APPENDIX 10G

Geologic and Foundation Design Criteria

APPENDIX 10G

Geologic and Foundation Design Criteria

10G.1 Introduction

This appendix includes the results of a preliminary geotechnical assessment for the project to support the Application for Certification (AFC). A letter report, prepared by Condor Earth Technologies, Inc. specific to this project, and a Geotechnical Engineering Study, previously prepared by Condor for Turlock Irrigation District's (TID's) Walnut Energy Center (WEC), are included as Attachments 10G-1 and 10G-2 to this appendix.

This appendix contains a description of the site conditions and preliminary foundation-related subsurface conditions. Soil-related hazards addressed include soil liquefaction, hydrocompaction (or collapsible soils), and expansive soils. Preliminary foundation and earthwork considerations are addressed based on the results of general published information available for the project area and collected for the AFC, and established geotechnical engineering practices.

Information contained in this appendix reflects the codes, standards, criteria, and practices that will be used in the design and construction of site and foundation engineering systems for the facility. More specific project information will be developed during execution of the project to support detailed design, engineering, material procurement specification and construction specifications. This information will be included in a geotechnical engineering study.

10G.2 Scope of Work

The scope of geotechnical services for the preparation of this appendix included an assessment of soils-related hazards, a summary of preliminary foundation and earthwork considerations, and preliminary guidelines for inspection and monitoring of geotechnical aspects of construction based on available published data as analyzed in Subsection 8.15 of this AFC.

10G.3 Site Conditions

The project site is located on a portion of a 69-acre parcel located on the southeast corner of the intersection of South Washington Road and the Union Pacific railroad tracks, south of West Main in Turlock, California. The site topography of the 69-acre parcel is relatively flat with a very gradual slope from the northeast corner to the southwest corner. Elevations range from 82 feet above sea level near the southwest corner of the property to 85 feet above sea level near the northeast corner of the property. The property has been "laser leveled" for irrigation purposes. The site currently drains towards the southwest corner of the property. The field is not located within the 100-year flood plane. Stormwater falling on the site percolates into the porous soil with no offsite runoff. The normal groundwater elevation is expected to be about 7 to 9 feet below ground surface, however, during recent heavy rains in 1997/1998, the groundwater documented reached as high as 1 to 4 feet below ground surface. The 69-acre parcel is currently used for agricultural purposes.

10G.4 Site Subsurface Conditions

10G.4.1 Stratigraphy

Generalized stratigraphy is discussed in Subsection 8.15. Borings will be performed at the project site to verify the soil consistency and characteristics.

10G.4.2 Seismicity/Ground-Shaking

The project site is subject to the probability of seismic activities. No known faults traverse through the local soils in or near the site, and the site is not located in an Alquist-Priolo Earthquake Fault Zone as defined by Special Publication 42 (revised 1997) published by the California Division of Mines and Geology (CDMG). The nearest fault system is located approximately 22 miles southwest of the project site. The San Andreas Fault is approximately 48 miles to the southwest. Per the Uniform Building Code (UBC), the site is located in seismic zone 3.

The project site is susceptible to ground-shaking during major earthquakes from the San Andreas Fault. The seismic risk to structures depends upon the distance to the epicenter; the characteristics of the earthquake, the geologic, groundwater, and soil conditions underlying the structures and their vicinity. Due to the site distance from the above faults and the subsurface conditions, maximum ground acceleration is expected to be on the order of about 0.17g (acceleration due to gravity).

10G.4.3 Ground Rupture

Ruptures along the surface trace of a fault tend to occur along lines of previous faulting. There is no evidence of potentially active fault trace at the nearby site; and thus the primary hazard of surface rupture at the project site is expected to be negligible. However, a ground rupture study at the project site will be performed as part of the geotechnical investigation in order to verify this assumption.

10G.4.4 Liquefaction Potential

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing cyclic shear stresses associated with earthquakes. Based on the anticipated relative density of the cohesionless sediments near the project site, it is expected that the potential for liquefaction is low. Additionally, any significant damage due to liquefaction potential can be mitigated through ground improvement techniques or through the use of piles. The geotechnical investigation will determine the extent, if any, of mitigation required.

10G.4.5 Groundwater

Groundwater is expected to occur at approximately 7 to 9 feet below ground surface. The groundwater elevation will be confirmed during the geotechnical investigation.

10G.5 Assessment of Soil-Related Hazards

10G.5.1 Liquefaction

Soil liquefaction is a process by which loose, saturated, granular deposits lose a significant portion of their shear strength due to pore water pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Soil liquefaction can lead to foundation bearing failures and excessive settlements when:

- The design ground acceleration is high (up to 0.4g)
- The water level is relatively shallow
- Low standard penetration tests (SPT) blow counts are measured in granular deposits (suggesting low soil density)

The results of the subsurface investigation at the nearby site indicate some soils with a potential for liquefaction. However, this must be verified by the subsurface investigation for this project site.

10G.5.2 Expansive Soils

Soil expansion is a phenomenon by which clayey soils expand in volume as a result of an increase in moisture content, and shrink in volume upon drying. Expansive soils are usually identified with index tests, such as percentage of clay particles and liquid limit. It is generally accepted that soils with liquid limits larger than about 50 percent, i.e., soils that classify as high plasticity clays (CH) or high plasticity silts (MH), may be susceptible to volume change when subjected to moisture variations.

Laboratory test results for representative soil samples at the top 10 feet below grade will be tested to determine overall soil expansiveness. The soils near the project site at WEC are generally not clayey and indicate no soils with a potential for expansion. A soil investigation will be performed at the project site.

10G.5.3 Collapsible Soils

Soil collapse (hydrocompaction) is a phenomenon that results in relatively rapid settlement of soil deposits due to addition of water. This generally occurs in soils having a loose particle structure cemented together with soluble minerals or with small quantities of clay. Water infiltration into such soils can break down the interparticle cementation, resulting in collapse of the soil structure. Collapsible soils are usually identified with index tests, such as dry density and liquid limit, and consolidation tests where soil collapse potential is measured after inundation under load.

Based on the available data, the potential for soil collapse at the site is expected to be remote. However, this will be confirmed by testing soil samples retrieved from borings at the project site.

10G.6 Preliminary Foundation Considerations

10G.6.1 General Foundation Design Criteria

For satisfactory performance, the foundation of any structure must satisfy two independent design criteria. First, it must have an acceptable factor of safety against bearing failure in the foundation soils under maximum design load. Second, settlements during the life of the structure must not be of a magnitude that will cause structural damage, endanger piping connections or impair the operational efficiency of the facility. Selection of the foundation type to satisfy these criteria depends on the nature and magnitude of dead and live loads, the base area of the structure and the settlement tolerances. Where more than one foundation type satisfies these criteria, then cost, scheduling, material availability and local practice will probably influence or determine the final selection of the type of foundation.

An evaluation of the information collected for the AFC indicates that no adverse foundation-related subsurface and groundwater conditions would be encountered that would preclude the construction and operation of the proposed structures. The site can be considered suitable for development of the proposed structures, pursuant to completion of a geotechnical investigation, and the preliminary foundation and earthwork considerations discussed in this appendix.

10G.6.2 Shallow Foundations

Completion of the geotechnical investigation will determine if the proposed structures can be supported directly on the native soils. Shallow foundation construction will require the earthwork measures discussed in Subsection 10G.7.

Allowable bearing pressures will include a safety factor of at least 3 against bearing failures. Settlements of footings are expected to be limited to 1 inch, and differential settlement between neighboring foundations to less than 1/2 inch. Tanks can usually undergo much larger settlements.

Frost depth is likely to be less than 5 inches at the site, but will be confirmed through a geotechnical investigation. Pursuant to a geotechnical investigation, exterior foundations and foundations in unheated areas should be placed at a depth of at least 1 foot below the ground surface for protection. Interior footings in permanently heated areas can be placed at nominal depths. The minimum recommended width is 3 feet for spread footings and 2 feet for wall footings.

10G.6.3 Deep Foundations

Compressible soils are not expected based on information analyzed for the AFC. However, if compressible soils are present at the project site, which would preclude use of shallow foundations mentioned above, piles will be needed. A typical pile could be a 12-inch or 14-inch square precast-prestressed concrete pile based on geotechnical investigation. These types of piles are expected to develop allowable loads of 60 to 80 tons in compression, 20 tons in uplift, and 4 tons laterally. The length, size, allowable bearing, uplift, and lateral capacity of the piles for the project site, if needed, will be determined using available software programs.

10G.6.4 Corrosion Potential and Ground Aggressiveness

Corrosivity tests will be conducted to determine whether the site soils to be noncorrosive or corrosive for buried steel based on the chloride content and pH values.

10G.7 Preliminary Earthwork Considerations

10G.7.1 Site Preparation and Grading

There are no trees, structures, or debris to be removed at the project site. The subgrade preparation would include the complete removal of all vegetation (agricultural crop) and topsoil. The majority of the vegetation on the site consists of corn stalks with a maximum root depth of less than a foot. Topsoil can be stockpiled and may be reused in remote areas of the site where no future construction is expected.

As discussed in Subsections 8.9, 8.14, and 8.16 and shown on the Proposed Drainage Facilities drawing (Figure 8.14-4), site grading will include fill to bring the site to a level grade. The site fill work should be performed as detailed below. All soil surfaces to receive fill should be proof-rolled with a heavy vibratory roller or a fully-loaded dump truck to detect soft areas.

10G.7.2 Temporary Excavations

It is anticipated that confined temporary excavations at the site will be required during construction for the installation of the circulation water pipes and the cooling tower forebay. All excavations should be sloped in accordance with Occupational Safety and Health Act (OSHA) requirements. Sheet piling could also be used to support any excavation. The need for internal supports in the excavation will be determined based on the final depth of the excavation. Any excavation below the water table should be dewatered using well points or other suitable system installed prior to the start of excavation. Since the water table is approximately 7 to 9 feet below the surface, the need for dewatering is expected for deep excavations.

10G.7.3 Permanent Slopes

Cut and fill slopes shall be 2h:1v (horizontal to vertical) maximum. Embankments for creek diversions, if required, shall be 5h:1v maximum.

10G.7.4 Backfill Requirements

All fill material will be free of organic matter, debris, or clay balls, with a maximum size not exceeding 3 inches. Structural fill will also have a Plastic Index of less than 20, a Liquid Limit of less than 40, and a maximum fine content (passing the 200 sieve) of 40 percent. Granular, uniformly graded material with a maximum aggregate size of 0.5 inch may be used for pipe bedding. Based on the available site grading, it is anticipated that fill material will be available onsite.

Structural fill will be compacted to at least 95 percent of the maximum dry density as determined by American Society for Testing and Materials (ASTM) D 1557 when used for raising the grade throughout the site, below footings or mats, or for rough grading. Fill placed behind retaining structures may be compacted to 90 percent of the maximum dry density as determined by ASTM D 1557. Initially, structural fill will be placed in lifts not exceeding 8-inches loose thickness. Thicker lifts may be used pursuant to approval based on results of field compaction performance. The moisture content of all compacted fill will fall within 3 percentage points of the optimum moisture content measured by ASTM D 1557, except the top 12 inches of subgrade will be compacted to 95 percent of ASTM D 1557 maximum density.

Pipe bedding can be compacted in 12-inch lifts to 90 percent of the maximum dry density as determined by ASTM D 1557. Common fill to be placed in remote and/or unsurfaced areas may be compacted in 12-inch lifts to 85 percent of the maximum dry density as determined by ASTM D 1557.

10G.8 Inspection and Monitoring

A California-registered Geotechnical Engineer or Engineering Geologist will monitor geotechnical aspects of foundation construction and/or installation and fill placement. At a minimum the Geotechnical Engineer/Engineering Geologist will monitor the following activities:

- Surfaces to receive fill will be inspected prior to fill placement to verify that no pockets of loose/soft or otherwise unsuitable material were left in place and that the subgrade is suitable for structural fill placement.
- Fill placement operations will be monitored by an independent testing agency. Field compaction control testing will be performed regularly and in accordance with the applicable specification to be issued by the Geotechnical Engineer.
- The Geotechnical Engineer will witness pile load testing or pile driving analysis, and the initial stages of production pile installation.
- Settlement monitoring of significant foundations and equipment is recommended on at least a quarterly basis during construction and the first year of operation, and then semi-annually for the next 2 years.

10G.9 Site Design Criteria

10G.9.1 General

The project will be located in the City of Turlock, California. The approximate 69-acre site is relatively flat, with no existing permanent type of structures. The site would be accessible from South Washington Road.

10G.9.2 Datum

The site grade varies between elevation 82 to 85 feet, mean sea level, based on the U.S. Geological Survey (USGS) Quad Map information and the 1929 National Geodetic Vertical Datum (NGVD). Final site grade elevation will be determined during detail design.

10G.92 Foundation Design Criteria

10G.9.1 General

Reinforced concrete structures (spread footings, mats, and deep foundations) will be designed consistent with Appendix 10B.

Allowable soil bearing pressures for foundation design will be in accordance with this Appendix and the detailed geotechnical investigation for the site.

10G.9.2 Groundwater Pressures

Hydrostatic pressures due to groundwater or temporary water loads will be considered.

10G.9.3 Factors of Safety

The factor of safety for structures, tanks and equipment supports with respect to overturning, sliding, and uplift due to wind and buoyancy will be as defined in Appendix 10B, Structural Engineering Design Criteria.

10G.9.4 Load Factors and Load Combinations

For reinforced concrete structures and equipment supports, using the strength method, the load factors and load combinations will be in accordance with Appendix 10B, Structural Engineering Design Criteria.

10G.10 References

California Building Code. 1998.

Department of the Navy.1982. "Identification and Classification of Soil and Rock." Chapter 1 in *Soil Mechanics Design Manual 7.1*. Naval Facilities Engineering Command. Alexandria, VA.

Caltrans. 1996. "California Seismic Hazards Map."

CET

CONDOR EARTH TECHNOLOGIES INC.

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August 27, 2002

Mr. Jeffrey T. Barton
Turlock Irrigation District
333 East Canal Drive
Turlock, CA 95381

Subject: Revised July 30, 2002 Letter
Anticipated Soil Conditions at Proposed Power Plant
Southeast Corner of Southern Tidewater and Washington Road, Turlock, California
Condor Project No. 3739

Reference: Geotechnical Engineering Study, 115 KV Walnut Substation
Condor Project No. 3002, July 19, 2000

Dear Mr. Barton:

We understand that Turlock Irrigation District (TID) is reviewing the feasibility of a proposed power generation plant on the southeast corner of the intersection of Southern Tidewater and Washington Road, west of Turlock, California. At your request, a Condor Earth Technologies, Inc. (Condor) Associate Geologist performed a site visit on July 29, 2002, to the proposed project site. The proposed location, shown on Figure 1, is located southeast of the existing TID Walnut 115 KV Substation located in Turlock, California. A site visit and a records review, including the referenced report, were conducted to better understand the anticipated soil conditions at the proposed site. This letter report documents our site visit and records review, and provides a summary of our findings.

The proposed site is located in the San Joaquin Valley within the Great Valley Geomorphic Province. The site is underlain by the Modesto Formation, which consists of arkosic alluvial deposits. The surface soils observed at the site during our visit were visually classified as light brown to medium brown silty sands. The site was covered with silage corn at the time of our site visit. Condor performed a Geotechnical Engineering Study (GES), Walnut 115 KV Substation, dated July 19, 2000 for the Turlock Irrigation District (Reference 1). The surface soils at the substation also consisted primarily of silty sands.

The USDA Soil Survey for Eastern Stanislaus Area California, Series 1957, No. 20, issued in September 1964, classifies the soils at both sites as the Dinuba sandy loam with a small area in the northwest corner of the proposed site as Dinuba sandy loam, slightly saline-alkali. A loam is a soil composed of a mixture of clay, silt, sand, and organic matter. The description is consistent with the observed soils at both sites. We also reviewed the TID Water Well Records in the vicinity of the site with the assistance of TID. Based on the review, the groundwater in the vicinity of the site (north ¼ mile) was 6.9 feet below ground surface (bgs) in July, 2002. The average groundwater depth for the last year has ranged from 7.0 to 9.0 feet bgs. According to TID records, during a heavy rainfall season in 1997/1998, the groundwater at the site was as shallow as 1 to 4 feet bgs.

The soil conditions at the substation site consisted of silty sands to depths of approximately 5.5 feet, underlain by silts, silty sands, and poorly graded sands in alternating layers at various depths, to a maximum depth of 31.5 feet. A lense of clay was encountered in boring SB-4 from 6.0 to 6.5 feet. Boring SB-6 encountered poorly graded sand with and without silt to 21.0 feet, underlain by silty sand to 31.0 feet. The site is located in a moderately active seismic area of California, and it is not transected by any known active faults. A more detailed discussion of the seismic setting is provided in the referenced GES. The GES for the substation is attached for your convenience.

Due to the close proximity of the two sites, we would anticipate the subsurface soils conditions to be similar. Based on the reviewed data discussed above, the proposed site appears feasible from a geotechnical standpoint. We anticipate that light to moderate foundation loads can be supported by shallow spread foundations. Heavy loads may be supported on mat foundations or driven piles. Shallow groundwater appears to be the most likely constraint to impact development of the site, particularly for subterranean structures.
This letter addresses the anticipated soil conditions at the proposed power plant site. Site-specific exploration is recommended to further assess the subsurface soils. We recommend the GES be performed in accordance with the 1997 Uniform Building Code (UBC) prior to any design or construction at the proposed site. The limitations section of the referenced report is hereby incorporated by reference.

Please call with any questions or concerns.

Respectfully submitted,

CONDOR EARTH TECHNOLOGIES, INC.

Ronald L. Skaggs, G.E.
Senior Geotechnical Engineer

Christopher C. Yarnell
Associate Geologist

Attachments: Figure 1
Geotechnical Engineering Study, Condor Project 3002

REGISTERED PROFESSIONAL ENGINEER
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EXP. 03/06
GEOTECHNICAL
STATE OF CALIFORNIA

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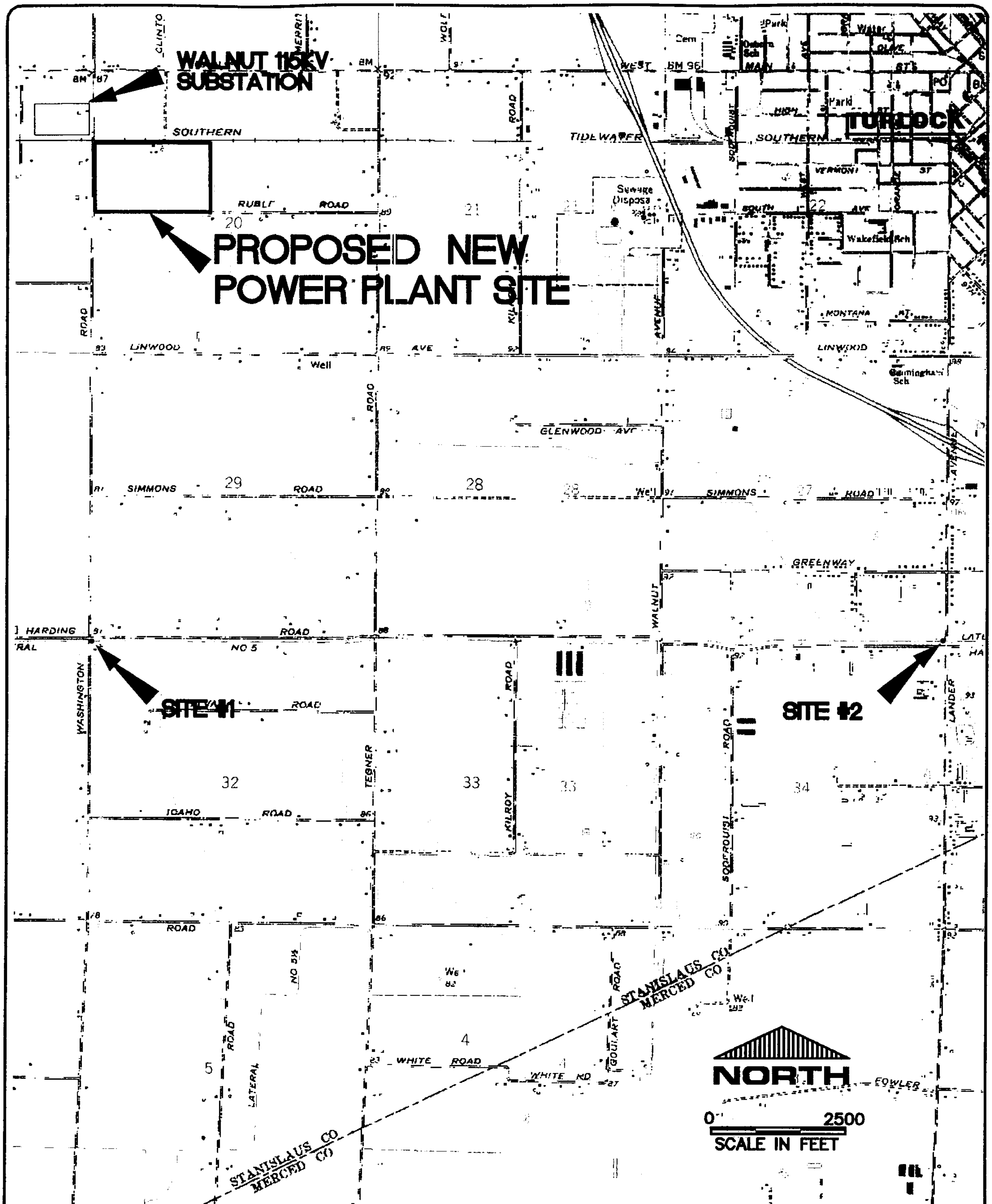

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Job No.	3739
Date	8/2/02
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VICINITY MAP
PROPOSED NEW POWER PLANT LOCATION
GEOTECHNICAL STUDY
TRULOCK IRRIGATION DISTRICT
TRULOCK, CALIFORNIA

FIGURE 1

File No. 3739F1

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GEOTECHNICAL ENGINEERING STUDY
TURLOCK IRRIGATION DISTRICT
WALNUT 115 KV SUBSTATION
TURLOCK, CALIFORNIA

Prepared for
Turlock Irrigation District
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July 19, 2000
Project No. 3002

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LIST OF APPENDICES

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GEOTECHNICAL ENGINEERING STUDY
TURLOCK IRRIGATION DISTRICT
WALNUT 115 KV SUBSTATION
TURLOCK, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering study for the Turlock Irrigation District proposed Walnut 115 KV Substation and two accompanying proposed transmission towers sites near Turlock, California. The proposed substation site is located approximately 2 miles west of Interstate 5 in a rural area west of Turlock., California. A site location map is shown on Figure 1, Appendix A. Also included in this report, are boring logs and soil testing data for two proposed transmission tower locations. These locations are referred to as site 1 and site 2 and are located along Harding Road west of Turlock. This report does not include geotechnical recommendations for the proposed towers.

The purpose of this study was to explore and evaluate subsurface soil conditions, and based on the information obtained, provide engineering recommendations for design and construction of the proposed substation structures.

This report is based upon data obtained from a total of six soil borings, field and laboratory testing, and general field observations made during the on-site investigation. The locations of the soil borings at the proposed substation are shown on the site map on Figure 2, Appendix A of this report.

2.0 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The proposed substation site is located at an elevation of approximately 83 feet above mean sea level (amsl) and covers approximately 1.5 acres. A fallow field is present along the west boundary, and Washington Road is parallel to the east boundary. The Tidewater Railroad is present at the south boundary, and the existing Walnut Substation borders the north boundary at 325 South Washington Road. The proposed substation site has been used for row crops, and the site is relatively flat.

We understand that the project will consist of two buildings utilizing slab-on-grade foundations and various high-voltage electrical structures associated with the construction of the substation. The buildings will be used for equipment storage and miscellaneous administrative requirements.

3.0 INVESTIGATIVE METHODS

A geotechnical field investigation of the site was conducted on June 7 and 8, 2000. A total of six exploratory soil borings were drilled under Condor's direction by Spectrum Exploration. Four borings were drilled at the proposed substation site, of which, two were to a depth of 15 feet and two to 30 feet. One boring was drilled for each of the two transmission tower locations to a depth of 40 feet. The boreholes were advanced using a CME 45 drilling rig, equipped with 6-inch diameter hollow stem augers. Soil boring logs documenting the subsurface soil conditions encountered were completed for each borehole. The project geotechnical engineer collected selected samples of each strata type for classification and review. The contacts between soil horizons were approximated based on field observations. The actual boundaries between different soil types may be gradual, and soil conditions may vary between borehole locations. The borehole logs are provided in Appendix B.

Discrete sampling methods alternated between a 2.0-inch O.D., split-spoon sampler fitted with three brass sleeves and a 2-inch O.D., standard penetration test (SPT) sampler. Samples were driven using a 140-pound hammer with a free fall of 30 inches. The sampling interval consisted of driving two samples near the surface (one at 0-1.5 feet and one from 1.5-3.0 feet) and additional samples were driven at 5-foot intervals to the maximum boring depth. The samples were logged according to the Unified Soil Classification System under the supervision of a California Registered Geotechnical Engineer.

Blow counts were recorded on the Soil Boring Logs, and samples were analyzed in the field for relative density, cohesiveness, field moisture, and visual grain size distribution. Brass sleeves were sealed with plastic end caps and labeled. Samples collected in the SPT sampler were placed in plastic bags, sealed, and labeled. A bulk sample of soil was also collected from 0-2 feet in the northeast portion of the proposed substation site at SB5 for compaction testing. Collected samples were transported to our Stockton office for laboratory testing and are retained pending review of this report.

Selected bulk and undisturbed soil samples recovered by Condor were tested in the laboratory to determine soil characteristics and to measure pertinent engineering and index properties. The tests performed include compaction testing, direct shear strength tests, grain size analysis, and moisture content and dry density determination. Soil resistively testing was also performed in the field at the proposed substation site. The results of the laboratory testing and resistivity testing are provided in Appendix C.

4.0 FINDINGS

4.1 GEOLOGIC SETTING

The project area is located in the Great Valley Geomorphic Province in the north central portion of the San Joaquin Valley at an elevation of approximately 83 feet above mean sea-level (amsl). The San Joaquin Valley is a northwest-trending, westward-dipping geosyncline filled with up to six vertical miles of lithified non-marine and marine, and unlithified non-marine sediments. Regionally, the lithology of the upper 3,000 feet of sediments is indicative of the Sierra Nevada to the east and, to a lesser degree, the Coast Range Mountains to the west. The Coast Range Mountains are approximately 25-mi southwest of the site, and the Sierra Nevada Mountains are approximately 15-mi northeast of the site.

The Coast Range Mountains generally consist of northwest trending ridges with Franciscan Complex and granitic basement rocks. The Sierra Nevada province is an asymmetric range with a steep fault-bounded eastern front and gentle western slope that dips under the sediments of the Great Valley to the west. The bedrock complex of the Sierra Nevada Mountains generally consists of metamorphosed sedimentary and volcanic rocks of Paleozoic and Mesozoic age (150 to 300 million years old) and plutonic rocks (chiefly granitic types) of Mesozoic age (80 to 150 million years old).

Structurally, the Coast Ranges - Sierra Nevada Block Boundary Zone, a regional geological boundary separating Franciscan basement rocks of the Coast Range from granitic basement rocks the Sierra Nevada Range, is present at depth near the western margin of the Great Valley Geomorphic Province.

4.2 PROJECT GEOLOGY AND GROUNDWATER CONDITIONS

Examination of drill cuttings and soil samples from the six boreholes indicates that the subsurface soils consist of unconsolidated alluvium. Groundwater at the site 2-transmission tower was encountered at approximately 17. 5 feet below ground and was not measured at the site 1-transmission tower location. At the site of the proposed substation, groundwater levels varied from approximately 4 to 6 feet below the existing ground surface. Resistant bedrock was not encountered in any of the soil borings.

4.3 SITE SUBSURFACE CONDITIONS

4.3.1 PROPOSED SUBSTATION SITE

The near surface soils, to a depth of approximately 3 feet below grade, generally consist of dry to damp, silty sand. On the north side of the site, these silty sands tend to grade into silts below 6 feet and back to silty sands at 10 feet. The silt layer was not encountered on the south side of the site. Poorly graded sand was encountered in all four boreholes at approximately 16 feet. From 16 feet to the bottom of the boreholes the soils varies from silty sand to poorly graded sands. The sand ranges from fine to coarse grained, and is predominantly fine to medium grained.

4.3.2 TRANSMISSION TOWER SITES

The soil stratigraphy at the transmission tower sites varies between beds of silts, silty sand, and poorly graded sands. Intermittent clay and silt lenses are also present.

The foregoing is a summary of the soil and rock conditions encountered during our borehole drilling for this investigation. More detailed descriptions of the subsurface soil conditions encountered are contained in the soil boring logs in Appendix B.

4.4 GROUND RESISTIVITY SURVEY

On June 28, 2000 a Wenner four point geophysical survey was conducted to evaluate the resistivity in the upper three feet of the ground surface at the proposed substation site. A total of nine soundings, spread evenly throughout the site, were tested using a Sting R1 (resistivity meter). At each location six separate resistivity measurements were collected using A-spacings of 1.0, 1.5, 2.5, 3.5, 5.5, and 7.5 feet. The calculated resistivities across the site ranged from approximately 257 ohm-meters in the east to 25 ohm-meters in the west. The average across the site is 56 ohm-meters, decreasing quickly from southeast to northwest. Tabulated results from the field tests are included in Table 1, Appendix C. An isopleth map of the average reisistivity, is included in Appendix A.

5.0 SEISMIC SETTING

The project site is located within Seismic Zone 3, according to Figure 16A-2-Seismic Hazard Map, 1994 UBC. Structures, therefore, should be designed in accordance the UBC for Seismic Zone 3. No known active or potentially active faults cross the proposed substation site, and the site is not located in a Fault-Rupture Hazard Zone, as established by the Alquist-Priolo Earthquake Fault Zoning Act (Hart, 1994).

6.0 BEARING CAPACITY

The bearing capacity of the subsurface soils was evaluated by visual classification, ease of excavation, pocket penetrometer and torvane measurements, and laboratory testing. Laboratory testing included unit weight analysis and direct shear testing for design of structural foundations.
Groundwater was encountered at approximately 4 to 6 feet below grade in SB3, SB4, SB5, and SB6, and the unsaturated soils at the project site are predominantly medium dense to dense. Based on the data collected at the locations of the soil borings, the project site materials are suitable for support of the proposed structure utilizing slab-on-grade construction with strip spread footings or isolated spread footings for supporting wall and roof loads when founded in native soil or engineered fill. The upper 12 inches of existing soil should be reworked to meet the requirements of engineered fill in all structural areas. All engineered fill and foundations should be designed and constructed in accordance with Sections 7.0 and 8.0 of this report. Results of the laboratory testing can be found in the appendix of this report.

7.0 GRADING AND EARTHWORK RECOMMENDATIONS

7.1 GENERAL RECOMMENDATIONS

All grading and site work should be performed in accordance with the 1997 Uniform Building Code (UBC), Chapter 33 (Site Work, Demolition and Construction), Appendix Chapter 33 (Excavation and Grading), Chapter 18 (Foundations and Retaining Walls), and with the recommendations of the Geotechnical Engineer of Record during construction. Where the recommendations of this report and the cited sections of UBC conflict, the owner should request clarification from the Geotechnical Engineer of Record. The recommendations of this report should not be waived without the consent of the Geotechnical Engineer of Record. Recommendations for additional work and construction monitoring are contained in Section 12.

7.2 GRADING RECOMMENDATIONS

Based on our study, the existing site soils are suitable for support of the proposed structures where the upper 12-inches of the existing site soils are properly stripped of organic matter, over-excavated, scarified, moisture conditioned, and backfilled with engineered fill compacted to the requirements of Section 7.2.3. The existing soils in building areas and a minimum of five feet beyond the limits of the building should be removed and recompacted. We anticipate that the near-surface site soils to be used for earthwork will consist of silty sand with a very low to low expansion potential. Such soils will dictate a minimum relative compaction of 90% in accordance with Section 7.2.3 Engineered Fill Placement.

The existing site soils are suitable for use as engineered fill when prepared and placed in accordance with the recommendations of this report. Fill materials may consist of the on-site native soils, relatively free of organic materials, placed on native materials, and compacted to a minimum of 90% relative compaction and a maximum relative compaction of 95% beneath pavements. Import materials meeting the requirements of Section 7.2.2 may also be used for earthwork construction.

7.2.1 Site Preparation and Grading

The site preparation and grading operations for the proposed areas to receive buildings, pavement, and other permanent structural improvements should commence with the removal of vegetation and the stripping of organic soil in all areas to receive improvements. Removal of vegetation should extend a minimum of five feet beyond the limits of the proposed improvements. Topsoil consisting of the natural accumulation of native grasses and mulch-like materials is generally not present on the site, but remnants of agricultural activities remain. Any organic laden material that is free from debris may be stockpiled for later use in non-structural areas approved by the owner, but should not be used for engineered fill.

Any voids left by the removal of trees, shrubs, or other buried objects, including existing site improvements, foundations, and buried utilities should be cleared of all loose soils and properly backfilled as described below. All stripped vegetation and debris should be removed from the areas receiving structural improvements.

The exposed non-organic soil surfaces in any areas to receive fill or structural improvements should be over-excavated to a minimum depth of 12 inches, the exposed subgrade scarified, and brought to a moisture content plus or minus 2% of optimum, and compacted to 90% relative compaction. After stripping and reworking the exposed subgrade, the Geotechnical Engineer of Record should view the native ground prior to fill placement. Any soft or loose pockets of native soil or fill encountered during the scarifying and compaction process should be fully excavated and replaced with engineered fill.

Relative compaction shall be determined based upon the ratio of field in-place dry density to the maximum dry density, as determined by the ASTM D-1557 test procedure. The upper 12 inches of subgrade soils in non-structural areas should be moisture conditioned to the above requirements and worked where necessary to achieve 90% relative compaction. Field density tests should be taken to verify compaction.

7.2.2 Engineered Fill Materials

We recommend any structural fill required to achieve finished grade should consist of on-site "non-organic" soils or imported soil having a maximum of 40% passing the #200 Sieve, a minimum R-Value (Resistance-Value) of 50, a maximum plasticity index of 12, and a maximum particle size of 4 inches. Any import material that does not meet the above criteria should be tested under the direction of the Geotechnical Engineer of Record to determine if the proposed import has engineering properties equivalent or better than the existing site materials. We recommend that samples of any proposed imported fill material be submitted to the Laboratory of Record for testing and approval by the Geotechnical Engineer of Record prior to being brought to the site.

7.2.3 Engineered Fill Placement

Engineered fill and structural or trench backfill should be placed in horizontal layers (lifts) with a maximum of 8 inches in loose thickness and compacted to the required compaction. Soils should be compacted to a minimum of 90% relative compaction. Relative compaction should be determined in accordance with ASTM D1557 in all structural areas. Compaction should be performed at moisture content of plus or minus 2% of optimum by aerating or adding moisture as needed prior to applying compactive effort.

We recommend that samples selected for compaction testing be collected when on-site earthwork operations have commenced. This procedure will allow for collection of samples which are representative of the materials used for any earthwork construction. All permanent cut or engineered fill slopes in soil should be graded too less than or equal to 2H: 1V.

7.2.4 Trench Backfill Compaction

Trench backfill should be placed in the same manner as required in Section 7.2.3, Engineered Fill Placement. A sandy material with a minimum sand equivalent of 20 should be used in the pipe zone where a manufacturer's specification is not available. The pipe zone material should be compacted to a minimum of 90% relative compaction or the manufacturer's recommendations where available. Trench backfill compaction criteria may by decreased to 85% relative compaction in landscape areas five feet beyond structural improvements, except in areas overlain by pavements, sidewalks, or other hardscapes.

7.2.5 Compaction Criteria in Landscape Areas

The compaction criteria in landscape areas may be reduced to 85% relative compaction when approved by the Geotechnical Engineer of Record during construction. The reduced compaction criteria should be limited to areas beyond five feet of structural improvements, and should not include hardscape areas such as sidewalks, hard courts, and other paved areas. Stripping in the areas of reduced compaction may be eliminated upon approval of the Geotechnical Engineer of Record at the time of grading.

8.0 FOUNDATION RECOMMENDATIONS

All foundation improvements should be designed and constructed in accordance with the 1997 Uniform Building Code (UBC), Chapter 17 (Structural Tests and Inspections), Chapter 18 (Foundations and Retaining Walls), and all other sections applicable to the proposed structural improvements. Shallow spread foundations and drilled piers less than 20 feet in maximum depth are anticipated and may be founded in native soils and engineered fill, provided the Grading and Earthwork Recommendations (Section 7) of this report are adhered to during the design and construction of earthwork and foundation improvements. Note that all stated bearing pressures in Section 8.2 are net values, and the weight of concrete in the portion of the foundations that extend below grade can be neglected in proportioning the foundations. Further evaluation of the subsurface may be warranted based on any other specific foundation designs not considered in this report. Foundation recommendations in this report are intended for the proposed substation site and not the proposed transmission tower sites.

8.1 FOOTING EMBEDDMENT REQUIREMENTS

Footings should extend at least 12 inches below the lowest adjacent grade for footings of single-story buildings and 18 inches below the lowest adjacent grade for two-story buildings. We recommend that the structural reinforcement be designed by the Structural Engineer. A minimum of one No. 4 bar in the top and one in the bottom are recommended in all footings. We recommend that a representative of the Geotechnical Engineer of Record prior to placing reinforcing steel observe all foundation excavations. This inspection should be conducted to ensure that the bottoms and sides of all footing trenches are level or suitably benched and are free of loose or soft soil, ponded water, and debris. If any loose pockets are encountered in the bottom of the footing excavations, they should be over-excavated, and the base of the trench should be recompacted or backfilled with lean concrete.

8.2 ALLOWABLE BEARING PRESSURES AND ESTIMATED SETTLEMENTS

8.2.1 Allowable Vertical Bearing Pressures

When the provisions of the previous sections are adhered to during design and construction, shallow foundations may be designed for vertical bearing pressure in accordance with the following:

$$Q = 1{,}500D + 900B$$

Where Q = allowable bearing pressure in pounds per square foot (psf) to a maximum of 3,500 psf
D = minimum embedment depth below lowest adjacent grade in feet (ft)
B = minimum footing width in feet (ft)

The following formula may be used in determining the depth of embedment for drilled piers required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement.

$$d = \frac{A}{2}\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right)$$

Where:

$$A = \frac{2.34P}{S_1 b}$$

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b = diameter of round post or footing or diagonal dimension of square post or footing, feet.
d = depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.
h = distance in feet from ground surface to point of application of "P."
P = applied lateral force in pounds.
S_l = allowable lateral soil-bearing pressure based on a depth of one third the depth of embedment using an equivalent fluid pressure of 300 pcf, to a maximum of 1,500 psf.

For piers that are five feet or less in length, the top one (1) foot of lateral soil bearing pressure should be ignored.

These bearing values are based on the results of direct shear testing of on-site soil and the results of field strength tests. The allowable vertical bearing capacity can be increased by one-third (1/3) for seismic and wind loading conditions.

8.2.2 Estimated Settlements

A design vertical movement for static and wind loading of 3/4 inch total vertical and 1/2 inch vertical over a span of 20 feet may be used for design of foundations, up to a maximum loading of 50 kips for column loads and loading of continuous footings of 3,500 pounds per linear foot. Lateral deflection at the ground surface of ½ to 1 inch should be anticipated for the above lateral bearing capacity equation.

Shallow foundations may be structurally tied to interior slabs-on-grade or float independently, but should be independent of concrete driveways and other hardscapes unless these members are structurally reinforced to form a mat type system.

8.2.3 Lateral Earth Pressures

Lateral earth pressures acting against footings and retaining walls on level ground may be designed using an equivalent fluid weight of 400 pounds per cubic feet (pcf), which can be increased for wind loading by one-third (1/3). The allowable lateral loading for seismic loading is 525 pcf where footings are founded in relatively level ground. The upper 12 inches of lateral resistance should be ignored unless concrete slab-on-grade or pavement protects the final grade. Lateral earth pressures may also be resisted using adhesion along the bottom of footings. An allowable coefficient of friction of 0.4 may be used for the bottom of footings for static and wind loading conditions.

8.3 SURFACE DRAINAGE CONTROL

Final grading around structures should be such that there is positive and enduring drainage away from the foundations. Ponding of water should not be allowed in structural areas, as ponded water will soften the sites clayey soils.

9.0 SLAB-ON-GRADE RECOMMENDATIONS

9.1 GENERAL RECOMMENDATIONS

All slabs should be poured at a maximum slump of less than 5 inches. Excessive water content is the major cause of concrete cracking. When fiber mesh and/or reinforcing bars are utilized, a water reducing agent or plasticizer may be utilized in the concrete to increase slump while maintaining a water/cement ratio, which will limit excessive shrinkage. Subgrades with clayey soil should be pre-moistened prior to concrete placement to close desiccation cracks.

9.2 EXTERIOR SLAB-ON-GRADE

If reinforcement is not used, a tool joint should be provided to separate the slab into sections of less than 150 square feet of surface area (400 square feet if fiber mesh is used with joints spaced not more than 20 feet apart in any direction). Concrete pavements that receive truck and forklift traffic should be a minimum of 6 inches in thickness.

9.3 INTERIOR SLAB-ON-GRADE

4 inches of free-draining gravel or clean crushed rock material should be placed between the finished subgrade and the floor slab, where interior slab-on-grade construction is employed. (Free draining gravel described herein should have a gradation of 100% passing a 1-inch sieve and have 5% passing a No. 4 sieve). We recommend that prior to placement of concrete, the subgrade should be moistened until a moisture equilibrium state is reached. Interior slabs-on-grade should be a minimum of 5 inches thick. The Structural Engineer should reinforce a 5-inch thick slab with No. 3 deformed bars at 24-inches on-center or closer, at the center of the structural section or as designed. Hot reinforcing steel should be cooled prior to concrete placement. The aforementioned reinforcement may be used for anticipated floor loads not exceeding 500 psf. If floor loads greater than 500 psf are anticipated, the Structural Engineer should evaluate the slab.

Where floor coverings are anticipated, a Visqueen-type membrane should be placed between the rock cushion and the slab to provide an effective vapor barrier and to minimize moisture condensation under the floor covering. It is suggested that a 1-inch thick sand layer be placed on top of the membrane to assist in the curing of the concrete. Where sand is used, the thickness of the gravel cushion may be decreased by the thickness of the sand. Where there is potential for moisture accumulation under the slab, special consideration should be given to allow gravity drainage of any water that could migrate into the subgrade of the slab or rock cushion.

10.0 RETAINING WALL RECOMMENDATIONS

In general, retaining walls should be designed to resist active lateral pressures exerted from a soil media having an equivalent fluid weight as follows:

Gradient of Backslope	Above Water Table Equivalent Fluid Weight (pcf)	Below Water Table Equivalent Fluid Weight (pcf)
Flat	40	82
2:1	55	90

For restrained conditions, an additional lateral loading of 10H psf, where H equals the restrained height, should be added to the equivalent fluid weight over the entire retained height of the wall.

For the "Above Water Table" conditions, we recommend that a blanket of filter material be placed behind all proposed walls. The blanket of filter material should be a minimum of 12 inches thick and should extend from the bottom of the wall to within 12 inches of the ground surface. The filter material should conform to Class One, Type B permeable material as specified in Section 68 of the California Department of Transportation Standard Specifications, current edition. A typical 1" x #4 concrete coarse aggregate mix approximates this specification. A clean pea-gravel is also acceptable. The top 12 inches of wall backfill should consist of a compacted native soil cap. Filter fabric should be placed around the gravel filter material to separate it from the wall backfill and the native soil cap. A perforated 4-inch-diameter drainpipe should be installed near the bottom of the filter blanket with perforations facing down.
Adequate gradients should be provided to discharge water that collects behind the retaining wall to an adequately controlled discharge system.

11.0 ASPHALT AND CONCRETE PAVEMENTS

All subgrade soils over which paving materials are placed should be compacted to a minimum depth of 8 inches and to a minimum of 95 percent of ASTM D-1557-78 maximum dry density. In addition, it is recommended that all pavements conform to the following criteria:

1. Asphalt pavements should consist of asphaltic concrete (A.C.) meeting current Caltrans specifications overlying Caltrans Class II aggregate base (A.B.).

2. All trench backfills, including culvert, utility, and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade. Ideally, these tasks should be completed following rough grading, and ahead of final grading and compaction.

3. An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the pavement subgrade soil.

4. Class 2 aggregate base rock should be compacted to a minimum of 95 percent of the ASTM D-1557-78 test density.

5. Class 2 aggregate base rock and asphalt concrete materials should conform to the specifications stated in Sections 26 and 39, respectively, of the State of California Standard Specifications, latest edition.

6. All curbs surrounding landscape areas should be embedded at least 6 inches below subgrade to minimize the movement of moisture beneath pavements, or other measures should be taken to drain aggregate base materials.

7. All driveways should be placed to minimize or eliminate drainage onto soil bearing surfaces.

8. All Portland cement concrete used for driveways and exterior uses should have a minimum compressive strength of 3,000 psi and should contain entrained air to help prevent freeze damage.

12.0 ADDITIONAL SERVICES

Because subsurface conditions are variable, it is impossible to include all construction details in plans and specifications. Design recommendations used as a basis of construction contracting are sensitive to a need for adjustment in the field. The adjustments are dependent upon findings during construction, which previously could only be assumed based on exploration activities. Since the intent of recommendations within this report are best understood by Condor representatives, we recommend that a review of final plans and field observations and testing during earthwork construction be performed by Condor.

We recommend that a registered engineer from our staff prior to construction bidding complete a review of all plans and specifications with regard to the earthwork. If Condor is not given the opportunity to review all plans and specifications with regard to earthwork, the reviewing registered engineer should thoroughly review this report and concur with its conclusions and recommendations, or provide additional recommendations. A qualified inspector should be present at the site during site preparation, earthwork, and grading. The inspector's observations should be supplemented with periodic compaction tests to establish substantial conformance with the recommendations contained in this report. The inspector should also observe all foundation excavations after cleaning and prior to placement of concrete to assess the depth of the footings and bearing conditions. A registered engineer should review the results of observations.

13.0 LIMITATIONS

The conclusions and recommendations contained in this report are for design purposes for the proposed Turlock Irrigation District proposed Walnut 115kV Substation in Turlock, California. They are invalid if:

1. The design loads change.

2. The report is used for adjacent or other property.

3. The recommendations contained in Additional Services (Section 12) are not followed.

4. Changes of grades and/or groundwater occur between the issuance of this report and construction.
5. Any other change is implemented which materially alters the project from that proposed at the time this report is prepared.

The analyses and recommendations submitted in this report are based upon the data obtained from the drilling of four boreholes at the locations shown on Figure 2, Appendix A. This report does not reflect variations, which may occur between the boreholes. The nature and extent of such variations may not become evident until construction is initiated. If variations then appear, a re-evaluation of the recommendations will be necessary after performing on-site observations during the excavation period and noting the characteristics of any variations.

The validity of the recommendations contained in this report are also dependent upon an adequate testing and observation program during the construction phase. Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform on-site testing and review during construction.

This report was prepared in accordance with the generally accepted standards of engineering geology, soils engineering, and civil engineering practice, which exist in Stanislaus County at the time the report was written. No other warranty, expressed or implied, is made.

It is the CLIENT'S responsibility to see that all parties to the project, including the designers, engineers, contractors, subcontractors, etc., are made aware of this report in its entirety.

We trust this report contains the information required. If you have any questions regarding this report, please contact us.

Sincerely,

CONDOR EARTH TECHNOLOGIES, INC.

Giovanni Del Papa, P.E.
Associated Engineer

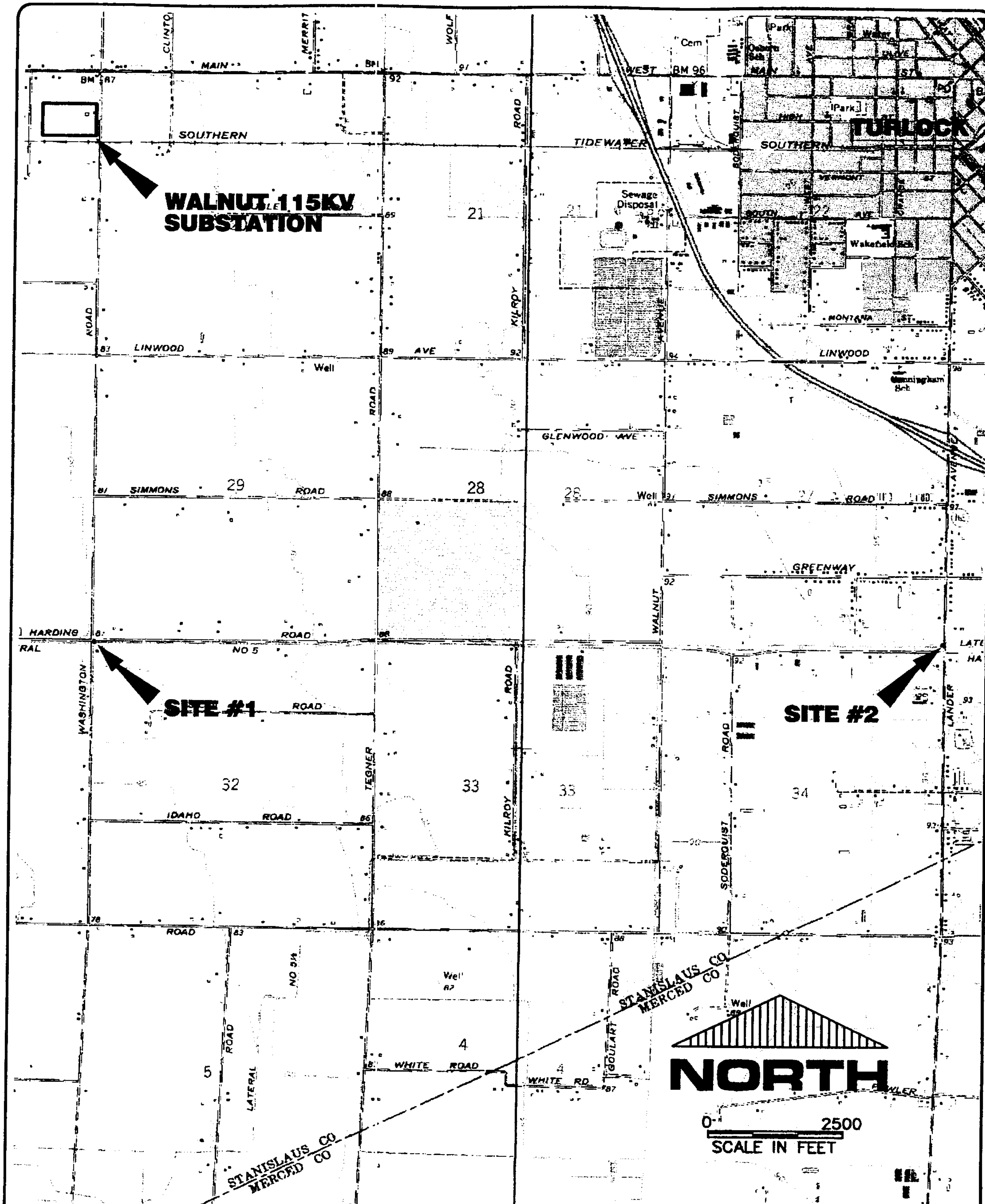
Ron Skaggs, G.E.
Division Manager

REFERENCES

Hart, E., 1994 (revised), Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps, California Division of Mines and Geology Special Publication 42, 34 pp.

ICBO (International Conference of Building Officials), 1997, Uniform Building Code.

APPENDIX A



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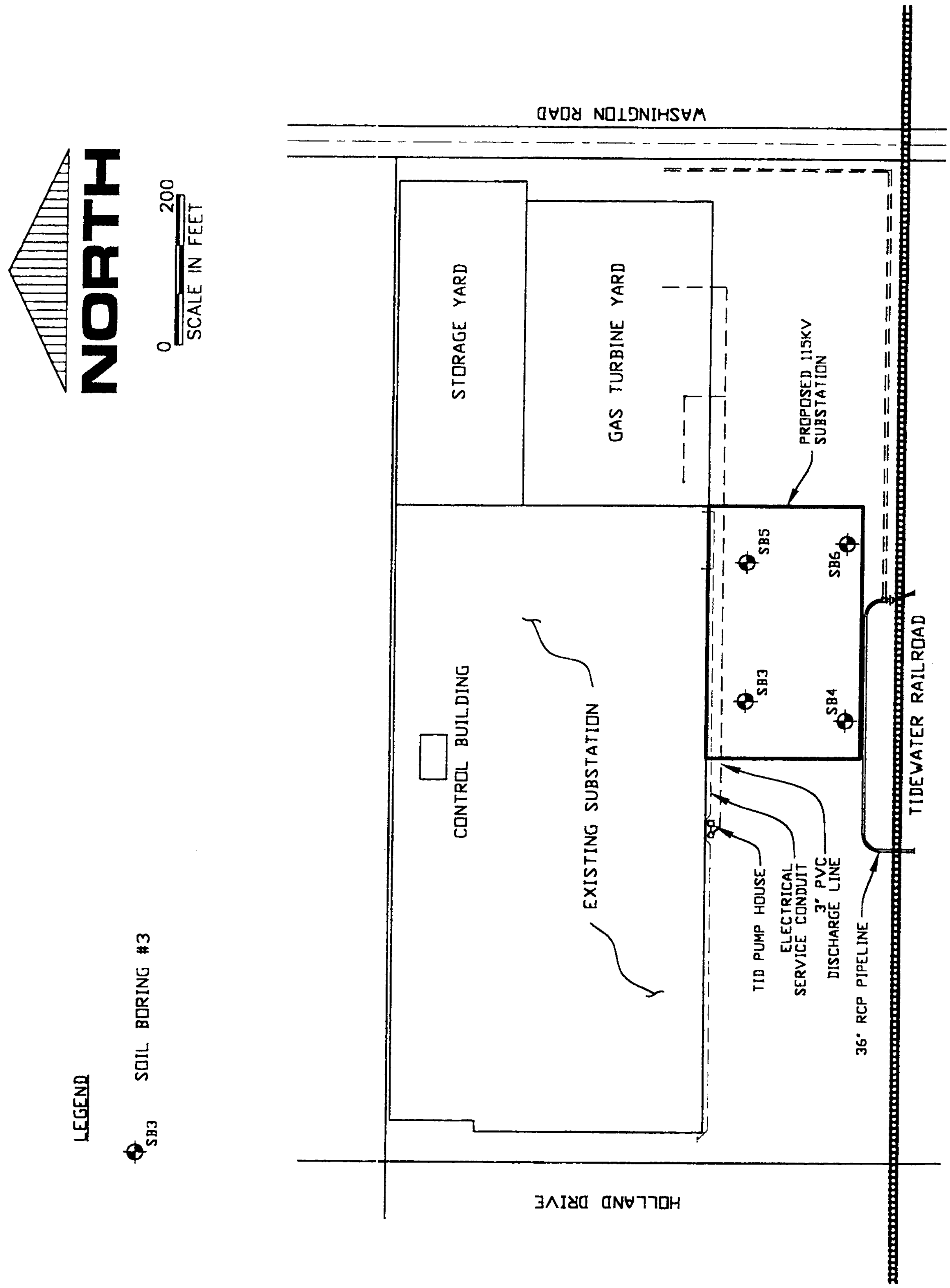
Job No.	3002
Date	6/13/00
Scale	AS SHOWN
Drawn / Chk'd	JKB / GDP

TURLOCK IRRIGATION DISTRICT
WALNUT 115KV SUBSTATION
GEOTECHNICAL STUDY
SITE LOCATION MAP

FIGURE 1

File No. 3002F1



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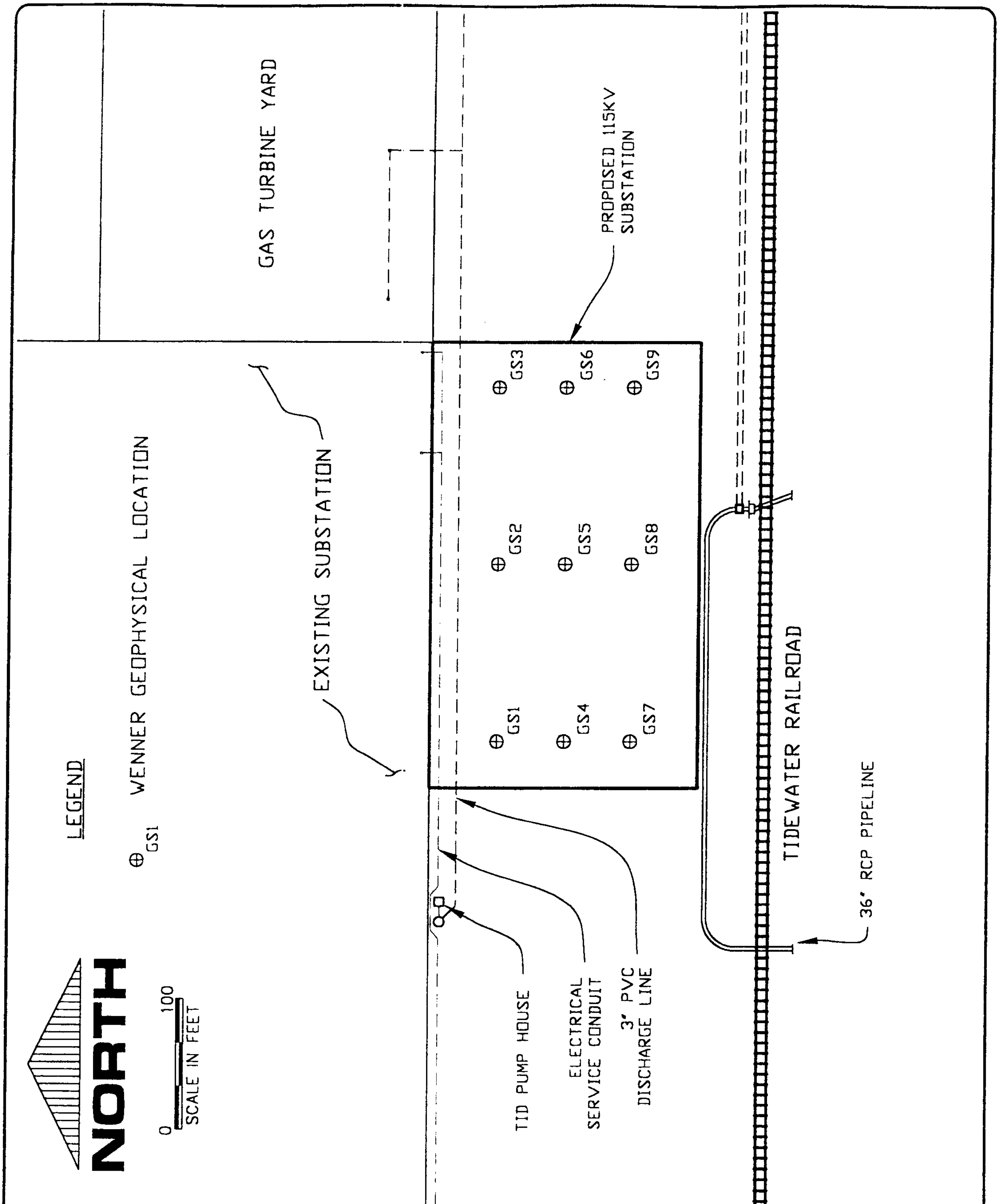
TURLOCK IRRIGATION DISTRICT
WALNUT 115KV SUBSTATION
GEOTECHNICAL STUDY
SOIL BORING MAP

FIGURE 2

File No. 3002F2



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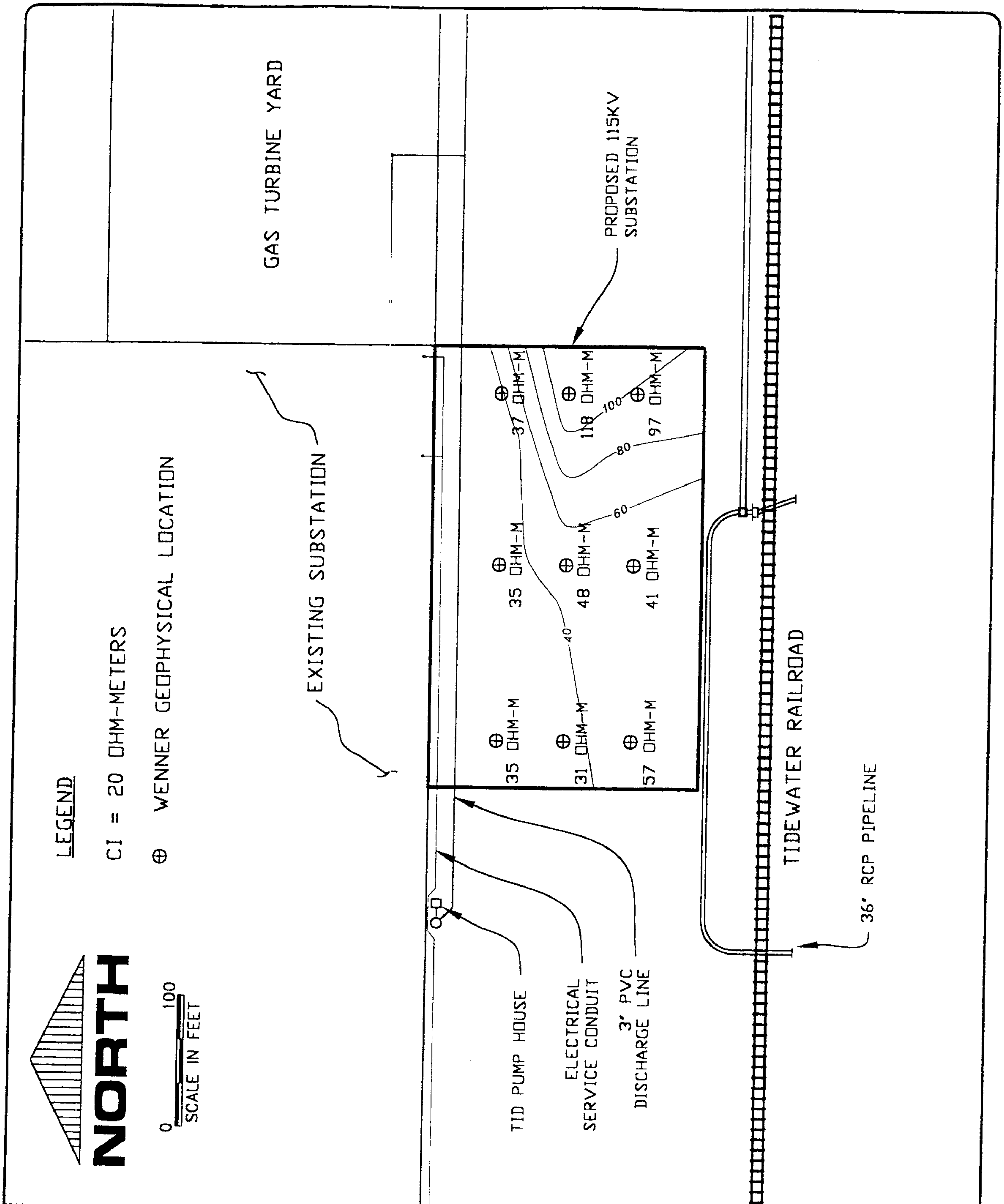
TURLOCK IRRIGATION DISTRICT
WALNUT 115KV SUBSTATION
GEOTECHNICAL STUDY
WENNER GEOPHYSICAL LOCATIONS

FIGURE 3

File No. 3002F3



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TURLOCK IRRIGATION DISTRICT
WALNUT 115KV SUBSTATION
GEOTECHNICAL STUDY
AVERAGE APPARENT RESISTIVITY

FIGURE 4

File No. 3002F4

APPENDIX B

LEGEND TO LOGS

TERMS AND SYMBOLS

SAMPLE

Sample types are indicated as follows:

■	2.5-inch liner sample	⊡	Unsuccessful attempt
◢	Standard penetration test sample	◿	Unsuccessful attempt

BLOW COUNT

The number of hammer blows required to drive the sampler the last 12 inches using a Standard Penetration Test hammer.

CLASSIFICATION AND DESCRIPTION

———	Conformable material change
———	Approximate material change
━━━	Bottom of hole

SYMBOLS and ABREVIATIONS

▼ Static water level

▽ First water

BOH: Bottom of Hole

OTHER TESTS

EI - Expansion Index
GS - Grain size distribution
RES – Resistivity
DS - Direct Shear

C - Consolidation
CH - Chemistry
RV – R-value

CP - Compaction
UC – Unconfined Compression
TV – Torvane

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SOIL BORING LOG

PROJECT T.I.D. - 3002 DATE 6/7/00 BORING NO. SB-1

SAMPLE ID	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PENETROMETER (tsf)	UNIFIED SOIL CLASSIFICATION	GEOTECHNICAL CLASSIFICATION AND DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	PLASTICITY INDEX (%)	LIQUID LIMIT (%)	UNDRAINED SHEAR STRENGTH (psf)	OTHER TESTS
					SP-SM	GRADING INTO... POORLY GRADED SAND WITH SILT BROWN, MOIST, DENSE, LOW PLASTICITY						
SB-1 BAG 36.5'	35		7 11 15		SP	POORLY GRADED SAND BROWN, WET, MED. DENSE, MEDIUM GRAIN SIZE						
SB-1 40.5' 41.0' 41.5'	40		11 18 27		SP-SM	POORLY GRADED SAND WITH SILT BROWN, VERY MOIST, DENSE, MICACIOUS FINES						UW
		BOH				TOTAL DEPTH: 41.5'						
	45											
	50											
	55											
	60											
	65											
	70											

CONDOR EARTH TECHNOLOGIES
STOCKTON, CALIFORNIA

SOIL BORING LOG

PROJECT: T.I.D. - 3002
DATE: 6/7/00
BORING NO.: SB-2
LOCATION: SITE #2
LOGGED BY: WR
DRILLING CONTRACTOR: SPECTRUM EXPLORATION
RIG TYPE: CME 45
HOLE DIAMETER: 6"
HAMMER WEIGHT AND FALL: 140LBS, 30"
TOTAL DEPTH: 41.5'
SURFACE CONDITIONS: SOIL
DEPTH TO GROUNDWATER: 17'-4"
SAMPLING METHOD: CAL MOD./SPT
DRILLING METHOD: HSA

NOTES: POWER IN NEARBY LINES TURNED OFF BY T.I.D.

SAMPLE ID	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PENETROMETER (tsf)	UNIFIED SOIL CLASSIFICATION	GEOTECHNICAL CLASSIFICATION AND DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	PLASTICITY INDEX (%)	LIQUID LIMIT (%)	UNDRAINED SHEAR STRENGTH (psf)	OTHER TESTS
SB-2 BAG 1.5			6 9 10		SM	SILTY SAND BROWN, DAMP, MED DENSE IRON MOTTLING						GS
SB-2 2.0 2.5 3.0			3 5 12			2.0': VERY MOIST TO WET. 3.0': FINE SILT FOUND IN PROBE TIP (NONE IN TUBE) WITH <5% FINE SAND.						UW DS
	5											
SB-2 BAG 11.5	10		4 6 6		SP	POORLY GRADED SAND BROWN, WET, MED DENSE FINE 100% SAND						
SB-2 15.5 16.0 16.5	15		6 9 13			15.5': TUBE SAMPLE SHOWING VARIATION TOWARDS FINER SAND						UW
				3.5 (SHOE)	CL	16.5': MATERIAL IN SHOE: DARK BRN./RED, SANDY CLAY						
SB-2 BAG 21.5	20		10 14 14		SP	20.0'-21.0': POORLY GRADED SAND, VERY CLEAN, LT BRN., MED. DENSE 21.0'-21.5': DARKER BROWN, TRACE OF CLAY. SOME COHESION (<5% FINES) 22.5': DRILLER'S NOTE: "HARDER AND SLOWER"						
SB-2 25.5 26.0 26.5	25		5 5 3		SC	CLAYEY SAND DARK BROWN, WET, LOOSE 40% FINES, SLIGHTLY CEMENTED, SILT MODULES.						UW UW
					ML	26.0': SILT LENSE						
					CL-ML	30.0-30.5': CLAYEY SILT WITH TRACE OF FINE SAND (<10%) AND CEMENTED MODULES.						
SB-2 BAG 31.5	30		2 5 12		SP	30.5': POORLY GRADED SAND MED. DENSE						

CONDOR

SOIL BORING LOG

PROJECT T.I.D. - 3002 DATE 6/7/00 BORING NO. SB-2

SAMPLE ID	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PENETROMETER (tsf)	UNIFIED SOIL CLASSIFICATION	GEOTECHNICAL CLASSIFICATION AND DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	PLASTICITY INDEX (%)	LIQUID LIMIT (%)	UNDRAINED SHEAR STRENGTH (psf)	OTHER TESTS
SB-2 35.5 36.0 36.5	35		10 17 21			35.5'~36.0' SMALL SILT/CLAY INCLUSIONS. POORLY CEMENTED.						GS UW
SB-2 BAG 41.5	40	BOH	17 20 23		ML	SILT WITH SAND BROWN, VERY MOIST, DENSE FINE-GRAINED SAND COMPONENT.						
						TOTAL DEPTH: 41.5'						
	45											
	50											
	55											
	60											
	65											
	70											

CONDOR EARTH TECHNOLOGIES
STOCKTON, CALIFORNIA

SOIL BORING LOG

PROJECT T.I.D. – 3002 DATE 6/8/00 BORING NO. SB-3
LOCATION WALNUT 115KV SUBSTATION LOGGED BY WR
DRILLING CONTRACTOR SPECTRUM EXPLORATION RIG TYPE CME 45
HOLE DIAMETER 6" HAMMER WEIGHT AND FALL 140LBS, 30" TOTAL DEPTH 31.5'
SURFACE CONDITIONS SOIL DEPTH TO GROUNDWATER 5'-1"
SAMPLING METHOD CAL MOD./SPT DRILLING METHOD HSA

NOTES: USED WATER SWIVEL TO KEEP STEM FULLY CHARGED WITH WATER TO REDUCE FLOWING SANDS

SAMPLE ID	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PENETROMETER (tsf)	UNIFIED SOIL CLASSIFICATION	GEOTECHNICAL CLASSIFICATION AND DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	PLASTICITY INDEX (%)	LIQUID LIMIT (%)	UNDRAINED SHEAR STRENGTH (psf)	OTHER TESTS
SB-3 BAG 1.5 SB-3 2.0 2.5 3.0			3 5 4 5 6 7		SM	SILTY SAND DARK BROWN, DAMP, LOOSE TO MED. DENSE, LOW PLASTIC FINES. SAND PORTION POORLY GRADED. ROOTS & STRONG EARTHY SMELL						UW
SB-3 BAG 6.5	5		3 16 11		ML	SILT LIGHT BROWN, DAMP, MED. DENSE, LOW PLASTICITY, WEAKLY CEMENTED SILT MODULES (<1/8 IN DIA)						GS
SB-3 10.5 11.0 11.5	10		20 20 20			10.0'–11.5' NO RECOVERY (SLUFF ONLY) DENSE						
					SM	SILTY SAND: LIGHT BROWN, WET, FIRM, LOW PLASTICITY.						
SB-3 BAG 16.5	15		6 5 6		SP	POORLY GRADED SAND DARK BROWN, WET, MED. DENSE, COARSE						
SB-3 20.5 21.0 21.5	20		8 18 20		SM	SILTY SAND BROWN, MOIST, DENSE, MEDIUM PLASTICITY						UW
					SP–SM	POORLY GRADED SAND W/SILT BROWN, VERY MOIST, DENSE LOW PLASTICITY						
SB-3 BAG 26.5	25		24 35 25		SP	POORLY GRADED SAND BROWN, WET, VERY DENSE						
SB-3 30.5 31.0 31.5	30	BOH	21 29 30									
						TOTAL DEPTH 31.5'						

SOIL BORING LOG

CONDOR EARTH TECHNOLOGIES
STOCKTON, CALIFORNIA

PROJECT: T.I.D. – 3002   DATE: 6/8/00   BORING NO. SB-6
LOCATION: WALNUT 115KV SUBSTATION   LOGGED BY: WR
DRILLING CONTRACTOR: SPECTRUM EXPLORATION   RIG TYPE: CME 45
HOLE DIAMETER: 6"   HAMMER WEIGHT AND FALL: 140LBS, 30"   TOTAL DEPTH: 31.5'
SURFACE CONDITIONS: SOIL   DEPTH TO GROUNDWATER: 3'–7"
SAMPLING METHOD: CAL MOD./SPT   DRILLING METHOD: HCA

NOTES: USING "WATER PIVOT" TO CHARGE AUGER STEM AND IMMOBILIZE FLOWING SANDS

SAMPLE ID	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PENETROMETER (tsf)	UNIFIED SOIL CLASSIFICATION	GEOTECHNICAL CLASSIFICATION AND DESCRIPTION	DRY DENSITY (pcf)	WATER CONTENT (%)	PLASTICITY INDEX (%)	LIQUID LIMIT (%)	UNDRAINED SHEAR STRENGTH (psf)	OTHER TESTS
			3 4 5		SP–SM	POORLY GRADED SAND W/ SILT						
SB–6 2.0' 2.5' 3.0'			4 5 4			DARK BROWN, DAMP, LOOSE, LOW PLASTICITY FINES, ROOTS, SURFACE ORGANICS, EARTHY SMELL, MICA FLAKES						UW
						2.0' ROOTS GONE BUT EARTHY ODOR REMAINS						
SB–6 BAG 6.5	5		2 2 3									
SB–6 10.5' 11.0' 11.5'	10		15 17 14			10.5' BROWN, VERY MOIST, DENSE						GS
SB–6 BAG 16.5	15		8 12 16									
					SP	POORLY GRADED SAND BROWN, MOIST, MED. DENSE, MEDIUM FINE GRAINED						
SB–6 20.5' 21.0' 21.5'	20		12 16 14			20.0' COARSE GRAINED, ROUNDED CLASTS						
					SM	SILTY SAND BROWN, MOIST, DENSE, NON–PLASTIC FINES						
						24.0' DRILLER REPORTS HITTING FLOWING SANDS						
	25		3 7 8			MED. DENSE (NO RECOVERY)						
	30		12 20 25			DENSE (NO RECOVERY)						
		BOH				TOTAL DEPTH: 31.5'						

APPENDIX C

DRY DENSITY AND MOISTURE CONTENT

Client: Turlock Irrigation District
Project: Walnut Substation
Condor Project No.: 3002

Sample #	SB-1	SB-1	SB-1	SB-1	SB-1	SB-2
Date	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00
Depth (ft.)	3.0	11.0	11.5	31.5	41.5	2.5
By	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens
Sample Location						
Dia. Sleeve (in.)	1.94	1.94	1.94	1.94	1.94	1.94
Length Sleeve (in.)	6.00	6.00	6.00	6.00	6.00	6.00
Tare Length (in.)	0.00	0.00	0.20	0.64	0.46	0.34
Sample Length (in.)	6.00	6.00	5.80	5.36	5.54	5.66
Volume (cu. in.)	17.74	17.74	17.14	15.84	16.38	16.73
Volume (cu. ft.)	0.0103	0.0103	0.0099	0.0092	0.0095	0.0097
Gross wt. (gms.)	732.6	739.1	773.4	738.1	742.9	694.4
Tare wt. (gms.)	184.0	185.3	186.0	186.6	185.4	184.4
Soil wt. wet (gms.)	548.6	553.8	587.4	551.5	557.5	510.0
Soil wt. (lbs.)	1.21	1.22	1.30	1.22	1.23	1.12
Wet Density (pcf)	117.4	118.6	130.8	132.2	129.4	115.9
Dry Density (pcf)	110.3	101.5	111.2	113.2	109.3	106.9
Tare #	AP	5	LZ	7	1	RS
Wet wt. & Tare (gms.)	662.3	745.0	779.5	741.4	746.8	701.5
Dry wt. & Tare (gms.)	629.2	665.1	691.7	662.2	660.3	661.8
Wt. of Water (gms.)	33.1	79.9	87.8	79.2	86.5	39.7
Wt. of Tare (gms.)	114.0	190.4	192.2	190.1	190.0	192.1
Wt. dry Soil (gms.)	515.2	474.7	499.5	472.1	470.3	469.7
Moisture Content%	6.4	16.8	17.6	16.8	18.4	8.5

Remarks:

DRY DENSITY AND MOISTURE CONTENT

Client: Turlock Irrigation District
Project: Walnut Substation
Condor Project No.: 3002

Sample #	SB-2	SB-2	SB-2	SB-3	SB-3	SB-4
Date	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00
Depth (ft.)	16.0	26.0	26.5	3.0	21.5	1.5
By	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens
Sample Location						
Dia. Sleeve (in.)	1.94	1.94	1.94	1.94	1.94	1.94
Length Sleeve (in.)	6.00	6.00	6.00	6.00	6.00	6.00
Tare Length (in.)	0.38	0.70	0.80	0.38	0.75	0.70
Sample Length (in.)	5.62	5.30	5.20	5.62	5.25	5.30
Volume (cu. in.)	16.61	15.67	15.37	16.61	15.52	15.67
Volume (cu. ft.)	0.0096	0.0091	0.0089	0.0096	0.0090	0.0091
Gross wt. (gms.)	762.3	696.3	675.2	737.6	738.5	649.7
Tare wt. (gms.)	185.1	186.9	187.0	184.6	185.5	184.7
Soil wt. wet (gms.)	577.2	509.4	488.2	553.0	553.0	465.0
Soil wt. (lbs.)	1.27	1.12	1.08	1.22	1.22	1.03
Wet Density (pcf)	132.6	123.4	121.0	127.0	135.5	112.7
Dry Density (pcf)	115.3	99.4	95.8	112.8	114.7	97.9
Tare #	AJ	BC	#3	#8	DD	#11
Wet wt. & Tare (gms.)	690.2	699.8	1445.8	1506.2	1483.0	1489.5
Dry wt. & Tare (gms.)	614.9	600.6	1344.3	1447.8	1401.3	1428.6
Wt. of Water (gms.)	75.3	99.2	101.5	58.4	81.7	60.9
Wt. of Tare (gms.)	113.0	190.5	957.6	983.2	951.5	1024.5
Wt. dry Soil (gms.)	501.9	410.1	386.7	464.6	449.8	404.1
Moisture Content%	15.0	24.2	26.2	12.6	18.2	15.1

Remarks:

DRY DENSITY AND MOISTURE CONTENT

Client: Turlock Irrigation District
Project: Walnut Substation
Condor Project No.: 3002

Sample #	SB-2	SB-2	SB-2	SB-3	SB-3	SB-4
Date	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00
Depth (ft.)	16.0	26.0	26.5	3.0	21.5	1.5
By	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens
Sample Location						
Dia. Sleeve (in.)	1.94	1.94	1.94	1.94	1.94	1.94
Length Sleeve (in.)	6.00	6.00	6.00	6.00	6.00	6.00
Tare Length (in.)	0.38	0.70	0.80	0.38	0.75	0.70
Sample Length (in.)	5.62	5.30	5.20	5.62	5.25	5.30
Volume (cu. in.)	16.61	15.67	15.37	16.61	15.52	15.67
Volume (cu. ft.)	0.0096	0.0091	0.0089	0.0096	0.0090	0.0091
Gross wt. (gms.)	762.3	696.3	675.2	737.6	738.5	649.7
Tare wt. (gms.)	185.1	186.9	187.0	184.6	185.5	184.7
Soil wt. wet (gms.)	577.2	509.4	488.2	553.0	553.0	465.0
Soil wt. (lbs.)	1.27	1.12	1.08	1.22	1.22	1.03
Wet Density (pcf)	132.6	123.4	121.0	127.0	135.5	112.7
Dry Density (pcf)	115.3	99.4	95.8	112.8	114.7	97.9
Tare #	AJ	BC	#3	#8	DD	#11
Wet wt. & Tare (gms.)	690.2	699.8	1445.8	1506.2	1483.0	1489.5
Dry wt. & Tare (gms.)	614.9	600.6	1344.3	1447.8	1401.3	1428.6
Wt. of Water (gms.)	75.3	99.2	101.5	58.4	81.7	60.9
Wt. of Tare (gms.)	113.0	190.5	957.6	983.2	951.5	1024.5
Wt. dry Soil (gms.)	501.9	410.1	386.7	464.6	449.8	404.1
Moisture Content%	15.0	24.2	26.2	12.6	18.2	15.1

Remarks:

DRY DENSITY AND MOISTURE CONTENT

Client: Turlock Irrigation District

Project: Walnut Substation

Condor Project No.: 3002

Sample #	SB-4	SB-4	SB-5	SB-5	SB-5	
Date	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	
Depth (ft.)	6.5	16.5	3.0	6.0	16.5	
By	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens	
Sample Location						
Dia. Sleeve (in.)	1.94	1.94	1.94	1.94	1.94	
Length Sleeve (in.)	6.00	6.00	6.00	6.00	6.00	
Tare Length (in.)	0.40	0.20	0.75	0.65	0.25	
Sample Length (in.)	5.60	5.80	5.25	5.35	5.75	
Volume (cu. in.)	16.55	17.14	15.52	15.81	17.00	
Volume (cu. ft.)	0.0096	0.0099	0.0090	0.0092	0.0098	
Gross wt. (gms.)	742.3	787.7	641.1	691.3	791.2	
Tare wt. (gms.)	185.1	185.9	184.7	185.8	187.5	
Soil wt. wet (gms.)	557.2	601.8	456.4	505.5	603.7	
Soil wt. (lbs.)	1.23	1.33	1.01	1.11	1.33	
Wet Density (pcf)	128.0	134.0	111.8	121.2	135.8	
Dry Density (pcf)	106.4	116.7	102.8	102.5	117.8	
Tare #	XL	QBB	ABC	X2	QBD	
Wet wt. & Tare (gms.)	1581.9	1599.0	1481.9	1451.6	1611.0	
Dry wt. & Tare (gms.)	1488.0	1521.2	1443.0	1374.4	1533.8	
Wt. of Water (gms.)	93.9	77.8	38.9	77.2	77.2	
Wt. of Tare (gms.)	1024.6	997.2	1000.5	951.5	1027.3	
Wt. dry Soil (gms.)	463.4	524.0	442.5	422.9	506.5	
Moisture Content%	20.3	14.8	8.8	18.3	15.2	

Remarks:

DRY DENSITY AND MOISTURE CONTENT

Client: Turlock Irrigation District

Project: Walnut Substation

Condor Project No.: 3002

Sample #	SB-4	SB-4	SB-5	SB-5	SB-5	
Date	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	07-Jun-00	
Depth (ft.)	6.5	16.5	3.0	6.0	16.5	
By	E. Spens	E. Spens	E. Spens	E. Spens	E. Spens	
Sample Location						
Dia. Sleeve (in.)	1.94	1.94	1.94	1.94	1.94	
Length Sleeve (in.)	6.00	6.00	6.00	6.00	6.00	
Tare Length (in.)	0.40	0.20	0.75	0.65	0.25	
Sample Length (in.)	5.60	5.80	5.25	5.35	5.75	
Volume (cu. in.)	16.55	17.14	15.52	15.81	17.00	
Volume (cu. ft.)	0.0096	0.0099	0.0090	0.0092	0.0098	
Gross wt. (gms.)	742.3	787.7	641.1	691.3	791.2	
Tare wt. (gms.)	185.1	185.9	184.7	185.8	187.5	
Soil wt. wet (gms.)	557.2	601.3	456.4	505.5	603.7	
Soil wt. (lbs.)	1.23	1.33	1.01	1.11	1.33	
Wet Density (pcf)	128.0	134.0	111.8	121.2	135.8	
Dry Density (pcf)	106.4	116.7	102.8	102.5	117.8	
Tare #	XL	QBE	ABC	X2	QBD	
Wet wt. & Tare (gms.)	1581.9	1599.0	1481.9	1451.6	1611.0	
Dry wt. & Tare (gms.)	1488.0	1521.2	1443.0	1374.4	1533.8	
Wt. of Water (gms.)	93.9	77.8	38.9	77.2	77.2	
Wt. of Tare (gms.)	1024.6	997.2	1000.5	951.5	1027.3	
Wt. dry Soil (gms.)	463.4	524.0	442.5	422.9	506.5	
Moisture Content%	20.3	14.8	8.8	18.3	15.2	

Remarks:

SIEVE ANALYSIS

CLIENT: Turlock Irrigation District
PROJECT: Walnut Substation
LOCATION: 16.0" – 16.5"
DESCRIPTION: Brown Sandy Silt (ML)

DATE: 06-21-00
PROJECT #: 3002
SAMPLE #: SB-1

TOTAL WEIGHT OF SAMPLE: 471.5

Sieve Openings (in.)	U.S. Standard Sieve #	Accumulated Weight Retained (gms)	Percent Retained Partial	Percent Retained Total	Percent Finer by Weight	Specification Operating Range
3.00	3-in.					
2.00	2-in.					
1.5	1-1/2-in.					
1.00	1-in.					
0.750	3/4-in.					
0.500	1/2-in.					
0.375	3/8-in.					
0 .187	No. 4					
0.132	No. 6					
0.094	No. 8	1.0		.20	99.8	
0.079	No. 10					
0.047	No. 16	7.0		1.5	98.5	
0.033	No. 20					
0.023	No. 30	50.2		10.6	89.4	
0.0165	No. 40					
0.0117	No. 50	126.5		26.8	73.2	
0.0098	No. 60					
0.0059	No. 100	181.2		38.4	61.6	
0.0041	No. 140					
0.0029	No. 200	225.2		47.8	52.2	
PAN						
TOTAL WEIGHT						

TECHNICIAN: D. Bailey

CHECKED BY: GDP

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SIEVE ANALYSIS

Turlock Irrigation District
: Walnut Substation
N: 26.0' – 26.5'
TION: Silty Sand (SM)

DATE: 06-21-00
PROJECT #: 3002
SAMPLE #: SB-1

EIGHT OF SAMPLE: 473.0

ve nings n.)	U.S. Standard Sieve #	Accumulated Weight Retained (gms)	Percent Retained Partial	Percent Retained Total	Percent Finer by Weight	Specification Operating Range
.00	3-in.					
.00	2-in.					
1.5	1-1/2-in.					
.00	1-in.					
.750	3/4-in.					
.500	1/2-in.					
.375	3/8-in.					
.187	No. 4	0.60		.10	99.9	
).132	No. 6					
).094	No. 8	1.00		.20	99.8	
).079	No. 10					
0.047	No. 16	5.7		1.2	98.8	
0.033	No. 20					
0.023	No. 30	40.6		8.6	91.4	
0.0165	No. 40					
0.0117	No. 50	236.3		50.0	50.0	
0.0098	No. 60					
0.0059	No. 100	368.4		77.9	22.1	
0.0041	No. 140					
0.0029	No. 200	396.1		83.7	16.3	
PAN						
TOTAL WEIGHT						

TECHNICIAN: D. Bailey

CHECKED BY: GDP

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SIEVE ANALYSIS

CLIENT: Turlock Irrigation District
PROJECT: Walnut Substation
LOCATION: 0 – 1.5'
DESCRIPTION: Brown Silty Sand (SM)

DATE: 06-21-00
PROJECT #: 3002
SAMPLE #: SB-2

TOTAL WEIGHT OF SAMPLE: 471.3

Sieve Openings (in.)	U.S. Standard Sieve #	Accumulated Weight Retained (gms)	Percent Retained Partial	Percent Retained Total	Percent Finer by Weight	Specification Operating Range
3.00	3-in.					
2.00	2-in.					
1.5	1-1/2-in.					
1.00	1-in.					
0.750	3/4-in.					
0.500	1/2-in.					
0.375	3/8-in.					
0 .187	No. 4	4.3		0.9	99.1	
0.132	No. 6					
0.094	No. 8	6.0		1.3	98.7	
0.079	No. 10					
0.047	No. 16	9.0		1.9	98.1	
0.033	No. 20					
0.023	No. 30	33.1		7.0	93.0	
0.0165	No. 40					
0.0117	No. 50	166.1		35.2	64.8	
0.0098	No. 60					
0.0059	No. 100	300.6		63.8	36.2	
0.0041	No. 140					
0.0029	No. 200	383.4		81.3	18.7	
PAN						
TOTAL WEIGHT						

TECHNICIAN: D. Bailey

CHECKED BY: GDP

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SIEVE ANALYSIS

CLIENT: Turlock Irrigation District
PROJECT: Walnut Substation
LOCATION: 35.0' – 36.0'
DESCRIPTION: Silty Sand (SM)

DATE: 06-21-00
PROJECT #: 3002
SAMPLE #: SB-2

TOTAL WEIGHT OF SAMPLE: 466.8

Sieve Openings (in.)	U.S. Standard Sieve #	Accumulated Weight Retained (gms)	Percent Retained Partial	Percent Retained Total	Percent Finer by Weight	Specification Operating Range
3.00	3-in.					
2.00	2-in.					
1.5	1-1/2-in.					
1.00	1-in.					
0.750	3/4-in.					
0.500	1/2-in.					
0.375	3/8-in.					
0 .187	No. 4					
0.132	No. 6					
0.094	No. 8	2.3		0.5	99.5	
0.079	No. 10					
0.047	No. 16	7.3		1.6	98.4	
0.033	No. 20					
0.023	No. 30	77.9		16.7	83.3	
0.0165	No. 40					
0.0117	No. 50	245.2		52.6	47.4	
0.0098	No. 60					
0.0059	No. 100	330.4		70.8	29.2	
0.0041	No. 140					
0.0029	No. 200	366.3		78.5	21.5	
PAN						
TOTAL WEIGHT						

TECHNICIAN: D. Bailey

CHECKED BY: GDP

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SIEVE ANALYSIS

CLIENT: Turlock Irrigation District
PROJECT: Walnut Substation
LOCATION: 5.0' – 6.5'
DESCRIPTION: Sandy Silt (ML)

DATE: 06-21-00
PROJECT #: 3002
SAMPLE #: SB-3

TOTAL WEIGHT OF SAMPLE: 467.9

Sieve Openings (in.)	U.S. Standard Sieve #	Accumulated Weight Retained (gms)	Percent Retained Partial	Percent Retained Total	Percent Finer by Weight	Specification Operating Range
3.00	3-in.					
2.00	2-in.					
1.5	1-1/2-in.					
1.00	1-in.					
0.750	3/4-in.					
0.500	1/2-in.					
0.375	3/8-in.					
0 .187	No. 4					
0.132	No. 6					
0.094	No. 8	1.1		.20	99.8	
0.079	No. 10					
0.047	No. 16	19.4		4.1	95.9	
0.033	No. 20					
0.023	No. 30	75.9		16.2	83.8	
0.0165	No. 40					
0.0117	No. 50	137.1		29.3	70.7	
0.0098	No. 60					
0.0059	No. 100	183.3		39.2	60.8	
0.0041	No. 140					
0.0029	No. 200	226.4		48.4	51.6	
PAN						
TOTAL WEIGHT						

TECHNICIAN: D. Bailey **CHECKED BY:** GDP

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SIEVE ANALYSIS

CLIENT: Turlock Irrigation District
DATE: 06-21-00
PROJECT: Walnut Substation
PROJECT #: 3002
LOCATION: 10.0" – 10.5"
SAMPLE #: SB-6
DESCRIPTION: Poorly Graded Sand with Silt (SP-SM)

TOTAL WEIGHT OF SAMPLE: 477.2

Sieve Openings (in.)	U.S. Standard Sieve #	Accumulated Weight Retained (gms)	Percent Retained Partial	Percent Retained Total	Percent Finer by Weight	Specification Operating Range
3.00	3-in.					
2.00	2-in.					
1.5	1-1/2-in.					
1.00	1-in.					
0.750	3/4-in.					
0.500	1/2-in.					
0.375	3/8-in.					
0 .187	No. 4					
0.132	No. 6					
0.094	No. 8					
0.079	No. 10					
0.047	No. 16	1.9		0.40	99.6	
0.033	No. 20					
0.023	No. 30	41.2		8.6	91.4	
0.0165	No. 40					
0.0117	No. 50	188.5		39.5	60.5	
0.0098	No. 60					
0.0059	No. 100	354.4		74.3	25.7	
0.0041	No. 140					
0.0029	No. 200	435.3		91.2	8.8	
PAN						
TOTAL WEIGHT						

TECHNICIAN: D. Bailey
CHECKED BY: GDP

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SOIL COMPACTION TEST (ASTM D-1557-A)

Client: Turlock Irrigation District
Project: Washington Substation

Project No.: 3002
Sample No.: CT-1

Date Tested: June 15, 2000
Tested By: Eva Spens

ASTM D-1557-C:
Vol. Of Mold: 1/30.0 (ft^3)
Length of Drop: 18"
Blows Per Layer: 25

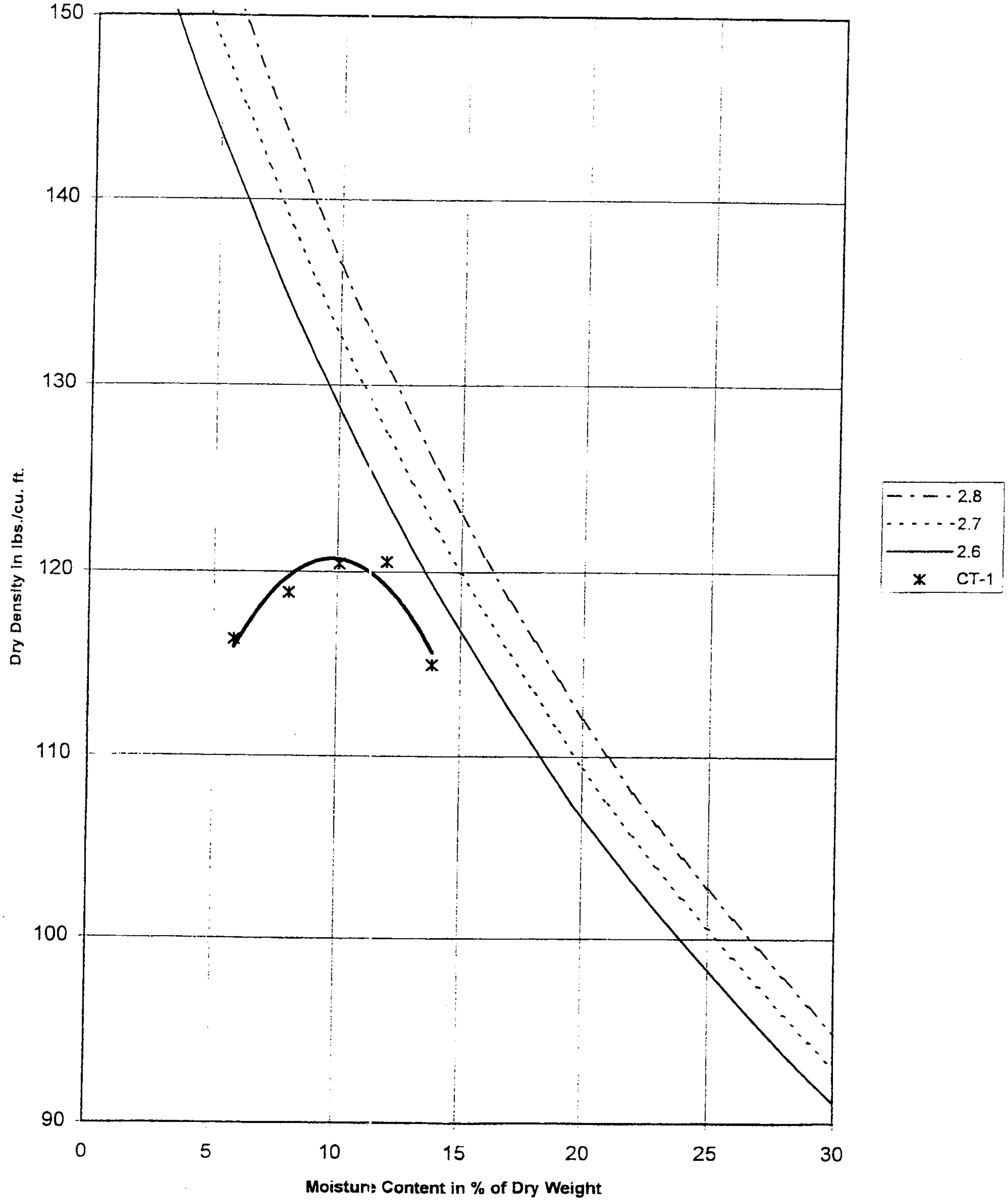
Weight of Hammer: 10 (lbs)
No. of Layers: 5

Soil Description:

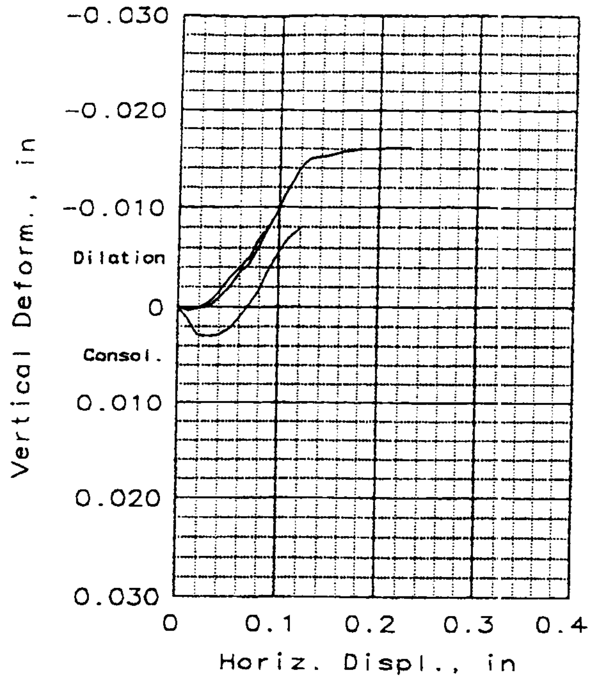
Can No.	AA	AS	AG	AS	AA
Can + Wet Weight (gms)	282.2	274.8	288.9	273.8	301.7
Can + Dry weight (gms)	273.9	264.1	274.3	258.5	280.9
Weight of Water (gms)	8.3	10.7	14.6	15.3	20.8
Weight of Can (gms)	130.9	130.9	129.6	130.9	130.9
Weight of Dry Soil (gms)	143.0	133.2	144.7	127.6	150.0
Water Content %	5.8	8.0	10.1	12.0	13.9
Weight of Soil + Mold (gms)	8.54	8.72	8.86	8.94	8.80
Weight of Mold (lbs)	4.44	4.44	4.44	4.44	4.44
Weight of Soil (lbs)	4.10	4.28	4.42	4.50	4.36
Wet Unit Weight Soil (pcf)	123.0	128.4	132.6	135.0	130.8
Dry Unit Weight Soil (pcf)	116.3	118.9	120.4	120.5	114.9

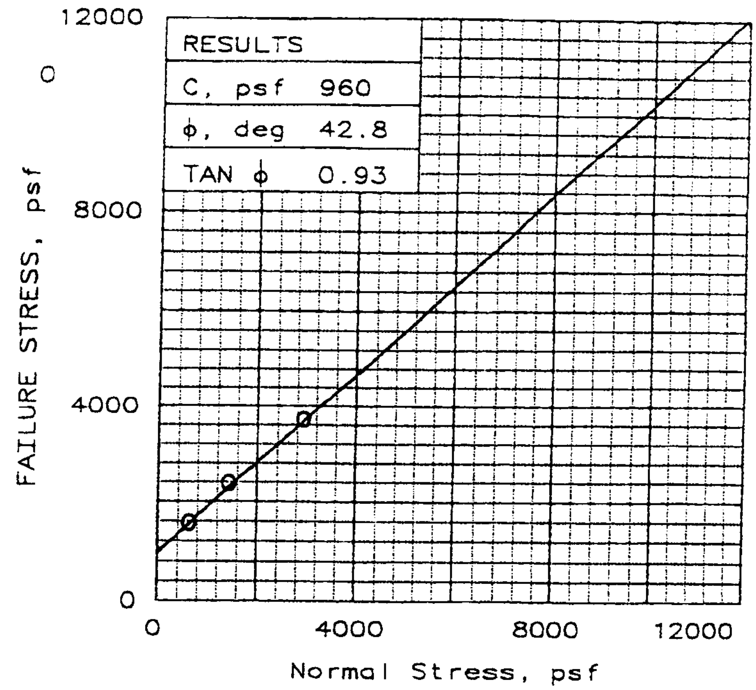
MAXIMUM DRY DENSITY	121.0	(pcf)
MAXIMUM DENSITY MOISTURE	11.0	%

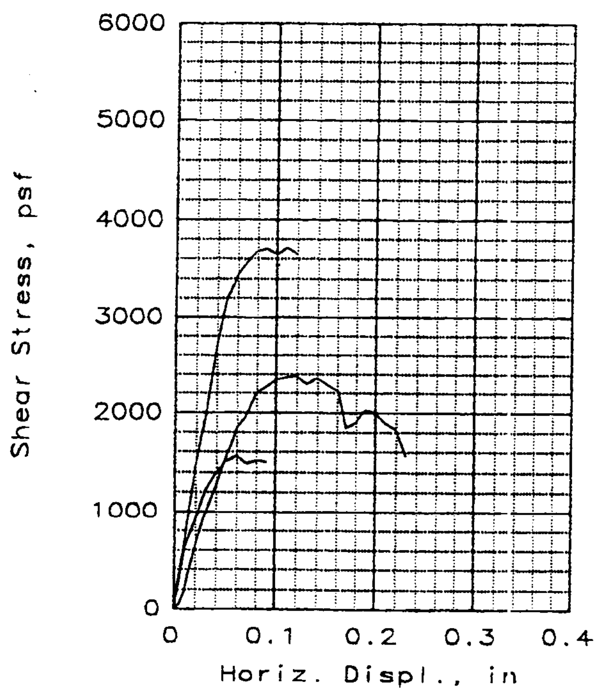
COMPACTION CURVE



MAX. DRY DENSITY 121.0 PCF @ 11.0% MOISTURE







	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	14.2	13.5	12.8
	DRY DENSITY, pcf	116.3	119.9	121.1
	SATURATION, %	85.6	89.6	88.0
	VOID RATIO	0.449	0.406	0.391
	DIAMETER, in	1.88	1.88	1.88
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	16.1	14.5	13.0
	DRY DENSITY, pcf	117.3	121.1	124.7
	SATURATION, %	99.7	100.0	99.7
	VOID RATIO	0.437	0.392	0.352
	DIAMETER, in	1.88	1.88	1.88
	HEIGHT, in	0.99	0.99	0.97
NORMAL STRESS, psf		700	1500	3000
FAILURE STRESS, psf		1571	2379	3714
DISPLACEMENT, in		0.06	0.12	0.11
ULTIMATE STRESS, psf				
DISPLACEMENT, in				
Strain rate, %/min		0.05	0.05	0.05

SAMPLE TYPE: undisturbed
DESCRIPTION: pending

SPECIFIC GRAVITY= 2.7
REMARKS: Condor# 3002

Fig. No.: 1

CLIENT: Condor Earth Technologies

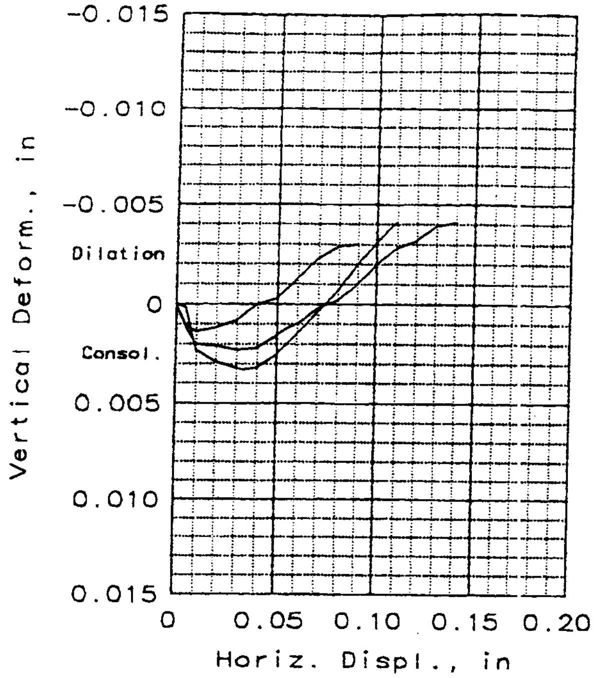
PROJECT: Walnut 115KV Substation

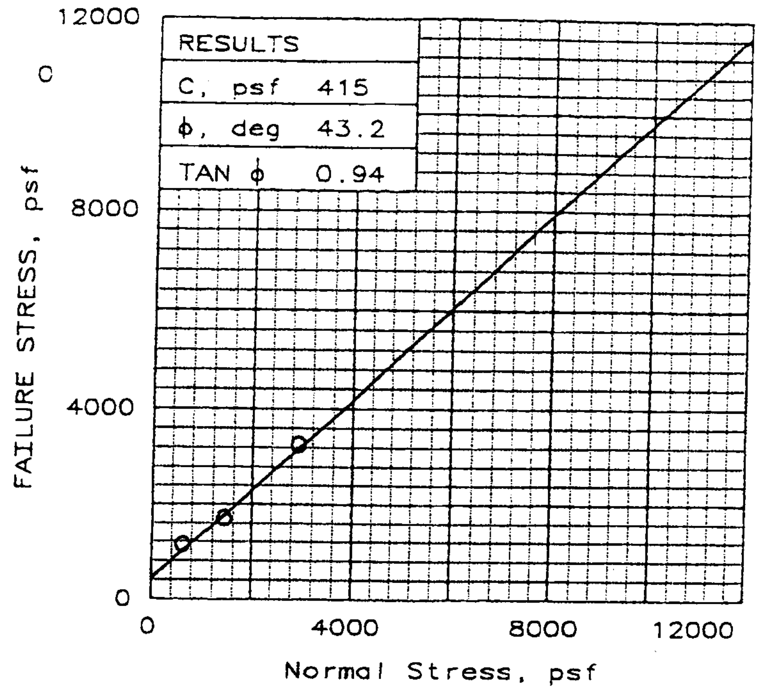
SAMPLE LOCATION: SB1@21.0'

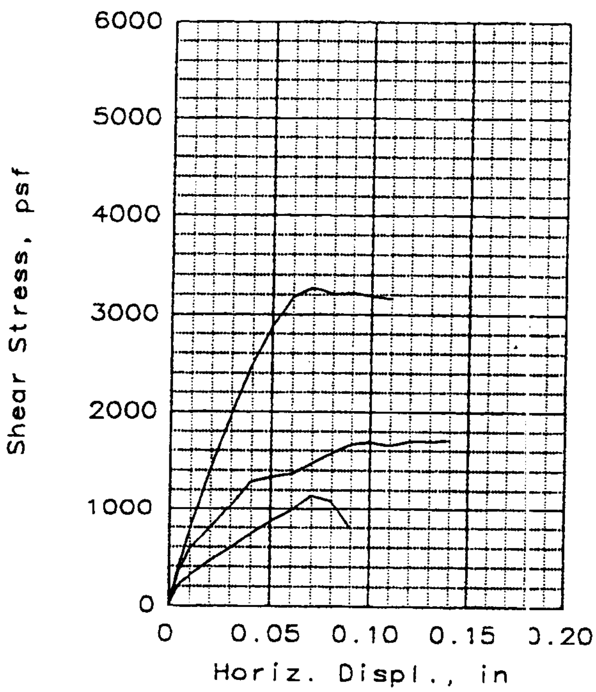
PROJ. NO.: 00-125 DATE: 6-26-2000

DIRECT SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.







	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	7.8	7.1	7.0
	DRY DENSITY, pcf	109.0	108.3	110.5
	SATURATION, %	38.8	34.6	35.9
	VOID RATIO	0.546	0.556	0.525
	DIAMETER, in	1.88	1.88	1.88
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	20.6	18.3	17.1
	DRY DENSITY, pcf	112.5	112.6	115.1
	SATURATION, %	111.7	99.6	99.5
	VOID RATIO	0.499	0.497	0.464
	DIAMETER, in	1.88	1.88	1.88
	HEIGHT, in	0.97	0.96	0.96
	NORMAL STRESS, psf	700	1500	3000
	FAILURE STRESS, psf	1144	1711	3269
	DISPLACEMENT, in	0.07	0.14	0.07
	ULTIMATE STRESS, psf			
	DISPLACEMENT, in			
	Strain rate, %/min	0.05	0.05	0.05

SAMPLE TYPE: undisturbed
DESCRIPTION: pending

SPECIFIC GRAVITY= 2.7
REMARKS: Condor#3002

Fig. No.: 2

CLIENT: Condor Earth Technologies

PROJECT: Walnut 115KV Substation

SAMPLE LOCATION: SB2@3.0'

PROJ. NO.: 00-125 DATE: 6-26-2000

DIRECT SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.

Table 1
Calculated Apparent Resistivities
Walnut 115KV Substation, June 2000

Station Number	A=1.0 ft ohm-m	A=1.5 ft ohm-m	A=2.5 ft ohm-m	A=3.5 ft ohm-m	A=5.5 ft ohm-m	A=7.5 ft ohm-m	Average ohm-m
GS1	49.4	39.7	33.5	30.4	29.3	29.1	35.2
GS2	43.4	38.9	30.5	30.3	34.3	35.2	35.4
GS3	45.4	35.8	35.2	35.6	35.6	35.3	37.2
GS4	39.6	29.6	29.9	24.6	28.8	31.5	30.7
GS5	68.3	59.8	55.0	40.8	31.0	32.6	47.9
GS6	257.0	124.4	118.3	90.0	67.4	53.2	118.4
GS7	106.5	73.7	52.8	42.5	35.9	33.0	57.4
GS8	53.3	46.6	44.7	34.9	33.6	32.3	40.9
GS9	139.7	128.5	119.2	94.6	47.1	50.0	96.5
					Cumulative Average =		55.5

Notes:

Ohm-m = ohm-meter

A = A-spacing distance between electrodes