly dip and ranges in elevation from +38 to +26 feet, NAVD 88, from southeast to northwest, respectively. Old paralic deposits were encountered beneath the colluvium to the maximum extent of the subsurface explorations performed for this investigation. Stratigraphic packages were not interpreted along Section C-C’ within the old paralic deposits due to the widely spaced subsurface explorations.

3.6.4 Geologic Cross Section D-D’

Geologic Cross Section D-D’ (Figure 9) was located diagonally across the site from south to north, and includes 4 borings, one test pit, and one CPT sounding. Section D-D’ indicates that the thickness of the fill is variable from south to north, ranging in depth from approximately 3 to 12 feet bgs. Similar to Section C-C’, the deepest fill was observed in the vicinity of Benton Boring 2. The basal contact of the underlying colluvium exhibits a uniform southerly dip, ranging in elevation from +38 to +26 feet NAVD 88, respectively from north to south. The colluvium also maintains a relatively consistent thickness across the site except in the vicinity of the noted deeper fill where it appears to have been removed. Old paralic deposits are encountered beneath the colluvium to the maximum extent of the subsurface explorations performed for this investigation. Stratigraphic packages were not interpreted along Section D-D’ within the old paralic deposits due to the widely spaced subsurface explorations.

4. GEOLOGIC HAZARDS

The conclusions and discussions below are based on the current field explorations, previous investigations by others, and geologic interpretation of the site-specific subsurface information.

4.1 Potential Site Faulting

The potential for fault surface rupture is generally considered to be significant along “active” faults (defined as exhibiting surface rupture within the past 11,000 years) and to a lesser degree along “potentially active” faults (surface rupture within the past 1.6 million years).

4.1.1 Desktop Faulting Evaluation

Prior to the field investigation, the position of the site relative to mapped fault traces was evaluated. A review of published geologic maps did not identify the presence of any active or potentially active faults crossing the project site. Further, the site is not located within a delineated earthquake fault rupture hazard zone as defined by the California Geological Survey (CGS), formerly known as the California Division of Mines and Geology [Hart and Bryant, 1997]. In addition, the site is not situated in the “Downtown Special Fault Zone,” designated by the City of San Diego Seismic Safety Study [City of San Diego, 2008] and is instead delineated as Geologic Hazard Category 53 (level or sloping terrain, unfavorable geologic structure, low to moderate risk). However, mapped segments of the RCFZ are located to the northeast and west of the site at distances of approximately 650 and 3,600 feet, respectively, and the site is mapped within the RCFZ [USGS and CGS, 2010].

4.1.2 Field Faulting Evaluation

The approach for the field data collection portion of the preliminary fault investigation was to advance explorations in two alignments roughly perpendicular to the direction of mapped faulting along the southern and northern property boundaries and to evaluate the continuity of the subsurface stratigraphy.

Local faults in San Diego are known to exhibit vertical offset (vertical separation), possibly as a result of significant lateral movement. Within an exploratory trench, a discrete planar surface (i.e., a fault plane) can often be directly observed separating dissimilar geologic units. Older geologic units at depth should also exhibit increasing amounts of vertical offset, as the deeper units have accumulated greater relative movement as a result of repeated fault movement over geologic time. Borings and/or CPT soundings on opposite sides of a fault would be expected to penetrate dissimilar stratigraphy, which would be indicated by strata of varying composition and thickness. Laterally continuous subsurface layers in borings and/or CPT soundings would suggest significant faulting does not pass between the explorations.

The CPT soundings advanced for this project were located at a close spacing (typically less than 10 feet) to reduce the uncertainty of correlating layers from adjacent explorations. If present, multiple laterally continuous layers at increasing depths below the site would increase the confidence that faulting does not exist at the site.

4.1.3 Faulting Evaluation Conclusions

The borings and CPT soundings advanced for this investigation penetrate several stratigraphic units and packages that can be correlated in the subsurface to depths in excess of 80 feet bgs. These units appear to reflect an overall slight west to southwesterly dip component which is common given the geologic setting of much of the western San Diego area. The stratigraphic contacts, which mark the boundary from one material type to another, are generally laterally continuous, although depth variations and thicknesses change across the site were observed in some of the stratigraphic packages; however, this is typical for near shore marine deposits.

The site appears to be underlain by a repetitive sequence of westerly dipping units within the older paralic deposits that are unlikely to have been offset by a fault. It is anticipated that repeated fault movement would induce vertical stratigraphic separations. Geologic cross sections designated Section A-A’ and Section B-B’ (Figures 6 and 7) were oriented to intercept the anticipated northwest-southeast and northeast-southwest trend of potential faults within the site vicinity. If present, a through-going fault would be expected to exhibit anomalous offsets of the underlying stratigraphic units on one or both of the cross sections. Such subsurface anomalies were not observed and fault related offsets are not indicated within the overlapping site area covered by the two sections. Areas where unit depth and/or thickness variations are indicated over short lateral distances are more difficult to interpret, but are likely to be a result of depositional features such as scoured channels which are typical given the depositional environment of the underlying deposits rather than faulting. Given the absence of mapped active or potentially active faulting projecting through the site, and the evaluation of the site subsurface stratigraphy, it is our opinion that the potential for fault-related surface rupture at the site is low.

4.2 <u>Strong Ground Shaking</u>

The project site is situated within a seismically-active region and will likely experience moderate to severe ground shaking in response to a large magnitude earthquake occurring on a local or more distant active fault during the expected lifespan of the substation. As a result, seismically-induced ground shaking in response to an earthquake occurring on a nearby active fault, such as the RCFZ, or a regional fault, such as the Elsinore fault zone, is considered to be the major geologic hazard affecting the project. Site-specific seismic design recommendations are presented in Section 5.

4.3 Liquefaction Potential

Seismically-induced liquefaction is a phenomenon in which saturated soils lose a significant portion of their strength and acquire some mobility from seismic shaking or other large cyclic loading. The material types considered most susceptible to liquefaction are granular and low-plasticity fine grained soils which are saturated and loose to medium dense. A rapid increase in groundwater pressures (excess pore water pressures) causes the loss of soil strength.

Manifestations of soil liquefaction can include sand boils, surface settlements and tilting in level ground, lateral spreading, and global instability (flow slides) in areas of sloping ground. The impact of liquefaction on structures can include loss of bearing capacity, drag loads on deep foundations, liquefaction-induced total and differential settlement, and increased lateral and uplift pressures on buried structures.

A summary of the liquefaction potential evaluation methodology, evaluation results, and potential impacts to the project are presented in the following subsections; detailed information regarding the liquefaction evaluation is presented in Appendix E of this report.

4.3.1 Evaluation Methodology

The potential for soil liquefaction can be evaluated based on in-situ measurements of the soil resistance, including Standard Penetration Test (SPT) blow counts in borings, CPT data, and shear wave velocity data. Each form of soil resistance measurement has advantages and disadvantages for the evaluation of liquefaction potential. For the current study, a detailed evaluation of soil liquefaction potential was performed using the CPT data, considering the nearly continuous subsurface characterization and the potential to identify thin layers that would be undetectable with the spatial resolution of the SPT data and shear wave velocity measurements, and in general, the larger number of CPT soundings. Liquefaction potential was evaluated using the Youd et al. [2001] methodology as implemented in the computer program CLiq [GeoLogismiki, 2013].

The procedure for evaluating liquefaction potential was empirical and is based on data and observations at sites that have and have not liquefied during an earthquake. The capacity of the soil to resist liquefaction is expressed as the cyclic resistance ratio. The seismic demand is a function of the anticipated peak ground acceleration (PGA) at the site generated by the design earthquake and is termed the cyclic stress ratio.

Details of seismic evaluations, including development of PGA and design earthquake magnitude, and liquefaction potential evaluation, are provided in Appendix E of this report. The values of PGA (expressed as units of gravity [g]) and earthquake magnitude used in the liquefaction evaluation were 0.60g and **M**6.6, respectively. The evaluation also utilized the design high groundwater level of 20 feet bgs.

Materials were considered potentially liquefiable if the factor of safety against liquefaction, calculated as the cyclic resistance ratio divided by the cyclic stress ratio, was less than 1.0. Estimates of the liquefaction-induced settlements were calculated using the Zhang et al. [2002] methodology implemented in the computer program CLiq [GeoLogismiki, 2013].

4.3.2 Evaluation Results

The results of the liquefaction analyses indicate that relatively thin and non-continuous lenses of the old paralic deposits have the potential for liquefaction. These layers generally occur within an approximately 40-foot range between the design high groundwater level (20 feet bgs) and -20 feet NAVD 88 (about 60 feet bgs). These analyses also indicate that the liquefaction-induced settlements could range from a fraction of an inch to more than 2.3 inches, with an average value less than 1 inch.

Other factors such as soil mineralogy, void ratio, overconsolidation ratio, and age are contributing factors to liquefaction susceptibility. In general, the older or denser a deposit, the less susceptible it is to liquefaction. Saturated cohesionless sediments within marine terraces of Pleistocene age are reported to have a low likelihood of being susceptible to liquefaction [Idriss and Boulanger, 2008]. In addition, the shear wave velocity measurements from this site, by inspection, suggest a low likelihood of liquefaction. These general conclusions support the results of the more detailed evaluation described above.

4.3.3 Liquefaction Impacts

Liquefaction potential will impact the site design, and may impact the site after development. Liquefaction would most likely be manifested at this site as local ground subsidence, settlement, and localized reduction in shear strength at depth.

The potential for lateral spreading at the site is considered to be low as (i) potentially liquefiable soil lenses do not appear to be continuous across the site; (ii) the site is relatively level; (iii) the potentially liquefiable lenses are located at depth; and (iv) a free face and/or steep slopes are not present in the immediate site vicinity.

Ground improvement to remediate the potentially liquefiable lenses is not considered economically feasible due to the thin and non-continuous lenses of material identified as potentially liquefiable. The magnitude of liquefaction-induced settlement is anticipated to be relatively small (on the order of 1.5 inch); however, it should be recognized that total and differential settlements may cause damage to surface improvements and subsurface utilities under seismic conditions. Foundation design recommendations incorporating the potential impacts of liquefaction are presented in Sections 5.7 and 5.8 of this report.

4.4 Expansive Soil

Soils with some expansion potential are present in the near surface of the site. Based on the plasticity characteristics of the soils encountered (typically indicating silty sand, sandy lean clay, and sandy low plasticity silt) and the results of two expansion index tests performed as part of the current investigation (expansion index values of 31 and 51), the near surface soils are considered to have a low to medium potential for expansion.

4.5 Flooding

The Federal Emergency Management Agency (FEMA) presents the flood hazard potential in the vicinity of the site as part of their Flood Insurance Rate Maps. FEMA Map No. 06073C1885G, dated 16 May 2012 [FEMA, 2012], indicates that the subject area is located in an un-shaded Zone X which is defined as “areas determined to be outside the 0.2% annual change flood plain”. Additionally, due to a lack of any reservoirs up gradient from the site, flooding as a result of dam failure is not considered to be a viable hazard. Based on our review of the FEMA mapping, the geologic setting, and the site elevation, the potential for flooding at the site is very low.

4.6 Hydroconsolidation

Hydroconsolidation is the collapse and compaction of silty to sandy soil having a low bulk density that has been saturated for sustained periods and the water is subsequently removed. Given the location and subsurface conditions observed at the site, when combined with the engineering recommendation provided for site earthwork activities (Section 5.3), the potential for hydroconsolidation is considered to be very low.

4.7 Other Geologic Hazards

Other potential geologic hazards evaluated which could possibly affect the site include slope instability, floods, seiches, and tsunamis. The site is relatively flat, and new slopes, if proposed at the site, will be engineered slopes designed at stable inclinations. Therefore, slope instability is not considered a hazard. Tsunamis are seismically-induced waves generated by sudden movements of the ocean bottom during submarine earthquakes, landslides or volcanic activity. Seiches are similarly generated but are oscillating waves within bodies of water such as reservoirs, lakes or bays. The site is not located within the County of San Diego [County of San Diego Office of Emergency Services, 2009] mapped tsunami run-up zone. Similarly, potential seiche inundation would not likely exceed the extent of tsunami run up. Based on the physiographic setting of this site, the distance to the ocean or other large water bodies, and the elevation of the site, it is our opinion that the potential for flooding from seismically-induced seiches and tsunamis is very low.

5. CONCLUSIONS AND RECOMMENDATIONS

The conclusions and preliminary design recommendations presented herein for the design of the proposed Vine Substation are based on our current understanding of the proposed project, previous investigations by others, and results of our field investigation, laboratory testing, engineering and geologic analyses, and professional judgment.

5.1 Design Development

In our opinion, the site is suitable for the construction of the project, provided the recommendations of this report are incorporated into planning, preliminary design, detailed design, and construction. However, interaction will be required during design development between SDG&E and Geosyntec, particularly with respect to re-evaluation and refinement of remedial grading and foundation design recommendations to optimize the substation design. Information regarding foundation type, layout, preliminary size, and settlement tolerances can be used to more specifically evaluate soil resistance and settlement potential.

5.2 Design Groundwater Level

The project design should incorporate provisions to account for the effect of groundwater. Based on the groundwater levels observed at the time of drilling in the geotechnical borings (24 to 25 feet bgs), and considering the potential for groundwater rise due to seasonal variation or nearby irrigation, a high groundwater level corresponding to a depth of 20 feet bgs is recommended for design.

5.3 Earthwork

Site earthwork will generally consist of demolition of existing site features and pavement, removal of unsuitable (loose, porous, soft, or expansive) soils, site grading and fill placement to construct a relatively level substation pad, foundation excavations, and backfill of utility trenches. Engineered fill is defined as fill meeting the material, placement, and compaction recommendations presented in this report. Earthwork should be performed in accordance with SDG&E requirements, the recommendations of this report, the Standard Specifications for Public Works Construction “Greenbook,” and California Occupational Safety and Health Administration (Cal OSHA) safety requirements. A preconstruction conference should be held at the site with SDG&E, the contractor, civil engineer, and geotechnical engineer in attendance. Existing structures identified by SDG&E to remain should be protected in place during earthwork construction.

5.3.1 Site Clearing and Demolition

General debris, construction debris, and vegetative matter in the project area should be cleared and properly disposed of off-site. Existing infrastructure within areas to be improved should be properly demolished and disposed of off-site. Existing utilities should be properly terminated and the portion within the proposed development area removed completely.

A portion of the site was previously developed, reportedly as a gas station. The Benton [1974 and 1977] reports reference a retaining wall (planned to be cut off below grade) and the removal of an underground storage tank at the site. Based on the referenced information, the potential to encounter buried infrastructure such as foundations for walls or buildings, piping, tanks, etc. associated with the previous site development during construction of the proposed site improvements is considered high.

5.3.2 Remedial Grading and Site Preparation

Based on the previous and current borings, the site is underlain by undocumented fill and colluvium to depths up to approximately 3.5 to 9 feet typically, and locally in the central and north-central portion of the site (as reported in Benton Boring 2 and Test Pit 14) up to a depth of approximately 12.5 feet. These materials are conventionally considered unsuitable soils and removed and recompacted to provide uniform support of new fill and structures.

However, the recommended depth of overexcavation and recompaction should be based on anticipated geologic conditions and the proposed development in an area. Given the depths of undocumented fill and colluvium observed and reported in field explorations, it may be impractical to perform full depth remedial grading at this site. Since many of the proposed site structures will be supported on mat foundations, on deep foundations, could tolerate estimated settlements, less than full depth remedial grading should be acceptable.

For preliminary planning and design purposes, we recommend that a minimum of five feet of the undocumented fill and colluvium below existing or finish grade, whichever is lower, be overexcavated and recompacted (if suitable fill material) prior to substation development. Foundation design parameters and estimated settlements provided in subsequent sections of this report incorporate this remedial grading recommendation. The depth of remedial grading should be based upon SDG&E’s knowledge of the design criteria, foundation type, and settlement tolerances for the proposed structures and equipment. These recommendations for remedial grading should be reevaluated as part of design development.

In addition, areas to be overexcavated and recompacted should extend a minimum of five feet beyond the footprint of the foundations in each direction. If variable depths of remedial grading are planned, areas of deeper remedial grading should transition to areas of shallower remedial grading to reduce the potential contrast in stiffness between materials. Specific guidelines for such transitions should be determined in the field during grading operations by a representative of the geotechnical engineer.

Due to the geologic conditions and the location of the site within the RCFZ, geologic mapping within the grading limits should also be performed during site earthwork operations to document subsurface conditions. Geologic mapping includes observation and documentation of exposed geologic strata (and potential shears or faults, if encountered) prior to recompaction (fill placement) under the supervision of a certified engineering geologist.

Based on historic site information, a retaining wall reportedly occupied the northeast corner of the site adjacent to the gas station and was planned to be removed by cutting off the wall a few feet below grade. It is likely that concrete debris or foundation remnants from the retaining wall or other structures may be present in the fill at the site. If foundations or other concrete debris greater than 6 inches in maximum dimension is encountered, they should be removed and properly disposed off site. Depending on the depth encountered, removal of such foundations may require localized remedial grading deeper than 5 feet bgs to restore areas disturbed by foundation removal.

Loose or soft soil, or soil disturbed by demolition activities within the proposed grading area, as identified by the geotechnical consultant during grading and foundation excavation, should be excavated or scarified as required, moisture conditioned, and then recompacted before placing additional fill or preparing subgrade. Soil containing organic or other deleterious matter, if encountered, should be removed from the site and properly disposed. Areas to receive new fill, including areas of overexcavation and recompaction, should be proof rolled and moisture conditioned prior to compacting new fill.

5.3.3 Fill Materials

Based on limited observation and laboratory testing performed for this investigation, the on-site fill and formational materials should meet the engineering properties for Common Fill; these materials may not meet the expansion index criteria for Select Fill. The colluvium demonstrated high plasticity characteristics and is not anticipated to meet the requirements for select fill or common fill; however, additional fill suitability confirmation testing should be performed during subsequent geotechnical investigation during design development or during construction. Biodegradable, organic, or other compressible material should not be used for Common Fill or Select Fill. The following material types are applicable to the project:

Common Fill should consist of native or import soil and be granular soil (less than 50% passing the No. 200 sieve) that has a plasticity index less than 40 and does not contain quantities of oversize material that could make compaction difficult. Rocks or hard lumps less than 6 inches in maximum dimension may be used, provided the distribution of rocks or hard lumps is satisfactory to the geotechnical consultant.

Select Fill should consist of granular native or import soil that contains at least 40 percent of material, by dry weight, less than ¼ inch in size. Select fill should not contain rocks or hard lumps greater than 3 inches in maximum dimension. In addition, select fill should have an expansion index less than 30, a liquid limit less than 30, and a plasticity index less than or equal to 15.

Class 2 Aggregate Base should conform to the State of California, Department of Transportation (Caltrans) “Standard Specifications” Section 26-1.02B.

We recommend that if import soil is needed to achieve the design site grades, the import soil should be non-expansive in accordance with California Building Code (CBC) Section 1803A.5.3. These soils generally correlate to materials with an expansion index of 20 or less and which have a plasticity index of 15 or less.

5.3.4 Fill Placement and Compaction

Fill should be moisture conditioned and compacted between 0 and 3 percent above the optimum moisture contents in layers that do not exceed 8-inch loose lifts for heavy equipment compaction and 4-inch loose lifts for hand-held equipment compaction. Each lift of fill should be compacted to a minimum 90 percent relative compaction unless otherwise specified. Relative compaction is defined as the ratio (in percent) of the in-place dry density to the maximum dry density determined using the latest version of ASTM D1557 as the compaction standard. Fill placed should demonstrate a moisture content within 3 percent of optimum moisture content, also determined with ASTM D1557. Class 2 aggregate base should be compacted to a minimum relative compaction of 95 percent.

SDG&E typical substation requirements include a 3-foot thick substation pad. This pad structural section consists of a 12-inch thick section of select fill compacted to a minimum relative compaction of 90 percent, overlain by a 12-inch thick section of select fill compacted to a minimum relative compaction of 95 percent, overlain by a

12-inch thick section of Class 2 aggregate base compacted to a minimum relative compaction of 95 percent.

5.3.5 Subdrains

Due to the proposed development grades and existing site topography, subdrains are not anticipated as part of the project.

5.3.6 Bulking and Shrinkage

The fill and colluvium may shrink in volume, and the old paralic deposits may bulk in volume when excavated and recompacted in accordance with the recommendations presented in this report. We anticipate that the range of material shrinkage and bulking is on the order of 5 to 10 percent.

5.4 Surface Drainage

Surface drainage should be planned to prevent ponding and promote the drainage of surface water away from structure foundations, slabs, edges of pavements and sidewalks, and towards suitable collection and discharge facilities. Paved and aggregate-surfaced areas should be sloped to drain water away from structures and pavements at a minimum gradient of 1 percent, and unpaved areas should be finish graded with a minimum slope of 2 percent away from structures and pavements. Stormwater collected by roof drainage systems should be discharged at suitable locations away from the structures to reduce the possibility of saturation of foundation soil. Even when these measures are taken, experience has shown that a shallow groundwater or surface-water condition can develop in areas where no such water condition existed before site development.

5.5 CBC Seismic Design Parameters

Seismic design parameters were developed in accordance with the 2010 and the 2013 California Building Code (CBC). The approximate geometric center of the site (latitude and longitude of 32.739 degrees and -117.179 degrees, respectively) was used to evaluate the minimum seismic design parameters presented in Tables 3a and 3b. The structural designer may utilize more conservative values at their discretion.

5.6 Seismic Qualification Level

Based on the Institute of Electrical and Electronics Engineers (IEEE) Standard 693 [IEEE, 2005], “IEEE Recommended Practice for Seismic Design of Substations,” we recommend the design of substation equipment use a seismic qualification level of “high”. A calculation evaluating the seismic qualification level is presented in Table 4.

5.7 Shallow Foundations

Substation features including control shelters, transformers, and screen walls may be founded on shallow foundations bearing on engineered fill. Shallow foundations may include spread footings, continuous perimeter footings, and mat foundations.

5.7.1 Footing Dimensions and Embedment

The minimum recommended shallow foundation embedment depth is 18 inches below finished grade for spread or continuous foundations or 12 inches below finished grade for larger mat foundations. The minimum recommended shallow foundation width is 18 inches. The structural designer should determine the footing embedment, size, and reinforcement based on anticipated loads and estimated settlements. Adjacent footings founded at different elevations should be located such that the slope from bearing level to bearing level is flatter than 1:1 (horizontal:vertical).

Structures and equipment foundations should not bear on different soil strata (i.e., engineered fill and old paralic deposits). If differing bearing conditions are encountered, the soil should be excavated and recompacted to a depth of at least 3 feet below the bottom of the structure foundation within the perimeter of the structure and at least 5 feet horizontally beyond the structure perimeter.

5.7.2 Allowable Foundation Pressure

Shallow foundations consisting of spread footings, continuous footings, or mat foundations bearing on engineered fill may be designed for an allowable bearing pressure of 2,000 pounds per square foot (psf). The allowable bearing pressure may be increased by 1,000 psf for each additional foot of depth and 500 psf for each additional foot of width beyond the minimum specified foundation dimensions, up to a maximum bearing pressure of 4,000 psf. Allowable bearing pressures may be increased by one-third for short-term wind and seismic loading.

5.7.3 Allowable Lateral Bearing

Resistance to lateral loads on shallow foundations may be provided by passive resistance along the outside face of footings and frictional resistance along the bottom of footings. The allowable passive resistance may be taken as equivalent to a fluid weighing 250 pounds per cubic foot (pcf) for footings poured neat against engineered fill.

An allowable friction coefficient of 0.35 may be used with the dead load to compute the frictional resistance of footings. If frictional and passive resistances are combined, the allowable friction coefficient should be reduced to 0.25.

The upper 12 inches of soil should be neglected in passive pressure calculations in areas where there will be no hardscape that extends from the outside edge of the footing to a horizontal distance equal to three times the footing depth. The resistance from passive pressure should also be neglected where utilities or similar excavations may occur in the future.

5.7.4 Settlement

The settlement of a shallow foundation for a given allowable bearing pressure depends on the size, shape, and embedment depth of the foundation, the relative compaction and stiffness of the engineered fill, and the saturation and density of the soil materials below.

Total settlement from structural loads (excluding seismically-induced settlement) based on the remedial grading recommendations and maximum recommended allowable bearing pressures are summarized below. Differential settlements between adjacent footings are expected to be approximately half the estimated total settlements. The majority of settlement due to structural loads should occur during or shortly after construction.

Shallow Foundation Condition	**Estimated Total Settlement (inches)**
Continuous foundations less than three feet in width up to a maximum bearing pressure of 4,000 psf	1
Isolated spread or mat foundations less than 25 feet in width up to a maximum bearing pressure of 3,000 psf	1
Isolated spread or mat foundations less than 25 feet in width up to a maximum bearing pressure of 4,000 psf	1.5

Shallow foundations may also be subject to potential liquefaction-induced settlement on the order of 0 to 2.3 inches, with an average of less than 1 inch at the exploration locations evaluated as summarized in Section 4.3.2 and detailed in Appendix E of this report.

5.7.5 Modulus of Subgrade Reaction

Deflections of mat foundations may also be estimated using the subgrade reaction (beam on elastic foundation) method of analysis. We recommend a modulus of subgrade reaction of 200 pounds per cubic inch (pci) for engineered fill.

5.8 Deep Foundations

5.8.1 General

We anticipate that Cast-In-Drilled-Hole (CIDH) deep foundations, also referred to as drilled piers, will be used to support structures such as racks, firewalls, terminal arrestors, and/or other structures with high lateral loads. Preliminary structural foundation design information (Section 1.1) was provided by SDG&E to support development of the preliminary geotechnical foundation design recommendations presented in this section and in Appendix F. We understand that the final foundation sizes, both diameter and depth, will be determined by SDG&E, and will depend on the foundation recommendations provided in this report, the design loads, and construction considerations. We recommend that deep foundations bear within the Old Paralic Deposits. Based on the preliminary design information provided by SDG&E, we anticipate that deep foundations will bear above the design groundwater level.

We understand that the deep foundations at the site may be designed for lateral and axial loading using computer programs such as the Electric Power Research Institute (EPRI) Moment Foundation Analysis and Design (MFAD) or Compression/Uplift Foundation Analysis and Design (CUFAD) programs, or the Ensoft program LPILE, or other spreadsheet or manual calculation methods. Regardless of the design methodology, there are several important considerations for deep foundation design at this site, including soil stratigraphy and design parameters, groundwater level, potential for surficial erosion, and the impacts of potential soil liquefaction.

We recommend the following approach to deep foundation design: 1) SDG&E confirms the preferred extent of remedial grading, equipment layout, foundation type, loads, and size based on the preliminary geotechnical recommendations in this report; then 2) as needed, Geosyntec reevaluates or confirms the deep foundation design recommendations and estimated settlement, analyzing specific foundations identified by SDG&E with respect to potentially liquefiable layers, subsurface modeling conditions, and group effects. Lateral capacity, axial capacity, or settlement may control the design of deep foundations.

Simplified and preliminary subsurface stratigraphy and soil parameters for deep foundation design incorporating the remedial grading recommendations of this report are presented in the following sections. A discussion of the groundwater conditions observed and the design groundwater level are presented in Sections 3.5 and Section 5.2 of this report, respectively. Deep foundation design should incorporate the effects of groundwater by modeling the recommended design groundwater level or buoyant soil unit weights, as appropriate with the foundation length and design methodology utilized. Within a developed SDG&E substation, the potential for erosion is considered low due to the site drainage and surfacing improvements. Additionally, remedial grading is planned. Therefore, due to the anticipated design and construction procedures, no discount depth of surficial materials is recommended for deep foundation design. The potential impact of liquefaction on deep foundation design will require careful consideration at the site as presented in the following section.

5.8.2 Impacts of Liquefaction

Deep foundation design should also consider the potential effects of liquefaction, including drag loads and downdrag settlement. As previously noted, ground improvement to remediate the potentially liquefiable lenses is not considered economically feasible due to the thin and non-continuous lenses of material identified as potentially liquefiable at the site.

Liquefaction-induced drag loads should be incorporated into the axial design of deep foundations. Drag load refers to the downward-acting force transferred to a deep foundation by surrounding soil that undergoes settlement. Load transfer is by shearing stress that develops at the soil-foundation interface. Drag loads occur in response to relative downward deformation of the surrounding soil to that of the foundation. The magnitude of relative movement required to develop full side resistance, and therefore full drag load, is reportedly 0.4 to 0.5 inches. However, it is prudent to assume that full drag loads will occur if any relative downward movement of soil is anticipated. Full drag loads for deep foundations are presented in Appendix F.

Foundation design for lateral loading is typically governed by the subsurface conditions within the upper portion of the soil profile. Since the depth to design groundwater and thus the shallowest potential location of liquefiable lenses is 20 feet bgsliquefied soil layers are not incorporated in the model for the lateral design of deep foundations. However, deep foundations should bear at least 1.5 foundation diameters above the shallowest location of potentially liquefiable lenses (20 feet bgs). The axial soil resistance charts presented in Appendix F terminate above potentially liquefiable layers. The influence of potentially liquefiable soil lenses on axial and lateral design and on estimated deep foundation settlement should be further evaluated during design development.

5.8.3 MFAD and CUFAD Design Parameters

The design soil parameters utilized in the MFAD and CUFAD computer programs include: subsurface stratigraphy; total unit weight; shear strength parameters; pressuremeter modulus; and various factors related to the soil stress conditions and soil/foundation interface conditions.

Estimates of these parameters were developed based on review of previous geotechnical information, site reconnaissance, field explorations, geotechnical laboratory testing, engineering analyses, empirical correlations, literature research, and professional judgment. Pressuremeter testing was not performed as part of this project. The recommended MFAD and CUFAD design parameters are presented in Table 5 and Table 6, respectively. These design parameters are intended for use in foundation design and may not reflect actual strengths. The ratio of operative to in-situ horizontal stress for CUFAD analysis is based on the specific construction method of the drilled shafts, and should be selected by SDG&E.

5.8.4 LPILE Design Parameters

We understand that SDG&E may perform deep foundation lateral pile analysis and design using the computer program LPILE. The design soil parameters used in lateral pile analysis using LPILE include: subsurface stratigraphy; soil type and p-y curve; effective unit weight; shear strength parameters; and modulus. The estimated LPILE soil design parameters are presented in Table 7. The results of preliminary LPILE modeling are presented in Appendix F.

5.8.5 Axial Resistance and Settlement

Deep foundation axial resistance, potential drag loads, and settlement are dependent on the size of the foundation, loading conditions, and the stratigraphy on the sides and base of the foundation (particularly with respect to the position of the potentially liquefiable layers).

The depth range estimated to have the potential for lenses of soil liquefaction is between the design groundwater level (20 feet bgs) and about 60 feet bgs. Currently, the deep foundations are anticipated to be founded above the design groundwater level, but some relative displacement is anticipated. Relative displacement between the surrounding soil and deep foundations can induce drag loads and downdrag settlement, which is the downward movement of the pile due to settlement of the surrounding ground.

Preliminary axial design information, including downward soil resistance, drag load, and uplift soil resistance for a range of foundation sizes is presented in Appendix F. The recommended drag load for a foundation should be added to the structural loads for foundation design.

Settlements for single foundations induced by the preliminary loads presented in Section 1.1 and by potential drag loads are anticipated to be less than 0.5 inches. This settlement does not include group effects or the potential liquefaction-induced settlement that may occur in soil layers below the foundations (See Section 4.3.2).

5.8.6 Group Effects

Construction of deep foundations in groups can reduce the available axial capacity of drilled piers due to the relaxation of the soil within the adjacent foundation excavations. Deep foundation groups can also demonstrate lower lateral capacity due to overlapping loads from adjacent piles within a group. Deep foundation groups can also demonstrate increased settlement due to the deeper zone of influence for the group than that of a single foundation.

We recommend a minimum center-to-center spacing of 2.5 foundation diameters for deep foundations. However, piers spaced closer than four foundation diameters (center to center) can have a total axial (downward and uplift) capacity less than the sum of the capacities of the individual piers. For preliminary design, we recommend a group efficiency factor for axial design of 0.65 and 1.0 for center-to-center spacing of 2.5 diameters and 4.0 diameters or more, respectively. Axial resistance group efficiency factors for intermediate spacing can be determined by linear interpolation between the noted values.

Group efficiencies for lateral design of deep foundations are presented in Table 8 as recommended for predominantly granular soils and for an individual foundation’s location within a group. The lateral group efficiency can be incorporated by magnifying the loads on the piers by the reciprocal of the efficiency.

Additional geotechnical evaluation should be performed during design development to check the group foundation capacity evaluation and settlement.

5.9 Retaining Walls

The site is relatively level; however, recommendations for conventional retaining walls are provided in the event that retaining walls are utilized for grade separations within the substation. Lateral earth pressures on retaining walls depend upon the type of wall, type of backfill material, and allowable wall movements.

Retaining walls should be backfilled with free draining granular material within a zone defined by a 1:1 (horizontal to vertical) slope up and away from the bottom of the foundation. Free draining granular material is not present in substantial quantities at the site and would require import to the site. Lateral loads on retaining wall foundations can be resisted by passive resistance and frictional resistance for soils adjacent to the foundations as outlined in Section 5.7.3 of this report.

Active lateral earth pressure conditions are applicable for walls which are not fixed at the top and where approximately ¼ inch of movement at the top of the wall per 5 feet of wall height is acceptable. An equivalent fluid pressure of 37 pcf may be used for the design of retaining walls for active earth pressure conditions. This recommended active earth pressure assumes a horizontal backfill surface, free-draining backfill, and does not include surcharge loads. For surcharge loads against the backfilled side of the wall, the resulting lateral load should be calculated based on a uniform lateral pressure equal to 0.5 times the vertical surcharge pressure acting on the backfilled side of the wall, applied over the full height of the wall in addition to the equivalent fluid pressure for walls up to 10 feet high.

5.10 Concrete Slabs and Hardscape

Concrete slabs and hardscape should be supported on select fill with low expansion potential, which may not be present in substantial quantities at the site. A modulus of subgrade reaction of 200 pci can be assumed for design of slabs and hardscape. The subgrade should be proof rolled prior to placing the concrete slabs and hardscape. The slab thickness and steel reinforcement should be designed by a California-registered civil engineer for the anticipated loads. Slab-on-grade concrete floors should have a minimum thickness of 4 inches.

We recommend that isolation joints be provided where slabs abut walls or columns. Isolation joints should be designed to separate the floor from the abutting element, to allow each part to move independently. Crack control or expansion/contraction joints should be provided at spacing appropriate for the slab thickness and the maximum concrete aggregate size, but should be provided at regular intervals not exceeding approximately 15 feet, each way.

Concrete slabs should be underlain by a minimum of 4 inches of clean (less than 5 percent passing the No. 200 sieve), coarse sand. Special care should be taken by the contractor so that a uniform thickness of sand is maintained to achieve uniformity in the concrete slab thickness. We recommend that the subgrade be wetted prior to placement of the clean, coarse sand beneath the slab. In lieu of the recommended clean, coarse sand, two-sack sand-cement slurry may be used in accordance with SDG&E design practices.

5.11 Utility Trenches

We understand that SDG&E has standards for utility trenches and trench backfill within their substations and for all electric power and distribution trenches outside of the substation consisting of slurry backfill. The following recommendations are meant to be applicable to those utility trenches not covered by the SDG&E standards.

Utilities should be placed above and outside the envelope defined by 2:1 (horizontal to vertical) lines drawn outward and down from the bottom edge of foundations. Trench backfill is defined as material placed in a trench starting 6 inches above the pipe, and bedding is all material placed in a trench below the backfill. Pipe trench backfill should conform to the recommendations presented in this report and Section 306-1.3 of the "Greenbook." Unless concrete bedding is required around utility pipes, free-draining clean sand should be used as bedding. Pavement and subgrade requirements provided in Section 5.10 should be incorporated for trench backfill. Compaction of backfill by water jetting should not be permitted.

5.12 Pavements

We recommend that the paved access roads within the site be designed for a traffic index selected by the project civil engineer. We understand that SDG&E typically utilizes a traffic index of 5.0 for flexible pavement design for substation access roads.

The flexible pavement section should consist of asphalt concrete (as defined in Section 39 of the Caltrans Standard Specifications) over Class 2 aggregate base (as defined in Section 26 of the Caltrans Standard Specifications) over properly prepared subgrade. Properly prepared pavement subgrade consists of the uppermost 12 inches of subgrade that is moisture conditioned and compacted to a minimum relative compaction of 95 percent. Asphalt and aggregate base should be compacted to a minimum relative compaction of 95 percent.

The actual pavement section may be chosen based on the cost for asphalt versus Class 2 aggregate base. We understand that the typical pavement section within an SDG&E substation is 4 inches of asphalt over 8 inches of Class 2 aggregate base; this structural section meets the criteria for the minimum recommended pavement section assuming a minimum resistance-value for pavement design (R-value) of 5 and a traffic index of 5.0. R-value testing should be performed during construction on samples representative of subgrade conditions to confirm the design assumptions.

Alternatively, portland cement concrete (PCC) can be used for pavements where heavy truck traffic is anticipated. A minimum of 7.5 inches of PCC should be used over properly prepared subgrade. We also recommend that concrete pavements be provided with expansion joints at regular intervals not exceeding 15 feet in each direction.

5.13 Corrosion Potential

The results of the corrosion testing performed for the current investigation are presented in Appendix D of this report. Corrosion testing was performed on two soil samples from this investigation. The results of the tests indicate the water soluble sulfate content and chloride content of the soil were 340 and 310 parts per million (ppm) and 160 and 1,110 ppm, respectively. The results of resistivity testing indicated minimum resistivity values of 1,215 and 265 ohms-centimeters (ohm-cm).

Sulfate contents in this range are generally considered to be negligible with respect to potential for sulfate attack of concrete in accordance with Table 4.2.1 of the 2011 American Concrete Institute Manual. Chloride contents in this range are generally considered to be negligible to high with respect to potential for chloride attack. Minimum resistivity values less than 500 ohm-cm are considered to be very corrosive, and a resistivity value of 1,215 is considered to be fairly corrosive. Metallic utility piping and conduits should be designed for a corrosive environment, and measures to enhance chloride resistance of concrete should be considered. An engineer specializing in corrosion resistance should be consulted if additional information is needed.

5.14 Low Impact Development and Hydromodification

The San Diego Regional Water Quality Control Board Order No. R9-20013-0001 was adopted on 8 May 2013 and became effective on 27 June 2013. This permit regulates discharges of urban runoff from municipal separate storm sewers (MS4s) in the San Diego Region. The MS4 permit requires that new developments and redevelopments implement source control, Low Impact Development (LID), treatment, and hydromodification management BMPs, depending on the type of development. The provisions of the new permit do not go into full effect until 27 June 2015, and until then, the provisions of the previous permit (Order No. R9-2007-0001) govern. The requirements of Order No. R9-2007-0001 and the additional requirements under Order No. R9-2013-0001 are discussed below.

5.14.1 LID and Treatment Control Requirements

Provision D.1.d of Order No. R9-2007-0001 requires each Co-permittee to implement a Standard Urban Storm Water Mitigation Plan designed to control the discharge of pollutants in storm water from the MS4 to the maximum extent practicable. Redevelopment projects that install and/or replace more than 5,000 square feet of impervious area are considered a Priority Development Project under the current City of San Diego Storm Water Standards Manual. The following LID BMPs are required for Priority Development Projects.

Site Design BMP Requirements:

- Drain impervious areas to pervious areas according to the infiltration ability of the pervious area prior to discharge from site;
- Landscape or pervious areas must be properly designed and constructed to function as areas for infiltration of storm water;
- Projects having areas with low traffic and appropriate soil conditions shall construct a portion of the area with permeable surfaces;

  and where applicable and feasible:
- Conserve natural areas, including existing trees, other vegetation, and soil;
- Construct streets, sidewalks, or parking lot aisles to the minimum widths necessary, provided that public safety and a walkable environment for pedestrians are not compromised;
- Minimize the impervious footprint of the project;
- Minimize soil compaction; and
- Minimize disturbances to natural drainages (e.g., natural swales, topographic depressions, etc.).

Source Control BMP Requirements:

- Minimize storm water pollutants of concern in urban runoff;
- Include storm drain system stenciling or signage;
- Include properly designed outdoor material storage areas;
- Include properly designed trash storage areas;
- Include efficient irrigation systems; and
- Include water quality requirements applicable to individual priority project categories.

Treatment Control BMP Requirements:

- Select treatment control BMPs to infiltrate, filter, or treat the volume of runoff produced from the runoff from the 85th percentile, 24-hour design storm.

5.14.2 Hydromodification Control Requirements

Priority Development Projects are also required to implement hydromodification control BMPs according to the Hydromodification Management Plan [County of San Diego, 2011]. However, if the project installs/replaces less than 5,000 square feet of impervious area, then the project would be considered a Standard Development Project and the following requirement would not apply.

Section D.1.g of Order No. R9-2007-0001 specifies that for applicable projects “post-project runoff flow rates and durations shall not exceed pre-project runoff flow rates and durations where the increased discharge flow rates and durations will result in increased potential for erosion or other significant adverse impacts to beneficial uses, attributable to changes in flow rates and durations.” The order further establishes hydromodification control criteria as follows:

> *Priority Development Projects shall implement the following criteria by comparing the pre-development (naturally occurring) and post-project flow rates and durations using a continuous simulation hydrologic model.*
>
> *a) For flow rates from 10 percent of the 2-year storm event to the 10-year storm event, the post-project peak flows shall not exceed predevelopment (naturally occurring) peak flows. Less restrictive standards are possible for more erosion-resistant receiving channel sections if the results from the Southern California Coastal Water Research Project channel screening indicated either a Medium or Low susceptibility to channel erosion.*
>
> *b) For flow rates from the 5-year storm event to the 10-year storm event the post-project peak flows may exceed pre-development (naturally occurring) flows by up to 10 percent for a 1-year frequency interval.*

The MS4 permit also contains language to support exemptions for projects located in highly urbanized areas. This project discharges directly to a hardened conveyance and may qualify for potential exemptions from hydromodification criteria. To qualify for this exemption, the existing hardened conveyance system must continue uninterrupted to the exempt system. Additionally, the project proponent must demonstrate that the hardened or rehabilitated conveyance system has capacity to convey the 10-year ultimate condition flow through the conveyance system.

5.14.3 Order No. R9-2013-0001 Requirements

Order No. R9-2013-0001 includes a new volume capture criteria which will go into effect in June 2015. This will require Priority Development Projects to implement treatment control BMPs designed to retain (i.e., intercept, store, infiltrate, evaporate, and evapotranspire) onsite the pollutants contained in the design capture volume (DCV). The DCV is equivalent to the volume of storm water produced from a 24-hour 85th percentile storm event. Or, at the discretion of the municipality, the project can incorporate:

- Properly sized biofiltration BMPs if it can demonstrate technical infeasibility of full DCV retention; or
- Properly sized flow-thru treatment control BMPs and offsite mitigation for the portion of the DCV not reliably retained onsite.

The above changes would increase the volume of storm water that would be required to be retained on-site and thus increase the footprint required to address long-term storm water treatment requirements for the project.

5.14.4 LID and Hydromodification Considerations

BMP selection and design is a function of site conditions, especially permeability rates of the underlying soil. The site is located within the hydrologic soil group designated “Undetermined” per the County of San Diego Hydrology Manual [County of San Diego, 2003]. However, a conservative approach is to assume “Group D” soils, which are characterized by a very slow infiltration rate when thoroughly wetted; chiefly clays that have a high shrink-swell potential; soils that have a high permanent water table; soils that have a clay layer at or near the surface; or soils that are shallow over nearly impervious material [County of San Diego, 2003]. The near-surface site materials are considered to have a very low infiltration rate due to the relatively high percentage of silt and clay sized particles.

The near surface site soils are assumed to have a low vertical and horizontal infiltration rate and a slow rate of water transmission. Due to the assumed low hydraulic conductivity of the near surface soils, the role of infiltration in LID and hydromodification control is limited. More specific recommendations can be provided by Geosyntec, if design development indicates that such BMP features are needed (i.e., storage basin design).

5.15 Additional Geotechnical Investigation

We recommend that additional geotechnical investigation, including subsurface explorations, geophysical surveys, laboratory testing, and engineering and geologic evaluations, is performed prior to final design. Noninvasive geophysical survey exploration methods could potentially delineate underground storage tanks, former tank pits, retaining wall foundations, and utilities or pipelines that may exist in the shallow subsurface, providing information to optimize environmental site assessment and/or support project planning and earthworks construction.

This additional geotechnical investigation should be performed during design development and include additional explorations (potentially including geophysical surveys) to further evaluate subsurface conditions in the specific areas of proposed structures and revisit the preliminary recommendations of this report for completeness and applicability with new information obtained. We recommend installation of a temporary piezometer and groundwater level observation until stabilized in at least one boring. The supplemental investigation should also include additional geotechnical laboratory testing on near-surface materials across the site to further evaluate suitability for re-use as fill.

Geophysical surveys would require closing the site (or portions of the site) to vehicular parking. Multiple geophysical survey methods should be utilized, including electro-magnetic, ground penetrating radar, and liner tracer methodologies, because each instrument senses different properties of the soil and potential buried objects. Electro-magnetic methods highlight buried metallic objects such as pipelines and underground storage tanks. Ground penetrating radar is the primary method applied for the detection of backfilled excavations, trenches, and nonmetallic pipelines and utilities and is also used for detailing anomalies detected with other geophysical instruments. Passive and active line tracer methods delineate pipelines, and active electrical and communication lines.

6. CONSTRUCTION CONSIDERATIONS

6.1 Reuse of Existing Fill Soils and Colluvium

The silty fine sand to sandy silt fill and colluvium is estimated to have a low to medium potential for expansion, and is anticipated to predominantly meet the geotechnical requirements for Common Fill. Two tests performed for this investigation indicated expansion indices of 31 and 51 which would not meet one criterion for Select Fill of an expansion index less than 30. Import of Select Fill materials and/or blending of native material with imported material to meet the Select Fill criteria may be needed.

The existing fill soils and colluvial materials excavated and intended to be reused as engineered fill should also be screened for the presence of contamination and potential for reuse in accordance with SDG&E guidelines. No PID readings or visual or olfactory observations of potential contamination were observed in soil samples and cuttings from the geotechnical borings advanced by Geosyntec. However, this does not preclude the possibility that impacted soil or groundwater is present at the site. The site reportedly was occupied by a gas station and is in an older, developed area of San Diego, with multiple gas stations and vehicle repair/rental facilities in relatively close proximity that can be potential sources of impacts.

During remedial grading, we recommend maintaining separate stockpiles for materials potentially meeting and not meeting the various fill criteria presented in Section 5.3.3 (specifically expansion index) and for potentially impacted soils, if needed.

6.2 Previous Site Development

The site was reportedly occupied by a gas station, with underground storage tank/s and a perimeter retaining wall. As discussed in Section 5.3.1 and Section 5.3.2 of this report, remnants from this development are likely present within the fill, including concrete debris, foundations, utility lines, or other construction debris. As described in Section 3.4.1, minor concrete debris was encountered in the vicinity of CPT C-18 and reported in Benton Test Pit 14.

6.3 Excavation Conditions

The hollo-stem auger borings performed for this investigation were advanced with no unusual difficulty to the target depths up to 51.5 bgs. The CPT soundings met refusal conditions in a few localized instances at depths less than five feet bgs and in a few instances at depths greater than 40 feet bgs. We anticipate that deep foundations can be excavated with easy to moderate effort with conventional heavy-duty drilling equipment.

The borings and CPT soundings performed for this investigation that extended below the groundwater level generally caved upon withdrawal of the hollow-stem augers or the CPT rods. Excavations that extend below the groundwater level (measured during drilling at elevations between +11 feet NAVD 88 and +19 feet NAVD 88) should use casing or other means of excavation stabilization approved by SDG&E.

6.4 Temporary Slopes

The design and excavation of temporary slopes and their maintenance during construction is the responsibility of the contractor. The contractor should have their geotechnical or geological professional evaluate the soil conditions encountered during excavation to determine permissible temporary slope inclinations and other measures required by Cal OSHA. For planning purposes, based on the materials observed in the borings, the design of temporary slopes for planning purposes may assume Type C conditions. Existing infrastructure within a 2:1 (horizontal:vertical) line projected up from the toe of temporary slopes should be monitored for potential movement during construction.

6.5 Construction Observation and Testing

During construction, R-value testing should be performed on subgrade materials below proposed asphalt-paved areas to ensure that the actual subgrade R-value exceeds that assumed for design. Laboratory compaction tests, Atterberg limits tests, and expansion index tests are also recommended during construction to evaluate fill material suitability and compaction requirements. Soil analytical testing may also be required if impacted soils are suspected.

Variations in subsurface conditions will likely be encountered during construction at the site. To permit correlation between the investigation data, design, and the conditions encountered during construction, and to provide conformance with the plans and specifications as originally contemplated, we recommend that Geosyntec be retained to provide continuous observations of earthwork construction operations, including geologic observation and mapping of remedial grading excavations, and to provide quality control testing of soil fill and backfill placement and compaction.

7. LIMITATIONS

The geotechnical investigation for this project observed only a small portion of the pertinent subsurface conditions. The recommendations made herein are based on the assumption that soil conditions do not deviate appreciably from those found during the current field investigation and the referenced previous investigations by others. This geotechnical investigation report has been prepared in accordance with current practices and the standard of care exercised by scientists and engineers performing similar tasks in this area. The conclusions contained in this report are based solely on the analysis of the conditions observed by Geosyntec personnel. We cannot make any assurances concerning the completeness of the data presented to us. Environmental characterization of soil and groundwater was beyond the scope of this investigation.

No warranty, expressed or implied, is made regarding the professional opinions expressed in this report. Site grading and earthwork, subgrade preparation under concrete slabs and paved areas, utility trench backfill, and foundation excavations should be observed by a qualified engineer or geologist to verify that the site conditions are as anticipated. If actual conditions are found to differ from those described in the report, or if new information regarding the site is obtained, Geosyntec should be notified and additional recommendations, if required, will be provided. Geosyntec is not liable for any use of the information contained in this report by persons other than SDG&E or their subconsultants, or the use of information in this report for any purposes other than referenced in this report without the expressed, written consent of Geosyntec.

California, including San Diego County, is an area of high seismic risk. It is generally considered economically unfeasible to design structures to resist earthquake loadings without damage. Proposed structures designed in accordance with the recommendations presented in this report could experience limited distress/damage if subjected to strong earthquake shaking.

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TABLES

Table 1. Summary of Current Explorations
Vine Substation

Exploration No.	Ground Surface Elevation (feet)	Exploration Depth (feet)	Reached Target Depth	CPT Refusal Due to High Tip Stress	CPT Refusal Due to High Sleeve Stress	CPT Shear Wave Velocity Measured
B-1	44.0	41.5	X			
B-2	36.0	51.5	X			
B-3	43.0	51.5	X			
B-4	38.0	41.5	X			
B-5	40.0	41.5	X			
C-1	37.0	80.1	X		X	X
C-2	37.5	74.3	X			
C-3	37.8	75.1	X			
C-4	38.3	72.7	X			
C-5	38.5	69.6	X			
C-6	38.8	65.9	X			
C-7	39.3	63.7		X		
C-8	39.8	61.2		X	X	X
C-9	40.3	48.2		X		
C-10	40.5	93.5	X	X	X	
C-11	41.0	88.8	X		X	
C-12B	41.7	44.8		X	X	
C-12	41.5	45.1		X		
C-13	41.8	43.3		X		
C-13A	42.0	80.3	X			
C-14	42.3	90.3	X		X	
C-15	42.5	80.3	X	X	X	X
C-16	39.5	90.4	X			X
C-17	39.0	100.4	X			
C-18	38.5	4.0		X		
C-18A	38.5	3.1		X		
C-18C	38.8	80.4	X			
C-19	38.0	80.1	X			
C-20	37.5	58.1		X		
C-21	37.0	80.3	X			
C-22	36.5	100.3	X			X

Table 2. Nearby Faults
Vine Substation

Fault Name	Distance and Direction from Site [a]	Maximum Moment Magnitude [b]
Rose Canyon	0.1 miles (0.20 km) to northeast 0.7 miles (1.1 km) to west	7.2
Coronado Bank	13.7 miles (22 km) to west	7.6
Elsinore (Julian Segment)	41.3 miles (66 km) to northeast	7.1
San Jacinto (Coyote Creek Segment)	62.6 miles (100 km) to northeast	6.8

Notes:

a. Distances from site noted are the closest distance to the surface trace or inferred projection of the fault as measured from the United States Geologic Survey Quaternary fault database [USGS & CGS, 2010].

b. Maximum moment magnitude values reported by California Geological Survey OFR 96-08 Appendix A, revised 2002 [Petersen et al., 1996].

Table 3a. 2010 CBC Seismic Parameters
Vine Substation

Parameter	Value
Site Soil Class	C
Mapped Spectral Response Acceleration at 0.2s Period, S_S	1.60 g
Mapped Spectral Response Acceleration at 1.0s Period, S_1	0.63 g
Short Period Site Coefficient at 0.2s Period, F_a	1.00
Long Period Site Coefficient at 1.0s Period, F_v	1.30
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.60 g
Adjusted Spectral Response Acceleration at 1.0s Period, S_{M1}	0.81 g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.06 g
Design Spectral Response Acceleration at 1.0s Period, S_{D1}	0.54 g

Notes:

a. Parameters based on ASCE 7-05 Standard and the 2009 International Building Code which use the 2008 USGS hazard data.

Table 3b. 2013 CBC Seismic Parameters
Vine Substation

Parameter	Value
Site Soil Class	C
Mapped Spectral Response Acceleration at 0.2s Period, S_S	1.23 g
Mapped Spectral Response Acceleration at 1.0s Period, S_1	0.48 g
Short Period Site Coefficient at 0.2s Period, F_a	1.00
Long Period Site Coefficient at 1.0s Period, F_v	1.32
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.23 g
Adjusted Spectral Response Acceleration at 1.0s Period, S_{M1}	0.63 g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	0.82 g
Design Spectral Response Acceleration at 1.0s Period, S_{D1}	0.42 g

Notes:

b. Parameters based on ASCE 7-10 Standard and the 2012 International Building Code which use the 2008 USGS hazard data.

Table 4. IEEE 693 Seismic Qualification Level
Vine Substation

Parameter	2009 IBC Values	2012 IBC Values
Site Soil Class	C	C
Maximum Considered Earthquake Ground Motion 0.2s Spectral Response Acceleration S_s	1.60 g	1.23 g
Short Period Site Coefficient at 0.2s Period, F_a	1.00	1.00
Adjusted Spectral Response Acceleration at 0.2s Period, S_{ms} (S_s*F_a)	1.60 g	1.23 g
Peak Ground Acceleration for seismic qualification selection (S_{ms}/2.5)	0.64 g	0.49 g
Recommended IEEE 693 Seismic Qualification Level	High	High

Table 5. MFAD Design Parameters
Vine Substation

Material Type	Depth bgs [a] (feet)	Total Unit Weight (pcf)	Effective Friction Angle (degrees)	Effective Cohesion (psf)	Pressuremeter Modulus, E_{pmt} (ksi)	Shear Strength Reduction Factor, α
Class 2 Base	0 to 1	130	40	0	1.5	1.0
Engineered Fill [b]	1 to 6	120	32	0	1.0	0.9
Undocumented Fill or Colluvium [b]	6 to 13	115	30	0	0.8	0.9
Old Paralic Deposits	>13	130	34	0	3.0	0.8

Notes:

a. The final project grading plan and equipment layout is not available at the time of this report. Design material depths are simplified and preliminary and will require re-evaluation during design development.

b. Existing undocumented fill and colluvium are preliminarily recommended to be removed and replaced with engineered fill to a minimum depth of 5 feet bgs. The depths shown herein represent the deepest interpreted areas of undocumented fill and colluvium.

c. No discount depth is recommended for surficial materials within the substation.

d. The design should consider groundwater below the design depth of 20 feet bgs.

e. pcf = pounds per cubic foot, psf = pounds per square foot, ksi = kips per square inch.

f. The shear strength reduction factor provided is for uncased drilled pier foundation excavations. Other factors should be provided for alternate construction methods.

g. Drilled shafts should bear a distance of at least 1.5 times the shaft diameter above the design groundwater depth of 20 feet bgs.

Table 6. CUFAD Design Parameters
Vine Substation

Material Type	Depth bgs [a] (feet)	Total Unit Weight (pcf)	Effective Friction Angle (degrees)	Undrained Strength (psf)	Adhesion Factor, α	Ratio of Operative to Insitu Coefficient of Horizontal Stress	In-Situ Coefficient of Horizontal Stress
Class 2 Base	0 to 1	130	40	0	NA	See note f	0.5
Engineered Fill [b]	1 to 6	120	32	0	NA	See note f	0.5
Undocumented Fill or Colluvium [b]	6 to 13	115	30	0	NA	See note f	0.5
Old Paralic Deposits	>13	130	34	0	NA	See note f	1.0

Notes:

a. The final project grading plan and equipment layout is not available at the time of this report. Design material depths are simplified and preliminary and will require re-evaluation during design development.

b. Existing undocumented fill and colluvium are preliminarily recommended to be removed and replaced with engineered fill to a minimum depth of 5 feet bgs. The depths shown herein represent the deepest interpreted areas of undocumented fill and colluvium.

c. No discount depth is recommended for surficial materials within the substation.

d. The design should consider groundwater below the design depth of 20 feet bgs.

e. pcf = pounds per cubic foot, psf = pounds per square foot, , NA = not applicable.

f. A value of 0.9 is recommended for dry, uncased drilled pier foundation excavations above the design groundwater level. A value of 0.67 is recommended if slurry is used, or a value of 0.83 is recommended if casing is used for construction below the groundwater level.

g. Drilled shafts should bear a distance of at least 1.5 times the shaft diameter above the design groundwater depth of 20 feet bgs.

Table 7. LPILE Design Parameters
Vine Substation

Material Type	Depth bgs [a] (feet)	Effective Unit Weight (pcf)	p-y Curve Type - Sand		
			Friction Angle (degrees)	Cohesion (psf)	Soil Modulus, k (pci)
Class 2 Base	0 to 1	130	40	0	225
Engineered Fill [b]	1 to 6	120	32	0	60
Undocumented Fill or Colluvium [b]	6 to 13	115	30	0	40
Old Paralic Deposits (unsaturated)	13 to 21	130	34	0	225
Old Paralic Deposits (saturated)	>21	68	34	0	125

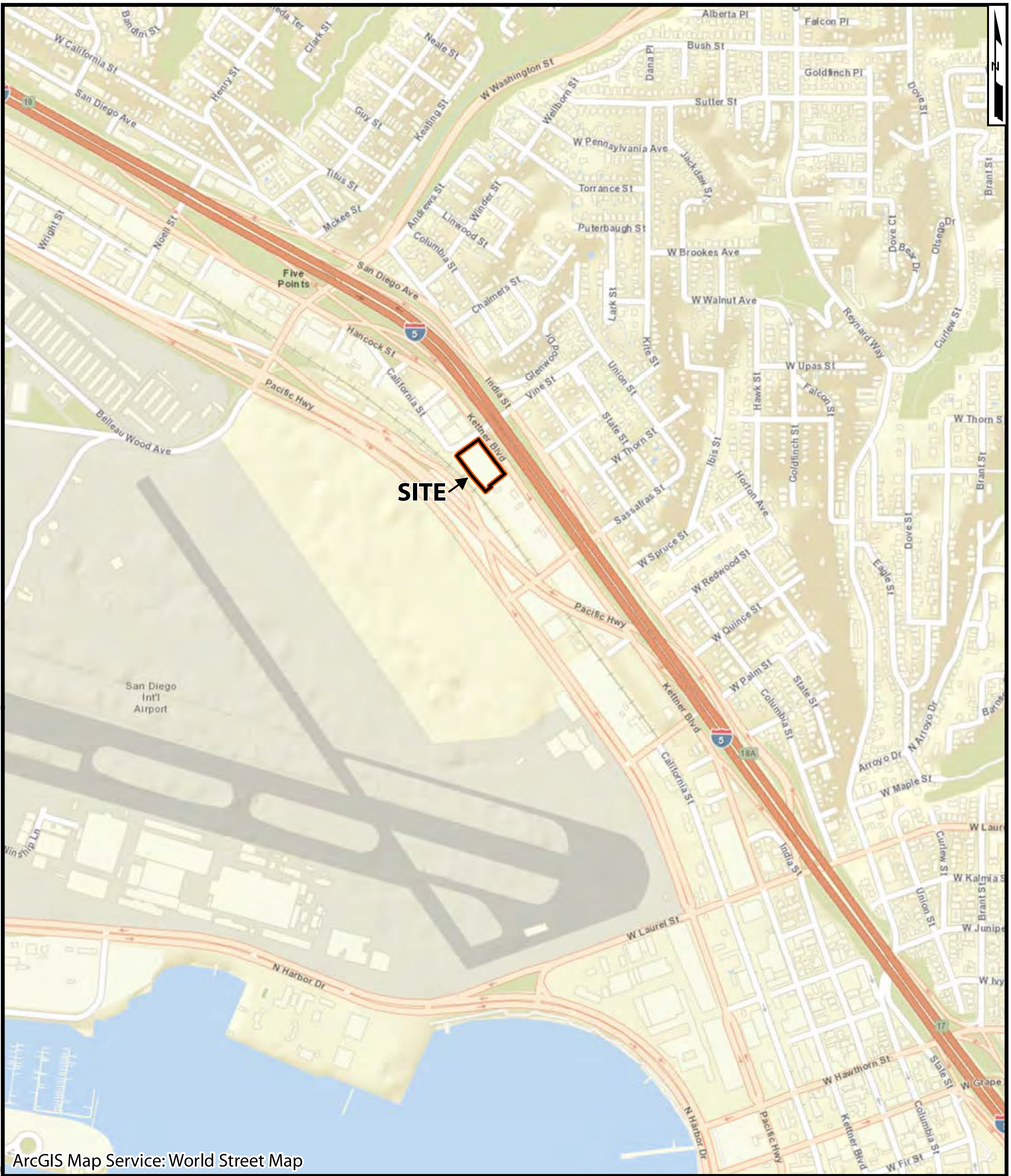
Notes:

a. The final project grading plan and equipment layout is not available at the time of this report. Design material depths are simplified and preliminary and will require re-evaluation during design development.
b. Existing undocumented fill and colluvium are recommended to be removed and replaced with engineered fill to a minimum depth of 5 feet bgs. The depths shown herein represent the deepest interpreted areas of undocumented fill and colluvium.
c. No discount depth is recommended for surficial materials within the substation.
d. The design should consider groundwater below the design depth of 20 feet bgs.
e. pcf = pounds per cubic foot, psf = pounds per square foot, pci = pounds per cubic inch.
f. Drilled shafts should bear a distance of at least 1.5 times the shaft diameter above the design groundwater depth of 20 feet bgs.

Table 8. Lateral Group Efficiencies for Deep Foundations
Vine Substation

Pile Spacing (in Diameters, D)	**Lateral P-multiplier, Pm**			
	3D	**4D**	**5D**	**>6D**
Lead Row	0.7	0.85	1.0	1.0
2nd Row	0.5	0.65	0.85	1.0
3rd and Higher Rows	0.35	0.5	0.7	1.0

FIGURES



1,000 500 0 1,000'

SITE LOCATION MAP
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

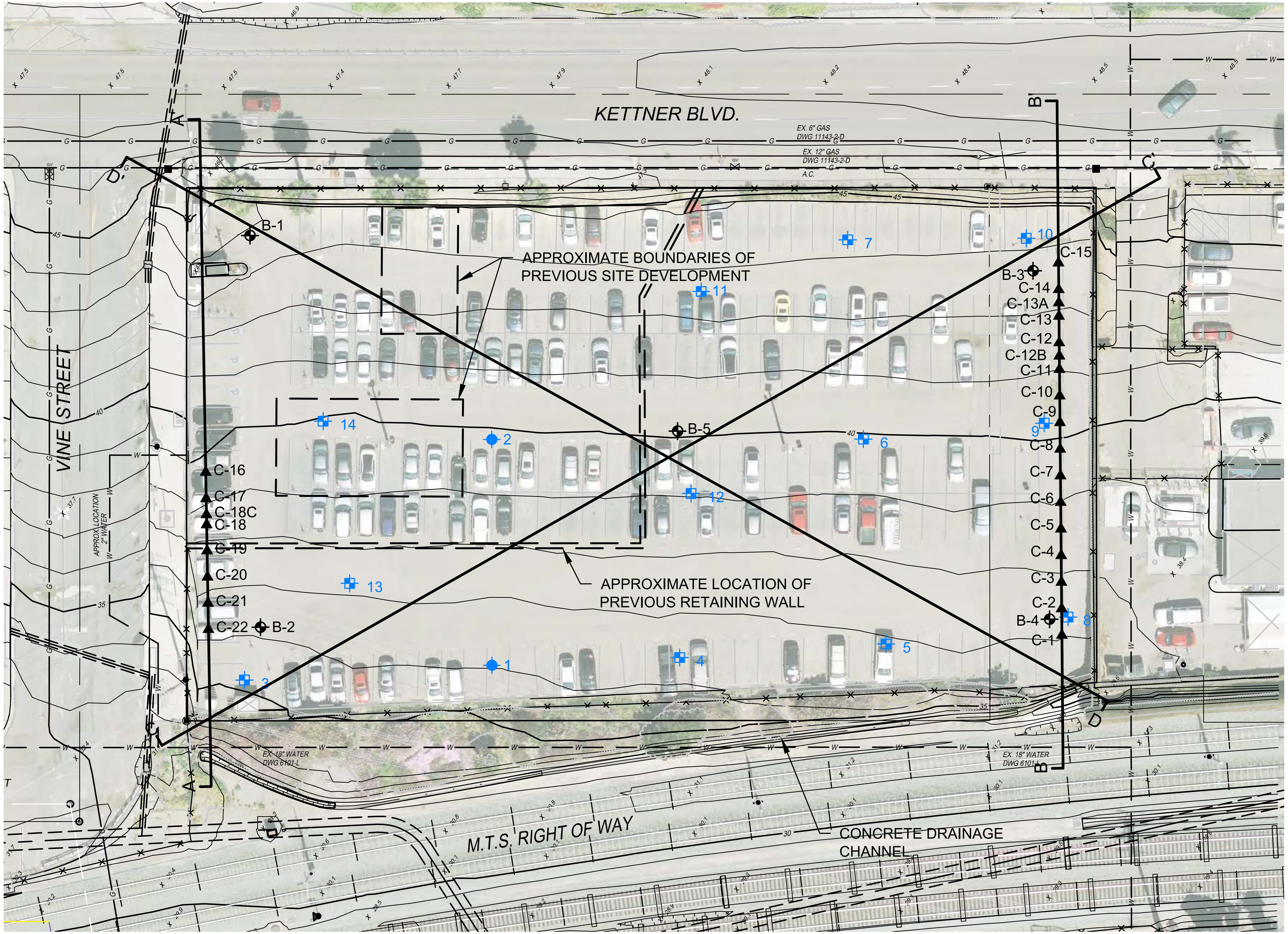
Geosyntec consultants

DATE:	DECEMBER 2013
PROJECT NO:	SC0368-30

FIGURE

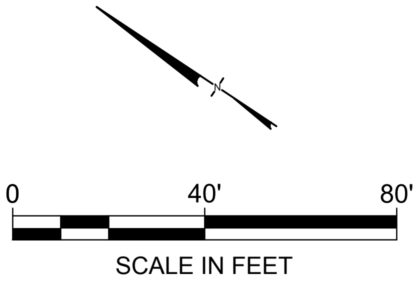
1

P:\GIS\SC0638_SDGE\SC0638_30_Kettner\SiteMap_Kettner.mxd



LEGEND

⌖	APPROXIMATE LOCATION OF BORING (GEOSYNTEC, 2013)
▲	APPROXIMATE LOCATION OF CPT (GEOSYNTEC, 2013)
●	APPROXIMATE LOCATION OF BORING (BENTON, 1974)
⌖	APPROXIMATE LOCATION OF TEST PIT (BENTON, 1977)
A ⌴ A'	APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION
——40——	EXISTING GROUND CONTOUR MAJOR (FEET)
～～～	EXISTING GROUND CONTOUR MINOR (FEET)
—×——×—	FENCELINE



SITE AND EXPLORATION LOCATION PLAN
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

Geosyntec consultants

DATE: DECEMBER 2013

PROJECT NO. SC0368-30

FIGURE 2

REFERENCE:
SURVEY AND AERIAL PHOTOGRAPHY FROM PDC, DATED 24 APRIL 2013.

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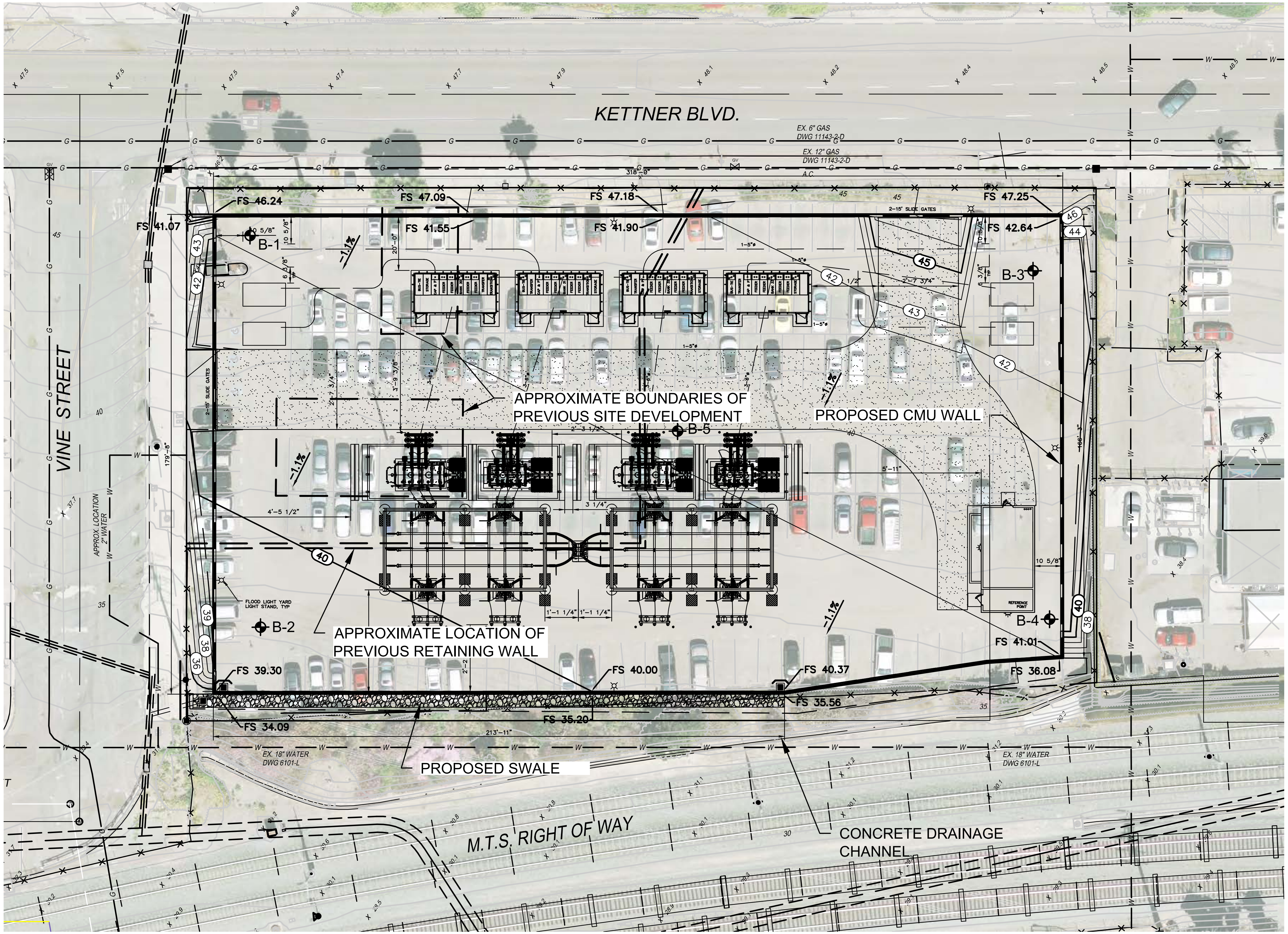
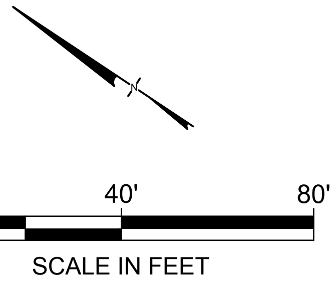


LEGEND

⊕	APPROXIMATE LOCATION OF BORING (GEOSYNTEC, 2013)
▲	APPROXIMATE LOCATION OF CPT (GEOSYNTEC, 2013)
⊕	APPROXIMATE LOCATION OF BORING (BENTON, 1974)
⊞	APPROXIMATE LOCATION OF TEST PIT (BENTON, 1977)
—40—	EXISTING GROUND CONTOUR MAJOR (FEET)
——	EXISTING GROUND CONTOUR MINOR (FEET)
—(45)—	PROPOSED GROUND CONTOUR MAJOR (FEET)
—(42)—	PROPOSED GROUND CONTOUR MINOR (FEET)
—×—×—	FENCELINE
▬	PROPOSED CMU WALL



REFERENCES:
1. SURVEY AND AERIAL PHOTOGRAPHY FROM PDC, DATED 24 APRIL 2013.
2. SUBSTATION ARRANGEMENT FROM SDG&E, DATED JULY 25, 2013.

P:\CAD\SC0368-30-04 - KETTNER SUBSTATION\FIGURES\SC0368 FUTURE SITE PLAN.DWG - DRAWN BY:JAMOS

SITE AND POTENTIAL EQUIPMENT LAYOUT
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

Geosyntec consultants

DATE: DECEMBER 2013

PROJECT NO. SC0368-30

FIGURE 3

NOT TO SCALE

AERIAL PHOTO SOURCE: USGS, 1972

1972 AERIAL PHOTOGRAPH
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

Geosyntec consultants

DATE:	DECEMBER 2013	FIGURE
PROJECT NO.	SC0368-30	4

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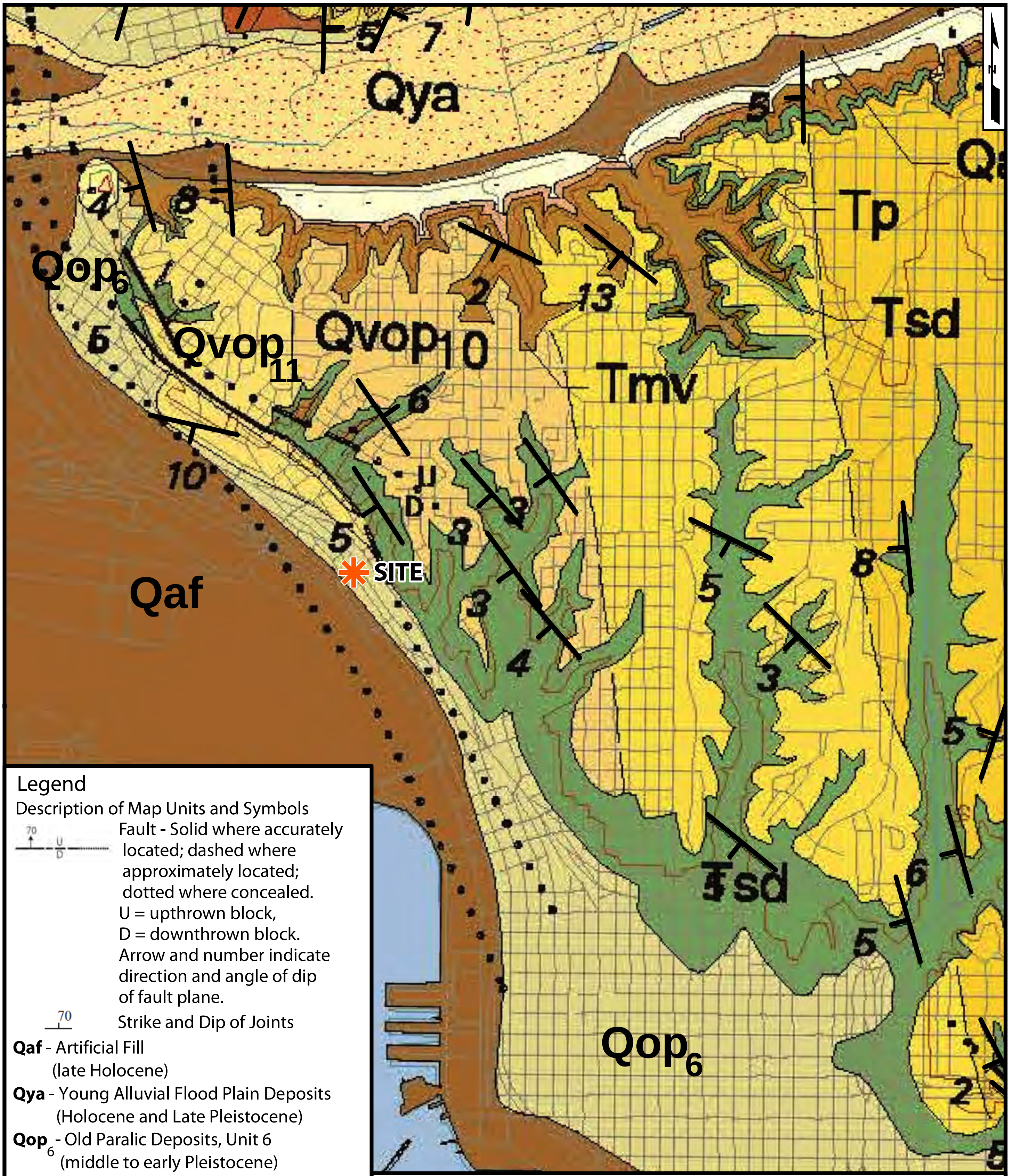


2,500 1,250 0 2,500 5,000'

Legend

Description of Map Units and Symbols

70 U D — Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

70 — Strike and Dip of Joints

Qaf - Artificial Fill (late Holocene)

Qya - Young Alluvial Flood Plain Deposits (Holocene and Late Pleistocene)

Qop$_6$ - Old Paralic Deposits, Unit 6 (middle to early Pleistocene)

Qvop$_{10}$ - Very Old Paralic Deposits, Unit 10 (middle to early Pleistocene)

Qvop$_{11}$ - Very Old Paralic Deposits, Unit 11 (middle to early Pleistocene)

Tsd - San Diego Formation (early Pleistocene and late Pliocene)

Tmv - Mission Valley Formation (Middle Eocene)

Map Source:
Kennedy, M.P. and Tan, S.S. (2005)

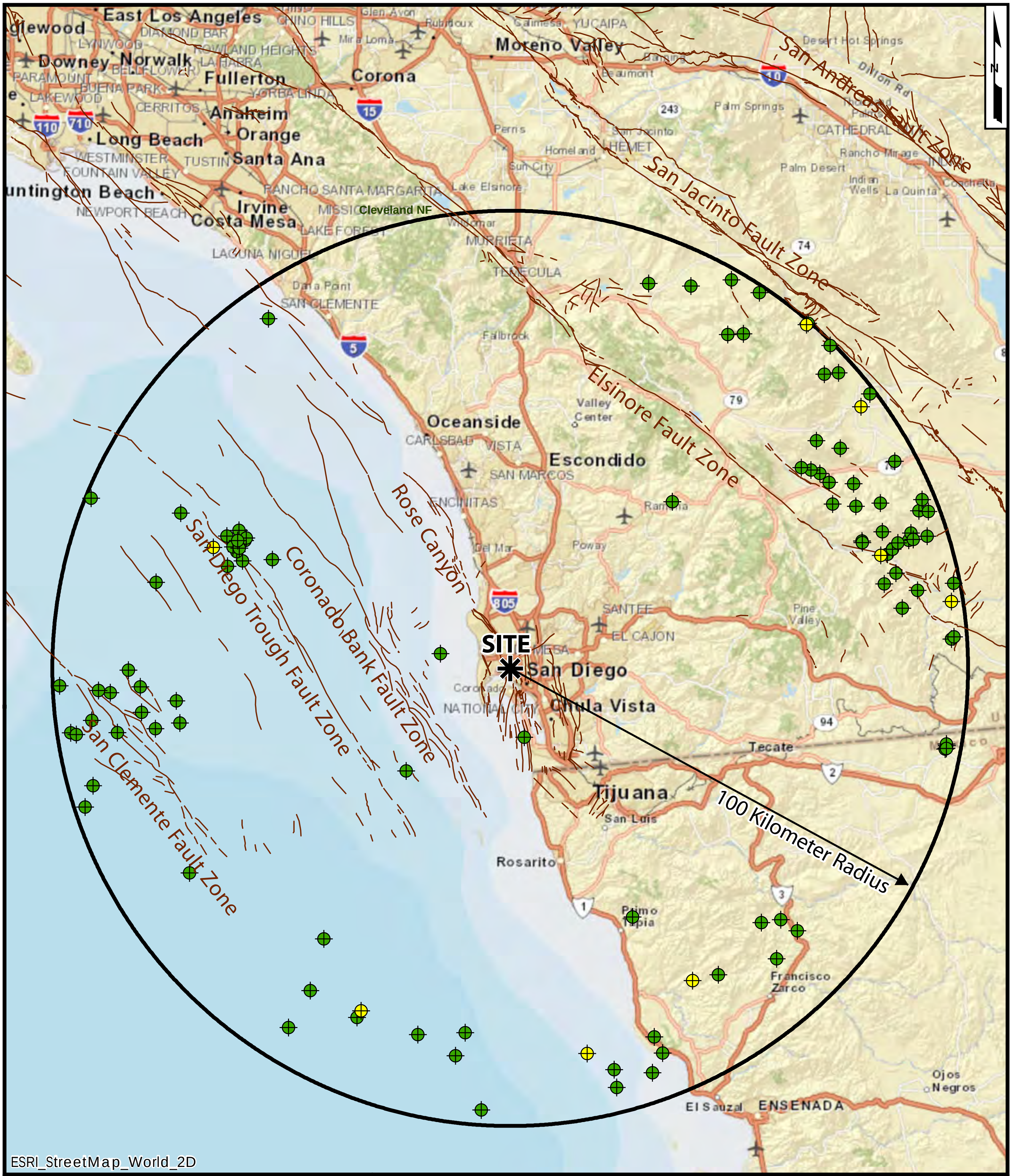
REGIONAL GEOLOGIC MAP
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

Geosyntec consultants

DATE: DECEMBER 2013

PROJECT NO: SC0368-30

FIGURE
5



25 12.5 0 25 50 Kilometers

Legend

Reported Earthquake Magnitudes 1932 - 2013

🟢 4.0 - 4.99

🟡 5.0 - 5.66

— Fault

Data Source: Southern California Earthquake Data Center

Image Source: ArcOnline NGS_Topo_US_2D

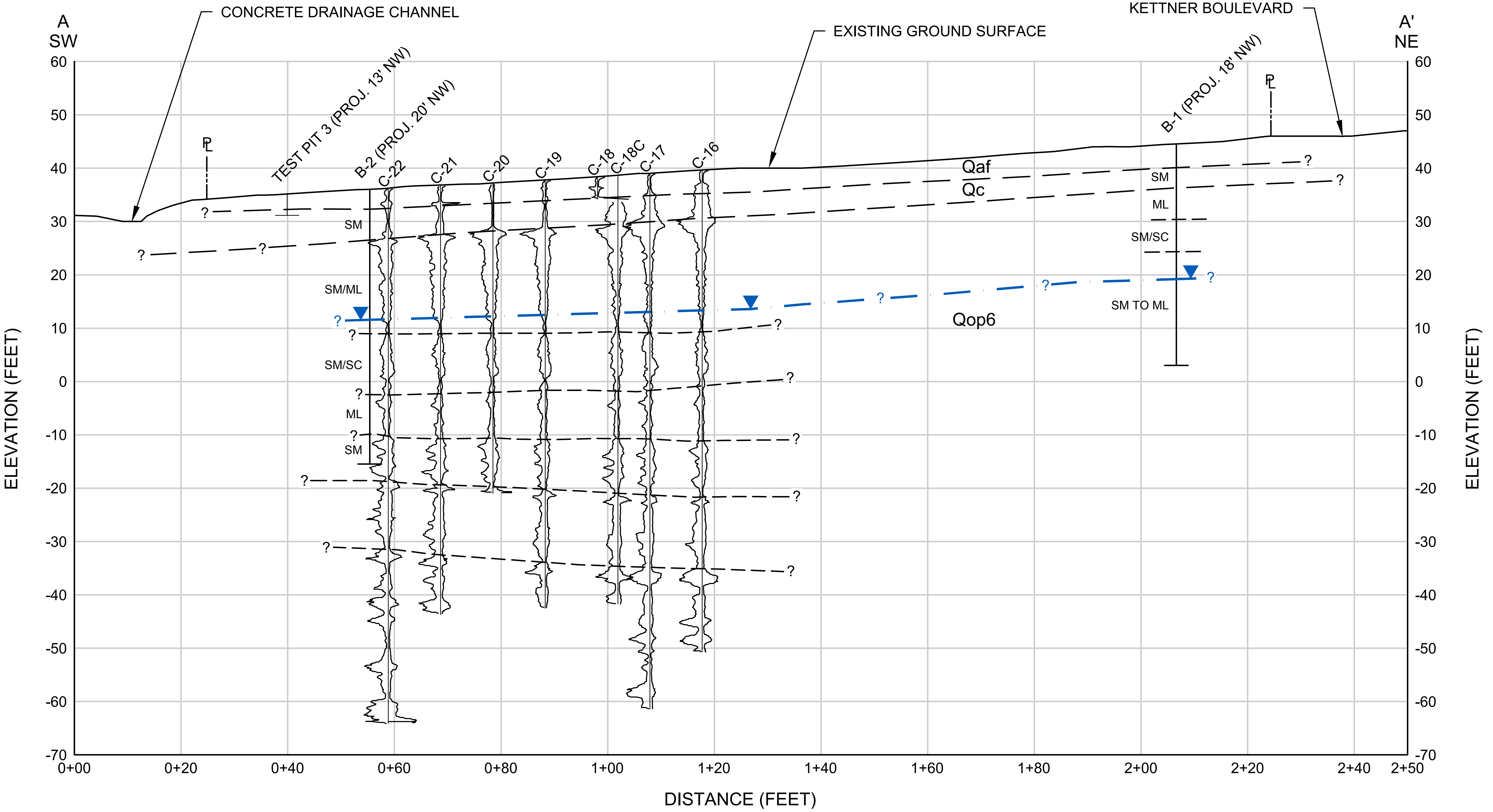
REGIONAL FAULT AND EPICENTER MAP
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

Geosyntec consultants

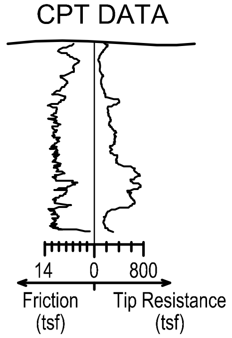
DATE: DECEMBER 2013

PROJECT NO: SC0368-30

FIGURE
6



LEGEND



EXISTING GROUND

GEOLOGIC CONTACT
(QUERIED WHERE UNCERTAIN)

LITHOLOGIC CONTACT
(APPROXIMATE)

INTERPRETED GROUNDWATER
BASED ON MEASUREMENTS
DURING EXPLORATION PROGRAM

APPROXIMATE LOCATION OF
BORING (GEOSYNTEC, 2013)

APPROXIMATE LOCATION OF
BORING (BENTON, 1974)
OR TEST PIT (BENTON, 1977)

PL PROPERTY LINE

Qaf	ARTIFICIAL FILL
Qc	COLLUVIUM
Qop6	OLD PARALIC DEPOSITS
SP, SM, SC, ML, CL, CH	USCS CLASSIFICATION SEE APPENDIX B

0 20' 40'

SCALE IN FEET
HORIZ: 1" = 20'
VERT: 1" = 20'

GEOLOGIC CROSS SECTION A-A'
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

Geosyntec
consultants

DATE: DECEMBER 2013
PROJECT NO. SC0368-30

FIGURE
7

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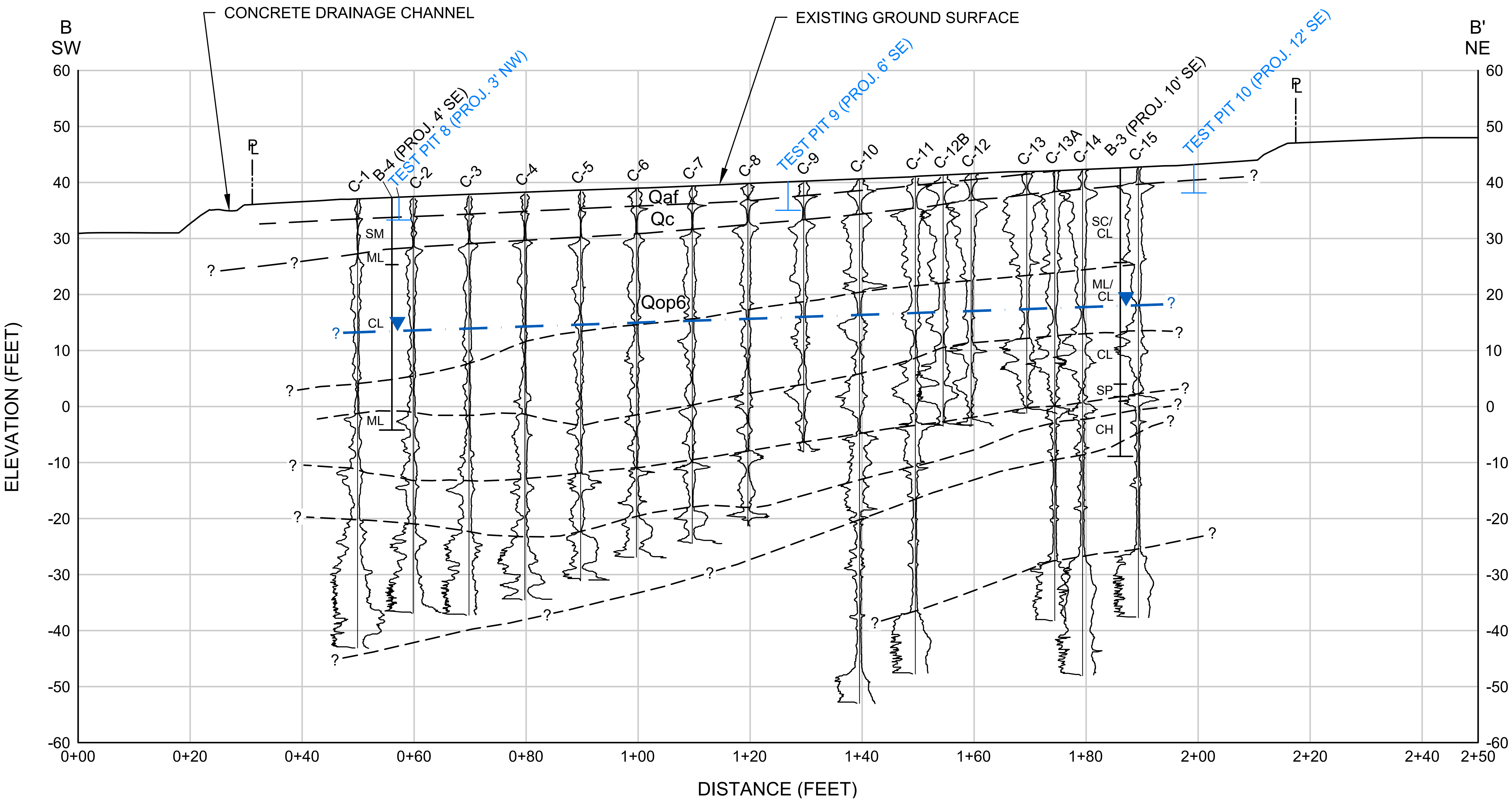
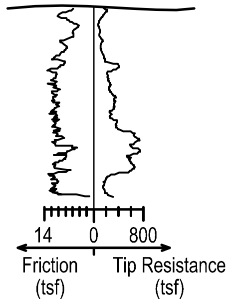


LEGEND

CPT DATA



EXISTING GROUND

GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)

LITHOLOGIC CONTACT (APPROXIMATE)

INTERPRETED GROUNDWATER BASED ON MEASUREMENTS DURING EXPLORATION PROGRAM

APPROXIMATE LOCATION OF BORING (GEOSYNTEC, 2013)

APPROXIMATE LOCATION OF BORING (BENTON, 1974) OR TEST PIT (BENTON, 1977)

℄ PROPERTY LINE

Qaf	ARTIFICIAL FILL
Qc	COLLUVIUM
Qop6	OLD PARALIC DEPOSITS
SP, SM, SC, ML, CL, CH	USCS CLASSIFICATION SEE APPENDIX B

0 20' 40'

SCALE IN FEET
HORIZ: 1" = 20'
VERT: 1" = 20'

GEOLOGIC CROSS SECTION B-B'
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

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DATE: DECEMBER 2013

PROJECT NO. SC0368-30

FIGURE 8

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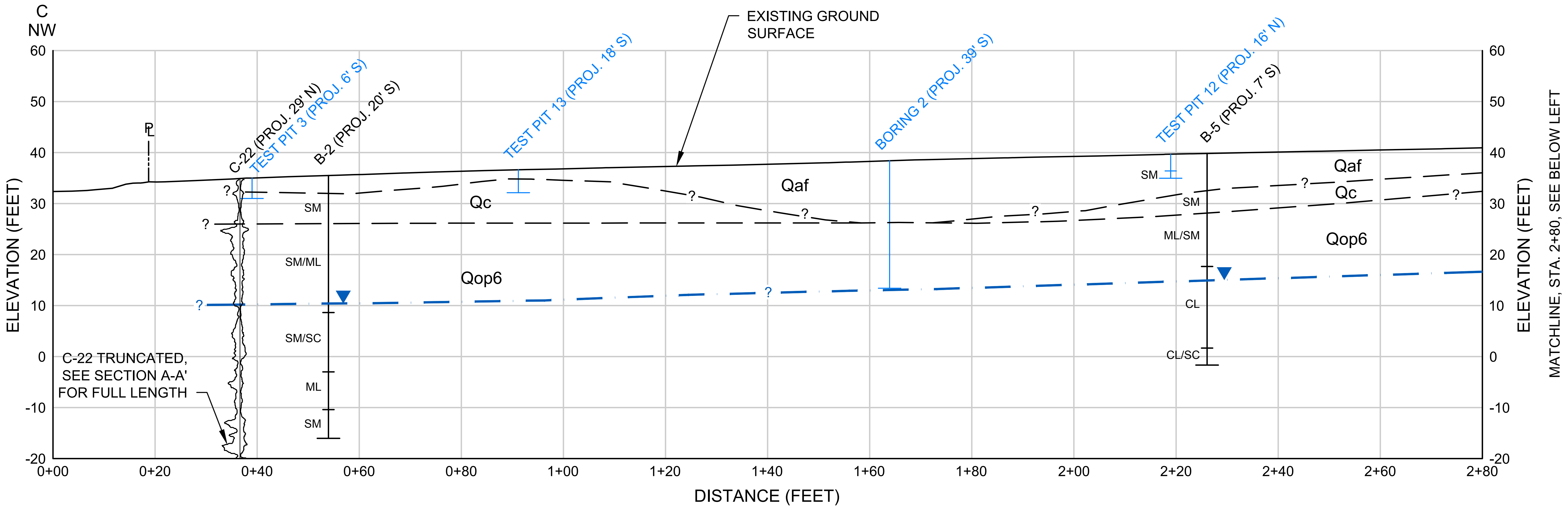
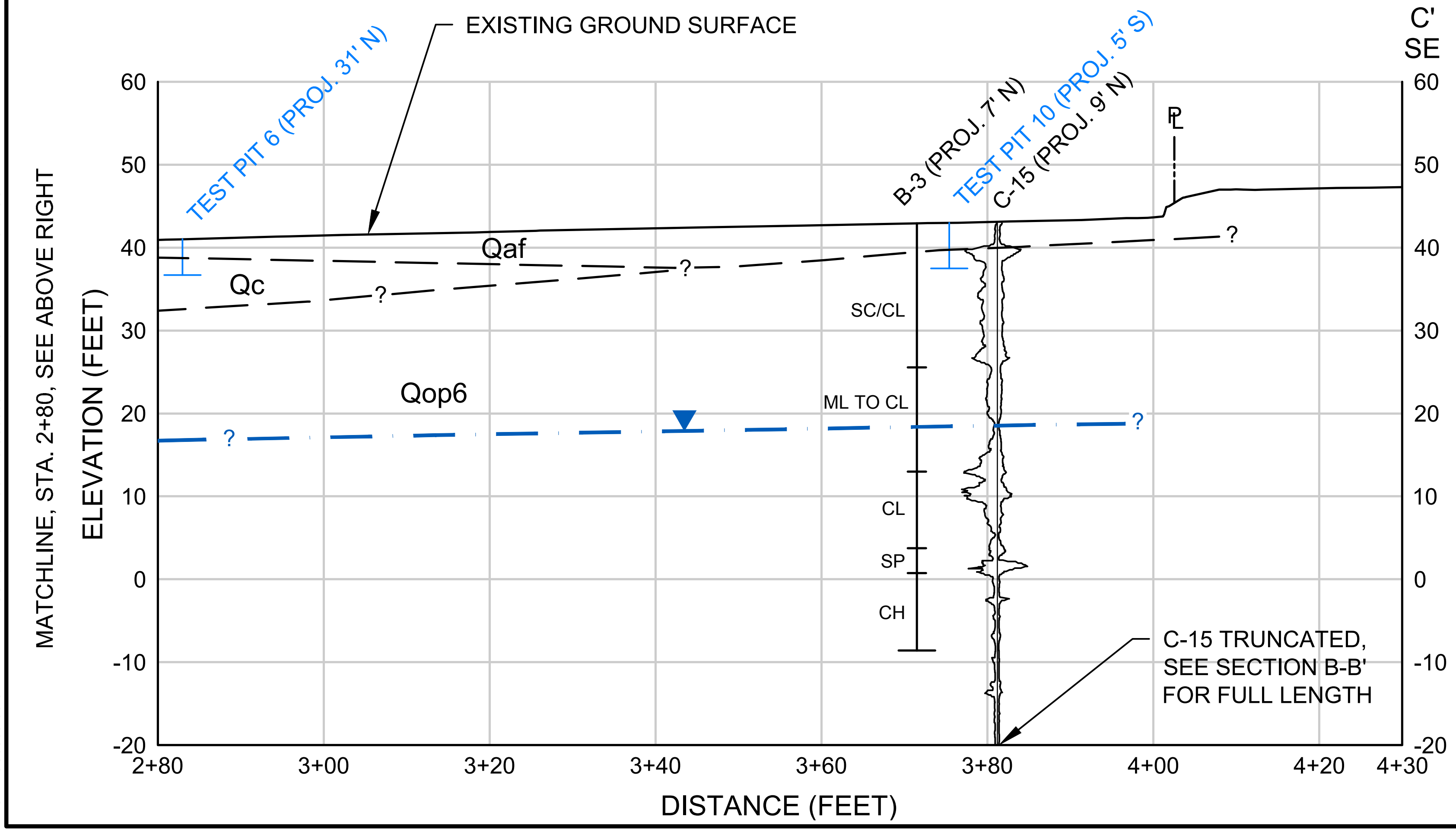




LEGEND

EXISTING GROUND

GEOLOGIC CONTACT
(QUERIED WHERE UNCERTAIN)

LITHOLOGIC CONTACT
(APPROXIMATE)

INTERPRETED GROUNDWATER
BASED ON MEASUREMENTS DURING
EXPLORATION PROGRAM

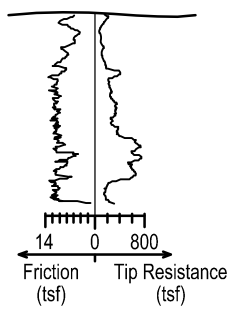
APPROXIMATE LOCATION OF BORING
(GEOSYNTEC, 2013)

APPROXIMATE LOCATION OF BORING
(BENTON, 1974)
OR TEST PIT (BENTON, 1977)

PL PROPERTY LINE

Qaf	ARTIFICIAL FILL
Qc	COLLUVIUM
Qop6	OLD PARALIC DEPOSITS
SP, SM, SC, ML, CL, CH	USCS CLASSIFICATION SEE APPENDIX B

CPT DATA



0 20' 40'

SCALE IN FEET
HORIZ: 1" = 20'
VERT: 1" = 20'

GEOLOGIC CROSS SECTION C-C'
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

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DATE: DECEMBER 2013

PROJECT NO. SC0368-30

FIGURE
9

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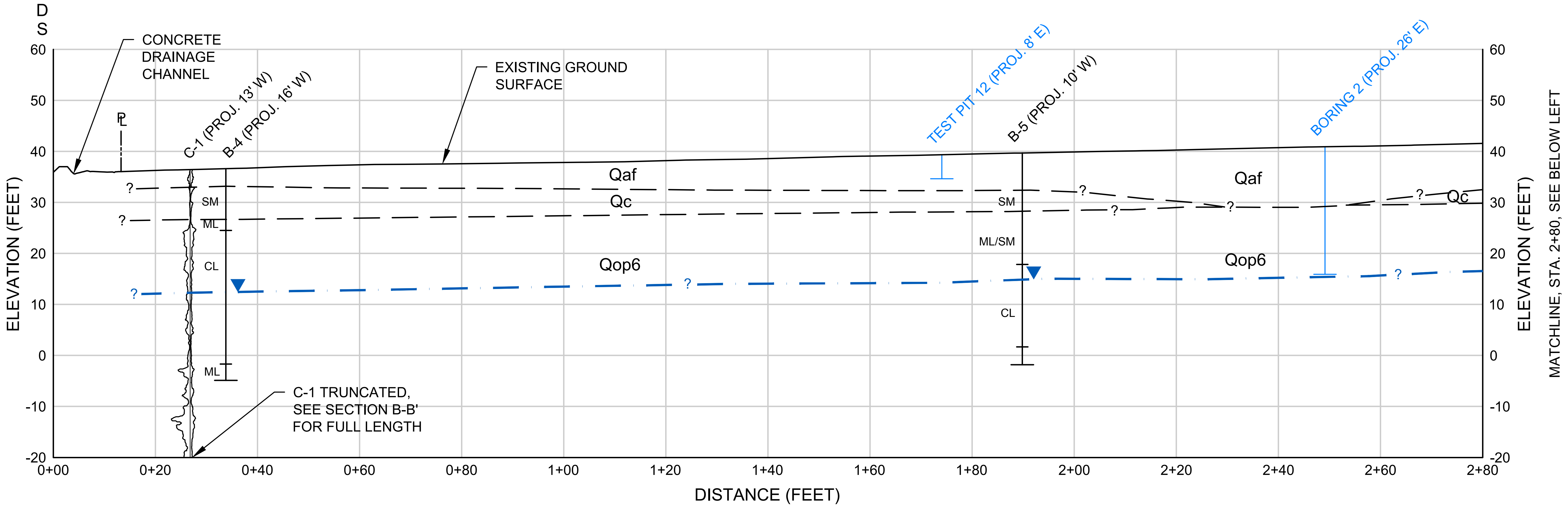
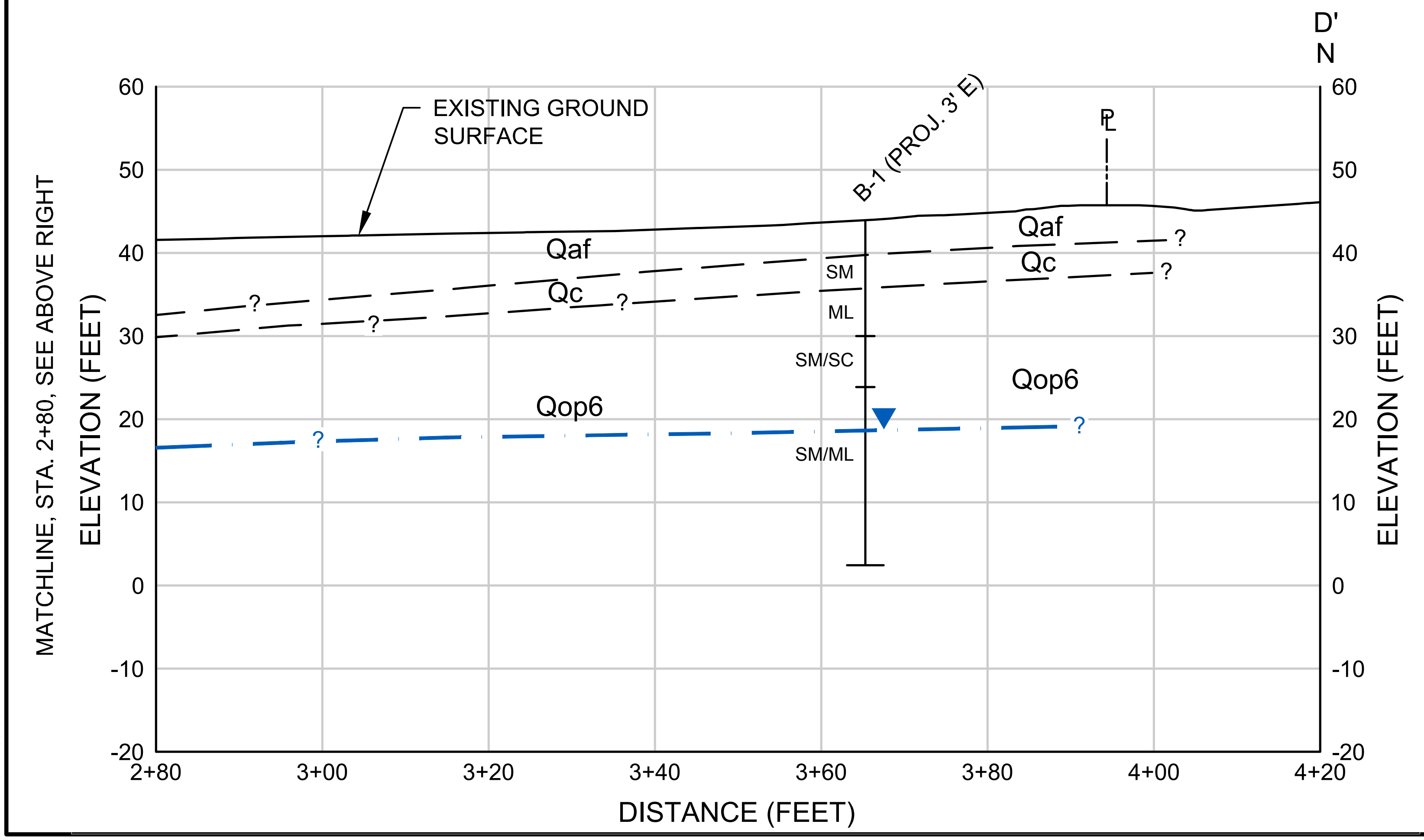


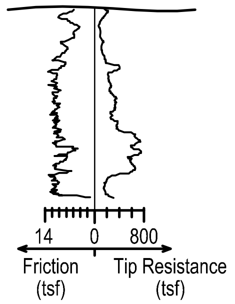


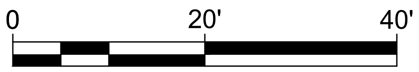
LEGEND

───	EXISTING GROUND
— — —	GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)
- - - -	LITHOLOGIC CONTACT (APPROXIMATE)
▼ — —	INTERPRETED GROUNDWATER BASED ON MEASUREMENTS DURING EXPLORATION PROGRAM
┴	APPROXIMATE LOCATION OF BORING (GEOSYNTEC, 2013)
┴	APPROXIMATE LOCATION OF BORING (BENTON, 1974) OR TEST PIT (BENTON, 1977)
℄	PROPERTY LINE

Qaf	ARTIFICIAL FILL
Qc	COLLUVIUM
Qop6	OLD PARALIC DEPOSITS
SP, SM, SC, ML, CL, CH	USCS CLASSIFICATION SEE APPENDIX B

CPT DATA





SCALE IN FEET
HORIZ: 1" = 20'
VERT: 1" = 20'

GEOLOGIC CROSS SECTION D-D'
VINE SUBSTATION
SAN DIEGO, CALIFORNIA

Geosyntec consultants

DATE: DECEMBER 2013
PROJECT NO. SC0368-30

FIGURE
10

P:\CAD\SC0368\SC0368-30-04 - KETTNER SUBSTATION\FIGURES\SC0368 F02.DWG - DRAWN BY:JAMOS

APPENDIX A

Previous Investigations

Benton, 1974

G271

Geotechnical Report

BENTON ENGINEERING, INC.
APPLIED SOIL MECHANICS — FOUNDATIONS
6717 CONVOY COURT
SAN DIEGO, CALIFORNIA 92111

PHILIP HENKING BENTON
PRESIDENT - CIVIL ENGINEER

TELEPHONE (714) 565-1955

October 14, 1974

San Diego Gas & Electric Company
P. O. Box 1831
San Diego, California 92112

Attention: Mr. E. J. Brancheau

Gentlemen:

This is to transmit to you two copies of our report of Project No. 74-10-2A entitled, "Preliminary Soils Investigation, Proposed Site No. 1 of Laurel Substation, South of the Intersection of Vine Street and Kettner Boulevard, San Diego, California," dated October 14, 1974.

If you should have any questions concerning any of the data presented in this report, please contact us.

Very truly yours,

BENTON ENGINEERING, INC.

Philip H Benton

Philip H. Benton, Civil Engineer

PRELIMINARY SOILS INVESTIGATION

Proposed Site No. 1 of
Laurel Substation
San Diego Gas & Electric Company
South of the Intersection of
Vine Street and Kettner Boulevard
San Diego, California

for the
San Diego Gas & Electric Company

Project No. 74-10-2A
October 14, 1974

BENTON ENGINEERING, INC.
APPLIED SOIL MECHANICS — FOUNDATIONS
6717 CONVOY COURT
SAN DIEGO, CALIFORNIA 92111

PHILIP HENKING BENTON
PRESIDENT - CIVIL ENGINEER

TELEPHONE (714) 565-1955

PRELIMINARY SOILS INVESTIGATION

Introduction

This is to present the results of a preliminary soils investigation conducted at Site No. 1 for the possible Laurel Substation construction. The proposed site is located southwesterly of the intersection of Kettner Boulevard and Vine Street, in San Diego, California.

The objectives of this investigation were to determine the general subsurface conditions of the site and certain physical properties of the soils so that sufficient subsurface information could be developed for site evaluation and selection.

In order to accomplish these objectives, two borings were drilled at selected locations, and both undisturbed and loose soil samples were obtained for laboratory testing.

Field Investigation

The two borings were drilled 24 and 36 inches in diameter, with a truck-mounted rotary bucket-type drill rig at the approximate locations shown on the attached Drawing No. 1, entitled "Location of Test Borings." The borings were drilled to depths of 15 to 25 feet below the existing ground surface. A continuous log of the soils encountered in the borings was recorded at the time of drilling and is shown in detail on Drawing Nos. 2 to 4, inclusive, each entitled "Summary Sheet."

The soils were visually classified by field identification procedures in accordance with the Unified Soil Classification Chart. A simplified description of this classification system is presented in the attached Appendix A at the end of this report.

Undisturbed samples were obtained at frequent intervals in the soils ahead of the drilling. The drop weight used for driving the sampling tube into the soils was the "Kelly" bar of the drill rig which weighs 1623 pounds, and the average drop was 12 inches. The general procedures used in the field sampling are described under "Sampling" in Appendix B.

Laboratory Tests

Laboratory tests were performed on all undisturbed samples of the soils in order to determine the dry density and moisture content. The results of these tests are presented on Drawing Nos. 2 to 4, inclusive. Consolidation tests were perfarmed on representative samples in order to determine the load-settlement characteristics of the soils and the results of these tests are presented graphically on Drawing Nos. 5 to 7, inclusive, each entitled "Consolidation Curves."

Compaction tests were performed on representative samples of the existing fill soil samples found at the site in order to establish compaction criteria. The soils were tested according to the A.S.T.M. D 1557-70 method of compaction which uses 25 blows of a 10 pound rammer dropping 18 inches on each of 5 layers in a 4 inch diameter 1/30 cubic foot compaction mold. The results of the tests are presented as follows:

Boring No.	Bag Sample	Depth in Feet	Soil Description	Maximum Dry Density lb/cu ft	Optimum Moisture Content % dry weight
1	1	1.0-2.0	Silty fine sand fill	119.9	11.4
2	1	1.0-2.0	Silty fine sand fill	108.2	17.4
2	3	9.0-10.0	Silty fine sand fill mottled with clayey fine sand	123.2	12.1

DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

Soil Strata

At Boring 1, an asphalt concrete paving was found in the upper 0.3 foot of the boring.

The asphalt concrete paving was underlain by loose silty fine sand fills to 3.2 feet. The fill soils were immediately underlain by a loose alluvial deposit consisting of porous silty fine sand and silt soils to 3.7 feet. Below 3.7 feet, a silty fine sand was encountered to the end of boring at 15.0 feet. The silty fine sand was medium firm between 3.7 and 8.0 feet in depth and very firm below.

No ground water was encountered in this boring.

At Boring 2, existing fill soils were found to a depth of 12.2 feet below existing ground surface. The fill soils consisted of loose silty fine sand and medium compact silty fine sand mottled with clayey fine sand. The fill soils were loose in the upper 0.5 foot, medium compact between 0.5 and 11.0 feet in depth and then merged to compact between 11.0 and 12.2 feet in depth. Below 12.2 feet, a medium firm silty fine sand was found to 18.0 feet and then merged to very firm silty fine to medium sand and silty fine sand to the end of boring at 25.0 feet.

No ground water was encountered in this boring.

Conclusions

It is concluded from the results of field explorations and laoboratory tests that the existing fill soils in the upper 3.2 and 12.2 feet, respectively, of Borings 1 and 2 have in-place densities varying from 78 to 89 percent of their maximum dry densities. The majority of the densities fall between 80.3 to 88.7 percent of maximum dry densities. The fill soils in the area of Boring 1 exhibited abrupt consolidation upon saturation with water indicating insufficient compaction at the time of fill placement. It is concluded that the fill soils in this area would have to be removed and recompacted under engineering inspection and testing to assure that these are uniformly compacted to at least 90 percent of maximum dry density and are then satisfactory for structural support.

The fill soils in the upper 12.2 feet of Boring 2 exhibited slight expansion upon saturation with water under a unit load of 1 kip per square foot in accordance with the results of consolidation tests. The fill soils encountered at Boring 2 are generally free of deleterious materials and have satisfactory load-settlement characteristics. However, it is not known whether these fill soils have been uniformly compacted throughout the entire fill area of the upper portion of the site.

Recommendations

In order to better determine the uniformity of the upper fill soils existing in the area of Boring 2, it would be desirable to drill additional borings and sample the soils in this portion of the site.

We were told that the site was an abandoned gas station site, and the fuel reservoir for the former gas station has been removed. The soil conditions around the former fuel reservoir are not known at this time. Additional borings in the former fuel reservoir be desirable to evaluate this area also. More definite conclusions can be drawn as to the suitability of the existing fill soils for load-supporting use once the evaluation of additional test results of the soil samples from additional borings is completed.

Specific recommendations regarding to pavement design, compaction specifications, and soil parameters for footing design will be presented in a supplemental report if the site is selected and we are authorized to complete the laboratory tests on the samples now stored in our laboratory.

Respectfully submitted,

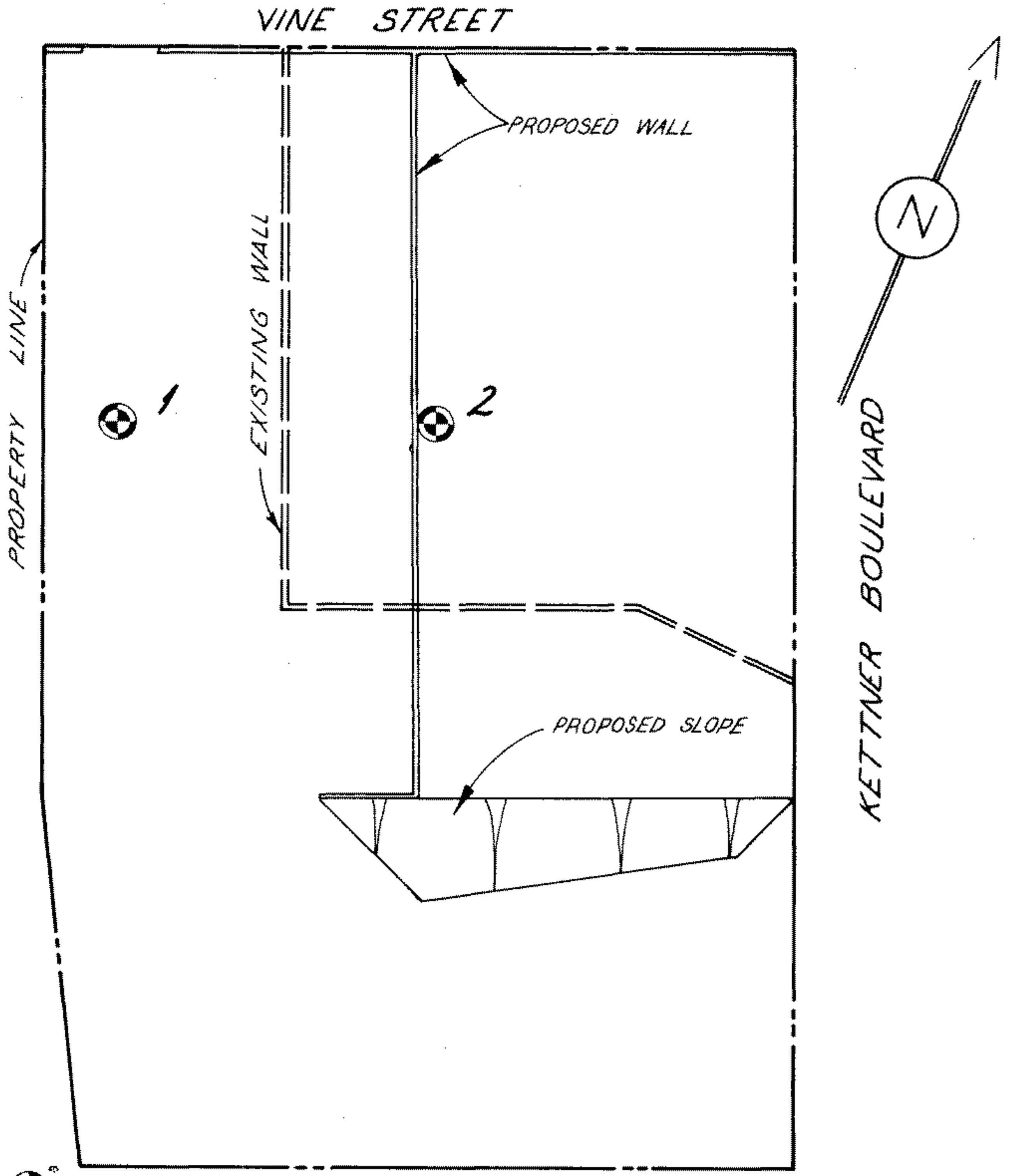
BENTON ENGINEERING, INC.

By
S. H. Shu, Civil Engineer

Reviewed by
Philip H. Benton, Civil Engineer

SHS/PHB/pk

Distr: (2) San Diego Gas & Electric Company
Attention: Mr. Ed Brancheau



⊗ Indicates Approximate Location of Test Boring

Date: 10-9-74 Drawn: M.R.C. Scale: 1"=40'

LOCATION OF TEST BORINGS

LAUREL SUBSTATION - SITE NO. 1
SAN DIEGO GAS & ELECTRIC COMPANY
VINE STREET AND KETTNER BOULEVARD
SAN DIEGO, CALIFORNIA

PROJECT NO.
74-10-2A

BENTON ENGINEERING, INC.

DRAWING NO.
1

JOB NAME San Diego Gas & Electric Company, Laurel Substation

SUMMARY SHEET

BORING NO. 1

ELEVATION 37.3' *

DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL			DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	
0			3 INCHES THICK	A.C. PAVING					
1	1	▨	Brown, Dry, Loose, Occasional Gravel	SILTY FINE SAND					FILL
2	①		Moist		1.6	5.5	93.6		
4	②	▨	Brown, Moist, Loose, Porous, Silty Clay Lenses, Alluvium	SILTY FINE SAND AND SILT	3.2	8.1	104.2		
5		▨	Brown, Moist, Medium Firm, Slightly Porous	SILTY FINE SAND					
7	③				0.8	9.0	9B.2		
8			Very Firm						
10	④				16.2	12.8	116.1		
14			Very Moist						
15	⑤				4.9	1B.7	108.4		

☐ - Indicates Loose Bag Sample

○ - Indicates Undisturbed Drive Sample

* - Elevations shown were obtained by interpolation between contour line on drawing furnished by San Diego Gas & Electric Company.

PROJECT NO.		DRAWING NO.
74-10-2A	BENTON ENGINEERING, INC.	2

JOB NAME San Diego Gas & Electric Company, Laurel Substation

SUMMARY SHEET

BORING NO. 2

ELEVATION 50.0'

DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL			DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	
0		▨	Yellow-brown, Moist, Laase	SILTY FINE SAND					FILL
1	$\boxed{1}$	▨	Medium Compact						
2	①				3.2	16.9	96.0		
4	②				4.9	20.4	101.4		
5		▨	Gray and Dark Gray, Moist, Medium Compact, Occasional Roots	SILTY FINE SAND MOTTLED WITH CLAYEY FINE SAND					
6	$\boxed{2}$								
7	③				3.2	16.7	108.2		
9	$\boxed{3}$								
10	④				2.4	16.5	99.1		
11		▨	Brown, Moist, Compact, Occasional Chunks of Light Brown Silty Fine Sand	SILTY FINE SAND					
12	$\boxed{4}$								
13	⑤	▨	Brown, Moist, Medium Firm	SILTY FINE SAND	4.9	9.6	102.2		
14			Loose to Medium Firm, Slightly Poraus						
16	⑥				1.6	6.0	98.8		
18		▨	Red-brown, Moist, Very Firm	SILTY FINE TO MEDIUM SAND					
20	⑦				26.0	3.7	120.6		

Continued on Drawing No. 4

PROJECT NO. 74-10-2A

BENTON ENGINEERING, INC.

DRAWING NO. 3

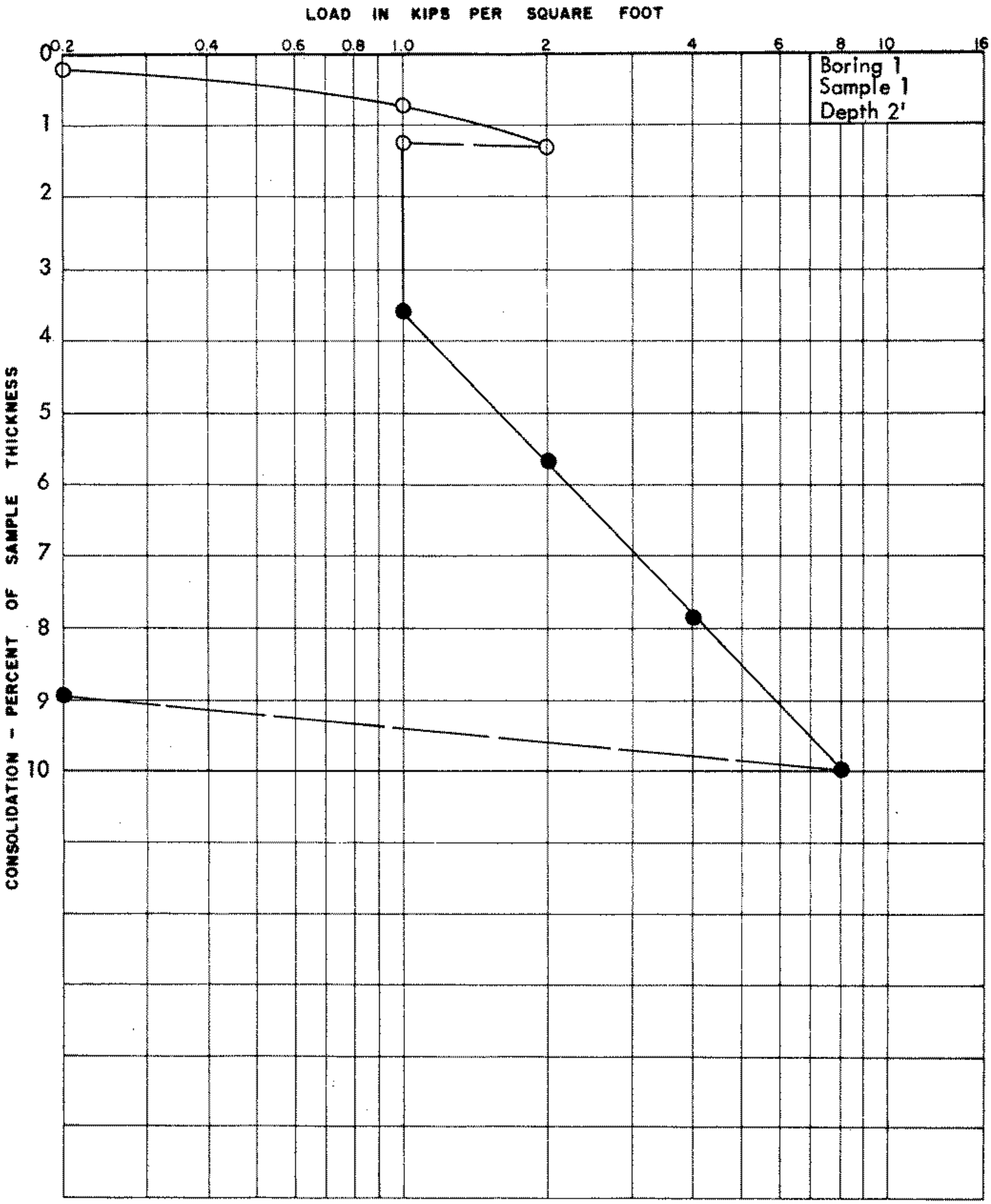
JOB NAME San Diego Gas & Electric Company, Laurel Substation

DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHEET BORING NO. 2		DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	
20		▨	Red-brown, Moist, Very Firm	SILTY FINE TO MEDIUM SAND					
21 22 23 24 25	⑧	▨	Dark Gray-brown, Moist, Very Firm	SILTY FINE SAND	15.0	13.2	102.8		

PROJECT NO.		DRAWING NO.
74-10-2A	BENTON ENGINEERING, INC.	4

JOB NAME S.D.G.&.E. - Laurel Substation - Site 1

CONSOLIDATION CURVES

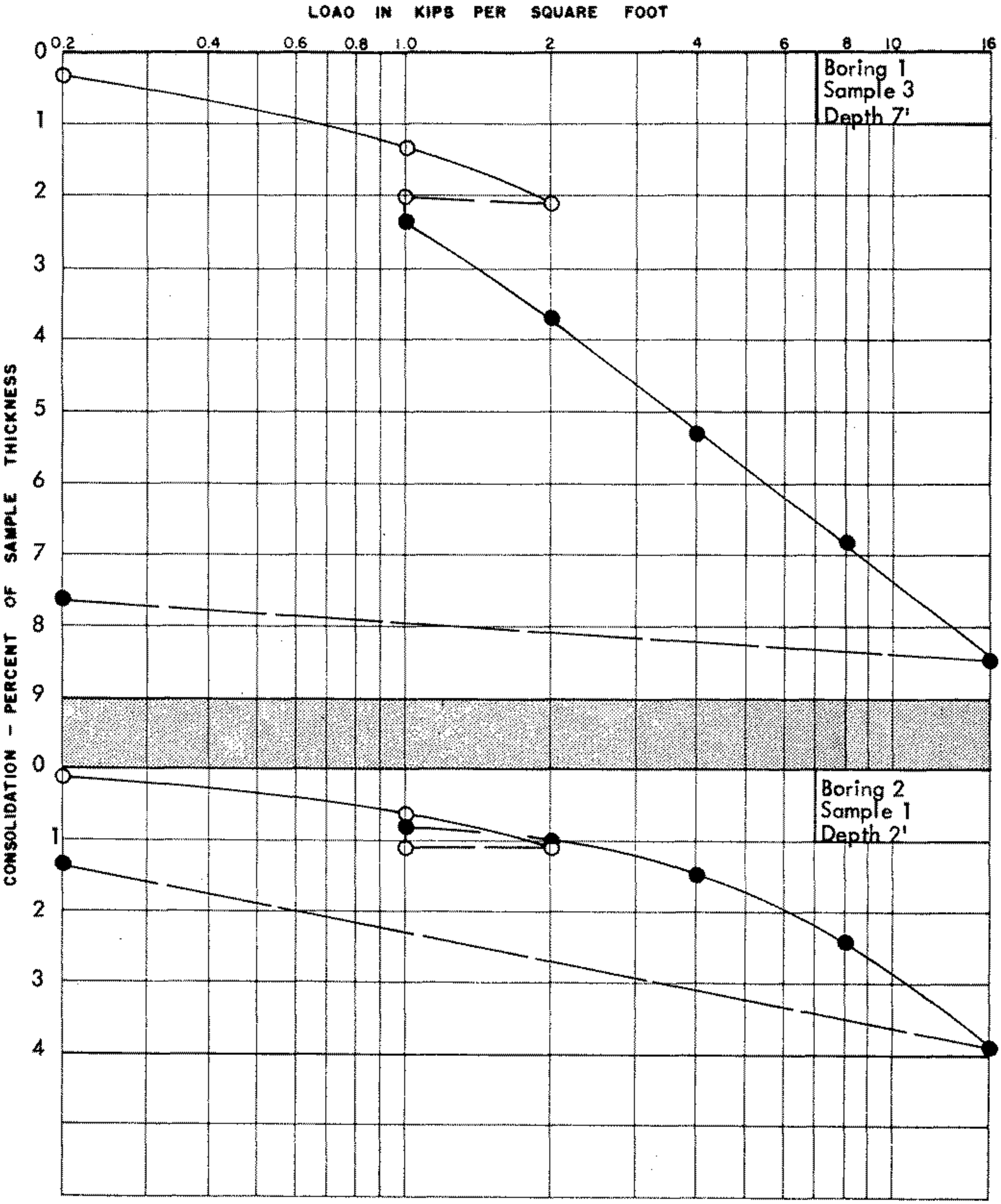


○ Indicates percent consolidation at field moisture

● Indicates percent consolidation after saturation

PROJECT NO.		DRAWING NO.
74-10-2A	**BENTON ENGINEERING, INC.**	5

CONSOLIDATION CURVES



○ Indicates percent consolidation at field moisture

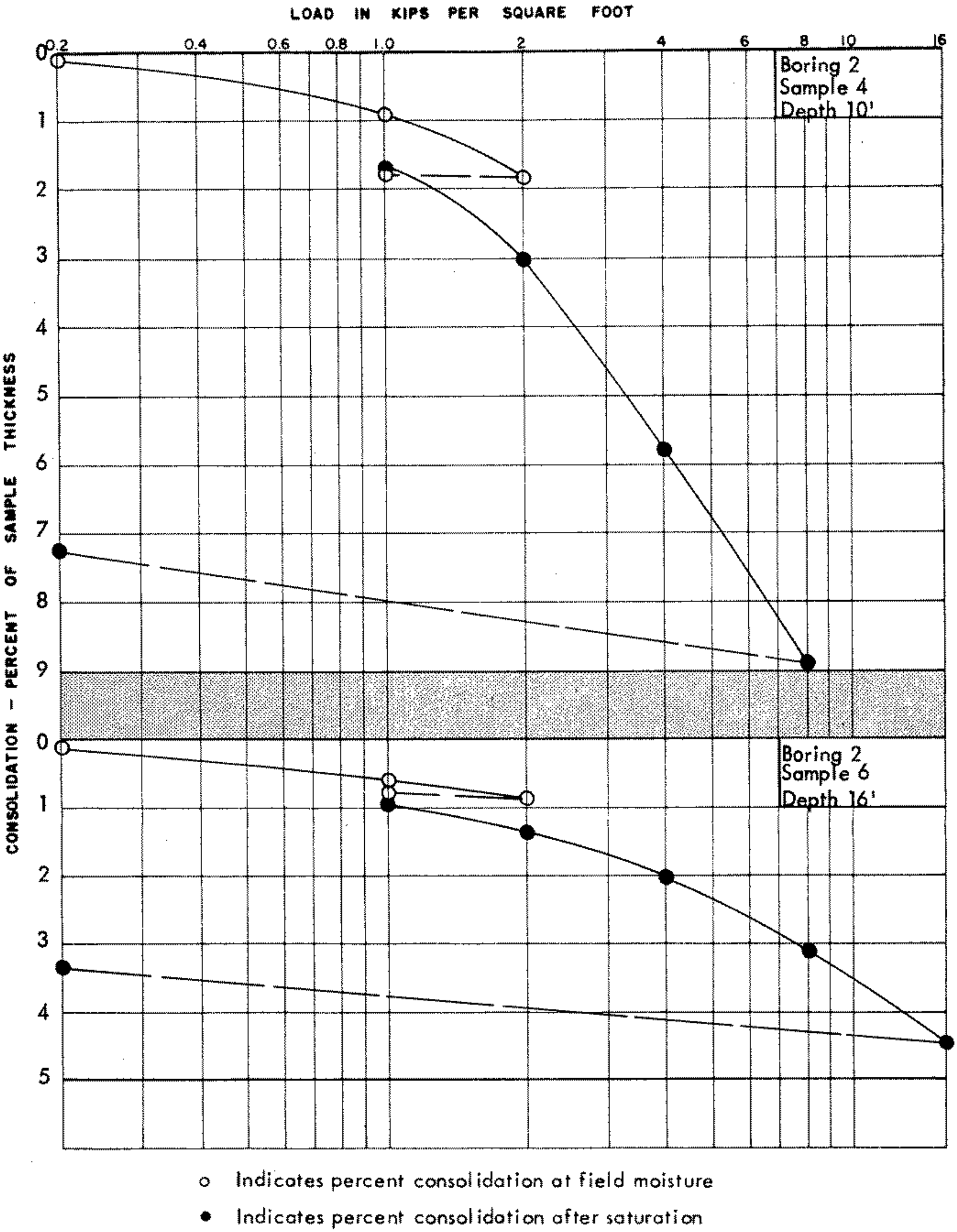
● Indicates percent consolidation after saturation

JOB NAME S.D.G.&.E. - Laurel Substation - Site 1

PROJECT NO.		DRAWING NO.
74-10-2A	BENTON ENGINEERING, INC.	6

JOB NAME S.D.G.&E. - Laurel Substation - Site 1

CONSOLIDATION CURVES



PROJECT NO.		DRAWING NO.
74-10-2A	**BENTON ENGINEERING, INC.**	7

Benton, 1977

G271

Geotechnical Report

BENTON ENGINEERING, INC.
APPLIED SOIL MECHANICS — FOUNDATIONS
6717 CONVOY COURT
SAN DIEGO, CALIFORNIA 92111

PHILIP HENKING BENTON
PRESIDENT - CIVIL ENGINEER

TELEPHONE (714) 565-1955

February 24, 1977

San Diego Gas & Electric Company
P. O. Box 1831
San Diego, California 92112

Attention: Mr. E. J. Brancheau

Subject: Project No. 77-2-17D
Report of Investigation to
Determine the Depths for
Removal of Unsuitable Soils
Proposed Site No. 1 of
Laurel Substation
South of the Intersection of
Vine Street and Kettner Boulevard
San Diego, California

Gentlemen:

This is to report the results of observations made on February 22, 1977 in order to determine the depths of unsuitable soils at the subject site.

The approximate locations of twelve additional exploration pits, Nos. 3 to 14, inclusive, excavated on February 22, 1977 are shown on the attached revised Drawing No. 1, entitled "Location of Exploration Pits."

The exploration pits were excavated using a backhoe supplied by the San Diego Gas & Electric Company. The pits were excavated to depths of 4 to 13 feet below the existing ground surface. A continuous log of the soils encountered in the pits was recorded and are described on pages 3 and 4, of this report.

The soils were visually classified by field identification procedures in accordance with the Unified Soil Classification Chart. A simplified description of this classification system is presented in the attached Appendix A at the end of this report.

It is concluded from the observations in the field exploration pits, that loose fill soils and/or porous alluvial soils exist from depths of 2.7 to 5.7 feet below existing grade in the lower area of the site, and to 0.5 foot in the upper area behind the existing retaining wall. Therefore, it is recommended that these unsuitable soils be removed and recompacted under engineering inspection and testing to assure that the recompacted soils are uniformly compacted to at least 90 percent of maximum dry density. The maximum dry density of the soils are to be determined according to the A.S.T.M. D 1557-70 method of compaction which uses 25 blows of a 10 pound rammer dropping 18 inches on each of 5 layers in a 4 inch diameter 1/30 cubic foot compaction mold.

The recommended depths of removal of the unsuitable fill and porous natural soils are listed below:

Pit No.	Depth of Removal Below Existing Grade in Feet
3	2.7
4	5.7
5	4.3
6	3.5
7	4.5
8	3.2
9	3.5
10	3.5
11	3.0
12	3.0
13	4.0
14	0.5

It is understood that at this time only the unsuitable soils in the lower area of the site are to be removed and recompacted. At a later date the soils behind the existing retaining wall will be removed and the wall cut off at 2.0 feet below the proposed finished grade.

Our office was informed by Mr. George Wiegand that the footing for the existing retaining wall extends approximately 0.5 foot beyond the face of the wall. This would indicate that the footing extends a greater distance underneath the fill behind the wall, therefore, it is our opinion that the unsuitable soils to be excavated in the lower portion of the site can be safely removed outside a slope of one horizontal to one vertical from a point at the top outside edge of the wall footing. The excavation, adjacent to the retaining wall, should be inspected at the time of excavating to verify that the stability of the wall is maintained.

Respectfully submitted,

BENTON ENGINEERING, INC.

By Carl R Johnson Jr
Carl R. Johnson, Civil Engineer
R.C.E. No. 27053

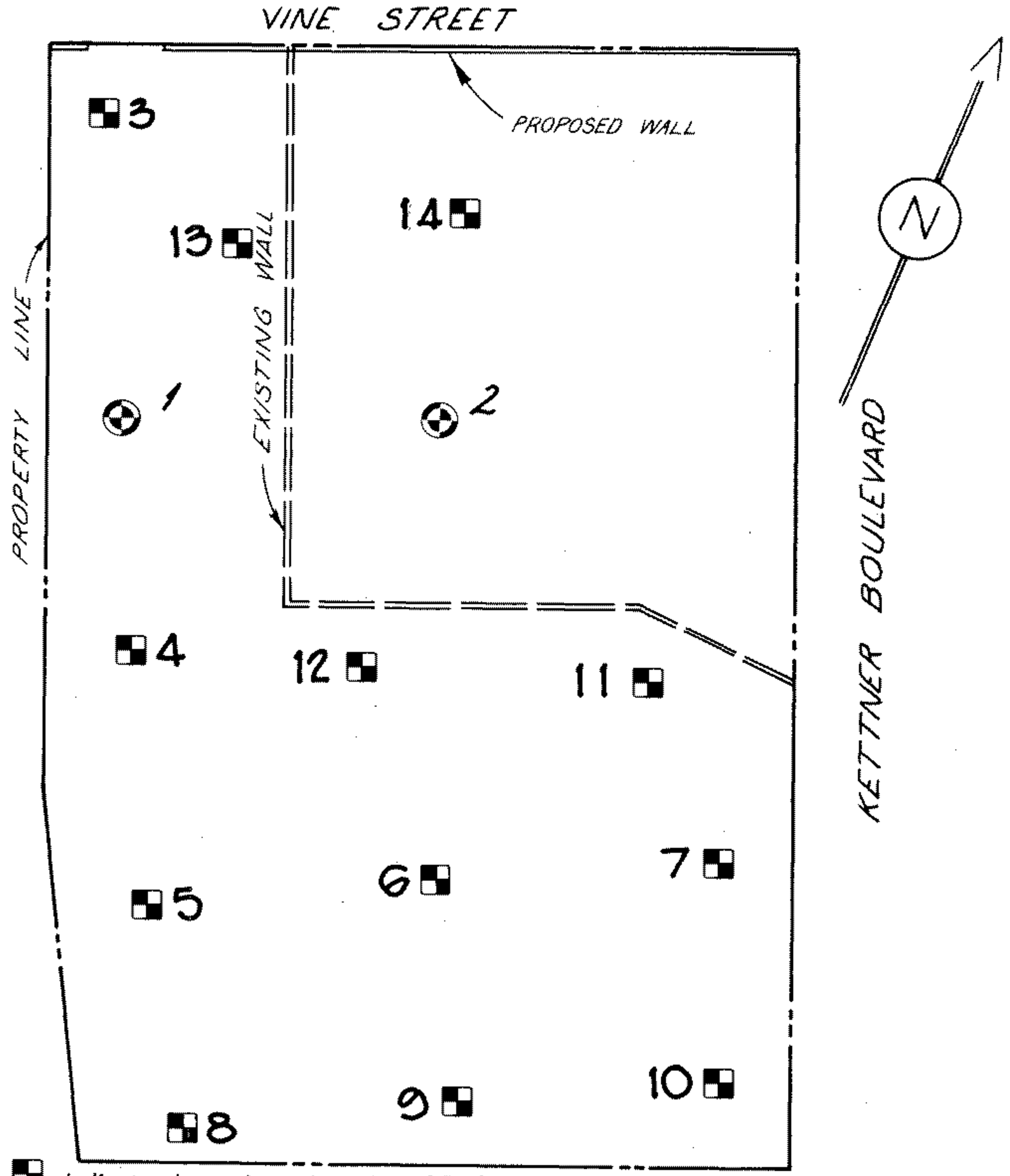
Distr: San Diego Gas & Electric Company
(2) Addressee
(1) Attention: Mr. William A. Davis
(1) Attention: Mr. J. F. Pendleton
(1) Attention: Mr. R. S. Peterson
(1) Attention: Mr. T. M. Nutt
(1) Attention: Ms. P. M. Stanfield

Reviewed by Phil H Benton
Philip H. Benton, Civil Engineer
R.C.E. No. 10332

CRJ/PHB/ew

Pit No.	Depth in Feet	Soil Description
3	0 - 0.3	A. C. Paving
	0.3 - 2.7	Brown, moist, loose fill, silty fine to medium sand
	2.7 - 3.2	Light gray brown, moist, firm, silty fine sand, slightly porous, alluvium
	3.2 -- 4.0	Light gray, moist, firm, slightly silty fine sand
4	0 - 0.2	A. C. Paving
	0.2 - 4.5	Brown, moist, loose to medium compact, fill, silty fine to medium sand
	4.5 - 5.0	Gray brown, moist, loose to medium firm, silty fine sand, alluvium
	5.0 - 5.7	Brown, moist, firm, silty fine sand, very porous, alluvium
	5.7 - 6.5	Brown, moist, firm, silty fine sand, slightly porous, alluvium
5	0-- 0.13	A. C. Paving
	0.13— 3.3	Brown, moist, loose to medium compact fill, silty fine sand
	3.3 - 4.3	Gray brown, moist, medium firm, silty fine sand, porous alluvium
	4.3 - 5.0	Light gray brown, moist, medium firm, silty fine sand
6	0 - 0.25	A.C. Paving
	0.25 - 2.2	Light gray brown, moist, loose to medium compact fill, silty fine sand
	2.2 - 3.5	Light gray brown, moist, loose to medium firm, silty fine sand, very porous alluvium
	3.5 - 4.4	Light gray brown, moist, medium firm, silty fine sand Slightly porous alluvium
7	0 - 0.13	A.C. Paving
	0.13 - 4.5	Brown, moist, firm, silty fine to medium sand with clay binder, porous alluvium
	4.5 - 5.0	Medium brown, moist, firm, silty fine sand, slightly porous alluvium
8	0 - 0.17	A. C. Paving
	0.17 - 3.2	Brown, moist, loose to medium compact fill, silty fine sand
	3.2 - 4.3	Light gray brown, moist, firm silty fine sand, alluvium

Pit No.	Depth in Feet	Soil Description
9	0 - 0.17	A. C. Paving
	0.17 - 1.0	Brown, moist, compact fill, silty fine sand
	1.0 - 3.5	Brown, slightly moist, medium firm, silty fine to medium sand with clay binder, porous alluvium
	3.5 - 5.0	Brown, slightly moist, medium firm, silty fine to medium sand with clay binder, slightly porous alluvium
10	0 - 0.2	A. C. Paving
	0.2 3.5	Brown, slightly moist, medium firm, silty fine to medium sand with clay binder, porous alluvium
	3.5 - 5.5	Brown, moist, medium firm, silty fine to medium sand with clay binder, slightly porous alluvium
11	0 - 0.2	A. C. Paving
	0.2 - 3.0	Brown, slightly moist, medium firm to very firm, silty fine to medium sand with clay binder, porous alluvium
	3.0 - 4.5	Brown, moist, medium firm to very firm, silty fine to medium sand with clay binder, slightly porous alluvium
12	0 - 0.17	A. C. Paving
	0.17 - 3.0	Brown, moist, loose to medium compact fill, silty fine sand
	3.0 - 4.7	Brown, moist, medium firm, silty fine sand
13	0 - 0.17	A. C. Paving
	0.17 - 1.0	Brown, slightly moist, loose fill, silty fine sand
	1.0 - 1.5	Brown, slight moist, loose to medium firm, silty fine sand
	1.5 - 4.0	Brown, slightly moist, medium firm, silty fine to medium sand with clay binder, very porous alluvium
	4.0 - 4.5	Brown, moist, medium firm, silty fine to medium sand with clay binder, slightly porous alluvium
14	0 - 0.5	Yellow brown, moist, loose, fill, silty fine sand
	0.5 - 6.0	Yellow brown, moist, medium compact, fill, silty fine sand with concrete debris
	6.0 - 10.5	Gray and dark gray, moist, medium compact fill, silty fine sand mottled with clayey fine sand
	10.5 - 13.0	Brown, moist, compact, silty fine sand



▚ Indicates Approximate Location of Exploration Pit.

⊕ Indicates Approximate Location of Test Boring, Our Report 74-10-2A - dated 10-9-74

Date: 2-23-1977 Drawn: K.L. Green Scale: 1"=40'

LOCATION OF EXPLORATION PITS

LAUREL SUBSTATION - SITE NO. 1
SAN DIEGO GAS & ELECTRIC COMPANY
VINE STREET AND KETTNER BOULEVARD
SAN DIEGO, CALIFORNIA

PROJECT NO.		DRAWING NO.
77-2-17D	**BENTON ENGINEERING, INC.**	1

APPENDIX B

Geotechnical Borings

Geosyntec consultants

10875 Rancho Bernardo Rd, Suite 200
San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

PROJECT Vine Substation
PROJECT LOCATION San Diego, California
PROJECT NUMBER SC0368-30

KEY SHEET - CLASSIFICATIONS AND SYMBOLS

GS FORM:
KEY 09/99

EMPIRICAL CORRELATIONS WITH STANDARD PENETRATION RESISTANCE N VALUES *

	N VALUE * (BLOWS/FT)	CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT)
FINE GRAINED SOILS	0 - 2	VERY SOFT	<0.25
	3 - 4	SOFT	0.25 - 0.50
	5 - 8	FIRM	0.50 - 1.00
	9 - 15	STIFF	1.00 - 2.00
	16 - 30	VERY STIFF	2.00 - 4.00
	31 - 50	HARD	>4.00
	>50	VERY HARD	

	N VALUE * (BLOWS/FT)	RELATIVE DENSITY
COARSE GRAINED SOILS	0 - 4	VERY LOOSE
	5 - 10	LOOSE
	11 - 30	MEDIUM DENSE
	31 - 50	DENSE
	>50	VERY DENSE

* ASTM D 1586; NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 IN. O.D., 1.4 IN. I.D. SAMPLER ONE FOOT.

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

MAJOR DIVISIONS			SYMBOLS	DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL COARSER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVELS LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES APPRECIABLE AMOUNT OF FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO.4 SIEVE	CLEAN SANDS LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES APPRECIABLE AMOUNT OF FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL FINER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENT

NOTE: DUAL SYMBOLS USED FOR BORDERLINE CLASSIFICATIONS

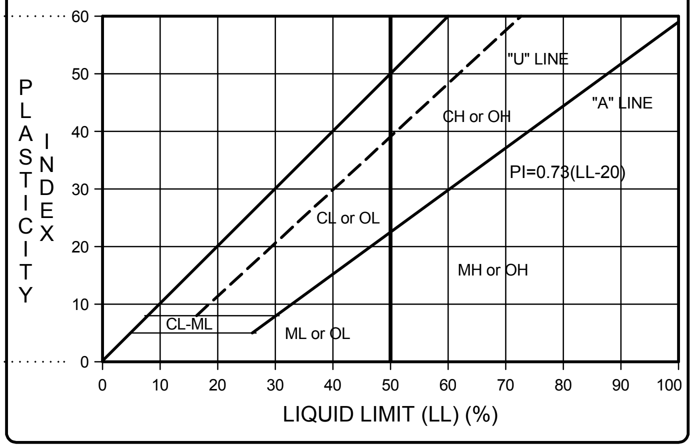
PARTICLE SIZE IDENTIFICATION

BOULDERS	>300 mm
COBBLES	75 - 300 mm
GRAVEL: COARSE	19.0 - 75 mm
GRAVEL: FINE	4.75 - 19 mm
SAND: COARSE	2.00 - 4.75 mm
SAND: MEDIUM	0.425 - 2.00 mm
SAND: FINE	0.075 - 0.425 mm
SILT	0.075 - 0.002 mm
CLAY	<0.002 mm

WELL GRADED - HAVING WIDE RANGE OF GRAIN SIZES AND APPRECIABLE AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES

POORLY GRADED - PREDOMINANTLY ONE GRAIN SIZE, OR HAVING A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING

PLASTICITY CHART



OTHER MATERIAL SYMBOLS

Siltstone
Sandstone
Siltstone/Claystone
Claystone
Shale
Siltstone/Sandstone
Conglomerate
Granitic

Sand
Silt
Silty Sand
Alluvium
Artificial Fill
Debris Fill
Asphalt
Cement

WELL SYMBOLS

GRANULAR BENTONITE
BENTONITE CEMENT GROUT
FILTER PACK
CONCRETE
NATIVE/ SLOUGH
CENTRAL-IZER

SAMPLER AND OTHER SYMBOLS

GRAB SAMPLE
SPLIT SPOON
STANDARD PENETRATION TEST (SPT)
SHELBY TUBE
CALIFORNIA SAMPLER
BULK SAMPLE

▽ Water Level at Time Drilling, or as Shown
▼ Static Water Level
MSL: Mean Sea Level
MC: Moisture Content
FC: Fines Content
DD: Dry Density
EI: Expansion Index
SA: Sieve Analysis
PI: Plasticity Index
LL: Liquid Limit
c: Cohesion
K: Hydraulic Conductivity
Phi: Friction Angle

KEY-GEOTECH SC0368-30.GPJ SAN DIEGO GINT LIBRARY.GLB 8/12/13

Geosyntec consultants

10875 Rancho Bernardo Rd, Suite 200
San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

GS FORM:
BORE 1/99

BOREHOLE RECORD

BORING B-1 **SHEET** 1 **OF** 2
START DATE 7/10/2013 **ELEVATION** 44 **FT MSL**
FINISH DATE 7/10/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOW COUNTS	SAMPLES % RECOVERY	SAMPLES N-VALUE	TIME	COMMENTS
	FILL 3" asphaltic concrete over moist, yellowish brown (10 YR 5/6) silty fine sand with trace clay.									
				B-1-1A	☰					CORR
	COLLUVIUM: (Qc) Medium dense, moist dark brown (10 YR 3/3) silty fine sand [SM] with angular to sub angular gravels									
5			39							
				B-1-1	◸		100	20		Cobble at 4.8 feet. MC (8) DD (108) EI (31)
	OLD PARALIC DEPOSITS (Qop6) Very stiff, moist, brown (7.5 YR 4/3) fine sandy silt [ML] with some soil development									
10			34							
				B-1-2	◢	10/10/9	100	19		PID (0.0) SA (62)
	Dense, moist, dark brown (10 YR 3/5) silty to clayey fine sand [SM/SC]									
15			29							
				B-1-3	◸	10/14/19	100	33		MC (15) DD (109)
20	Medium dense, moist, dark yellowish brown to light gray (10 YR 4/6 to 6/2) silty fine sand [SM]		24							
				B-1-4	◢	6/7/8	100	15		PID (0.0) WA (63)
25	Increase in moisture, becomes wet, dark reddish brown (10 YR 3/2)		19							
				B-1-5	◸	2/7/8	10	15		
30			14							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73917
LONGITUDE 117.17899
ANGLE Vertical
BEARING ------
PRINTED August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

COORDINATE SYSTEM:

SEE KEY SHEET FOR SYMBOLS AND ABBREVIATIONS

Geosyntec consultants

10875 Rancho Bernardo Rd, Suite 200
San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

GS FORM: BORE 1/99

BOREHOLE RECORD

BORING B-1 **SHEET** 2 **OF** 2
START DATE 7/10/2013 **ELEVATION** 44 **FT MSL**
FINISH DATE 7/10/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOW COUNTS	SAMPLES % RECOVERY	SAMPLES N-VALUE	TIME	COMMENTS
	Becomes dark yellowish brown (10 YR 3/6)			B-1-6	◢	4/7/6	0	13		Description made from shoe.
35			9							Sample wet.
				B-1-7	◿	6/13/11	0	24		
40			4							(Slough) Borehole caved to 24.3 feet after augers pulled.
				B-1-8	◢	3/2/1	100	3		
	Bottom of boring at 41.5 feet. Boring backfilled with approximately 10.7 cubic feet of bentonite grout, 0.2 cubic feet of bentonite chips, and 0.2 cubic feet of concrete.									
45			-1							
50			-6							
55			-11							
60			-16							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73917
LONGITUDE 117.17899
ANGLE Vertical
BEARING ------
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REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

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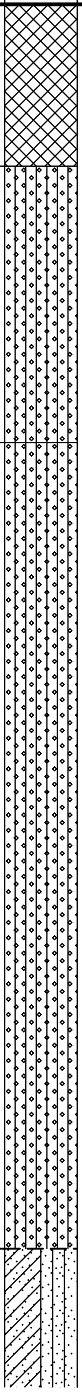
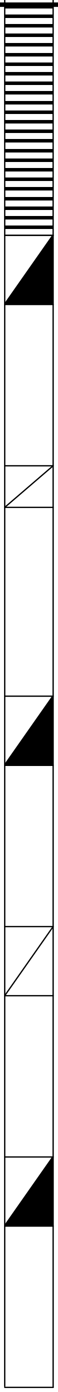
San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

GS FORM: BORE 1/99

BOREHOLE RECORD

BORING B-2 **SHEET** 1 **OF** 2
START DATE 7/10/2013 **ELEVATION** 36 FT MSL
FINISH DATE 7/11/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30





DEPTH (ft)	MATERIAL DESCRIPTION	ELEVATION (ft)	SAMPLES NUMBER	TYPE	BLOW COUNTS	% RECOVERY	N-VALUE	TIME	COMMENTS
	FILL: 1" asphaltic concrete over moist brownish yellow to dark yellowish brown (10 YR 6/6 to 4/4) fine sandy silt to silty sand with clay. Contains angular to sub angular gravels.								Hand auger to 5 feet.
			B-2-1A						COMP
	COLLUVIUM: (Qc) Loose, moist, dark brown (10 YR 3/3) silty fine sand [SM] with trace clay								
5		31							PID (0.0)
			B-2-1		5/3/4	100	7		
10	OLD PARALIC DEPOSITS: (Qop6) Very dense to hard, moist, dark yellowish brown (10 YR 3/4) fine sandy silt to silty fine sand [SM/ML]	26	B-2-2		26/50 for 5"	100	50		Hard drilling at 10 feet. MC (13) DD (103)
15	Decrease in density, becomes stiff to medium dense	21							PID (0.0)
			B-2-3		5/6/9	100	15		
20	Increase in moisture, becomes dark yellowish brown (10 YR 3/6) with increase in clay	16	B-2-4		4/9/11	100	20		DD (114) MC (16) WA (57)
25		11							
			B-2-5		4/4/5	0	9		
	Medium dense, wet, dark yellowish brown (10 YR 3/6) clayey to silty fine sand [SM/SC]								
30		6							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73893
LONGITUDE 117.17937
ANGLE Vertical
BEARING ------
PRINTED August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

COORDINATE SYSTEM:

SEE KEY SHEET FOR SYMBOLS AND ABBREVIATIONS

Geosyntec consultants

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San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

GS FORM:
BORE 1/99

BOREHOLE RECORD

BORING B-2 **SHEET** 2 **OF** 2
START DATE 7/10/2013 **ELEVATION** 36 **FT MSL**
FINISH DATE 7/11/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	BLOW COUNTS	% RECOVERY	N-VALUE	TIME	COMMENTS
				B-2-6	◿	3/3/11	100	14		Slough? MC (20) DD (114) PI (NP) SA (48)
35			1	B-2-7	◢	2/2/2	100	4		Slough?
	Hard, wet, strong brown (7.5 YR 4/6) fine sandy silt [ML]									
40			-4	B-2-8	◿	12/19/14	100	33		MC (19) DD (115) WA (66)
45			-9	B-2-9	◢	1/2/3	100	5		Slough?
	Dense, moist to wet, dark yellowish brown to brownish yellow (10 YR 5/6 to 4/6) silty fine to very fine sand [SM]									
50			-14	B-2-10	◿	14/17/24	100	41		MC (23) DD (108)
	Bottom of boring at 51.5 feet. Boring backfilled with approximately 10.7 cubic feet of bentonite grout and 0.3 cubic feet of concrete.									
55			-19							
60			-24							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73893
LONGITUDE 117.17937
ANGLE Vertical
BEARING ------
PRINTED August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

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Fax: (858) 674-6586

GS FORM:
BORE 1/99

BOREHOLE RECORD

BORING B-3 **SHEET** 1 **OF** 2
START DATE 7/10/2013 **ELEVATION** 43 **FT MSL**
FINISH DATE 7/11/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOW COUNTS	SAMPLES % RECOVERY	SAMPLES N-VALUE	TIME	COMMENTS
	FILL: 3" asphaltic concrete over moist, dark yellowish brown (10 YR 5/6) fine sandy silt									Hand auger to 5 feet.
				B-3-1A						WA (54) R-value
	OLD PARALIC DEPOSITS: (Qop6) Very dense, moist, dark yellowish brown (10 YR 4/6) clayey fine sand [SC/CL] with few angular to sub angular gravels. Contains localized carbonate cementation.									
5			38							MC (13) DD (116)
				B-3-1		12/23/34	100	57		
	Increase in clay becomes sandy lean clay									
10	Becomes dark yellowish brown (10 YR 3/6)		33							PID (0.0) MC (11) WA (61)
				B-3-2		12/18/15	100	33		
15			28							MC (11) DD (113)
	Increase in oxidation; contains angular to sub angular gravels			B-3-3		14/16/24	100	40		
	Medium dense, moist, mottled strong brown to pale brown (7.5 YR 4/6 to 10 YR 6/3) silty to clayey fine sand [SC/SM]									
20			23							PID (0.0) WA (56)
				B-3-4		10/11/13	100	24		
25			18							MC (15) DD (121) SG (2.7)
				B-3-5		6/10/18	100	28		
30			13							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73851
LONGITUDE 117.17845
ANGLE Vertical
BEARING ------
PRINTED August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

COORDINATE SYSTEM:

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Tel: (858) 674-6559
Fax: (858) 674-6586

GS FORM:
BORE 1/99

BOREHOLE RECORD

BORING B-3 **SHEET** 2 **OF** 2
START DATE 7/10/2013 **ELEVATION** 43 **FT MSL**
FINISH DATE 7/11/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	BLOW COUNTS	% RECOVERY	N-VALUE	TIME	COMMENTS
	Hard to medium stiff, wet, dark yellowish brown (10 YR 4/6) fine sandy lean clay [CL]	▨		B-3-6	◣	5/18/14	100	32		WA (53) LL (23) PL (17) PI (6)
	Decrease in sand content	▨								
		▨								Easy drilling @ 34'.
35		▨	8	B-3-7	◿	2/5/6	100	11		WA (62) LL (27) PL (18) PI (9)
40	Dense, wet, yellowish brown (10 YR 5/6) poorly graded fine sand [SP]	░	3	B-3-8	◣	4/13/32	100	45		
	Very stiff, wet, mottled light grayish brown to brownish yellow (10 YR 6/2 to 6/8) and light yellowish brown (2.5 YR 6/4) fat clay [CH] with trace fine sand.	▨								Hard drilling @ 44'.
45		▨	-2	B-3-9	◣	3/7/9	100	16		SA (95) LL (69) PL (24) PI (45)
50		▨	-7	B-3-10	◣	4/8/7	100	15		
	Bottom of boring at 51.5 feet. Boring backfilled with approximately 10.7 cubic feet of bentonite grout, 0.2 cubic feet of bentonite chips, and 0.3 cubic feet of concrete.									
55			-12							
60			-17							

CONTRACTOR Tri-County Drilling **LATITUDE** 32.73851
EQUIPMENT CME-85 **LONGITUDE** 117.17845
DRILL MTHD HSA **ANGLE** Vertical
DIAMETER 8 inches **BEARING** ------
LOGGER J.Warner **REVIEWER** A.Greene **PRINTED** August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

COORDINATE SYSTEM:

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San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

GS FORM:
BORE 1/99

BOREHOLE RECORD

BORING B-4 **SHEET** 1 **OF** 2
START DATE 7/11/2013 **ELEVATION** 38 **FT MSL**
FINISH DATE 7/11/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	BLOW COUNTS	% RECOVERY	N-VALUE	TIME	COMMENTS
	FILL: 1" asphaltic concrete over moist, dark yellowish brown (10 YR 3/4 to 4/6) silty fine sand with sub rounded gravels.			B-4-1A						Hand auger to 5 feet. Gravels 2.5"x1.5"x1.0"
	COLLUVIUM: (Qc) Very loose, moist, dark brown (10 YR 3/3) silty fine sand [SM] with trace clay									
5			33	B-4-1		1/2/1	100	3		PID (0.0) CORR
	OLD PARALIC DEPOSITS: (Qop6) Hard, moist, yellowish brown (10 YR 5/4) lean silt [ML] with trace sand. Contains carbonate cementation.									
10			28	B-4-2		12/15/17	100	32		WA (66)
	Very stiff to hard, moist, dark yellowish brown (10 YR 3/4) fine sandy lean clay [CL]									
15			23	B-4-3		5/8/11	100	19		PID (0.0) WA (59)
20			18	B-4-4		9/17/22	100	39		MC (14) DD (118) WA (57) LL (29) PL (16) PI (13)
25	Stiff, wet, dark yellowish brown (10 YR 3/6) lean clay [CL] with fine sand.		13	B-4-5		3/6/8	100	14		MC (14) WA (63) LL (30) PL (14) PI (16)
30			8							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73828
LONGITUDE 117.17879
ANGLE Vertical
BEARING ------
PRINTED August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

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GS FORM:
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BOREHOLE RECORD

BORING B-4 **SHEET** 2 **OF** 2
START DATE 7/11/2013 **ELEVATION** 38 **FT MSL**
FINISH DATE 7/11/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOW COUNTS	SAMPLES % RECOVERY	SAMPLES N-VALUE	TIME	COMMENTS
	Stiff to very stiff, wet, dark yellowish brown (10 Yr 4/6) to brown (7.5 YR 4/4) fine sandy lean clay [CL].	▨		B-4-6	◢	6/5/7	100	12		WA (58) LL (28) PL (14) PI (15)
35			3	B-4-7	◢	6/11/11	100	22		WA (50) LL (23) PL (14) PI (9)
	Dense, moist, dark yellowish brown (10 Yr 4/6) to strong brown (7.5 YR 4/6) silty fine sandy silt [ML]	▥								
40			-2	B-4-8	◢	12/17/17	100	34		Harder drilling. WA (69)
	Bottom of boring at 41.5 feet. Boring backfilled with approximately 10.7 cubic feet of bentonite grout, 1.0 cubic feet of bentonite chips, and 0.3 cubic feet of concrete.									
45			-7							
50			-12							
55			-17							
60			-22							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73828
LONGITUDE 117.17879
ANGLE Vertical
BEARING ------
PRINTED August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

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GS FORM:
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BOREHOLE RECORD

BORING B-5 **SHEET** 1 **OF** 2
START DATE 7/11/2013 **ELEVATION** 40 **FT MSL**
FINISH DATE 7/11/2013
PROJECT Vine Substation
LOCATION San Diego, California
PROJECT NUMBER SC0368-30

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES					TIME	COMMENTS
				NUMBER	TYPE	BLOW COUNTS	% RECOVERY	N-VALUE		
	FILL: 1" asphaltic concrete over moist, light olive brown (25 YR 5/4) silty fine sand with angular to sub angular gravels.	▨			▤					Hand auger to 5 feet.
				B-5-1A	▤					
	Decrease in gravels at 3.6 feet									
5	Becomes dark yellowish brown (10 YR 4/6) silt. Contains metal debris at 6.0 feet.		35	B-5-1	◣	4/3/5	100	8		PID (0.0) MC (8) EI (51)
	COLLUVIUM: (Qc) Medium dense, moist, mottled dark brown (10 YR 3/3) silty fine sand [SM]. Porous.	⁘								
10			30	B-5-2	◿	11/11/11	100	22		Porous soil. MC (12) DD (106)
	OLD PARALIC DEPOSITS: (Qop6) Medium dense, stiff, moist, dark yellowish brown (10 YR 3/4) fine sandy silt to silty fine sand [ML/SM] with localized manganese nodules.	▥								
15			25	B-5-3	◣	6/7/5	100	12		
20	Becomes hard to very dense		20	B-5-4	◿	11/15/17	100	32		Difficult drilling @ 20'. MC (14) DD (119) WA (59)
	Stiff, moist to wet, dark yellowish brown (10 YR 3/4) lean clay [CL] with fine sand.	▧								
25			15	B-5-5	◣	7/5/9	100	14		WA (56) LL (27) PL (17) PI (10)
30			10							

CONTRACTOR Tri-County Drilling
EQUIPMENT CME-85
DRILL MTHD HSA
DIAMETER 8 inches
LOGGER J.Warner **REVIEWER** A.Greene

LATITUDE 32.73874
LONGITUDE 117.17887
ANGLE Vertical
BEARING ------
PRINTED August 12, 2013

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

COORDINATE SYSTEM:

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Geosyntec consultants

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Tel: (858) 674-6559
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GS FORM:
BORE 1/99

BOREHOLE RECORD

BORING **B-5** **SHEET 2 OF 2**
START DATE **7/11/2013** **ELEVATION** **40** **FT MSL**
FINISH DATE **7/11/2013**
PROJECT **Vine Substation**
LOCATION **San Diego, California**
PROJECT NUMBER **SC0368-30**

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES					TIME	COMMENTS
				NUMBER	TYPE	BLOW COUNTS	% RECOVERY	N-VALUE		
	Very stiff, wet, dark yellowish brown (10 YR 3/4) fine to medium sandy lean clay [CL].	▨		B-5-6	◿	12/14/13	100	27		WA (53) LL (23) PL (16) PI (7)
35		▨	5							
	Contains fine to coarse gravels	▨		B-5-7	◢	5/10/18	100	28		WA (56) LL (25) PL (16) PI (9)
	Hard to very dense, wet, light yellowish brown (2.5 YR 6/4) to light olive brown (2.5 YR 5/3) lean clay to clayey fine sand [CL/SC].	▒								
40		▒	0	B-5-8	◿	4/9/42	100	51		Difficult drilling @ 39'. MC (22) DD (103) WA (57)
	Bottom of boring at 41.5 feet. Boring backfilled with approximately 13.4 cubic feet of bentonite grout and 0.3 cubic feet of concrete.									
45			-5							
50			-10							
55			-15							
60			-20							

CONTRACTOR **Tri-County Drilling**
EQUIPMENT **CME-85**
DRILL MTHD **HSA**
DIAMETER **8 inches**
LOGGER **J.Warner** **REVIEWER** **A.Greene**

LATITUDE **32.73874**
LONGITUDE **117.17887**
ANGLE **Vertical**
BEARING **------**
PRINTED **August 12, 2013**

REMARKS: Approximate lat/long estimated from Google Earth. Approximate elevation estimated from site survey data provided by SDG&E.

COORDINATE SYSTEM:

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APPENDIX C

Cone Penetration Test Soundings

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

Kettner Substation Relocation
Kettner Blvd. & Vine Street
San Diego, CA
July 30, 2013 - August 2, 2013

Prepared for:

Ms. Jennifer Nevius
Geosyntec Consultants, Inc.
10875 Rancho Bernardo Road, Ste 200
San Diego, CA 92127
Office (858) 674-6559 / Fax (858) 674-6586

Prepared by:

KTE

KEHOE TESTING & ENGINEERING
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Huntington Beach, CA 92649-1518
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www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY
OF
CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Kettner Substation Relocation project located at Kettner Blvd. & Vine Street in San Diego, California. The work was performed by Kehoe Testing & Engineering (KTE) on July 30, 2013-August 2, 2013. The scope of work was performed as directed by Geosyntec Consultants, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at 26 locations to determine the soil lithology. Groundwater measurements and hole collapse depths provided in **TABLE 2.1** are for information only. The readings indicate the apparent depth to which the hole is open and the apparent water level (if encountered) in the CPT probe hole at the time of measurement upon completion of the CPT. KTE does not warranty the accuracy of the measurements and the reported water levels may not represent the true or stabilized groundwater levels.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
C-1	80	Refusal, groundwater @ 25.0 ft
C-2	74	Refusal, groundwater @ 25.0 ft
C-3	75	Groundwater @ 21.0 ft
C-4	73	Refusal, groundwater @ 24.0 ft
C-5	70	Refusal, groundwater @ 23.0 ft
C-6	66	Refusal, groundwater @ 23.0 ft
C-7	64	Refusal, groundwater @ 23.0 ft
C-8	61	Refusal, groundwater @ 23.0 ft
C-9	48	Refusal, groundwater @ 24.0 ft
C-10	93	Groundwater @ 24.0 ft
C-11	89	Refusal, groundwater @ 21.0 ft
C-12	45	Refusal, groundwater @ 21.0 ft
C-12B	45	Refusal, groundwater @ 22.0 ft
C-13	43	Refusal, groundwater @ 21.0 ft
C-13A	80	No cave depth taken
C-14	90	Groundwater @ 24.0 ft
C-15	80	Groundwater @ 24.0 ft

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
C-16	90	Groundwater @ 27.0 ft
C-17	100	Groundwater @ 27.0 ft
C-18	4	Refusal, hole open to 4.0 ft (dry)
C-18A	4	Refusal, hole open to 4.0 ft (dry)
C-18C	80	Groundwater @ 27.0 ft
C-19	80	Groundwater @ 26.0 ft
C-20	58	Refusal, groundwater @ 25.5 ft
C-21	80	Groundwater @ 25.0 ft
C-22	100	Groundwater @ 25.0 ft

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

At location C-1, C-8, C-10, C-15, C-16 & C-22, shear wave measurements were obtained at approximately 10-foot intervals. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the attached CPT Classification Chart (Robertson) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Tables of basic CPT output from the interpretation program CPeT-IT are provided for CPT data averaged over one foot intervals in the Appendix. Spreadsheet files of the averaged basic CPT output and averaged estimated geotechnical parameters are also included for use in further geotechnical analysis. We recommend a geotechnical engineer review the assumed input parameters and the calculated output from the CPeT-IT program. A summary of the equations used for the tabulated parameters is provided in the Appendix.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

08/15/13-ag-3997

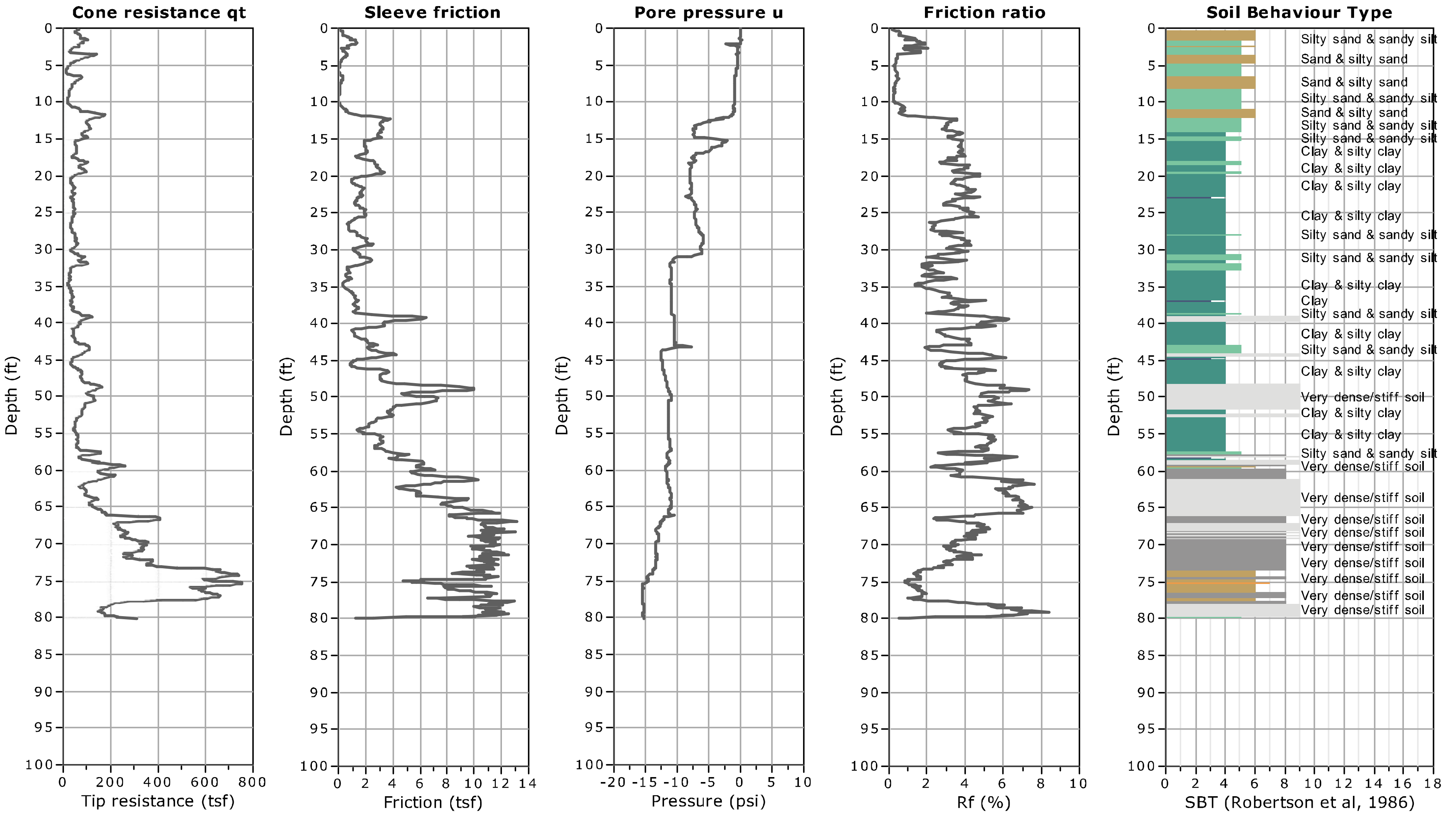
APPENDIX

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: C-1

Total depth: 80.10 ft, Date: 7/30/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

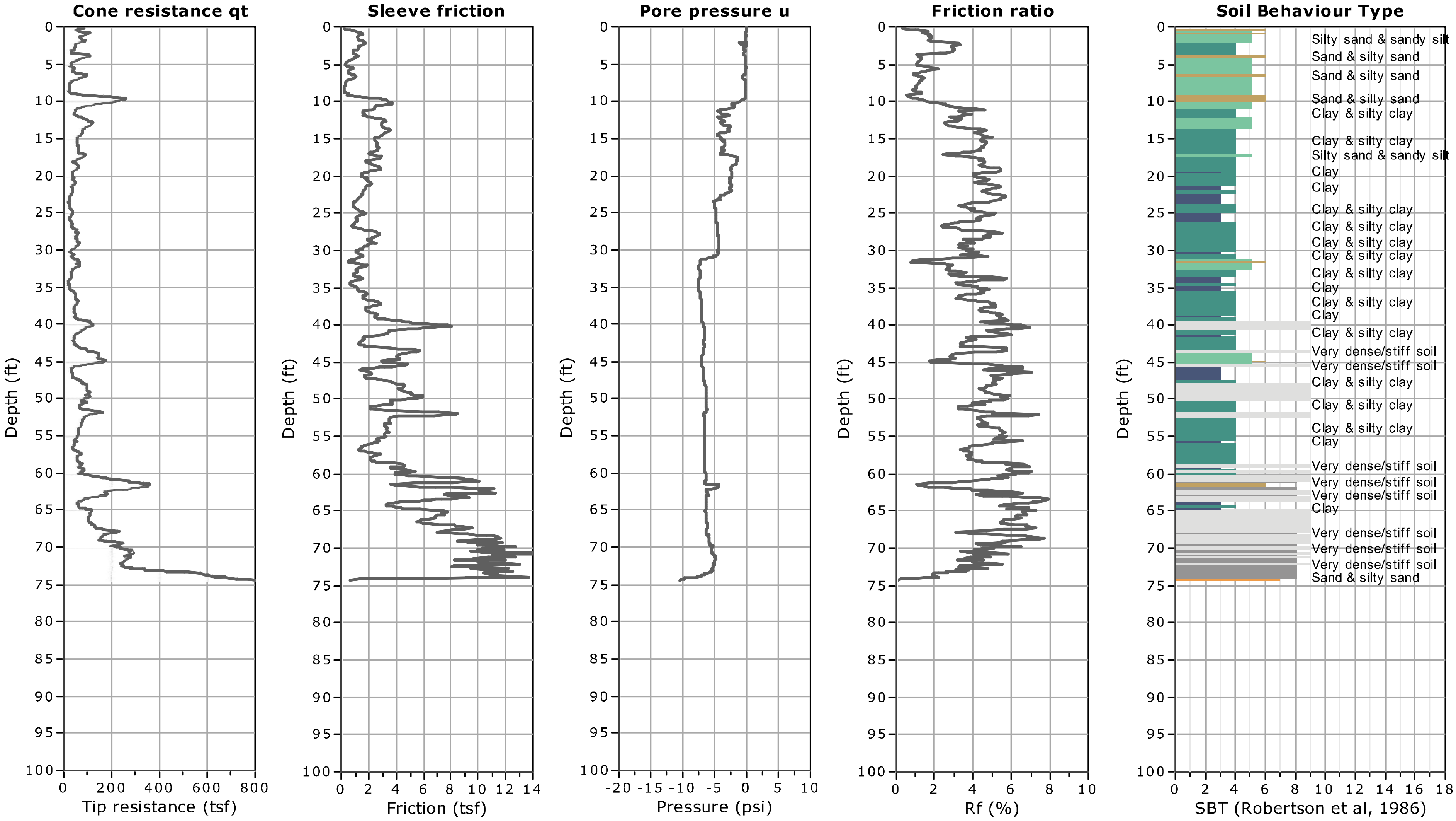
CPT: C-2

Total depth: 74.37 ft, Date: 7/30/2013

Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**

Location: **Kettner Blvd. & Vine St. San Diego, CA**

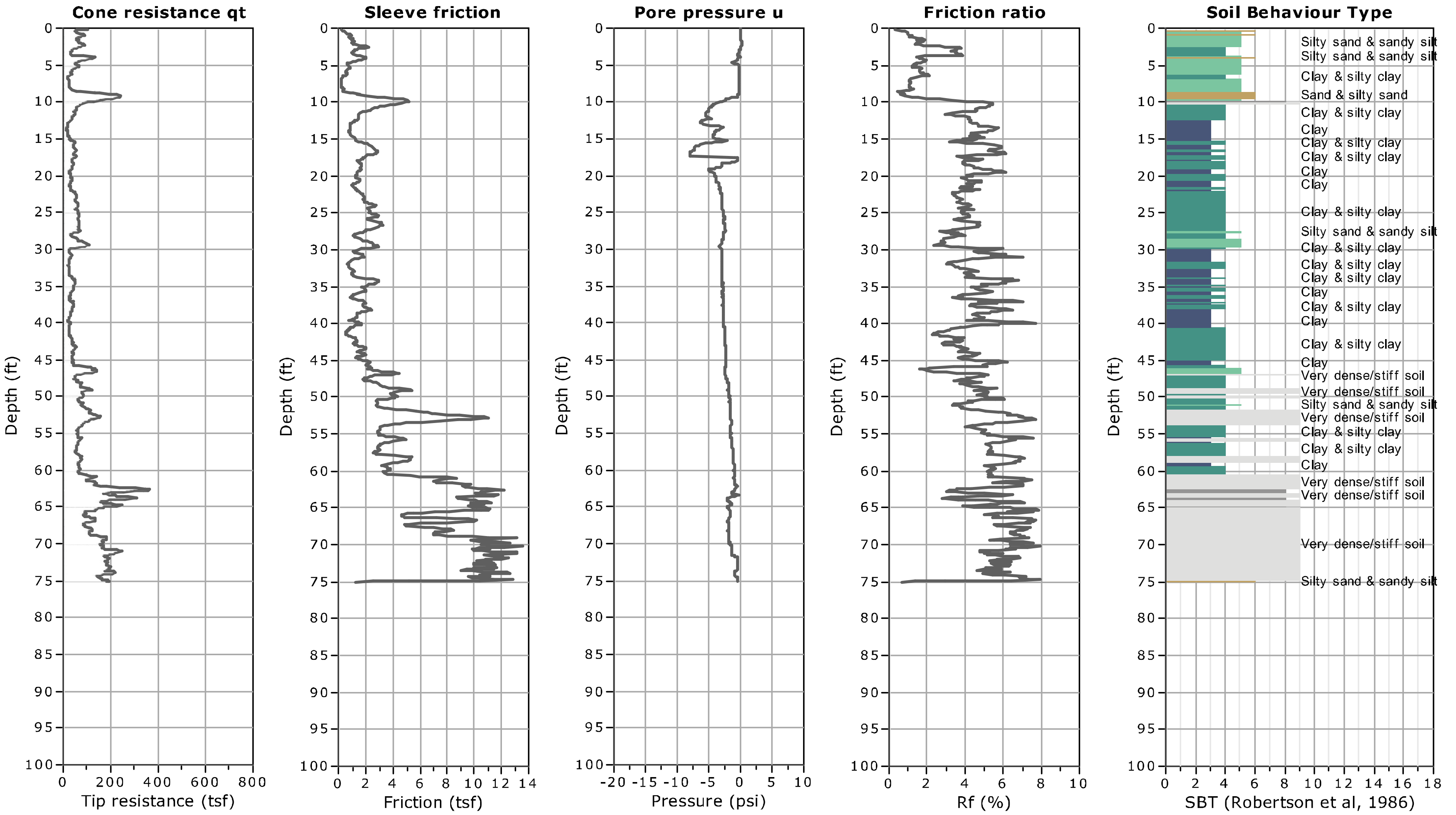


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: C-3

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 75.14 ft, Date: 7/30/2013
Cone Type: Vertek



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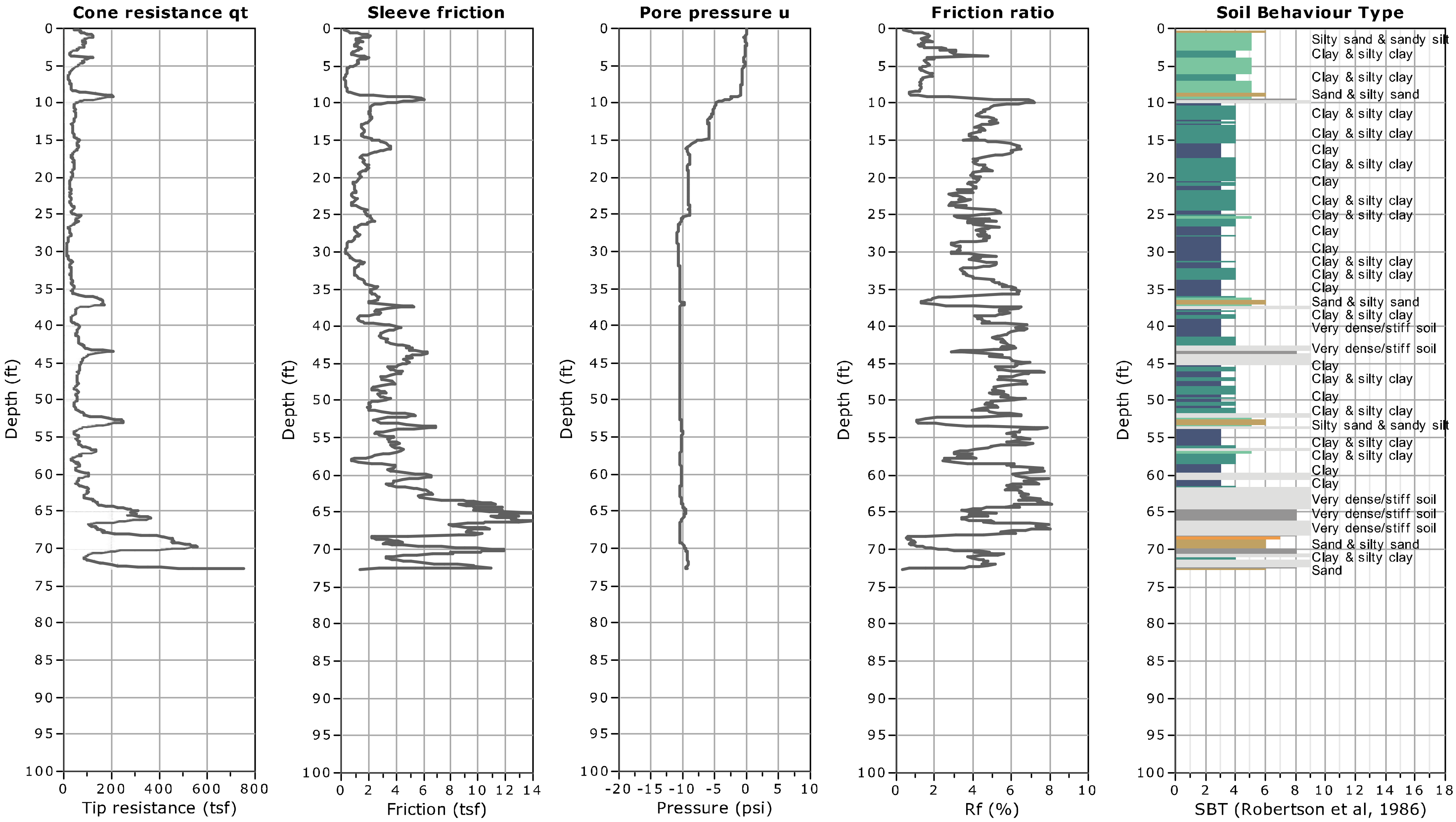
CPT: C-4

Total depth: 72.80 ft, Date: 7/30/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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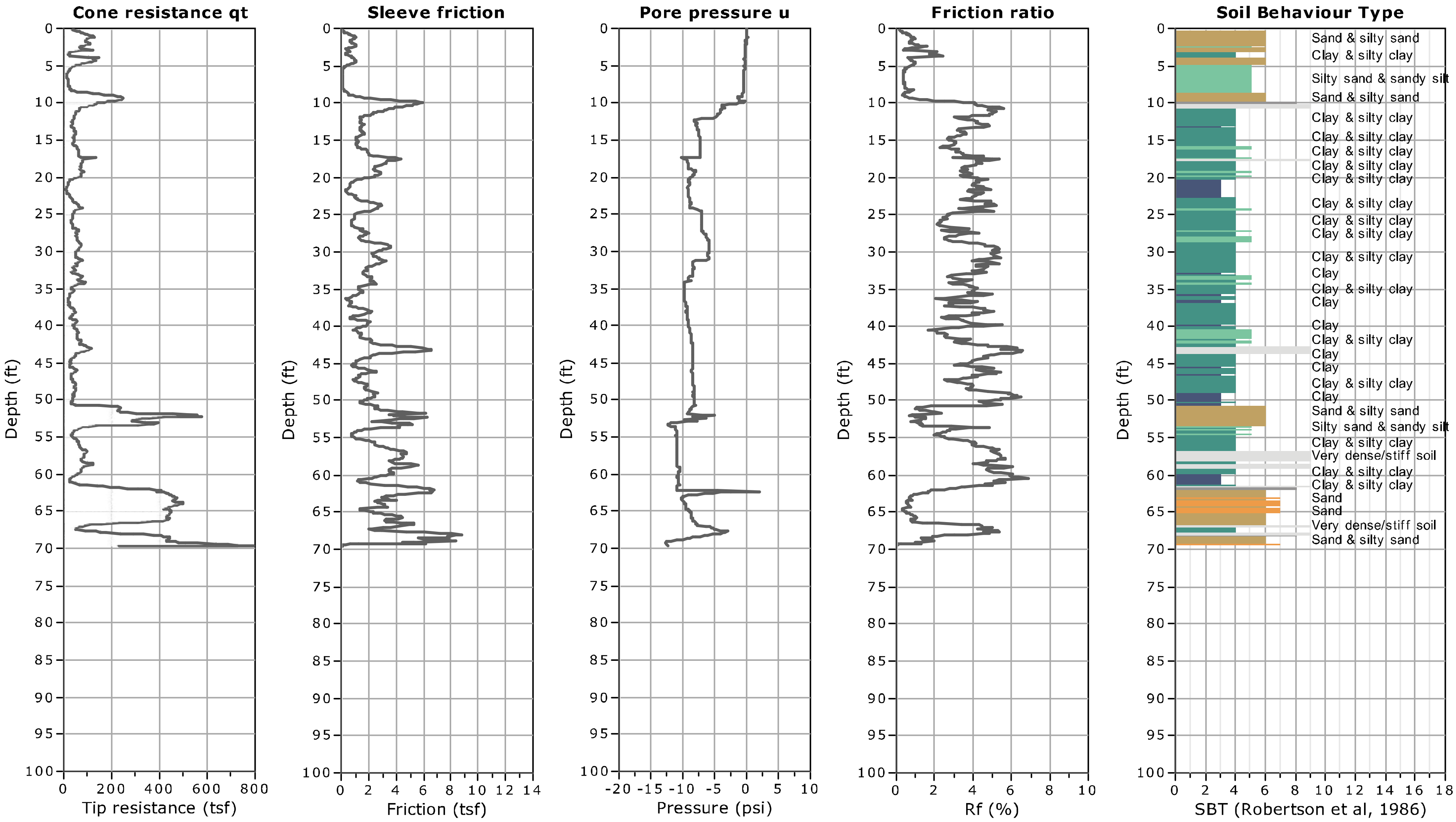
CPT: C-5

Total depth: 69.63 ft, Date: 7/30/2013

Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**

Location: **Kettner Blvd. & Vine St. San Diego, CA**



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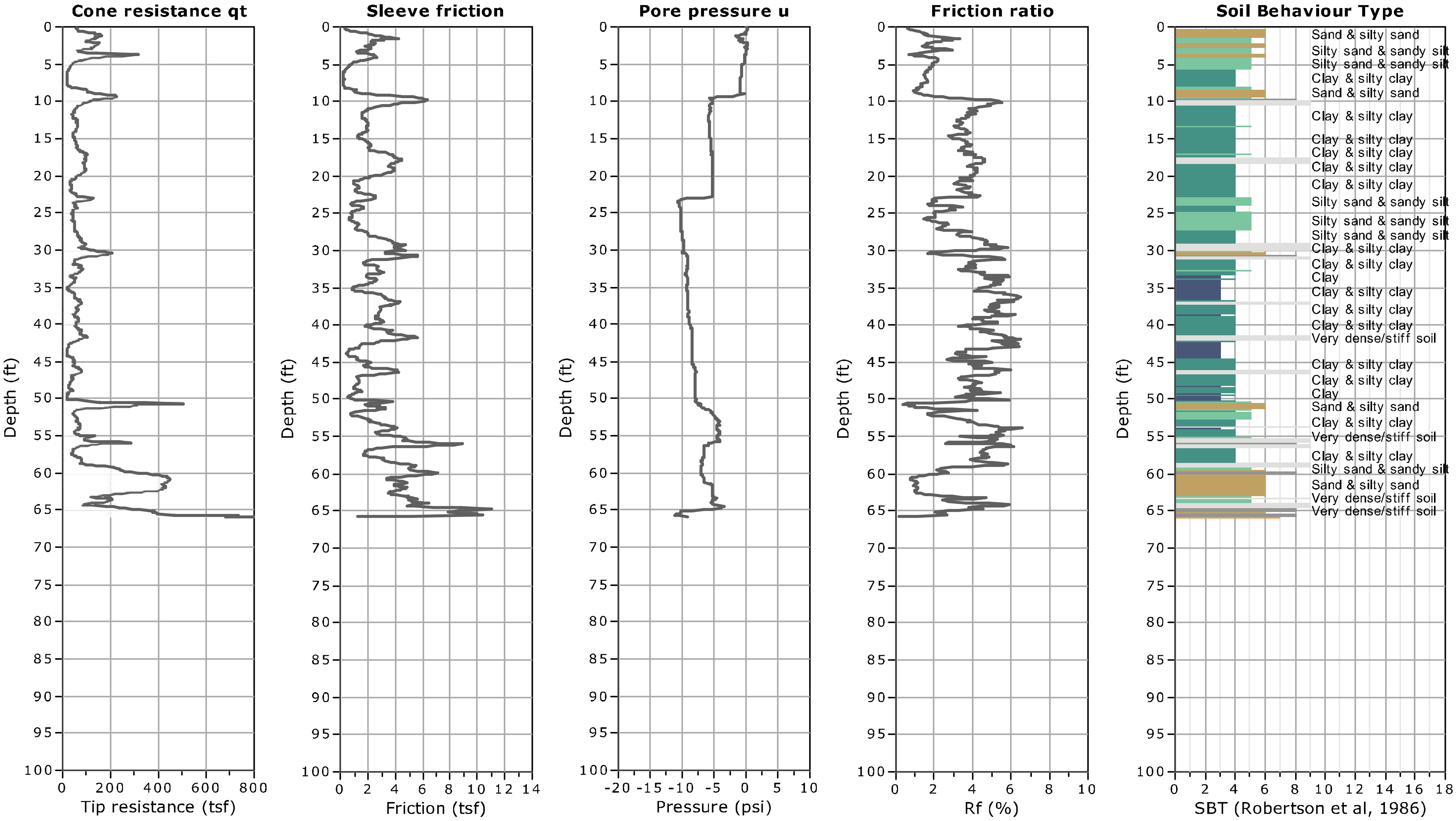
CPT: C-6

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 65.90 ft, Date: 7/30/2013
Cone Type: Vertek



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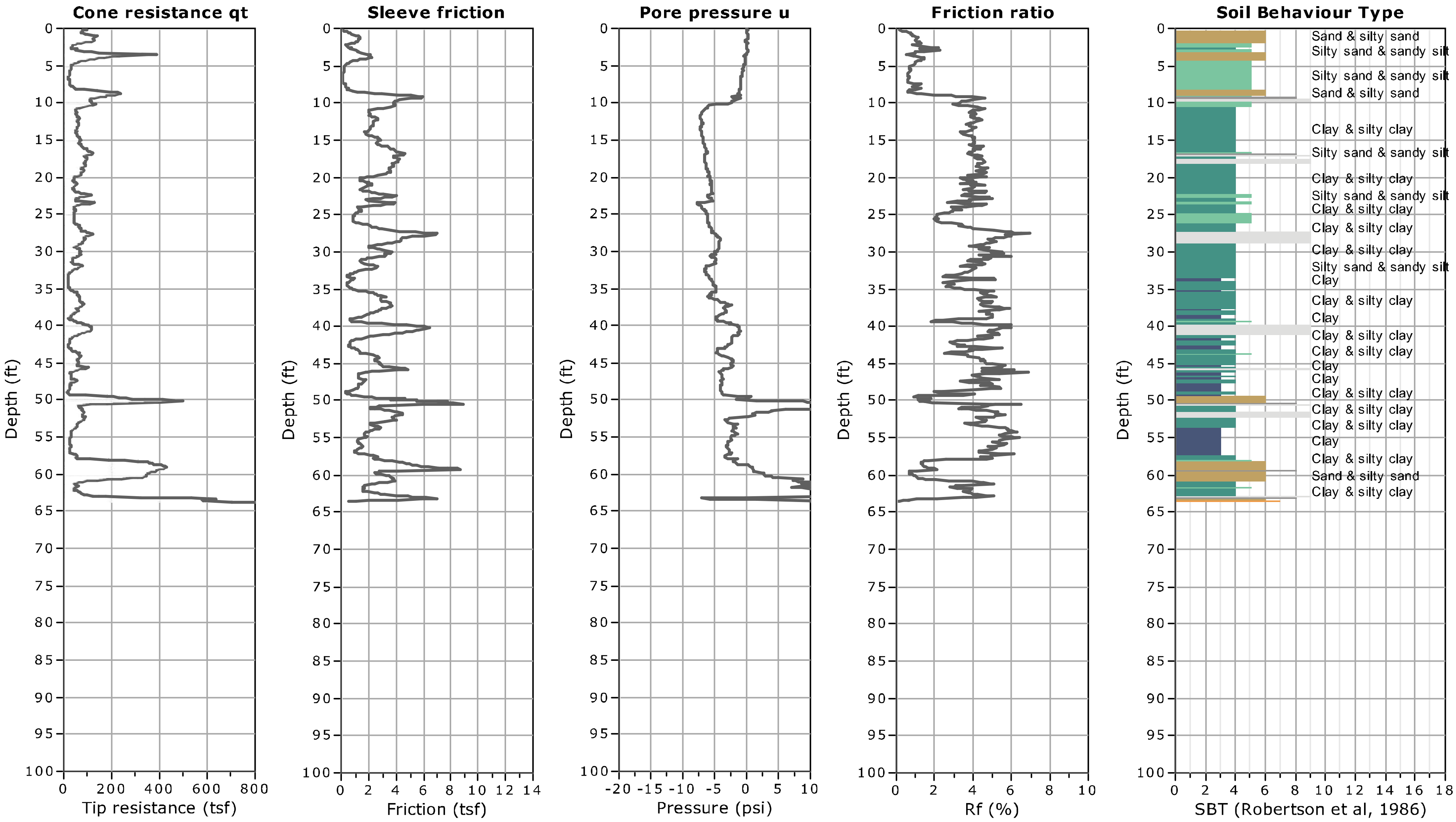
CPT: C-7
Total depth: 63.72 ft, Date: 7/31/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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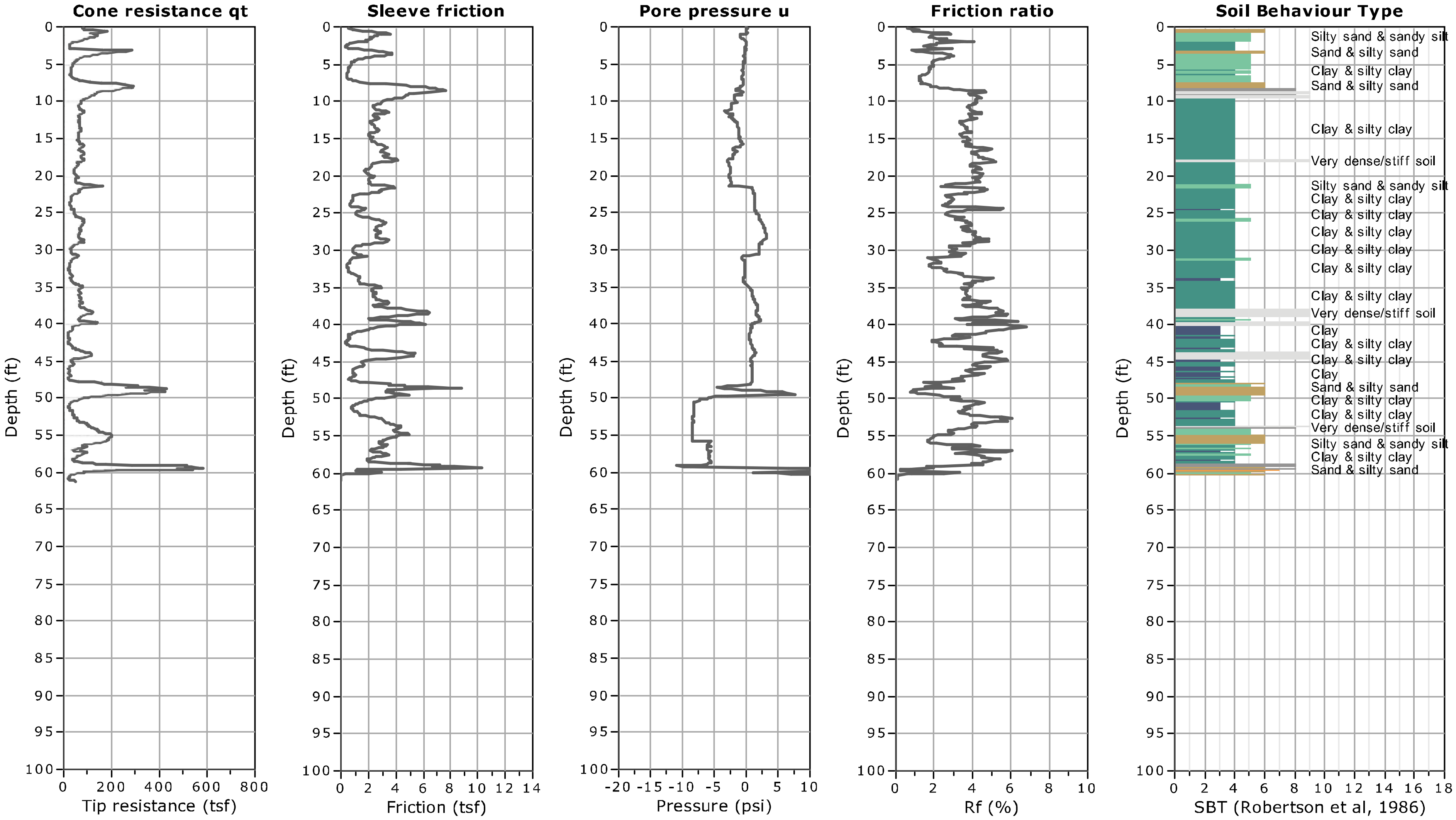
CPT: C-8

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 61.16 ft, Date: 7/31/2013
Cone Type: Vertek



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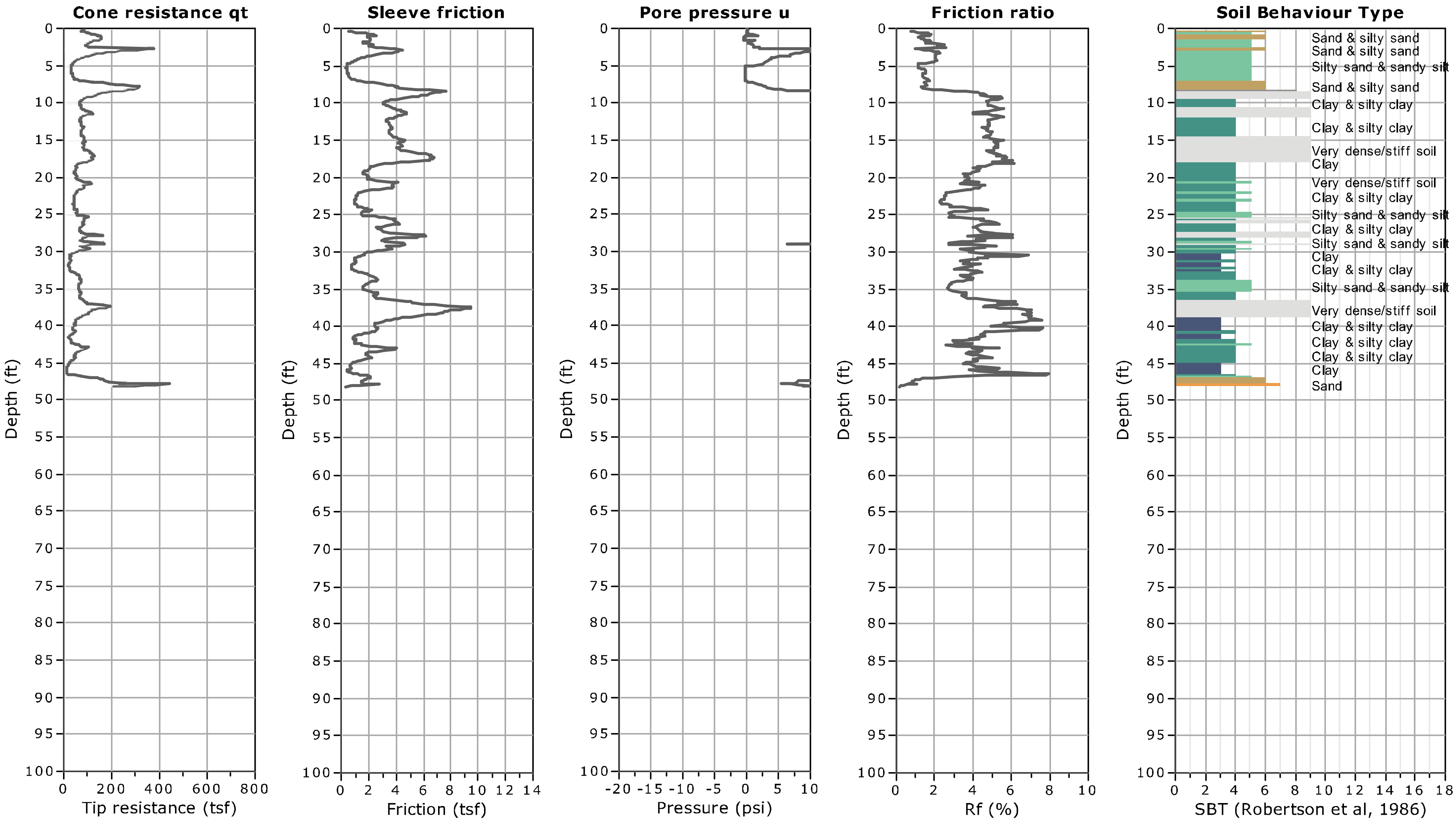
CPT: C-9

Total depth: 48.21 ft, Date: 7/31/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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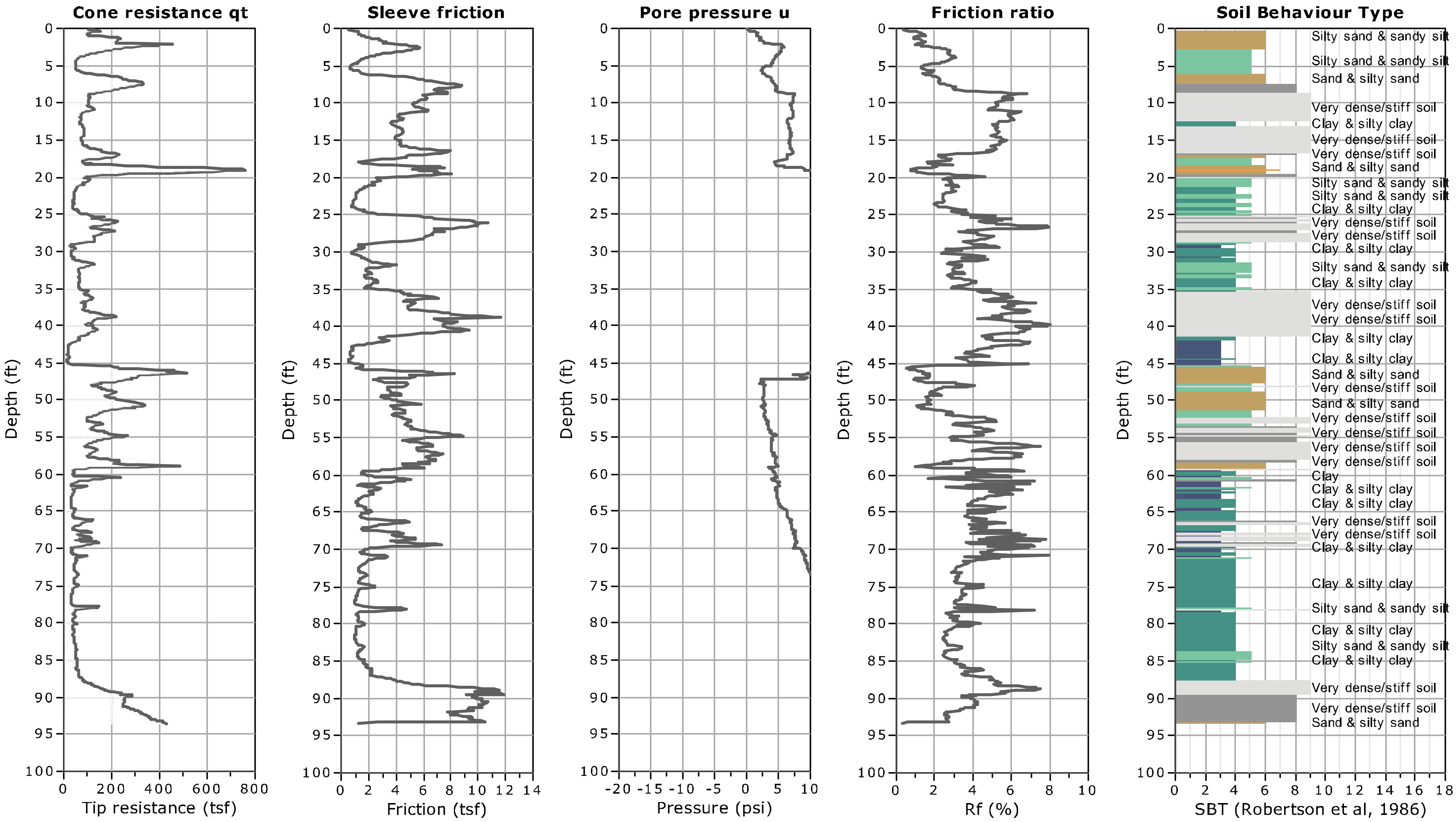
Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

CPT: C-10
Total depth: 93.47 ft, Date: 7/31/2013
Cone Type: Vertek



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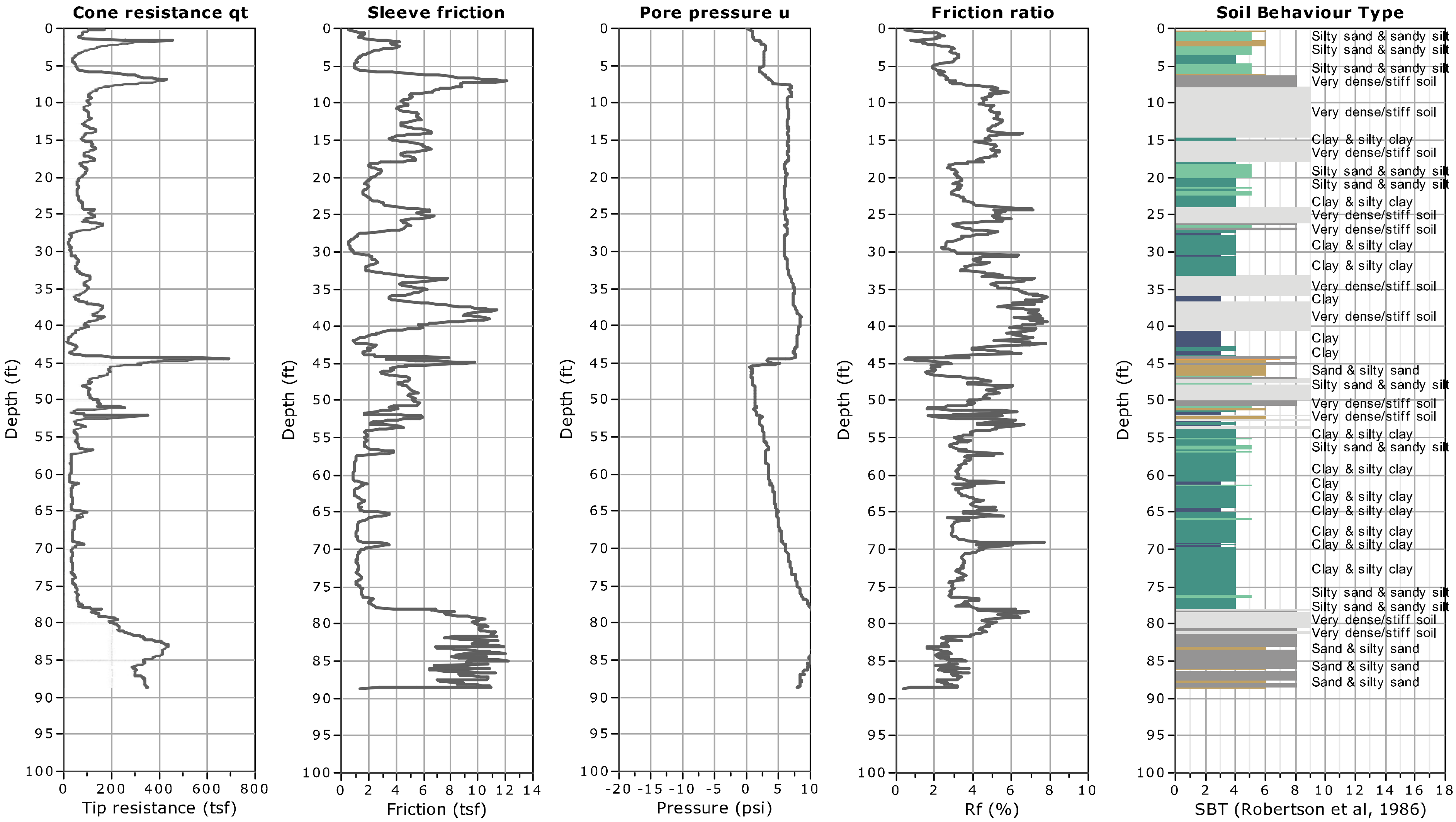
CPT: C-11

Total depth: 88.77 ft, Date: 7/31/2013

Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**

Location: **Kettner Blvd. & Vine St. San Diego, CA**



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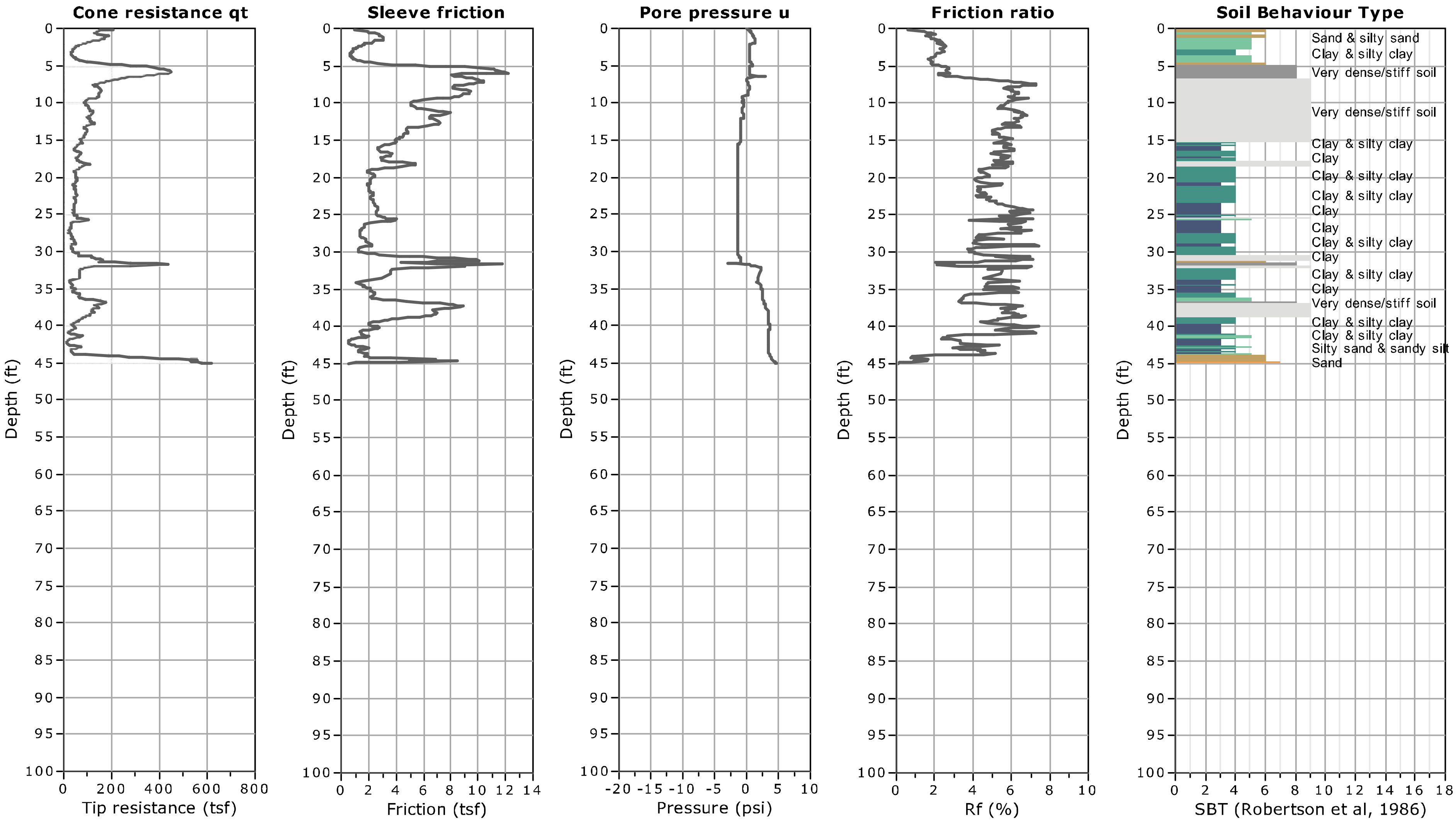
CPT: C-12

Total depth: 45.10 ft, Date: 7/31/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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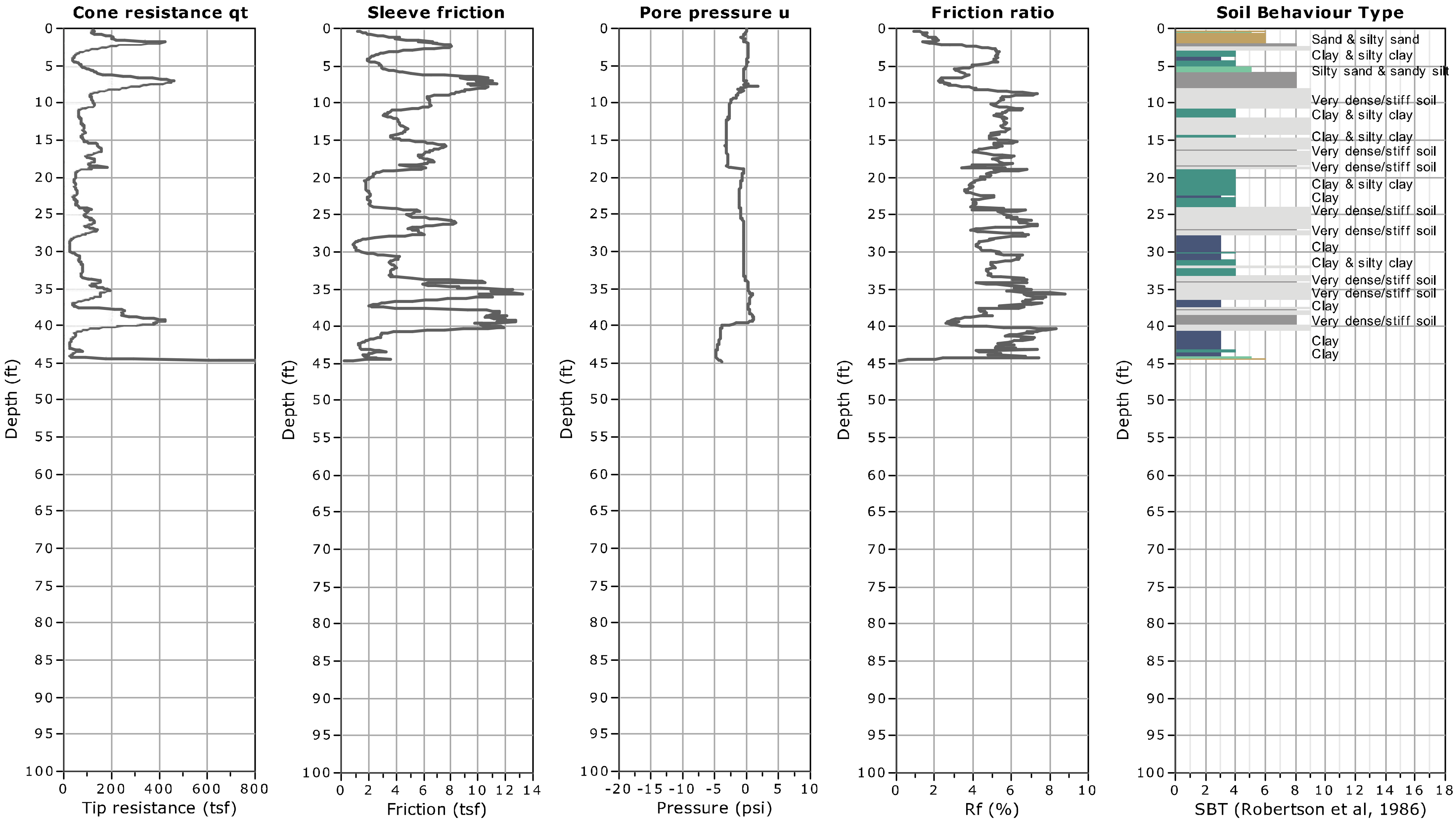
CPT: C-12B

Total depth: 44.82 ft, Date: 8/2/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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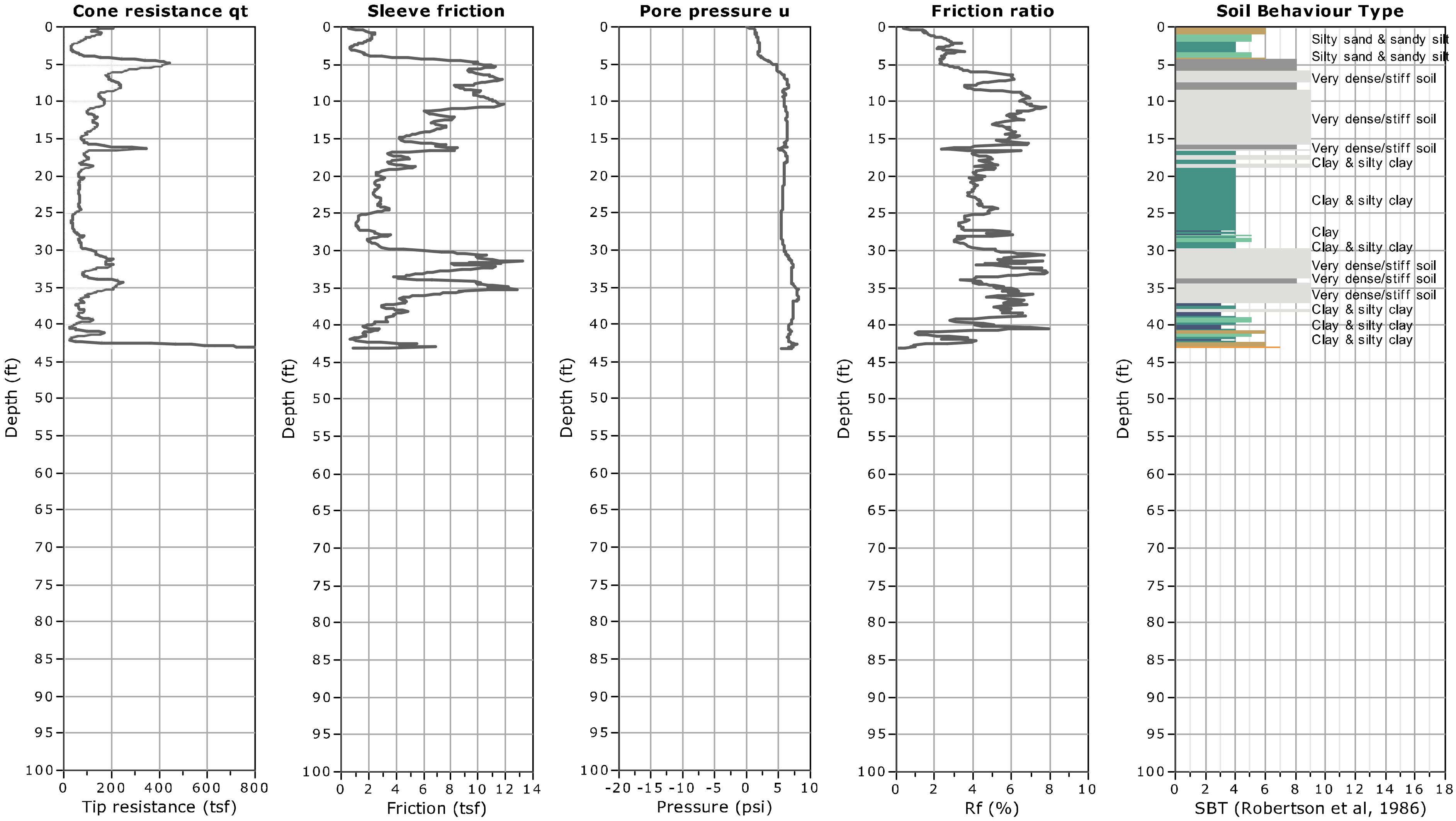
CPT: C-13

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 43.30 ft, Date: 7/31/2013
Cone Type: Vertek



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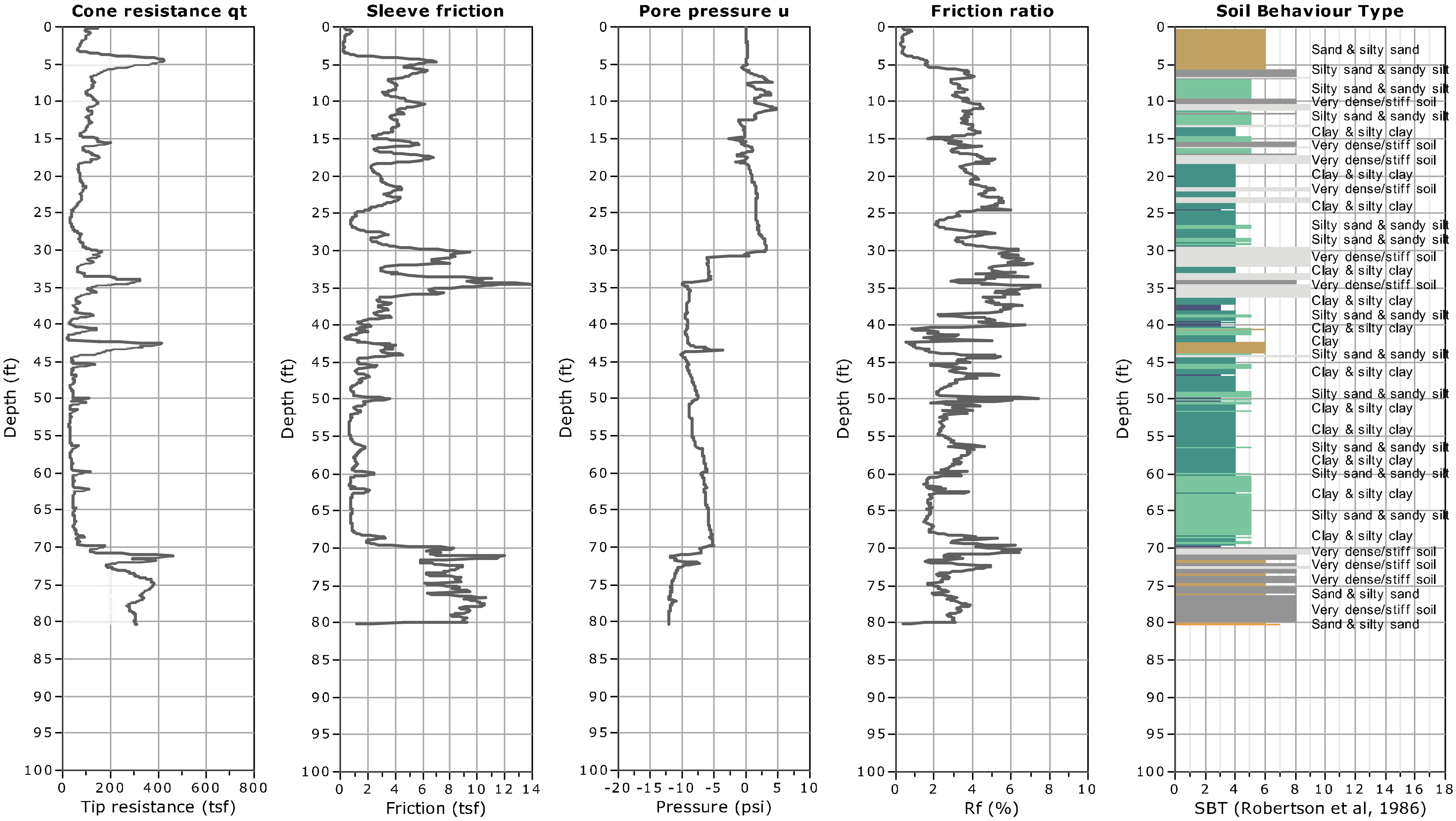
CPT: C-13A

Total depth: 80.29 ft, Date: 8/2/2013

Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**

Location: **Kettner Blvd. & Vine St. San Diego, CA**



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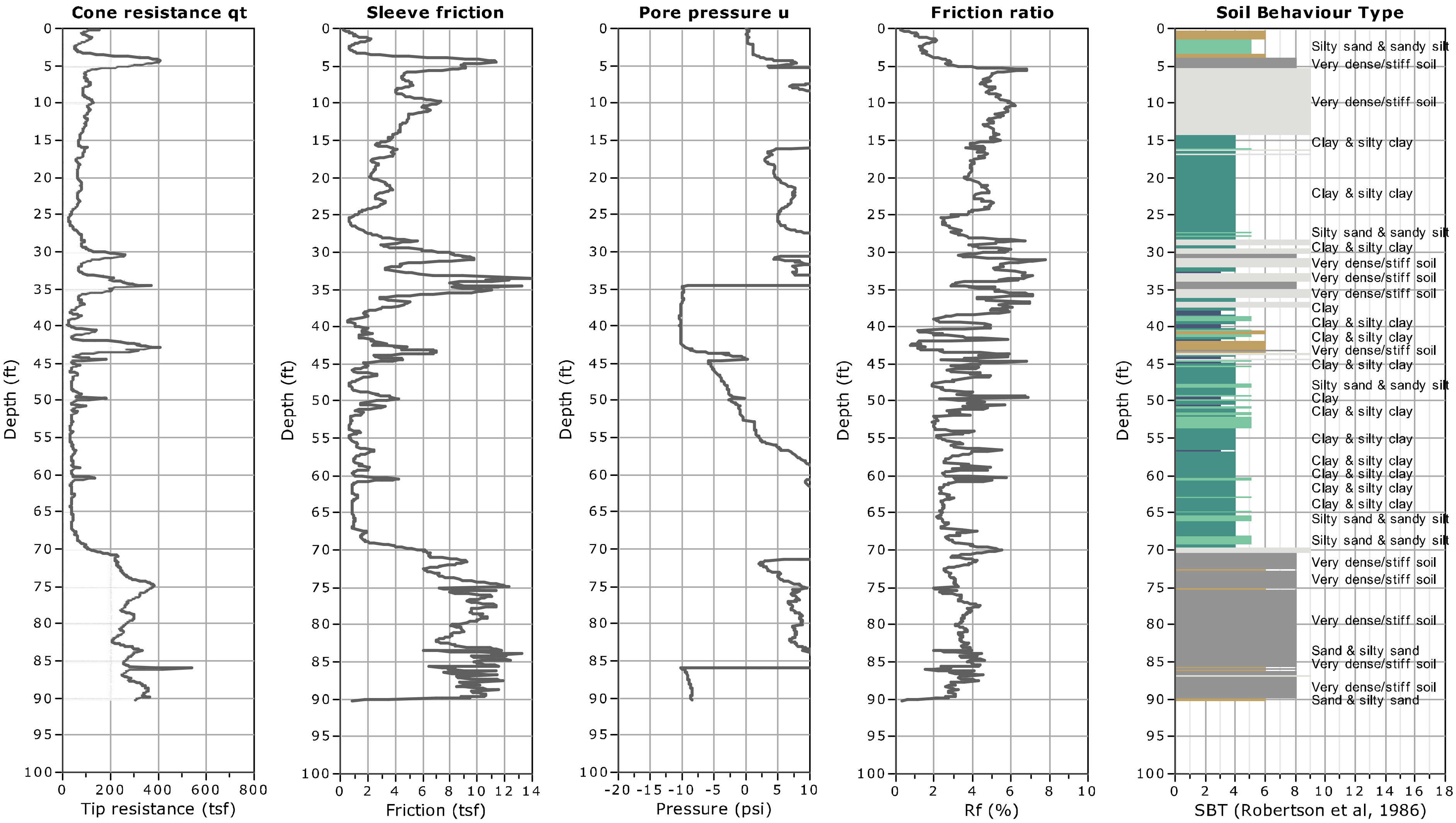
CPT: C-14

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 90.30 ft, Date: 8/1/2013
Cone Type: Vertek



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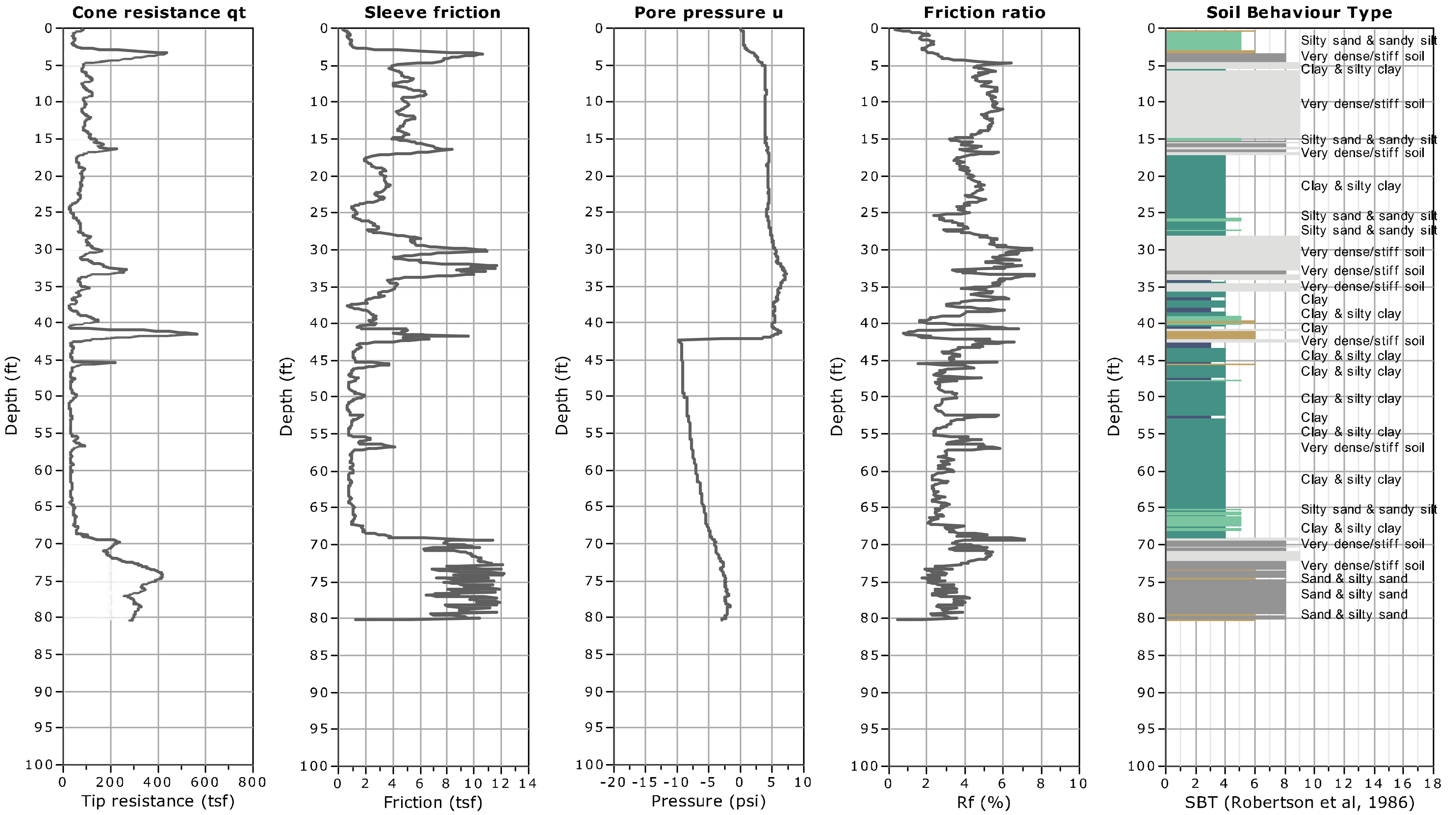
CPT: C-15

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 80.32 ft, Date: 8/1/2013
Cone Type: Vertek



CPeT-IT v.1.7.6.3 - CPTU data presentation & interpretation software - Report created on: 8/5/2013, 3:24:59 PM
Project file: C:\GEOSYN~1\CPETDA~1\PLOTDA~1\Plots.cpt

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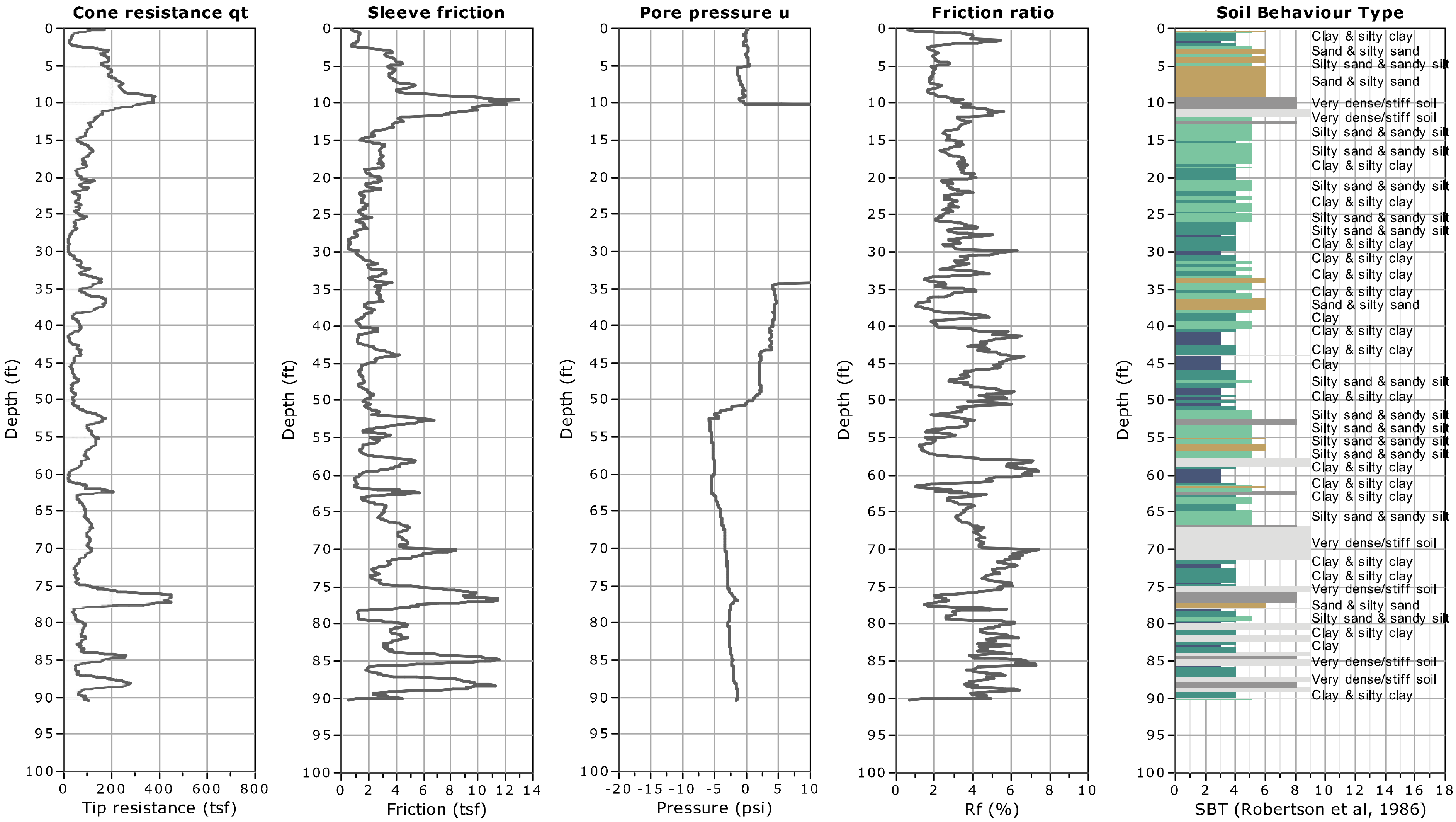
Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

CPT: C-16
Total depth: 90.37 ft, Date: 8/1/2013
Cone Type: Vertek



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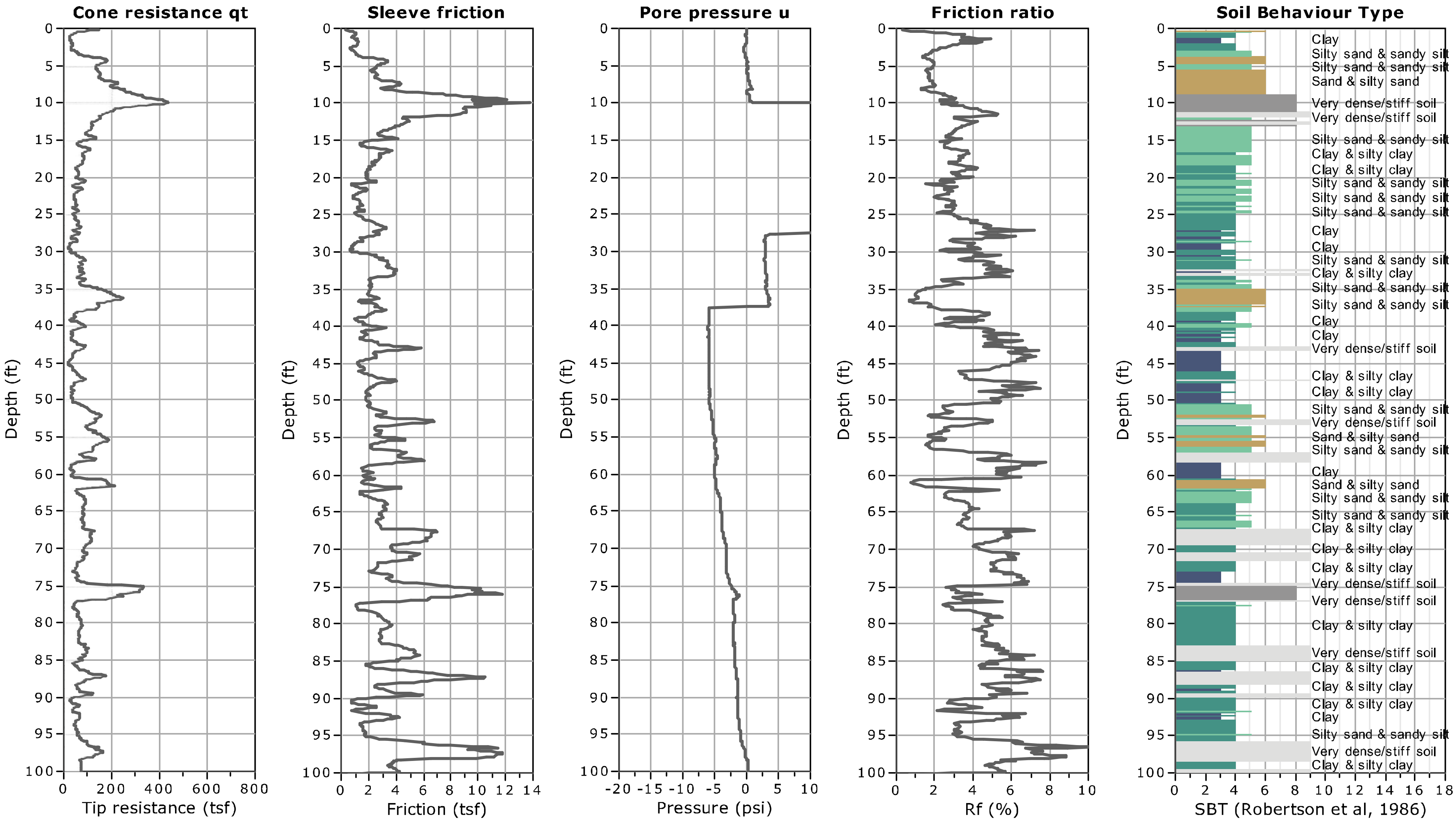
CPT: C-17

Total depth: 100.39 ft, Date: 8/1/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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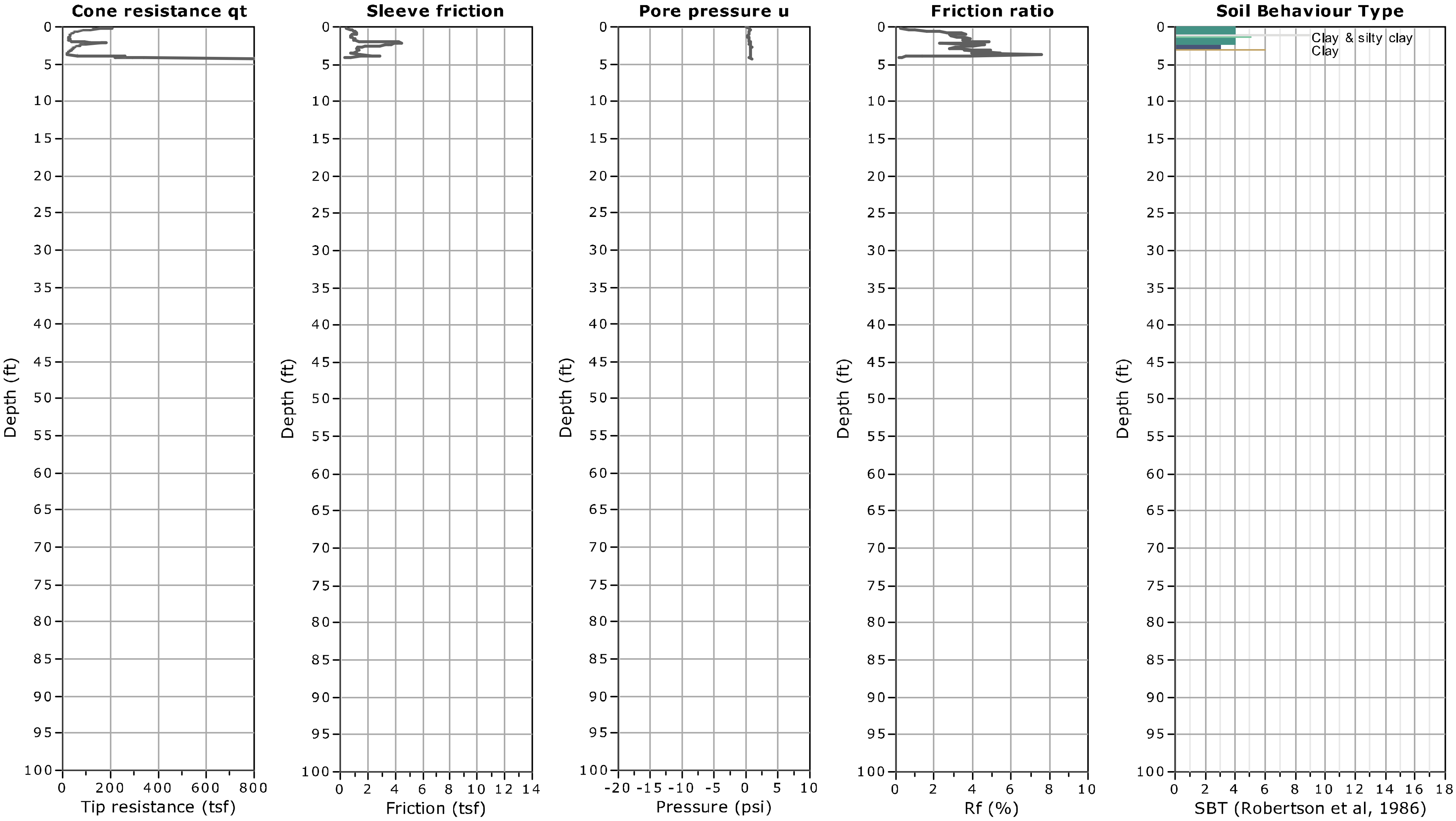
CPT: C-18
Total depth: 4.25 ft, Date: 8/1/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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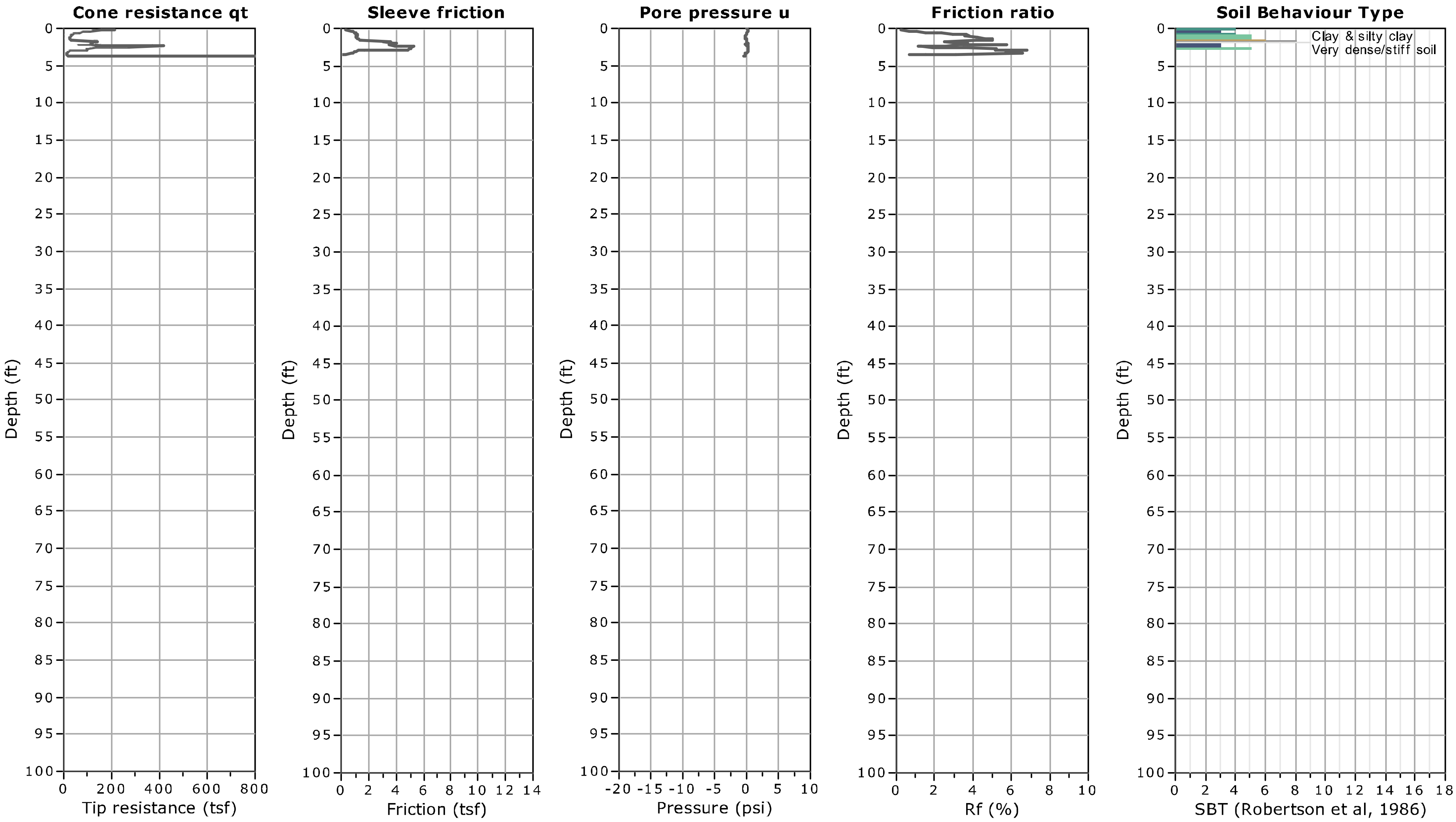
CPT: C-18A

Total depth: 3.74 ft, Date: 8/1/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



CPeT-IT v.1.7.6.3 - CPTU data presentation & interpretation software - Report created on: 8/5/2013, 3:28:27 PM
Project file: C:\GEOSYN~1\CPETDA~1\PLOTDA~1\Plots.cpt

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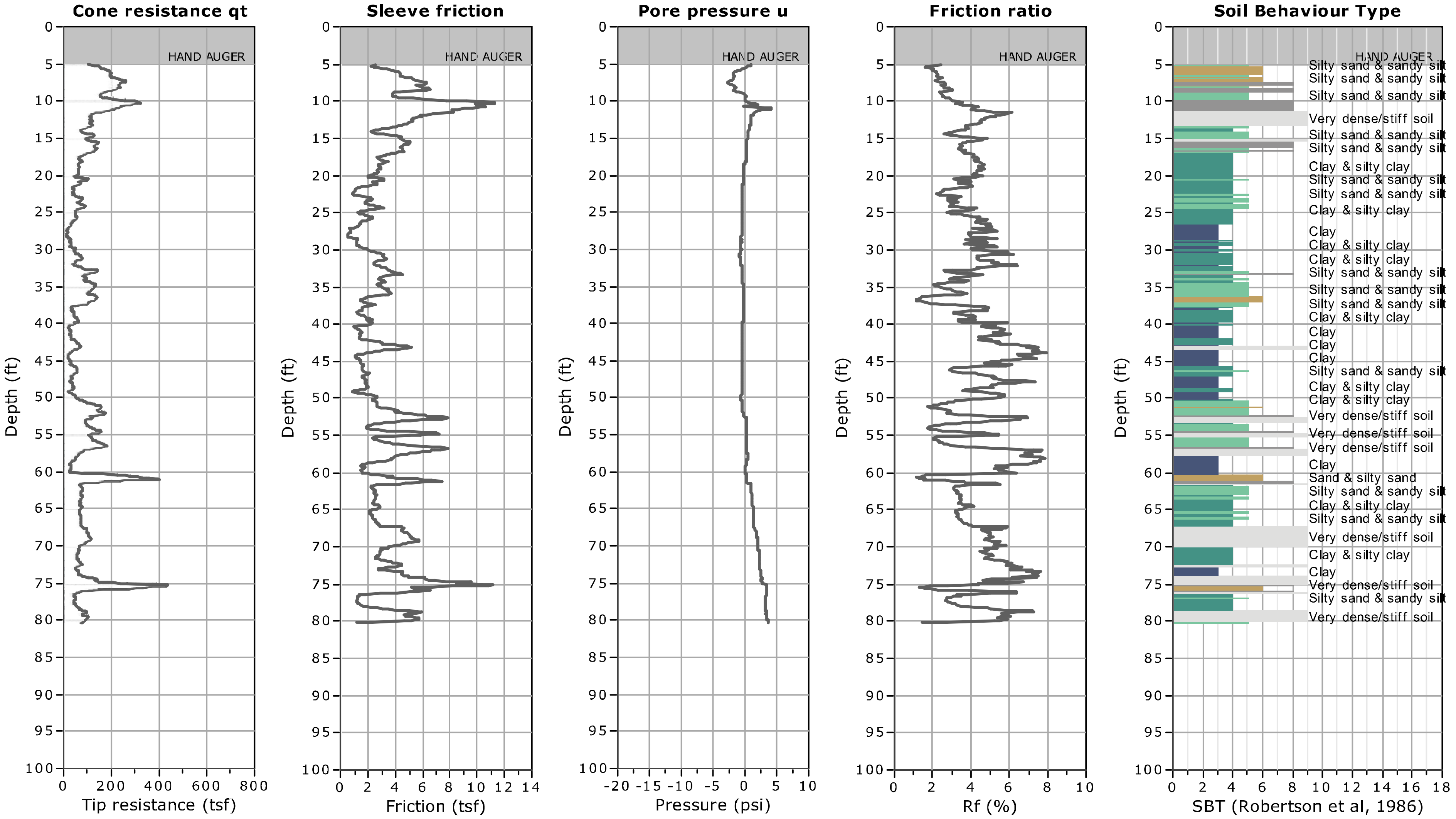
CPT: C-18C

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 80.35 ft, Date: 8/2/2013
Cone Type: Vertek



CPeT-IT v.1.7.6.3 - CPTU data presentation & interpretation software - Report created on: 8/5/2013, 4:53:28 PM
Project file: C:\GEOSYN~1\CPETDA~1\PLOTDA~1\Plots.cpt

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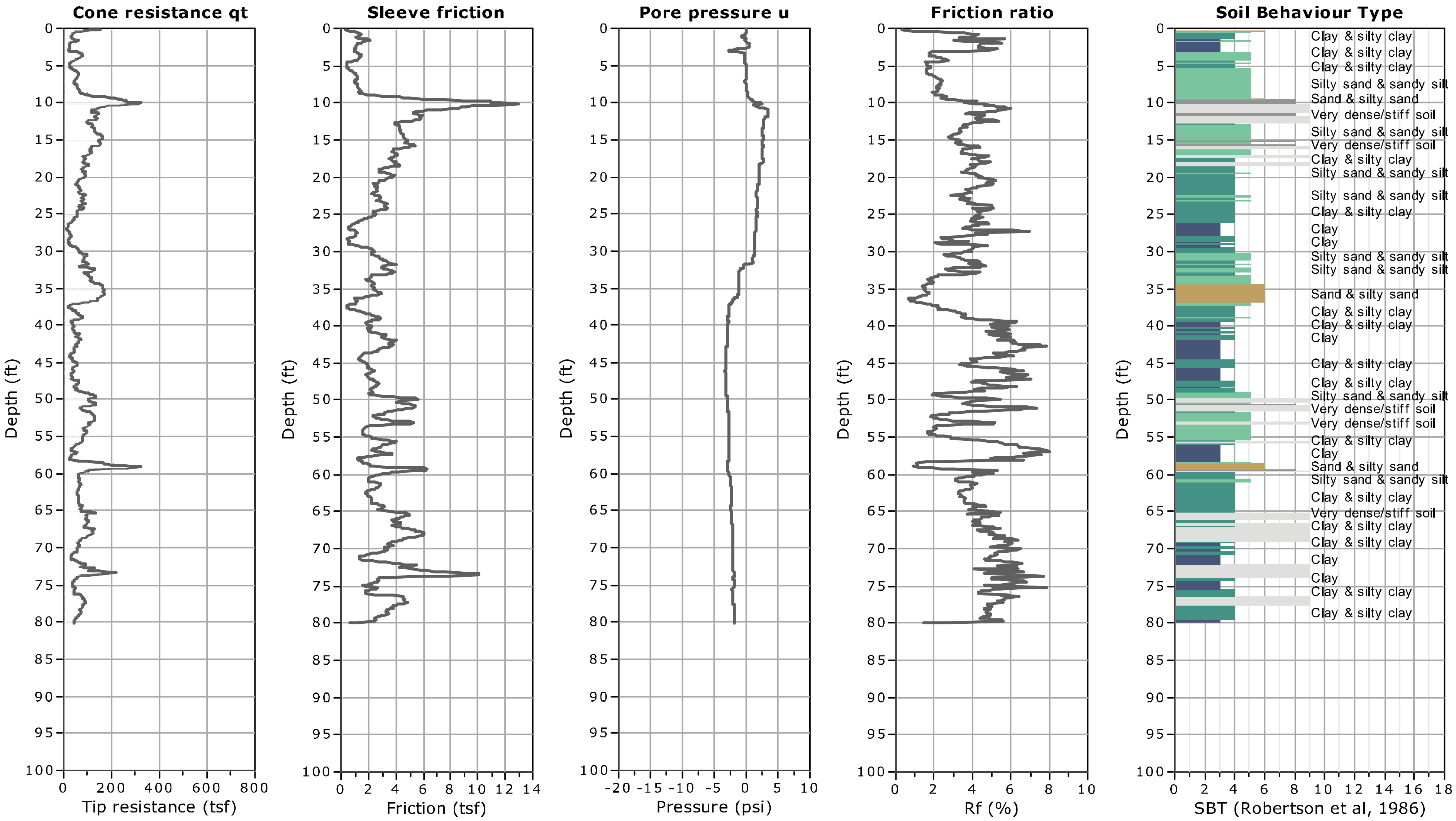
CPT: C-19

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 80.12 ft, Date: 8/1/2013
Cone Type: Vertek



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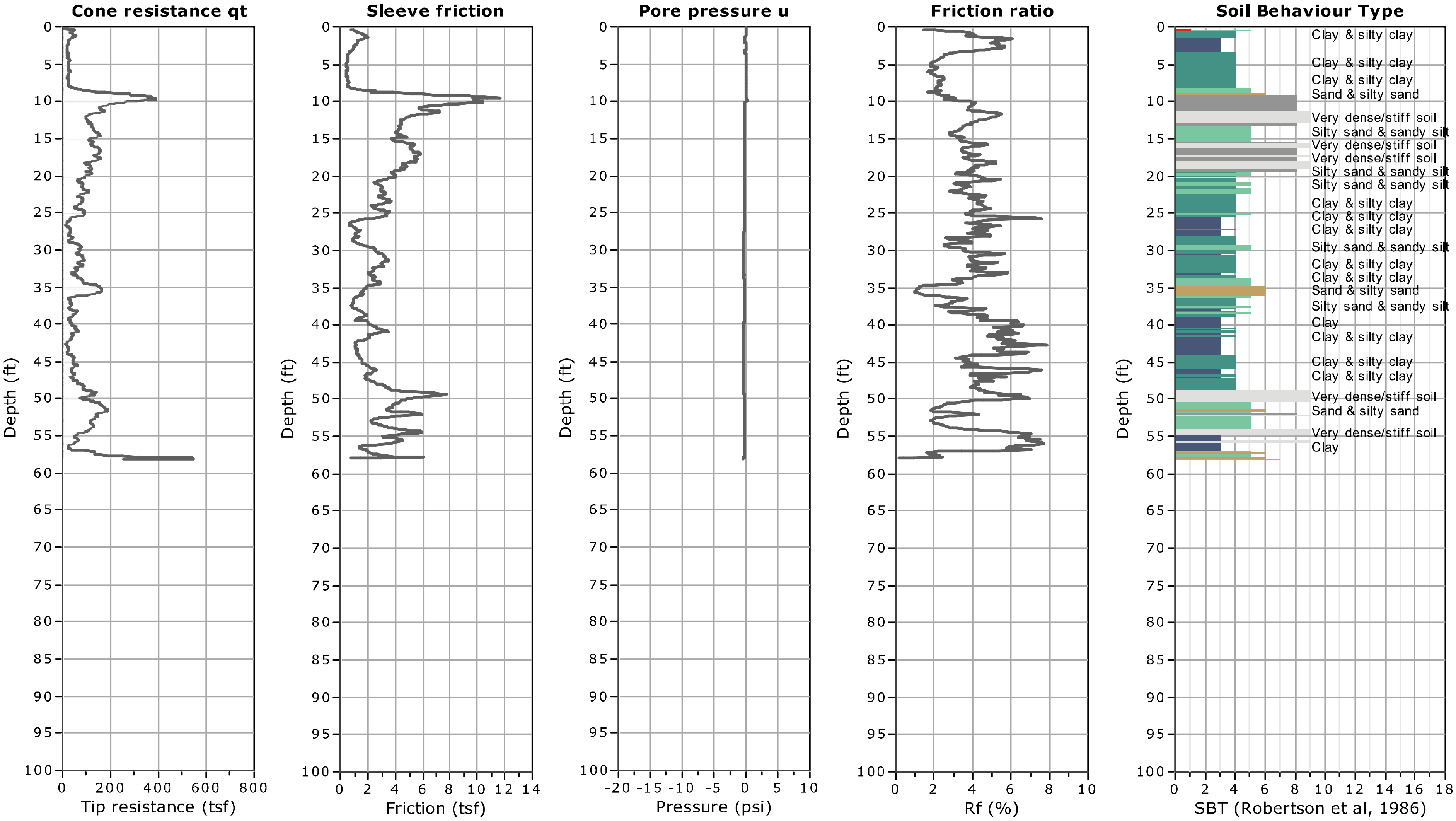
CPT: C-20
Total depth: 58.09 ft, Date: 8/1/2013
Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**
Location: **Kettner Blvd. & Vine St. San Diego, CA**



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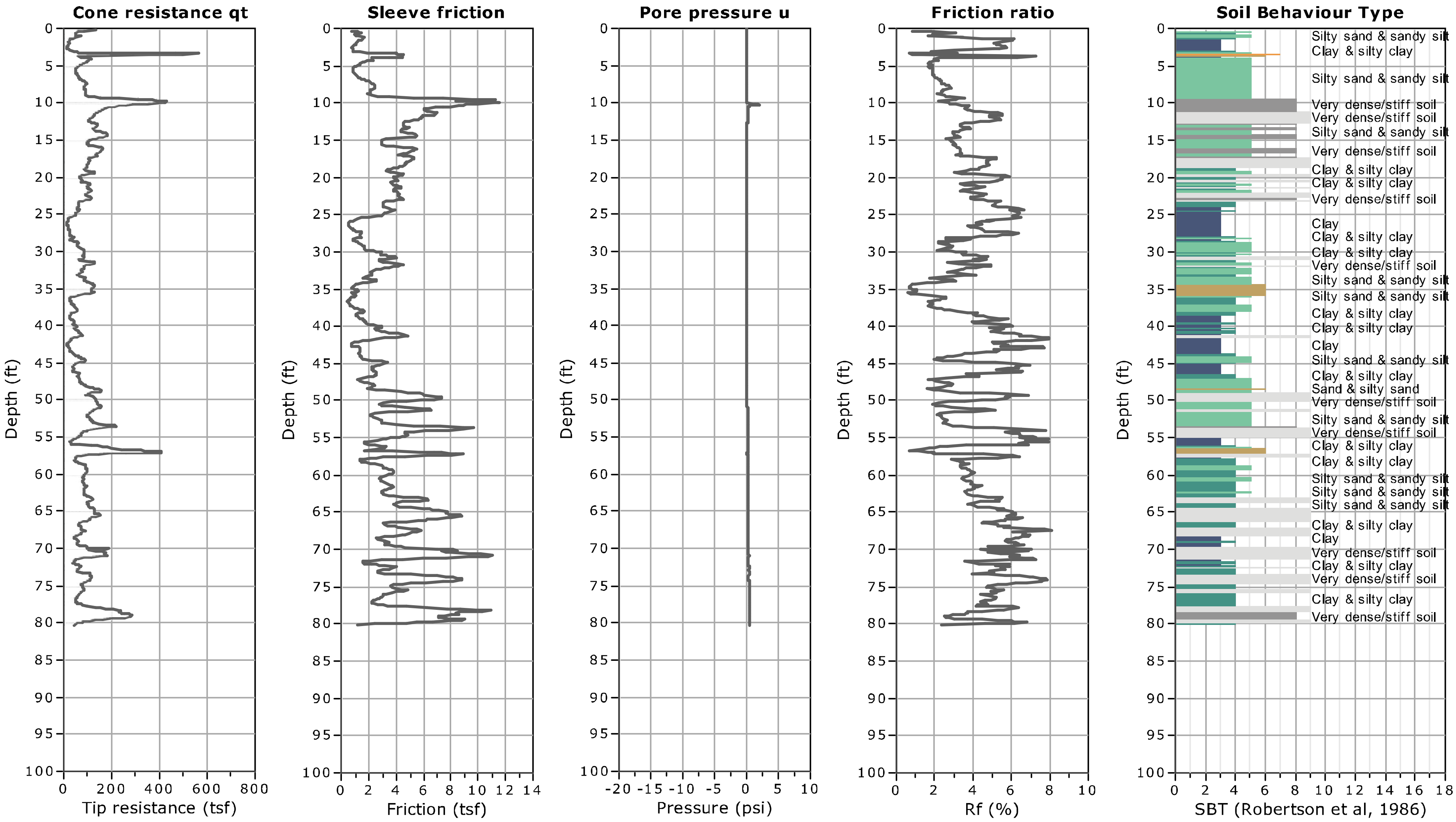
CPT: C-21

Project: **Geosyntec/Kettner Substation Relocation**

Location: **Kettner Blvd. & Vine St. San Diego, CA**

Total depth: 80.27 ft, Date: 8/1/2013

Cone Type: Vertek



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CPT: C-22

Total depth: 100.25 ft, Date: 8/2/2013

Cone Type: Vertek

Project: **Geosyntec/Kettner Substation Relocation**

Location: **Kettner Blvd. & Vine St. San Diego, CA**