APPENDIX C:
GEOTECHNICAL INVESTIGATION REPORT

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September 29, 2010
File No. 112664.GEO

Mr. Peter Lum
Pacific Gas and Electric System Engineering
1919 Webster Street, Room 493
Oakland, California 94612

Subject: Geotechnical Investigation Report
Proposed PG&E Shepherd Substation
Fresno County, California

Dear Mr. Lum:

Kleinfelder is pleased to present the results of our geotechnical services performed for the proposed project at the Pacific Gas and Electric Company (PG&E) Shepherd Substation located on North Sunnyside Avenue, in Fresno County, California.

The purpose of the geotechnical investigation was to evaluate the subsurface conditions at the site in order to provide geotechnical recommendations for design and construction of the proposed project. Based on the present information, it is Kleinfelder's professional opinion that the proposed site is geotechnically suitable for construction of the proposed project provided the recommendations presented in this report are incorporated into the project design.

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We appreciate the opportunity to be of service on this project. Please do not hesitate to contact the undersigned if you have any questions, comments, or require additional information.

Respectfully submitted,
KLEINFELDER WEST, INC.

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GEOTECHNICAL INVESTIGATION REPORT
PROPOSED PG&E SHEPHERD SUBSTATION
FRESNO COUNTY, CALIFORNIA

September 29, 2010

Prepared For:

Pacific Gas and Electric System Engineering
1919 Webster Street, Room 493
Oakland, California 94612

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1 INTRODUCTION

1.1 GENERAL

The proposed project site is located north of Shepherd Avenue and west of North Sunnyside Avenue in Fresno County, California. Kleinfelder West, Inc. (Kleinfelder) was retained by Pacific Gas and Electric System Engineering to provide geotechnical engineering services for the project. The proposed site is shown on Plate 1, Site Vicinity Map. The Site Plan, presented on Plate 2, shows the proposed substation and the approximate boring locations.

This report includes recommendations related to the geotechnical aspects of project design. Conclusions and recommendations presented in this report are based on subsurface conditions encountered at the locations of the exploration, as well as the provisions and requirements outlined in the "Additional Services" and "Limitations" sections of this report. Recommendations presented herein should not be extrapolated to other areas or used for other projects without prior review.

1.2 PROJECT DESCRIPTION

The proposed site for the Shepherd Substation consists of an existing almond orchard with mature trees spaced in narrow rows approximately 25 feet on center.

Equipment planned for this facility includes:

- 115kv standard dead ends and rigid ring bus conductor support structures supported with cast-in-drilled hole (CIDH) piers.
- Two to three 45 MVA transformers and circuit breakers supported on structural slab or mat foundations. The maximum transformer weight is about 200 kips.
- A switchgear enclosure structure supported on a continuous perimeter foundation.

A new spill prevention control and countermeasures (SPCC) pond will be excavated at the southwest corner of the site. The yard surface will have asphalt paved roadways. It is understood that typical facility roads consist of 2 inches of asphalt concrete over a 4-

inch thick compacted Class 2 aggregate base layer for the pavement section. Earthwork is anticipated to be performed to provide a relatively level substation pad and proper drainage. Other than for the SPCC pond, cuts and fills are expected to be less than about 2 feet in vertical height.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore and evaluate the site subsurface conditions in order to develop geotechnical recommendations for project design and construction.

The authorized scope of work for this study was outlined in our proposal dated June 16, 2010 (File No. FRE10P135) and included the following tasks:

- Drilling and sampling of five (5) exploratory borings to depths of about 31½ to 51½ feet below the ground surface;
- Excavating two (2) test pits to depths of about 6 and 10 feet below the ground surface and conducting two (2) double-ring infiltration (DRI) tests at the proposed SPCC pond,
- Geotechnical laboratory testing of soil samples collected from the borings;
- Engineering analyses to develop geotechnical recommendations, and;
- Preparation of this report.

This report addresses the following items:

- ❑ A description of the proposed project, including a vicinity map showing the location of the site and a site plan showing the locations of the explorations for this study;
- ❑ A description of the site surface and subsurface conditions encountered during the field investigation, including logs of borings and test pits;
- ❑ A summary of the field exploration and laboratory testing programs;
- ❑ Discussion of regional and local geology, including faults, seismicity and liquefaction potential, seismic settlement, and associated effects;

- Recommended 2007 CBC seismic design criteria;
- Recommendations for site preparation and earthwork;
- Recommendations for shallow foundation design, including available bearing capacity of foundation soil for sustained and total combined loading and anticipated settlement;
- Recommendations for resistance of lateral loads on shallow foundations;
- Recommendations for axial capacity design of CIDH piers;
- Geotechnical parameters for use in L-Pile lateral analysis of CIDH piers;
- Recommendations for temporary excavations including OSHA soil type and shoring recommendations, if appropriate;
- Comments on the infiltration rates for the SPCC pond;
- Comments on the corrosion potential of on-site soils to buried metal and concrete;
- Recommendations to aid in the design of site drainage; and,
- Recommendations for plan review, grading observations, and compaction testing.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration program, conducted on August 13, 2010, included advancement of five (5) test borings and excavation of two (2) test pits to facilitate infiltration testing. Three (3) of the borings were drilled in the planned equipment areas to depths of about 31½ feet below the ground surface (bgs). One (1) boring was drilled in the area of the planned dead end structure to a depth of about 51½ feet bgs. One (1) boring was advanced in the vicinity of the proposed SPCC pond to a depth of 31½ feet bgs. The borings were advanced utilizing a CME 55, truck-mounted drill rig using a hollow-stem auger. Two test pits were excavated to depths of approximately 6 and 10 feet bgs with a rubber-tired backhoe equipped with a 24-inch wide bucket. The approximate locations of the test borings and test pits are indicated on the Site Plan, Plate 2.

The soils encountered in the borings and test pits were visually classified in the field and continuous logs were recorded. Relatively undisturbed samples were collected from the borings at selected depths by driving a 2.5-inch inside diameter (I.D.) split barrel sampler containing brass liners into the undisturbed soil with a 140-pound automatic hammer free falling a distance of 30 inches. The 2.5-inch I.D. sampler is in general conformance with American Society of Testing Materials (ASTM) D3550. Relatively undisturbed soil samples may experience some minor disturbance due to hammer impact, retrieval, and handling. In addition, a 1.4-inch I.D. Standard Penetration Test (SPT) sampler was driven at selected depths in general accordance with ASTM D1586 test procedures. The SPT sampler was used without liners. Resistance to sampler penetration was noted as the number of blows over the last 12 inches of sampler penetration on the boring logs. The blow counts listed on the boring logs have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Correction factors were applied to estimate the sample relative density descriptions noted in the boring logs. The consistency terminology used in the soil descriptions is based on ASTM D2488. Bulk samples were also obtained from auger cuttings at the boring locations. Each soil sample was classified in accordance with the Unified Soil Classification (USCS) system presented in ASTM D 2487 and D2488. Logs of the borings are attached in Appendix A. At the completion of fieldwork, the borings and test pits were backfilled with the soil cuttings.

2.2 FIELD AND LABORATORY TESTING

Sampler penetration rates, determined in general accordance with ASTM D1586, were used as an aid in evaluating the relative density, compression, and strength characteristics of the foundation soils.

Two (2) double-ring infiltration (DRI) tests were performed during the field exploration program in general accordance with ASTM D3385. The DRI tests were performed within the test pits at depths of approximately 6 and 10 feet bgs. The DRI test results are summarized in Section 5.7 of this report.

Kleinfelder performed laboratory tests on selected samples collected from the borings to evaluate physical and engineering characteristics of the site soils. The following laboratory tests were used to develop the design geotechnical parameters included in this report:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Soluble Sulfate Content (California Test Method No. 417)
- Soluble Chloride Content (California Test Method No. 422)
- pH and Minimum Resistivity (California Test Method No. 643)
- Direct Shear (ASTM D3080)
- Plasticity Index (ASTM D4318)
- Material Passing No. 200 Sieve (ASTM D1140)
- Maximum Density/Optimum Moisture (ASTM D1557)

Test specimens for used for unit weight, moisture content, and direct shear tests were obtained from the 2.5-inch I.D. driven samples. Each test specimen was unique to the test performed. The dry density, moisture content, plasticity index, and material passing the No. 200 sieve test results are shown on the boring logs in Appendix A. The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 5.8, Corrosion Potential. The other test results are provided in Appendix B.

3 SITE AND SUBSURFACE CONDITIONS

3.1 SURFACE CONDITIONS

The proposed substation site is currently occupied by an existing almond orchard with narrow rows of trees, spaced approximately 25 feet on center. The site is located about a half mile north of the intersection of Sunnyside and Shepard Avenues. The site area measures approximately 466 by 441 feet in plan dimensions, and is relatively flat. The south and west sides of the property are bounded by an adjacent almond orchard. The east side of the site is bounded by Sunnyside Avenue. The north side is bounded by an open field with annual grasses. The current ground surface elevation at the site is approximately 383 to 384 feet above mean sea level, based on the project datum.

3.2 SUBSURFACE SOIL CONDITIONS

The earth materials encountered at the site are alluvial soil deposits consisting predominantly of medium dense silty sand extending to depths of about 8 to 27½ feet bgs which are underlain by discontinuous layers of stiff sandy lean clay and medium dense to dense clayey sand.

The preceding soil descriptions provide a general summary of the subsurface conditions encountered during the field exploration program. For more thorough descriptions of the actual conditions encountered at specific boring or test pit locations, refer to the boring logs presented in Appendix A (Plates A-3 through A-9).

3.3 GROUNDWATER

Groundwater was encountered in Boring B-4 at a depth of approximately 40½ feet bgs. The four shallower borings did not encounter groundwater. The State of California Department of Water Resources, "Lines of Equal Elevation of Water in Wells" Spring 2006, indicates the depth to groundwater in the project site vicinity to be on the order of 40 to 50 feet bgs. It is possible that groundwater conditions at the site could change at some time in the future due to variations in the rainfall, groundwater withdrawal or recharge, construction activities, or other factors not apparent at the time the test borings were explored. However, groundwater is presently not anticipated to effect design or construction.

4 GEOLOGIC CONDITIONS

4.1 REGIONAL GEOLOGY

The site is located in the eastern portion of the San Joaquin Valley in central California. The valley is a large northwestward trending, asymmetric structural trough that has been filled with as much as 6 vertical miles of sediment. The trough is situated between the Sierra Nevada Mountains on the east and the Coast Range Mountains on the west. Both of these mountain ranges were initially formed by uplifts that occurred during the Jurassic and Cretaceous periods of geologic time (greater than 65 million years ago). Renewed uplift began in the Sierra Nevada during late Tertiary time, and is continuing today.

4.2 AREA AND SITE GEOLOGY

The majority of the native sediments in the project area have been mapped (Fresno 2 degree geologic sheet) by the California Geological Survey (formerly Division of Mines and Geology, CDMG) as Holocene age alluvial fan deposits (Q_f).

4.3 LOCAL FAULTS

The site is located in a region traditionally characterized by low to moderate seismic activity, but with the potential for relatively high activity. The site is not in an Alquist-Priolo Earthquake Fault Zone and no known active faults traverse the site. The project site is located approximately 34 miles southwest of the Foothills Fault System, approximately 49 miles southwest of the Great Valley fault, and approximately 76 miles northeast of the San Andreas Fault. A major seismic event on these faults could cause ground shaking at the site.

4.4 SEISMICITY

4.4.1 Seismic Design Parameters

Seismic design information based upon the 2007 California Building Code (CBC) is presented below. The Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1) were estimated based on Section 1613 of the 2007 CBC using the Java calculator provided at the USGS National Seismic Hazards Mapping Program (NSHMP) website. The mapped acceleration values and associated soil amplification factors (F_a and F_v) based on the 2007 CBC are presented in Table 4.4-1 below. Corresponding site modified (S_{MS} and S_{M1}) and design spectral accelerations (S_{DS} and S_{D1}) are also presented in Table 4.4-1. The Site Class is D.

TABLE 4.4-1
SEISMIC DESIGN PARAMETERS

Parameter	Value	2007 CBC Reference
S_S	0.467g	Section 1613.5.1
S_1	0.211g	Section 1613.5.1
Site Class	D	Table 1613.5.2
Seismic Design Category	D	Table 1613.5.6(2)
F_a	1.427	Table 1613.5.3(1)
F_v	1.978	Table 1613.5.3(2)
S_{MS}	0.666g	Section 1613.5.3
S_{M1}	0.418g	Section 1613.5.3
S_{DS}	0.444g	Section 1613.5.4
S_{D1}	0.278g	Section 1613.5.4

The peak horizontal ground acceleration (PHGA) based on the Maximum Considered Earthquake (MCE) is 0.27g. The design earthquake has a PHGA of 0.18g.

4.4.2 Liquefaction

In order for soil liquefaction due to ground shaking to occur, it is generally accepted that four conditions will exist:

- The subsurface soils are in a relatively loose state,
- The soils are saturated,
- The soils have low plasticity, and

- Ground shaking is of sufficient intensity to act as a triggering mechanism.

Based on the depth of groundwater, relative density of the subsurface soils, and evaluation based on Youd et al (2001), the anticipated cyclic stress associated with the design PHGA (0.18g) is not likely sufficient to result in liquefaction or seismically induced settlement.

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. The subsurface conditions encountered in the borings advanced at the site are not generally considered conducive to such seismically induced ground deformation. Based on methods by Tokimatsu and Seed (1987), it is estimated no significant settlement (less than 0.2 inch) due to dynamic compaction would occur at the site during the design earthquake.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

The geotechnical conditions at the project site appear suitable for the proposed construction. It is anticipated that transformers and similar equipment will be founded on mat slab foundations. Reinforced concrete drilled piers are expected to be used to support the overhead switch gear and bus structures as well as dead end structures. Shallow spread foundations may be used to support auxiliary structures and control buildings, as necessary.

It is anticipated that site grading can be performed with conventional grading equipment and techniques. General recommendations regarding the geotechnical aspects of project earthwork are presented in subsequent sections of this report. All references to compaction, maximum density and optimum moisture are based on American Society of Testing and Materials (ASTM) Test Method D1557, unless otherwise noted.

5.2 SITE EARTHWORK

5.2.1 Stripping and Existing Tree Removal

All surface vegetation including existing trees should be removed along with their major root systems. This should include removal of all roots greater than ½ inch in diameter. The amount of soil lost or disturbed within tree removal areas will likely vary depending on the extent of root systems and the methods of removal.

To provide uniform support of proposed and future site improvements, it is recommended that soil disturbance from tree removal activities be mitigated by excavating to at least the depths of the major root systems (estimated at about 2 to 3 feet below existing grades) over the entire site. The intent is to enable compaction of all disturbed soils in a uniform manner across the site. Following removals, the exposed soils should be processed and compacted as recommended in Section 5.2.4 of this report. Excavated on-site soil can be reused as engineered fill provided it meets the criteria provided in Section 5.2.4.1. Organic materials, organic-laden soils, and debris are not suitable for use as engineered fill and should be removed from proposed improvement areas.

5.2.2 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

If not documented during clearing and demolition, initial site grading should include a reasonable search to locate soil disturbed by previous activities and tree removal, any undocumented fill soils and any abandoned underground structures, irrigation systems or utilities that may exist within the areas of construction. Any obstructions or deleterious materials should be removed from the project area. Special attention should be paid to potential irrigation systems on the property due to its past agricultural use. Any disturbed or loose soils, animal burrows, or undocumented fill encountered during grading should be over-excavated to expose firm native material.

5.2.3 Scarification and Compaction

After stripping and performing all necessary removals, exposed areas to receive fill should be scarified at least 8 inches below the exposed subgrade elevation. The subgrade soil should be uniformly moisture conditioned to slightly above the optimum moisture content and compacted to at least 90% of the maximum dry density.

5.2.4 Engineered Fill

5.2.4.1 Materials

All engineered fill soils should be free of organic materials, debris, or other deleterious materials and have a maximum particle size less than 3 inches in maximum dimension. Excavated on-site soil that is free of organic materials, debris, or other deleterious material, may be used as engineered fill.

Recommended requirements for imported engineered fill, as well as applicable test procedures to verify material suitability are provided in Table 5.2-1.

TABLE 5.2-1
ENGINEERED FILL REQUIREMENTS

Fill Requirement		Test Procedures	
		ASTM[1]	Caltrans[2]
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	C 136	202
¾ inch	70-100	C 136	202
No. 4	50-100	C 136	202
No. 200	20-70	C 136	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D 4318	204
Organic Content			
No visible organics		---	---
Expansion Index			
20 or less		D 4829	---
Corrosion Potential			
Soluble Sulfates	<2000 ppm	---	417
Soluble Chloride	<300 ppm	---	422
Resistivity	>2000 ohm-cm	---	643

[1]American Society for Testing and Materials Standards (latest edition)
[2]State of California, Department of Transportation, Standard Test Methods (latest edition)

Any imported fill materials to be used for engineered fill should be sampled and tested by the project Geotechnical Engineer prior to being transported to the site.

5.2.4.2 Compaction Criteria

Soils used for engineered fill should be uniformly moisture conditioned to slightly above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. Disking and/or blending will likely be required to uniformly moisture-condition soils used for engineered fill.

5.2.5 Construction Considerations

Should site grading be performed during or subsequent to wet weather, near-surface site soils may be significantly above the optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact site soils to the recommended compaction criteria. Disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile fabric or grid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork operations. The project Geotechnical Engineer should be consulted to provide specific recommendations for wet soil mitigation, if needed at the time of construction.

5.3 TEMPORARY EXCAVATIONS

5.3.1 General

All excavations must comply with applicable local, state, and federal safety regulations including the current the Occupational Safety & Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. Kleinfelder is providing the information below solely as a service to the client. Under no circumstances should the information provided be interpreted to mean that Kleinfelder is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

5.3.2 Temporary Excavations

The near surface soils encountered during the field investigation consisted predominantly of silty sand. These soils would likely be considered as Type B or C soils with regard to OSHA regulations.

5.4 SHALLOW FOUNDATIONS

5.4.1 General

The proposed transformers, inverters, and other equipment may be supported by mat foundations supported on engineered fill prepared as recommended herein. Other structures such as control buildings and similar structures may be supported by conventional shallow spread foundations bearing in engineered fill. Recommendations are provided below for design of mat slab and spread footing foundation systems. The following recommendations are based on the assumption that the recommendations in Section 5.2, "SITE EARTHWORK", have been implemented.

5.4.2 Spread Foundations

5.4.2.1 Allowable Bearing Pressures

We recommend spread footings constructed of reinforced concrete and founded on engineered fill be used for support of buildings and similar structures. Perimeter spread footings for buildings with interior floor slabs should be continuous. Interior column foundations may be continuous of isolated. Continuous footings should be a minimum of 12 inches wide and embedded a minimum of 12 inches below the lowest final adjacent subgrade[1]. Isolated footings should be a minimum of 24 inches wide and embedded a minimum of 12 inches below the lowest final adjacent subgrade. An allowable bearing pressure of 2,000 pounds per square foot (psf) may be used for design of spread foundations with the above minimum dimensions.

The allowable bearing pressure will vary with footing width and embedment. Therefore, the minimum allowable bearing pressure provided above may be increased by 500 psf for each additional foot of width and by 1,000 psf for each additional foot of embedment up to a maximum allowable bearing pressure of 4,000 psf.

[1] **Within this report, subgrade refers to the top surface of undisturbed native soil, native soil compacted during site preparation, or engineered fill.*

The allowable bearing pressure provided above is a net value. Therefore, the weight of the foundation (that extends below grade) may be neglected when computing footing contact pressures. The allowable bearing pressure applies to dead plus live loads, includes a calculated factor of safety of at least 3, and may be increased by ⅓ for short-term loading due to wind or seismic forces.

5.4.2.2 Estimated Settlements

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on anticipated foundation dimensions and loads, we estimate maximum settlement of foundations designed and constructed in accordance with the preceding recommendations to be on the order of ½-inch. Differential settlement between similarly loaded, adjacent footings is expected to about half of the total settlement provided footings are founded on similar materials (e.g., all on engineered fill, native soil). Settlement of all foundations is expected to occur rapidly and should be essentially complete shortly after initial application of the loads.

Footings may experience an overall loss in bearing capacity or an increased potential to settle where located in close proximity to existing or future utility trenches and/or foundations. Furthermore, stresses imposed by the footings on the utility lines may cause excessive cracking, collapse and/or a loss of serviceability. To reduce these risks, footings should extend below a 2(h): 1(v) plane projected upward from the closest bottom edge of the adjacent utility trench or foundations.

5.4.2.3 Construction Considerations

Prior to placing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by the project Geotechnical Engineer just prior to placing steel or concrete to verify the recommendations contained herein are implemented during construction.

5.4.3 Mat Foundations

Reinforced concrete mat foundations may be used to support transformers and various equipment. We anticipate mat slabs may be as large as about 20 by 40 feet in plan dimensions. We understand typical transformer weights are expected to be on the order of 700 to 800 kips.

5.4.3.1 Mat Foundation Subgrades

Subgrades to support mat foundations should be constructed as recommended in Section 5.2 of this report. We recommend mat slabs be underlain by at least 6 inches of Caltrans Class 2 aggregate base material. The material should be compacted to at least 95 percent relative compaction at a moisture content slightly above optimum.

5.4.3.2 Allowable Bearing Pressure

An allowable bearing pressure of 1,000 pounds per square foot (psf) should be used for design of mat slab foundations supported on engineered fill prepared as recommended in this report. If higher loads or larger mat slabs are needed than mentioned above, the allowable bearing pressure and anticipated settlement should be evaluated on a case-by-case basis.

The allowable bearing pressure provided above is a net value. Therefore, the weight of the foundation (that extends below grade) may be neglected when computing dead loads. The allowable bearing pressure applies to dead plus live loads, includes a calculated factor of safety of at least 3, and may be increased by 1/3 for short-term loading due to wind or seismic forces.

Mat foundations should have their bearing surfaces situated below or beyond an imaginary 2(h):1(v) plane projected upward from the nearest bottom edge of adjacent footings, parallel utility trenches, or other excavations.

5.4.3.3 Anticipated Settlement

Post construction settlement of mat foundations will be dependent on the slab dimensions and loadings. Settlement of mat foundations designed and supported as recommended herein is expected to be on the order of ¾ inch. Differential settlement between the outer edges and center of the slab is expected to be about half the total settlement.

5.4.3.4 Modulus of Subgrade Reaction

A modulus of subgrade reaction, K_{V1} = 250 pounds per cubic inch (based on a one square foot bearing plate) can be used for mat slab subgrades prepared as recommended in this report. The subgrade modulus is applicable for consideration of static loads with subgrade deformations within an elastic range.

5.4.4 Resistance to Lateral Loads on Shallow Foundations

Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottoms of concrete spread or mat foundations and the underlying soils, and by passive soil pressure acting against the sides of the foundations. An allowable coefficient of friction of 0.40 may be used between cast-in-place concrete foundations and the underlying soil. This value contains a factor of safety of approximately 2 and assumes good contact between a concrete foundation and the underlying soil. Passive pressure available in engineered fill may be taken as equivalent to the pressure exerted by a fluid weighing 350 pounds per cubic foot (pcf). Passive pressure should be neglected in the top one foot of soil unless confined by slabs or pavements. This value includes a factor of safety of approximately 1.5, which generally corresponds to a predicted lateral deflection of less than about ½ inch.

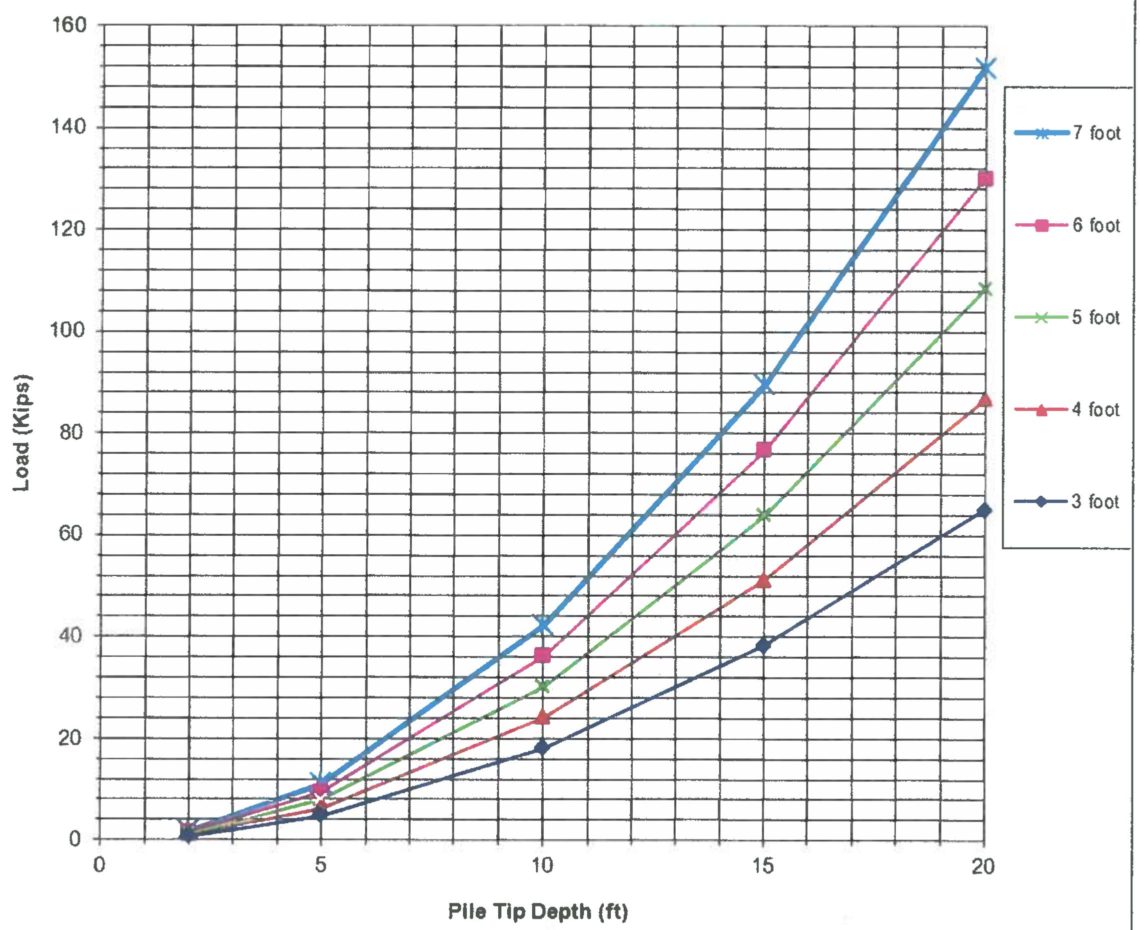
5.5 DRILLED PIER FOUNDATIONS

5.5.1 Axial Capacities

5.5.1.1 Static Loading

Cast-in-drilled-hole (CIDH) piers are considered appropriate for support of dead-end structures, towers, or other applicable structures. Axial loads imposed by the structures can be supported by the frictional capacity of the pier. Figure 5.5-1 presents the allowable downward capacity for 3, 4, 5, 6, and 7-foot diameter CIDH piers. The allowable downward (compressional) capacity may be increased by one third for the total of all loads, including wind. The uplift capacity of piers should be taken as 70% of the compressive frictional capacity plus the weight of the pier. A factor of safety of 3 was used on skin friction to develop the allowable downward capacity. End bearing was neglected due to strain incompatibility and construction issues.

Figure 5.5-1
Allowable Frictional Axial Capacity vs Pile Tip Depth



The frictional capacity (compression or uplift) is proportional to the pier diameter at a corresponding depth.

The total settlement of friction piers, designed in accordance with the above recommendations, should be about 0.002 times the pier diameter. The concrete mix and reinforcement for CIDH piers should be designed by the project structural engineer.

5.5.2 Lateral Capacity

The ability of reinforced concrete drilled piers to resist lateral loads applied at the tops of the piers can be evaluated using LPILE (computer software developed by Ensoft Inc.). The geotechnical parameters summarized in Table 5.5-1 are based on a generalized soil profile and can be used for evaluation of lateral loading of piles at the site.

TABLE 5.5-1
SOIL INPUT PARAMETERS FOR LPILE

Depth (ft)		p-y	γ'	ϕ	k
From	To	Curve	(pci)	(°)	(psi)
0	5	Sand	0.069	34	190
5	25	Sand	0.069	34	250

5.5.3 Construction Considerations

5.5.3.1 Anticipated Excavation Conditions

Based on the subsurface conditions encountered in the borings and the anticipated depths of the proposed drilled pier foundations (i.e., about 15 to 20 feet below the ground surface), caving of drilled pier excavations is not expected for piers that are less than about 4 feet in diameter. Larger diameter piers may be subject to caving especially within the upper silty sand soils. Groundwater levels are expected to be below the bottoms of the pier shafts. However, a clay layer exists at depths between about 8 and 27½ feet bgs that could trap infiltrating surface water seasonally and create a perched groundwater condition. This condition could cause caving of drilled pier excavations.

5.5.3.2 Groundwater and Caving

If perched groundwater or caving conditions are encountered in the pier holes, use of temporary casings or slurry drilling methods should be considered. Such pier drilling methods should be attempted only by experienced foundation drilling contractors. Otherwise, severe caving, loss of pier capacity, and other serious conditions could result.

We recommend steel reinforcement and concrete be placed on the same day of completion of each pier excavation. Additionally, drilled excavations should be scheduled to allow concrete in each pier to set over night before drilling adjacent holes that are closer than 4 diameters center-to-center.

Concrete used for pier construction should be discharged vertically into the drilled holes to reduce aggregate segregation. Under no circumstances during pile construction should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation.

In order to develop the design skin friction values provided above, concrete used for CIDH pile construction should have a slump ranging from 4 to 6 inches if placed in a dry shaft without temporary casing, and from 6 to 8 inches if temporary casing or slurry drilling methods are used. The concrete mix should be designed with appropriate admixtures and/or water/cement ratios to achieve these recommended slumps. Adding water to a conventional mix to achieve the recommended slump should not be allowed. For concrete mixes with slumps over 6 inches, vibration of the concrete during placement is generally not recommended as aggregate settlement may result in the lack of aggregate within the upper portion of the pier. Careful vibration of the concrete around anchor bolt assemblies is recommended.

Groundwater was not encountered during our explorations on this site. However, if more than 6 inches of water or drilling fluids are present in the pier excavations during concrete placement, concrete should be placed into the hole using tremie methods. Tremie concrete placement should be performed in accordance with ACI 304R. The tremie pipe should be rigid and remain several feet below the surface of the in-place concrete at all times to maintain a seal between the water or slurry and the fresh concrete. The upper concrete seal layer will likely become contaminated with excess water and soil as the concrete is placed and should be removed to expose uncontaminated concrete during or immediately following completion of concrete placement. It has been our experience that the concrete seal layer may be on the order of 3 to 5 feet in thickness but will depend on the pile diameter, amount of water seepage, and construction workmanship.

We recommend concrete used for tremie construction have a slump of 6 to 8 inches and a minimum cement content of 6 sacks per cubic yard. The concrete mix should be designed with an appropriate water/cement ratio for the design strength and use water reducing/plasticizing admixtures to achieve the recommended slump. Adding water to a conventional mix to achieve the recommended slump should not be allowed. Vibration of pier concrete under water during placement is generally not recommended as it may result in contamination of the concrete and/or cause aggregate settlement within the pier. Careful vibration of the tops of the piles following removal of the seal layer is recommended to consolidate the concrete around anchor bolt assemblies.

5.5.4 Construction Observation

The allowable axial capacity provided in Figure 5.5-1 is based on the frictional capacity of the soil and no end-bearing component. As such, inspection of the pile bottom is not required.

Consistent with Chapter 17 of the 2006 IBC/2007 CBC, CIDH pier borings should be inspected and approved by the geotechnical engineer prior to installation of reinforcement. Concrete placement by pumping or tremie tube to the bottoms of the pier borings is recommended. Sufficient space should be provided in the pier reinforcement cage during fabrication to allow the insertion of a pump hose or tremie tube for concrete placement. The pier reinforcement cage should be installed and the concrete pumped immediately after drilling is completed.

5.6 SITE PAVEMENTS

Design of pavement sections was not a part our scope of work for this project. However, a typical pavement section has been provided by PG&E for evaluation. The section includes 2 inches of asphalt concrete over 4 inches of Caltrans Class 2 aggregate base material. It is our opinion that this pavement section is minimal for light automobile traffic and is not adequate to support typical PG&E service vehicles and line trucks without excessive cracking and surface deformation.

In equipment access areas, consideration should be given to the use of thicker asphalt concrete and aggregate base sections. Alternatively, unpaved access ways could be constructed using at least 12 inches of Caltrans Class 2 aggregate base material underlain by a stabilization fabric (Mirafi 500X or equal). The subgrade soils and aggregate base materials should be prepared as recommended in Section 5.2 of this report, and be compacted to at least 95 percent relative compaction at a moisture content slightly above optimum. The stabilization fabric should be laid out and overlapped in accordance with the manufacturer's instructions.

If desired, Kleinfelder can evaluate appropriate pavement sections for various design lives based on the anticipated vehicle loading conditions.

5.6.1 Construction Considerations

In the event unstable (pumping) subgrades are encountered within planned pavement areas, it is recommended a heavy, rubber-tired vehicle (typically a loaded water truck) be used to test the load/deflection characteristics of the finished subgrade materials. It is recommended this vehicle have a minimum rear axle load (at the time of testing) of 16,000 pounds with tires inflated to at least 65-psi pressure. If the tested surface shows a visible deflection extending more than about 6 inches from the wheel track at the time of loading, or a visible crack remains after loading, corrective measures should be implemented. Such measures could include disking to aerate, chemical treatment, replacement with drier material, or other methods. It is recommended Kleinfelder be retained to assist in developing which method (or methods) would be applicable for this project.

5.7 INFILTRATION TESTS

Data collected from DRI tests conducted in the two test pits in the area of the proposed SPCC pond are presented in Table 5.7-1. No factors of safety have been applied. Infiltration tests were performed in general accordance to ASTM D3385.

TABLE 5.7-1
INFILTRATION TEST RESULTS

Location	Depth (feet)	Soil Type	Percolation Rate	Infiltration Rate
			min / inch	feet / day
TP-1	6	SM	74	1.6
TP-2	10	SM	74	1.6

Field exploration performed in the area of the basin encountered fine to coarse grained silty sand material in the upper 20 feet with trace amounts of clay. More detailed descriptions of the subsurface soils encountered are shown on the boring log for Boring B-5 on Plate A-7 in Appendix A.

The small scale testing from the double-ring infiltrometer cannot model the effects that interbedded soil layering has on large area pond infiltration. In using the double-ring data to estimate large area infiltration, it is necessary to apply some type of reduction factor, which is usually based on observation and/or water level drop measurements from large area ponds. For example, the EPA suggests using 2 to 4 percent of the small scale test result. Recent testing at some 30-acre ponds provided similar relationships (3.2%) between double-ring tests and drop in measurements.

Pond maintenance procedures should consider skimming and removal of any sediment build-up. Such an approach will tend to optimize infiltration. Bottom disking and/or ripping will tend to gradually increase fines content of the bottom soil and likely lead to long-term reduction of infiltration rates.

5.8 CORROSION POTENTIAL

Chemical analyses were performed on a sample of near surface soils to estimate pH, minimum resistivity, soluble sulfate content, and soluble chloride content in general accordance with Caltrans Standard Test Methods No. 643 (pH and resistivity), No. 417 (sulfates), and No. 422 (chlorides). The results of the corrosivity testing are provided in Table 5.8-1.

TABLE 5.8-1
CORROSION TEST RESULTS

Sample ID	Sulfates (ppm)	Chloride (ppm)	pH	Minimum Resistivity (ohm-cm)
B-3@0-5 feet	5.5	41.9	7.3	6,000

The test results suggest that relatively low levels of soluble sulfate content and low levels of soluble chloride content are present in on-site soils. Normal Type II cement is anticipated to be adequate in foundation concrete that comes in contact with the foundation soils.

The minimum electrical resistivity is generally representative of an environment that could be moderately corrosive to buried, unprotected metals. Corrosion is dependent upon a complex variety of conditions, which are beyond typical geotechnical engineering practice. Kleinfelder does not practice corrosion engineering. It is recommended that a competent corrosion engineer evaluate the corrosion potential of the site to the proposed project, to recommend further testing as required, and to provide specific corrosion mitigation methods appropriate for the project. It is recommended that specific testing be performed once site-grading activities are near completion to provide a better assessment of the actual soils present in the areas of interest.

It should be noted that the resistivity is a minimum value associated with potential future moisture increases. The value noted is not appropriate for use in evaluating any potential grounding system.

5.9 SITE DRAINAGE AND MOISTURE PROTECTION

Foundation, concrete slab, and pavement performance depends greatly on how well runoff waters drain from the site. This drainage should be maintained both during construction and over the entire life of the project. The ground surface around structures should be graded so that water flows rapidly away from structures without ponding. The surface gradient needed to do this depends on the surface conditions (i.e., surfacing, landscaping, pavements, etc.).

In general, the elevation of exterior grades should not be higher than the elevation of the subgrade beneath interior floor slabs to help prevent water intrusion beneath them. All utility trenches that pass beneath perimeter building foundations should be backfilled with compacted non-pervious fill material or a lean concrete trench plug to reduce the potential for external water to migrate beneath the building through the utility trenches. Special care should be taken during installation of sub-floor water and sewer lines to reduce the possibility of leaks.

6 ADDITIONAL SERVICES

6.1 PLANS AND SPECIFICATION REVIEW

It is recommended Kleinfelder be retained to review preliminary foundation and earthwork plans and specifications. It has been Kleinfelder's experience that this service provides an opportunity to review whether or not the recommendations have been properly interpreted and to correct possible misunderstandings of the recommendations prior to the start of construction. In the event Kleinfelder is not retained to perform this recommended review, Kleinfelder will assume no responsibility for misinterpretation of the recommendations.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that Kleinfelder be retained to provide observation and testing services during site earthwork and construction of foundations. This will allow us the opportunity to compare actual subsurface soil conditions with those encountered during the field exploration and, if necessary, to provide supplemental recommendations, if warranted due to unanticipated subsurface conditions.

7 LIMITATIONS

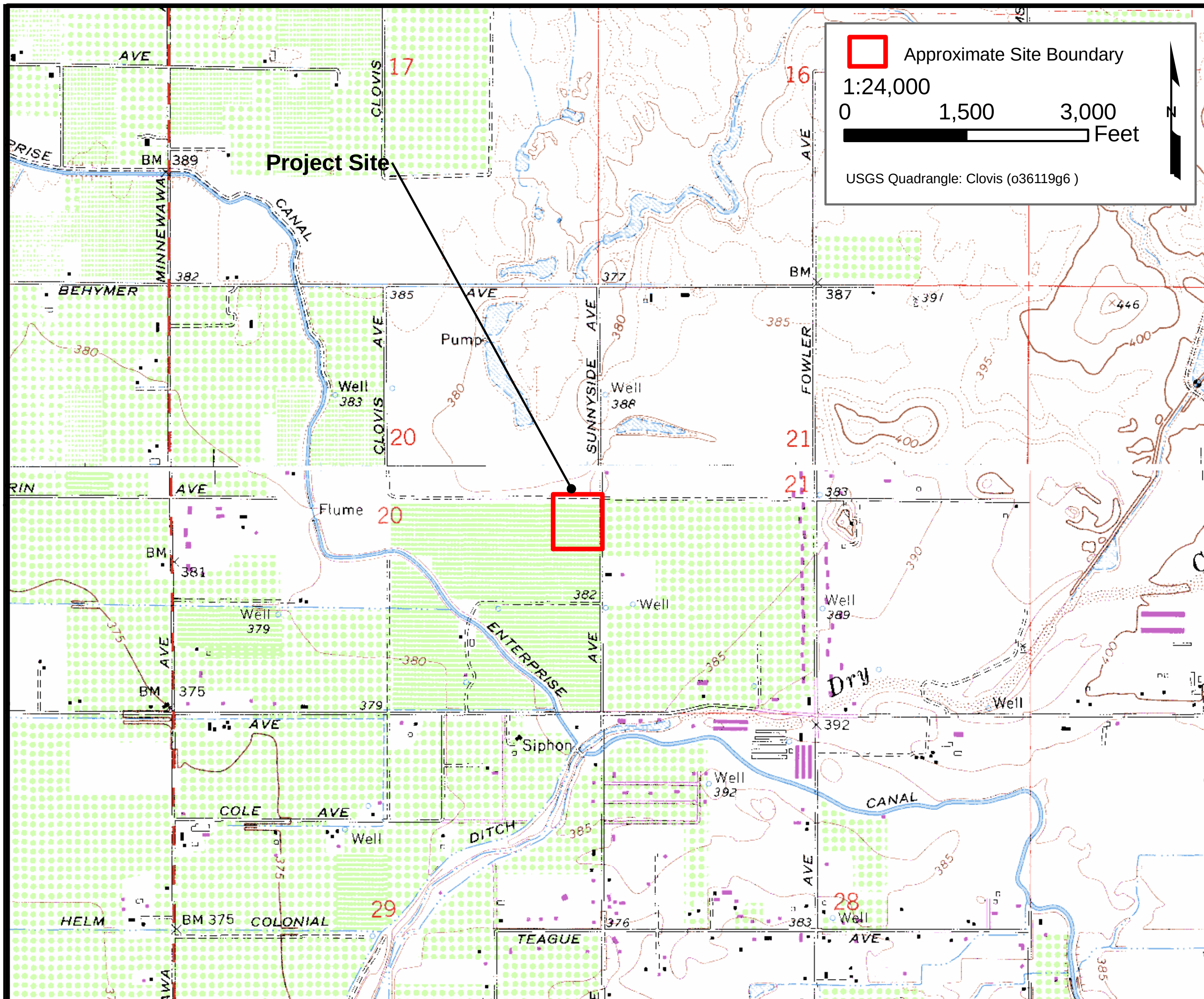
Recommendations contained in this report are based on the field observations, subsurface explorations, laboratory tests, and present knowledge of the proposed construction, as described in this report. It is possible that soil conditions vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, Kleinfelder should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction changes from that described in this report, the recommendations should also be reviewed. Kleinfelder has not reviewed the final grading plans or foundation plans for the project.

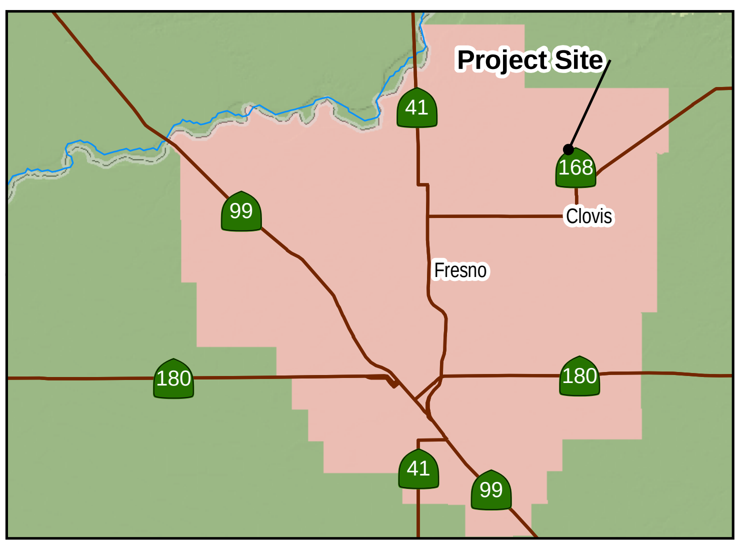
Corrosion recommendations are preliminary. Kleinfelder is not a corrosion engineering consultant. Specific recommendations for corrosion protection should be obtained from a corrosion specialist.

Kleinfelder has strived to present the findings, conclusions and recommendations in this report in a manner consistent with the standards of care and skill ordinarily exercised by members of this profession practicing under similar conditions in Fresno County, California, and at the time the services were performed. No warranty, express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by Kleinfelder during Project construction in order to evaluate compliance with the recommendations and/or to provide supplemental recommendations, as needed, if anticipated subsurface conditions are encountered.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year (without review) from the date of the report. Land use, site conditions or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party, and client agrees to defend, indemnify, and hold harmless Kleinfelder from any claim or liability associated with such unauthorized use or non-compliance.

The scope of the geotechnical services did not include any environmental site assessment for the presence or absence of hazardous/toxic materials. Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials.





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PROJECT NO.	112664
DRAWN:	09/17/10
DRAWN BY:	V.Oceguera
CHECKED BY:	R.Shilee
FILE NAME:	112664_P1_09/17/10

Site Vicinity Map

PG&E Substation
Shepherd Avenue and Sunnyside Avenue
Clovis, California

PLATE

1

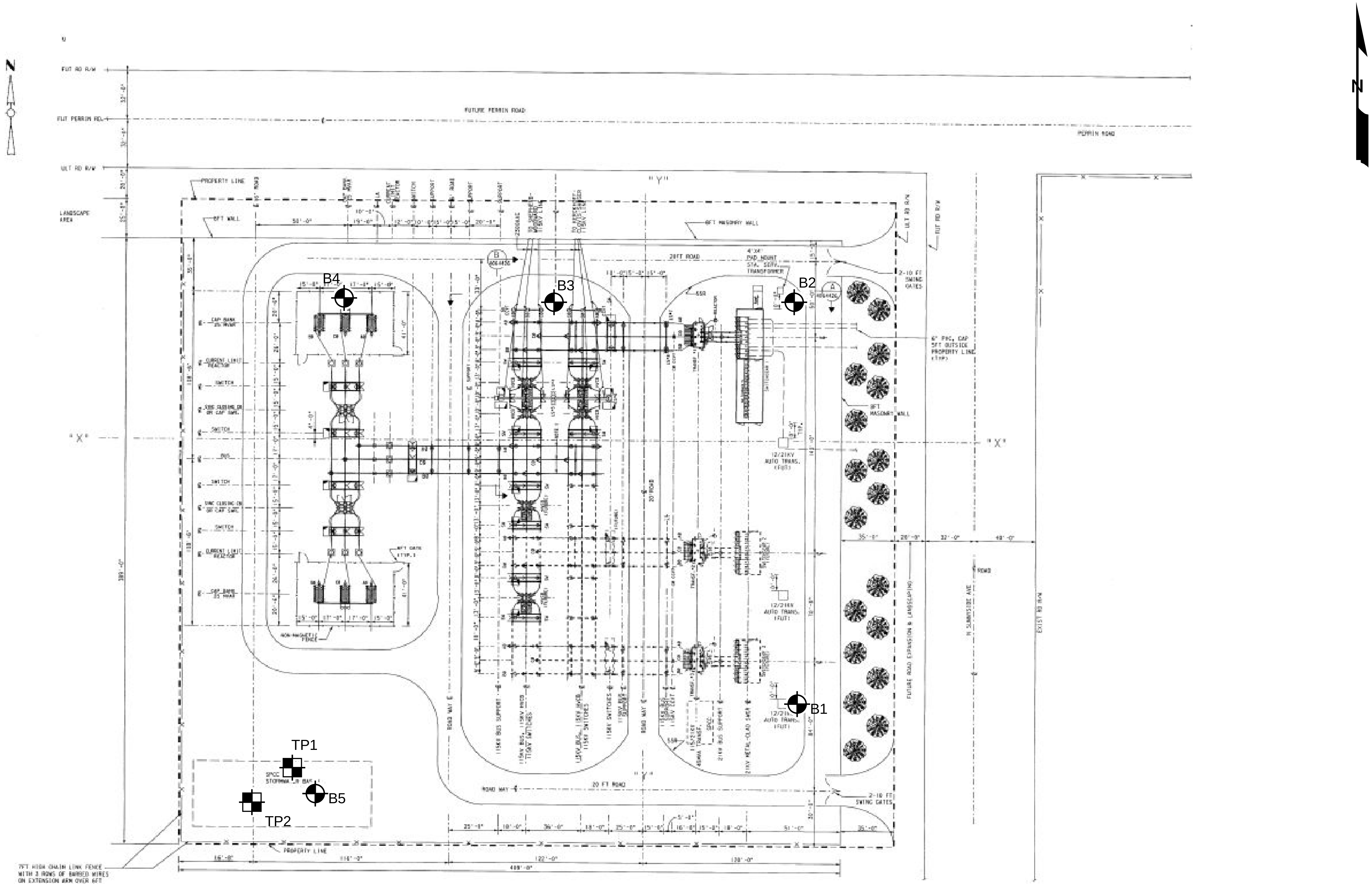

Image Source: PG&E

Approximate Boring Location

Approximate Test Pit Location

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PROJECT NO.	112664
DRAWN:	09/17/10
DRAWN BY:	V.Oceguera
CHECKED BY:	R.Shiplee
FILE NAME:	112664_P2_091710

Site Plan

PG&E Substation
Shepherd Avenue and Sunnyside Avenue
Clovis, California

PLATE

2

LOG SYMBOLS

	BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
	MODIFIED CALIFORNIA SAMPLER (2 1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
	CALIFORNIA SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
	NX SIZE CORE BARREL	DS	DIRECT SHEAR (ASTM Test Method D 3080)
		COL	COLLAPSE POTENTIAL
	CONTINUOUS SAMPLER (3 inch outside diameter)	UC	UNCONFINED COMPRESSION
▼	WATER LEVEL (level after completion)	MC	MOISTURE CONTENT
▽	WATER LEVEL (level where first encountered)	NFGWE	NO FREE GROUND WATER ENCOUNTERED
	SEEPAGE		

GENERAL NOTES

1. **Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.**
2. **No warranty is provided as to the continuity of soil conditions between individual sample locations.**
3. **Logs represent general soil conditions observed at the point of exploration on the date indicated.**
4. **In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.**
5. **A temporary benckmark for relative elevation was located at:**

KEYLOGG 112664.GPJ 9/23/10

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Drafted By: Date:	Project No.: **112664** File Number:	**LOG KEY** **PG&E SHEPHERD SUBSTATION** **SHEPHERD AND SUNNYSIDE AVENUES** **FRESNO COUNTY, CALIFORNIA**	**PLATE** **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

MAJOR DIVISIONS				GRAPHIC LOG	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/or 1>Cc>3	GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5 to 12% FINES	Cu ≥4 and 1≤ Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/or 1>Cc>3	GP-GM	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH >12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/or 1>Cc>3	SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5 to 12% FINES	Cu ≥6 and 1≤ Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/or 1>Cc>3	SP-SM	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH >12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)			ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY,
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

USCS (2487) 112664.GPJ KLEINFELDER FRESNO.GDT 9/23/10

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Drafted By: R. Shiplee
Date: 9/23/2010
Project No.: 112664
File Number:

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
A-2

Surface Conditions: Flat Bare Soil

Groundwater: No free groundwater encountered.

Method: Hollow Stem Auger

Equipment: CME 45

Date Completed: 8/13/2010

Logged By: R. Shiplee

Total Depth: 31.5 feet

Boring Diameter: 6" H.S.

Depth (feet)	Sample Type	Sample No.	Blows/foot	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Graphic Log	DESCRIPTION
	FIELD				LABORATORY								
0													SILTY SAND (SM) - dark brown, moist, dense, fine to medium grained, some cementation.
2			46		70.0	8.0							
5			7										... medium dense, sample contains rootlets, no cementation.
10			56/6"		122.3	8.2							... red brown, moist, very dense, fine to coarse.
15			27										... medium dense, trace clay, fine grained.
17													SANDY LEAN CLAY (CL) - red brown, moist, very stiff, fine grained sand.
20			49							57			
25			20										
28													SILTY SAND (SM) - red brown to gray, moist, medium dense, fine to coarse grained.
30			15										
31.5													**Notes:** **1.) Bottom of boring at 31.5 feet. 2.) No free groundwater encountered. 3.) Boring backfilled with soil cuttings 8/13/2010.**
35													

P-LOG_2006 BLOWS PER FOOT 112664.GPJ KA_2008_SAC.GDT 9/23/10

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Drafted By:
Date: 9/23/2010

Project No.: 112664
File Number:

LOG OF BORING B-1
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
1 of 1
A3

Surface Conditions:	Flat Bare Soil	Date Completed:	8/13/2010
Groundwater:	No free groundwater encountered.	Logged By:	R. Shiplee
		Total Depth:	31.5 feet
Method:	Hollow Stem Auger	Boring Diameter:	6" H.S.
Equipment:	CME 45		

Depth (feet)	Sample Type	Sample No.	Blows/foot (FIELD)	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Graphic Log	DESCRIPTION
0													**SILTY SAND (SM) -** red brown, moist, medium dense, fine to medium grained.
			3										
5			45		121.3	11.7							... fine to coarse grained, dense.
10			36										
15			79		123.3	14.1							... very dense, trace clay.
18													**CLAYEY SAND (SC) -** red brown, moist, medium dense, fine to coarse grained sand.
20			20										
24													**CLAYEY SILT with SAND (ML) -** gray, moist, firm, fine sand.
25			46										
28.5													**SILTY SAND (SM) -** brown, moist, dense, fine to coarse grained.
30			32										
31.5													**Notes:** **1.) Bottom of boring at 31.5 feet. 2.) No free groundwater encountered. 3.) Boring backfilled with soil cuttings 8/13/2010.**
35													

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Drafted By:
Date: 9/23/2010
Project No.: 112664
File Number:

LOG OF BORING B-2
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
1 of 1
A4

P-LOG_2006 BLOWS PER FOOT 112664.GPJ KA_2008_SAC.GDT 9/23/10

Surface Conditions:	Flat Bare Soil	Date Completed:	8/13/2010
		Logged By:	R. Shiplee
Groundwater:	No free groundwater encountered.	Total Depth:	31.5 feet
Method:	Hollow Stem Auger	Boring Diameter:	6" H.S.
Equipment:	CME 45		

Depth (feet)	Sample Type	Sample No.	Blows/foot	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Graphic Log	DESCRIPTION
FIELD					LABORATORY								
0													**SILTY SAND (SM) -** red brown, moist, medium dense, fine to coarse grained, trace clay with rootlets.
2			12		120.8	11.4							
5			16										... no rootlets, no clay, some cementation.
10			65		112.1	16.4							... very dense.
15			14										... brown, medium dense, fine grained.
20			60										... red brown, moist, very dense, trace clay.
25			30										... dense.
28													**CLAYEY SAND (SC) -** red brown, moist, very dense, fine to coarse grained.
30			50/6"										
35													**Notes:** **1.) Bottom of boring at 31.5 feet.** **2.) No free groundwater encountered.** **3.) Boring backfilled with soil cuttings 8/13/2010.**

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LOG OF BORING B-3
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
1 of 1
A5

Drafted By:
Date: 9/23/2010
Project No.: 112664
File Number:

Surface Conditions: Flat Bare Soil

Groundwater: Free groundwater encountered at 40.5 feet.

Method: Hollow Stem Auger

Equipment: CME 45

Date Completed: 8/13/2010

Logged By: R. Shiplee

Total Depth: 51.5 feet

Boring Diameter: 6" H.S.

Depth (feet)	FIELD: Sample Type	Sample No.	Blows/foot	Pocket Penetrometer (tsf)	LABORATORY: Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Graphic Log	DESCRIPTION
													SILTY SAND (SM) - dark brown, moist, loose, fine to medium grained.
			2										
5			4		110.1	16.2							... fine to coarse grained.
													SANDY LEAN CLAY (CL) - red brown, moist, firm, fine to coarse grained.
10			15										
15			31		118.5	14.0							
20			13				44	28					
													CLAYEY SAND (SC) - red brown, moist, dense, fine to coarse grained.
25			33		117.0	14.3							
30			26										**SANDY LEAN CLAY (CL)** - red brown, moist, firm, fine to medium grained.
35													

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Drafted By:
Date: 9/23/2010

Project No.: 112664
File Number:

LOG OF BORING B-4
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
1 of 2
A6

P-LOG_2006 BLOWS PER FOOT 112664.GPJ KA_2008_SAC.GDT 9/23/10

Depth (feet)	Sample Type	Sample No.	Blows/foot	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Graphic Log	DESCRIPTION
			22		114.3	16.3							...increased sand content.
40			13				27	9		28			**CLAYEY SAND (SC)** - red brown, wet, medium dense, fine to coarse grained.
45			33		66.6	55.5							**SANDY LEAN CLAY (CL)** - gray, wet, firm to hard, fine grained.
50			50/6"										... red brown, fine to coarse grained.
55													
60													
65													
70													
75													

Notes:
1.) Bottom of boring at 51.5 feet.
2.) Free groundwater encountered at 40.5 feet.
3.) Boring backfilled with soil cuttings 8/13/2010.

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Drafted By:
Date: 9/23/2010
Project No.: 112664
File Number:

LOG OF BORING B-4
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE

A6

P-LOG_2006 BLOWS PER FOOT 112664.GPJ KA_2008_SAC.GDT 9/23/10

Surface Conditions:	Flat Bare Soil	Date Completed:	8/13/2010
Groundwater:	No free groundwater encountered.	Logged By:	R. Shiplee
Method:	Hollow Stem Auger	Total Depth:	31.5 feet
Equipment:	CME 45	Boring Diameter:	6" H.S.

Depth (feet)	FIELD: Sample Type	Sample No.	Blows/foot	Pocket Penetrometer (tsf)	LABORATORY: Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Graphic Log	DESCRIPTION
0													**SILTY SAND (SM) -** red brown, moist, medium dense, fine to coarse grained.
2			13		117.4	6.5					Phi =34 deg C =100 psf		
5			2										... loose.
10			72		126.1	10.5							... very dense, trace clay.
15			19										... brown, medium dense, no clay.
20			36										... dense, decreased content of fines.
25			17										... red brown, medium dense.
28													**SANDY LEAN CLAY (SC) -** red brown, moist, firm, fine sand.
30			31										
35													**Notes:** **1.) Bottom of boring at 31.5 feet.** **2.) No free groundwater encountered.** **3.) Boring backfilled with soil cuttings 8/13/2010.**

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Date: 9/23/2010
Project No.: 112664
File Number:

LOG OF BORING B-5
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
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P-LOG_2006 BLOWS PER FOOT 112664.GPJ KA_2008_SAC.GDT 9/23/10

Surface Conditions:	Flat Bare Soil	Date Completed:	8/18/2010
Groundwater:	No free groundwater encountered.	Logged By:	M.Shubert
Method:		Total Depth:	6.0 feet
Equipment:		Boring Diameter:	

Depth (feet)	FIELD: Sample Type	FIELD: Sample No.	FIELD: Blows/foot	FIELD: Pocket Penetrometer (tsf)	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Liquid Limit	LABORATORY: Plasticity Index	LABORATORY: Passing #4 Sieve (%)	LABORATORY: Passing #200 Sieve (%)	LABORATORY: Other Tests	Graphic Log	DESCRIPTION
													SILTY SAND (SM) - red brown, moist, medium dense, fine to coarse grained.
5													... trace of clay, moderate cementation.
													Notes: **1.) Bottom of test pit at 6 feet.** **2.) No free groundwater encountered.** **3.) Test pit backfilled with soil cuttings 8/18/2010.**
10													
15													
20													

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Drafted By:
Date: 9/27/2010
Project No.: 112664
File Number:

LOG OF BORING TP-1
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
1 of 1
A-8

P-LOG_2006 BLOWS PER FOOT 112664.GPJ KA_2008_SAC.GDT 9/27/10

Surface Conditions:	Flat Bare Soil	Date Completed:	8/18/2010
Groundwater:	No free groundwater encountered.	Logged By:	M.Shubert
		Total Depth:	10.0 feet
Method:		Boring Diameter:	
Equipment:			

Depth (feet)	FIELD: Sample Type	FIELD: Sample No.	FIELD: Blows/foot	FIELD: Pocket Penetrometer (tsf)	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Liquid Limit	LABORATORY: Plasticity Index	LABORATORY: Passing #4 Sieve (%)	LABORATORY: Passing #200 Sieve (%)	LABORATORY: Other Tests	Graphic Log	DESCRIPTION
0													**SILTY SAND (SM)** - red brown, moist, medium dense, fine to coarse grained.
5													... trace of clay, moderate cementation.
													... weak cementation.
10													**Notes:** **1.) Bottom of test pit at 10 feet.** **2.) No free groundwater encountered.** **3.) Test pit backfilled with soil cuttings 8/18/2010.**
15													
20													

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Drafted By:
Date: 9/27/2010
Project No.: 112664
File Number:

LOG OF BORING TP-2
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
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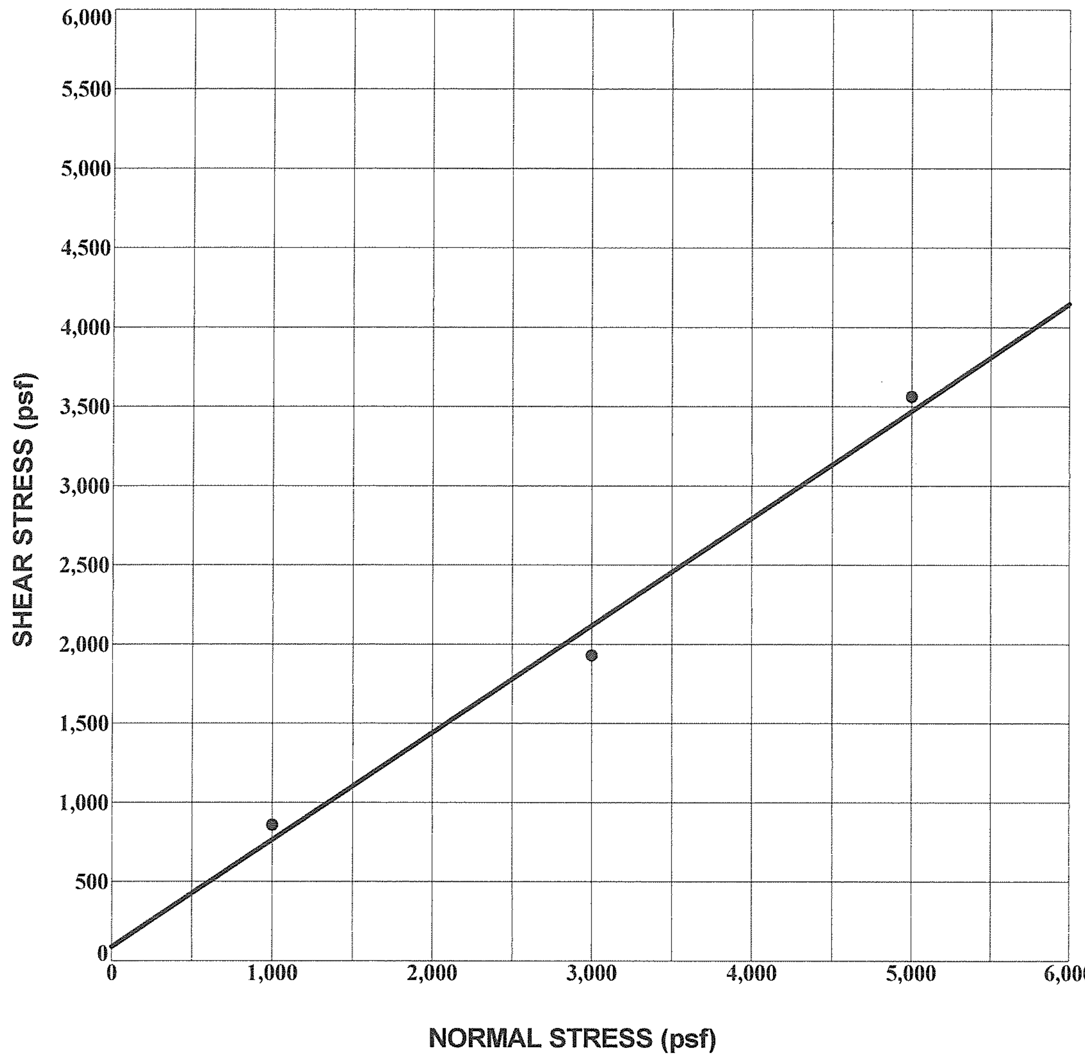
DIRECT SHEAR



Source:	**B-5**	**Friction Angle =**	**34 deg**
Depth:	**2.0 ft**	**Cohesion =**	**100 psf**
Test Type:	**Consolidated - Drained**		
Soil Description:	**Silty Sand (SM)**		

Dry Density (pcf)	117.4	117.4	117.4
Initial Water Content (%)	6.5	6.5	6.5
Final Water Content (%)	15.0	14.0	13.0
Normal Stress (psf)	1000	3000	5000
Shear Stress(psf)	858	1929	3563

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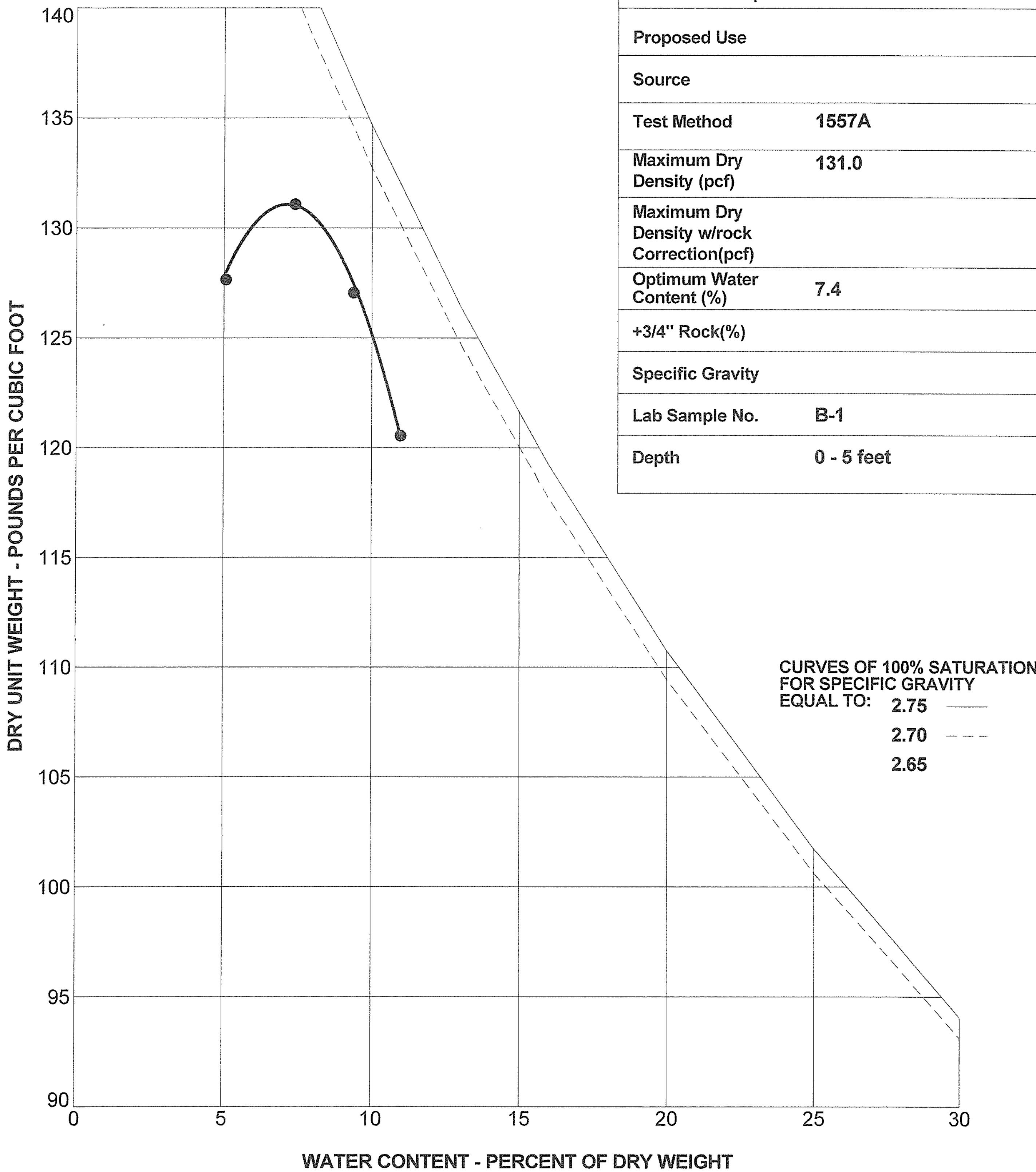
PROJECT NO. 112664

DIRECT SHEAR TEST
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

PLATE
B-1



SUMMARY OF TEST RESULTS	
Material Description	Silty Sand
Proposed Use	
Source	
Test Method	1557A
Maximum Dry Density (pcf)	131.0
Maximum Dry Density w/rock Correction(pcf)	
Optimum Water Content (%)	7.4
+3/4" Rock(%)	
Specific Gravity	
Lab Sample No.	B-1
Depth	0 - 5 feet

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MOISTURE DENSITY RELATIONSHIP
PG&E SHEPHERD SUBSTATION
SHEPHERD AND SUNNYSIDE AVENUES
FRESNO COUNTY, CALIFORNIA

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PROJECT NO. 112664