REPORT

GEOTECHNICAL INVESTIGATION BANK 82 ADDITION, 2ND 500 KV TIE LINE, AND NEW 230 KV YARD IMPERIAL VALLEY SUBSTATION IMPERIAL COUNTY, CALIFORNIA

PREPARED FOR:

SAN DIEGO GAS & ELECTRIC

URS PROJECT NO. 27668011.00010

JUNE 10, 2009

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GEOTECHNICAL INVESTIGATION BANK 82 ADDITION, 2ND 500 KV TIE LINE, AND NEW 230 KV YARD IMPERIAL VALLEY SUBSTATION IMPERIAL COUNTY, CALIFORNIA

Prepared for

San Diego Gas & Electric Company 8316 Century Park Court, CP-52G San Diego, California, 92123

URS Project No. 27668011.00010

June 10, 2009



1615 Murray Canyon Road, Suite 1000 San Diego, CA 92108-4314 619.294.9400 Fax: 619.293.7920 June 10, 2009

Mr. Ron Brunton San Diego Gas & Electric Company 8316 Century Park Court, CP-52G San Diego, California, 92123

Subject: Geotechnical Investigation Bank 82 Addition, 2nd 500 kV Tie Line, and New 230 kV Yard Imperial Valley Substation Imperial County, California URS Project No. 27668011.00010

Dear Mr. Brunton:

URS Corporation Americas (URS) is pleased to submit the following report presenting the results of our geotechnical investigation for the proposed improvements at the existing San Diego Gas & Electric Company Imperial Valley Substation. This investigation was performed in general accordance with our proposal dated May 27, 2008 and our Standard Service Agreement Nos. 5660004780 and 6160015090.

This report presents the results of our investigation and geotechnical recommendations for design. The results of the study indicate that the site is suitable for the proposed development, provided that the geotechnical and geologic considerations discussed in this report are incorporated into the design.

We are pleased to be part of this important project, and if you have any questions, please contact us at (619) 294-9400.

Sincerely,

URS CORPORATION

Jernifer L. Nevius, R.C.E Project Engineer

JLN/MEH:kl

Enclosure



No. 1925

Michael E. Hatch, C.E.G. 1925 Principal Geologist

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ASTM	American Society for Testing and Materials		
Benton	Benton Engineering Inc.		
bgs	below ground surface		
Cal OSHA	California Occupational Safety and Health Administration		
Caltrans	State of California Department of Transportation		
CBC	California Building Code		
CBR	California Bearing Ratio		
EI	Expansion Index		
E _{pmt}	Pressuremeter Modulus		
EPRI	Electric Power Research Institute		
ft	feet		
Fugro	Fugro, Inc.		
g	units of gravity		
IBC	International Building Code		
IEEE	Institute of Electrical and Electronics Engineers, Inc.		
in	inches		
kips	kilopounds		
ksi	kips per square inch		
kV	Kilovolt		
MCE	Maximum Considered Earthquake		
MFAD	Moment Foundation Analysis and Design		
MSL	Mean Sea Level		
Ν	Blowcount		
ohm-cm	Ohm-centimeters		
pcf	pounds per cubic foot		
PGA	peak ground acceleration		
psf	pounds per square foot		
R-Value	Resistance Value (for pavement design)		
SDG&E	San Diego Gas & Electric Company		
SPT	Standard Penetration Test		
Substation	Imperial Valley Substation		
TI	Traffic Index		
URS	URS Corporation Americas		
USCS	Unified Soil Classification System		
USGS	United States Geological Survey		

SECTION 1 INTRODUCTION

This report presents the results of URS Corporation Americas' (URS) geotechnical investigation for the proposed improvements at the existing San Diego Gas & Electric Company (SDG&E) Imperial Valley Substation (Substation) located in Imperial County near El Centro, California. The Substation is located south of Interstate 8 and north of State Route 98, just west of the Westside Main Canal (Figure 1). URS prepared this report for SDG&E and their consultants for use in project planning and design.

1.1 **PROJECT DESCRIPTION**

The currently planned additions are on the west side of the Substation and include the Bank 82 Addition, a 2nd 500 kilovolt (kV) Tie Line and a new 230 kV Yard. An area is also available at the southwest corner of the site for the terminus of the proposed Sunrise Powerlink Transmission Line. Figure 2 presents a site plan with the current Substation layout and the approximate locations of the proposed additions. Elements of the additions include switch stands, circuit breakers, transformers, firewalls, "A" frames, "H" frames, and bus supports. Foundation types for this equipment typically include drilled piers, shallow strip foundations and mat foundations. Final foundation layouts and structural loads are not available at this time. Preliminary foundation information for the Bank 82 Addition is summarized below based on existing and similar designs at the Imperial Valley Substation. The information was provided by Mr. Ron Brunton via email on June 16, 2008 and in subsequent discussions. Safety factors of 1.5 to 2.0 are incorporated in the loads shown.

Structure	Foundation Thickness (in)	Foundation Plan Area	Vertical Load (kips)	Maximum Soil Pressure Due to Seismic Overturning (psf)	
Disconnect Switch Stand	18	13'-0" x 22'-6"	14	500 to 1,000	
Circuit Breaker	18	9'-6" x 12'-0"	17	1,500 to 2,000	
Transformer Mat	36	16'-0"x 34'-0"	862	3,760	

Shallow Foundations

Deep Foundations

Structure	Planned Pier Diameter (in)	Planned Pier Length (ft)	Vertical Load (kips)	Lateral Load (kips)	Moment (ft-kips)
Firewall Support Piers	66	26.5	60	125	1,800
60' Dead End A-Frame	60 to 66	18 to 24	150 downward 110 uplift	60	400
Single Column Bus Support	30	6 to 8	1	6	40
SL&P Rack Support Steel	36	8 to 10	6	12	140
38' Dead End H-Frame	48 to 60	14 to 16	20	12	40 to 60

In addition to specific recommendations for the planned improvements, we understand that SDG&E desires general foundation design information for the entire Substation to accommodate future additions not yet planned.

1.2 PURPOSE AND SCOPE OF INVESTIGATION

The purpose of our services was to explore subsurface conditions at the substation and provide geotechnical recommendations for design and construction. The scope of our investigation included a review of the area geology and available previous investigations at the Substation, field explorations, pressuremeter testing, geotechnical laboratory testing, engineering analyses and evaluations, and development of design recommendations. Our scope of services was expanded from addressing only the currently planned additions to include an evaluation of the full Substation area based on available information.

The results of the investigation were used to develop geotechnical recommendations regarding:

- General subsurface soil and groundwater conditions;
- Geologic setting and assessment of geologic and seismic hazards including surface faulting, strong ground motion, liquefaction, and seismic settlement;
- Presence and effect of near-surface expansive and collapsible soils;
- Earthwork recommendations;
- Appropriate foundation types;
- Allowable vertical and lateral capacities of shallow foundations;
- Modulus of subgrade reaction for mat foundations;
- Estimated total and differential shallow foundation settlements;
- Allowable axial capacities of deep foundations;
- Parameters for lateral deep foundation design;
- Surface drainage;
- Flexible pavement design;
- Corrosion potential of soil;
- Substation equipment seismic qualification level; and
- Construction considerations.

Detailed results of previous information, current field exploration, pressuremeter testing, and geotechnical laboratory testing are provided in the appendices of this report.

SECTION 2 GEOTECHNICAL INVESTIGATION

The geotechnical investigation included a review of previous studies and available information, a site reconnaissance, geotechnical borings, pressuremeter testing, and laboratory testing.

2.1 PREVIOUS STUDIES

URS previously performed geotechnical investigations for improvements at the Substation, 1) for the access road from State Route 98, and 2) for associated transmission line structures. URS has also been provided with copies of previous investigations by others at and near the site. A detailed list of references is provided in Section 6. The investigations within the Substation are summarized below. The approximate locations of the explorations from these previous studies are presented on Figure 2. Geotechnical information from these investigations were evaluated and the data was incorporated into our current investigation where appropriate. Copies of the boring logs and selected geotechnical laboratory test results from these investigations are presented in Appendix A.

2.1.1 Fugro, 1980

Fugro, Inc. (Fugro) performed the feasibility and preliminary geotechnical investigation for construction of the Substation. The investigation included four hollow stem auger borings advanced to depths up to about 52 feet below grade. Representative soil samples were tested for grain size distribution. The ground surface elevations from the Fugro borings are inconsistent with the historical topographic elevations at the site. For the current study, the ground surface elevations at the Fugro borings have been approximated based on historical topographic information from the Substation construction.

2.1.2 Benton, 1980

Benton Engineering Inc. (Benton) performed the design level geotechnical investigation for the Substation. The investigation included twenty-one borings advanced to depths between about 12 and 50 feet below grade. Twenty of the borings were rotary wash and one of the borings (Boring 21) was a large diameter (30-inch) bucket-type boring. The geotechnical laboratory test program included shear strength, consolidation, expansion, California Bearing Ratio (CBR), unconfined compressive strength, and chemical analysis for corrosion potential.

2.1.3 URS, 2003

URS performed a geotechnical investigation for new transformers and firewalls and a retrofit of existing firewall structures. Three hollow stem auger borings were advanced in the central portion of the Substation to depths of up to 37 feet below grade. Laboratory testing included unconfined compression, particle size distribution, and laboratory compaction tests.

2.2 SUBSURFACE EXPLORATIONS

The current field investigation included a visual reconnaissance of the existing surface conditions and drilling and logging eight test borings in the western half of the Substation. The borings were designated

Boring B-1 through Boring B-8 and their approximate locations are shown on Figure 2. Boring B-1 and Boring B-2 were advanced in the area of the new 230 kV yard. Boring B-3 and Boring B-4 were advanced in the area of the Bank 82 Addition. Boring B-5 and Boring B-6 were advanced in the area of the 2nd 500 kV Tie Line. Boring B-7 and Boring B-8 were advanced where future transmission line improvements are being considered. The geotechnical borings were advanced using a truck mounted drill rig with hollow-stem augers. The borings were advanced to depths of 51.5 feet below the ground surface (bgs). Additional borings were advanced immediately adjacent to Borings B-3, B-6, and B-7 using mud rotary drilling for pressuremeter testing. The subsurface investigation was performed between June 2 and 9, 2008.

An engineering geologist from our firm logged the borings, and the soil encountered was classified in general accordance with the Unified Soil Classification System (USCS). Disturbed and relatively undisturbed soil samples were collected from the borings. A Key to Logs is presented in Appendix B as Figure B-1. Logs of the borings are presented in Appendix B in Figures B-2 through Figure B-9. Further details of the field exploration are presented in Appendix B.

2.3 PRESSUREMETER TESTING

A total of 18 pressuremeter tests were performed in borings immediately adjacent to Boring B-3, Boring B-6, and Boring B-7 to provide the basis of design values of the pressuremeter modulus (E_{pmt}) used for lateral deep foundation design. The specific locations and depths for the pressuremeter tests were selected after a review of the subsurface conditions encountered in the hollow-stem auger borings to provide data in a variety of strata within the expected foundation depths.

The pressuremeter testing is the start of a regional effort to collect pressuremeter modulus data for the various formations and deposits present in the SDG&E service area. The methodology and results of the pressuremeter testing are summarized in Appendix C.

2.4 LABORATORY TESTING

The materials encountered in the field were visually classified and evaluated with respect to consistency, density, and moisture content. The samples were then returned to our laboratory for further examination and testing. Grain size and plasticity analyses on representative samples of the soils substantiated the visual classifications. The strength and compressibility of the soil were evaluated by considering the density and moisture content of relatively undisturbed samples, results of direct shear tests and unconsolidated undrained triaxial tests, and by other empirical correlations between soil characteristics and physical properties. A laboratory compaction test was performed on a sample of near surface soil to evaluate the moisture density relationship. Resistivity, sulfate content, pH, and chloride content tests were performed to evaluate the potential corrosivity of the soils. Testing was performed in general accordance with applicable American Society for Testing and Materials (ASTM) standards. Results of laboratory testing are shown at the corresponding sample locations on the boring logs in Appendix B; detailed results are presented in Appendix D.

SECTION 3 SITE CONDITIONS

Knowledge of the site conditions was developed from a review of available information, site reconnaissance and the current investigation.

3.1 GEOLOGIC SETTING

The site is in the Imperial Valley, which is part of the Salton Trough physiographic province. The Salton Trough is bounded by the Western Mojave Desert, the Peninsular Ranges Batholith, the Basin and Ranges and the Gulf of California. The Salton Trough is a deep, structural basin characterized by high seismicity, high heat flux, extensional tectonics, crustal thinning, and rapid sedimentation (Damiata *et al.*, 1986). Geophysical studies (Tarbet 1951; Biehler *et al.*, 1964) suggest that upwards of 5.5 kilometers (3.3 miles) of sediment have accumulated in the Salton Trough since the Miocene epoch (Eberley and Standley 1978).

The Salton Trough represents the transition zone between the crustal spreading centers in the Gulf and the right-lateral transform boundary between the North America and Pacific plates (Crowell and Sylvester 1979; Crowell 1981). The San Andreas fault zone is the principal element in this transform plate boundary; however, the total plate motion is distributed across a broad zone of deformation. Major elements in the right-slip system of faults related to the plate boundary and near the Salton Trough include the San Andreas, San Jacinto, Elsinore-Laguna Salada, Brawley, Imperial, and Cerro Prieto faults. Figure 3 presents a regional geologic map.

3.2 TECTONIC SETTING

The tectonic setting of the Imperial Valley is influenced by plate boundary interaction between the Pacific and North American lithospheric plates. This crustal interaction occurs along a broad belt of northwest-trending, predominately right-slip faults that span the width of the Peninsular Ranges and extend into the offshore Continental Borderland province. The major southern California fault systems include the San Andreas, San Jacinto, and Imperial fault zones to the east; the San Clemente, Coronado Bank, San Jacinto, Elsinore and Rose Canyon fault zones to the west; and the Agua Blanca and San Miguel fault zones to the southwest.

3.3 LOCAL FAULTS

The Imperial Valley is historically an area of high seismic activity and it is characterized by numerous active faults as shown on Figure 4, a regional fault and epicenter map. High-activity faults in the Imperial Valley include the San Andreas, Imperial, and San Jacinto faults.

Active faults within 25 kilometers of the site include: the Laguna Salada-Elsinore, San Jacinto, Imperial, and Yuha Wells faults. The Laguna Salada-Elsinore fault zone and the San Jacinto fault zone lie west and north of the site at distances of 14 and 20 kilometers, respectively.

The Imperial fault is 23 kilometers east of the site and is characterized by a high rate of slip estimated at 20 millimeters per year. The Imperial fault has ruptured the ground surface twice historically, once in 1940 and again in 1979.

The Yuha Wells fault is approximately 10 kilometers northwest of the site. This secondary fault is considered active although it has not been extensively studied. It is a northeasterly striking, left-lateral fault and is thought capable of generating earthquakes of approximately magnitude 6. The importance of left-lateral faults in the Imperial Valley was brought to light in 1987 when the left-lateral Elmore Ranch fault generated a magnitude 6.2 event that then triggered a magnitude 6.6 event on the adjacent branch of the San Jacinto fault. The Yuha Wells fault appears to be a similar cross fault structure.

3.4 SURFACE CONDITIONS

The Imperial Valley Substation site has been graded and developed and is relatively flat. Elevations across the site range from approximately +3.0 to +10.5 feet Mean Sea Level (MSL) based on grading plans from the original Substation construction. SDG&E also provided surveyed ground surface elevation information at each boring location; this information is presented on the boring logs.

The Substation is in an area of undeveloped low-lying desert with sparse vegetation. A gravel access road leads to the Substation from the south, approaching the west side of the Substation. The Westside Main Canal runs generally north-south and is located east of the Substation. Drainage within the general area typically flows in a northeasterly to easterly direction. Prior to construction, the site drained to the northeast. Currently, the site drains to the east and northeast.

3.5 SUBSURFACE CONDITIONS

This section describes the subsurface conditions at the site as encountered in our recent geotechnical borings and previous geotechnical borings by URS and by others. These explorations indicate that a minor veneer of fill typically overlies interbedded alluvial and lacustrine deposits at the site. These units are described in the following paragraphs, and are described in more detail on the boring logs in Appendix A and Appendix B. Figures 5 and 6 present generalized geologic cross sections through the Substation. The locations of the cross sections are shown on Figure 2.

3.5.1 Fill

In general, it appears the previous site development involved little grading. A plan from the original construction of the Substation shows cuts on the order of 2 feet in the southwestern corner and fills on the order of 2 to 5 feet in the northwest corner of the Substation. Typical SDG&E Substation design also includes a 1-foot thick wearing surface.

The URS 2003 investigation reported fill approximately 1.5 to 2 feet deep in three explorations in the center of the Substation. The fill was described as sandy gravel and is likely a wearing surface constructed for the Substation. Fill thicknesses were not identified during the recent investigation.

3.5.2 Alluvial and Lacustrine Deposits

The borings at the Substation encountered alluvial and lacustrine deposits. Alluvium deposition resulted from valley fill or distal fan processes. Lacustrine deposits are associated with the ancient Lake Cahuilla. These deposits are layered and interfingered, consisting of alternating layers of clean sand, silty and clayey sand, silt, lean clay, and fat clay. The blowcounts in these materials indicate medium dense to

dense sands and stiff to hard silts and clays. For simplicity, we have categorized the subsurface materials into 1) granular alluvial deposits and 2) fine-grained lacustrine deposits.

The cross sections presented on Figures 5 and 6 suggest that alluvial deposits predominate on the south side of the Substation and lacustrine deposits predominate on the north side. However, significant interlayering is present and subsurface conditions vary significantly, even between adjacent borings. Hence, different areas of the site require separate characterization and engineering evaluation.

3.6 **GROUNDWATER**

Groundwater was encountered at a depth of approximately 35 to 36 feet below existing grade in Borings B-7 and B-8 at the time of the field investigation in June 2008. A temporary piezometer was placed in Boring B-8 and the water level was recorded one day after drilling. Although direct observation of groundwater was not observed in the remaining borings, wet soil conditions were observed at similar depths. Groundwater was observed at the time of drilling in Boring B-3 of the URS 2003 investigation at a depth of 34 feet bgs.

SECTION 4 DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

The discussions, conclusions, and recommendations presented in this report are based on information provided to us, review of available information, results of our field investigation, pressuremeter testing, laboratory testing, empirical correlations, engineering and geologic analyses, and professional judgment.

4.1 GEOLOGIC HAZARDS

4.1.1 Surface Faulting Hazard

Based on a review of previous investigations, published and unpublished mapping and an analysis of historic stereographic aerial photographs, surface faulting is not considered a significant hazard at the site.

4.1.2 Seismic Coefficients

Seismic coefficients have been developed in accordance with the seismic criteria provided in the 2007 California Building Code (CBC). Based on the site location and site conditions described in Section 3, the values listed in Table 1 should be used for design.

4.1.3 Liquefaction and Seismic Compaction

Seismically induced liquefaction is a phenomenon in which loose, saturated, nonplastic (typically granular) materials develop high pore water pressure and lose strength because of ground vibrations induced by earthquakes.

The site specific subsurface information gathered as part of this geotechnical investigation and for previous investigations at the Substation typically demonstrates a low liquefaction potential for the site. The groundwater surface is estimated at a depth of approximately 36 feet below the site, so the upper sands are not saturated. Below groundwater, the materials encountered in the majority of our borings to the depths explored (51.5 feet) are typically fine grained and not considered susceptible to liquefaction. Liquefaction is typically limited to within 50 feet of the ground surface.

A two to four foot thick layer of potentially liquefiable material was encountered at a depth of about 35 feet bgs in Borings B-7 and B-8 and was also observed just below groundwater in Fugro Boring B-3. This potential for liquefaction should be considered as part of foundation design for the future transmission line or other foundations in Zone A (see Figure 2) of the Substation. Liquefaction-induced settlement is estimated to be on the order of one inch in this area.

Seismic compaction occurs in loose to medium dense dry sandy materials above groundwater due to particle rearrangement during seismic shaking. The magnitude of seismic compaction at the Substation is estimated to be on the order of 1 inch or less for the soil profile variability and the range of ground motions anticipated at the site. The higher seismic settlements are expected to be on the south side of the site where coarse-grained alluvial material dominates the near-surface.

4.1.4 Expansion and Collapse Potential

Fine-grained soils with expansion potential are present in the near surface in many areas of the site. The occurrence of clayey soils is greater in the northern half of the site.

Benton considered the clay, sandy clay, and silty clay encountered at the site to have high potential for expansion and the silt and sandy silt to be non expansive to slightly expansive. For the construction of the Substation, they recommended that the northerly half of the site be capped with at least 3 feet of nonexpansive soil. Borings B-1 through B-4 performed for this investigation in the northern portion of the site demonstrated granular (non expansive) materials in the near surface. One Expansion Index (EI) test, performed on a sample from the recently excavated Bank 82 mat foundation subgrade indicated a low expansion potential.

Loosely deposited alluvium can be subject to collapse due to wetting and/or inundation. Collapse can occur in dry soils that have an unstable soil structure due to deposition or irrigation processes, typically with a skeletal structure that is weakly cemented by soluble salts or clay. Increases in moisture content can cause the interparticle cementation to reduce, causing changes in volume (collapse), especially when loaded. The Substation site is developed and graded to drain to prevent inundation. In addition, the existing site has been subject to storms since the original construction, and is not likely to experience additional collapse settlement. Therefore, the potential for collapse settlement to affect the planned improvements is low.

Given the variable nature of materials across the site, expansion and collapse potential should be evaluated on a foundation specific basis. Mitigation of expansion and collapse potential can be performed by selective earthwork practices during construction if needed.

4.1.5 Other Hazards

The local geologic conditions indicate that the probability of other geologic hazards (such as slope instability, subsidence, seiches, tsunamis, and flooding) affecting the site is very low. There are no significant slopes near the project; therefore, slope instability is not considered a hazard. The potential for seiches or tsunamis to affect the site is considered low given the site location. Similarly, the site is not in an active flood plain and the risk from flooding is considered low. Land subsidence resulting from fluid withdrawal is not currently occurring in the site vicinity and is not a hazard at the site. These hazards do not constitute constraints to site development.

4.2 EARTHWORK

4.2.1 General

Site earthwork will generally consist of removal of unsuitable (loose, soft, expansive or collapsible) material, foundation excavations, and backfills of utility trenches. Earthwork should be performed in accordance with SDG&E requirements and the latest edition of the Standard Specifications for Public Works Construction ("Green Book"). A preconstruction conference should be held at the site with the owner, contractor, civil engineer, and geotechnical engineer in attendance.

4.2.2 Site Clearing and Demolition

Any vegetation and construction debris within areas that are to be improved should be cleared and properly disposed of off-site. Roots and other vegetative matter, if encountered, should be removed and disposed of off-site.

Existing infrastructure within areas that are to be improved should be properly demolished and disposed of at an appropriate facility off-site. Existing utilities may be abandoned by backfilling with sand-cement slurry, subject to approval by the Geotechnical Engineer.

4.2.3 Subgrade Preparation

Foundation and slab-on-grade subgrades should be inspected by a representative of the Geotechnical Engineer to observe and document the absence of loose, soft, potentially expansive, or collapsible soils. If expansive soils are encountered at the foundation level, the potentially expansive material should be removed and replaced with nonexpansive material as directed by the Geotechnical Engineer. If collapsible soils are encountered at the foundation level, they should be prewetted to initiate collapse prior to the construction of the foundation, as directed by the Geotechnical Engineer.

The surface within areas to receive fill should be scarified, moisture conditioned as necessary, and compacted prior to fill placement. Areas temporarily vacated during earthwork should be similarly scarified, moisture conditioned and reworked to the satisfaction of a Geotechnical Engineer before placing additional fill to avoid drying and lamination along the fill interface.

Demolition excavations should be backfilled and compacted with suitable material. Each lift of fill should be benched into competent material to the satisfaction of the Geotechnical Engineer. The minimum width and height of the bench should not be less than the lift thickness. Likewise, contiguous areas that have been disturbed by demolition should be removed to the satisfaction of the Geotechnical Engineer and replaced with engineered fill.

4.2.4 Fill Materials

In our opinion, based on the laboratory testing completed for this study, the granular alluvial materials should be suitable for Common Fill and Select Fill as defined by SDG&E. Some of the clayey lacustrine materials demonstrated high plasticity characteristics and do not meet the requirements for Select Fill. If blended with nonplastic granular materials, the clayey materials would likely meet the requirements of Common Fill and may meet the requirements of Select Fill. Blending operations would require selective stockpiling during grading. We also note that some of the insitu materials could be corrosive to metal based on the limited corrosion potential laboratory testing completed for this study.

The onsite materials are generally suitable for use as engineered fill. Perishable, spongy or other compressible material should not be used for engineered fill. The following material types are applicable to the project:

• <u>Common Fill</u>. This material should consist of native or import soils that are approved for use by the Geotechnical Engineer. These materials should generally be granular soils (less than 50%

passing the No. 200 sieve) that are not excessively plastic (plasticity index less than 40) and/or 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 is satisfactory to the Geotechnical Engineer.

- <u>Select Fill</u>. This material may consist of on-site excavated soil or imported soils that are approved by the Geotechnical Engineer. The material should not contain rocks or hard lumps greater than 3 inches in maximum dimension and at least 40% of material should be smaller than ¹/₄-inch in size. In addition, the material should have an Expansion Index of less than 50, a Liquid Limit (LL) less than 30 and a Plasticity Index (PI) less than or equal to 15.
- <u>Class 2 Aggregate Base Material</u>. Aggregate base material should conform to the State of California Department of Transportation (Caltrans) "Standard Specifications" Section 26-1.02A Class 2 Aggregate Base.

4.2.5 Fill Placement and Compaction

Fill material should be placed in loose lifts no thicker than 8 inches, moisture conditioned, and processed as necessary to achieve uniform moisture content above optimum. Each lift should be compacted to not less than 90% relative compaction. Relative compaction is defined as the ratio of the in-place dry density to the maximum dry density determined using the latest version of ASTM D1557 as the compaction standard.

Class 2 aggregate base should be compacted to 95% of the maximum dry density as determined by ASTM D1557. Each lift should be compacted before the next lift is placed, except where specifically designated by the Geotechnical Engineer to facilitate mixing of materials.

4.3 SHALLOW FOUNDATIONS

Vault structures, transformers, switch stands, circuit breakers, control buildings, and other lightly loaded structures may be supported on conventional shallow spread and continuous footings or mat foundations.

4.3.1 Footing Dimensions and Embedment

The recommended minimum spread or strip footing embedment depth is 12 inches below finished grade. The recommended minimum spread or continuous foundations width is 12 inches. The recommended minimum mat foundation width is 5 feet. The Structural Engineer should determine the footing embedment, size and reinforcement based on anticipated loads and estimated differential 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).

4.3.2 Allowable Foundation Pressure

Shallow foundations consisting of continuous and isolated spread footings or mat foundations bearing on engineered fill, alluvial, or lacustrine deposits may be designed using an allowable bearing pressure of 1,500 pounds per square foot (psf). For footings deeper or wider than 12 inches, the allowable bearing pressure may be increased by 500 psf for each additional 12 inches of depth, or by 1,000 psf for each

additional 12 inches of width, up a maximum of 4,000 psf. Allowable bearing pressures may be increased by 33 percent for short term wind or seismic loads.

4.3.3 Allowable Lateral Bearing

Resistance to lateral loads on the shallow foundations may be provided by passive resistance along the outside face of footings and frictional resistance along the bottom of the 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 or native allovial or lacustrine deposits.

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 resistance is combined, the allowable friction coefficient should be reduced to 0.3.

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 be neglected where utilities or similar excavations may occur in the future.

4.3.4 Settlement

The settlement of a shallow foundation for a given allowable bearing pressure will depend upon the size, shape, embedment depth of the foundation, the relative compaction and stiffness of the fill or the relative density of underlying native materials, in addition to other factors.

A total settlement of less than one inch has been estimated for isolated spread and strip foundations designed with the minimum allowable vertical foundation pressures provided in this report and a minimum embedment depth of 12 inches. This settlement could increase by up to 50 percent if the design adopts the maximum allowable bearing pressure for increased embedment. This settlement estimate only considers dead loads. The maximum differential settlement between identical footings supporting similar loads should not exceed ¹/₂-inch, when only building loads are considered. The majority of the settlement due to building loads should occur during construction.

A total settlement of about 2 inches has been estimated for the static load and foundation size provided in Section 1.1 for the transformer mat foundations that are part of the Bank 82 Addition. Approximately half of the total should occur due to elastic settlement as the load is applied; the remaining settlement will result from consolidation of the clay below the groundwater level and is expected to be substantially complete within 2 to 6 months. Four mat foundations are planned, approximately 7.5 feet apart. Adjacent mat foundations will impose some additional stresses at depth below the edge of the mat under consideration; the additional consolidation settlement is estimated to be approximately ¹/₂-inch; however, it is expected that the stiffness of the mat should attenuate the majority of this additional settlement. Some additional minor settlement may occur upon the application of live loads.

Similar settlement may occur for other large mat foundations at the site. Actual settlement will depend upon the location on the site (due to varying subsurface conditions) and the foundation size and loading. Evaluation of individual mat foundations should be performed once this information is available. If the estimated settlements cannot be tolerated by the structures, mitigation could include the following:

- Preloading the site using a surcharge fill or other load;
- Staging the construction to allow the majority of the settlement to occur prior to construction of critical elements;
- Supporting the mats on drilled piers or mini piles. Foundations would likely need to extend about 25 to 30 feet below the ground surface to reduce the elastic settlement and 40 to 50 feet deep to substantially reduce total settlement; and
- Improving the settlement properties of the underlying clay soil by soil improvement techniques such as deep soil mixing with the addition of lime or cement. Improvement depths would also likely need to extend 40 to 50 feet.

The structural or soil improvement methods are likely to be costly, although they could be performed relatively quickly if schedule is an issue. If scheduling and logistics permit, preloading and/or staging the construction is likely to be the most cost effective option. Settlement monitoring is recommended during preloading or staged construction.

4.3.5 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 150 and 100 pounds per cubic inch for undisturbed granular alluvial or clayey lacustrine deposits, respectively.

4.4 DEEP FOUNDATIONS

Drilled pier foundations should have a minimum diameter of 2 feet and a minimum embedment of 6 feet below the ground surface. The base of the pier excavations should be free of any loose or disturbed materials.

The planned foundations are not anticipated to extend deeper than 30 feet below grade. If deeper foundations are planned in the future, URS should be contacted for further recommendations. In particular, the potential for liquefaction below 30 feet should be considered for foundation design in Zone A (Figure 2).

4.4.1 Pressuremeter Tests Results

As discussed previously, the pressuremeter testing performed for this project is the start of an effort to collect elastic pressuremeter modulus (E_{pmt}) data for the various formations and deposits present in the SDG&E service area. The E_{pmt} is a parameter required for the lateral pile design methodology used by SDG&E. Historically, correlations with SPT N value and soil properties have been used to estimate E_{pmt} , however, little field testing has been performed in the San Diego area to validate the parameters used for design. While field data can be technically applied only to the location of the test, these results and future pressuremeter test results provide a database of information that can be used to calibrate results for similar materials and to help estimate parameters where pressuremeter testing is not performed.

The pressuremeter testing at the Imperial Valley Substation was performed in Quaternary alluvial and lacustrine deposits of the Imperial Valley. The results of the testing are summarized in Table C-1 in Appendix C. The E_{pmt} in the clayey lacustrine deposits ranged between 1.5 and 2.2 kips per square inch (ksi), excluding the single highest and lowest values. The E_{pmt} in the alluviual sands and silty sands ranged between 1.2 and 1.6 ksi, excluding the single highest and lowest values. One test was performed in clayey sand, which resulted in an E_{pmt} of 1.9 ksi, intermediate between the clay and sand. Two tests were performed in silt, which can be a highly variable material, with results of 1.8 and 2.7 ksi. No substantive correlations of increasing E_{pmt} with increasing depth or density/stiffness were observed with the data.

Other than a few outliers, the measured E_{pmt} values were within a relatively narrow range for each material type, and are generally within the range that would be predicted using blowcount and other empirical correlations.

4.4.2 MFAD Design Parameters

We understand deep foundations at the site will be designed for lateral loads using the Electric Power Research Institute (EPRI) computer program, Moment Foundation Analysis and Design (MFAD). This program analyzes and designs drilled shaft foundations subject to high overturning moment loading. The design soil parameters required to use the MFAD program include:

- Soil Layer Depths
- Groundwater Depth
- Total Unit Weight
- Internal Friction Angle
- Cohesion
- Elastic Pressuremeter Modulus
- Strength Reduction Factor

Estimates of the required parameters were developed based on the results of our site observations, borings, laboratory testing, engineering evaluation and analysis, empirical correlation, literature research, and professional judgment. The estimated design parameters are presented in Table 2. Due to the variety of subsurface conditions, the Substation site was divided into Zones A through H, as shown on Figure 2, for the purpose of providing MFAD recommendations.

We recommend a design groundwater depth of 30 feet below the existing ground surface. The design does not need to discount surficial soils within the Substation; however, the upper 1 to 2 feet of soil should be discounted in design of pole foundations outside the Substation that may be subject to erosion. It should be noted that the design parameters presented in Table 2 are intended for use in the MFAD computer program and may not reflect actual strengths.

4.4.3 Allowable Skin Friction

Deep foundations at the site will also be designed to resist axial compression and uplift loads. The allowable skin friction on the sides of the drilled pier begins at grade and increases with depth as indicated on Figure 7 for downward loads and Figure 8 for uplift loads. The weight of concrete of the drilled pier was not included in our analyses but may be added to resist uplift. The allowable downward load can be increased by one third for loads that include wind or seismic forces.

4.4.4 Capacity Reduction for Group Effects

Construction of deep foundations in groups reduces the available capacity of drilled piers due to the relaxation of the soils within the adjacent foundation excavation. Design of piers spaced closer than 8 pier diameters on center can have total axial (downward and uplift) and lateral capacity less than the sum of the capacities of the individual piers.

A table of group efficiencies for piers founded in granular soils is presented in Table 3. The axial group efficiency effect can be incorporated by reducing the allowable skin friction and end bearing values to obtain the allowable axial capacities. The lateral group efficiency effect can be incorporated by magnifying the loads on the piers by the reciprocal of the efficiency.

For piers founded in clayey soils, group effects are generally less significant and the efficiency is evaluated based on the geometry of an equivalent pier having the shape of the outside boundary of the group. The piers for the Bank 82 firewalls (Zone C) are founded predominantly in clayey soils and spaced approximately 5 pier diameters apart. Based on our evaluation of the group block, no group capacity reduction is needed for these foundations.

4.5 SLABS-ON-GRADE

Slab-on-grade concrete floors for control buildings or similar facilities should be at least four inches thick. The Structural Engineer should design the thickness and reinforcement of concrete slab-on-grade floor slabs to accommodate concentrated loads and heavy distributed loads. Expansion joints and crack control sawcuts should be included at regular intervals.

4.6 SURFACE DRAINAGE

Positive measures should be taken to properly finish grade the area to direct drainage waters away from foundations and floor slabs. All runoff water should be directed to proper drainage areas and not be allowed to pond.

Even when these measures have been taken, experience has shown that a shallow groundwater or surface water condition can develop in areas where no such water condition existed prior to site development.

4.7 PAVEMENTS

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. For preliminary evaluation purposes, we have adopted a Traffic Index

(TI) of 5.0. The traffic index should be confirmed prior to final design. We have assumed a Resistance-Value (R-value) of 35 as representative of the as-graded subgrade conditions consisting of non-expansive granular materials. Confirmation R-values test should be completed on samples obtained from the final subgrade materials where pavements are planned.

For preliminary design purposes, we recommend that the pavement structural section within the Substation consist of at least 3 inches of asphalt concrete over 5 inches of Class 2 Aggregate Base. The section assumes a properly prepared subgrade consisting of at least 12 inches of soil compacted to a minimum of 95% relative compaction. The aggregate base materials should be placed at a minimum relative compaction of 95%. Construction materials (asphalt and aggregate base) should conform to the current Standard Specifications for Public Works Construction.

4.8 CORROSION POTENTIAL

The results of pH, resistivity, and water-soluble sulfate and chloride tests are summarized in Table 4. It has been our experience with local corrosion engineers that resistivity results between 0 and 1,000 ohmcentimeters (ohm-cm) may be considered very corrosive and between 1,000 and 2,000 may be considered fairly corrosive to metallic utility piping and conduits. The corrosion testing performed by Benton for the construction of the Substation indicated severely corrosive materials are present on the site. A corrosion engineer should be consulted for additional design information.

The results of the testing indicate that the potential for chloride attack is low to moderate. The results of the tests indicated that sulfate attack to concrete may be considered negligible.

4.9 SUBSTATION EQUIPMENT SEISMIC QUALIFICATION LEVEL

The selection of the seismic qualification level for the performance evaluation of Substation equipment is based on IEEE Standard 693-2005. Following the seismic exposure map methodology presented in Section 8.6 of the IEEE Standard (IEEE 2005), a high qualification level is suggested based on a calculated peak ground acceleration of 0.57g. Table 5 presents the selected and calculated values following the procedures outlined in IEEE 693-2005 and based on the 2006 International Building Code (IBC) and the Maximum Considered Earthquake (MCE) ground motion presented on regional seismic hazard maps.

4.10 CONSTRUCTION CONSIDERATIONS

4.10.1 Excavation Characteristics

Trench excavation is expected to encounter little difficulty using modern trenching machines or backhoes, except where there are remnant obstructions associated with demolition. Conventional earth moving equipment (large diameter auger rigs, excavators, dozers, scrapers, etc.) should be able to excavate the soils encountered at the site with no unusual difficulty. Dry, cohesionless sands were observed in the borings. Vertical or steeply sloping excavations for trenches or drilled piers in cohesionless materials will not stand without predrilling stabilization, prewetting and/or casing or shoring.

4.10.2 Temporary Slopes

The design and excavation of temporary slopes as well as their maintenance during construction is the responsibility of the contractor. The contractor should have a geotechnical or geological professional evaluate the soil conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by California OSHA (Cal OSHA).

Based on the existing data interpreted from the borings, the design of temporary slopes and benches for planning purposes may assume the conditions summarized in Table 6. The contractor's geotechnical or geological professional may use the information provided in this report to assess the stability of temporary slopes, as well as any additional data they may need to acquire, to prepare a specific temporary slope analysis and design. Existing infrastructure that is within a 2:1 (horizontal:vertical) line projected up from the toe of temporary slopes should be monitored during construction.

The contractor should note that the materials encountered in construction excavations may vary significantly across the site. The above assessment of soil type for temporary excavations is based on preliminary engineering classifications of material encountered in widely spaced explorations. The contractor's geotechnical or geological professional should observe and map mass excavations and temporary slopes at regular intervals during excavation and assess the stability of temporary slopes, as necessary.

4.10.3 Construction Observation and Testing

Earthwork and placement of engineered fill should be performed under the observation and testing services of a geotechnical professional supervised by a California–registered Geotechnical Engineer. Tests should be taken to determine the in-place moisture and relative compaction of engineered fill.

Removal excavations should be observed and mapped by a geologic or geotechnical professional during grading. All soils at foundation level should be observed by a geotechnical or geologic professional to observe that the subgrade is satisfactory. Excavations should be free of soft fill or loose and disturbed soils. The installation of drilled piers should also be observed.

A California-registered Geotechnical Engineer should prepare a final report of earthwork and foundation construction testing and observation at the completion of the project.

SECTION 5 LIMITATIONS

We have observed only a very 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 our field investigation. Specific details for may of the proposed projects are not available at this time. The recommendations presented in this report are intended to assist SDG&E and their consultants in the planning and design of the projects. The professional judgments and interpretations presented in this report are based on our current knowledge of the proposed improvements, our interpretations of the subsurface conditions in the project area, and our understanding of the geologic and tectonic setting of the project site. This knowledge is based on the information provided to us, published literature, previous studies by others, and our investigations.

We recommend that URS review the foundation plans for the currently proposed and future improvements to verify that the intent of the recommendations presented herein has been properly interpreted and incorporated into the contract documents. We further recommend that any site grading and earthwork, subgrade preparation under concrete slabs and paved areas, utility trench backfill, and foundation excavations be observed by a qualified engineer or geologist to verify that site conditions are as anticipated, or to provide revised recommendations, if necessary.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet current professional standards; we do not guarantee the performance of the project in any respect.

SECTION 6 REFERENCES

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6-2

Table 1
2007 CBC Seismic Coefficients
Imperial Valley Substation

Parameter	Value	2007 CBC Reference		
Site Class	D	Table 1613.5.2		
Mapped Spectral Acceleration - Short Period, $S_s(g)$	1.45	Figure 1613.5 ¹		
Mapped Spectral Acceleration - 1 Sec. Period, S ₁ (g)	0.56	Figure 1613.5 ¹		
Site Coefficient - Short Period, F_a	1.0	Table 1613.5.3(1) ¹		
Site Coefficient - 1 Sec. Period, F_{ν}	1.5	Table 1613.5.3(2) ¹		
MCE ² Spectral Response Acceleration - Short Period, S _{MS} (g)	1.45	Equation 16-37, $S_{MS}=F_aS_S$		
MCE ² Spectral Response Acceleration - 1 Sec. Period, S _{M1} (g)	0.85	Equation 16-38, $S_{M1}=F_vS_1$		
Design Spectral Response Acceleration - Short Period, S _{DS} (g)	0.96	Equation 16-39, SDS=2/3*SMS		
Design Spectral Response Acceleration - 1 Sec. Period, S _{D1} (g)	0.56	Equation 16-40, S _{D1} =2/3*S _{M1}		

Notes:

Calculated using USGS program "Earthquake Ground Motion Parameters" Version 5.0.8.
 MCE – Maximum Considered Earthquake.

3. Site latitude and longitude obtained from Google Maps: 32.7181; -115.7156 for the center of the Substation.

Table 2
MFAD Design Parameters and Subsurface Characterization
Imperial Valley Substation

Depth Below Proposed Grade (feet)	Anticipated USCS Range	Total Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	E _{pmt} (ksi)	Shear Strength Reduction Factor, α	
Zone A – Future Transmission L	ine						
0 to 8	SM, SP-SM, SP	110	33	0	1.5	1.0	
8 to 13	CL	110	29	0	1.75	0.6	
13 to 24	SM, SP-SM, SP	110	33	0	1.5	1.0	
24 to 30	CL	110	29	0	1.75	0.6	
Zone B – 2 nd 500 kV Tie Line							
0 to 18	SM to SP-SM	110	33	0	1.5	1.0	
18 to 30	CL	110	29	0	1.75	0.6	
Zone C – Bank 82 Addition	Zone C – Bank 82 Addition						
0 to 8	SC	110	33	0	1.5	1.0	
8 to 14	СН	120	27	0	1.75	0.6	
14 to 30	CL	110	29	0	1.75	0.6	
Zone D – New 230 kV Yard							
0 to 8	SP to SP-SC	110	33	0	1.5	1.0	
8 to 14	СН	120	27	0	1.75	0.6	
14 to 30	CL	110	29	0	1.75	0.6	

Notes:

1. The design groundwater level is 30 feet below existing grade.

2. No discount depth is recommended for deep foundations within the Substation. Foundations outside the Substation should include a discount depth of 1 to 2 feet.

3. USCS acronyms defined on the boring logs.

4. Epmt = Modulus of deformation as would be determined from a pressuremeter test in kips per square inch.

Table 2 Recommended MFAD Design Parameters and Subsurface Characterization Imperial Valley Substation (Continued)

Depth Below Proposed Grade (feet)	Anticipated USCS Range	Total Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	E _{pmt} (ksi)	Shear Strength Reduction Factor, α
Zone E						
0 to 7	SM	110	33	0	1.5	1.0
7 to 10	ML	110	29	0	1.5	1.0
10 to 25	SM	115	33	0	1.5	1.0
25 to 30	SW, SM	120	35	0	2.0	1.0
Zone F						
0 to 5	SM/CL	110	30	0	1.5	1.0
5 to 15	SM	110	33	0	1.5	1.0
15 to 20	SW	110	33	0	1.5	1.0
20 to 30	SP to SM	115	35	0	2.0	1.0
Zone G						
0 to 5	SP	110	33	0	1.5	1.0
5 to 20	ML, CL, CL-ML	115	29	0	1.75	0.6
20 to 25	СН	120	27	0	2.0	0.6
25 to 30	SP to SM	125	35	0	2.0	1.0
Zone H	Zone H					
0 to 15	CL-ML, CL, ML	110	29	0	1.75	0.6
15 to 30	SM	115	33	0	1.5	1.0

Notes:

1. The design groundwater level is 30 feet below existing grade.

2. No discount depth is recommended for deep foundations within the Substation. Foundations outside the Substation should include a discount depth of 1 to 2 feet.

3. USCS acronyms defined on the boring logs.

4. Epmt = Modulus of deformation as would be determined from a pressuremeter test in kips per square inch.

Table 3	
Group Efficiencies for Drilled Piers Founded in Granular Ma	terial
Imperial Valley Substation	

Pier Spacing	Axial Group Efficiency ^{a, c}	Lateral Group Efficiency ^{b, c} (in line w/group)	Lateral Group Efficiency ^{b, c} (perpendicular to group)
2B	0.65	0.76	1.0
3B	0.70	0.80	1.0
4B	0.75	0.84	1.0
5B	0.85	0.88	1.0
6B	0.90	0.92	1.0
7B	0.95	0.96	1.0
8B	1.00	1.00	1.0

Notes:

1. For both downward and uplift capacities.

2. For lateral capacity.
 3. Efficiency factors can be interpolated for intermediate spacings.

Table 4
Summary of Corrosivity Testing
Imperial Valley Substation

Test Location	рН	Minimum Resistivity (ohm-cm)	Water Soluble Sulfates (ppm)	Chloride (ppm)
Boring B-1 Depth of 1ft	8.9	1,750	784	285
Boring B-1 Depth of 10 ft	8.2	350	368	420
Boring B-1 Depth of 30 ft	8.1	240	544	765
Boring B-6 Depth of 10 ft	8.5	2,500	736	240
Boring B-7 Depth of 3 ft	7.0	5,000	816	195
Caltrans Guidelines Indicate Corrosive Environment if:	<5.5	<1,000	>2,000	>500

Table 5
Seismic Qualification Level Calculation
Imperial Valley Substation

Parameter	Value	Reference
Site Soil Class	D	2006 IBC Table 1615.1.1
MCE Ground Motion 0.2s Spectral Response Acceleration, Ss	1.45 <i>g</i>	2006 IBC Figure 1615 (3)
Site Coefficient, F _a	1.0	2006 IBC Table 1615.1.2 (1)
Adjusted MCE Spectral Response Acceleration -short period, S_{ms} (= S_sF_a)	1.45 <i>g</i>	IEEE 8.6.2.1 (d) ; IBC Equation 16-38
Peak Ground Acceleration (PGA) for seismic qualification selection $(S_{ms}/2.5)$	0.57 <i>g</i>	IEEE 8.6.2.1 (e)
Selected Seismic Qualification Level	High	IEEE 8.6.2.1 (f)

Note:

1. g = indicates units of gravity

Table 6
Preliminary Cal OSHA Soil Types
Imperial Valley Substation

Geological Unit	Cal OSHA Soil Type
Fill	Туре С
Fine Grained Deposits	Туре В
Granular Deposits	Туре С



Jun 08, 2009 – 1:17pm X:\27668011\IV Substation Vicinity Map.dwg


LI	EGEND
B-1	INDICATES APPROXIMATE LOCATION OF TEST BORING (THIS INVESTIGATION)
- (В-1	➡ INDICATES APPROXIMATE LOCATION OF TEST BORING (URS, 2003)
17 (INDICATES APPROXIMATE LOCATION OF TEST BORING (BENTON ENGINEERING, 1980)
в-3	INDICATES APPROXIMATE LOCATION OF TEST BORING (FUGRO, INC. 1980)
	A INDICATES APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION
	- INDICATES ZONE BOUNDARY (URS, 2008)
NOTE Loca 1980 with rougl	: tions of Benton 1980 borings and Fugro Inc, borings approximated from historical plans no common reference and are considered n approximates.
- <u>SEE_IV-S620 & IV-S</u> 6	321
12KV SL&P (IID SOURCE)	
EL CENTRO (IID) TL 230S SEMPRA #1 TL 23048	
TL 23047, <u>INTERGEN #2</u> TL 23046, <u>INTERGEN #1</u>	
TL 23045. <u>FUTURE</u> 230KV LINE	
LA ROSITA (CFE) TL 23050	
REFERENCE "500/230 Imperial Va Rev No. 23	: <v arrangement",<br="" general="" switchyard="">lley Substation, Drawing No, IV—S—501, 6, SDG&E (undated).</v>

SDG&E - IMPERIAL VALLEY SUBSTATIC IMPERIAL VALLEY, CALIFORNIA

100 200'	CHECKED BY: JLN		DATE: 06-11-09		FIG. NO:	
200'	PM: KCG	PROJ.	NO: 2766	8011.00010	2	



2.5 5 miles	CHECKED B	Y: JLN	DATE: 06-11-09	FIG. NO.
5 miles	PM: KCG	PROJ.	NO: 27668011.00010	3



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LEGEND:

REPORTED EARTHQUAKE MAGNITUDES.

- 4.0 TO 4.9
 5.0 TO 5.9
- 6.0 TO 6.9
- 7.0 AND GREATER

SOUTH OF LATITUDE 35'N, EPICENTER AND MAGNITUDE DATA ARE FROM THE CALIFORNIA GEOLOGICAL SURVEY (2000) EARTHQUAKE CATALOG FOR THE PERIOD FROM 1769 TO 2000. ONLY EARTHQUAKES OF MAGNITUDE 4.0 AND LARGER ARE SHOWN. AFTERSHOCKS ARE NOT INCLUDED.

APPROXIMATE LOCATION OF HOLOCENE AND PLEISTOCENE FAULTS, DOTTED WHERE CONCEALED, QUERIED WHERE CONJECTURAL. FAULT LOCATIONS BASED ON: ZIONY AND JONES, 1989; CDMG GEOLOGIC MAP SERIES OF CALIFORNIA, 1977–1986 (1:250,000 SCALE); CDMG GEOLOGIC MAP SERIES, CALIFORNIA CONTINENTAL MARGIN, 1987 (1:250,000 SCALE); HAUKSSON, 1990; SHAW AND SHEARER, 1999; WRIGHT, 1991; GRANT ET AL. 1999.

REGIONAL FAULT AND EPICENTER MAP SDG&E - IMPERIAL VALLEY SUBSTATION IMPERIAL VALLEY, CALIFORNIA

10 20 Miles	CHECKED B	r: JLN	DATE:	06-10-09	FIG. NO:
0 miles	PM: KCG	PROJ.	NO: 2766	68011.00010	4





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B'



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LEGEND

BORING NUMBER, (REFERENCE), PROJECTD DISTANCE AND DIRECTION STANDARD PENETRATION TEST (SPT) SAMPLE LOCATION MODIFIED (MODCAL) CALIFORNIA SAMPLE LOCATION

36 SPT N-VALUE BLOWCOUNT (REDUCED BY 0.8 FOR URS MODCAL SAMPLES)

– — — APPROXIMATE LITHOLOGIC CONTACT

REPORTED GROUNDWATER AT TIME OF DRILLING

SP, SM USCS SYMBOL – GRANULAR ALLUVIAL DEPOSITS

ML, CL USCS SYMBOL – FINE–GRAINED LACUSTRINE DEPOSITS

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NOTES: 1. Locations of Benton 1980 borings and Fugro Inc, 1980 borings approximated from historical plans with no common reference and are considered rough approximates.

2. Ground surface approximated from as-built plans for substation construction.



GENERALIZED GEOLOGIC CROSS SECTIONS C-C' AND D-D' SDG&E - IMPERIAL VALLEY SUBSTATION IMPERIAL VALLEY, CALIFORNIA





Copies of boring logs and laboratory test data from previous investigations by URS and others are provided in this appendix.

BENTON, 1980

SOILS INVESTIGATION

PROPOSED IMPERIAL VALLEY SUBSTATION PHASE III, SITE 7A IMPERIAL COUNTY, CALIFORNIA

Prepared for the

SAN DIEGO GAS & ELECTRIC COMPANY

by

BENTON ENGINEERING, INC.

PROJECT NO. 80-9-10A NOVEMBER 14, 1980

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BENTON ENGINEERING, INC. APPLIED BOIL MECHANICE — FOUNDATIONS BBAD RUFFIN ROAD SAN DIEGO, CALIFORNIA \$2123

PHILIP HENKING BENTON PRESIDENT - GIVIL ENGINEER

SOILS INVESTIGATION

TELEPHONE (714) 565-1955

Introduction

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This is to present the results of a soils investigation conducted at Site 7A of the proposed Imperial Valley Substation of San Diego Gas & Electric Company in Imperial Valley, San Diego County, California.

The site was previously investigated by FUGRO, INCORPORATED of Long Beach, California In order to study its feasibility for meeting construction permit requirements. In their study, five borings were drilled at the site, and the findings from the borings were used as reference for this investigation.

It is understood that the objectives of this investigation were to develop certain soil parameters, commensurate with the tested soil properties, for the foundation design of the proposed substation.

In order to accomplish these objectives, twenty-one borings were drilled by our firm, and both loose and undisturbed soil samples were obtained for laboratory testing.

This investigation was based on our proposal to San Diego Gas & Electric Company dated August 15, 1980.

Field Investigation

Borings 1 to 20 inclusive were drilled 4.5 inches in diameter with a rotary wash rig and Boring 21 was drilled 30 inches in diameter with a 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 12 to 50 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 44, 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, where possible, in the soils ahead of the drilling. The drop weight used for driving the sampling tube into the soils was either 300 pounds for the rotary wash drill rig or 3,770 pounds for the rotary bucket-type drill rig. The average drop was 12 inches for the rotary bucket-type drill rig and 30 inches for the rotary wash drill rig. The general procedures used in field sampling are described under "Sampling" in Appendix B. During the course of drilling, the blow counts required to drive a standard splitspoon sampler for a distance of 6 inches were recorded. The recorded blow counts are shown on the attached Drawing Nos. 2 to 44, inclusive. To obtain the required blow counts for driving the sampler for a distance of 12 inches, the last two recorded blow counts should be combined. Laboratory Tests

Laboratory tests were performed on all undisturbed samples of the soils in order to determine the dry density, moisture content and shearing strength. To test the shearing strength of the soils, the soil samples were sheared under a normal pressure equal to their effective overburden pressures. For those recovered above ground water table, the soil samples were sheared under field moisture condition. For those recovered below water table, they were sheared in a saturated undrained condition. The results of these tests are presented on Drawing Nos. 2 to 44, inclusive. Consolidation tests were performed 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. 45 to 52, inclusive each entitled "Consolidation Curves." The general procedures used for the laboratory tests are described briefly in Appendix B.

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In addition to the above laboratory tests, expansion tests were performed on some of the clayey soils encountered to determine their volumetric change characteristics with change In moisture content. The recorded expansions of the samples are presented as follows:

Boring	Sample	Depth of Sample, in Feet	Soil Description	Under Unit Load of 500 Pounds per Square Foot from Air Dry to Saturation
<u>No.</u>		2.5	Clayey Silt	0.27
14 20	1	2.5	Silty Clay Clayey Silt	3.1 1 6.71

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California Bearing Ratio tests were performed on representative samples of the subgrade soils at the site. The tests were performed in accordance with A.S.T.M. D1883 and D1557 Method D modified per City of San Diego standards. The test results are presented below:

CBR Sample No	Depth in Feet	Molded Dry Density Ib/cu ft	Initial Moisture Content % dry wt	Pene- tration Inch	Lood in Pounds on 3 Sq. In. Plunger	CBR % of Std.	Percent Expansion During Soaking	Percent Moisture After Penetration
2	0 to 1.0	110.7	7.0	0.1 0.2 0.3 0.4 0.5	316.0 527.8 558.5 589.1 650.3	10.5 11.7 9.8 8.5 8.3	0.06	17.0
4	0 to 1.0	114.8	17.3	0.1 0.2 0.3 	110.6 229.1 323.9 402.9	3.69 5.09 5.68 5.84 5.98	3.46	25.6
5	0 to 1.0	121.8	15.4	0.1 0.2 0.3 0.4 0.5	102.7 165.9 229.1 284.4 331.8	3.42 3.69 4.02 4.12 4.25	5.04	25.6

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Unconfined compression tests were conducted on certain clayey soil samples in order

to determine their strength characteristics. The results of the tests are presented below:

Boring Nos.	Sample Nos.	Depth Below Existing Ground Surface In Feet	Soil Description	Compre ssive Strength (KSF)	Shearing Strength (KSF)
9	3	10.0	Clay	13,02	6.51
-	4	15.0	Clay	5.48	2.74
	5	20.0	Silt with lenses of silty clay	3.36	1.68
11		2.5	Silty clay	14.57	7.28
••	2	5.0	Stilty clay with silt lenses	4.71	2.35
	3	10.0	Clay	11.99	6.00
	4	15.0	Silt, thinly bedded	3,00	1.50
12 -	i	2.5	Silty clay with silt lenses	8,52	4.26
	2	5.0	Silty clay with silt lenses	20.74	10.37
	3	10.0	Clay	9.72	4.86
	4	15.0	Silt interbedded with sandy silt	2.68	1.34
	5	20.0	Silt with lenses of silty clay	4.16	2,08
13	3	10.0	Clay	5.34	2.67
14	4	15.0	Clay	6.22	3.11
15	1	2.5	Clay	12.33	6.17
	2	5.0	Silt with lenses of silty clay	0.77	0.39
	3	10.0	Silt with lenses of silty clay	4.81	2.41
16	1	2.5	Silt interbedded with silty clay	15.05	7.53
	2	5.0	Silt interbedded with silty clay	14.83	7.42
	3	10.0	Fine sandy silt	3.42	1.71
17	Ĩ	2.5	Silty clay with thin layers of silt	8.68	4.34
20	3	10.0	Silt with lens of fine sandy silt and silty clay	1.40	0.70

Chemical analysis was conducted on selected loose soil samples in order to determine their corrosive characteristics. The tests were conducted by the Environmental Engineering Laboratory of San Diego, California. Their test results are attached at the end of this report as Appendix C.

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In order to differentiate the strength characteristics of soils between field moisture and saturated conditions, certain selected soil samples were also tested under both field moisture

and saturated conditions. The results of the tests are presented below:

Boring Nos.	Sample Nos.	Sample Depth Feet	Normal Load Kips/Sq.Ft.	Maximum Shear Load Tested Under Field Moisture Condition Kips/Sq.Ft.	Maximum Shear Load Tested Under Saturated, Undrained Condition Kips/Sq.Ft.
1	3	10.0	1.147	1.38	1.23
1	4	15.0	1.696	1.87	1.78
2	1	2.5	0.245	0.32	0.28
2	3	10.0	1.126	1.41	1,22
2	5	20.0	2,220	2.51	2.16
3 .	6	25.0	2.690	2.70	2.42
4	3	10.0	1,120	1.49	1.22
4	5	20.0	2.260	2.70	2.40
5	4	15.0	1.665	1.94	1.53
5	5	20.0	2.195	2.65	2.17
7	1	2.5	0.245	0.36	0.36
7	3	10.0	1,068	1.50	1.04
7	5	20.0	2.220	2.40	1.96
8	1	2.5	0.245	0.26	0.12
8	5	20.0	2.130	1,99	1.74
10	4	15.0	1.572	1.27	1.06
13	2	5.0	0.550	3.23	0.47
13	4	15.0	1.525	1,51	1.14
13	5	20.0	2.090	2.25	1.58
14	3	10.0	1.100	3.09	0.74
14	5	20.0	2.130	.2.56	1.66
14	· 6	25.0	2.700	3,81	2.13
17	3	10.0	1.210	1.88	1.56
17	4	15.0	1,760	2,31	1.37
17	5	20.0	2.250	2,44	1.21
18.	2	5.0	0.527	1.58	0.67
18	3	10.0	1.017	1.11	0.67
18	4	15.0	1,572	2.88	2.01
18	5	20.0	2.185	3.04	1.94
19	4	15.0	1,810	- 3,86	1.50
20	2	5.0	0.550	1.80	1.14
21	1	4.0	0,493	0.55	0.50
21	2	6.0	0.693	0.65	0.52
21	3	8.0	0.975	1.17	0.91

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Subsurface Conditions

At the site, two distinctive soil deposits, described as alluvial fan deposit and lacustrine deposits of ancient Lake Cahuilla by Fugro Incorporated of Long Beach, California, were also encountered in our twenty-one borings drilled.

The alluvial fan deposit consists mainly of more granular and permeable soils, such as silty sand, slightly silty sand and sandy soils of various grain sizes with occasional inclusion of thin layers of fine sandy silt, fine sandy clay and clay and silty clay soils. The lacustrine deposit of ancient Lake Cahuilla consists mainly of interbedded clay, silty clay, silt and clayey silt which were fine-grained and less permeable.

Based on the results of field exploration, the alluvial fan deposit covers the southern portion of the site where Borings 1 to 8 inclusive, Borings 10 and 21 were drilled. Borings 2 and 4 by Fugro Incorporated were also located in the alluvial deposit area. On the northerly portion of the site, Boring 9 and Barings 11 to 20 inclusive, were drilled in the lacustrine deposit. The two deposits seem to averlap in the vicinity of Barings 4 and 5 drilled by Fugro Incorporated. Along the south boundary of the site, the alluvial fan deposit extended to Elevation -37.19 feet or to a depth of 46.7 feet below the existing ground surface at Baring 4 location. Below 46.7 feet at Baring 4 location, silty clay of lacustrine deposit was encountered. The soils of both deposits were found to be mostly medium firm to very firm or dense to very dense except the upper portion of the site where thin layers of loose to very loose soil mantle were encountered in most of the twenty-one barings drilled. The loose to very loose soil mantle consists mainly of silty sand and slightly silty sand of various grain sizes.

The depths where the loose to very loose soil mantle exists are summarized and presented on the following page.

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Elevation of	Depth Below Existing Ground Surface
Existing Ground	Where Loose to Very Loose Soil Mantle
Surface in Feet*	Was Encountered, in Feet
5,56	0 to 5.0
7.68	0 to 1.5
8.21	0 to 1.5
9.51	0 to 1.0
9,99	0 to 1.0
10.60	0 to 1.5
5.18	0 to 1.0
8.91	0 to 1.0
1.64	Not Encountered
8.95	0 to 1.5
-1.44	Not Encountered
-0.36	0 to 0.8
0.32	0 to 0.8
3.04	0 to 0.4
-1.62	Not Encountered
0.42	0 to 1.0
- 0,48	0 to 0.5
3.22	0 to 2.4
5.89	0 to 1.5
3.06	0 to 0.7
5.06	0 to 1.0
	Elevation of Existing Ground Surface in Feet* 5.56 7.68 8.21 9.51 9.59 10.60 5.18 8.91 1.64 8.95 -1.44 -0.36 0.32 3.04 -1.62 0.42 -0.48 3.22 5.89 3.06 5.06

* MSL datum

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Due to rotary wash type of drilling, no clear ground water level was observed during our field exploration; however, very moist or saturated soil zones were established after soil sampling and laboratory moisture test. The total depth of exploration, the depth to the surface of very moist or saturated soil zone and the existing ground surface elevation at each boring location are presented below:

Boring Nos.	Existing Ground Surface Elevation In Feet	Total Depth of Boring In Feet	Depth To The Surface of Saturated Soil Zone in Feet (**)
1	5,56	30.0	Below 24.0
2	7,68	30.0	Very moist below 23.0
3	8.21	30.0	Below 29.0
4	9.51	50.0	Below 28.5
5	9.99	30.0	*
6	10.60	30,0	*
7	5,18	30.0	Very moist below 26.0
8	8.91	30.0	Very moist below 29.0
9	1.64	30.0	Below 24.0
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Boring Nos.	Existing Ground Surface Elevation In Feet	Total Depth of Boring in Feet	Depth To The Surface of Saturated Soil Zone in Feet (**)	
10	8.95	30.0	Very moist below 28.0	
11	-1.44	30.0	Below 25.0	
12	-0.36	30,0	Very moist to saturated below 24.0 feet	
13	0.32	50.0	Below 21.0	
14	3.04	30.0	Very moist to saturated below 25.5 feet	
15	-1.62	30,0	Below 26.0	
16	0.42	30,0	Below 29.0	
17	-0.48	30.0	Below 28.0	
18	3.22	30.0	Below 28.5	
19'	5.89	30.0	Below 28.0	
20	3,06	30.0	Below 29.8	
21	5.06	12.0	*	

* No saturated soil zone was encountered within the limit of exploration. Boring 21 could not be drilled below 12.0 feet because of the use of the bucket rig in drilling this boring. ** Surface of saturated soil zone may indicate ground water table. Conclusions

It is concluded from the results of field exploration, laboratory tests and office analysis of both field and laboratory data that:

1. During earthquake, soil liquefaction at the site is considered unlikely because of favorable subsurface conditions. The high blow count of a standard sampler, the deep ground water table, the high clay content of the subsoils and the favorable consistency of the soils all tend to negate the possibility of soil liquefaction.

2. On the northerly half of the site where Boring 9 and Borings 11 to 20 inclusive were drilled, expansive, fine-grained soils of lacustrine deposit were encountered in the upper portion of these borings. Based on the results of laboratory tests and the visual identification of the soils, the soils with high expansive potential are clay, sandy clay and silty clay soils. The clayey silt soils are marginally expansive depending upon the amount of silty clay or clay in the clayey silt soils. The silt and fine sandy silt soils were found to be non-expansive to slightly expansive.

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3. On the southerly half of the site where Borings 1 to 8 inclusive, Borings 10 and 21 were drilled, the upper soils were found to be non-expansive. The non-expansive soils consist of silty sand, slightly silty sand and sandy soils of various grain size, silt and fine sandy silt soils. These soils are suitable for use as compacted fill materials.

4. Due to the presence of expansive soils in the upper portion of the northerly half of the site, it is desirable to cap the northerly half of the site with at least 3.0 feet of compacted fill ground that consists of the non-expansive soils obtained from the southerly half of the site.
5. During field exploration, loss of water circulation in drilling through cracked clay layers was observed in some of the borings drilled. The cracks and loss of water circulation may require special attention during construction. The depths where the cracks and the loss of water circulation in the clay layers are summarized and presented below:

Boring Nos	Depth In Feet	Soil Description
9	13.0 to 15.5	Clay with cracks, thin lenses of silt, loss of water circulation during drilling
14	9.5 to 11.0	Clay with shrinkage cracks
14	14.5 to 15.5	Clay with shrinkage cracks
19	19.0 to 20.0	Clay with cracks and fissures, blocky structure, loss of water circulation during drilling
20	4.8 to 7.20	Clay with shrinkage cracks, loss of water circulation during drilling

6. The soils and the ground water at the site were found to be severely corrosive. Any underground structures made of steel and concrete require special preventive measures for the adverse corrosion environment.

Recommendations

1. Soil Density

The tested dry densities of the on-site soils are shown in the eighth column of the attached Drawing Nos. 2 to 44, inclusive of this report. The dry density and the tested field moisture content of the soils, shown in the seventh column of these drawings, can be used as the basis for determining moist, saturated and submerged unit weights of the soils by assuming 2.65

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for the specific gravity of the soils.

2. Ultimate Vertical Bearing Capacity

The recommended bearing capacities of soils for the design of conventional square and continuous footings at the site are presented below:

	Footings On Medium Firm To Very Firm, Undisturbed Natural Soils Of Alluvial Fan Deposit (*) Square Continuous		Footings On Medium Firm To Very Firm, Undisturbed Natural Soils Of Lacustrine Deposit (*) Square Continuous		Footings On Compacted Filled Ground Consisting of Non-Expansive On-Site Soils For Both Square and
	Footing	Footing	Footing	Footing	Continuous Footing
a) Bearing value of two-foot wide square footing and one- foot wide continuous footing at 1.0 foot below final ground surface under field moisture			0.500	0.000	
condition. (psf)	4,000	3,200	3,500	3,000	3,000 under field
b) Rate of allowable increase in bearing value for each additional foot of depth below 1.0 foot (psf/ft)					
Field Moisture Condition	1,100	1,100	210	210	
Submerged Condition	350	350	90	90	
c) Rate of allowable increase in bearing value for each additional foot of width increase (osf/ft)					
Field Moisture Condition	480	600	25	35	
Submerged Condition	140	180	20	25	
d) Recommended maximum					
bearing value (psf)	6,000	6,000	4,200	4,200	3,000 under field
* See Subsurface Conditions"sec	tion and Co	nclusions 2 an	d 3 ⁻		moisture condition
3. Ultimate Moist Skin Friction Condition.	and the Rat	io of Skin Frid	ction under	Moist and Si	ubmerged

For the design of deep foundations which obtain their supporting capacity mainly from

skin friction, the unit skin friction and the corresponding depth may be estimated from the shearing

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resistances of soils presented in the last two columns (columns 9 and 10) of Drawing Nos. 2 to 44, inclusive. The shearing resistances presented in column 9 were obtained by shearing the soil samples at their existing overburden pressures under field moisture condition for those obtained above water table. For those obtained below water table or below saturated soil zones, the soil samples were sheared at their existing effective overburden pressure under saturated-undrained condition. The shearing resistances presented in the last column (column 10) were obtained by saturating selected soil samples above ground water table before test and then shearing the soil samples under a saturated-undrained condition.

The skin friction under field moisture condition may be obtained from the shearing strengths of the soils shown on column 9 of Drawing Nos. 2 to 44 inclusive, divided by 3 for those above ground water table. For example, the shearing resistance of the fine to coarse sand encountered between 19.0 feet and 21.0 feet at Boring 1 was tested to be 3.27 kips per square foot under field moisture condition as shown on Drawing No. 3. Therefore, the estimated skin friction of this zone will be $3.27 \div 3 = 1.09$ kips per square foot. The skin friction under submerged condition may be derived by the shearing strengths of the soils shown on column 10 of Drawing Nos. 2 to 44 inclusive, divided by 2. Where no saturated shearing strengths are shown on column 10, the saturated shearing strengths above ground water table may be derived by the following procedures which were developed on the basis of the average reduction in shearing resistances tested under both field moisture and saturated-undrained condition:

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S9: the shearing resistance of soils above ground water table shown in column 9 of Drawing Nos. 2 to 44 inclusive.

 $S9 \times R = S10$

- R: Average reduction factor for shearing resistance tested under saturatedundrained condition.
- S10: The desired saturated-undrained shearing resistance of soils in column 10 for determining submerged skin friction of the soils above ground water table. If saturated shearing resistance of the soils was tested, the tested value shown on column 10 should be used in lieu of S10.

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The average reduction factors developed on the results of the laboratory tests are

presented below:

	Recommended Average Reduction Factor, R.
The soils of alluvial fan deposit such as silty sand, slightly silty sand, fine to coarse sand and fine sandy silt	0.82
The clay, silt clay, sandy clay and clayey silt soils of lacustrine deposit	0.38
The silt and silty sand of lacustrine deposit	0.67

To obtain the submerged skin friction values of the solls above ground water table,

divide the derived \$10 values by 2.

4. Apparent Cohesion and Angle of Internal Friction

The recommended apparent cohesion and the angle of internal friction values per

laboratory soil tests are presented below:

	Field Moisture Condition		Saturated Condition	
	Apparent Cohesion In psf	Angle of Internal Friction in Degrees	Apparent Cohesion in psf	Angle of Internal Friction In Degrees
Alluvial fan deposit and compacted fills consisting mainly of silty sand, slightly silty sand, fine to coarse sand and fine sandy silt soils	200	33.0	150	28.0
 Lacustrine-deposit-consisting-mainly- of clay, silty clay, sandy clay, clayey silt and silt soils 	820	15.5	330	13.5

5. Coefficient of Subgrade Reaction

The recommended coefficients of subgrade reactions for the soils at the site, on the basis of laboratory test results are presented on the following page.

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	Square Inch per Inch of Deflection			
	Vertical		Horizontal	
	Field Moisture Condition	Submerged Condition	Field Moisture Condition	Submerged Condition
Alluvial fan deposit and compacted non-expansive fill	320.0	160.0	64.0	32.0
Lacustrine deposit	64.0	32.0	13.0	6.5

6. Allowable Maximum Passive Resistance of Solls

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Assuming that the failure of the isolated pier footings will be governed by the soil strength at the time of failure, the recommended maximum possive resistances of soils at various depths below final ground surface and/or mudline, are presented below:

Depth Below Final Ground Surface	Allowable Maximum Passive Resistance of Soils, Kips/Sq.Ft.			
(Feet)	Alluvial Fan Deposit	Lacustrine Soil Deposit		
0	0.37	1.04		
1	0.56	1.13		
5	1.35	1,48		
10	2.33	1 91		
15	3.31	2.34		
20 *	4,29	2.78		
25	4.84	2.89		
30	5.40	3.01		
· 35	5,95	3.12		
40	. 6.51	3.24		

* Assuming ground water or saturated soil zone will be at a depth of 20 feet below final ground surface.

The failure of isolated pier foundation may sometimes be governed by the pier structure itself or the degree of soil structure interaction below ground surface. The degree of soil structure interaction, in turn, will depend on the restrained condition or the allowable deflection condition of the pier foundation at ground surface level.

7. Uplift Resistance of Pier

To estimate the ultimate uplift resistance of an isolated belled pier, the contributory components of the resistance may be assumed to equal the combined weight of the footing plus the inverted cone of a soil prism. The inverted cone of the soil prism is defined by an imaginary plane

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Recommended Subgrade Reaction Pounds Per

projected upwardly and outwardly from the bottom of the footing at an inclined angle with the vertical equal to the tested angle of internal friction presented in Section (4), Page 12. The unit weight of the soils may be computed from the values shown in columns 7 and 8 of Drawing Nos. 2 to 44, inclusive.

The ultimate uplift resistance of a straight cylindrical pier may be assumed to be one-half of the downward supporting capacity of the pier as maximum, computed from the skin friction values described in Section (3), Page 10.

8. Estimated Total Settlement Under Building Foundations, the Transformer and the Oil Circuit Breaker Supporting Pad.

Assuming that the stressed bearing soils will be saturated sometime in the future, the estimated total settlement of shallow building foundations are presented below:

(A) Building Foundations

1. Building foundations on medium firm to very firm undisturbed surface of alluvial fan

deposit:

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Project No. 80-9-10A

Estimated Maxumum Settlement in Inches						
	Conti	invous Footing		S	quare Footing	
	Unit Bearing	Unit Bearing	Unit Bearing	Unit Bearing	Unit Bearing	Unit Bearing
Footing Width	Pressure of	Pressure of	Pressure of	Pressure of	Pressure of	Pressure of
(Feet)	2000 psf	4000 psf	6000 psf	2000 psf	4000 psf	6000 psf
1	0.08	0.09	0.10			
2	0.14	0.15	0.19	0,10	0.12	0.14
3	0.19	0.21	0.25	0.14	0.15	0.19
4	0.23	0.26	0.31	0.17	0.19	0.23
5	0.26	0.29	0.35	0.19	0.22	0.26
6				0.21	0.24	0.29
7				0.23	0.26	0.31
8				0.25	0.28	0.33

2. Building foundation on medium firm to very firm undisturbed surface of lacustrine

deposit such as clayey silt or silt soils:

		Continuous Footing		Square Footing		
	Unit	Unit	Unit	Unit	Unit	Unit
	Foundation	Foundation	Foundation	Foundation	Foundation	Foundation
Footing Width	Pressures of	Pressure of	Pressure of	Pressures of	Pressures of	Pressures of
(Feet)	1500 psf	3000 psf	5000 psf	1500 psf	3000 psf	5000 psf
1	0.11	0.14	0.21	ينبو جده		
2	0.21	0.27	0.41	0.15	0.19	0.30
3	0.30	0.38	0.58	0.21	0.28	0.42
4	0.38	0.49	0.74	0.27	0.36	0.54
5	0.44	0.57	0.88	0.32	0.42	0.64
6				0.37	0.48	0.73
7				0.41	0.53	0.80
8				0.44	0.57	0.87

Estimated Total Settlement in Inches

Where expansive silty clay, clay or sandy clay soils are exposed on the proposed finished grades in the cut area, the upper 3 feet of expansive soils below finished grades should be replaced with a uniformly compacted filled ground consisting of non-expansive or slightly expansive soils such as silty sand, slightly silty sand and fine to coarse sand soils. The anticipated total settlement of footings on a compacted fill ground, uniformly compacted to at least 90 percent of the maximum dry density, is estimated to be less than 1/2 inch for continuous footings less than 4 feet in width and for square footings less than 7 feet in width.

(B) Settlement of Transformer and Oil Circuit Breaker Pad

Assuming that (a) the soils beneath the pad will be saturated sometime in the future, and (b) the pad will be structurally rigid, the estimated total settlements of the pad supporting transformer and oil circuit breaker are presented below:

1. Pad on medium firm to very firm undisturbed alluvial fan deposit or compacted nonexpansive filled ground

Size of Pad	Unit Pressure At	Estimated Total
(Width x Length in feet)	Bottom of Pad, psf	Settlement, Inches
15 x 20 15 x 25	2500 2500	0.39

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2, Pod on medium firm to very firm undisturbed lacustrine deposit:

Size of Pad	Unit Pressure At	EstImated Total
Width x Length in feet	Bottom of Pad, psf	Settlement, Inches
15 x 20	2500	1.9
15 x 25	2000	1.6

For pad of smaller size or pad of less applied unit pressure, the total settlement will

be less than those presented above.

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Project No. 80–9–10A

9. Combined Sections of Asphaltic Concrete Pavement

Using a CBR value of 10.5 tested on the representative non-expansive subgrade soils, the required thicknesses of asphaltic concrete pavement to support various wheel loads are presented below:

	Required Thickness, Inches		
	Parking Area	Driveway Area	
Design Wheel Loads in Pounds	12,000	16,000	
(a) A.C. paving surface, inches	2.0	2.5	
(b) Base course materials with minimum CBR of 80 at 95 percent relative compaction or Caltran's Class 2 aggregate	4.0	6.0	
(c) Clay-free on-site soils or selected non-expansive import soils uniformly compacted to at least 95 percent of maximum dry density, inches	12.0	12.0	

During construction, the proposed paved area should be excavated to a depth equivalent to the combined total of the thicknesses indicated on lines (a), (b) and (c). If expansive clay, silty clay or sandy clay soils are encountered, these should be disposed of off site and be replaced with select non-expansive fill soils for compaction. The exposed surface should then be scarified to a depth of 6 inches, moistened or dried, as necessary to an optimum moisture

BENTON ENGINEERING, INC.

content, and uniformly compacted to at least 90 percent of the maximum dry density. Thereafter, the excavated on-site soils and the base course materials should be placed and uniformly compacted to the required percentages of relative compaction and thicknesses indicated above.

The recommended gradation for the base course materials are as follows:

U.S. Standard Sieve Size	Percentage Pa ssing By Weight
]#	100
3/4"	90 to 100
No. 4	35 to 55
No. 30	10 to 30
No.200	2 to 9

10. Site Groding

It is recommended that the earthwork operations at the site be conducted under a continuous observation by a soils engineer and in accordance with the applicable sections of the attached Appendix AA entitled "Standard Specifications For The Placement Of Compacted Filled Ground."

The maximum dry density of soils to be compacted shall be obtained by the ASTM D1557-70 method of compaction that requires 25 blows of a 10 pound rammer falling from a height of 18 inches on each of 5 layers in a 4-inch diameter 1/30 cubic foot compaction mold.

11. Possible Subsurface Problems Associated with Installation of Drilled and Cast-in-Place Concrete Piers

In drilling Boring 21 using a rotary bucket drill rig, extensive caving of the boring wall occurred. The caving is attributed to the dry nature of the desert soils and is expected to occur during pier construction in the future. The dry desert soils would require pre-drilling stabilization or moistening with water before drilling.

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Respectfully submitted,

BENTON ENGINEERING, INC.

By S.H. Shu, Civil Engineer

RCE No. 19913

Distribution: (5) Addressee

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE boring no. <u>1</u> elevation <u>5,56</u> '	SUMMARY SHEET BORING NO ELEVATION5.56' (*)				SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq. Ft.	
	1-			Light Brown, Dry, Very Loose	SILTY FINE TO MEDIUM SAND						
	2 - 3 - 4 -	িৰিল্ল		Gray Brown, Dry, Loose, With Scattered Fine Gravel	SLIGHTLY SILTY FINE TO MEDIUM SAND	12.8	2.6	112.3	0.43		
	5_	2				15.0	11.4	99.1	0.63		
	6-	10 18 21		Light Brown, Dry, Firm Merges	SILTY FINE SAND		-				
v	 8	8		Olive Gray Brown, Dry, Medium Firm to Firm	FINE SANDY SILT		\$				
<mark>y Substation – San Diego G & Electri</mark>	9 10	<u> </u>		Light Brown, Dry, Very Firm		33. 0	5.4	112.0	1.38	1.23	
	11- 12- 13- 14- 15-	<u>200</u>			SILTY FINE TO MEDIUM SAND	30. 0	4.3	109.1	1.87	1.78	
Na I	-			Continued	I on Drawing No. 3	-		•			
peria		- Indicates Undisturbed Drive Sample									
ji iame Imi				 Indicates Sample Not Recovered indicates the Blowcount of a Indicates the Blowcount of a Indicates the Blowcount of a Is. of Penetration 0" to 6", 6" to 6",	Indicates Sample Not Recovered Indicates the Blowcount of a Standard Splitspoon Sampler for each 6" of Penetration 0" to 6", 6" to 12" and 12" to 18". The Standard Penetration Resistance Is based on adding the number of blows between 6" and 12" to the blows between 12" and 18". The Elevations were provided by San Diego Gas & Electric						
	PROJECT NO. 80-9-10A BENTON ENGINEERING, INC.					DRAWING NO. 2).	

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	DEPTH/FEET	SAMPLE NUMBER	SOIL SOIL SYMBOL	SUMMARY SHI BORING NO. 1 Co	DRIVE ENERGY FT, KIPS/FT,	FIELD MOISTURE % DRY WT,	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq. Ft.	
	10 17- 18	252124		Olive Gray Brown, Dry, Firm, Micaceous, Small Calcareous Lenses and Scattered Lenses of Clay	SILTY FINE TO MEDIUM SAND					
n - San Diego Gas Electric	20-	5		Gray Brown, Dry, Very Dense, With Fine Gravel Merges	FINE TO COARSE SAND	30, 0	16.0	110.0	3.27	
	21-22-23-23-23-23-23-23-23-23-23-23-23-23-	প্রতন্ত্র		Gray Brown, Dry, Dense, With Scattered Coarse Sand Grains and Fine Gravel	SLIGHTLY SILTY FINE TO MEDIUM SAND					
	24- - 25- -	6	Water	Saturated Very Dense		40. 5	13.0	123.6	4.32	
	26- 27 28- 29- 	522		Light Brown, Saturated, Very Dense	SLIGHTLY SILTY FINE SAND	67. 5	17.7	106.7	3.26	
.xc we Imperial Valley Substati										
	project no. 80910A			BENTON ENGINEERING, INC. 3				NG NO.		

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	****	SUMMARY SHE BORING NO. 2 ELEVATION 7.68	DRIVE ENERGY FT, KIPS/FT,	FIELD MOISTURE & DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq. Ft.	
	0 - 1-			Ligh	nt Brown , Dry , Very Loose	SILTY FINE TO MEDIUM SAND					
	2 - - 3 -	() () () () () () () () () () () () () (Gra Wit Sca	y Brown ,Dry , Very Loose , h Sea Shell Fragments and ttered Fine Gravel	SLIGHTLY SILTY FINE TO COARSE SAND	11.3	13.1	98.4	0,32	0.28
	4 5			Lighi Dry / Silty [Slig	Brown and Light Gray Brown, Medium Firm, With Lenses of Fine To Medium Sand and htly Silty Fine to Coarse Sand	SILTY FINE SAND	15.0				
				Ligh Dry of S	nt Brown and Gray Brown, ,Medium Firm,With Lenses Silty Fine Sand						
ectric	8- - 9-	878		Wit	h Lenses of Clayey Silt						
go Gas 71	10- 11-	(3) [5]		Firr	n	FINE TO MEDIUM SAND	19.5	8.6	106.1	1.41	1.22
- San Die	12- 13-	57		Ver	y Firm						
me Imperial Valley Substation	14- 15-	A R		Wi	th Thin Lens of Clayey Silf		45.0				
	16- 17-	23 32 50		Lig	ht Brown, Dry, Very Dense	SLIGHTLY SILTY FINE TO MEDIUM SAND					
	* 18- 19- 20-			Lig	ht Brown, Dry, Very Firm	SILTY FINE TO MEDIUN SAND	34. 5	4.0	5 105.2	2 2.35	1.83
NOP NO	21-		021117		Continued on Draw	ing No. 5	J				
	ряојест NO. 80-9-10А				BENTON ENG	INEERING, INC.			DRA	wing n 4	0.

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHEE BORING NO ELEVATION8.21*	:T 	DRIVE ENERGY FT, KIPS/FT.	FIELD MOISTURE X DRY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
	1 2	0		Gray Brown, Dry, Loose, With Shell Fragments Firm	SILTY FINE TO MEDIUM SAND	18. 8	19.3	98.2	0.36	
	3-4	RNE(R) SEE		Light Brown , Dry , Medium Firm to Firm , Slightly Micaceous Merges	SILTY FINE SAND	15. 8				
lectric	7 - 8 - 9 - -	424		Gray Brown, Dry, Medium Firm, Interbedded with Silty Fine Sand	FINE SANDY SILT	07.8	11 4	103 4	1 42	0.85
n' - San Diego Gas E	10 - - 11 - 12 - 13 - -	्राम्य		Light Brown , Dry , Dense to Very Dense , With Lenses of Fine Sand	slightly silty fine to medium sand					0.00
alley Substatio	14- 15- 16-	(4)		Olive Gray, Dry, Medium Firm, Micaceous, Thinly Bedded	FINE SANDY SILT	13. 5	15.6	82.1	1.78	0.73
Imperial V	17- 18- 19-	NIQ 1		Light Brown , Dry , Very Dense	SLIGHTLY SILTY FINE SAND					
JO AME -	 20 	5	UUDI.	Light Brown , Dry , Very Dense	SLIGHTLY SILTY FINE TO MEDIUM SAND	60. 0	13.0	106.6	2.51	2.16
-	1	proje 30-9-	ct no. 10A	Continued on Drawi BENTON ENGIN	ing No. 7			DRAI 6	NING NO).

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<i>4</i>	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE BORING NO3 CO	ET nt.	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE X DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SO. FT.	Sat. Shear Kips/Sq. Ft.
	20 21 22 22	18 28 33		Light Brown, Dry, Very Firm	SILTY FINE TO MEDIUM SAND					
	23 24 25 26 27 -	<u>କୁଷ</u> ା ଅପାସ		Light Brown, Moist, Very Firm, With Small Calcareous Nodules Olive Gray Brown, Micaceous	SILTY FINE SAND	39.0	7.6	105.7	2.70	2.42
go Gas { lectric	28 - 29 - - - - - - - - - - - - - - - - - - -	0	W of er	Saturated		22.5	20.5	107.3	3.38	
lley Substation – San Die										
Jon time Imperial Val										
	۹ 80	ROJEC -9-1(<u>1</u> 17 NO. DA	BENTON ENGIN	EERING, INC.	0,		draw 7	'ING NO.	

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	S DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SH BORING NO. 4 C	EET ont.	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE X DAY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq. Ft.
	20	5 30 43		Light Brown , Dry Very Dense Gray Brown , Dry , Very Firm	SLIGHTLY SILTY FINE SAND SILTY FINE TO MEDIUM					
	25- 25- 26-	<u>र</u> ह		Gray Brown, Dry, Very Dense, With Fine Gravel	SAND FINE TO COARSE SAND	45.0	940 940 -			
U	27 28	ភា		Gray Brown, Slightly Moist, Very Dense, With Fine Gravel	SLIGHTLY SILTY FINE TO COARSE SAND					
5 Electri	29_ 30	Ø		Red Brown and Light Brown, Very Moist to Saturated, Very Firm, With Lenses of Fine to Medium Sand	SILTY CLAY	45.0	18,2	107.8	3.07	
Substation - San Diego Go	31- 32- 33- 34- 35-	۵ ک ک ک		Light Brown, Very Moist, Very Dense	FINE TO MEDIUM SAND	64. 5	20.9	101.3	3.57	
Imperial Valley :	- 36- 37- 38- - 20	58 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1		Thin Lens of Silty Fine to						
JOB NAME	3 7 - 40- -	9		Medium Sand Continued on Drawir	ng No. 10	75.0	17.2	99,1	3.32	
		 PROJE 809-	ст no. -10А	BENTON ENGI	NEERING, INC.			draw 9	ING NO.	

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	<u>,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,</u>	SUMMARY Soring NO4	SHEE Con	ET <u>t.</u>	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE X DRY WT.	DRY DENSITY LBS JCU. FT.	SHEAR RESISTANCE KIPS/SO. FT.	Sat. Shear Kips. Sq. Ft.
4	0 41 42 43 43 44 45 46 -			Light Dens Silty With	Brown, Very Moist, Ver e, With Lenses of Slight Fine Sand Thin Lenses of Clay	y y	SLIGHTLY SILTY FINE TO MEDIUM SAND	52.5	18.9	102.5	3.47	
ctric	47- 48- 49- 50 -			Red Satu Scat of SI Med	Brown to Light Brown, rated, Very Firm, With tered Thin Lenses lightly Silty Fine to ium Sand		SILTY CLAY	27.0	23.1	104,0	3.62	
Valley Substation - San Diego Gas & I												
JOB NAME Imperial									<u> </u>	DR	AWING	NO.
		ряо. 80	иест NO. 9-10А		BENTON	ENG	INEERING, INC.				10	

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	D DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL		SUMMARY S	HEE (Cor	:T n <u>t</u> •)	DRIVE ENERGY FT, KIPS/FT,	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
	21 22 23 24 25	<u>ଅମ୍</u> ପର ୧୯୦୦		Gray Brown ,	Dry , Very Firm		SILTY FINE TO MEDIUM SAND	39.0	16.7	109.5	4.11	
	26 27 28	<u></u>		Gray Brown, With Lenses Sand, With F	Dry , Very Dense , of Fine to Coarse ine Gravel	/	SLIGHTLY SILTY FINE TO MEDIUM SAND					
r v Electric	20 - 29 - 20 -	T		Light Gray E Moist, Very I of Fine to Co Fine Gravel	Brown, Slightly Dense, With Lense parse Sand, With	es	FINE TO MEDIUM SAND	46.5	14.5	106.3	3,85	
JOB NAME Imperial Valley Jubstation - Jan Liego O												
`		<u>рно</u> л 9 -08	L ect no. 2-10A		BENTON EN	NGH	NEERING, INC.			DRA 1	wing n	0.

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL		SUMMARY SHE boring no. <u>6</u> elevation <u>10,60</u>	ET	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DAY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
1			<u> </u>	Li	ght Brown, Dry, Loose						
	1 2 3)		G Fi	ray Brown, Dry, Loose, With ne to Medium Gravel	FINE TO COARSE SAND	12.0	6.6	99.7	0.47	
				Gi Le Si	irm ray Brown,Dry,Firm,With enses of Fine Sandy Silt and Ity Fine to Medium Sand	SILTY FINE SAND	21.0	1.0	106.1	0.49	
U	8-			Gr Fi	ray Brown,Slightly Moist, m	CLAYEY FINE SAND					
Ga Electri	9- - 10- - 11-			G	ray Brown, Dry, Very Firm		42,0				
<u>bstation - San Diego</u>	- 12- - 13- - 14_ - - 15-	334		Sil	ty Fine Sand	SILTY FINE TO MEDIUM SAND	46.5	1.9	105.7	1.59	
Imperial Valley Su	16_ 17_ 17_ 18_	14 10 12									
ME	19 20	5P B		Gr Fir	ay Brown , Dry , Firm , With ne to Medium Gravel"	SILTY FINE TO COARSE SAND	24.0				
10 ⊳ .(A	-				Continued on Drawi	ng No. 14					
	PROJECT NO. 80-9-10A				BENTON ENGIN	BENTON ENGINEERING, INC.			DRAW 13	ING NO.	

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE BORING NO6 (C	ET ont.)	DRIVE ENERGY FT, KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
	20 21 - 22 - 23 - -	21722		Light Gray Brown, Dry, Firm, With Fine Gravel	SILTY FINE TO COARSE SAND					
	24	<u>کا تا تا</u>		Light Gray Brown, Dry, Very Firm, With Fine Gravel	SLIGHTLY SILTY FINE TO COARSE SAND	45.0	17.4	108.1	3.81	
& tlectric	28 29 	D		Brown , Slightly Moist , Firm , Slightly Micaceous	CLAYEY FINE SAND	56.3	14.8	106.6	3.84	~-
JOB AME Imperial Valley Substation = San Diego G										
		1 рвојі 80-9	<u>ест no.</u> -10А	BENTON ENG	INEERING, INC.			DRA	wing n 14	0 <i>,</i>

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	DEPTH/PEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE	ET nt.)	DRIVE ENERGY FT, KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
	21 - 22 -	25		Light Brown, Dry, Very Firm, With Lenses of Slightly Silty Fine Sand	SILTY FINE SAND					
	23 24 25	6		Red Brown , Moist to Very Moist , Very Firm , With Scattered Lime Cemented Nodules—	CLAY	37.5	20.5	105.1	5.44	** =
ric	26 27 27 28 -	ଅତ୍ୟୁ		Gray Brown,Slightly Moist, Very Dense	SLIGHTLY SILTY FINE SAND					
. Elect	29 - - 30	Ī		Very Moist		71.3	23.6	98.0	3.81	
JOB NAME Imperial Valley Substation – San Diego Go.					· · · · · · · · · · · · · · · · · · ·					
		PROJ 80-1	ест NO. 7-10А	BENTON ENG	INEERING, INC.			DRA 1	wing n 8	0.

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE boring no. <u>10</u> elevation <u>8,95</u> 1	ET	DRIVE ENERGY FT, KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
	1	<u> _</u>		Gray Brown, Dry, Very Loose, With Scattered Fine to Medium Gravel Medium Dense	SLIGHTLY SILTY FINE TO COARSE SAND	10.5	1.4	108.2	0.39	gran bit
	4 , 5-	ান্ধ্রু চ্য		Gray Brown, Dry, Medium Firm	SILTY FINE TO MEDIUM SAND	· 12. 8·				
v	6- - 7- 8- -	<u>S</u> <u>S</u> <u>S</u> <u>S</u> <u>S</u> <u>S</u> <u>S</u> <u>S</u>		Gray Brown, Dry, Medium Dense	SLIGHTLY SILTY FINE TO COARSE SAND					
- San Diego نحمت ما المحلية	9			Brown to Light Brown, Dry, Firm to Very Firm	SILTY FINE TO MEDIUM SAND	34.5	2.8	108.7	1.16	
ey Substation	14 15 16	4		Red Brown, Dry, Very Firm, With Clay Lenses and Lime Cemented Lenses	FINE SANDY SILT	34.5	9.2	89.0	1.27	1 .0 6
Imperial Vall	17- 17- 18-	202140		Light Brown to Light Gray Brown ,Dry , Very Firm ,Inter- bedded with Fine Sandy Silt	SLIGHTLY SILTY FINE SAND					
ME	19- 	5		Light Brown, Dry, Very Firm, Thinly Bedded	FINE SANDY SILT	45.0	6.9	98.1	2.18	
AM BOL				Continued on Draw	ring No. 22		des 700 cg 140 c/190	140-14	Redeviderty: Crimenant	4-41-4-4-4-4
		PROJE 80-9-	ст no. -10А	BENTON ENGI	NEERING, INC.			DRAM 2	ing no	-

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHEI BORING NO. 10 (CO	E T n <u>t</u> .)	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SO, FT.	Sat. Shear Kips/Sq.Ft.
	21-	দিরা		Light Brown, Dry to Slightly Moist, Very Firm, With Lenses of Silty Fine Sand	FINE SANDY SILT					
	22- 23- 23-	1000		Red Brown, Moist, Very Firm	CLAY					
	24	6		Red Brown, Moist, Very Firm	FINE SANDY SILT	33.0	2.1	101.3	2.56	
	26 27			Red Brown, Very Moist, Very Firm, With Lenses of Silty Clay and Clayey Silt Medium Firm	CLAY					
a Electric	28- 29- 	Ø		Light Brown to Brown, Very Moist, Very Firm With Lenses of Silty Fine Sand	FINE SANDY SILT	45.0	25.2	91.1	2.66	
JOB NAME Imperial Valley Substation - San Diego Gas				·						
	-	рво. 80-1	іест NO . 9–10А	BENTON ENG	NEERING, INC.			DR	awing n 22	0.

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL		SUMMARY SHE boring no. <u>11</u> elevation <u>-1,44</u>	ET	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
	-0			Red Firr	Brown and Light Brown , Dry , n to Very Firm , Merges	CLAYEY SILT					
	2			Red Firr Bed	Brown and Light Brown, Dry n to Very Firm, With Thinly Ided Lenses of Silt and Clay	SILTY CLAY	63.8	7.9	116.4	3.64	
	4- - 5_	<u>ق</u>					44, 3	15.8	98.8	1.18	
	- 6- -			Gro Thi	ay Brown, Dry, Very Firm, nly Bedded	SILT					
ctric	7- - 8- - 9-	121423		Bro Mo Clo Me	wn and Red Brown, Slightly ist, Very Firm, With Lenses of ayey Silt irges	SILTY CLAY					
G. & Ele	10-	3		Rec Fin Cic	t Brown,Slightly Moist,Very m,With Scattered Lenses of ayey Silt		22, 5	18.4	106.0	3.00	
n - San Diego		SET SET				CLAY					
rial Valley Substation	14- 15- 16- 17-	<u>ৰ</u>		Lig Dry Wi Sar	ht Brown and Yellow Brown, , Very Firm, Thinly Bedded, th Scattered Lenses of Fine ndy Silt	SILT	24.8	6.4	99.5	0.75	
ame Impe	18- 19- 20-	9		Lig Mo	ht Brown, Dry to Slightly ist, Very Firm, Thin Bedding	SILTY FINE SAND	39.0	6.6	88.4	2.23	
108 M	-			No	te: Used Unconfined Compress Resistance Figure On Sam	wing 190, 24 sion Test Data Figu ple No. 1,2,3 and	re Divid 14.	led By	2 for	Shear	
	I	PROJE 80-9	ст но. 10А		BENTON ENGIN	IEERING, INC.			DRAW 23	ing no. }	

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-20	SAMPLE SAMPLE	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE BORING NO <u>]] (C</u>	ET ont.)	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % ORY WT.	DAY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
2	-		Light Brown, Dry to Slightly Moist, Very Firm, Thin Bedding Merges	SILTY FINE SAND					
22	2-123		Brown and Light Brown, Moist, Firm	FINE SANDY SILT					
23	5		Light Brown, Very Moist to Saturated, Very Firm, With Scattered Lenses of Clayey						
2:	;_]@ 	Water	Silt Saturated	CILITY	40.5	23.8	99.2	2.62	
20			•	FINE SAND		-			
	3-1								
ି <u>କ୍</u> ରୁ ୧୯ ୧୯)- _ @				40.5	23.3	102.9	3.21	
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lley Sub									
erial Va	,		······································			<u> </u>			
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	рво. 2-08	iect no. -104	BENTON ENG	NEERING, INC.	ana ana ang ang ang ang ang ang ang ang		DRAV 24	VING NO	•







DEPTH/FEET SAMPLE NI BARER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE BORING NO13 (C	ET Cont.)	DRIVE ENERGY FT, KIPS/FT,	FIELD MOISTURE % DAY WT.	DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.	
20 21 - - Z	Water 1	Light Brown, Dry, Firm to Very Firm, With Scattered Lime Cemented Nodules	FINE SANDY SILT						
		Brown, Saturated, Medium Firm	CLAY						
23 - 24 - 25 - - - - - - - - - - - - -		Brown and Yellow Gray Brown, Saturated, Firm, Interbedded with Silty Clay, Thinly Bedded	CLAYEY SILT	22.5	29.8	94.0	2.81		
30_7 31_ 31_ 32_13		Light Brown , Saturated , Dense	SLIGHTLY SILTY FINE SAND	-58.5-	• 24.9	100.5	2.98	P (10) (10) (10)	
33-		Firm	FINE SANDY SILT						
34-		Brown, Saturated, Firm	SILTY FINE SAND	28.5	24.0	100.2	3.10		
35- 36- 5- 36- 5- 5- 5- 5- 5- 5- 5- 5- 5- 5		Brown, Saturated, Firm, Inter- bedded with Clayey Silt and <u>Fine Sandy Silt</u> Medium Firm Continued on D	CLAY rawing No. 29	19.5	27.6	96.4	2.49		
PROJE	і <u> </u>	BENTON ENGINEERING, INC.				drawing no. 28			

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	D DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL		SUMMARY SHE BORING NO. 14 ELEVATION 3.04	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE X DRY WT.	DAY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.			
				Gray	Brown , Dry , Loose	GRAVELLY SILTY FINE TO MEDIUM SAND							
	2_	\bigcirc		Red B Frac	rown , Dry , Firm , Brittle and tured	ſ	10, 5	10.4	97.2	1.20			
	3_ - 4-			Red B Thinl	rown and Brown, Dry, Firm, y Bedded, With Clayey Silt	SILT	10.0		60 4	0.00			
U	5	<u>ি</u> গি বি		Red E With Thin	Brown and Brown, Dry, Firm, Shrinkage Cracks, With Lenses of Clayey Silt	. CLAY	.12.8	.11.4.	.99.4	- 2. 09-			
Electri	-9-	21	777	Yelle	ow Gray Brown, Dry, Firm	SILT							
۲ , & I	10-	3		Brow to V Crac	n to Red Brown,Dry,Firm ery Firm,With Shrinkage iks	CLAY	21.0	13.8	100.7	3.09	0.74		
tation - San Diego	11	140128 20128		Brow Bedd	n,Dry,Very Firm,Thinly led,With Lenses of Silt	CLAYEY SILT							
y Subs	- 15-	\bigcirc		Red	Brown . Moist , Very Firm ,	CLAY	18.0	14.0	107.3	1.56			
Imperial Valle	16- 17- 18-	1931 1931 1931 1931 1931 1931 1931 1931		Brov Dry	vn and Yellow Gray Brown, ,Very Firm,Thinly Bedded -	· SILT							
AME	19- 	5			Continued on Dr	awing No. 31	26.3	3 6.9	94.9	2.5	5 1.66		
N BOC	-			Not	e: Used Unconfined Compr Resistance Figure On Sc	nconfined Compression Test Data Figure, Dividence Figure On Sample No. 3,5 and 6.				id By 2 For Shear			
		PRO. 80-	ject no. 9-10A		BENTON ENGINEERING, INC.					DRAWING NO. 30			

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DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE BORING NO. 17 (0	SUMMARY SHEET BORING NO. 17 (Cont.)			DRY DENSITY LASJOU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sa.Ft.
21	2004		Brown, Dry, Very Firm, Thinly Laminated With Fine Sandy Silt and Clayey Silt						
23- 24-			Merges	SILT					
25-	6		Light Brown to Light Gray Brown, Dry to Slightly Moist, Very Firm	SILTY	51.0	12.4	02.7	3.52	9 99 9 -1
20 27 	500		Moist, Firm	FINE SAND					
28- 29- 	$\overline{\mathcal{O}}$	Water	Brown and Light Brown, Saturated, Firm to Very Firm	SILTY CLAY	36.0	24.6	100.5	1.48	111
					· · ·				
 PROJECT NO. 80-9-10A		ст no. -10А	BENTON ENGI	VEERING, INC.			DRAV 3	ving no 7	

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JOB NAME San Diego Gas & Electric - Imperial Valley Sub., Jion

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San Diego Gas & Electric - Imperial Valley Sub. Jtion

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHEET BORING NO. <u>18 (Co</u> nt.)			FIELD MOISTURE X. DRY WT,	DRY DENSITY LBS /CU. FT.	SHEAR RESISTANCE KIPS/BO. FT.	Sat. Shear Kips/Sq.Ft.
	21- 21- 22- -	<u> 목이</u> (1)		Brown, Dry, Very Firm, Interbedded With Silty Clay, Thinly Laminated	CLAYEY SILT					
	23- 24- 25-	6		Brown, Dry, Very Firm, With Lenses Of Fine Sandy Silt	SILTY FINE SAND	52.2	12.4	95.7	2.62	
	26- 27- 28-	<u> হিা</u> ণ্ডব		Brown, Dry, Very Firm, Thinly Laminated With Clayey Silt and Fine Sandy Silt	SILT					
ubs. ion	29- -30	Ī	Water -	Brown, Saturated, Very Firm, Interbedded With Silty Clay and Fine Sandy Silt and Lime Cemented Nodules	CLAY	36.0	25.8	99.6	4.10	***
JOB NAME San Diego Gas & Electric - Imperial Valley S									·	
	,	PROJEC 80-9-	т но. -10А	BENTON ENGIN	IEERING, INC.			DRAW	/ing no . 39	

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DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE BORING NO. 19 (1	SUMMARY SHEET BORING NO. 19 (Cont.)			DRY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
21	N 49		Red Brown and Light Brown, <u>Slightly Moist, Very Firm</u> Moist	CLAY			· · · · · · · · · · · · · · · · · · ·		
23 24- 25- 26- 27-			Light Brown, Moist, Very Firm, Thinly Bedded Very Moist, With Scattered Lenses Of Silty Clay and Clayey Silt	S ILT	60.0	14.0	92.2	2.79	
28- 29- 	T	Water	Brown, Saturated, Very Firm, Interbedded With Silty Clay and Clayey Silt	CLAY	19.5	24.5	100.8	2.39	
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	PROJE 80-9	ст no. -10А	BENTON ENGIN	IEERING, INC.			DRAW 4	/ing no.	

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DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHE BORING NO. 20 ELEVATION 3.06	ET	DRIVE ENERGY FT, KIPS/FT.	FIELD MOISTURE % DRY WT.	DAY DENSITY LBS./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
-0			Gray Brown, Dry, Loose	SLIGHTLY SILTY FINE TO MEDIUM SAND					
2-			Brown, Dry, Firm to Very Firm, With Lenses Of Silty Clay		11.3	10.8	103.3	3.11	
3	14			CLAYEY SILT		÷			
5-	281		Red Brown, Slightly Moist, Firm With Shrinkage Cracks and Thin Lenses Of Silt, Loss of Water		20,3	13.2	98.9	-1.80	1.14
7-			Circulation During Drilling Light Brown, With Scattered	CLAY					
8-	212		Lenses Of Silt Merges	- - 					
10-	3		Brown and Light Gray Brown, Dry, Firm, Thinly Bedded		28.5	10.4	97.3	0.35	
11 - - 12 -	18 25 25								
13-				SILT					
14- 15-	4				26.3	11.5	99.5	2.36	
	6 53		Brown, Dry, Very Firm, With						
17-	3		Lenses OF Clayey Silt	SILTY CLAY					
19-			Brown, Dry, Very Firm, With Lenses Of Fine Sandy Silt	SILTY FINE SAND	30.0	85	93.8	2.02	
20- 			Continue Note: Used Unconfined Compre Resistance Figure On Sa	l ed On Drawing No. ession Test Data Figu mple No. 3.	43 ire, Di	vided	By 2 F	or She	ar
	л рясј -08	ect no. 9-10A	BENTON ENG	INEERING, INC.			DR/	wing n 42	0.

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	DEPTH/FEET	SAMPLE NUMBER	SOIL CLASSIFICATION SYMBOL	SUMMARY SHEL	ET (Cont.)	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE X. DRY WT.	DRY DENSITY LBS /CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	Sat. Shear Kips/Sq.Ft.
	±∙ 21-	11		Brown, Dry, Very Firm, With Lenses Of Fine Sandy Silt	SILTY FINE SAND					
	22 23- 24			Brown, Dry to Slightly Moist, Firm, Interbedded With Thin Lenses Of Silt and Clayey Silt and Scattered Calcareous Nodules	FINE SANDY SILT					
	25- 26- 	ك الالا الالا		Brown to Red Brown, Moist, Firm, With Scattered Lime Cemented Nodules	SILTY CLAY	- 27.0	-21.0-	-93.2-	-2,60-	•
	27 28 29	3		Brown and Gray Brown, Saturated, Firm, Thinly Bedded With Thin Layers of Silt	CLAYEY SILT					
					SILT					
*		рясіје 80-9	ст no. 10А	BENTON ENGIN	EERING, INC.	,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,		DRAY	VING NO).

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BENTON ENGINEERING, INC.

APPLIED SOIL MECHANICS - FOUNDATIONS

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PHILIP HENKING BENTON PRESIDENT - CIVIL ENGINEER

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APPENDIX AA

TELEPHONE (714) 565-1955

STANDARD SPECIFICATIONS FOR PLACEMENT OF COMPACTED FILLED GROUND

1. General Description. The objective is to attain uniformity and adequate internal strength in filled ground by proven engineering procedures and tests so that the proposed structures may be safely supported. The procedures include the clearing and grubbing, removal of existing structures, preparation of land to be filled, filling of the land, the spreading, and compaction of the filled areas to conform with the lines, grades, and slopes as shown on the accepted plans.

The owner shall employ a qualified soils engineer to inspect and test the filled ground as placed to verify the uniformity of compaction of filled ground to the specified 90 percent of maximum dry density. The soils engineer shall advise the owner and grading contractor immediately if any unsatisfactory conditions are observed to exist and shall have the authority to reject the compacted filled ground until such time that corrective measures are taken necessary to comply with the specifications. It shall be the sole responsibility of the grading contractor to achieve the specified degree of compaction.

- 2. Clearing, Grubbing, and Preparing Areas to be Filled.
 - (a) All brush, vegetation and any rubbish shall be removed, piled, and burned or otherwise disposed of so as to leave the areas to be filled free of vegetation and debris. Any soft, swampy or otherwise unsuitable areas shall be corrected by draining or removal, or both.
 - (b) The natural ground which is determined to be satisfactory for the support of the filled ground shall then be plowed or scarified to a depth of at least six inches (6"), and until the surface is free from ruts, hummocks, or other uneven features which would tend to prevent uniform compaction by the equipment to be used.
 - (c) Where fills are made on hillsides or exposed slope areas, greater than 10 percent, horizontal benches shall be cut into firm undisturbed natural ground in order to provide both lateral and vertical stability. This is to provide a horizontal base so that each layer is placed and compacted on a horizontal plane. The initial bench at the toe of the fill shall be at least 10 feet in width on firm undisturbed natural ground at the elevation of the toe stake placed at the natural angle of repose or design slope. The soils engineer shall determine the width and frequency of all succeeding benches which will vary with the soil conditions and the steepness of slope.

- 2 -

- (d) After the natural ground has been prepared, it shall then be brought to the proper moisture content and compacted to not less than ninety percent of maximum density in accordance with A.S.T.M. D-1557-70 method that uses 25 blows of a 10 pound hammer falling from 18 inches on each of 5 layers in a 4" diameter cylindrical mold of a 1/30th cubic foot volume.
- 3. Materials and Special Requirements. The fill soils shall consist of select materials so graded that at least 40 percent of the material passes a No. 4 sieve. This may be obtained from the excavation of banks, borrow pits of any other approved sources and by mixing soils from one or more sources. The material used shall be free from vegetable matter, and other deleterious substances, and shall not contain rocks or lumps of greater than 6 inches in diameter. If excessive vegetation, rocks, or soils with inadequate strength or other unacceptable physical characteristics are encountered, these shall be disposed of in waste areas as shown on the plans or as directed by the soils engineer. If during grading operations, soils not encountered and tested in the preliminary investigation are found, tests on these soils shall be performed to determine their physical characteristics. Any special treatment recommended in the preliminary or subsequent soil reports not covered herein shall become an addendum to these specifications.

The testing and specifications for the compaction of subgrade, subbase, and base materials for roads, streets, highways, or other public property or rights-of-way shall be in accordance with those of the governmental agency having jurisdiction.

4. Placing, Spreading, and Compacting Fill Materials.

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- (a) The suitable fill material shall be placed in layers which, when compacted shall not exceed six inches (6"). Each layer shall be spread evenly and shall be throughly mixed during the spreading to insure uniformity of material and moisture in each layer.
- (b) When the moisture content of the fill material is below that specified by the soils engineer, water shall be added until the moisture content is near optimum as specified by the soils engineer to assure thorough bonding during the compacting process.
- (c) When the moisture content of the fill material is above that specified by the soils engineer, the fill material shall be aerated by blading and scarifying or other satisfactory methods until the moisture content is near optimum as specified by the soils engineer.
- (d) After each layer has been placed, mixed and spread evenly, it shall be thoroughly compacted to not less than ninety percent of maximum density in accordance with A.S.T.M. D-1557-70 modified as described in 2 (d) above. Compaction shall be accomplished with sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other approved types of compaction equipment, such as vibratory equipment that is specially designed for certain soil types. Rollers shall be of such design that they will be able

- 3 -

to compact the fill material to the specified density. Rolling shall be accomplished while the fill material is at the specified moisture content. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to insure that the desired density has been obtained. The entire areas to be filled shall be compacted.

(e) Fill slopes shall be compacted by means of sheepsfoot rollers or other suitable equipment. Compacting operations shall be continued until the slopes are stable but not too dense for planting and until there is no appreciable amount of loose soil on the slopes. Compacting of the slopes shall be accomplished by backrolling the slopes in increments of 3 to 5 feet in elevation gain or by other methods producing satisfactory results.

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- (f) Field density tests shall be taken by the soils engineer for approximately each foot in elevation gain after compaction, but not to exceed two feet in vertical height between tests. Field density tests may be taken at intervals of 6 inches in elevation gain if required by the soils engineer. The location of the tests in plan shall be so spaced to give the best possible coverage and shall be taken no farther apart than 100 feet. Tests shall be taken on corner and terrace lots for each two feet in elevation gain. The soils engineer may take additional tests as considered necessary to check on the uniformity of compaction. Where sheepsfoot rollers are used, the tests shall be taken in the compacted material below the disturbed surface. No additional layers of fill shall be spread until the field density tests indicate that the specified density has been obtained.
- (g) The fill operation shall be continued in six inch (6") compacted layers, as specified above, until the fill has been brought to the finished slopes and grades as shown on the accepted plans.
- 5. <u>Inspection</u>. Sufficient inspection by the soils engineer shall be maintained during the filling and compacting operations so that he can certify that the fill was constructed in accordance with the accepted specifications.
- 6. <u>Seasonal Limits</u>. No fill material shall be placed, spread, or rolled if weather conditions increase the moisture content above permissible limits. When the work is interrupted by rain, fill operations shall not be resumed until field tests by the soils engineer indicate that the moisture content and density of the fill are as previously specified.
- 7. All recommendations presented in the "Conclusions" section of the attached report are a part of these specifications.

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APPENDIX A Unified Soil Classification Chart*

SOIL DESCRIPTION		GROUP SYMBOL	TYPICAL NAMES
I. COARSE GRAIN material is large size.**	NED, More than half of ar than No. 200 sieve	<u></u>	
GRAVELS More than half of	CLEAN GRAVELS	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
coarse fraction is larger than No. 4		GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
sieve size but smaller than 3 inches	GRAVELS WITH FINES (Appreciable amount	GM	Silty gravels, poorly graded gravel- sand-silt mixtures.
	of fines)	GC	Clayey gravels, poorly graded gravel- sand-clay mixtures.
SANDS More than half of	CLEAN SANDS	SW	Well graded sand, gravelly sands, little or no fines.
coarse fraction is smaller than No. 4		SP	Poorly graded sands, gravelly sands, little or no fines.
sieve size	SANDS WITH FINES (Appreciable amount	SM	Silty sands, poorly graded sand-silt mixtures.
	of fines)	SC .	Clayey sands, poorly graded sand-clay mixtures.
II. FINE GRAINED	, More than half of er than No. 200		
sieve size.**	SILTS AND CLAYS	ML	Inorganic silts and very fine sands, rock flour, sandy silt or clayey–silt–sand mixtures with slight plasticity.
	Liquid Limit Less than 50	CL	Inorganic clays of low to medium plas- ticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
	Liquid Limit Greater than 50	СН	Inorganic clays of high plasticity, fat clays.
		ОН	Organic clays of medium to high plasticity

III. HIGHLY ORGANIC SOILS

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Peat and other highly organic soils.

* Adopted by the Corps of Engineers and Bureau of Reclamation in January, 1952.

** All sieve sizes on this chart are U.S. Standard.

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APPENDIX B

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Sampling

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The undisturbed soil samples are obtained by forcing a special sampling tube into the undisturbed soils at the bottom of the boring, at frequent intervals below the ground surface. The sampling tube consists of a steel barrel 3.0 inches outside diameter, with a special cutting tip on one end and a double ball valve on the other, and with a lining of twelve thin brass rings, each one inch long by 2.42 inches inside diameter. The sampler, connected to a twelve inch long waste barrel, is either pushed or driven approximately 18 inches into the soil and a six inch section of the center portion of the sample is taken for laboratory tests, the soil being still confined in the brass rings, after extraction from the sampler tube. The samples are taken to the laboratory in close fitting waterproof containers in order to retain the field moisture until completion of the tests. The driving energy is calculated as the average energy in foot-kips required to force the sampling tube through one foot of soil at the depth at which the sample is obtained.

Shear Tests

The shear tests are run using a direct shear machine of the strain control type in which the rate of deformation is approximately 0.05 inch per minute. The machine is so designed that the tests are made without removing the samples from the brass liner rings in which they are secured. Each sample is sheared under a normal load equivalent to the weight of the soil above the point of sampling. In some instances, samples are sheared under various normal loads in order to obtain the internal angle of friction and cohesion. Where considered necessary, samples are saturated and drained before shearing in order to simulate extreme field moisture conditions.

Consolidation Tests

The apparatus used for the consolidation tests is designed to receive one of the one inch high rings of soil as it comes from the field. Loads are applied in several increments to the upper surface of the test specimen and the resulting deformations are recorded at selected time intervals for each increment. Generally, each increment of load is maintained on the sample until the rate of deformation is equal to or less than 1/10000 inch per hour. Porous stones are placed in contact with the top and bottom of each specimen to permit the ready addition or release of water.

Expansion Tests

One inch high samples confined in the brass rings are permitted to air dry at 105° F for at least 48 hours prior to placing into the expansion apparatus. A unit load of 500 pounds per square foot is then applied to the upper porous stone in contact with the top of each sample. Water is permitted to contact both the top and bottom of each sample through porous stones. Continuous observations are made until downward movement stops. The dial reading is recorded and expansion is recorded until the rate of upward movement is less than 1/10000 inch per hour.

APPENDIX C

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Chemical Analysis Tests by Environmental Engineering Laboratory

ENVIRONMENTAL ENGINEERING LABORATORY



November 10, 1980

Benton Engineering

San Diego, Ca 92123

5540 Ruffin Rd.

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3538 HANCOCK STREET SAN DIEGO, CALIF. 92110 P. O. BOX 81789 SAN DIEGO, CALIF. 92138 PHONE: 298-6131

LABORATORY REPORT

Soil samples from Imperial Valley 80-9-10-A received October 20, 1980

Results are mg/L from 1:1 extract

<u>Hole #</u>	Bag #	Depth	Chloride	Sulfate
1	4	14' to 155	39	148
4	Spt 5	^{لۇ} 17.5'	34	181
6	Spt 6	22.5'	. 37	280
8	Spt 6	22.5'	39	239
11	4	14' to 15'	576	651
12	6	24' to 25'	. 915	836
14	4	14' to 15'	265	235
16	Spt 7	27.5'	688	634
17	5	'19' to 20'	311	416
19	5	19' to 20'	384	466

hurd. Chamber Submitted by,

Robert L. Chambers

Director

BIOLOGICAL AND CHEMICAL CONSULTING SERVICES WATER AND AIR POLLUTION STUDIES WATER TREATMENT CHEMICAL AND MICROBIOLOGICAL LABORATORY SERVICES APPROVED BY CALIFORNIA STATE DEPARTMENT OF PUBLIC HEALTH

Benton, Englineering Skillon, Ed. 37123 Date Collected Skillon, Ed. 3712	Benton Encineexing Sido Rutrith Rd. Date Sido Rutrith Rd. Date Collected Sido Rutrith Rd. Date Collected Rutrith Rd. Date Collected Rutrit Rd. Date Collected Rutrith Rd.	Renton Engineering Date	1		Hole # 2	Bag # 1, Depth: 1.
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Annraved Water Laboratory, California State Department of Public Health

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Annroved Water Laboratory, California State Department of Public Health

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5540 Ruffin Rd. San Diego, Ca 92123 incipal Constituents ations: Calcium Magnesium Mg	Date	November	10, 1980		т этоц	J Dag opt	z . Uel	171: 0.J
San Dí.ego, Ca 92123 incipal Constituents ations: Calcium Magnesium Mg	Date Colle	rtad	والمعالمة المعالية ا	Conductivity	1422	microm	hos/cm	0.25°C
incipal Constituents ations: Calcium Magnesium Mg	Date Rece	ived Octo	ber 20, 1980	pH	8.51			2
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Magnesium Mg			Aluminum	A		Barium	Ba	
			Zinc	Zn		Cadmium	Cđ	
Sodium			Hexavalent Chromíu	ш Сr		Silver	Ag	
Potassium K			Total Chromium	ර්		Mercury	Hg	
Ammonia NH ₄	12		Arsenic	As	-	Gold	Au	
% Sodium			Lead	Pb				
			Copper	Cu				
nions:			Selenium	Se				
Hydroxide	0		Nickel	Ni				
Carbonate CO ₃	48							
Bicarbonate HCO ₃	24		Cyanide	CN				
Sulfate Sold	383		Phenols	•				
Chloride Cl	243		MBAS					
Nitrate NO ₃	1.9.		Grease & Oil					
Fluoride			Sulfides	•	< 2		_	
			Volatile Acids					
Boron B			Suspended Solids					
Silica SiO2			Volatile Suspended	Solids				
Iron Fe			Dissolved Solids				-	
Manganese Mn			Volatile Dissolved S	solids				
Total Phosphate PO4			Settleable Solids					
Ortho Phosphate PO4			BOD, 5 day 20°C					
Nitrite	0		Oxygen Consumed					
Nitrate	0.44		Coliform, MPN/100	Jm (
Ammonia N	9.5		Plate Count/ml					
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5540 Ruffin Ra. Date Collected Conducted San Diego, Ca 92123 Date Received 0Ctober 20, 1980 Pal Constituents mg/l me/l Aluminum Date Received October 20, 1980 pH Pal Constituents mg/l me/l Aluminum Date Received October 20, 1980 pH Date Received Conductor Conductor Calcium Na Magnesium Mg Namonia Ni 7.6 Aluminum Anmonia Ni 7.6 Aluminum Anmonia Ni 7.6 Copper Sodium Ni 7.6 Copper Sodium Nickel Copper Selenium Nickel Nickel Nickel Solida 251 Cyanide Sulfate Solid Sulfate Sulfate Solids Solids Manganese Mi Sulfate Micon Nickel Volatile Suge Sulfate Solids Solids Sulfate Solids Solids Sulfate Solids Solids Sulfate Solids Solids Sulfate Solids <t< th=""><th>tivity 990 8.33 Al mg/l Zn Zn mg/l Cr Cr mg/l As Pb Ni Ni Ni cv cv</th><th>micromhos/cm ©</th></t<>	tivity 990 8.33 Al mg/l Zn Zn mg/l Cr Cr mg/l As Pb Ni Ni Ni cv	micromhos/cm ©
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URS, 2003

Project: SDG&E Imperial Valley Substation		Key to Logs						
Project Location: Imperial County, California		Sheet 1 of 1						
Elevation, feet Type Craphic Log Graphic Log	DESCRIPTION	Water Content, % Dry Density, pcf DtHEL LESLS						
1 2 3 4 5 6	7	8 9 10						
COLUMN DESCRIPTIONS								
 <u>Elevation:</u> Elevation in feet referenced to mean sea level (MSL) or site datum. <u>Depth</u>: Depth in feet below the ground surface. <u>Sample Type</u>: Type of soil sample collected at depth interval shown; sampler symbols are explained below. <u>Sample Number</u>: Sample identification number. Unnumbered sample indicates no sample recovery. <u>Sampling Resistance</u>: Number of blows required to advance driven sampler 12 inches beyond first 6-inch interval, or distance noted, using a 140-lb hammer with a 30-inch drop. <u>Graphic Log</u>: Graphic depiction of subsurface material encountered; typical symbols are explained below. <u>Material Description</u>: Description of material encountered; may include relative density/consistency, moisture, color, particle size; texture, weathering, and strength of formation material. <u>Silty SAND (SM)</u> Sand CLAY 	 8 Water Content: Water of laboratory, expressed as performance of laboratory, expressed as performance of laboratory, in pounds per of laboratory, i	content of soil sample measured in ercentage of dry weight of specimen. ensity of soil sample measured in ubic foot. <u>a:</u> Comments and observations regarding y driller or field personnel. (% KP=nonplastic (% <#200 sieve) sh sieve (% <#200 sieve) npressive strength test (Qu in ksf) g SILT						
TYPICAL SAMPLER GRAPHIC SYMBOLS	OTHER GRAPHIC SYME	<u>IOLS</u>						
Sample collected in a bag or bucket 2.5-inch-OD Modified California sampler	 ✓ First water encounter (ATD) ✓ Water level measured of drilling and sampling 	ed at time of drilling and sampling I at specified time after completion						
	Minor change in mate	erial properties within a stratum						
	— — Inferred or gradationa	l contact between strata						
 —— Inferred or gradational contact between strata GENERAL NOTES 1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests. 2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times. 								

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Report: GEO_10_SNA_KEY; File: 27643870.GPJ; 8/28/2008 keyhsa

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Figure A-1

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Log of Boring B-1

Date(s) Drilled	11/7/02	Logged By	S. Fitzwilliam	Checked By	S. Fitzwilliam
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"	Total Depth of Borehole	31 feet
Drill Rig Type	Mobile B-61	Drilling Contractor	F&C Drilling	Approximate Surface Elevation	
Water Level None encountered		Sampling Method(s)	Bulk/ModCal	Hammer 140#/	'30" drop
Borehole Backfill	Soil cuttings	Location	See Site Plan		



Log of Boring B-1



Log of Boring B-2

-					
Date(s) Drilled	11/7/02	Logged By	S. Fitzwilliam	Checked By	S. Fitzwilliam
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"	Total Depth of Borehole	31.5 feet
Drill Rig Type	Mobile B-61	Drilling Contractor	F&C Drilling	Approximate Surface Elevation	
Water Leve Depth (Fee	el None encountered	Sampling Method(s)	Bulk/ModCal	Hammer 140#/	30" drop
Borehole Backfill	Soil cuttings	Location	See Site Plan		

			5	SAMP	ES				ğ	
	Elevation, feet	Depth, feet	Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, p	REMARKS AND OTHER TESTS
		-0					FILL - Dry to moist, pale light brown, sandy gravel (up to 2")			
		- -					EOLIAN DEPOSITS			
		-					with trace gravel	-		
		-					Becomes light brown medium to fine sand	-		
		4								
		- 6-	N	2-1	33		ALLUVIAL/LACUSTRINE DEPOSITS Very stiff to hard, moist, medium brown to yellow brown, sandy lean CLAY (CL) with – trace silt	-		
		-	V	2-2				-		
		o					Very dense, moist, light brown, fine sandy SILT (ML)	-		
		10	Y	2-3	66		Very hard, moist, medium brown to yellow brown, lean CLAY (CL) with fine sand	-		
		12						-		
		12-					Very dense, moist, light brown, fine sandy SILT (ML), with thin interbeds of silty clay			
B-2		-								
8/2008		-								
GPJ; 8/2		-	X	2-4	55					
7643870.0		-					- · ·			
VA; File: 2		- 18								
GEO_10_SI		-					Very dense, moist, light brown, fine sandy lean CLAY (CL)			
Report:	-	20				[////		1		Figure A-3

Log of Boring B-2



Log of Boring B-3

Date(s) Drilled	11/7/02	Logged By	S. Fitzwilliam	Checked By	S. Fitzwilliam
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"	Total Depth of Borehole	37 feet
Drill Rig Type	Mobile B-61	Drilling Contractor	F&C Drilling	Approximate Surface Elevation	
Water Leve Depth (Fee	el 34 feet	Sampling Method(s)	Bulk/ModCal	Hammer 140#/3	30" drop
Borehole Backfill	Soil cuttings	Location	See Site Plan		



Log of Boring B-3



Our field investigation included the drilling and sampling of eight hollow stem auger test borings (Borings B-1 through B-8) advanced between June 2 and 9, 2008. Three additional mud rotary borings were advanced immediately adjacent to Borings B-3, B-6, and B-7 to facilitate pressuremeter testing. Pacific Drilling of San Diego, California performed the drilling. The borings were supervised and logged by an engineering geologist from our firm. The borings were backfilled soil cuttings and bentonite chips. The approximate boring locations are shown on Figure 2.

The locations of the borings were approximated a handheld Global Positioning System (GPS) unit and the plans provided to us. SDG&E provided ground surface elevation information for each of the boring locations. A Key to Logs is presented as Figure B-1. Final logs of the borings are presented on Figures B-2 through B-9. The descriptions on the boring logs are based on field logs, sample inspections, and results of laboratory tests.

Samples of the subsurface materials were obtained using Standard Penetration Test (SPT) or modified California samplers. Grab samples were also collected of the soil cuttings. Where the modified California sampler was used, the blowcounts on the boring logs have been corrected to indicate SPT N values by multiplying the field blowcount by a factor of 0.8. The samples were sealed to preserve the natural moisture content and returned to our laboratory for examination and testing. The results of laboratory tests are shown at the corresponding sample location on the boring logs and in Appendix D.

Project: SDG&E - Imperial Valley Substation Key to Logs									
Project Location: Imperial Valley, CA Project Number: 27668011.00010		Sheet 1 of 1							
Elevation, feet Type Blows per foot t cog Graphic Log Graphic Log	CRIPTION	Water Content, % Dry Density, pcf	REMARKS AND OTHER TESTS						
1 2 3 4 5 6 7		89	10						
COLUMN DESCRIPTIONS									
 Elevation: Elevation in feet referenced to mean sea level (MSL) or site datum. Depth: Depth in feet below the ground surface. Sample Type: Type of soil sample collected at depth interval shown; sampler symbols are explained below. Sample Number: Sample identification number. Unnumbered sample indicates no sample recovery. Sampling Resistance: Number of blows required to advance driven sampler 12 inches beyond first 6-inch interval, or distance noted, using a 140-lb hammer with a 30-inch drop. Graphic Log: Graphic depiction of subsurface material encountered; typical symbols are explained below. Material Description: Description of material encountered; may include relative density/consistency, moisture, color, particle size; texture, weathering, and strength of formation material. 	Water Content: Water calaboratory, expressed as per claboratory, expressed as per claboratory, in pounds per claboratory in pounds per claboratory in pounds per claboratory in pounds per claboratory com claboratory clabora	ontent of soil sample me rcentage of dry weight o ensity of soil sample mea- ubic foot. <u>a</u> : Comments and obser y driller or field personne n Atterberg limits test), % [LL - PL], %; NP=nonpla %<#200 sieve) th sieve (%<#200 sieve) st Suite upaction Undrained Compression	asured in f specimen. asured in vations regarding l. 6 stic						
TYPICAL MATERIAL GRAPHIC SYMBOLS									
Silty SAND (SM) Poorly graded SAND (SP)	SAND with clay (SP-SC	C) SAND	with silt (SP-SM)						
SILT (ML) High plasticity SILT (MH)	Lean CLAY (CL)	Fat CL	AY (CH)						
Lean to fat CLAY (CL/CH) SILT/CLAY (ML/CL)	Clayey SAND (SC)								
TYPICAL SAMPLER GRAPHIC SYMBOLS	THER GRAPHIC SYMB	<u>OLS</u>							
Bulk sample Modified California sampler	 First water encountere (ATD) 	ed at time of drilling and s	sampling						
Standard Penetration Test sampler	Water level measured of drilling and sampling	at specified time after co g	ompletion						
Ţ.			ratum						
-	mened of gradalional	Someon Derween Sudid							
 <u>GENERAL NOTES</u> Soil classifications are based on the Unified Soil Classification System. Descrip lithologic changes may be gradual. Field descriptions may have been modified 	tions and stratum lines are to reflect results of lab tests	interpretive; actual							
 Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times. 									
Reported blows per foot are SPT-N values. Blowcounts for samples obtained w reduced by a factor of 0.8.	ith the Modified California s	ampler were							

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Figure B-1

Log of Boring B-1

Date(s) Drilled	06/03/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	6.60 feet, MSL
Water Leve Depth (Fee	el None encountered	Sampling Method(s)	SPT/ModCal	Hammer Data 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3621016 E 620286		



Log of Boring B-1



Log of Boring B-2

Date(s) Drilled	06/03/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	5.74 feet, MSL
Water Leve Depth (Fee	el None encountered	Sampling Method(s)	SPT/ModCal	Hammer Data 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3620962 E 620334		



Log of Boring B-2



Log of Boring B-3

Date(s) Drilled	06/03/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	7.12 feet, MSL
Water Leve Depth (Fee	el None encountered	Sampling Method(s)	SPT/ModCal	Hammer Data 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3620849 E 620256		



Log of Boring B-3



Log of Boring B-4

Date(s) Drilled	06/09/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	6.06 feet, MSL
Water Leve Depth (Fee	el None encountered	Sampling Method(s)	SPT/ModCal	Hammer Data 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3620884 E 620336		



Log of Boring B-4



Log of Boring B-5

Date(s) Drilled	06/09/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	7.68 feet, MSL
Water Leve Depth (Fee	el None encountered	Sampling Method(s)	SPT/ModCal	Hammer 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3620669 E 620239		



Log of Boring B-5



Log of Boring B-6

Date(s) Drilled	06/02/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	7.25 feet, MSL
Water Leve Depth (Fee	el None encountered	Sampling Method(s)	SPT/ModCal	Hammer Data 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3620658 E 620279		



Log of Boring B-6



Log of Boring B-7

Date(s) Drilled	06/02/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	9.80 feet, MSL
Water Leve Depth (Fee	el 35	Sampling Method(s)	SPT/ModCal	Hammer Data 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3620525 E 620179		



Log of Boring B-7



Log of Boring B-8

Date(s) Drilled	06/09/08	Logged By	A. Podawiltz	Checked By	K. Giesing
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" ID/7" OD finger bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Unimog Marl M5	Drilling Contractor	Pacific Drilling	Approximate Surface Elevation	8.40 feet, MSL
Water Leve Depth (Fee	el 36.3	Sampling Method(s)	SPT/ModCal	Hammer Data 140 lk	os/30" drop
Borehole Backfill	Soil cuttings and bentonite	Location	N 3620527 E 620275		



Log of Boring B-8



This appendix presents the results of 18 prebored pressuremeter tests conducted adjacent to Boring B-3, Boring B-7, and Boring B-8. Pressuremeter test borings were advanced using rotary wash equipment approximately 7 feet away from the primary hollow stem auger geotechnical borings. When possible, pressuremeter tests were performed at the same depths as the driven geotechnical samples in the adjacent borehole to facilitate correlation of SPT N-values to pressuremeter modulus. Testing was done in general accordance with ASTM Test Standard D4719.

The pressuremeter test is a loading test carried out in situ in a borehole. For prebored pressuremeter tests, an inflatable cylindrical probe attached to drill rods is set at the selected test depth in a borehole predrilled below the bottom of the probe. The pressuremeter then uses a hydraulic control unit to load and monitor the soil response. The data collected defines the stress-strain relationship of the soil. The pressuremeter data is used to determine the limit pressure and the pressuremeter modulus.

The results of this test method are dependent on the degree of disturbance during drilling of the borehole and insertion of the pressuremeter probe. Since disturbance cannot be completely eliminated, the interpretation of the test results should include consideration of drilling conditions. This disturbance is particularly significant in very soft clays and very loose sands.

EQUIPMENT

A TEXAM pressuremeter manufactured by Roctest Ltd. was used to complete all pressuremeter tests. This pressuremeter uses a monocellular, hydraulically inflated probe. A piston displaces a controlled volume of water into a 74-millimeter diameter pressuremeter probe. This piston is housed in the control unit and is advanced by an actuator attached to the control unit. Pressure is measured by pressure gauges on the control unit. Teclan tubing is used to connect the probe to the control unit. Additional equipment information can be found on Roctest's website.

PRESSUREMETER TESTING

The pressuremeter test borings were advanced using a 2-15/16-inch drag bit and rotary wash methods. To achieve a boring of uniform diameter within the prescribed tolerances (between 1.03 and 1.2 times the uninflated probe diameter) and reduce borehole disturbance, pump speeds and rod advancement rates were closely monitored. Typically, the boring was advanced approximately 3.5 feet beyond the desired test depth (center of the probe) to allow space for drill cuttings not evacuated by the drilling mud to settle prior to insertion of the test probe. Following a test, the borehole was advanced to a depth sufficient to conduct the next pressuremeter test.

Calibration of the pressuremeter system for compressibility and probe membrane resistance was done prior to transport to the site. Further calibrations were completed in the field during testing and after completion of the pressuremeter test in Boring B-3 (the first test location). The control box was also routinely checked for accumulated air, which is sometimes generated during testing.

In general, testing was performed with little difficulty, though the drillers had trouble maintaining fluid circulation while drilling pressuremeter Boring B-3. In some instances, the probe was mechanically advanced using static weight or light down pressure using the drill rig hydraulics. This was necessary likely due to relaxation of borehole walls.

The pressuremeter tests were performed following ASTM Test Standard D4719 Procedure B, in which the pressuremeter is inflated in increments of equal volume. Volume increments of 80 cubic centimeters were introduced into the probe and pressure readings were recorded after 30 seconds. In addition, unload-reload cycles were performed at some locations. Readings were recorded manually and entered into a spreadsheet that performs data corrections and calculations.

DATA REDUCTION

The pressuremeter test data was reduced using Pressio Companion V.15 by Roctest Ltd. 2007. The program uses the raw data recorded in the field and applies appropriate corrections. A pressuremeter plot is generated using the corrected data. Typical plots consist of three zones. The first zone is where the probe has not made contact with the borehole wall. This occurs in the initial loading increments and is seen as a relatively small pressure increase with increasing volume. The second zone is the linear portion of the plot. This segment represents the pseudo-elastic behavior of the tested material and is used to compute the pressuremeter modulus. Poisson's ratio is assumed to be 0.33 for all soils in the calculations of this modulus (Briaud 1992). The third portion of the plot will generally show an acceleration of deformation towards the failure point, which is reflected as a decreasing pressure increase for successive increase. This portion of the plot represents plastic deformation of the soil and is used to define the limit pressure.

Table C-1, Summary of Pressuremeter Test Results, presents a summary of the pressuremeter test results. Figures C-1 through C-18 present the data for each test, a plot of the data, and the values of the pressuremeter modulus, limit pressure and yield pressure.

The pressuremeter testing was intended to provide in situ engineering properties of the specific materials and locations tested. Test results should not be construed as representative for the entire site due to variations in subsurface materials. These variations occur both vertically and horizontally. The data and values presented do not reflect any conservatism or factors of safety, which may be appropriate for design.

APPENDIXC

Summary of Pressuremeter Test Results **Imperial Valley Substation Table C-1**

mple	USCS ³	SC	CL	ML	CL	CL	CL	CL	SP-SM	SM	SM	ML	CL	SP-SM	SP	SP	SP	SP-SM	
Nearest Sa	SPT N ²	18	17	35	22	22	15	12	14	36	26	24	14	18	15	21	22	32	
	Sample Depth ¹ (ft)	9	11	16	21	26	31	36	9	11	16	21	31	9	11	16	21	31	
	P∟/PF	1.74	2.50	2.32	2.46	2.47	2.20	1.50	N/A	2.01	2.50	2.25	1.91	2.01	N/A	1.50	1.64	2.55	
	E/P _L	8.83	9.28	7.56	7.38	8.52	9.01	4.78	N/A	4.82	5.18	7.15	8.06	7.15	N/A	6.15	5.37	10.68	
Viold	Pressure Pr (psi)	123	95	100	78	143	LL	211	86	124	123	170	121	108	48	145	172	171	
timato	Pressure P _L (psi)	215	236	233	191	353	170	316	N/A	251	309	382	232	218	N/A	217	282	436	
Reload	Modulus Eri (psi)	4,372	7,950	6,100	5,490	15,850	4,810	6,810	N/A	12,190	N/A	N/A	N/A	11,320	N/A	14,320	17,050	N/A	
Unload	Modulus E _{ul} (psi)	4,843	8,873	6,737	6,642	18,690	5,740	10,260	N/A	14,580	V/N	N/A	V/N	12,880	V/N	16,490	19,870	N/A	
	E _{pmt} (ksi)	1.89	2.19	1.76	1.41	3.01	1.53	1.51	1.45	1.21	1.60	2.73	1.87	1.56	0.92	1.33	1.52	4.65	
	Test ID	3-1	3-2	3-3	3-4	3-5	3-6	3-7	6-1	6-2	6-3	6-4	9-2	L-7	7-2	7-3	7-4	7-5	
	Depth (ft)	4.5	9.5	16.5	21	26	31.5	36	5	11	15	21	31	4.5	11	16	21	31	
	Location	B-3	B-3	B-3	B-3	B-3	B-3	B-3	B-6	B-6	B-6	B-6	B-6	B-7	B-7	B-7	B-7	B-7	Notes:

Reported sample depth to the center of the interval over which the blow count was determined. Modified California blow counts were multiplied by 0.8 to convert to an equivalent SPT N value. Unified Soil Classification System group symbol.

TEXAM Pressuremeter Test

Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Raw Readings

Volume

in³

0.0

4.9

9.8

14.6

19.5

24.4

29.3

34.2

39.1

43.9

48.8

53.7

48.8

53.7

58.6

63.5

68.3

73.2

78.1

83.0

Pressure

psi

0

0

1

1

2

3

6

11 20

36

57

78

21

73

104

127

146

165

181

195

Imperial substation B-3 06/11/2008 3-1 N

Pressure

psi

3

0

0

0

1

1

3

7

16

33

54

75

18

70

101

123

143

161

177

191

Corrected Readings

Volume

in³

0.0

4.9

9.8

14.6

19.5

24.4

29.3

34.1

39.0

43.8

48.6

53.4

48.7

53.4

58.2

63.0

67.8

72.6

77.5

82.3

 $\Delta R/R_0$

%

0.00

2.47

4.88

7.23

9.53

11.79

14.00

16.16

18.28

20.34

22.37

24.37

22.42

24.37 26.32

28.25

30.16

32.04

33.89

35.73

◀

◀

Test Results	
Pressiometric modulus E:	1,893 psi
Unload Modulus	4843 psi
Reload Modulus	4372 psi
Ultimate pressure P _L :	215 psi
Yield pressure P _F :	123 psi
Ratio E / P _L :	8.83
Ratio P _L / P _F :	1.74

Calibration Sheet Reference

Remarks



TEXAM Pressuremeter Test

Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-3 06/11/2008 3-2 N

Raw Re	eadings	Corrected Readings					
Pressure	Volume	Pressure	Volume ∆R/R				
psi	in ³	psi	in ³	%			
0	0.0	5	0.0	0.00			
1	4.9	3	4.9	2.46			
2	9.8	4	9.8	4.87			
4	14.6	5	14.6	7.23			
7	19.5	8	19.5	9.52			
16	24.4	16	24.4	11.77			
36	29.3	35	29.2	13.95			
67	34.2	66	33.9	16.07			
96	39.1	95	38.7	18.16			
120	43.9	119	43.5	20.22			
139	48.8	138	48.3	22.25			
35	43.9	33	43.8	20.35			
129	48.8	128	48.4	22.27			
152	53.7	150	53.2	24.26			
163	58.6	162	58.0	26.24			
172	63.5	171	62.9	28.19			
180	68.3	178	67.7	30.11			
188	73.2	186	72.6	32.01			
193	78.1	191	77.4	33.88			

Test Results	
Pressiometric modulus E:	2,191 psi
Unload Modulus	8873 psi
Reload Modulus	7950 psi
Ultimate pressure P_L :	236 psi
Yield pressure P_F :	95 psi
Ratio E / P _L :	9.28
Ratio P _L / P _F :	2.50

Calibration Sheet Reference

Remarks



TEXAM Pressuremeter Test

Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Raw Readings

Pressure

psi

0

1

3

4

7

15

30

54

78

99

18

91

116

132

143

154

162

171

Volume

in³

0.0

4.9

9.8

14.6

19.5

24.4

29.3

34.2

39.1

43.9

48.8

43.9

48.8

53.7

58.6

63.5

68.3

73.2

78.1

Imperial substation B-3 06/11/2008 3-3 N

Pressure

psi

8

6

6

7

8

9

17

32

56

80

100

20

93

118

134

145

156

164

172

Corrected Readings

Volume

in³

0.0

4.9

9.8

14.6

19.5

24.4

29.2

34.1

38.9

43.7

48.5

43.9

48.5

53.3

58.1

63.0

67.8

72.7

77.5

 $\Delta R/R_0$

%

0.00

2.47

4.87

7.23

9.53

11.78

13.98

16.13

18.22

20.28

22.31

20.37

22.32

24.31 26.28

28.23

30.15

32.04

33.91

◀

◀

Test Results	
Pressiometric modulus E:	1,764 psi
Unload Modulus	6,737 psi
Reload Modulus	6,100 psi
Ultimate pressure P_L :	233 psi
Yield pressure P_F :	100 psi
Ratio E / P_L :	7.56
Ratio P_L / P_F :	2.32

Calibration Sheet Reference

Remarks


Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-3 06/11/2008 3-4 N

Raw Readings		Cor	Corrected Readings	
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$
psi	in³	psi	in³	%
0	0.0	10	0.0	0.00
2	4.9	9	4.9	2.46
3	9.8	10	9.8	4.87
4	14.6	10	14.6	7.22
6	19.5	12	19.5	9.53
14	24.4	19	24.4	11.77
36	29.3	41	29.2	13.95
57	34.2	61	34.0	16.09
74	39.1	78	38.8	18.19
32	36.6	36	36.5	17.20
67	39.1	71	38.8	18.20
88	43.9	92	43.6	20.27
99	48.8	103	48.5	22.31
110	53.7	113	53.3	24.32
117	58.6	120	58.2	26.30
125	63.5	129	63.0	28.25
133	68.3	136	67.9	30.18
139	73.2	142	72.7	32.07
145	78.1	149	77.6	33.94
149	83.0	152	82.5	35.79
153	87.9	157	87.3	37.61
158	92.8	161	92.2	39.41

Use of a slotted casing:	No
rest depth:	21.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000

Test Results					
Pressiometric modulus E:	1,410 psi				
Unload Modulus	6,642 psi				
Reload Modulus	5,490 psi				
Ultimate pressure P_L :	191 psi				
Yield pressure P_F :	78 psi				
Ratio E / P_L :	7.38				
Ratio P_L / P_F :	2.46				

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-3 06/11/2008 3-5 N

Raw Re	eadings	Сог	rected Read	ings
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$
psi	in ³	psi	in ³	%
0	0.0	12	0.0	0.00
1	4.9	11	4.9	2.46
3	9.8	12	9.8	4.87
4	14.6	12	14.6	7.22
7	19.5	14	19.5	9.52
12	24.4	19	24.4	11.77
49	29.3	56	29.1	13.93
95	34.2	101	33.8	16.03
137	39.1	143	38.6	18.10
169	43.9	175	43.3	20.15
65	41.5	71	41.3	19.26
155	43.9	161	43.4	20.17
194	48.8	200	48.1	22.17
215	53.7	221	52.9	24.17
232	58.6	238	57.8	26.14
247	63.5	253	62.6	28.08
262	68.3	268	67.4	30.00
273	73.2	278	72.3	31.89
283	78.1	288	77.1	33.76
292	83.0	297	82.0	35.60
297	87.9	303	86.8	37.42

Use of a slotted casing:	No
Test depth:	26.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000

Test Results					
Pressiometric modulus E:	3,007 psi				
Unload Modulus	18,690 psi				
Reload Modulus	15,850 psi				
Ultimate pressure P_L :	353 psi				
Yield pressure P_F :	143 psi				
Ratio E / P _L :	8.52				
Ratio P _L / P _F :	2.47				

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-3 06/12/2008 3-6 N

Raw Readings		Corrected Readings		ings
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$
psi	in ³	psi	in ³	%
0	0.0	14	0.0	0.00
6	4.9	17	4.9	2.46
-6	9.8	6	9.8	4.89
-3	14.6	8	14.7	7.24
28	19.5	38	19.4	9.49
45	24.4	54	24.3	11.72
68	29.3	77	29.1	13.90
88	34.2	96	33.9	16.04
102	39.1	110	38.7	18.15
113	43.9	121	43.5	20.23
77	41.5	86	41.2	19.24
107	43.9	116	43.6	20.24
119	48.8	128	48.4	22.28
126	53.7	135	53.3	24.30
131	58.6	139	58.1	26.28
134	63.5	143	63.0	28.24
137	68.3	146	67.9	30.17
140	73.2	148	72.7	32.07
142	78.1	150	77.6	33.95
145	83.0	153	82.5	35.79
147	87.9	155	87.4	37.62

Use of a slotted casing:	No
Test depth:	31.50 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000

Test Results	
Pressiometric modulus E:	1,529 psi
Unload Modulus	5,740 psi
Reload Modulus	4,810 psi
Ultimate pressure P _L : Yield pressure P _F :	170 psi 77 psi psi
Ratio E / P _L :	9.01
Ratio P _L / P _F :	2.20

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial sustation B-3 06/12/2008 3-7 N

Raw Re	eadings	Cor	rected Read	ings	
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$	
psi	in ³	psi	in ³	%	
0	0.0	16	0.0	0.00	
4	4.9	18	4.9	2.46	
5	9.8	18	9.7	4.87	
6	14.6	18	14.6	7.22	
8	19.5	20	19.5	9.52	
19	24.4	30	24.3	11.76	
34	29.3	45	29.2	13.96	
53	34.2	63	34.0	16.10	
74	39.1	84	38.8	18.20	
92	43.9	102	43.6	20.27	
30	41.5	41	41.4	19.31	
73	43.9	83	43.7	20.30	
103	48.8	113	48.5	22.32	
127	53.7	137	53.3	24.31	
146	58.6	157	58.1	26.28	
165	63.5	175	62.9	28.21	
184	68.3	194	67.7	30.12	
200	73.2	211	72.6	32.01	-
214	78.1	224	77.4	33.87	
224	83.0	234	82.3	35.71	
239	87.9	248	87.1	37.52	
249	92.8	259	91.9	39.31	

Use of a slotted casing:	No
Test depth:	36.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000

Test Results				
Pressiometric modulus E:	1,511 psi			
Unload Modulus	10,260 psi			
Reload Modulus	6,810 psi			
Ultimate pressure P_L :	316 psi			
Yield pressure P_F :	211 psi			
Ratio E / P_L :	4.78			
Ratio P_L / P_F :	1.50			

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-3 06/12/2008 3-8 N

Use of a slotted casing:	No
Test depth:	46.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000

Raw Readings		Corrected Readings		
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$
psi	in ³	psi	in ³	%
0	0.0	21	0.0	0.00
28	4.9	45	4.8	2.42
12	9.8	29	9.7	4.86
58	14.6	75	14.5	7.14
72	19.5	88	19.3	9.43
101	24.4	116	24.1	11.64
114	29.3	129	28.9	13.84
124	34.2	139	33.8	16.00
134	39.1	149	38.6	18.12
94	36.6	108	36.3	17.11
127	39.1	142	38.6	18.13
138	43.9	152	43.5	20.21
144	48.8	158	48.4	22.26
148	53.7	162	53.2	24.28
151	58.6	165	58.1	26.27
155	63.5	170	63.0	28.23
159	68.3	173	67.8	30.16
162	73.2	177	72.7	32.06
165	78.1	179	77.6	33.93
166	83.0	180	82.5	35.78
170	87.9	184	87.3	37.60
172	92.8	186	92.2	39.41
174	97.6	188	97.1	41.18

Test Results				
Pressiometric modulus E:	1,848 psi			
Unload Modulus	6,310 psi			
Reload Modulus	5,130 psi			
Ultimate pressure P_L :	197 psi			
Yield pressure P_F :	116 psi			
Ratio E / P _L :	9.37			
Ratio P _L / P _F :	1.70			

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-6 06/13/2008 6-1 N

Poisson's coefficient: 0.33 Fluid density: 1.000	Use of a slotted casing: Test depth: Manometer height above ground: Poisson's coefficient: Fluid density:	No 5.00 ft 1.64 ft 0.33 1.000
---	---	---

Raw Readings		Cor	Corrected Readings		
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$	
psi	in ³	psi	in ³	%	
0	0.0	3	0.0	0.00	
1	4.9	3	4.9	2.40	
3	9.8	3	9.8	4.74	
3	14.6	3	14.6	7.04	
4	19.5	3	19.5	9.28	
4	24.4	3	24.4	11.48	
6	29.3	3	29.3	13.64	
6	34.2	3	34.1	15.75	
7	39.1	4	39.0	17.83	
9	43.9	5	43.9	19.87	
11	48.8	7	48.8	21.88	
12	53.7	8	53.7	23.86	
19	58.6	14	58.5	25.79	
27	63.5	23	63.4	27.70	
35	68.3	30	68.2	29.57	
48	73.2	43	73.0	31.41	
62	78.1	57	77.9	33.23	
75	83.0	70	82.7	35.02	
92	87.9	86	87.5	36.78	
104	92.8	98	92.3	38.53	
116	97.6	110	97.2	40.26	

Test Results				
Pressiometric modulus E:	1,451 psi			
Unload Modulus	n.a.			
Reload Modulus	n.a.			
Ultimate pressure P_L :	n.a.			
Yield pressure P_F :	86 psi			
Ratio E / P _L :	n.a.			
Ratio P _L / P _F :	n.a.			

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Raw Readings

Pressure

psi

0

1 3

4

6

8

12

19

29

43

56

70

84

99

113

127

58

117

139

152

162

172

181

Volume

in³

0.0

4.9

9.8

14.6

19.5 24.4

29.3

34.2

39.1

43.9

48.8

53.7

58.6

63.5

68.3

73.2

70.8

73.2

78.1

83.0

87.9

92.8

97.6

Imperial substation B-6 06/13/2008 6-2 Ν

Pressure

psi

5

6

6

7

7

9

12

19

28

42

54

68

82

97

111

124

56

114

136

149

159

169

177

Corrected Readings

Volume

in³

0.0

4.9

9.8

14.6

19.5

24.4

29.2

34.1

38.9

43.8

48.6

53.4

58.2

63.1

67.9

72.7

70.6

72.8

77.5

82.4

87.2

92.1

96.9

 $\Delta R/R_0$

%

0.00

2.40

4.74

7.03

9.28

11.47

13.63

15.73

17.80

19.82

21.81

23.76

25.69

27.58

29.45

31.29

30.47

31.31

33.11

34.90

36.68

38.43

40.16

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Use of a slotted casing: Test depth: Manometer height above ground: Poisson's coefficient: Fluid density:	11.00 ft 1.64 ft 0.33 1.000
---	--------------------------------------

Test Results	\$
Pressiometric modulus E:	1,208 psi
Unload Modulus	14,580 psi
Reload Modulus	12,190 psi
Ultimate pressure P_L :	251 psi
Yield pressure P_F :	124 psi
Ratio E / P _L :	4.82
Ratio P _L / P _F :	2.01

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-6 06/13/2008 6-3 N

Use of a slotted casing:	No
Test depth:	15.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000
Fluid density:	1.000

Raw Readings		Corrected Readings			
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$	
psi	in ³	psi	in ³	%	
0	0.0	7	0.0	0.00	
2	4.9	9	4.9	2.40	
3	9.8	7	9.8	4.74	
3	14.6	7	14.6	7.04	
6	19.5	9	19.5	9.28	
8	24.4	11	24.4	11.47	
23	29.3	25	29.2	13.61	
42	34.2	43	34.0	15.69]◀
64	39.1	65	38.8	17.73	
84	43.9	85	43.6	19.75	
104	48.8	104	48.4	21.72	
123	53.7	123	53.2	23.67	◀
141	58.6	141	58.0	25.59	
155	63.5	155	62.8	27.49	
170	68.3	169	67.7	29.36	
185	73.2	184	72.5	31.20	
197	78.1	196	77.3	33.02	
207	83.0	206	82.1	34.82	
218	87.9	217	87.0	36.59	
228	92.8	227	91.8	38.35	
236	97.6	235	96.7	40.08	

Test Results	
Pressiometric modulus E:	1,598 psi
Unload Modulus	n.a.
Reload Modulus	n.a.
Ultimate pressure P_L :	309 psi
Yield pressure P_F :	123 psi
Ratio E / P _L :	5.18
Ratio P _L / P _F :	2.50

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-6 06/13/2008 6-4 N

Use of a slotted casing:	No
Test depth:	21.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000

Raw Readings		Corrected Readings		
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$
psi	in ³	psi	in ³	%
0	0.0	10	0.0	0.00
1	4.9	10	4.9	2.40
1	9.8	8	9.8	4.74
1	14.6	8	14.6	7.04
2	19.5	8	19.5	9.28
2	24.4	8	24.4	11.48
4	29.3	8	29.3	13.64
12	34.2	16	34.1	15.74
99	39.1	102	38.7	17.67
133	43.9	137	43.4	19.66
167	48.8	170	48.1	21.62
196	53.7	199	52.9	23.55
220	58.6	223	57.7	25.47
239	63.5	242	62.5	27.36
257	68.3	259	67.3	29.22
270	73.2	272	72.1	31.07
281	78.1	283	77.0	32.89
291	83.0	292	81.8	34.69
300	87.9	301	86.6	36.47
308	92.8	309	91.5	38.23
314	97.6	315	96.4	39.97

Test Results	6
Pressiometric modulus E:	2,727 psi
Unload Modulus	n.a.
Reload Modulus	n.a.
Ultimate pressure P _L :	382 psi
Yield pressure P _F :	170 psi
Ratio E / P_L :	7.15
Ratio P_L / P_F :	2.25

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Raw Readings

Pressure

psi

0 3

4

5

6

7

9

13

23

57

84

93

115

127

138

146

154

160

167

172

177

Volume

in³

0.0

4.9

9.8

14.6

19.5

24.4

29.3

34.2

39.1

43.9

48.8

53.7

58.6

63.5

68.3

73.2

78.1

83.0

87.9

92.8

97.6

Imperial substation B-6 06/13/2008 6-5 N

Pressure

psi

14

16

15

16

16

16

18

21

31

64

91

100

121

134

144

152

160

166

173

177

183

Corrected Readings

Volume

in³

0.0

4.9

9.7

14.6

19.5

24.4

29.3

34.1

39.0

43.7

48.5

53.3

58.1

62.9

67.8

72.6

77.5

82.3

87.2

92.1

96.9

 $\Delta R/R_0$

%

0.00

2.40

4.74

7.03

9.28

11.48

13.63

15.74

17.81

19.79

21.76

23.72

25.64

27.54

29.41

31.26

33.09

34.89

36.67

38.43

40.17

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Use of a slotted casing: Test depth: Manometer height above ground: Poisson's coefficient: Fluid density:	31.00 ft 1.64 ft 0.33 1.000
---	--------------------------------------

Test Results	\$
Pressiometric modulus E:	1,873 psi
Unload Modulus	n.a.
Reload Modulus	n.a.
Ultimate pressure P_L :	232 psi
Yield pressure P_F :	121 psi
Ratio E / P _L :	8.06
Ratio P _L / P _F :	1.91

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-7 06/12/2008 7-1 N

Raw Re	eadings	Cor	rected Readi	ngs		
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$		
psi	in ³	psi	in³	%		
0	0.0	3	0.0	0.00		
1	4.9	3	4.9	2.40		
3	9.8	3	9.8	4.74		
4	14.6	4	14.6	7.03		
6	19.5	5	19.5	9.28		
8	24.4	6	24.4	11.47		
13	29.3	10	29.2	13.62		
21	34.2	18	34.1	15.73		
36	39.1	33	38.9	17.78	◀	
53	43.9	49	43.7	19.80		
78	48.8	74	48.5	21.77		
96	53.7	92	53.3	23.72		
113	58.6	108	58.1	25.64	◀	
46	56.1	42	56.0	24.78		
106	58.6	101	58.2	25.65		
126	63.5	121	62.9	27.54		
138	68.3	133	67.8	29.41		
149	73.2	143	72.6	31.26		
154	78.1	149	77.5	33.09		
164	83.0	158	82.3	34.89		
170	87.9	164	87.2	36.67		

Test Results	
Pressiometric modulus E:	1,558 psi
Unload Modulus	12,880 psi
Reload Modulus	11,320 psi
Ultimate pressure P_L :	218 psi
Yield pressure P_F :	108 psi
Ratio E / P_L :	7.15
Ratio P_L / P_F :	2.01

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-7 06/12/2008 7-2 N

Raw Re	eadings	Cor	rected Read	ings
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$
psi	in³	psi	in³	%
0	0.0	5	0.0	0.00
2	4.9	7	4.9	2.40
3	9.8	6	9.8	4.74
4	14.6	6	14.6	7.03
5	19.5	6	19.5	9.28
6	24.4	7	24.4	11.48
6	29.3	6	29.3	13.64
6	34.2	6	34.1	15.75
7	39.1	6	39.0	17.83
8	43.9	6	43.9	19.88
8	48.8	7	48.8	21.89
10	53.7	8	53.7	23.86
12	58.6	10	58.5	25.80
14	63.5	12	63.4	27.72
17	68.3	15	68.3	29.60
21	73.2	18	73.1	31.46
26	78.1	23	78.0	33.28
33	83.0	30	82.9	35.08
41	87.9	38	87.7	36.86
51	92.8	48	92.5	38.61
59	97.6	56	97.4	40.34

Use of a slotted casing:	No
Test depth:	11.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000
Fluid density:	1.000

Test Results	
Pressiometric modulus E:	918 psi
Unload Modulus	n.a.
Reload Modulus	n.a.
Ultimate pressure P_L :	n.a.
Yield pressure P_F :	48 psi
Ratio E / P _L :	n.a.
Ratio P _L / P _F :	n.a.

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Raw Readings

Pressure

psi

0

2

3

4

6

8

9

12

17

26

40

57

72

87

103

117

132

146

73

137

157

171

180

Volume

in³

0.0

4.9

9.8

14.6

19.5 24.4

29.3

34.2

39.1

43.9

48.8

53.7

58.6

63.5

68.3

73.2

78.1

83.0

80.6

83.0

87.9

92.8

97.6

Imperial substation B-7 06/12/2008 7-3 N

Pressure

psi

8

9

9

9

9

12

11

13

18

27

41

58

72

87

103

117

131

145

72

136

156

170

179

Corrected Readings

Volume

in³

0.0

4.9

9.7

14.6

19.5

24.4

29.3

34.1

39.0

43.8

48.7

53.5

58.3

63.1

67.9

72.7

77.6

82.4

80.3

82.4

87.2

92.1

96.9

 $\Delta R/R_0$

%

0.00

2.40

4.74

7.03

9.28

11.47

13.63

15.74

17.82

19.85

21.83

23.78

25.71

27.60

29.47

31.31

33.12

34.91

34.12

34.93

36.69

38.43

40.16

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Use of a slotted casing: Test depth: Manometer height above ground: Poisson's coefficient: Fluid density:

Test Results	
Pressiometric modulus E:	1,334 psi
Unload Modulus	16,490 psi
Reload Modulus	14,320 psi
Ultimate pressure P _L :	217 psi
Yield pressure P _F :	145 psi
Ratio E / P _L :	6.15
Ratio P _L / P _F :	1.50

Calibration Sheet Reference



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Ρ

Imperial substation B-7 06/13/2008 7-4 Ν

Use of a slotted casing:	No
Test depth:	21.00 ft
Manometer height above ground:	1.64 ft
Poisson's coefficient:	0.33
Fluid density:	1.000

Raw Re	eadings	Cor	rected Read	ngs	Test Results			
ressure	Volume	Pressure	Volume	$\Delta R/R_0$				
psi	in ³	psi	in ³	%				
0	0.0	10	0.0	0.00		Pressiometric modulus E:	1,516 psi	
3	4.9	12	4.9	2.40		Unload Modulus	19,870 psi	
4	9.8	11	9.7	4.74		Reload Modulus	17,050 psi	
6	14.6	12	14.6	7.03				
7	19.5	13	19.5	9.27		Ultimate pressure PL:	282 psi	
10	24.4	15	24.4	11.47		Yield pressure P _F :	172 psi	
15	29.3	19	29.2	13.62				
23	34.2	27	34.1	15.72		Ratio E / P _L :	5.37	
34	39.1	38	38.9	17.79		Ratio P _L / P _F :	1.64	
50	43.9	53	43.7	19.80	<			
67	48.8	70	48.5	21.79				
83	53.7	86	53.4	23.74	1	Calibration Sheet Be	foronco	
102	58.6	105	58.2	25.66		Calibration Sheet he	leience	
120	63.5	122	63.0	27.55	1			
139	68.3	141	67.8	29.41				
156	73.2	158	72.6	31.25				
170	78.1	172	77.4	33.06	◀	Bomarke		
82	75.7	84	75.3	32.28		Heilidiks		
160	78.1	161	77.5	33.08	1			
184	83.0	186	82.2	34.86				
199	87.9	200	87.1	36.62				
211	92.8	212	91.9	38.37				
220	97.6	222	96.7	40.10				



Project name:
Borehole name:
Test date: (mm/dd/yyyy)
Test number:
Probe size:

Imperial substation B-7 06/13/2008 7-5 N

Use of a slotted casing:	No
Manometer height above ground:	31.00 ft 1.64 ft
Poisson's coefficient: Fluid density:	0.33 1.000

Raw Re	eadings	Corrected Readings		
Pressure	Volume	Pressure	Volume	$\Delta R/R_0$
psi	in ³	psi	in³	%
0	0.0	14	0.0	0.00
3	4.9	16	4.9	2.40
4	9.8	15	9.7	4.74
5	14.6	16	14.6	7.03
6	19.5	16	19.5	9.28
10	24.4	20	24.4	11.47
20	29.3	29	29.2	13.61
46	34.2	54	34.0	15.68
103	39.1	111	38.6	17.67
163	43.9	171	43.3	19.61
208	48.8	215	48.0	21.55
244	53.7	251	52.7	23.47
273	58.6	280	57.5	25.38
293	63.5	300	62.3	27.27
309	68.3	315	67.1	29.14
324	73.2	330	71.9	30.98
334	78.1	339	76.7	32.81
343	83.0	349	81.6	34.61
352	87.9	358	86.4	36.39

Test I	Results
Pressiometric mod	ulus E: 4,653 psi
Unload Modulus	n.a. psi
Reload Modulus	n.a. psi
Ultimate pressure F	P _L : 436 psi
Yield pressure P _F :	171 psi
Ratio E / P _L :	10.68
Ratio P _L / P _F :	2.55

Calibration Sheet Reference

Remarks

Coarse sand in drill cuttings



The materials observed in the borings were visually classified and evaluated with respect to strength, density and consistency, and moisture content. The strength of the soil was evaluated by considering the density and moisture content of the samples, the penetration resistance of the sampler, and the results of direct shear and triaxial compression tests. A laboratory compaction test was performed on a sample of near surface soil to evaluate the moisture density relationship. Resistivity, sulfate content, pH, and chloride content tests were performed to evaluate the potential corrosivity of the soils. An Expansion Index test was performed to evaluate shrink/swell potential of soil below the proposed mat foundations. Testing was performed in general accordance with applicable ASTM standards.

The results of the grain size analyses, Atterberg Limits, moisture content, and dry density tests are shown with the penetration resistance of the sampler, where applicable, at the corresponding sample location on the logs, Figures B-2 through B-9.

Graphical laboratory test results are presented on the following figures in this appendix:

- Sieve analysis (Figure D-1 through D-8);
- Laboratory compaction (Figure D-9);
- Atterberg limits tests (Figures D-10 and D-11);
- Direct shear tests (Figures D-12 through D-16);
- Unconsolidated undrained triaxial compression tests (Figures D-17 through D-20);
- Corrosion testing (Figures D-21 through D-25);
- Expansion Index testing (Figure D-26); and
- Wash analysis (Figure D-27).



(SNA) sieve only (04/2000)



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(SNA) sieve only (04/2000)





Fig D3 Sieve IVS B06002

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(SNA) sieve only (04/2000)



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Fig D8 Sieve IVS B08035

URS
























Project Number:	27668011	Exploration No.:	B-1
Project Name:	Imperial Valley Substation	Sample No.:	1
Project Engineer:	KG	Depth (ft):	3.0

Initial Visual Classification Symbol: SC

State of Specimen before Processing

X Passing soil through #8 sieve X Moist State Air Dried Oven Dried at 60 C

Set-Up	Minus No. 8	
Water Content	or ()	
Container No.	x31	
Mass Container + Wet Soil (g), M1	168.63	
Mass Container + Dry Soil (g), M2	166.56	
Mass Container (g), M3	134.54	
Water Content, w (%)	6.46	

Resistivity Test: California Test Method 643

Mininum Resistence value: 1,750 ohm-cm

	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Weight of Soil in bowl (g):	689.75	707.98	711.12	713.9	
Weight of mixing bowl (g):	509.16	509.16	509.16	509.16	
Wet weight of Soil (g):	180.59	198.82	201.96	204.74	
Amount of water added (ml):	0	20	5	5	
Soil Box + Wet Soil (g), M5	259.11	267.26	266.76	266.23	
Weight of Soil Box (g), M6	127.18	127.18	127.18	127.18	
Wt. of Wet Soil for test (g), M7	131.93	140.08	139.58	139.05	
Volume of Soil Box (cm ³)	79.2	79.2	79.2	79.2	79.2
Est. Saturation (%)	24.2	38.3	40.3	39.9	
Soil Box Constant (cm)	1.00	1.00	1.00	1.00	1.00
Resistivity Reading (ohm)	3,500	1,900	1,750	1,750	
Resistence (ohm-cm)	3,500	1,900	1,750	1,750	

pH Test : California Test Method 532	pH of slurry:	3.93
50g wet weight of soil mixed with 50 mL of de-ionized water.	Temperature : 2	24.0 Celsius
Sulfate Content: California Test Method 417		
100g of soil mixed with 300 mL of de-ionized water.	SO ₄ (ppm) :	784
mg /kg of SO ₄ = (mg of SO ₄ X 3000) / mL of sample		
recorded mg of SO ₄ in sample = <u>49</u> mg		
above value X = 16 = <u>784</u> mg/ L = ppm		
Chloride Content: California Test Method 422		
100g of soil mixed with 300 mL of de-ionized water.	Cl ⁻ (ppm) :	285
mg/L of Cl ⁻ = ((A-B) x N x 35453) x 3		
$A = mL \text{ of } AgNO_3$ $A = 19$		
B = 23 mL of the blank		
$N = 0.0493 \text{ N}$, normality of the titrant $Cl^{-}(mg/L) = A^{-1}$	* 5 * 3	
Tested By: BG Date:6/11/2008	Che	cked By: TJO
URS		Figure : I

Project Number:	27668011	Exploration No.:	B-1
Project Name:	Imperial Valley Substation	Sample No.:	3
Project Engineer:	KG	Depth (ft):	10.0

Initial Visual Classification Symbol: CL

Resistivity Test: California Test Method 643

State of Specimen before Processing

X Passing soil through #8 sieve X Moist State Air Dried Oven Dried at 60 C

Set-Up	Minus No. 8	
Water Content	or ()	
Container No.	sf4	
Mass Container + Wet Soil (g), M1	181.83	
Mass Container + Dry Soil (g), M2	174.11	
Mass Container (g), M3	130.61	
Water Content, w (%)	17.75	

Mininum Resistence value: 350 ohm-cm

[Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Weight of Soil in bowl (g):	744.42	760.65	775.26		
Weight of mixing bowl (g):	507.34	507.34	507.34		
Wet weight of Soil (g):	237.08	253.31	267.92		
Amount of water added (ml):	0	20	20		
Soil Box + Wet Soil (g), M5	253.66	261.15	256.84		
Weight of Soil Box (g), M6	127.40	127.40	127.40		
Wt. of Wet Soil for test (g), M7	126.26	133.75	129.44		
Volume of Soil Box (cm ³)	79.2	79.2	79.2	79.2	79.2
Est. Saturation (%)	48.4	60.6	61.9		
Soil Box Constant (cm)	1.00	1.00	1.00	1.00	1.00
Resistivity Reading (ohm)	570	350	350		
Resistence (ohm-cm)	570	350	350		

pH Test : California Test Method 532	pH of slurry: 8.19
50g wet weight of soil mixed with 50 mL of de-ionize	d water. Temperature : 24.2 Celsius
Sulfate Content: California Test Method 417 100g of soil mixed with 300 mL of de-ionized wa mg /kg of SO ₄ = (mg of SO ₄ X 3000) / mL of sa	ater. SO₄ (ppm) : <u>368</u> mple
recorded mg of SO ₄ in sample = 23	_mg
above value X = 16 = <u>368</u>	_mg/ L = ppm
Chloride Content: California Test Method 422	
100g of soil mixed with 300 mL of de-ionized water.	Cl⁻ (ppm) : 420
mg/L of Cl ⁻ = ((A-B) x N x 35453) x 3	
A = mL of AgNO ₃ A= 28	
B = 23 mL of the blank	
N = 0.0493 N, normality of the titrant	Cl ⁻ (mg/L) = A * 5 * 3
Tested By: BG Date	<u>6/11/2008</u> Checked By: <u>TJO</u>
URS	Figure : D

Project Number:	27668011	Exploration No.:	B-1
Project Name:	Imperial Valley Substation	Sample No.:	7
Project Engineer:	KG	Depth (ft):	30.0

Initial Visual Classification Symbol: CL

State of Specimen before Processing

X Passing soil through #8 sieve X Moist State Air Dried Oven Dried at 60 C

Set-Up	Minus No. 8	
Water Content	or ()	
Container No.	x18	
Mass Container + Wet Soil (g), M1	139.4	
Mass Container + Dry Soil (g), M2	137.67	
Mass Container (g), M3	127.6	
Water Content, w (%)	17.18	

Resistivity Test: California Test Method 643

Mininum Resistence value: 240 ohm-cm

	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Weight of Soil in bowl (g):	631.35	640.08	646.6	650.57	
Weight of mixing bowl (g):	510.66	510.66	510.66	510.66	
Wet weight of Soil (g):	120.69	129.42	135.94	139.91	
Amount of water added (ml):	0	10	10	10	
Soil Box + Wet Soil (g), M5	252.04	258.27	262.09	265.26	
Weight of Soil Box (g), M6	130.93	130.93	130.93	130.93	
Wt. of Wet Soil for test (g), M7	121.11	127.34	131.16	134.33	
Volume of Soil Box (cm ³)	79.2	79.2	79.2	79.2	79.2
Est. Saturation (%)	43.5	51.6	58.2	61.1	
Soil Box Constant (cm)	1.00	1.00	1.00	1.00	1.00
Resistivity Reading (ohm)	470	260	240	240	
Resistence (ohm-cm)	470	260	240	240	

pH Test : California Test Method 532	pH of slurry:	8.05	
50g wet weight of soil mixed with 50 mL of de-ionized water.	Temperature :	24.4	Celsius
Sulfate Content: California Test Method 417			
100g of soil mixed with 300 mL of de-ionized water.	SO₄ (ppm) :	544	
mg /kg of SO ₄ = (mg of SO ₄ X 3000) / mL of sample			
recorded mg of SO ₄ in sample = <u>34</u> mg			
above value X = 16 = <u>544</u> mg/ L = ppm			
Chloride Content: California Test Method 422			
100g of soil mixed with 300 mL of de-ionized water.	Cl ⁻ (ppm) :	765	
mg/L of Cl ⁻ = ((A-B) x N x 35453) x 3			
$A = mL \text{ of } AgNO_3$ $A = 51$			
B = 23 mL of the blank			
N = 0.0493 N, normality of the titrant $Cl^{-}(mg/L) = A^{*}$	5*3		
Tested By: BG Date: 6/11/2008		Checked By:	TJO
URS			Figure : [

Project Number:	27668011	Exploration No.:	B-6
Project Name:	Imperial Valley Substation	Sample No.:	3
Project Engineer:	KG	Depth (ft):	10.0

Initial Visual Classification Symbol: SM

State of Specimen before Processing

Resistivity Test: California Test Method 643

X Passing soil through #8 sieve X Moist State Air Dried Oven Dried at 60 C

Set-Up	Minus No. 8	
Water Content	or ()	
Container No.	sna1	
Mass Container + Wet Soil (g), M1	194.03	
Mass Container + Dry Soil (g), M2	191.61	
Mass Container (g), M3	145.65	
Water Content, w (%)	5.27	

Mininum Resistence value: 2,500 ohm-cm

-					
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Weight of Soil in bowl (g):	772.06	789.65	805.49	813.98	
Weight of mixing bowl (g):	510.9	510.9	510.9	510.9	
Wet weight of Soil (g):	261.16	278.75	294.59	303.08	
Amount of water added (ml):	0	20	20	10	
Soil Box + Wet Soil (g), M5	256.83	272.97	282.20	273.71	
Weight of Soil Box (g), M6	131.69	131.69	131.69	131.69	
Wt. of Wet Soil for test (g), M7	125.14	141.28	150.51	142.02	
Volume of Soil Box (cm ³)	79.2	79.2	79.2	79.2	79.2
Est. Saturation (%)	17.9	33.9	50.4	38.9	
Soil Box Constant (cm)	1.00	1.00	1.00	1.00	1.00
Resistivity Reading (ohm)	8,100	3,500	2,500	2,600	
Resistence (ohm-cm)	8,100	3,500	2,500	2,600	

pH Test : California Test Method 532	pH of slurry:	8.47	
50g wet weight of soil mixed with 50 mL of de-ionized water.	Temperature :	23.9	Celsius
Sulfate Content: California Test Method 417			
100g of soil mixed with 300 mL of de-ionized water.	SO ₄ (ppm) :	736	
mg /kg of SO ₄ = (mg of SO ₄ X 3000) / mL of sample			
recorded mg of SO ₄ in sample = <u>46</u> mg			
above value X = 16 = <u>736</u> mg/ L = ppm			
Chloride Content: California Test Method 422			
100g of soil mixed with 300 mL of de-ionized water.	Cl ⁻ (ppm) :	240	
mg/L of Cl ⁻ = ((A-B) x N x 35453) x 3			
$A = mL \text{ of } AgNO_3$ $A = 16$			
B = 23 mL of the blank			
$N = 0.0493 \text{ N}$, normality of the titrant $Cl^{-}(mg/L) = A^{+}$	* 5 * 3		
Tested By: BG Date:6/11/2008		Checked By:	TJO
URS			Figure : [

Project Number:	27668011	Exploration No.:	B-7
Project Name:	Imperial Valley Substation	Sample No.:	1
Project Engineer:	KG	Depth (ft):	1.0

Initial Visual Classification Symbol: SM

State of Specimen before Processing

Resistivity Test: California Test Method 643

X Passing soil through #8 sieve X Moist State Air Dried Oven Dried at 60 C

Set-Up	Minus No. 8	
Water Content	or ()	
Container No.	x15	
Mass Container + Wet Soil (g), M1	178.1	
Mass Container + Dry Soil (g), M2	177.21	
Mass Container (g), M3	149.31	
Water Content, w (%)	3.19	

Mininum Resistence value: 5,000 ohm-cm

_					
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Weight of Soil in bowl (g):	224.56	228.86	232.76	236.51	245
Weight of mixing bowl (g):	104.67	104.67	104.67	104.67	104.67
Wet weight of Soil (g):	119.89	124.19	128.09	131.84	140.33
Amount of water added (ml):	0	5	5	5	10
Soil Box + Wet Soil (g), M5	241.03	246.34	250.09	257.94	263.22
Weight of Soil Box (g), M6	127.19	127.19	127.19	127.19	127.19
Wt. of Wet Soil for test (g), M7	113.84	119.15	122.9	130.75	136.03
Volume of Soil Box (cm ³)	79.2	79.2	79.2	79.2	79.2
Est. Saturation (%)	9.2	16.5	23.9	27.3	36.4
Soil Box Constant (cm)	1.00	1.00	1.00	1.00	1.00
Resistivity Reading (ohm)	14,000	9,700	8,200	6,000	5,000
Resistence (ohm-cm)	14,000	9,700	8,200	6,000	5,000

pH Test : California Test Method 532	pH of slurry:	7.01
50g wet weight of soil mixed with 50 mL of de-ionized water.	Temperature :	23.7 Celsius
Sulfate Content: California Test Method 417		
100g of soil mixed with 300 mL of de-ionized water.	SO₄ (ppm) :	816
mg /kg of SO ₄ = (mg of SO ₄ X 3000) / mL of sample		
recorded mg of SO ₄ in sample = <u>51</u> mg		
above value X = 16 = <u>816</u> mg/ L = ppm	I	
Chloride Content: California Test Method 422		
100g of soil mixed with 300 mL of de-ionized water.	Cl ⁻ (ppm) :	195
mg/L of Cl ⁻ = ((A-B) x N x 35453) x 3		
$A = mL \text{ of } AgNO_3$ $A = 13$		
B = 23 mL of the blank		
N = 0.0493 N, normality of the titrant $Cl^{-}(mg/L) = A$	A * 5 * 3	
Tested By: BG Date: 6/11/200	<u>08</u> C	hecked By: TJO
URS		Figure : D

Expansion Index (ASTM D4829)

G Force Lab No.	6451		
Date Sampled:	09/17/08	By:	Client
Date Submitted:	09/17/08	By:	Client
Sample Location:	Bank 82, Pad 2		
Sample Description:	Light brown silty clay		

Potential Expansion	Very Low
Expansion Index	9
Final Water Content, %	16.2
Saturation, %	47.5
Dry Density, pcf	118.6
Initial Water Content, %	7.4

Reviewed by: John Inlow, Lab Manager



8788 Balboa Avenue 🔶 San Diego, CA 92123 🔌 619-583-6633 🍎 Fax 619-583-6654 3536 Concours Avenue, Suite 110 🔶 Ontario, CA 91764 🍨 909-481-6833 🇳 Fax 909-481-4642 www.gforceca.com

Material Finer Than #200 Sieve

ASTM D1140

G Force Lab No.	6451		
Date Sampled:	9/17/2008	By:	Client
Date Submitted:	9/17/2008	By:	Client
Sample Location:	Bank 82, Pad 2		
Sample Description:	Silty clay		

Test Results

Percent finer than #200 Sieve = 61

Reviewed by: John Inlow, Lab Manager



8788 Balboa Avenue 🔸 San Diego, CA 92123 🔸 619-583-6633 🔌 Fax 619-583-6654 3536 Concours Avenue, Suite 110 🔸 Ontario, CA 91764 🔸 909-481-6833 🔺 Fax 909-481-4642 www.gforceca.com