Appendix H

Geotechnical Investigation Reports

H1

PG&E Lockeford Substation
Geotechnical Investigation Report

GEOTECHNICAL INVESTIGATION REPORT
PG&E LOCKEFORD SUBSTATION
12861 EAST KETTLEMAN LANE
LODI, CALIFORNIA

PROJECT NO. 20193961.001A

JUNE 10, 2019

June 10, 2019
Project No. 20193961.001A

Pacific Gas and Electric Company
6111 Bollinger Canyon Road, Room 2460-A
San Ramon, CA 94583

Attention: Grant Wilcox, PE, PG, CEG
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Joseph Sun, PhD, PE, GE
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SUBJECT: Geotechnical Investigation Report

PROJECT: PG&E Northern San Joaquin Reinforcement – Lockeford Substation
PG&E Order No. / Operation Code: 74007150/3750
12861 East Kettleman Lane
Lodi, California

Dear Mr. Wilcox and Dr. Sun:

The attached report presents the results of Kleinfelder's geotechnical investigation for the Northern San Joaquin Reinforcement at the Lockeford Substation, located in Lodi, California. The report describes the study, findings, conclusions, and recommendations for use in project design and construction. Kleinfelder's services are authorized by our proposal dated February 26, 2019 and revised on March 6, 2019 and were performed in general accordance with the terms of our Master Services Agreement No. 4400007810.

The primary geotechnical concern at this site is shallow foundation support and potential caving of drilled pier excavations due to the loose silty sand and perched groundwater encountered in the upper 5 feet of all borings performed outside the existing substation. Based on the information gathered during this study, it is Kleinfelder's professional opinion that the subject site is geotechnically suitable for construction of the proposed improvements using conventional grading, shallow and deep foundation systems. Recommendations for shallow slab, spread footing, and drilled pier foundations are provided in this report. The recommendations presented herein should be incorporated into project design and construction

Recommendations for design of foundations, site grading, and other geotechnical considerations are presented in this report. The recommendations presented in this report should be incorporated into project design and construction. Kleinfelder appreciates the opportunity to provide geotechnical engineering services to PG&E during the design phase of this project. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully Submitted,

KLEINFELDER, INC.

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1 INTRODUCTION

This report presents the results of a geotechnical investigation conducted for the Northern San Joaquin Reinforcement at the PG&E Lockeford Substation, located at 12861 East Kettleman Lane, in Lodi, California. A site vicinity map is shown on Figure 1. Kleinfelder was retained by PG&E to provide geotechnical engineering services for the project. The purpose of the investigation was to evaluate the subsurface conditions at the site and develop geotechnical engineering recommendations to aid in project design and construction. Kleinfelder has previously submitted a report titled, "*Geotechnical Investigation Report, PG&E Lockeford Substation Improvements, 12861 East Kettleman Lane, Lodi, California*," dated June 9, 2016. This report was referenced during the development of the conclusions and recommendations.

1.1 PROPOSED CONSTRUCTION

Project understanding is based on the Geotechnical Investigation Request (GIR) dated January 17, 2019 and email and telephone correspondence with Grant Wilcox and Joseph Sun through March 1, 2019. We understand that PG&E plans to expand the existing Lockeford Substation as part of the Northern San Joaquin Reinforcement Project. The expansion will include construction of four breaker-and-a-half (BAAH) bays to support a total of eight element positions as well as a storm water basin, an SMP building, a battery building, a new substation fence, a new access road, and a new entrance into Lockeford Substation. At this time, foundation loading and dimensions for the aforementioned structures has not been provided.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore and evaluate subsurface conditions at the site and develop geotechnical conclusions and recommendations for use in project design, specification development, and construction. To accomplish these purposes, Kleinfelder's scope of services includes the following:

- Review of existing geologic and geotechnical data for the site vicinity.
- Drilling and sampling of four soil borings to explore subsurface conditions and to obtain samples for laboratory testing.

- Laboratory testing of selected samples to assess pertinent geotechnical properties.
- Evaluation of the available data to develop conclusions and recommendations to guide geotechnical aspects of design and construction.
- Preparation of this report.

Environmental evaluations and analyses, including detailed review of possible contaminants in the foundation soils, are outside of our scope of services.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

Prior to subsurface exploration, exploration locations were marked, and Underground Service Alert (USA) was contacted to provide utility clearance in the public right-of-way. A project-specific safety plan (PSSP) was prepared for the field exploration activities. This plan was discussed with the field crews prior to the start of field exploration work.

2.1.1 Exploratory Borings

Four borings, labeled B-1 through B-4, were drilled by Gregg Drilling of Martinez, California using a S-24 drill rig capable of hollow stem augers. Approximate exploration locations are shown on Figure 2. Exploration locations were designated in the field by measuring from existing landmarks. Horizontal coordinates and elevations of the borings were not surveyed.

The borings were drilled between April 9 and April 10, 2019. Borings B-1, B-2, B-3, and B-4 were drilled to depths of approximately 26½ feet, 51 ½ feet, 26½ feet, and 31 ½ feet, respectively.

Logs of the borings are provided in Appendix A. Our borings were cleared to a depth of about 4 feet below the ground surface using hand auger methods to confirm the absence of a grounding grid or other buried conflicts. Borings B-1 through B-4 were drilled using hollow stem auger methods from depths of about 4 to the end of each respective boring.

A Kleinfelder field-engineer maintained logs of the borings, visually classified the soils encountered per the Unified Soil Classification System (presented on Figure A-3 through A-6 in Appendix A) and obtained samples of the subsurface materials. Soil classifications made in the field from samples and auger cuttings were made in accordance with ASTM D2488. These classifications were re-evaluated in the laboratory after further examination and testing in accordance with ASTM D2487. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring logs. The blow counts listed on the boring logs are raw values and have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency.

Keys to the soil descriptions and symbols used on the boring logs are presented on Figures A-1, A-2 of Appendix A.

2.1.2 Sampling Procedures

Below the hand auger depth, soil samples were collected from the borings at depth intervals of approximately 2½ to 5 feet. Samples were collected from the borings at selected depths by driving either a 2.5-inch inside diameter (I.D.) California sampler or a 1.4-inch I.D. Standard Penetration Test (SPT) sampler driven 18 inches (unless otherwise noted) into undisturbed soil. The samplers were driven using a 140-pound automatic hammer free-falling a distance of 30 inches. Blow counts were recorded at 6-inch intervals for each sample attempt and are reported on the logs.

The SPT sampler did not contain liners. The 2.5-inch I.D. California sampler contained stainless steel liners. The California sampler was in general conformance with ASTM D3550. The SPT sampler was in conformance with ASTM D1586.

Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance. Following drilling, the samples were returned to our laboratory for further examination and testing. After the borings were completed they were backfilled with cement grout. Drilling spoils were contained in 55-gallon drums for analytical testing and staged inside the substation, for future disposal by our subcontractor.

2.2 LABORATORY TESTING

Laboratory tests were performed on selected samples to evaluate the physical and engineering properties of the materials encountered. Tests included the following:

- Percent passing the No. 200 sieve (ASTM D1140)
- Atterberg limits (ASTM D4318)
- Natural water content (ASTM D2216)
- Corrosion Suite:
    - Soluble Sulfate Content (ASTM D4327)
    - Soluble Chloride Content (ASTM D4327)
    - pH (ASTM D4972)
    - Minimum Resistivity (ASTM G57)
    - Redox (ASTM D1498)
    - Sulfide (ASTM D4658)

Results of most of the laboratory tests are included on the boring logs in Appendix A. Complete laboratory test data are presented in Appendix B.

3 GEOLOGIC CONDITIONS

3.1 AREA AND SITE GEOLOGY

According to geologic mapping by Marchand and Bartow (1979), the substation area is underlain by Quaternary aged terrace and alluvial fan deposits of the Upper Modesto and Lower Riverbank formations. In the project area, these soils generally consist of silts, sands, and gravels with minor clays. Regional groundwater levels in the area are greater than 70 feet deep based on DWR well records near the site.

3.2 LOCAL AND REGIONAL FAULTING

The substation is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site. The nearest zoned faults to the project site are the Greenville fault (located about 40 miles to the southwest), Calavaras fault (located about 51 miles to the southwest), Hayward fault (located about 59 miles to the southwest), and San Andreas fault zone (located about 78 miles to the southwest).

4 SITE CONDITIONS

4.1 SITE AND SURFACE DESCRIPTION

The existing Lockeford Substation is located at 12861 East Kettleman Lane in Lodi, California. The site is located within a rural agricultural area and is about 1,500 feet northeast of the Bear Creek. The site is bounded to the east and west by orchards, to the south by East Kettleman Lane and vineyards, and to the north by a vineyard and a field of annual crops (not identified). The site is relatively flat inside the existing substation where the grounds are covered with gravel. The expansion area to the northwest of the existing fence line is also relatively flat.

4.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered in our borings are in general agreement with the mapped geology. The following description provides a general summary of the subsurface conditions encountered during this study. For more thorough descriptions of the actual conditions encountered at specific boring locations, refer to the boring logs located in Appendix A.

Approximately ½ a foot of topsoil was encountered at the surface of the boring locations. The topsoil was underlain by a variation of sandy lean clay and clayey sand to an approximate depth of 4 feet, after which, a hard clayey layer was encountered which is commonly referred to as hardpan. Perched ground water was also found in Borings 3 and 4 at the depth of the hardpan.. The hard pan was underlain by a variation of clayey sand, sandy lean clay, and interbedded silty sand and sandy silt layers, until the end of each respective boring. Apparent densities of coarse-grained soils beneath the hardpan ranged from dense to very dense, and the consistency of fine-grained soils was generally hard.

4.3 GROUNDWATER

According to regional well record data published by the State Water Resources Control Board (https://www.waterboards.ca.gov/), regional groundwater levels are generally greater than 70 feet below the ground surface. Regional groundwater was not encountered during our explorations. However, it should be noted that perched water was encountered at a depth of about 5 feet within the hardpan layer encountered in Boring B-3 and B-4.

It is possible that groundwater conditions at the site could change due to variations in rainfall and runoff, regional groundwater withdrawal or recharge, construction activities, or other factors not apparent at the time the study was performed.

4.4 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions at the site are based on the conditions encountered in the borings drilled for this project. The conclusions and recommendations that follow are based on those interpretations. If soil or groundwater conditions exposed during construction vary from those presented in this report, Kleinfelder should be notified to evaluate whether our conclusions or recommendations should be modified.

5 CONCLUSIONS AND RECOMMENDATIONS

The following sections discuss conclusions and recommendations with respect to geologic and seismic hazards, California Building Code (CBC) design considerations, site preparation and grading, and foundation design.

5.1 2016 CBC SEISMIC DESIGN PARAMETERS

5.1.1 Site Class

In developing seismic design criteria, the characteristics of the soils underlying the site are an important input to evaluate the site response. According to the 2016 California Building Code (CBC), the project site may be classified as Site Class D, Stiff Soil, according to Section 1613.3.2 of 2016 CBC and Table 20.3-1 of American Society of Civil Engineers (ASCE) 7-10 (2010). Site Class D is defined as a soil profile consisting of stiff soil profile with a shear wave velocity between 600 feet per second and 1,200 feet second, standard penetration test (SPT) blow counts (N-value) between 15 blows per foot and 50 blows per foot, or undrained shear strength between 1,000 pounds per square foot and 2,000 pound per square foot in the top 100 feet.

5.1.2 Seismic Design Parameters

Approximate coordinates for the site are noted below.

- Latitude: 38.117944°N
- Longitude: 121.158938°W

For a 2016 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1), associated soil amplification factors (F_a and F_v), and mapped peak ground acceleration (PGA) are presented in Table 5-1. Corresponding site modified (S_{MS} and S_{M1}) and design (S_{DS} and S_{D1}) spectral accelerations, PGA modification coefficient (F_{PGA}), PGA_M, risk coefficients (C_{RS} and C_{R1}), and long-period transition period (T_L) are also presented in Table 5-1. Presented values were estimated using Section 1613.3 of the 2016 California Building Code (CBC), chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps (https://seismicmaps.org/).

Table 5-1
Ground Motion Parameters Based on 2016 CBC

Parameter	Value	Reference
S_S	0.662g	2016 CBC Section 1613.3.1
S_1	0.279g	2016 CBC Section 1613.3.1
Site Class	D	2016 CBC Section 1613.3.2
F_a	1.270	2016 CBC Table 1613.3.3(1)
F_v	1.842	2016 CBC Table 1613.3.3(2)
PGA	0.225g	ASCE 7-10 Figure 22-7
S_{MS}	0.841g	2016 CBC Section 1613.3.3
S_{M1}	0.514g	2016 CBC Section 1613.3.3
S_{DS}	0.561g	2016 CBC Section 1613.4.4
S_{D1}	0.343g	2016 CBC Section 1613.4.4
F_{PGA}	1.351	ASCE 7-10 Table 11.8-1
PGA_M	0.303g	ASCE 7-10 Section 11.8.3
C_{RS}	1.111	ASCE 7-10 Figure 22-17
C_{R1}	1.148	ASCE 7-10 Figure 22-18
T_L	12 seconds	ASCE 7-10 Figure 22-12

5.2 LIQUEFACTION

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table but can also occur in non-plastic to low-plasticity, finer-grained soils. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or “cyclic mobility,” increased lateral earth pressures on retaining walls, liquefaction settlement, and lateral spreading or “flow failures” in slopes.

Based on the relative density, soil type, and depth to groundwater at the site, the potential for liquefaction is considered negligible.

5.3 EXPANSIVE SOILS

Based on the results of an Atterberg limits test performed on a near-surface sample of sandy clay (Boring B-1 at a depth of about 3 feet), the surficial soils have low expansion potential (Liquid Limit of 23 and Plasticity Index of 10). Based on the low expansion potential and density of these soils, we do not anticipate they will shrink or swell significantly as a result of soil moisture content changes. Given the presence of perched ground water however, we do recommend replacing the upper 6-inches with import non-expansive fill material beneath all slabs, which is discussed further in Section 5.10.2.5.

5.4 SITE PREPARATION

5.4.1 General

Considering site grades are presently well established, site grading is anticipated to be minimal, minus the grading for the proposed pond. General recommendations for site preparation and earthwork construction are presented in the following sections of this report. All earthwork, including excavation, backfill and preparation of subgrade, should be performed in accordance with the geotechnical recommendations presented in this report and applicable portions of the grading code of local regulatory agencies. The grading contractor is responsible to notify governmental agencies, as required, and the geotechnical engineer at the start of site cleanup, the initiation of grading and any time that grading operations are resumed after an interruption. All earthwork should be performed under the observation and testing of a Kleinfelder representative. All references to compaction, maximum density and optimum moisture content are based on ASTM D1557, unless otherwise noted.

5.4.2 Stripping and Grubbing

Any miscellaneous surface obstructions, vegetation, debris or other deleterious materials should be removed from the project area prior to any site grading. The stripped materials should not be incorporated into any engineered fill. Existing pavements to be demolished should include removal of the pavement and aggregate base materials.

5.4.3 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

Initial site grading should include a reasonable search to locate soil disturbed by previous activity and abandoned underground structures or existing utilities that may exist within the areas of construction. Any loose or disturbed soils, void spaces that may be encountered should be over-excavated to expose firm native soil, as approved by a representative of Kleinfelder.

Unless approved otherwise by an on-site representative of Kleinfelder during grading, undocumented fills at the locations of any future grading or shallow foundations should be over-excavated and replaced with engineered fill as recommended below in the "Engineered Fill-Placement and Compaction Criteria" section of this report.

5.4.4 Scarification and Compaction

In areas requiring placement of fill, it is recommended the fill be placed and compacted as engineered fill. Following site stripping and any required grubbing and/or over-excavation, it is recommended areas to receive engineered fill be scarified to a depth of 8 inches, uniformly moisture conditioned to at least the optimum moisture content for sandy soils (SP, SM, SC) or at least 3 percent above the optimum moisture content for clayey soils (CL, CH) and compacted to at least 90 percent relative compaction for sandy soils or between 88 and 92 percent relative compaction for clayey soils, as determined by ASTM D1557.

5.5 ENGINEERED FILL

5.5.1 Onsite Materials

The on-site soil appears suitable for use as engineered fill. All engineered fill should be free of debris, significant organics, or other deleterious materials, and have a maximum particle size less than 3 inches in maximum dimension. Where imported material is brought in, it is recommended that it be granular in nature and conform to the minimum criteria discussed in Table 5-2.

5.5.2 Non-Expansive Engineered Fill Requirements

Specific requirements for engineered fill as well as applicable test procedures to verify material suitability are provided below:

Table 5-2
Engineered Fill Requirements

Fill Requirement		**Test Procedures**	
		ASTM	**Caltrans**
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D6913	202
¾ inch	70-100	D6913	202
No. 200	20-50	D6913	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D4318	204
Organic Content			
No visible organics		---	---
Expansion Potential			---
20 or less		D4829	---
Soluble Sulfates			
Less than 2,000 ppm		---	417
Soluble Chloride			
Less than 300 ppm		---	422
Resistivity			
Greater than 2,000 ohm-cm		---	643

Materials to be used for engineered fill should be sampled and tested by Kleinfelder prior to being transported to the site. Highly pervious materials such as clean crushed stone or pea gravel are not recommended for use in engineered fill because they can permit transmission of water into the underlying materials. We recommend representative samples of imported materials proposed for use as engineered fill be submitted to Kleinfelder for testing and approval at least one week prior to the start of grading and import of this material.

In addition, we recommend that a laboratory corrosion test series (pH, resistivity, redox, sulfides, chlorides, and sulfates) be performed on all proposed import materials.

5.5.3 Placement and Compaction Criteria

Non-expansive soils that meet the criteria outlined in Table 5-2 that are to be used for engineered fill should be uniformly moisture conditioned to at least the optimum moisture content, placed in horizontal lifts less than about 8 inches in loose thickness, and compacted to at least 90 percent relative compaction, as determined by ASTM D1557. Onsite clayey soils to be used for general fill where engineered fill is not required should be uniformly moisture conditioned to at least 4 percent over the optimum moisture content, placed in horizontal lifts no more than about 8 inches in loose thickness, and compacted to between 88 and 92 percent relative compaction, as determined by ASTM D1557.

Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or moisture content, or if soil conditions are not stable. Disking or blending may be required to uniformly moisture condition soils used for engineered fill. Ponding or jetting compaction methods should not be allowed.

All site preparation and fill placement should be observed by Kleinfelder. It is important that during the stripping and scarification processes, a representative of Kleinfelder be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during the geotechnical site exploration.

5.6 WET WEATHER CONSIDERATIONS

Should construction be performed during or subsequently after 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 geogrid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork and construction operations.

5.7 SITE DRAINAGE

Final site grading should provide surface drainage away from all structures and areas to be traversed by vehicles and maintenance equipment. In general, we recommend consideration be given to providing at least 2 percent slope away from structure foundations or access ways.

5.8 TEMPORARY EXCAVATIONS

5.8.1 General

All excavations should comply with applicable local, state, and federal safety regulations including the current Occupational Safety & Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the Contractor, who is 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.

5.8.2 Excavation and Slopes

Excavated slope height, slope inclination, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties.

Underground utilities should be located above a 1H:1V (horizontal to vertical) plane projected down and out from the bottoms of new footings to avoid undermining the footings during the excavation of the utility trench.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should be kept sufficiently away from the top of any excavation to prevent any unanticipated surcharging. Alternatively, excavation slopes and shoring systems can be designed to accommodate surcharge loadings, if necessary. Shoring, bracing, or underpinning required for the project (if any), should be designed by a professional engineer registered in the State of California.

5.9 TRENCH BACKFILL

All trench backfill should be placed and compacted in accordance with recommendations provided for engineered fill (see Section 5.5.3). Mechanical compaction is recommended. Ponding or jetting should not be used as a sole means of soil compaction.

5.10 SHALLOW FOUNDATIONS

This section provides general recommendations for shallow foundations. Kleinfelder should review the design to ensure compliance with the intent of the geotechnical conclusions and recommendations provided in this report.

Foundations should satisfy two independent criteria with respect to foundation soils. First, the foundation should have an adequate safety factor against bearing failure with respect to the shear strength of the foundation soils. Second, the vertical movements of the foundation due to settlement (both immediate elastic settlement and consolidation settlement) should be within tolerable limits for the structure. Depending on the settlement tolerance of planned structures, design loading, and foundation dimensions, the general recommendations presented in this report may be subject to modification. If future project needs require additional foundation capacity, Kleinfelder should be contracted to evaluate this potential for specific foundation designs.

Lightly-loaded structures may be supported on conventional, shallow, reinforced concrete mat foundations or spread footings, provided the site structures can tolerate the anticipated settlement.

5.10.1 Spread Footings

5.10.1.1 Allowable Bearing Pressure

Shallow spread footings constructed of reinforced concrete may be founded on approved undisturbed native soil and/or engineered fill. The footings should be founded at least 18 inches below lowest adjacent finished grade on subgrade soils that have been prepared in accordance with the recommendations provided in this report. Continuous and isolated rectangular footings should have a minimum width of 12 inches.

For foundation subgrade prepared in accordance with the recommendations provided in this report, spread and strip footings may be designed for a net allowable bearing pressure of up to 3,000 pounds per square foot (psf) due to dead plus live loads. The weight of the foundation that extends below grade may be neglected when computing dead loads. The allowable bearing pressure includes a safety factor of at least 3 with respect shear failure of the foundation soils and may be increased by one-third for transient loading due to wind or seismic forces.

To maintain the desired support, foundations adjacent to utility trenches or other existing foundations should be deepened so that their bearing surfaces are below an imaginary plane having an inclination of 1 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent foundations or utility trenches.

5.10.1.2 Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.40 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 360 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches should be neglected unless the area in front of the footing is protected from disturbance by concrete or pavement. The allowable friction coefficient and passive resistance may be used concurrently.

5.10.1.3 Settlement

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Foundation dimensions and loads have not been provided for the proposed structures, we estimate maximum total settlement of foundations designed and constructed in accordance with the preceding recommendations of up to about ½ inch or less. Differential settlement between similarly loaded, adjacent footings are estimated to be about half the total settlement. The majority of foundation settlement is expected to occur rapidly and should be essentially complete shorty after initial application of the loads.

5.10.1.4 Shallow Foundation Construction Considerations

Prior to placing steel or concrete, foundation excavations should be cleaned of any debris, disturbed soil or water. All foundation excavations should be observed by a representative of Kleinfelder just prior to placing fill and/or steel or concrete. The purpose of these observations is to check that the bearing soils actually encountered in the foundation excavations are similar to those assumed in analysis and to verify the recommendations contained herein are implemented during construction.

5.10.2 Mat Foundations

Recommendations for design and construction of small mat slab foundations up to about 25 feet wide are presented below. Kleinfelder should be consulted to provide supplementary mat foundation recommendations if larger mat slab foundations are planned in the future.

5.10.2.1 Allowable Bearing Pressure

For subgrades prepared as recommended in this report, reinforced concrete mat foundations may be designed for a net allowable bearing pressure of 3,000 psf. If higher allowable bearing capacity for mat foundations is required, Kleinfelder should be consulted to provide supplemental engineering and construction recommendations on a case-by-case basis. The allowable bearing pressure applies to dead plus live loads, includes a safety factor of at least 3 with respect to shear failure of the foundation soils, and may be increased by one-third for short-term loading due to wind or seismic forces.

5.10.2.2 Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.40 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 360 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches should be neglected unless the area in front of the foundation is protected from disturbance by concrete or pavement. The friction coefficient and passive resistance may be used concurrently.

5.10.2.3 Subgrade Modulus

For preliminary design purposes, a modulus of subgrade reaction, K_{v1}, of 150 pounds per square inch per inch of deflection (for a 1 square-foot bearing plate) may be used for design of mat slabs. The modulus should be adjusted for the actual slab size using appropriate formulas or software.

5.10.2.4 Mat Slab Settlement

For foundations with design pressures equal to or less than the net allowable pressure provided above, and under static loading conditions, total post-construction foundation settlement is expected to be less than about ½ inch at the center of the mat foundations. Post-construction differential settlement of individual foundation elements is expected to be about one-half the total settlement.

These settlement estimates are based on the assumption that the foundation subgrade is properly prepared, and the foundations are designed and constructed in accordance with the recommendations presented in this report.

5.10.2.5 Mat Foundation Construction Considerations

Underground utilities that are 4 feet deep or shallower and that run parallel to shallow mat foundations generally should be located no closer than 2 feet horizontally away from the perimeter edges of the slab. Deeper utilities should be located above a 1H:1V (horizontal to vertical) slope projected downward from the bottom edges of the slab. Utility plans should be reviewed by Kleinfelder prior to trenching to evaluate conformance with this requirement.

Beneath exterior cast-in-place concrete mat foundations, we recommend the design include a base course of well-graded crushed aggregate base at least 6 inches thick. Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate base. Under slabs that will be subject to vehicle loading, the aggregate base course thickness should be increased to a minimum of 6 inches. The base course should be compacted to at least 95 percent relative compaction at optimum moisture content. Thickened slab edges embedded to at least 18 inches below grade need not be underlain by the gravel base course.

If a capillary break and vapor mitigation under mat slabs are required by the architect, the 6-inch thick layer of Class 2 aggregate base may be omitted and replaced with 6 inches of crushed rock with less than 5 percent passing the No. 4 sieve, such as Caltrans Class 2 permeable material. The capillary break rock layer should be overlain by a vapor retarder membrane that meets ASTM E1745 requirements. Installation should meet architectural and manufacturer recommendations. Although capillary break and vapor retarded systems are commonly used, these systems will not "moisture proof" the floor slab or otherwise ensure floor slab moisture transmission rates will meet project requirements.

5.11 DRILLED PIER FOUNDATIONS

Recommendations for design and construction of drilled pier foundations are presented in the following sections of this report.

5.11.1 Axial Capacity

Axial pile capacity was developed based on Federal Highway Administration methods using the commercial computer software SHAFT, version 2017, produced by Ensoft, Inc. Static soil strength parameters are based on strength and soil properties measured during the field and laboratory testing phases of this investigation.

Axial loads on drilled piers should be supported by the frictional capacity of the pier. End bearing is not considered in the axial capacity due to strain incompatibility issues between skin friction and end bearing, settlement issues, and the potential for loose materials to exist at the bottoms of the pier holes during construction that cannot be effectively cleaned out. If additional axial capacity is required beyond what is provided in this report, Kleinfelder should be consulted to provide a portion of end bearing capacity and additional construction recommendations.

A curve illustrating the ultimate axial compressive capacity of a unit (1-foot) diameter straight-sided drilled pier installed from the existing grade under static conditions is shown on Figure 3a. Corresponding tabulated values are presented on Figure 3b. Capacities for drilled piers with diameters other than 1 foot may be obtained by multiplying the capacity for the 1-foot diameter pier by the actual pier diameter (in feet). Ultimate tensile capacity may be obtained by multiplying the compressive capacity by a factor of 0.8 and adding the weight of the foundation. For evaluation of allowable axial capacity under static conditions, we recommend a factor of safety of 3 be applied to the ultimate capacity (per the General Order 95 code). Note that the weight of the foundation need not be considered for evaluation of allowable axial capacity. For allowable tension capacity under transient flood, wind or seismic conditions, a safety factor of at least 1.5 should be used. For allowable sustained tension, a safety factor of 3 should be used.

5.11.1.1 Estimated Settlement

Based on the methods outlined by Brown et al. (2010), total static settlement of each drilled pier should be on the order of 0.1 percent of the pier diameter for a drilled pier designed and constructed in accordance with the recommendations presented in this report. This value includes elastic compression of the pile under design loads. The majority of the settlement should occur during and shortly after application of the structure loads. We suggest allowing for about ¼ inch of settlement to accommodate potential long-term settlement, construction issues, and some soil variability across the site.

5.11.1.2 Axial Capacity Group Effects

The axial capacity of piers developed in accordance with the recommendations provided above applies to single, isolated piers. Consideration of group effects on axial capacity of drilled piers is usually not necessary for piers with center-to-center spacings of at least 3 effective diameters. For closer spacings the capacity of individual piers will be reduced. For these cases Kleinfelder should be consulted to evaluate axial capacity on a case-by-case basis. Note that group effects should also be considered where new foundations are constructed immediately adjacent to existing foundations.

5.11.2 Lateral Response

5.11.2.1 LPILE Analysis Soil Parameters

Lateral capacity of drilled piers may be developed through analysis of pier response due to a range of design loads. Table 5-3 contains recommended input soil parameters for lateral response analysis of deep foundations using the LPILE computer program (by Ensoft, Inc., Version 2018. Program default values may be used for strain factor (E_{50}) and horizontal subgrade reaction (K).

Table 5-3
LPILE Geotechnical Parameters
Static Conditions

Depth (feet)	Model P-Y Curve	Effective Unit Weight (lb/ft^3)	Cohesion c (psf)	Internal Friction Angle, Φ (degrees)
0 to 30	Sand (Reese)	120	-	35

5.11.3 Drilled Pier Construction Considerations

Successful completion of drilled pier foundations requires good construction procedures. Drilled pier excavations should be constructed by a skilled operator using techniques that allow the excavations to be completed, the reinforcing steel placed, and the concrete poured in a continuous manner to reduce the time that excavations remain open. Steel reinforcement and concrete should be placed on the same day of completion of each pier excavation. Additionally, drilled pier excavations should be scheduled to allow concrete in each pile to set over night before drilling adjacent holes that are closer than 4 diameters center-to-center.

The following considerations should be implemented during construction of drilled shaft foundations. We recommend the contractor follow the procedures for drilled pier construction contained in the Federal Highway Administration (FHWA) manual on drilled shaft construction (Brown et al., 2010).

Consistent with Chapter 17 of the 2016 CBC, drilled pier excavations should be inspected and approved by the geotechnical engineer prior to installation of reinforcement. The depths of all pier excavations should be checked immediately prior to concrete placement to verify excessive sloughing and/or caving has not reduced the required hole depth. This may be done with a weighted tape measure or similar measuring device.

As described above, perched groundwater may be encountered at shallower depths depending on local rainfall and runoff patterns at the time of construction. The contractor should be prepared to handle shallow groundwater and possibly caving sandy soil conditions during construction of drilled piers at the site.

Drilled shaft excavations extending below groundwater levels should be cleaned such that less than about 1 inch of loose soil remains at the bottom of the drilled hole. Since the piers should be designed to derive their support in skin friction along the sides of the shafts, consideration could be given to over-drilling the shafts to accommodate any sloughing that may occur between drilling and concrete placement. It is recommended that a representative from Kleinfelder observe each drilled shaft excavation to verify soil and excavation conditions prior to placing steel reinforcement or concrete.

Steel reinforcement and concrete should be placed on the same day the drilled hole is completed to reduce the potential for caving and reduce the quantity of suspended soil particles that may settle to the bottom of the hole during wet-method construction. Excavation depths should be checked several times before concrete placement to ensure excessive sedimentation has not occurred. Concrete used for pier construction should be discharged vertically into the drilled hole to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during shaft construction. 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.

In order to develop the design skin friction values provided in the axial capacity figures, concrete used for drilled pier 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 pile.

If water or drilling fluids are present during concrete placement, concrete should be placed into the hole using tremie methods. Tremie concrete placement should be performed in strict accordance with ACI 304R. The tremie pipe should be rigid and remain below the surface of the in-place concrete at all times to maintain a seal between the water or slurry and 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 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 thick but will depend on the pile diameter, amount of water seepage, and construction workmanship.

As noted above, perched groundwater and caving sandy soils may be encountered during drilled pier construction. Use of slurry drilling methods may be needed to reduce the potential for caving in the drilled pier excavations where groundwater levels are above the bottom of the excavation. Use of slurry drilling methods normally requires experienced construction personnel to batch and mix the slurry, test the slurry for proper mixing, hydration, viscosity and other important properties, and to monitor slurry performance during drilling. If slurry drilling methods are used, we recommend use of a polymer slurry that meets Caltrans requirements for drilled shaft construction or bentonite-based slurry, mixed and used in accordance with the guidelines in the FHWA Drilled Shaft Manual (Brown et al., 2010). This guideline recommends bentonite slurry mixtures not be left in the hole for more than about 4 hours in order to avoid potential side friction losses that may be caused by excessive thickness of bentonite filter cake on the hole wall.

If caving conditions are encountered in a drilled pier excavation and there are no overhead clearance issues, temporary casing could be used to help mitigate this condition. If temporary steel casing is used, it should be removed from the hole as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete during casing withdrawal and concrete placement operations. Casing should not be withdrawn until sufficient quantities of concrete have been placed into the excavation to balance the groundwater head outside the casing. Continuous vibration of the casing or other methods may be required to reduce the potential for voids occurring within the concrete mass during casing withdrawal. Corrugated metal pipe should not be used as casing. In no case should casing material be left in the excavation after concrete has been placed without the approval of the project structural and geotechnical engineers. Concrete should be in direct contact with the surrounding soil or the design parameters and recommendations in the geotechnical report are not valid.

5.12 SOIL CORROSION

Kleinfelder has completed laboratory testing to provide data regarding corrosivity of onsite soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

Laboratory chloride concentration, sulfate concentration, pH, oxidation reduction potential, redox, sulfide and electrical resistivity tests were performed for a near surface soil sample. The results of the tests are attached and are summarized in Table 5-4. If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material.

Table 5-4
Chemistry Laboratory Test Results

Boring and Depth	Material	Resistivity, ohm-cm	Resistivity, ohm-cm (Saturated)	pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
						Chloride	Sulfide	Sulfate
B-1 & B-2 @ 1-4 ft.	Sand	6,500	8,600	6.92	320	N.D.*	N.D.*	N.D.*
B-3 & B-4 @ 1-4 ft.	Sand	6,100	6,400	6.74	340	N.D.*	N.D.*	N.D.*

*N.D. - None Detected

Ferrous metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices.

Based on the “10-point” method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the potential for the soils at the site to be corrosive to buried ferrous metal piping, cast iron pipes, or other objects made of these materials is negligible. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication “Guide to Durable Concrete” (ACI 201.2R-08) provides guidelines for this assessment. The samples had sulfate concentrations of non-dedectible (N.D.), which indicates the potential for deterioration of concrete is mild to negligible, and no special requirements should be necessary for the concrete mix.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater. Chloride tests indicated the sample had concentrations below the detectable limit.

6 ADDITIONAL SERVICES

6.1 PLANS AND SPECIFICATIONS REVIEW

Kleinfelder should conduct a general review of plans and specifications to evaluate that the earthwork and foundation recommendations presented in this report have been properly interpreted and implemented during design. In the event Kleinfelder is not retained to perform this recommended review, no responsibility for misinterpretation of the recommendations by Kleinfelder is accepted.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that all earthwork and foundation construction be monitored by a representative from Kleinfelder, including site preparation, placement of all engineered fill and trench backfill, construction of slab and all foundation excavations. The purpose of these services is to observe the soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

7 LIMITATIONS

This report presents information for planning, permitting, design, and construction of the new electrical structures and perimeter fence at the Lockeford Substation in Lodi, California. Recommendations contained in this report are based on materials encountered in Borings B-1 through and B-4, geologic interpretation based on published articles and geotechnical data, and our present knowledge of the proposed construction.

It is possible that soil conditions could vary beyond the points explored. If the scope of the proposed construction, including the proposed location, changes from that described in this report, we should be notified immediately in order that a review may be made, and any supplemental recommendations provided.

We have prepared this report in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty expressed or implied is made.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on-site and off-site) 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.

8 REFERENCES

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FIGURES

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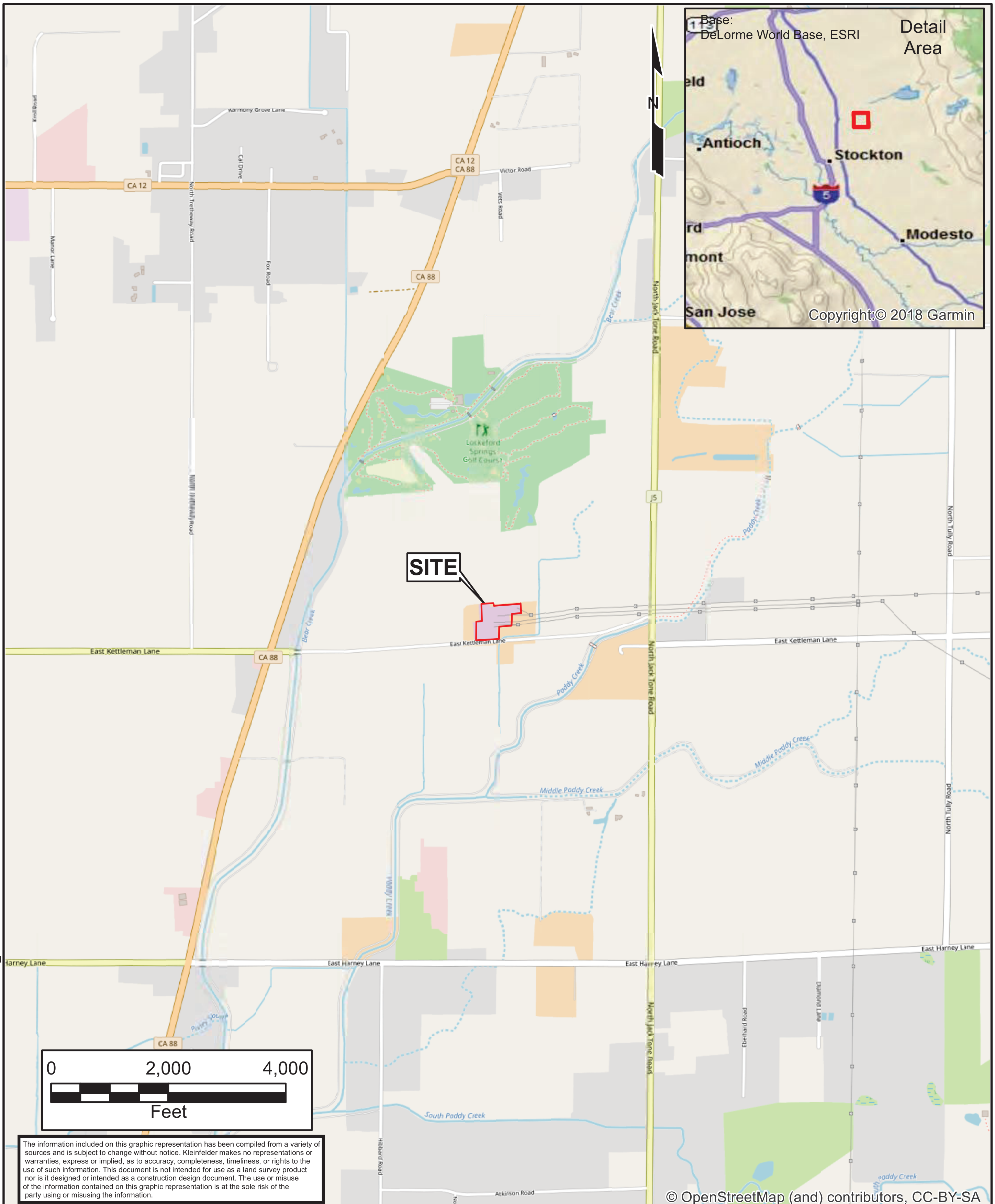

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KLEINFELDER Bright People. Right Solutions.	PROJECT NO. 20193961 DRAWN BY: JDS CHECKED BY: AS DATE: 5/9/2019 REVISED:	SITE VICINITY MAP PG&E LOCKEFORD SUBSTATION 12861 EAST KETTLEMAN LANE LODI, CALIFORNIA	FIGURE 1

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SCALE: 1" = 200' SCALE IN FEET

REFERENCE:
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LEGEND

FENCE LINE

SOIL BORING

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CHECKED BY:	AS
DATE:	05/02/2019
REVISED:	

EXPLORATION LOCATION PLAN

PG&E LOCKEFORD SUBSTATION
12861 EAST KETTLEMAN LANE
LODI, CALIFORNIA

FIGURE

2

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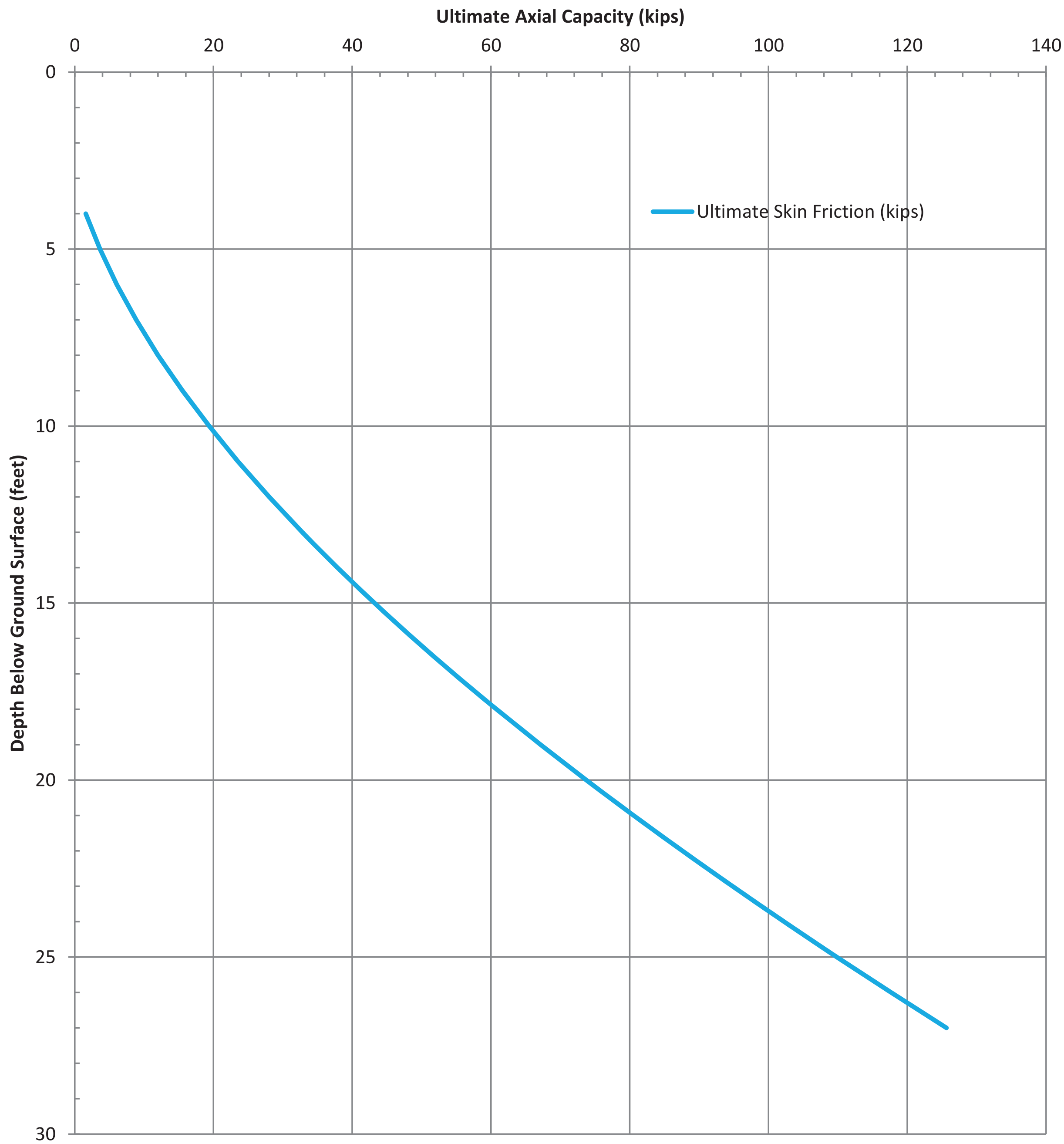

Notes:

1. Axial capacities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the pier (in feet).
2. Ultimate tensile capacity may be obtained by multiplying the ultimate compressive capacity by a factor of 0.8.
3. The curve represents ultimate axial capacity of a straight-sided drilled pier. See text discussion for factor of safety and group effects.

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PROJECT NO.: 20193961	ULTIMATE AXIAL CAPACITY TABLE UNIT DIAMETER (1-FOOT) DRILLED PIER STATIC CONDITION	FIGURE
DRAWN BY: HF		
CHECKED BY: KG	PG&E LOCKEFORD SUBSTATION	3A
DATE: 5/7/2019	12861 EAST KETTLEMAN LANE	
REVISED:	LODI, CA	

Depth (ft)	Ultimate Axial Capacity (Kips)	Depth (ft)	Ultimate Axial Capacity (Kips)
4	1.6	18	60.8
5	3.6	19	67.1
6	6.0	20	73.7
7	8.8	21	80.5
8	12.0	22	87.6
9	15.5	23	94.8
10	19.4	24	102.2
11	23.5	25	109.8
12	28.0	26	117.7
13	32.8	27	125.6
14	37.8		
15	43.2		
16	48.8		

Notes:

1. Axial capcities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the pier (in feet).
2. Ultimate tensile capacity may be obtained by multiplying the ultimate compressive capacity by a factor of 0.8.
3. The curve represents ultimate axial capacity of a straight-sided drilled pier. See text discussion for factor of safety and group effects.

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PROJECT NO.: 20193961	ULTIMATE AXIAL CAPACITY TABLE UNIT DIAMETER (1-FOOT) DRILLED PIER STATIC CONDITION	FIGURE	
DRAWN BY: HF			
CHECKED BY: KG	PG&E LOCKEFORD SUBSTATION	3B	
DATE: 5/7/2019	12861 EAST KETTLEMAN LANE		
REVISED:	LODI, CA		

APPENDIX A

FIELD EXPLORATION

SAMPLE/SAMPLER TYPE GRAPHICS

- BULK SAMPLE
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

- ▽ WATER LEVEL (level where first observed)
- ▼ WATER LEVEL (level after exploration completion)
- ▼ WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

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 GRAVEL 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

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PROJECT NO.:	20193961
DRAWN BY:	JDS
CHECKED BY:	AS
DATE:	4/22/2019
REVISED:	-

GRAPHICS KEY

PG&E LOCKEFORD SUBSTATION
12861 EAST KETTLEMAN LANE
LODI, CALIFORNIA

FIGURE

A-1

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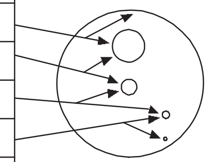
GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT	
	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

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

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25 ≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5 ≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 ≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 ≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 ≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.: 20193961	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: JDS		
CHECKED BY: AS	PG&E LOCKEFORD SUBSTATION 12861 EAST KETTLEMAN LANE LODI, CALIFORNIA	A-2
DATE: 4/22/2019		
REVISED: -		

Date Begin - End:	4/09/2019	**Drilling Company:**	Gregg Drilling		**BORING LOG B-1**
Logged By:	A. Sadat	**Drill Crew:**	Louie Wens		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	S24/S97/Liftgate	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Clear	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.11871°
Longitude: -121.16069°
Surface Condition: Grass

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	TOPSOIL: Grass and roots												hand auger
	Clayey SAND (SC): non-plastic, reddish brown, wet, soft, roots from the adjacent trees	Bulk #1											
		Bulk #2					19.4				23	10	
	Sandy Silty CLAY (CL-ML): non-plastic, reddish brown, moist, hard, strongly cemented (Hard Pan)	Bulk #3											
5		Bulk #4		BC=8 18 24	18"	CL-ML	11.7			56	20	4	switched to hollow stem auger
	Clayey SAND (SC): fine-grained sand, low plasticity, reddish brown, moist, dense	1											
	Sandy SILT (ML): medium plasticity, reddish brown, moist, hard	2		BC=7 16 26	18"	ML				61	20	3	
10	**Clayey SAND (SC)**: low plasticity, reddish brown, dry, very dense	3		BC=23 38 50	18"								
15	yellowish brown, dense, blacks streaks	4		BC=11 21 23	18"								
	trace gravel												
20		5		BC=9 15 24	18"								
25	very dense	6		BC=9 14 36	18"								
	Sandy SILT (ML): low plasticity, yellowish brown, moist, hard												

The boring was terminated at approximately 26.5 ft. below ground surface. The boring was backfilled with grout on April 09, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

30

Date Begin - End:	4/09/2019	**Drilling Company:**	Gregg Drilling		**BORING LOG B-2**
Logged By:	A. Sadat	**Drill Crew:**	Louie Wens		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	S24/S97/Liftgate	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Clear	**Exploration Diameter:**	8 in. O.D.		

Latitude: 38.11859°
Longitude: -121.16019°
Surface Condition: Grass

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. / Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	**TOPSOIL**: Grass with roots												hand auger
	Sandy Lean CLAY (CL): low plasticity, reddish brown, moist	BULK #1	Bulk		18"								
	Sandy Lean CLAY (CL): fine-grained sand, low plasticity, reddish brown, dry, hard, strongly cemented (Hard Pan)												switched to hollow stem auger
5		1	SPT	BC=11 50	18"								
	Sandy Lean CLAY (CL): low plasticity, yellowish brown, dry, hard, weakly cemented	2	SPT	BC=10 22 29	18"								
10	**Clayey SAND (SC)**: low plasticity, yellowish brown, moist, very dense	3	SPT	BC=18 29 38	18"								
15	coarse-grained sand, trace gravel	4	SPT	BC=6 12 42	18"					33			
20	olive brown, very dense, moderately cemented	5	SPT	BC=12 15 26	6"								
25	**Clayey SAND with Gravel (SC)**: non-plastic, yellowish brown, moist, very dense	6	SPT	BC=9 32 42	18"								
	Sandy SILT (ML): low plasticity, yellowish brown, moist, hard, weakly cemented	7	SPT	BC=7 18 26	18"								
30	**SILT with Sand (ML)**: low plasticity, yellowish brown, moist, hard	8	SPT	BC=10 14 26									

Date Begin - End:	4/09/2019	Drilling Company:	Gregg Drilling	BORING LOG B-2
Logged By:	A. Sadat	Drill Crew:	Louie Wens	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	S24/S97/Liftgate	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger	
Weather:	Clear	Exploration Diameter:	8 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.11859°
Longitude: -121.16019°
Surface Condition: Grass

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Sandy Lean CLAY (CL): low plasticity, yellowish brown, moist, very stiff	9		BC=9 13 22 PP=3.5 PP=4.5	12"								
40		**Poorly graded SAND with Clay (SP-SC)**: non-plastic, yellowish brown, moist, very dense	10		BC=18 30 50 PP=3.5	18"								
45		**Sandy Lean CLAY (CL)**: low plasticity, yellowish brown, moist, hard	11		BC=11 11 27	18"								
50		**SILT with Sand (ML)**: low plasticity, yellowish brown, moist, hard	12		BC=7 13 25	18"								

55

60

65

The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with grout on April 09, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.:	20193961	BORING LOG B-2	FIGURE
DRAWN BY:	JDS		
CHECKED BY:	AS	PG&E LOCKEFORD SUBSTATION 12861 EAST KETTLEMAN LANE LODI, CALIFORNIA	A-4
DATE:	4/22/2019		
REVISED:	-		PAGE: 2 of 2

Date Begin - End:	4/10/2019	**Drilling Company:**	Gregg Drilling			**BORING LOG B-3**
Logged By:	A. Sadat	**Drill Crew:**	Louie Wens			
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	S24/S97/Liftgate	**Hammer Type - Drop:**	140 lb. Auto - 30 in.	
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger			
Weather:	Clear	**Exploration Diameter:**	8 in. O.D.			

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.11861°
Longitude: -121.15908°
Surface Condition: Grass

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	TOPSOIL: Grass and organic roots												hand auger to 4 feet
	Clayey SAND (SC): fine-grained sand, low plasticity, dark reddish brown, wet, soft, roots from the adjacent trees	BULK #1											
		BULK #2					17.0						
5	**Sandy CLAY (CL)**: low plasticity, wet, hard, strongly cemented (Hard Pan)												switched to hollow stem auger perched groundwater encountered
	Poorly graded SAND (SP): fine to medium-grained sand, non-plastic, reddish brown, moist, very dense	1		BC=5 6 8	18"								
	Clayey SAND (SC): medium-grained sand, low plasticity, reddish yellow, moist, very dense, weakly cemented	2		BC=8 16 47	18"								
10		3		BC=13 24 29	18"								
15	fine to medium-grained, dark brown	4		BC=13 31 37	18"								
20	**Sandy SILT (ML)**: medium plasticity, light brown gray, moist, hard, trace gravel	5		BC=14 27 39	18"								
	Poorly graded SAND (SP): fine to coarse-grained sand, non-plastic, wet, very dense												
	Silty SAND (SM): fine-grained sand, low plasticity, olive yellow, moist, medium dense												
25		6		BC=5 10 12	12"								
30													

The boring was terminated at approximately 26.5 ft. below ground surface. The boring was backfilled with grout on April 10, 2019.

GROUNDWATER LEVEL INFORMATION:
Perched groundwater was encountered at approximately 4 ft. below ground surface during drilling.
GENERAL NOTES:

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PROJECT NO.: 20193961	BORING LOG B-3		FIGURE
DRAWN BY: JDS			A-5
CHECKED BY: AS	PG&E LOCKEFORD SUBSTATION 12861 EAST KETTLEMAN LANE LODI, CALIFORNIA		
DATE: 4/22/2019			
REVISED: -			PAGE: 1 of 1

Date Begin - End:	4/10/2019	**Drilling Company:**	Gregg Drilling		**BORING LOG B-4**
Logged By:	A. Sadat	**Drill Crew:**	Louie Wens		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	S24/S97/Liftgate	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Clear	**Exploration Diameter:**	8 in. O.D.		

Latitude: 38.11770°
Longitude: -121.15919°
Surface Condition: Grass

Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		TOPSOIL: Grass and organic roots												hand auger to 4 feet 4 inches
		Clayey SAND (SC): low plasticity, dark reddish brown, moist, weakly cemented	Bulk #1											
		Sandy CLAY (CL): low plasticity, reddish brown, moist, hard	Bulk #2					17.0						
5		heavily cemented (Hard Pan)												
		Clayey SAND (SC): fine to coarse-grained sand, low plasticity, yellowish brown, moist, very dense	1		BC=9 27 27	18"								switched to hollow stem auger
		reddish brown, moderately cemented	2		BC=48 27 50/3"	18"								
10		fine-grained sand, yellowish brown, weakly cemented	3		BC=9 23 38	18"								
15		non-plastic, reddish brown, dense, trace gravel, weakly to moderately cemented	4		BC=5 12 17	18"								
20			5		BC=6 12 19	15"								
		Poorly graded SAND (SP): fine to medium-grained sand, non-plastic, yellowish brown, moist, dense												
25		fine to coarse-grained sand, yellowish brown, medium dense	6		BC=6 10 12	18"								
		Poorly graded SAND (SP): fine to coarse-grained sand, non-plastic, olive yellow, moist, medium dense												
30			7		BC=7 11 15	18"								

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with grout on April 10, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.: 20193961	BORING LOG B-4		FIGURE
DRAWN BY: JDS			
CHECKED BY: AS	PG&E LOCKEFORD SUBSTATION 12861 EAST KETTLEMAN LANE LODI, CALIFORNIA		A-6
DATE: 4/22/2019			
REVISED: -			PAGE: 1 of 1

PLOTTED: 05/10/2019 10:32 AM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20193961.001A
gINT FILE: Klf_gint_master_2019
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB [_KLF_BORING/TEST PIT SOIL LOG]

APPENDIX B

LABORATORY TEST RESULTS

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	3.0	Bulk #3	REDDISH BROWN SANDY LEAN CLAY (CL)	19.4					23	13	10	
B-1	5.0	1	REDDISH BROWN SANDY SILTY CLAY (CL-ML)	11.7				56	20	16	4	
B-1	7.5	2	REDDISH BROWN SANDY SILT (ML)					61	20	17	3	
B-2	15.0	4	REDDISH BROWN CLAYEY SAND (SC)					33				
B-3	3.0	BULK #2		17.0								
B-4	3.0	Bulk #2		17.0								

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

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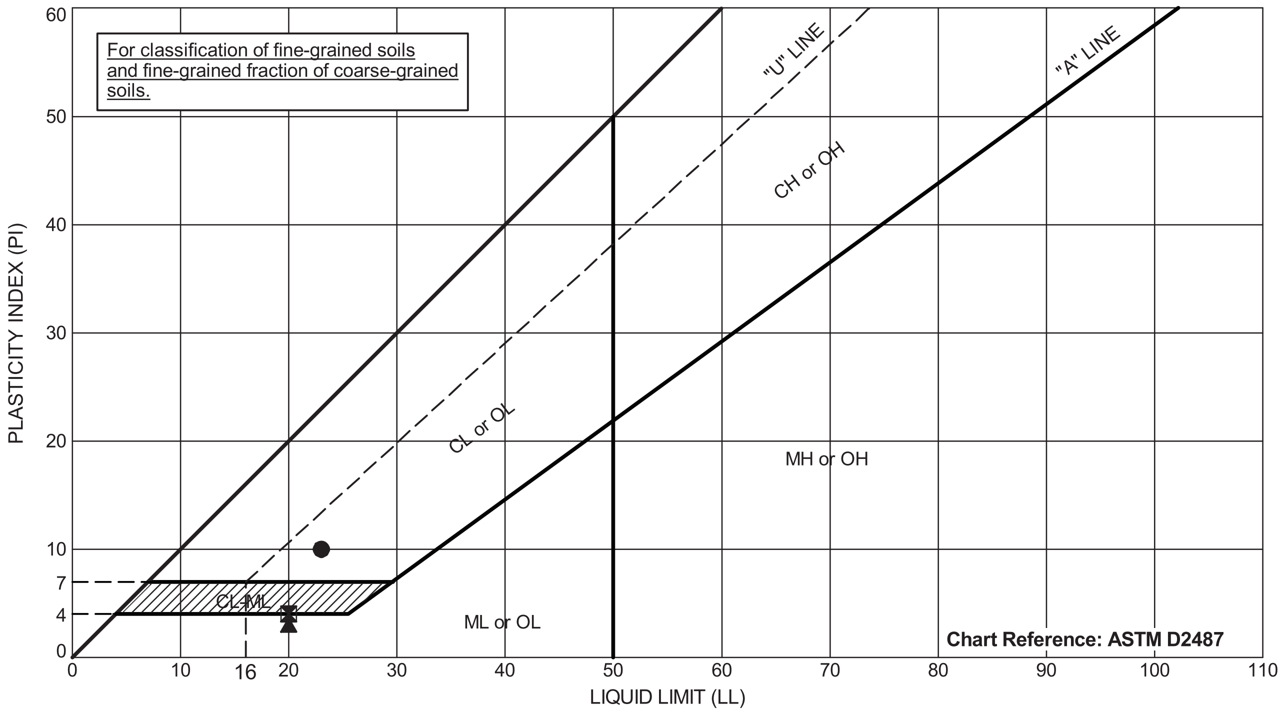
PROJECT NO.:	20193961
DRAWN BY:	JDS
CHECKED BY:	AS
DATE:	4/22/2019
REVISED:	-

LABORATORY TEST
RESULT SUMMARY

PG&E LOCKEFORD SUBSTATION
12861 EAST KETTLEMAN LANE
LODI, CALIFORNIA

FIGURE

B-1



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	B-1	3	Bulk #3	REDDISH BROWN SANDY LEAN CLAY (CL)	NM	23	13	10
⧗	B-1	5	1	REDDISH BROWN SANDY SILTY CLAY (CL-ML)	56	20	16	4
▲	B-1	7.5	2	REDDISH BROWN SANDY SILT (ML)	61	20	17	3

Testing performed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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PROJECT NO.: 20193961	ATTERBERG LIMITS	TABLE
DRAWN BY: JDS		
CHECKED BY: AS	PG&E LOCKEFORD SUBSTATION 12861 EAST KETTLEMAN LANE LODI, CALIFORNIA	B-2
DATE: 4/22/2019		
REVISED: -		

California State Certified Laboratory No. 2153

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

Client: Kleinfelder
Client's Project No.: 20193961.001A
Client's Project Name: PG&E Lockeford Substation
Date Sampled: 04/09 - 10/19
Date Received: 23-Apr-2019
Matrix: Soil
Authorization: Laboratory Testing Program

Date of Report: 26-Apr-2019

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1904167-001	B-1/B-2, All Bulks	+320	6.92	6,500	8,600	N.D.	N.D.	N.D.
1904167-002	B-3/B-4, All Bulks	+340	6.74	6,100	6,400	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	24-Apr-2019	24-Apr-2019	25-Apr-2019	25-Apr-2019	25-Apr-2019	24-Apr-2019	24-Apr-2019

Cheryl McMillen
Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

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

KLEINFELDER

CA1904167 12312 Fed Ex to Cerco

LAB NUMBER: HL12049

LABORATORY TESTING PROGRAM - GEOTECHNICAL

HFattal@kleinfelder.com

925-484-4514

Test Method
- [x] ASTM
- [ ] AASHTO
- [ ] DOT
- [ ] Other (see remarks)

Project: PG&E Lockeford Substation

Date Sampled: 4/9/19 to 4/10/19 | Project No.: 20193961.001A

Date Submitted: 4/17/2019 | Task No.: 04-0001 & 05-0000

Submitted By: Abdul Sadat | Report To: Liana Serrano, H. Fattal

Hadi Fattal

Boring / Test Pit	Sample	Depth, ft.	Sample Type	CLASSIFICATN. VISUAL	WATER CONTENT	WATER/DENSITY, SOIL	SIEVE ANALYSIS,-200	SIEVE ANALYSIS, F	SIEVE ANALYSIS, C&F	HYDROMETER ANALYSIS	ATTERBERG, MULTI-PT.	ATTERBERG, ONE-PT.	SOIL ORGANIC CONTENT	SOIL SPEC. GRAV.	STANDARD PROCTOR, 4"	MODIFIED PROCTOR, 4"	R-VALUE	EXPANSION INDEX	SOIL PH	SOIL RESISTIVITY	CHLORIDE CONTENT	SULFATE CONTENT	SAMPLE MIX&PROCESS	SAMPLE PREPARATION	Remarks
B-1	3	3	Bulk		x						x														Standard ASTM + 5 - 3 redox as rec'd
B-1	1	5	SPT		x		x				x														
B-1	2	7.5	SPT				x				x														(RUSH) TAT
B-1/B-2	All bulks		Bulk																x	x	x	x			Add: Sufide & Redox, composite into ~~one sample~~
B-3/B-4	All Bulks		Bulk																x	x	x	x			Add: Sufide & Redox, composite into ~~one sample~~
B-2	4	15	SPT				x																		
B-1	Bulk 4	4	Bulk		~~x~~						~~x~~														
B-4	Bulk 2	3	Bulk		x																				
B-3	Bulk 2	3	Bulk		x																				
																									Diana Saud 4/23/19 0940
Quantity of Tests																									
Unit				L00220	L00294	L00295	L00267	L00270	L00269	L00248	L00223	L00224	L00271	L00275	L00279	L00251	L00264	L00246	L00272	L00273	L00227	L00281	L00905	L00907	

Boring / Test Pit	Sample	Depth, ft.	Sample Type	UNCONFINED COMP STR.	DIRECT SHEAR, 1 PT.	DIRECT SHEAR, 3 PTS.	DIRECT SHEAR, RESID.	TRIAX. CU, 1 POINT	TRIAX. CU, 3 POINTS	TRIAX CD, 1 POINT	TRIAX CD, 3 POINTS	TRIAX. UU, 1 POINT	TRIAX. UU, 3 POINTS	TRIAX. UU SAT, 1 PT.	TRIAX. UU SAT, 3 PT.	PERMEABILITY, FLEX	PERMEABILITY, RIGID	PINHOLE DISPERSION	CONSOL. W/O TR	CONSOL. W/2 TR	CONSOL. ALL TR	CONSOL. ADD. REBOUND	ONEDIM FREE SWELL	ONEDIM SWELL % HEAVE	ONEDIMSWELL W/CONSOL	SAMPLE REMOLDING	Remarks
Quantity of Tests																											
Unit				L00293	L00242	L00243	L00244	L00286	L00287	L00285	Loo336	L00289	L00290	L00288	L00325	L00258	L00259	L00261	L00233	L00232	L00231	L00230	L00255	L00256	L00257	L00908	

Test requested | Test in progress | Test complete

Rev. 02/17/2014

McCampbell Analytical, Inc.

"When Quality Counts"

Analytical Report

WorkOrder: 1904607

Report Created for: Kleinfelder, Inc.

2601 Barrington Court
Hayward, CA 94545

Project Contact: Hadi Fattal
Project P.O.:
Project: 20193961.001A; Lockeford Substation

Project Received: 04/11/2019

Analytical Report reviewed & approved for release on 04/18/2019 by:

Yen Cao

Yen Cao
Project Manager

The report shall not be reproduced except in full, without the written approval of the laboratory. The analytical results relate only to the items tested. Results reported conform to the most current NELAP standards, where applicable, unless otherwise stated in the case narrative.

NELAP ACCREDITED TNI LABORATORY

1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Glossary of Terms & Qualifier Definitions

Client: Kleinfelder, Inc.
Project: 20193961.001A; Lockeford Substation
WorkOrder: 1904607

Glossary Abbreviation

%D	Serial Dilution Percent Difference
95% Interval	95% Confident Interval
DF	Dilution Factor
DI WET	(DISTLC) Waste Extraction Test using DI water
DISS	Dissolved (direct analysis of 0.45 μm filtered and acidified water sample)
DLT	Dilution Test (Serial Dilution)
DUP	Duplicate
EDL	Estimated Detection Limit
ERS	External reference sample. Second source calibration verification.
ITEF	International Toxicity Equivalence Factor
LCS	Laboratory Control Sample
MB	Method Blank
MB % Rec	% Recovery of Surrogate in Method Blank, if applicable
MDL	Method Detection Limit
ML	Minimum Level of Quantitation
MS	Matrix Spike
MSD	Matrix Spike Duplicate
N/A	Not Applicable
ND	Not detected at or above the indicated MDL or RL
NR	Data Not Reported due to matrix interference or insufficient sample amount.
PDS	Post Digestion Spike
PDSD	Post Digestion Spike Duplicate
PF	Prep Factor
RD	Relative Difference
RL	Reporting Limit (The RL is the lowest calibration standard in a multipoint calibration.)
RPD	Relative Percent Deviation
RRT	Relative Retention Time
SPK Val	Spike Value
SPKRef Val	Spike Reference Value
SPLP	Synthetic Precipitation Leachate Procedure
ST	Sorbent Tube
TCLP	Toxicity Characteristic Leachate Procedure
TEQ	Toxicity Equivalents
TZA	TimeZone Net Adjustment for sample collected outside of MAI's UTC.
WET (STLC)	Waste Extraction Test (Soluble Threshold Limit Concentration)

1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Glossary of Terms & Qualifier Definitions

Client: Kleinfelder, Inc.
Project: 20193961.001A; Lockeford Substation
WorkOrder: 1904607

Analytical Qualifiers

J	Result is less than the RL/ML but greater than the MDL. The reported concentration is an estimated value.

McCampbell Analytical, Inc.
"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Received:	4/11/19 16:16	**Extraction Method:**	SW3550B
Date Prepared:	4/11/19	**Analytical Method:**	SW8081A/8082
Project:	20193961.001A; Lockeford Substation	**Unit:**	mg/kg

Organochlorine Pesticides + PCBs

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Composite	1904607-001A	Soil	04/10/2019 15:00	GC23 04111947.d	176099

Analytes	Result	RL	DF	Date Analyzed
Aldrin	ND	0.0010	1	04/12/2019 02:26
a-BHC	ND	0.0010	1	04/12/2019 02:26
b-BHC	ND	0.0010	1	04/12/2019 02:26
d-BHC	ND	0.0010	1	04/12/2019 02:26
g-BHC	ND	0.0010	1	04/12/2019 02:26
Chlordane (Technical)	ND	0.025	1	04/12/2019 02:26
a-Chlordane	ND	0.0010	1	04/12/2019 02:26
g-Chlordane	ND	0.0010	1	04/12/2019 02:26
p,p-DDD	ND	0.0010	1	04/12/2019 02:26
p,p-DDE	ND	0.0010	1	04/12/2019 02:26
p,p-DDT	ND	0.0010	1	04/12/2019 02:26
Dieldrin	ND	0.0010	1	04/12/2019 02:26
Endosulfan I	ND	0.0010	1	04/12/2019 02:26
Endosulfan II	ND	0.0010	1	04/12/2019 02:26
Endosulfan sulfate	ND	0.0010	1	04/12/2019 02:26
Endrin	ND	0.0010	1	04/12/2019 02:26
Endrin aldehyde	ND	0.0010	1	04/12/2019 02:26
Endrin ketone	ND	0.0010	1	04/12/2019 02:26
Heptachlor	ND	0.0010	1	04/12/2019 02:26
Heptachlor epoxide	ND	0.0010	1	04/12/2019 02:26
Hexachlorobenzene	ND	0.010	1	04/12/2019 02:26
Hexachlorocyclopentadiene	ND	0.020	1	04/12/2019 02:26
Methoxychlor	ND	0.0010	1	04/12/2019 02:26
Toxaphene	ND	0.050	1	04/12/2019 02:26
Aroclor1016	ND	0.050	1	04/12/2019 02:26
Aroclor1221	ND	0.050	1	04/12/2019 02:26
Aroclor1232	ND	0.050	1	04/12/2019 02:26
Aroclor1242	ND	0.050	1	04/12/2019 02:26
Aroclor1248	ND	0.050	1	04/12/2019 02:26
Aroclor1254	ND	0.050	1	04/12/2019 02:26
Aroclor1260	ND	0.050	1	04/12/2019 02:26
PCBs, total	ND	0.050	1	04/12/2019 02:26

Surrogates	REC (%)	Limits		
Decachlorobiphenyl	114	69-143		04/12/2019 02:26

Analyst(s): CN

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Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Received:	4/11/19 16:16	**Extraction Method:**	SW3050B
Date Prepared:	4/11/19	**Analytical Method:**	SW6020
Project:	20193961.001A; Lockeford Substation	**Unit:**	mg/Kg

CAM / CCR 17 Metals

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Composite	**1904607-001A**	**Soil**	**04/10/2019 15:00**	**ICP-MS1 072SMPL.D**	**176125**

Analytes	Result	RL	DF	Date Analyzed
Antimony	ND	0.50	1	04/12/2019 17:15
Arsenic	**2.5**	0.50	1	04/12/2019 17:15
Barium	**81**	5.0	1	04/12/2019 17:15
Beryllium	ND	0.50	1	04/12/2019 17:15
Cadmium	ND	0.25	1	04/12/2019 17:15
Chromium	**25**	0.50	1	04/12/2019 17:15
Cobalt	**6.7**	0.50	1	04/12/2019 17:15
Copper	**9.7**	0.50	1	04/12/2019 17:15
Lead	**4.6**	0.50	1	04/12/2019 17:15
Mercury	ND	0.050	1	04/12/2019 17:15
Molybdenum	ND	0.50	1	04/12/2019 17:15
Nickel	**12**	0.50	1	04/12/2019 17:15
Selenium	ND	0.50	1	04/12/2019 17:15
Silver	ND	0.50	1	04/12/2019 17:15
Thallium	ND	0.50	1	04/12/2019 17:15
Vanadium	**45**	0.50	1	04/12/2019 17:15
Zinc	**29**	5.0	1	04/12/2019 17:15

Surrogates	REC (%)	Limits	
Terbium	125	70-130	04/12/2019 17:15

Analyst(s): MIG

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Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Received:	4/11/19 16:16	**Extraction Method:**	SW5030B
Date Prepared:	4/11/19	**Analytical Method:**	SW8021B/8015Bm
Project:	20193961.001A; Lockeford Substation	**Unit:**	mg/Kg

Gasoline Range (C6-C12) Volatile Hydrocarbons as Gasoline with BTEX and MTBE

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Composite	**1904607-001A**	**Soil**	**04/10/2019 15:00**	**GC19 04111925.D**	**176084**

Analytes	Result	RL	DF	Date Analyzed
TPH(g) (C6-C12)	ND	1.0	1	04/12/2019 01:01
MTBE	ND	0.050	1	04/12/2019 01:01
Benzene	ND	0.0050	1	04/12/2019 01:01
Toluene	ND	0.0050	1	04/12/2019 01:01
Ethylbenzene	ND	0.0050	1	04/12/2019 01:01
m,p-Xylene	ND	0.010	1	04/12/2019 01:01
o-Xylene	ND	0.0050	1	04/12/2019 01:01
Xylenes	ND	0.0050	1	04/12/2019 01:01

Surrogates	REC (%)	Limits		
2-Fluorotoluene	85	62-126		04/12/2019 01:01

Analyst(s): IA

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Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Received:	4/11/19 16:16	**Extraction Method:**	SW9045C
Date Prepared:	4/11/19	**Analytical Method:**	SW9045C
Project:	20193961.001A; Lockeford Substation	**Unit:**	pH units @ 25°C

pH

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Composite	1904607-001A	Soil	04/10/2019 15:00	WetChem	176123

Analytes	Result	Accuracy	DF	Date Analyzed
pH	**7.76**	±0.1	1	04/11/2019 20:34

Analyst(s): PHU

Analytical Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Received:	4/11/19 16:16	**Extraction Method:**	SW3550B
Date Prepared:	4/11/19	**Analytical Method:**	SW8015B
Project:	20193961.001A; Lockeford Substation	**Unit:**	mg/Kg

Total Extractable Petroleum Hydrocarbons w/out SG Clean-Up

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
Composite	**1904607-001A**	**Soil**	**04/10/2019 15:00**	**GC39A 04111946.D**	**176146**

Analytes	Result	RL	DF	Date Analyzed
TPH-Diesel (C10-C23)	ND	1.0	1	04/12/2019 00:38
TPH-Motor Oil (C18-C36)	ND	5.0	1	04/12/2019 00:38

Surrogates	REC (%)	Limits		
C9	98	74-123		04/12/2019 00:38

Analyst(s): JIS

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Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Prepared:	4/11/19	**BatchID:**	176099
Date Analyzed:	4/11/19	**Extraction Method:**	SW3550B
Instrument:	GC20, GC23	**Analytical Method:**	SW8081A/8082
Matrix:	Soil	**Unit:**	mg/kg
Project:	20193961.001A; Lockeford Substation	**Sample ID:**	MB/LCS/LCSD-176099

QC Summary Report for SW8081A/8082

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
Aldrin	ND	0.00027	0.0010	-	-	-
a-BHC	ND	0.00010	0.0010	-	-	-
b-BHC	ND	0.00025	0.0010	-	-	-
d-BHC	ND	0.00037	0.0010	-	-	-
g-BHC	ND	0.000097	0.0010	-	-	-
Chlordane (Technical)	ND	0.016	0.025	-	-	-
a-Chlordane	ND	0.00047	0.0010	-	-	-
g-Chlordane	ND	0.00021	0.0010	-	-	-
p,p-DDD	ND	0.00014	0.0010	-	-	-
p,p-DDE	ND	0.00032	0.0010	-	-	-
p,p-DDT	ND	0.00043	0.0010	-	-	-
Dieldrin	ND	0.00033	0.0010	-	-	-
Endosulfan I	ND	0.00065	0.0010	-	-	-
Endosulfan II	ND	0.00020	0.0010	-	-	-
Endosulfan sulfate	ND	0.00063	0.0010	-	-	-
Endrin	ND	0.00042	0.0010	-	-	-
Endrin aldehyde	ND	0.00020	0.0010	-	-	-
Endrin ketone	ND	0.00013	0.0010	-	-	-
Heptachlor	ND	0.00021	0.0010	-	-	-
Heptachlor epoxide	ND	0.00020	0.0010	-	-	-
Hexachlorobenzene	ND	0.00027	0.010	-	-	-
Hexachlorocyclopentadiene	ND	0.00040	0.020	-	-	-
Methoxychlor	ND	0.00089	0.0010	-	-	-
Toxaphene	ND	0.035	0.050	-	-	-
Aroclor1016	ND	0.0051	0.050	-	-	-
Aroclor1221	ND	0.011	0.050	-	-	-
Aroclor1232	ND	0.0063	0.050	-	-	-
Aroclor1242	ND	0.0067	0.050	-	-	-
Aroclor1248	ND	0.0040	0.050	-	-	-
Aroclor1254	ND	0.0068	0.050	-	-	-
Aroclor1260	ND	0.0061	0.050	-	-	-
PCBs, total	ND	N/A	0.050	-	-	-
Surrogate Recovery						
Decachlorobiphenyl	0.061			0.050	121	75-136

(Cont.)

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Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Prepared:	4/11/19	**BatchID:**	176099
Date Analyzed:	4/11/19	**Extraction Method:**	SW3550B
Instrument:	GC20, GC23	**Analytical Method:**	SW8081A/8082
Matrix:	Soil	**Unit:**	mg/kg
Project:	20193961.001A; Lockeford Substation	**Sample ID:**	MB/LCS/LCSD-176099

QC Summary Report for SW8081A/8082

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
Aldrin	0.060	0.060	0.050	120	120	92-133	0	20
a-BHC	0.064	0.065	0.050	129	129	96-140	0	20
b-BHC	0.056	0.056	0.050	113	113	77-137	0	20
d-BHC	0.060	0.060	0.050	120	120	89-145	0	20
g-BHC	0.060	0.060	0.050	119	119	92-134	0	20
a-Chlordane	0.057	0.057	0.050	115	114	72-134	0.315	20
g-Chlordane	0.059	0.059	0.050	118	118	86-132	0	20
p,p-DDD	0.046	0.046	0.050	92	91	35-140	1.11	20
p,p-DDE	0.057	0.057	0.050	114	115	83-138	0.769	20
p,p-DDT	0.053	0.053	0.050	105	106	70-137	0.655	20
Dieldrin	0.065	0.065	0.050	129	129	99-141	0	20
Endosulfan I	0.057	0.057	0.050	114	115	93-121	0.0973	20
Endosulfan II	0.054	0.054	0.050	108	108	74-125	0	20
Endosulfan sulfate	0.060	0.061	0.050	120	122	66-138	1.09	20
Endrin	0.061	0.061	0.050	121	121	92-137	0	20
Endrin aldehyde	0.060	0.060	0.050	120	121	77-135	0.670	20
Endrin ketone	0.056	0.056	0.050	113	113	72-126	0	20
Heptachlor	0.058	0.058	0.050	116	115	89-136	0.169	20
Heptachlor epoxide	0.055	0.055	0.050	111	111	85-121	0	20
Hexachlorobenzene	0.052	0.052	0.050	105	105	87-127	0	20
Hexachlorocyclopentadiene	0.035	0.037	0.050	69	74	41-145	7.08	20
Methoxychlor	0.049	0.050	0.050	99	100	82-142	1.23	20
Aroclor1016	0.15	0.14	0.15	103	95	90-125	7.68	20
Aroclor1260	0.16	0.16	0.15	108	107	77-122	0.820	20
Surrogate Recovery								
Decachlorobiphenyl	0.065	0.065	0.050	130	131	75-136	0.440	20

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Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Prepared:	4/11/19	**BatchID:**	176125
Date Analyzed:	4/11/19	**Extraction Method:**	SW3050B
Instrument:	ICP-MS1	**Analytical Method:**	SW6020
Matrix:	Soil	**Unit:**	mg/Kg
Project:	20193961.001A; Lockeford Substation	**Sample ID:**	MB/LCS/LCSD-176125

QC Summary Report for Metals

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
Antimony	ND	0.094	0.50	-	-	-
Arsenic	ND	0.14	0.50	-	-	-
Barium	ND	0.97	5.0	-	-	-
Beryllium	ND	0.072	0.50	-	-	-
Cadmium	ND	0.058	0.25	-	-	-
Chromium	ND	0.092	0.50	-	-	-
Cobalt	ND	0.056	0.50	-	-	-
Copper	ND	0.069	0.50	-	-	-
Lead	ND	0.094	0.50	-	-	-
Mercury	ND	0.0050	0.050	-	-	-
Molybdenum	ND	0.23	0.50	-	-	-
Nickel	ND	0.072	0.50	-	-	-
Selenium	ND	0.13	0.50	-	-	-
Silver	ND	0.055	0.50	-	-	-
Thallium	ND	0.10	0.50	-	-	-
Vanadium	ND	0.064	0.50	-	-	-
Zinc	ND	1.4	5.0	-	-	-
Surrogate Recovery						
Terbium	500			500	99	70-130

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Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Prepared:	4/11/19	**BatchID:**	176125
Date Analyzed:	4/11/19	**Extraction Method:**	SW3050B
Instrument:	ICP-MS1	**Analytical Method:**	SW6020
Matrix:	Soil	**Unit:**	mg/Kg
Project:	20193961.001A; Lockeford Substation	**Sample ID:**	MB/LCS/LCSD-176125

QC Summary Report for Metals

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
Antimony	54	55	50	107	110	75-125	2.50	20
Arsenic	49	50	50	98	99	75-125	1.34	20
Barium	500	510	500	99	102	75-125	2.41	20
Beryllium	49	50	50	99	100	75-125	1.17	20
Cadmium	48	48	50	97	96	75-125	1.02	20
Chromium	48	49	50	97	97	75-125	0	20
Cobalt	48	49	50	97	98	75-125	1.53	20
Copper	49	48	50	98	97	75-125	0.927	20
Lead	49	49	50	97	99	75-125	1.68	20
Mercury	1.2	1.3	1.25	99	102	75-125	3.35	20
Molybdenum	49	51	50	99	101	75-125	2.60	20
Nickel	48	48	50	96	96	75-125	0	20
Selenium	50	50	50	99	101	75-125	1.76	20
Silver	48	49	50	95	97	75-125	1.83	20
Thallium	47	48	50	94	96	75-125	2.12	20
Vanadium	49	49	50	97	98	75-125	0.205	20
Zinc	490	490	500	98	98	75-125	0	20
Surrogate Recovery								
Terbium	510	530	500	103	105	70-130	2.46	20

Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Prepared:	4/10/19	**BatchID:**	176084
Date Analyzed:	4/11/19	**Extraction Method:**	SW5030B
Instrument:	GC3	**Analytical Method:**	SW8021B/8015Bm
Matrix:	Soil	**Unit:**	mg/Kg
Project:	20193961.001A; Lockeford Substation	**Sample ID:**	MB/LCS/LCSD-176084

QC Summary Report for SW8021B/8015Bm

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
TPH(g) (C6-C12)	0.22,J	0.090	1.0	-	-	-
MTBE	ND	0.0023	0.050	-	-	-
Benzene	ND	0.0010	0.0050	-	-	-
Toluene	ND	0.0012	0.0050	-	-	-
Ethylbenzene	ND	0.0020	0.0050	-	-	-
m,p-Xylene	ND	0.0013	0.010	-	-	-
o-Xylene	ND	0.0013	0.0050	-	-	-
Surrogate Recovery						
2-Fluorotoluene	0.098			0.10	98	75-134

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
TPH(btex)	0.60	0.70	0.60	101	116	82-118	14.1	20
MTBE	0.092	0.088	0.10	92	88	61-119	4.26	20
Benzene	0.10	0.10	0.10	104	102	77-128	1.96	20
Toluene	0.11	0.11	0.10	108	107	74-132	0.269	20
Ethylbenzene	0.10	0.10	0.10	105	105	84-127	0	20
m,p-Xylene	0.21	0.21	0.20	106	107	80-120	0.223	20
o-Xylene	0.10	0.10	0.10	100	102	80-120	2.10	20
Surrogate Recovery								
2-Fluorotoluene	0.098	0.097	0.10	98	97	75-134	1.03	20

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Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Prepared:	4/11/19	**BatchID:**	176123
Date Analyzed:	4/11/19	**Extraction Method:**	SW9045C
Instrument:	WetChem	**Analytical Method:**	SW9045C
Matrix:	Water	**Unit:**	pH units @ 25°C
Project:	20193961.001A; Lockeford Substation	**Sample ID:**	CCV-176123

QC Summary Report for pH

Analyte	CCV Result	CCV Limits
pH	6.99	6.8-7.2

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Quality Control Report

Client:	Kleinfelder, Inc.	**WorkOrder:**	1904607
Date Prepared:	4/11/19	**BatchID:**	176146
Date Analyzed:	4/12/19	**Extraction Method:**	SW3550B
Instrument:	GC39A, GC39B	**Analytical Method:**	SW8015B
Matrix:	Soil	**Unit:**	mg/Kg
Project:	20193961.001A; Lockeford Substation	**Sample ID:**	MB/LCS/LCSD-176146 1904607-001AMS/MSD

QC Report for SW8015B w/out SG Clean-Up

Analyte	MB Result	MDL	RL	SPK Val	MB SS %REC	MB SS Limits
TPH-Diesel (C10-C23)	ND	0.83	1.0	-	-	-
TPH-Motor Oil (C18-C36)	ND	3.8	5.0	-	-	-
Surrogate Recovery						
C9	23			25	93	72-122

Analyte	LCS Result	LCSD Result	SPK Val	LCS %REC	LCSD %REC	LCS/LCSD Limits	RPD	RPD Limit
TPH-Diesel (C10-C23)	38	38	40	95	96	75-128	0.204	30
Surrogate Recovery								
C9	23	23	25	91	91	72-122	0	30

Analyte	MS DF	MS Result	MSD Result	SPK Val	SPKRef Val	MS %REC	MSD %REC	MS/MSD Limits	RPD	RPD Limit
TPH-Diesel (C10-C23)	1	39	41	40	ND	98	102	71-134	4.22	30
Surrogate Recovery										
C9	1	24	24	25		97	97	78-126	0	30

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CHAIN-OF-CUSTODY RECORD

WorkOrder: 1904607 **ClientCode: KFH**

☐ WaterTrax ☐ WriteOn ☐ EDF ☐ Excel ☒ EQuIS ☐ Email ☐ HardCopy ☐ ThirdParty ☐ J-flag
☒ Detection Summary ☒ Dry-Weight

Report to:
Hadi Fattal
Kleinfelder, Inc.
2601 Barrington Court
Hayward, CA 94545
(916) 366-1701 FAX: 510-887-5932

Email: hfattal@kleinfelder.com
cc/3rd Party:
PO:
Project: 20193961.001A; Lockeford Substation

Bill to:
Accounts Payable
Kleinfelder, Inc.
550 West C Street, Ste. 1200
San Diego, CA 92101
AccountsPayableUS@kleinfelder.com

Requested TAT: **5 days;**

Date Received: **04/11/2019**
Date Logged: **04/11/2019**

Lab ID	Client ID	Matrix	Collection Date	Hold	Requested Tests (See legend below) 1	2	3	4	5	6	7	8	9	10	11	12
1904607-001	Composite	Soil	4/10/2019 15:00	☐	A	A	A	A	A	A						

Test Legend:

1	8081PCB_S	2	CAM17MS_TTLC_S	3	G-MBTEX_S	4	PH_S
5	STLC_MSEXTRACTONLY	6	TPH(DMO)_S	7		8	
9		10		11		12	

Project Manager: Yen Cao

Prepared by: Nancy Palacios

Comments:

NOTE: Soil samples are discarded 60 days after results are reported unless other arrangements are made (Water samples are 30 days).
Hazardous samples will be returned to client or disposed of at client expense.

McCampbell Analytical, Inc.
"When Quality Counts"

1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

WORK ORDER SUMMARY

Client Name: KLEINFELDER, INC.
Client Contact: Hadi Fattal
Contact's Email: hfattal@kleinfelder.com

Project: 20193961.001A; Lockeford Substation
Comments:

Work Order: 1904607
QC Level: LEVEL 2
Date Logged: 4/11/2019

☐ WaterTrax ☐ WriteOn ☐ EDF ☐ Excel ☒ EQuIS ☐ Email ☐ HardCopy ☐ ThirdParty ☐ J-flag

Lab ID	Client ID	Matrix	Test Name	Containers /Composites	Bottle & Preservative	De-chlorinated	Collection Date & Time	TAT	Sediment Content	Hold	SubOut
1904607-001A	Composite	Soil	SW8015B (Diesel & Motor Oil)	4 / (4:1)	16OZ GJ, Unpres	☐	4/10/2019 15:00	5 days		☐	
			STLC (rotated) Extraction Only			☐		5 days*		☐	
			SW9045C (pH)			☐		5 days		☐	
			SW8021B/8015Bm (G/MBTEX)			☐		5 days		☐	
			SW6020 (CAM 17)			☐		5 days		☐	
			SW8081A/8082 (OC Pesticides+PCBs)			☐		5 days		☐	

NOTES:
- **STLC and TCLP extractions require 2 days to complete; therefore, all TATs begin after the extraction is completed (i.e., One-day TAT yields results in 3 days from sample submission).**
- **MAI assumes that all material present in the provided sampling container is considered part of the sample - MAI does not exclude any material from the sample prior to sample preparation unless requested in writing by the client.**

General COC

MAI Work Order # 1904607

McCAMPBELL ANALYTICAL, INC.

1534 Willow Pass Rd. Pittsburg, Ca. 94565-1701
Telephone: (877) 252-9262 / Fax: (925) 252-9269
www.mccampbell.com main@mccampbell.com

CHAIN OF CUSTODY RECORD

Turn Around Time: 1 Day Rush ☐ 2 Day Rush ☐ 3 Day Rush ☐ STD X Quote #
J-Flag / MDL ☐ ESL ☐ Cleanup Approved ☐ Bottle Order #
Delivery Format: PDF ☐ GeoTracker EDF ☐ EDD ☐ Write On (DW) ☐ EQuIS ☐

Report To: Hadi Fattal
Bill To:
Company: Kleinfelder
Email: HFattal@Kleinfelder.com
Alt Email:
Tele:
Project Name: Lockeford Substation
Project #: 20193961.001A
Project Location: 12861 E. Kettleman Lane
PO# Lodi, CA
Sampler Signature: [signature]

Analysis Requested

SAMPLE ID Location / Field Point	Sampling Date	Sampling Time	#Containers	Matrix	Preservative	BTEX & TPH as Gas (8021/ 8015) MTBE	TPH as Diesel (8015) + Motor Oil Without Silica Gel	TPH as Diesel (8015) + Motor Oil With Silica Gel	Total Oil & Grease (1664 / 9071) Without Silica Gel	Total Petroleum Hydrocarbons - Oil & Grease (1664 / 9071) With Silica Gel	Total Petroleum Hydrocarbons (418.1) With Silica Gel	EPA 505/ 608 / 8081 (Cl Pesticides)	EPA 608 / 8082 PCB's ; Aroclors only	EPA 524.2 / 624 / 8260 (VOCs)	EPA 525.2 / 625 / 8270 (SVOCs)	EPA 8270 SIM / 8310 (PAHs / PNAs)	CAM 17 Metals (200.8 / 6020)*	Metals (200.8 / 6020)	Baylands Requirements	Lab to filter sample for dissolved metals analysis	PH	STLC (6010B) or equivalent		
Composite	4/9/19 4/10/19	2:45pm 3:00pm	4	Soil	Yes	X	X					X	X				X				X	X		

MAI clients MUST disclose any dangerous chemicals known to be present in their submitted samples in concentrations that may cause immediate harm or serious future health endangerment as a result of brief, gloved, open air, sample handling by MAI staff. Non-disclosure incurs an immediate $250 surcharge and the client is subject to full legal liability for harm suffered. Thank you for your understanding and for allowing us to work safely.

* If metals are requested for water samples and the water type (Matrix) is not specified on the chain of custody, MAI will default to metals by E200.8.

Please provide an adequate volume of sample. If the volume is not sufficient for a MS/MSD a LCS/LCSD will be prepared in its place and noted in the report.

Relinquished By / Company Name	Date	Time	Received By / Company Name	Date	Time
[signature] / Kleinfelder	4/11/19	4:15PM	[signature]	4/11/19	16:16

Comments / Instructions: 4- Point Composite

Matrix Code: DW=Drinking Water, GW=Ground Water, WW=Waste Water, SW=Seawater, S=Soil, SL=Sludge, A=Air, WP=Wipe, O=Other

Preservative Code: 1=4°C 2=HCl 3=H_2SO_4 4=HNO_3 5=NaOH 6=ZnOAc/NaOH 7=None

Temp 6.8 °C Initials TP

Page ___ of ___

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Sample Receipt Checklist

Client Name:	**Kleinfelder, Inc.**	Date and Time Received	**4/11/2019 16:16**
Project:	**20193961.001A; Lockeford Substation**	Date Logged:	**4/11/2019**
		Received by:	Tina Perez
WorkOrder №:	**1904607** Matrix: Soil	Logged by:	Nancy Palacios
Carrier:	Client Drop-In		

Chain of Custody (COC) Information

	Yes	No	NA
Chain of custody present?	☑	☐	
Chain of custody signed when relinquished and received?	☑	☐	
Chain of custody agrees with sample labels?	☑	☐	
Sample IDs noted by Client on COC?	☑	☐	
Date and Time of collection noted by Client on COC?	☑	☐	
Sampler's name noted on COC?	☑	☐	
COC agrees with Quote?	☐	☐	☑

Sample Receipt Information

	Yes	No	NA
Custody seals intact on shipping container/cooler?	☐	☐	☑
Shipping container/cooler in good condition?	☑	☐	
Samples in proper containers/bottles?	☑	☐	
Sample containers intact?	☑	☐	
Sufficient sample volume for indicated test?	☑	☐	

Sample Preservation and Hold Time (HT) Information

	Yes	No	NA
All samples received within holding time?	☑	☐	☐
Samples Received on Ice?	☑	☐	
(Ice Type: WET ICE)			
Sample/Temp Blank temperature	Temp: 6.8°C		☐
Water - VOA vials have zero headspace / no bubbles?	☐	☐	☑
Sample labels checked for correct preservation?	☑	☐	
pH acceptable upon receipt (Metal: <2; Nitrate 353.2/4500NO3: <2; 522: <4; 218.7: >8)?	☐	☐	☑
UCMR Samples:			
pH tested and acceptable upon receipt (200.8: ≤2; 525.3: ≤4; 530: ≤7; 541: <3; 544: <6.5 & 7.5)?	☐	☐	☑
Free Chlorine tested and acceptable upon receipt (<0.1mg/L)?	☐	☐	☑

Comments:

APPENDIX C

GBA INFORMATION SHEET

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

H2

PG&E Thurman Switching Station Site
Geotechnical Investigation Report

GEOTECHNICAL INVESTIGATION REPORT
PG&E THURMAN SWITCHING STATION
1215 EAST THURMAN ROAD
LODI, CALIFORNIA

PROJECT NO. 20193892.001A

JUNE 27, 2019

June 27, 2019
Project No. 20193892.001A

Pacific Gas and Electric Company
6111 Bollinger Canyon Road, Room 2460-A
San Ramon, CA 94583

Attention: Grant Wilcox, PE, PG, CEG
Grant.wilcox@pge.com

Joseph Sun, PhD, PE, GE
jis4@pge.com

SUBJECT: Geotechnical Investigation Report

PROJECT: PG&E Northern San Joaquin Reinforcement – Thurman Switching Station
PG&E Order No. / Operation Code: 74000935/3750
1215 East Thurman Road
Lodi, California

Dear Mr. Wilcox and Dr. Sun:

The attached report presents the results of Kleinfelder's geotechnical investigation for the Northern San Joaquin Reinforcement at the Thurman Switching Station, located in Lodi, California. The report describes the study, findings, conclusions, and recommendations for use in project design and construction. Kleinfelder's services are authorized by our proposal dated February 26, 2019 and revised on March 6, 2019 and were performed in general accordance with the terms of our Master Services Agreement No. 4400007810.

The primary geotechnical concern at this site is shallow foundation support and potential caving of drilled pier excavations due to the loose to medium dense sand soils encountered in the upper 5 feet of all borings performed outside the existing substation. Based on the information gathered during this study, it is Kleinfelder's professional opinion that the subject site is geotechnically suitable for construction of the proposed improvements using conventional grading and shallow and deep foundation systems. Recommendations for shallow slab, spread footing, and drilled pier foundations are provided in this report. The recommendations presented herein should be incorporated into project design and construction

Recommendations for design of foundations, site grading, and other geotechnical considerations are presented in this report. The recommendations presented in this report should be incorporated into project design and construction. Kleinfelder appreciates the opportunity to provide geotechnical engineering services to PG&E during the design phase of this project. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully Submitted,

KLEINFELDER, INC.

Hadi Fattal, EIT
Staff Engineer

Stephen Plauson, PE, GE
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 9/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

CC: Kris Johnson (kjjohnson@kleinfelder.com)
Liana Serrano (lserrano@kleinfelder.com)

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APPENDIX A – FIELD EXPLORATION

APPENDIX B – LABORATORY TEST RESULTS

APPENDIX C – INFILTRATION TEST DATA

APPENDIX D – GBA INFORMATION SHEET

1 INTRODUCTION

This report presents the results of a geotechnical investigation conducted for the Northern San Joaquin Reinforcement at the PG&E Thurman Switching Station, located at 1215 East Thurman Road, in Lodi, California. An exploration location plan and vicinity map are shown on Figure 1. Kleinfelder was retained by PG&E to provide geotechnical engineering services for the project. The purpose of the investigation was to evaluate the subsurface conditions at the site and develop geotechnical engineering recommendations to aid in project design and construction.

1.1 PROPOSED CONSTRUCTION

Project understanding is based on the Geotechnical Investigation Request (GIR) dated January 17, 2019 and email and telephone correspondence with Grant Wilcox and Joseph Sun through March 1, 2019. We understand that PG&E plans to expand the existing Thurman Switching Station as part of the Northern San Joaquin Reinforcement Project. The expansion will include construction of one (1) SMP building, one (1) battery building, four BAAH 230 kV bus arrangements, and the connection of two (2) 230kV lines and 230kV connections/provisions to a third-party facility (City of Lodi) for two (2) transmission autotransformers. At this time, foundation loading and dimensions for the aforementioned structures has not been provided.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore and evaluate subsurface conditions at the site and develop geotechnical conclusions and recommendations for use in project design, specification development, and construction. To accomplish these purposes, Kleinfelder's scope of services includes the following:

- Review of existing geologic and geotechnical data for the site vicinity.
- Drilling and sampling of five soil borings to explore subsurface conditions and to obtain samples for laboratory testing, as well as one percolation test.
- Laboratory testing of selected samples to assess pertinent geotechnical properties.
- Evaluation of the available data to develop conclusions and recommendations to guide geotechnical aspects of design and construction.
- Preparation of this report.

Environmental evaluations and analyses, including detailed review of possible contaminants in the foundation soils, are outside of our scope of services.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

Prior to subsurface exploration, exploration locations were marked, and Underground Service Alert (USA) was contacted to provide utility clearance in the public right-of-way. A project-specific safety plan (PSSP) was prepared for the field exploration activities. This plan was discussed with the field crews prior to the start of field exploration work.

2.1.1 Exploratory Borings

Five borings, labeled B-1 through B-5, were drilled by Taber Drilling of Sacremento, California using a CME-55 drill rig equipped with both solid stem and hollow stem augers. Approximate exploration locations are shown on Figure 1. Exploration locations were designated in the field by measuring from existing landmarks. Horizontal coordinates and elevations of the borings were not surveyed.

The borings were drilled on May 23, 24, and 28, 2019. Borings B-1, B-2, B-3, and B-5 were drilled to a depth of approximately 31 ½ feet below ground surface, and B-4 was drilled to a depth of 51 ½ feet below ground surface.

Logs of the borings are provided in Appendix A. Our borings were cleared to a depth of about 5 feet below the ground surface using hand auger methods to confirm the absence of a utilities or other buried conflicts. Borings B-1, 2, 3, and 5 were drilled using solid stem auger methods from depths of about 5 feet to 31 ½ feet below ground surface, while B-4 was drilled using hollow stem auger methods, from depth of about 5 feet to 51 ½ feet below ground surface.

The borings were located in the field by measuring from existing landmarks. Horizontal coordinates and elevations of the borings were not surveyed. A Kleinfelder field-engineer maintained logs of the borings, visually classified the soils encountered per the Unified Soil Classification System (presented on Figure A-3 through A-6 in Appendix A) and obtained samples of the subsurface materials. Soil classifications made in the field from samples and auger cuttings were made in accordance with ASTM D2488. These classifications were re-evaluated in the laboratory after further examination and testing in accordance with ASTM D2487. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring logs. The blow counts listed on the boring logs are raw values and have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency.

Keys to the soil descriptions and symbols used on the boring logs are presented on Figures A-1, A-2 of Appendix A.

2.1.2 Sampling Procedures

Below the hand auger depth, soil samples were collected from the borings at depth intervals of approximately 2½ to 5 feet. Samples were collected from the borings at selected depths by driving either a 2.5-inch inside diameter (I.D.) California sampler or a 1.4-inch I.D. Standard Penetration Test (SPT) sampler driven 18 inches (unless otherwise noted) into undisturbed soil. The samplers were driven using a 140-pound automatic hammer free-falling a distance of 30 inches. Blow counts were recorded at 6-inch intervals for each sample attempt and are reported on the logs. Near-surface bulk samples were also obtained from auger cuttings.

The SPT sampler did not contain liners. The 2.5-inch I.D. California sampler contained stainless steel liners. The California sampler was in general conformance with ASTM D3550. The SPT sampler was in conformance with ASTM D1586.

Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance. Following drilling, the samples were returned to our laboratory for further examination and testing. After the borings were completed they were backfilled with cement grout and approved in the field by an inspector from San Joaquin County. Drilling spoils were off-hauled in 55-gallon drums to be disposed of by our drilling subcontractor. Given the uniqueness of this project, in that the project site is not owned by PG&E, the protocol for performing analytical testing wasn't required, and thereby not performed.

2.1.3 Infiltration Testing

On May 23, 2019, a single percolation test was performed. One boring, PT-1, was excavated using hand auger methods to a depth of approximately 5 feet in the area suggested by PG&E. Percolation testing was performed using a Model 2840K2 Aardvark Permeameter, a digital scale, and a laptop in general accordance with ASTM D5126.

A constant head percolation test was performed using the scale to measure the rate of water infiltration over time. Automatic readings were recorded within the Aardvark Permeameter Module at 1-minute intervals, until a stabilized percolation rate was reached.

The approximate testing location is shown in Figure 1. Upon completion of the percolation testing, the borings were backfilled with auger cuttings. Results from the percolation test are discussed in Section 4.3 of this report and are included in Appendix C.

2.2 LABORATORY TESTING

Laboratory tests were performed on selected samples to evaluate the physical and engineering properties of the materials encountered. Tests included the following:

- Percent passing the No. 200 sieve (ASTM D1140)
- Atterberg limits (ASTM D4318)
- Natural water content (ASTM D2216)
- Unconsolidated Undrained Compression (ASTM D2850)
- Corrosion Suite:
  - Soluble Sulfate Content (ASTM D4327)
  - Soluble Chloride Content (ASTM D4327)
  - pH (ASTM D4972)
  - Minimum Resistivity (ASTM G57)
  - Redox (ASTM D1498)
  - Sulfide (ASTM D4658)

Results of most of the laboratory tests are included on the boring logs in Appendix A. Complete laboratory test data are presented in Appendix B.

3 GEOLOGIC CONDITIONS

3.1 AREA AND SITE GEOLOGY

The site is located along the central section of the Great Valley geomorphic province 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 mountain ranges were initially formed by regional 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. The deepest and oldest of the sediments that fill the structural trough are marine sediments deposited before the uplift of the Coast Ranges. A mix of marine and continental deposits formed over these older units as seas advanced and retreated in the Sacramento and San Joaquin Valleys. The upper and youngest sediments in the basin are continental deposits consisting of alluvial fan deposits and flood-basin, lake, and marsh deposits.

According to geologic mapping by Marchand and Bartow (1979), the substation area is underlain by Quaternary aged terrace and alluvial fan deposits of the Upper Modesto and Lower Riverbank formations. In the project area, these soils generally consist of silts, sands, and gravels with minor clays, which are relatively comparable to the mapped deposits. Regional groundwater levels in the area are greater than 70 feet deep based on DWR well records near the site.

3.2 LOCAL AND REGIONAL FAULTING

The substation is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site. The nearest zoned faults to the project site are the Greenville fault (located about 32 miles to the southwest), Calavaras fault (located about 44 miles to the southwest), Hayward fault (located about 52 miles to the southwest), and San Andreas fault zone (located about 71 miles to the southwest).

4 SITE CONDITIONS

4.1 SITE AND SURFACE DESCRIPTION

The existing Thurman Switching Station is located at 1215 East Thurman Road in Lodi, California. The area of exploration is located within an undeveloped area, directly west of the existing substation at the aforementioned address. The site is bordered by developed commercial property on all sides, with East Thurman Road to its south, and rail lines with East Lodi Avenue beyond to the north. The exploration area is relatively level and covered with dry grass.

4.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered in our borings are in general agreement with the mapped geology. The following description provides a general summary of the subsurface conditions encountered during this study. For more thorough descriptions of the actual conditions encountered at specific boring locations, refer to the boring logs located in Appendix A.

Approximately ½ foot of topsoil was encountered at the surface of all the boring locations. The topsoil was underlain by a variation of silty sand, poorly graded sand, and clayey sands to the final depth of 31 ½ feet, at B-1 through 5. At B-4, below 31 ½ feet, poorly graded sands were encountered to the final depth of 51 ½ feet. Apparent densities were loose in the upper 5 feet, followed by a range of medium dense to dense, with a relatively consistent increase in density with subsequent depth.

4.3 INFILTRATION RATE

A summary table showing the stabilized water percolation rates and corresponding saturated hydraulic conductivity values, K_{sat}, are presented in Table 4.1. Detailed test data is presented in Appendix C. The field percolation rates measured are based on the poorly-graded sand soil conditions encountered at the location of the test. The percolation rate is anticipated to differ throughout the site due to the various soil layers present, such as silts, sands and gravels.

Table 4.1
Hydraulic Conductivity, K_{sat}

Test Location ID	Hole Depth (ft)	Steady Flow Rate (ml/min)	Hydraulic Conductivity, K_{sat} (cm/sec)
PT-1	5	16.5	3.69×10^{-5}

4.4 GROUNDWATER

According to regional well record data published by the State Water Resources Control Board (https://www.waterboards.ca.gov/), regional groundwater levels are generally greater than 70 feet below the ground surface. Regional groundwater was not encountered during our explorations.

It is possible that groundwater conditions at the site could change due to variations in rainfall and runoff, regional groundwater withdrawal or recharge, construction activities, or other factors not apparent at the time the study was performed.

4.5 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions at the site are based on the conditions encountered in the borings drilled for this project. The conclusions and recommendations that follow are based on those interpretations. If soil or groundwater conditions exposed during construction vary from those presented in this report, Kleinfelder should be notified to evaluate whether our conclusions or recommendations should be modified.

5 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, the proposed construction is feasible provided the recommendations presented in this report are incorporated into the project design and construction. The following sections discuss conclusions and recommendations with respect to geologic and seismic hazards, California Building Code (CBC) design considerations, site preparation and grading, and foundation design.

5.1 2016 CBC SEISMIC DESIGN PARAMETERS

5.1.1 Site Class

Based on information obtained from the investigation, published geologic literature and maps, and on our interpretation of the 2016 California Building Code (CBC) criteria, it is our opinion that the project site may be classified as Site Class D, Stiff Soil, according to Section 1613.3.2 of 2016 CBC and Table 20.3-1 of American Society of Civil Engineers (ASCE) 7-10 (2010). Site Class D is defined as a soil profile consisting of stiff soil profile with a shear wave velocity between 600 feet per second and 1,200 feet second, standard penetration test (SPT) blow counts (N-value) between 15 blows per foot and 50 blows per foot, or undrained shear strength between 1,000 pounds per square foot and 2,000 pounds per square foot in the top 100 feet.

5.1.2 Seismic Design Parameters

Approximate coordinates for the site are noted below.

- Latitude: 38.12929 °
- Longitude: -121.24990 °

For a 2016 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1), associated soil amplification factors (F_a and F_v), and mapped peak ground acceleration (PGA) are presented in Table 5-1. Corresponding site modified (S_{MS} and S_{M1}) and design (S_{DS} and S_{D1}) spectral accelerations, PGA modification coefficient (F_{PGA}), PGA_M, risk coefficients (C_{RS} and C_{R1}), and long-period transition period (T_L) are also presented in Table 5-1. Presented values were estimated using Section 1613.3 of the 2016 California Building Code (CBC), Chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps (https://seismicmaps.org/).

Table 5-1
Ground Motion Parameters Based on 2016 CBC

Parameter	Value	Reference
S_S	0.724g	2016 CBC Section 1613.3.1
S_1	0.295g	2016 CBC Section 1613.3.1
Site Class	D	2016 CBC Section 1613.3.2
F_a	1.22	2016 CBC Table 1613.3.3(1)
F_v	1.811	2016 CBC Table 1613.3.3(2)
PGA	0.249g	ASCE 7-10 Figure 22-7
S_{MS}	0.884g	2016 CBC Section 1613.3.3
S_{M1}	0.533g	2016 CBC Section 1613.3.3
S_{DS}	0.589g	2016 CBC Section 1613.4.4
S_{D1}	0.356g	2016 CBC Section 1613.4.4
F_{PGA}	1.301	ASCE 7-10 Table 11.8-1
PGA_M	0.324g	ASCE 7-10 Section 11.8.3
C_{RS}	1.1	ASCE 7-10 Figure 22-17
C_{R1}	1.142	ASCE 7-10 Figure 22-18
T_L	12 seconds	ASCE 7-10 Figure 22-12

5.2 LIQUEFACTION

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table but can also occur in non-plastic to low-plasticity, finer-grained soils. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or “cyclic mobility,” increased lateral earth pressures on retaining walls, liquefaction settlement, and lateral spreading or “flow failures” in slopes.

Based on the relative density, soil type, and depth to groundwater at the site, the potential for liquefaction is considered negligible.

5.3 EXPANSIVE SOILS

Based on the results of an Atterberg limits test performed on a near-surface sample of silty sand (Boring B-4 at a depth of about 5.5 feet), the sampled soils at shallow surface measured to be non-plastic. Based on the aforementioned test results, potential and density of these soils, we do not anticipate the surficial soils will shrink or swell significantly as a result of soil moisture content changes.

5.4 SITE PREPARATION

5.4.1 General

Considering site grades are presently well established, site grading is anticipated to be minimal, minus the grading for the proposed pond. General recommendations for site preparation and earthwork construction are presented in the following sections of this report. All earthwork, including excavation, backfill and preparation of subgrade, should be performed in accordance with the geotechnical recommendations presented in this report and applicable portions of the grading code of local regulatory agencies. The grading contractor is responsible to notify governmental agencies, as required, and the geotechnical engineer at the start of site cleanup, the initiation of grading and any time that grading operations are resumed after an interruption. All earthwork should be performed under the observation and testing of a Kleinfelder representative. All references to compaction, maximum density and optimum moisture content are based on ASTM D1557, unless otherwise noted.

5.4.2 Stripping and Grubbing

Any miscellaneous surface or encountered subsurface obstructions, vegetation, debris, or other deleterious materials should be removed from the project area prior to any site grading. At the time of the investigation, the site surface was loose to a depth of approximately 6 inches due to previous disking for weed control. The loose, disked soil was also blended with a moderate amount of visible organics of seasonal vegetation. The depth of stripping at the time of construction should be enough to remove the visible organics. The stripped materials should not be incorporated into any engineered fill unless they can be thoroughly blended to achieve an organic content less 3 percent by weight and no visible organic matter.

5.4.3 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

Initial site grading should include a reasonable search to locate soil disturbed by previous activity and abandoned underground structures or existing utilities that may exist within the areas of construction. Any loose or disturbed soils, void spaces that may be encountered should be over-excavated to expose firm and relatively unyielding native soil, as approved by a representative of Kleinfelder.

Unless approved otherwise by an on-site representative of Kleinfelder during grading, undocumented fills at the locations of any future grading or shallow foundations should be over-excavated and replaced with engineered fill as recommended below in the "Engineered Fill-Placement and Compaction Criteria" section of this report.

5.4.4 Scarification and Compaction

In areas requiring placement of fill, it is recommended the fill be placed and compacted as engineered fill. Following site stripping and any required grubbing and/or over-excavation, it is recommended areas to receive engineered fill be scarified to a depth of 8 inches, uniformly moisture conditioned to at least the optimum moisture content for sandy soils (SP, SM, SC) or at least 3 percent above the optimum moisture content for clayey soils (CL, CH) and compacted to at least 90 percent relative compaction for sandy soils or between 88 and 92 percent relative compaction for clayey soils, as determined by ASTM D1557.

5.5 ENGINEERED FILL

5.5.1 Onsite Materials

The on-site soil appears suitable for use as engineered fill. All engineered fill should be free of debris, visible organics, or other deleterious materials, and have a maximum particle size less than 3 inches in maximum dimension. Where imported material is brought in, it is recommended that it be granular in nature and conform to the minimum criteria discussed in Table 5-2.

5.5.2 Non-Expansive Engineered Fill Requirements

Specific requirements for engineered fill as well as applicable test procedures to verify material suitability are provided below:

Table 5-2
Engineered Fill Requirements

Fill Requirement		Test Procedures	
		ASTM	Caltrans
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D6913	202
¾ inch	70-100	D6913	202
No. 200	20-50	D6913	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D4318	204
Organic Content			
No visible organics		---	---
Expansion Potential			---
20 or less		D4829	---
Soluble Sulfates			
Less than 2,000 ppm		---	417
Soluble Chloride			
Less than 300 ppm		---	422
Resistivity			
Greater than 2,000 ohm-cm		---	643

Materials to be used for engineered fill should be sampled and tested by Kleinfelder prior to being transported to the site. Highly pervious materials such as clean crushed stone or pea gravel are not recommended for use in engineered fill because they can permit transmission of water into the underlying materials. We recommend representative samples of imported materials proposed for use as engineered fill be submitted to Kleinfelder for testing and approval at least one week prior to the start of grading and import of this material.

In addition, we recommend that a laboratory corrosion test series (pH, resistivity, redox, sulfides, chlorides, and sulfates) be performed on all proposed import materials.

5.5.3 Placement and Compaction Criteria

Non-expansive soils that meet the criteria outlined in Table 5-2 that are to be used for engineered fill should be uniformly moisture conditioned to at least the optimum moisture content, placed in horizontal lifts less than about 8 inches in loose thickness, and compacted to at least 90 percent relative compaction, as determined by ASTM D1557. Onsite clayey soils to be used for general fill where engineered fill is not required should be uniformly moisture conditioned to at least 4 percent over the optimum moisture content, placed in horizontal lifts no more than about 8 inches in loose thickness, and compacted to between 88 and 92 percent relative compaction, as determined by ASTM D1557.

Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or moisture content, or if soil conditions are not stable. Disking or blending may be required to uniformly moisture condition soils used for engineered fill. Ponding or jetting compaction methods should not be allowed.

All site preparation and fill placement should be observed by Kleinfelder. It is important that during the stripping and scarification processes, a representative of Kleinfelder be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during the geotechnical site exploration.

5.6 WET WEATHER CONSIDERATIONS

Should construction be performed during or subsequently after 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 geogrid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork and construction operations.

5.7 SITE DRAINAGE

Final site grading should provide surface drainage away from all structures and areas to be traversed by vehicles and maintenance equipment. In general, we recommend consideration be given to providing at least 2 percent slope away from structure foundations or access ways.

5.8 TEMPORARY EXCAVATIONS

5.8.1 General

All excavations should comply with applicable local, state, and federal safety regulations including the current Occupational Safety & Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the Contractor, who is 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.

5.8.2 Excavation and Slopes

Excavated slope height, slope inclination, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties.

Underground utilities should be located above a 1H:1V (horizontal to vertical) plane projected down and out from the bottoms of new footings to avoid undermining the footings during the excavation of the utility trench.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should be kept sufficiently away from the top of any excavation to prevent any unanticipated surcharging. Alternatively, excavation slopes and shoring systems can be designed to accommodate surcharge loadings, if necessary. Shoring, bracing, or underpinning required for the project (if any), should be designed by a professional engineer registered in the State of California.

5.9 TRENCH BACKFILL

All trench backfill should be placed and compacted in accordance with recommendations provided for engineered fill (see Section 5.5). Mechanical compaction is recommended. Ponding or jetting should not be used as a sole means of soil compaction.

5.10 SHALLOW FOUNDATIONS

This section provides general recommendations for shallow foundations. Kleinfelder should review the design to ensure compliance with the intent of the geotechnical conclusions and recommendations provided in this report.

Foundations should satisfy two independent criteria with respect to foundation soils. First, the foundation should have an adequate safety factor against bearing failure with respect to the shear strength of the foundation soils. Second, the vertical movements of the foundation due to settlement (both immediate elastic settlement and consolidation settlement) should be within tolerable limits for the structure. Depending on the settlement tolerance of planned structures, design loading, and foundation dimensions, the general recommendations presented in this report may be subject to modification. If future project needs require additional foundation capacity, Kleinfelder should be contracted to evaluate this potential for specific foundation designs.

Structures may be supported on conventional, shallow, reinforced concrete mat foundations or spread footings, provided the site structures can tolerate the anticipated settlement.

5.10.1 Spread Footings

5.10.1.1 Allowable Bearing Pressure

Shallow spread footings constructed of reinforced concrete may be founded on approved undisturbed native soil and/or engineered fill. The footings should be founded at least 18 inches below lowest adjacent finished grade on subgrade soils that have been prepared in accordance with the recommendations provided in this report. Continuous and isolated rectangular footings should have a minimum width of 12 inches.

For foundation subgrade prepared in accordance with the recommendations provided in this report, spread and strip footings may be designed for a net allowable bearing pressure of up to 2,000 pounds per square foot (psf) due to dead plus live loads. The weight of the foundation that extends below grade may be neglected when computing dead loads. The allowable bearing pressure includes a safety factor of at least 3 with respect shear failure of the foundation soils and may be increased by one-third for transient loading due to wind or seismic forces.

To maintain the desired support, foundations adjacent to utility trenches or other existing foundations should be deepened so that their bearing surfaces are below an imaginary plane having an inclination of 1 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent foundations or utility trenches.

5.10.1.2 Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.39 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 360 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches should be neglected unless the area in front of the footing is protected from disturbance by concrete or pavement. The allowable friction coefficient and passive resistance may be used concurrently.

5.10.1.3 Settlement

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Foundation dimensions and loads have not been provided for the proposed structures, we estimate maximum total settlement of foundations designed and constructed in accordance with the preceding recommendations of up to about ½ inch or less. Differential settlement between similarly loaded, adjacent footings are estimated to be about half the total settlement. The majority of foundation settlement is expected to occur rapidly and should be essentially complete shorty after initial application of the loads.

5.10.1.4 Shallow Foundation Construction Considerations

Prior to placing steel or concrete, foundation excavations should be cleaned of any debris, disturbed soil or water. All foundation excavations should be observed by a representative of Kleinfelder just prior to placing fill and/or steel or concrete. The purpose of these observations is to check that the bearing soils actually encountered in the foundation excavations are similar to those assumed in analysis and to verify the recommendations contained herein are implemented during construction.

5.10.2 Mat Foundations

Recommendations for design and construction of small mat slab foundations up to about 25 feet wide are presented below. Kleinfelder should be consulted to provide supplementary mat foundation recommendations if larger mat slab foundations are planned in the future.

5.10.2.1 *Allowable Bearing Pressure*

For subgrades prepared as recommended in this report, reinforced concrete mat foundations may be designed for a net allowable bearing pressure of 2,000 psf. If higher allowable bearing capacity for mat foundations is required, Kleinfelder should be consulted to provide supplemental engineering and construction recommendations on a case-by-case basis. The allowable bearing pressure applies to dead plus live loads, includes a safety factor of at least 3 with respect to shear failure of the foundation soils, and may be increased by one-third for short-term loading due to wind or seismic forces.

5.10.2.2 *Lateral Load Resistance*

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.39 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 360 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches should be neglected unless the area in front of the foundation is protected from disturbance by concrete or pavement. The friction coefficient and passive resistance may be used concurrently.

5.10.2.3 *Subgrade Modulus*

For preliminary design purposes, a modulus of subgrade reaction, K_{v1}, of 150 pounds per square inch per inch of deflection (for a 1 square-foot bearing plate) may be used for design of mat slabs. The modulus should be adjusted for the actual slab size using appropriate formulas or software.

5.10.2.4 *Mat Slab Settlement*

For foundations with design pressures equal to or less than the net allowable pressure provided above, and under static loading conditions, total post-construction foundation settlement is expected to be less than about ½ inch at the center of the mat foundations. Post-construction differential settlement of individual foundation elements is expected to be about one-half the total settlement.

These settlement estimates are based on the assumption that the foundation subgrade is properly prepared, and the foundations are designed and constructed in accordance with the recommendations presented in this report.

5.10.2.5 Mat Foundation Construction Considerations

Underground utilities that are 4 feet deep or shallower and that run parallel to shallow mat foundations generally should be located no closer than 2 feet horizontally away from the perimeter edges of the slab. Deeper utilities should be located above a 1H:1V (horizontal to vertical) slope projected downward from the bottom edges of the slab. Utility plans should be reviewed by Kleinfelder prior to trenching to evaluate conformance with this requirement.

Beneath exterior cast-in-place concrete mat foundations, we recommend the design include a base course of well-graded crushed aggregate base at least 6 inches thick. Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate base. Under slabs that will be subject to vehicle loading, the aggregate base course thickness should be increased to a minimum of 6 inches. The base course should be compacted to at least 95 percent relative compaction at optimum moisture content. Thickened slab edges embedded to at least 18 inches below grade need not be underlain by the gravel base course.

5.11 DRILLED PIER FOUNDATIONS

Recommendations for design and construction of drilled pier foundations are presented in the following sections of this report.

5.11.1 Axial Capacity

Axial pile capacity was developed based on Federal Highway Administration methods using the commercial computer software SHAFT, version 2017, produced by Ensoft, Inc. Static soil strength parameters are based on strength and soil properties measured during the field and laboratory testing phases of this investigation.

Axial loads on drilled piers should be supported by the frictional capacity of the pier. End bearing is not considered in the axial capacity due to strain incompatibility issues between skin friction and end bearing, settlement issues, and the potential for loose materials to exist at the bottoms of the pier holes during construction that cannot be effectively cleaned out. If additional axial capacity is required beyond what is provided in this report, Kleinfelder should be consulted to provide a portion of end bearing capacity and additional construction recommendations.

A curve illustrating the ultimate axial compressive capacity of a unit (1-foot) diameter straight-sided drilled pier installed from the existing grade under static conditions is shown on Figure 2a. Corresponding tabulated values are presented on Figure 2b. Capacities for drilled piers with diameters other than 1 foot may be obtained by multiplying the capacity for the 1-foot diameter pier by the actual pier diameter (in feet). For evaluation of allowable axial capacity under static conditions, we recommend a factor of safety of 3 be applied to the ultimate capacity (per the General Order 95 code). Note that the weight of the foundation need not be considered for evaluation of allowable axial capacity.

Ultimate tensile capacity may be obtained by multiplying the compressive capacity by a factor of 0.8 and adding the weight of the foundation. For allowable tension capacity under transient flood, wind or seismic conditions, a safety factor of at least 1.5 should be used. For allowable sustained tension, a safety factor of 3 should be used.

5.11.1.1 Estimated Settlement

Based on the methods outlined by FHWA Drilled Shaft Manual, Brown et al. (2010), total static settlement of each drilled pier should be on the order of 0.1 percent of the pier diameter for a drilled pier designed and constructed in accordance with the recommendations presented in this report. This value includes elastic compression of the pile under design loads. The majority of the settlement should occur during and shortly after application of the structure loads. We suggest allowing for about ¼ inch of settlement to accommodate potential long-term settlement, construction issues, and some soil variability across the site.

5.11.1.2 Axial Capacity Group Effects

The axial capacity of piers developed in accordance with the recommendations provided above applies to single, isolated piers. Consideration of group effects on axial capacity of drilled piers is usually not necessary for piers with center-to-center spacings of at least 3 effective diameters. For closer spacings the capacity of individual piers will be reduced. For these cases Kleinfelder should be consulted to evaluate axial capacity on a case-by-case basis. Note that group effects should also be considered where new foundations are constructed immediately adjacent to existing foundations.

5.11.2 Lateral Response

5.11.2.1 LPILE Analysis Soil Parameters

Lateral capacity of drilled piers may be developed through analysis of pier response due to a range of design loads. Table 5-3 contains recommended input soil parameters for lateral response analysis of deep foundations using the LPILE computer program (by Ensoft, Inc., Version 2018. Program default values may be used for strain factor (E_{50}) and horizontal subgrade reaction (K).

Table 5-3
LPILE Geotechnical Parameters
Static Conditions

Depth (feet)	Model P-Y Curve	Effective Unit Weight (lb/ft^3)	Cohesion c (psf)	Internal Friction Angle, Φ (degrees)
0 to 30	Sand (Reese)	115	-	30

LPILE analyses and Canedo Q Value determinations could not be performed at this time, as the loading of individual piles have not yet been established by PG&E. When loading is available, Kleinfelder can provide Lpile analysis and evaluate the Canedo Q Value for an additional fee.

5.11.3 Drilled Pier Construction Considerations

Successful completion of drilled pier foundations requires good construction procedures. Drilled pier excavations should be constructed by a skilled operator using techniques that allow the excavations to be completed, the reinforcing steel placed, and the concrete poured in a continuous manner to reduce the time that excavations remain open. Steel reinforcement and concrete should be placed on the same day of completion of each pier excavation. Additionally, drilled pier excavations should be scheduled to allow concrete in each pile to set over night before drilling adjacent holes that are closer than 4 diameters center-to-center.

The following considerations should be implemented during construction of drilled shaft foundations. We recommend the contractor follow the procedures for drilled pier construction contained in the Federal Highway Administration (FHWA) manual on drilled shaft construction (Brown et al., 2010).

Consistent with Chapter 17 of the 2016 CBC, drilled pier excavations should be inspected and approved by the geotechnical engineer prior to installation of reinforcement. The depths of all pier excavations should be checked immediately prior to concrete placement to verify excessive sloughing and/or caving has not reduced the required hole depth. This may be done with a weighted tape measure or similar measuring device.

As described above, loose sandy soils may be encountered during drilled pier construction. In addition, perched groundwater depending on local rainfall and runoff patterns may also be present at the time of construction. The contractor should be prepared to handle caving sandy soil and possibly of perched groundwater conditions during construction of drilled piers at the site.

The depth to regional groundwater is on the order of 70 feet bgs, therefore, it is unlikely that drilled shafts will encounter regional groundwater. If drilled shaft excavations extend below groundwater levels, the excavations should be cleaned such that less than about 1 inch of loose soil remains at the bottom of the drilled hole. Since the piers should be designed to derive their support in skin friction along the sides of the shafts, consideration could be given to over-drilling the shafts to accommodate any sloughing that may occur between drilling and concrete placement. It is recommended that a representative from Kleinfelder observe each drilled shaft excavation to verify soil and excavation conditions prior to placing steel reinforcement or concrete.

Steel reinforcement and concrete should be placed on the same day the drilled hole is completed to reduce the potential for caving and reduce the quantity of suspended soil particles that may settle to the bottom of the hole during wet-method construction. Excavation depths should be checked several times before concrete placement to ensure excessive sedimentation has not occurred. Concrete used for pier construction should be discharged vertically into the drilled hole to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during shaft construction. 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.

In order to develop the design skin friction values provided in the axial capacity figures, concrete used for drilled pier 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 pile.

If water or drilling fluids are present during concrete placement, concrete should be placed into the hole using tremie methods. Tremie concrete placement should be performed in strict accordance with ACI 304R. The tremie pipe should be rigid and remain below the surface of the in-place concrete at all times to maintain a seal between the water or slurry and 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 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 thick but will depend on the pile diameter, amount of water seepage, and construction workmanship.

Loose sandy soils will likely be encountered during drilled pier construction. Use of slurry drilling methods will likely be needed to reduce the potential for caving in the drilled pier excavations. Use of slurry drilling methods normally requires experienced construction personnel to batch and mix the slurry, test the slurry for proper mixing, hydration, viscosity and other important properties, and to monitor slurry performance during drilling. If slurry drilling methods are used, we recommend use of a polymer slurry that meets Caltrans requirements for drilled shaft construction or bentonite-based slurry, mixed and used in accordance with the guidelines in the FHWA Drilled Shaft Manual (Brown et al., 2010). This guideline recommends bentonite slurry mixtures not be left in the hole for more than about 4 hours in order to avoid potential side friction losses that may be caused by excessive thickness of bentonite filter cake on the hole wall.

If caving conditions are encountered in a drilled pier excavation and there are no overhead clearance issues, temporary casing could be used to help mitigate this condition. If temporary steel casing is used, it should be removed from the hole as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete during casing withdrawal and concrete placement operations. Casing should not be withdrawn until sufficient quantities of concrete have been placed into the excavation to balance the groundwater head outside the casing. Continuous vibration of the casing or other methods may be required to reduce the potential for voids occurring within the concrete mass during casing withdrawal. Corrugated metal pipe should not be used as casing. In no case should casing material be left in the excavation after concrete has been placed without the approval of the project structural and geotechnical engineers. Concrete should be in direct contact with the surrounding soil or the design parameters and recommendations in the geotechnical report are not valid.

5.12 SOIL CORROSION

Kleinfelder has completed laboratory testing to provide data regarding corrosivity of onsite soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

Laboratory chloride concentration, sulfate concentration, pH, oxidation reduction potential, redox, sulfide and electrical resistivity tests were performed for a near surface soil sample. The results of the tests are attached and are summarized in Table 5-4. If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material.

Table 5-4
Chemistry Laboratory Test Results

Boring and Depth	Material	Resistivity, ohm-cm	Resistivity, ohm-cm (Saturated)	pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
						Chloride	Sulfide	Sulfate
B-4 (1-5 feet)	Sand	29,000	22,000	7.21	320	N.D.*	N.D.*	N.D.*

*N.D. - None Detected

Ferrous metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices.

Based on the “10-point” method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the potential for the soils at the site to be corrosive to buried ferrous metal piping, cast iron pipes, or other objects made of these materials is negligible. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication "Guide to Durable Concrete" (ACI 201.2R-08) provides guidelines for this assessment. The samples had sulfate concentrations of non-detectible (N.D.), which indicates the potential for deterioration of concrete is mild to negligible, and no special requirements should be necessary for the concrete mix.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater. Chloride tests indicated the sample had concentrations below the detectable limit.

6 ADDITIONAL SERVICES

6.1 PLANS AND SPECIFICATIONS REVIEW

Kleinfelder should conduct a general review of plans and specifications to evaluate that the earthwork and foundation recommendations presented in this report have been properly interpreted and implemented during design. In the event Kleinfelder is not retained to perform this recommended review, no responsibility for misinterpretation of the recommendations by Kleinfelder is accepted.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that all earthwork and foundation construction be monitored by a representative from Kleinfelder, including site preparation, placement of all engineered fill and trench backfill, construction of slab and all foundation excavations. The purpose of these services is to observe the soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

7 LIMITATIONS

This report presents information for planning, permitting, design, and construction of the new expansion, planned at the Thurman Switching Station in Lodi, California. Recommendations contained in this report are based on materials encountered in Borings B-1 through and B-5, geologic interpretation based on published articles and geotechnical data, and our present knowledge of the proposed construction.

It is possible that soil conditions could vary beyond the points explored. If the scope of the proposed construction, including the proposed location, changes from that described in this report, we should be notified immediately in order that a review may be made, and any supplemental recommendations provided.

We have prepared this report in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty expressed or implied is made.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on-site and off-site) 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.

8 REFERENCES

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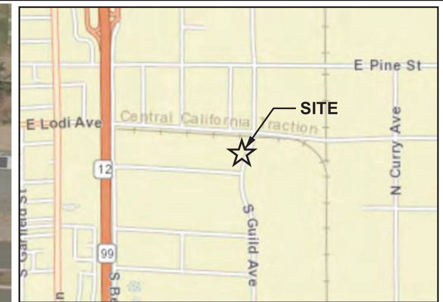
FIGURES



VICINITY MAP

NOT TO SCALE

NOTE:
BASE MAPPING AND VICINITY MAP CREATED FROM LAYERS COMPILED BY ESRI PRODUCTS AND 2019 MICROSOFT CORPORATION.
COORDINATE SYSTEM:
NAD 1983 2011 STATEPLANE CALIFORNIA III FIPS 0403

LEGEND

SOIL BORING

PERCOLATION TEST HOLE

N

0 100 200

1 " = 100 SCALE IN FEET

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PROJECT NO.	20193892
DRAWN BY:	HFattal
CHECKED BY:	L. Serrano
DATE:	06-13-2019
REVISED:	-

EXPLORATION LOCATION PLAN AND VICINITY MAP

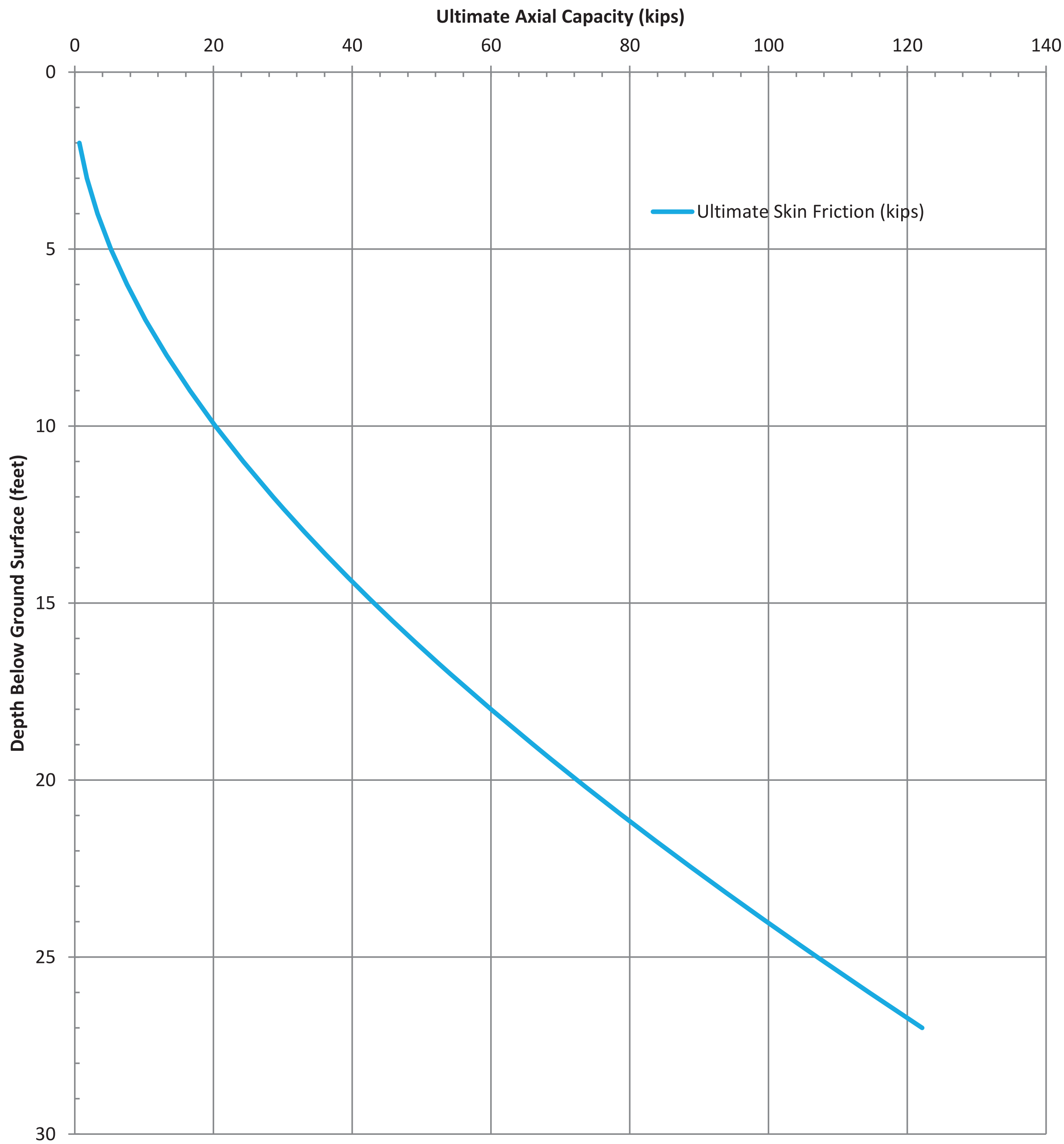
PG&E Thurman Substation
1215 East Thurman Road
Lodi, CA

FIGURE

1

PLOTTED: 6/13/2019 2:44:50 PM BY: HFattal

GIS FILE PATH: \\azrgisstorp01\Working_clients_Automated_Exploration Plans



Notes:

1. Axial capacities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the pier (in feet).
2. Ultimate tensile capacity may be obtained by multiplying the ultimate compressive capacity by a factor of 0.8.
3. The curve represents ultimate axial capacity of a straight-sided drilled pier. See text discussion for factor of safety and group effects.

KLEINFELDER
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PROJECT NO.: 20193892	ULTIMATE AXIAL CAPACITY TABLE UNIT DIAMETER (1-FOOT) DRILLED PIER STATIC CONDITION	FIGURE
DRAWN BY: HF		
CHECKED BY: SP	PG&E THURMAN SUBSTATION	2A
DATE: 6/13/2019	1215 EAST THURMAN ROAD	
REVISED: 6/21/2019	LODI, CA	

Depth (ft)	Ultimate Axial Capacity (Kips)	Depth (ft)	Ultimate Axial Capacity (Kips)
2	0.7	15	48.5
3	1.7	16	54.1
4	3.3	17	60.0
5	5.2	18	66.1
6	7.5	19	72.4
7	10.2	20	78.9
8	13.2	21	85.7
9	16.6	22	92.6
10	20.3	23	99.7
11	24.3	24	107.0
12	28.6	25	114.5
13	33.1	26	122.1
14	38.0	27	

Notes:

1. Axial capcities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the pier (in feet).
2. Ultimate tensile capacity may be obtained by multiplying the ultimate compressive capacity by a factor of 0.8.
3. The curve represents ultimate axial capacity of a straight-sided drilled pier. See text discussion for factor of safety and group effects.

KLEINFELDER
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PROJECT NO.: 20193892	ULTIMATE AXIAL CAPACITY TABLE UNIT DIAMETER (1-FOOT) DRILLED PIER STATIC CONDITION	FIGURE
DRAWN BY: HF		
CHECKED BY: SP	PG&E THURMAN SUBSTATION	2B
DATE: 6/13/2019	1215 EAST THURMAN ROAD	
REVISED: 6/21/2019	LODI, CA	

APPENDIX A

FIELD EXPLORATION

BORING LOG B-1

Date Begin - End:	5/24/2019	**Drilling Company:**	Taber		
Logged By:	H. Fattal	**Drill Crew:**	Rick & David		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Solid Stem Auger	**Hammer Efficiency:**	89%
Weather:	Not Available	**Exploration Diameter:**	4 in. O.D.	**Hammer Cal. Date:**	10/26/2018

Latitude: 38.12986°
Longitude: -121.24990°
Surface Condition: Topsoil/Grass

Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	approximately 6 inches of topsoil, dry											hand auger to 5 feet
	Poorly Graded SAND with Silt (SP-SM): fine to coarse-grained sand, non-plastic, dark brown, moist, trace roots											
5	non-plastic, loose	1	BC=4 4 4			8.7	88.7					switch to solid stem auger at 5 feet
		2	BC=3 5 5						14			
	Silty Clayey SAND (SC-SM): fine to coarse-grained sand, low to medium plasticity, olive yellow, moist, loose											
10	**Poorly Graded SAND with Silt (SP-SM)**: fine to coarse-grained sand, low plasticity, dark brown, dry, dense	3	BC=8 15 23									
	Clayey SAND (SC): fine to coarse-grained sand, low to medium plasticity, yellowish brown, dry, very dense											
15		4	BC=14 23 30							23	10	
	Poorly Graded SAND with Clay (SP-SC): low plasticity, dark brown, dry, medium dense, weakly cemented layer											
20		5	BC=3 12 11									
25	low plasticity, dense	6	BC=12 15 19									
30	low plasticity, medium dense	7	BC=3 12 14									upper 6" of blow counts is slush

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with neat cement on May 24, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:

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PROJECT NO.:	20193892
DRAWN BY:	JDS
CHECKED BY:	HF
DATE:	5/29/2019
REVISED:	6/17/2019

BORING LOG B-1

PG&E Thurman Substation
1215 East Thurman Road
Lodi, CA

FIGURE

A-2

BORING LOG B-2

Date Begin - End:	5/24/2019	**Drilling Company:**	Taber		
Logged By:	H. Fattal	**Drill Crew:**	Rick & David		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Solid Stem Auger	**Hammer Efficiency:**	89%
Weather:	Not Available	**Exploration Diameter:**	4 in. O.D.	**Hammer Cal. Date:**	10/26/2018

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.12999°
Longitude: -121.24940°
Surface Condition: Topsoil/Grass

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	approximately 6 inches of topsoil, dark brown, moist												hand auger to 5.5 feet
0.5	**Silty Clayey SAND (SC-SM)**: fine to coarse-grained sand, low plasticity, dark brown, moist												
5	increased silt with depth	1		BC=7 7 6			15.3	94.8		63			switch to solid stem auger at 5.5 feet TXUU: c = 1.95 ksf
6	**Sandy Lean CLAY (CL)**: fine to coarse-grained sand, low plasticity, dark brown, moist, medium dense, trace roots dry, dense, strongly cemented, thin layer at 8.5'	2		BC=4 17 23									
10		3		BC=3 15 33									poured water down boring to assist in drilling difficulty
15	low to medium plasticity, very dense	4		BC=3 22 35									poured water down boring to assist in drilling difficulty
20	low to medium plasticity, medium dense	5		BC=3 11 12									
23	**Poorly Graded SAND (SP)**: fine to coarse-grained sand, non-plastic, reddish brown, dense												
25		6		BC=9 17 17									strong grinding noise during advancement of solid stem auger
30	non-plastic, medium dense	7		BC=13 18 20									
30.5	**Clayey SAND (SC)**: fine to coarse-grained sand, low plasticity, gray, dry, dense												

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with neat cement on May 24, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:

KLEINFELDER
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PROJECT NO.:	20193892
DRAWN BY:	JDS
CHECKED BY:	HF
DATE:	5/29/2019
REVISED:	6/17/2019

BORING LOG B-2

PG&E Thurman Substation
1215 East Thurman Road
Lodi, CA

FIGURE

A-3

Date Begin - End:	5/28/2019	**Drilling Company:**	Taber		**BORING LOG B-3**
Logged By:	H. Fattal	**Drill Crew:**	Rick & David		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Solid Stem Auger	**Hammer Efficiency:**	89%
Weather:	Not Available	**Exploration Diameter:**	4 in. O.D.	**Hammer Cal. Date:**	10/26/2018

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.12947°
Longitude: -121.24890°
Surface Condition: Topsoil/Dry Grass

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0		approximately 6 inches of topsoil, dark brown, dry												hand auger to 5 feet
		Poorly Graded SAND with Silt (SP-SM): fine to coarse-grained sand, non-plastic, dark brown, moist, more silt with depth												
5		loose	1		BC=5 5 5			10.5	112.2					switch to solid stem auger at 5 feet
		Silty SAND (SM): fine to coarse-grained sand, non-plastic, yellowish brown, moist, medium dense, trace clay	2		BC=3 4 11									
		Poorly Graded SAND (SP): fine to coarse-grained sand, non-plastic, reddish brown, dry, medium dense												
10			3		BC=4 12 10									
		Clayey SAND (SC): fine to coarse-grained sand, low to medium plasticity, yellowish brown, dry, medium dense												
15		**Poorly Graded SAND (SP)**: fine to coarse-grained sand, non-plastic, dark brown, dry, dense	4		BC=13 17 20									
		Clayey SAND (SC): fine to coarse-grained sand, low to medium plasticity, yellowish brown, dry, dense												
		Poorly Graded SAND (SP): fine to coarse-grained sand, non-plastic, dark brown, dry, medium dense												
20			5		BC=8 9 10									
														difficultly advancing solid stem augers grinding noise. Poured water down boring to assist in drilling difficulty
25			6		BC=11 10 11									
		Silty SAND (SM): fine to coarse-grained sand, trace clay, low plasticity, olive brown, dry, dense												
30			7		BC=10 13 17						16			

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with neat cement on May 28, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20193892	BORING LOG B-3	FIGURE
	DRAWN BY: JDS		
	CHECKED BY: HF	PG&E Thurman Substation 1215 East Thurman Road Lodi, CA	A-4
	DATE: 5/29/2019		
	REVISED: 6/17/2019		PAGE: 1 of 1

BORING LOG B-4

Date Begin - End:	5/23/2019	**Drilling Company:**	Taber
Logged By:	H. Fattal	**Drill Crew:**	Rick & David
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Not Available	**Exploration Diameter:**	6 in. O.D.

Hammer Type - Drop:	140 lb. Auto - 30 in.
Hammer Efficiency:	89%
Hammer Cal. Date:	10/26/2018

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.12929°
Longitude: -121.24990°
Surface Condition: Topsoil/Grass

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	approximately 6 inches of topsoil, dark brown, dry												hand auger to 5 feet
	Poorly Graded SAND (SP-SM): fine to medium-grained sand, non-plastic, dark brown, moist, trace roots, with silt												
	Silty SAND (SM): fine to medium-grained sand, non-plastic, dark brown, moist												
5		1		BC=4 6 4						28	NP	NP	switch to hollow stem auger at 5 feet
		2		BC=3 4 7									
10		3		BC=7 7 6						23			
15		4		BC=6 8 9									
	Poorly Graded SAND with Clay (SP-SC): fine to coarse-grained sand, low plasticity, reddish brown, moist, medium dense												
	Poorly Graded SAND (SP): fine to coarse-grained sand, non-plastic, reddish brown, dry, dense												
20		5		BC=14 16 19									
													very difficult to auger
25	non-plastic, reddish brown, medium dense	6		BC=7 12 13									
	Clayey SAND (SC): fine to medium-grained sand, low plasticity, gray, dry, medium dense												
	Poorly Graded SAND (SP): fine to coarse-grained sand, non-plastic, reddish brown, dry, dense												
30		7		BC=9 16 20									

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PROJECT NO.:	20193892
DRAWN BY:	JDS
CHECKED BY:	HF
DATE:	5/29/2019
REVISED:	6/17/2019

BORING LOG B-4

PG&E Thurman Substation
1215 East Thurman Road
Lodi, CA

FIGURE

A-5

BORING LOG B-4

Date Begin - End:	5/23/2019	**Drilling Company:**	Taber		
Logged By:	H. Fattal	**Drill Crew:**	Rick & David		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	89%
Weather:	Not Available	**Exploration Diameter:**	6 in. O.D.	**Hammer Cal. Date:**	10/26/2018

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.12929°
Longitude: -121.24990°
Surface Condition: Topsoil/Grass

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Poorly Graded SAND (SP): fine to coarse-grained sand, non-plastic, reddish brown, dry, dense non-plastic, reddish brown gray, medium dense	8		BC=9 9 15									
40		reddish brown, dense	9		BC=10 14 20									
45			10		BC=7 11 13									
50			11		BC=11 11 12									
55														
60														
65														

The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with neat cement on May 23, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.:	20193892	BORING LOG B-4	FIGURE
DRAWN BY:	JDS		A-5
CHECKED BY:	HF	PG&E Thurman Substation 1215 East Thurman Road Lodi, CA	
DATE:	5/29/2019		
REVISED:	6/17/2019		

BORING LOG B-5

Date Begin - End:	5/23/2019	**Drilling Company:**	Taber		
Logged By:	H. Fattal	**Drill Crew:**	Rick & David		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	**Hammer Efficiency:**	89%
Weather:	Not Available	**Exploration Diameter:**	4 in. O.D.	**Hammer Cal. Date:**	10/26/2018

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.12889°
Longitude: -121.24940°
Surface Condition: Topsoil/Dry Grass

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximately 6 inches of topsoil, dark brown, moist												hand auger to 5 feet
	Poorly Graded SAND with Silt (SP-SM): fine to coarse-grained sand, non-plastic, orangish brown, dry												
	Clayey SAND (SC): fine to coarse-grained sand, low plasticity, dark brown, moist												
5	**Silty SAND (SM)**: fine to coarse-grained sand, low plasticity, reddish brown, dry, medium dense	1		BC=6 9 8			18.8	103.7					switch to solid stem auger at 5 feet
	Clayey SAND (SC): fine to coarse-grained sand, low plasticity, reddish brown, dry, medium dense, thin strongly cemented layer, cementation weaker with depth	2		BC=12 7 12									
10	**Poorly Graded SAND (SP)**: fine to coarse-grained sand, non-plastic, reddish brown, dry, medium dense, trace roots, trace clay	3		BC=12 18 26							24	9	
	Sandy Lean CLAY (CL): fine to medium-grained sand, medium plasticity, reddish brown, dry, hard												
15	**Clayey SAND (SC)**: fine to coarse-grained sand, low plasticity, reddish brown, dry, medium dense	4		BC=14 23 32									
	Poorly Graded SAND with Silt (SP-SM): fine to coarse-grained sand, low plasticity, reddish brown, dry, very dense												
	Poorly Graded SAND with trace Silt (SP): fine to coarse-grained sand, non-plastic, dark brown, dry, dense												poured water down boring
20		5		BC=6 19 22									
	Poorly Graded SAND with Clay (SP-SC): low plasticity, dark brown, dry, dense												
25		6		BC=13 16 19									
	Clayey SAND (SC): fine to coarse-grained sand, non-plastic, dark brown, dry, medium dense												
30		7		BC=11 14 17									
	Poorly Graded SAND (SP): fine to coarse-grained sand, non-plastic, dark brown, dry, dense, trace clay												

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with neat cement on May 23, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.:	20193892	BORING LOG B-5	FIGURE
DRAWN BY:	JDS		
CHECKED BY:	HF	PG&E Thurman Substation 1215 East Thurman Road Lodi, CA	A-6
DATE:	5/29/2019		
REVISED:	6/17/2019		PAGE: 1 of 1

APPENDIX B

LABORATORY TEST RESULTS

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	6.0			8.7	88.7							
B-1	8.0		DARK YELLOWISH BROWN SILTY CLAYEY SAND (SC-SM)					14				
B-1	15.0	4	DARK YELLOWISH BROWN CLAYEY SAND (SC)						23	13	10	
B-2	6.0			15.3	94.8							TXUU: c = 1.95 ksf
B-2	6.5		OLIVE BROWN SANDY LEAN CLAY (CL)					63				
B-3	6.0			10.5	112.2							
B-3	30.0	7	LIGHT OLIVE BROWN SILTY SAND (SM)					16				
B-4	5.5		DARK YELLOWISH BROWN SILTY SAND (SM)					28	NP	NP	NP	
B-4	10.0	3	DARK YELLOWISH BROWN SILTY SAND (SM)					23				
B-5	6.0			18.8	103.7							
B-5	10.0	3	DARK YELLOWISH BROWN SANDY LEAN CLAY (CL)						24	15	9	

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

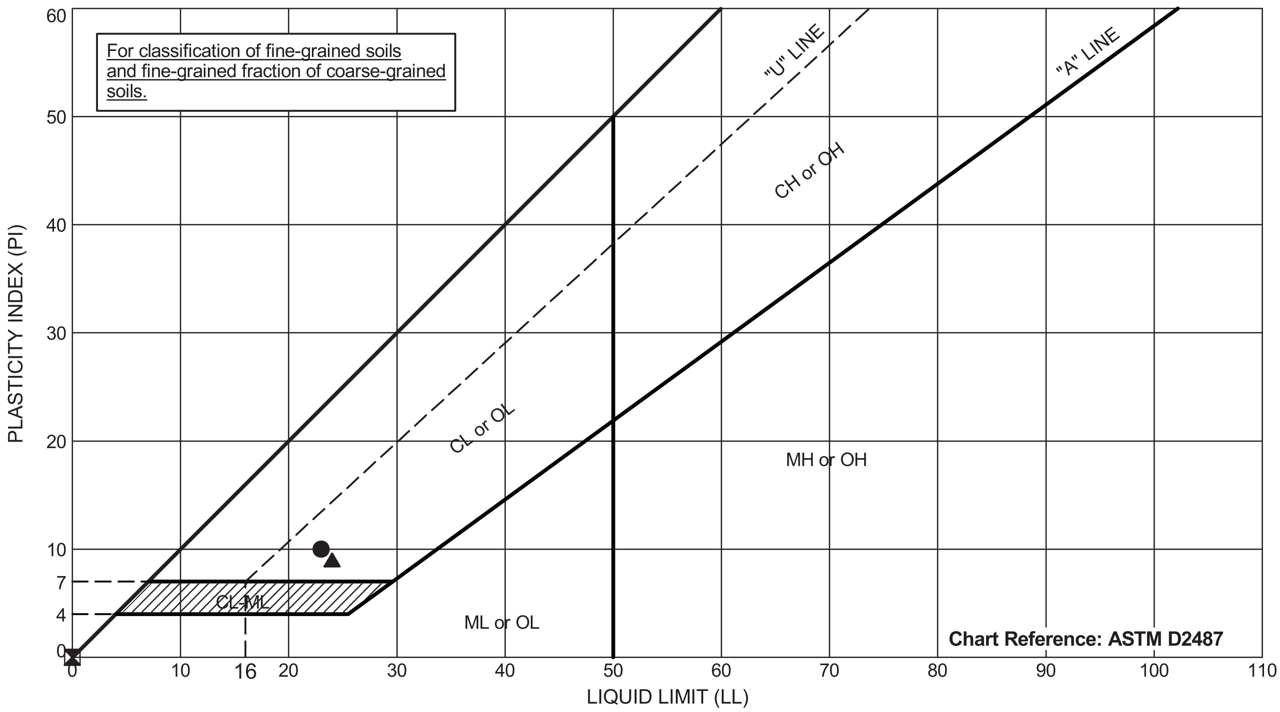
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20193892
DRAWN BY: JDS
CHECKED BY: HF
DATE: 5/29/2019
REVISED: -

LABORATORY TEST RESULT SUMMARY

PG&E Thurman Substation
1215 East Thurman Road
Lodi, CA

FIGURE
B-1



Exploration ID		Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	B-1	15	4	DARK YELLOWISH BROWN CLAYEY SAND (SC)	NM	23	13	10
⊠	B-4	5.5	NA	DARK YELLOWISH BROWN SILTY SAND (SM)	28	NP	NP	NP
▲	B-5	10	3	DARK YELLOWISH BROWN SANDY LEAN CLAY (CL)	NM	24	15	9

Testing performed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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PROJECT NO.:	20193892
DRAWN BY:	JDS
CHECKED BY:	HF
DATE:	5/29/2019
REVISED:	-

ATTERBERG LIMITS

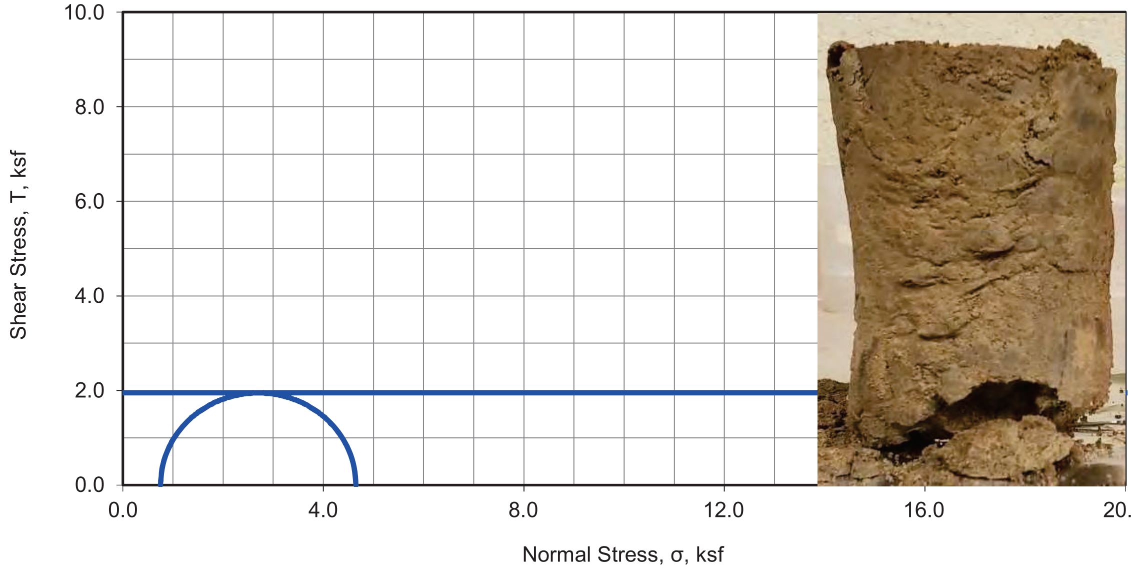
PG&E Thurman Substation
1215 East Thurman Road
Lodi, CA

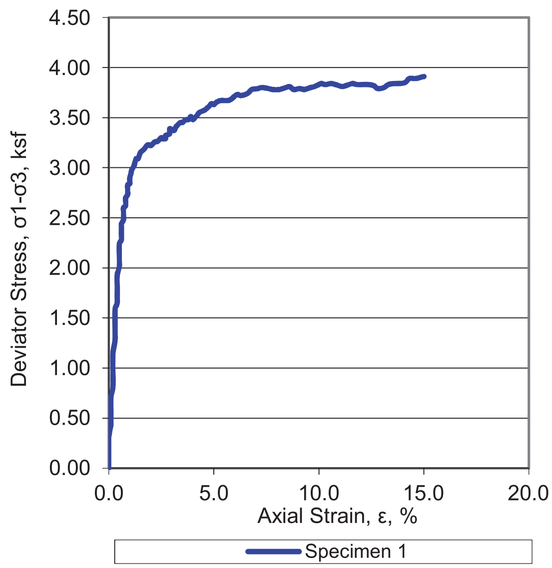
FIGURE

B-1

Total	
c = 1.95	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.63
	Water Content, %	ω_O	15.3
	Dry Density, lbs/ft^3	γd_o	94.8
	Saturation, %	S_O	54
	Void Ratio	e_O	0.744
Minor Principal Stress, ksf		σ_3	0.75
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	3.91
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	15.02
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	3.91
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	15.02

Description of Specimen: Dark Yellowish Brown Sandy Silt (ML)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

Boring:	B-2
Sample:	1B
Depth, ft:	6.0
Test Date:	6/5/19

Remarks: nm= not measured, na = not applicable

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Bright People. Right Solutions.

2601 Barrington Ct, Hayward, CA 94545

Project No.:	20193892
Date:	6/10/19
Entry By:	CP
Checked By:	CP
File Name:	HL12171

TRIAXIAL COMPRESSION TEST (UU)

PG&E THURMAN SUBSTATION
1215 EAST THURMAN ROAD
LODI, CALIFORNIA

Figure

California State Certified Laboratory No. 2153

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

Client: Kleinfelder
Client's Project No.: 20193892
Client's Project Name: PG&E Thurman Substation
Date Sampled: 05/24 - 28/19
Date Received: 5-Jun-2019
Matrix: Soil
Authorization: Laboratory Testing Program

Date of Report: 13-Jun-2019

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1906023-001	4 Bulk	+320	7.21	29,000	22,000	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	75
Date Analyzed:	12-Jun-2019	11-Jun-2019	7-Jun-2019	7-Jun-2019	7-Jun-2019	11-Jun-2019	11-Jun-2019

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

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

APPENDIX C

INFILTRATION TEST DATA

SimplyData Software Suite

Aardvark Permeameter

Location: Thurman Sub
Site: Thurman Sub Boring B-5

Date of Readings:

Time interval: 2 minutes

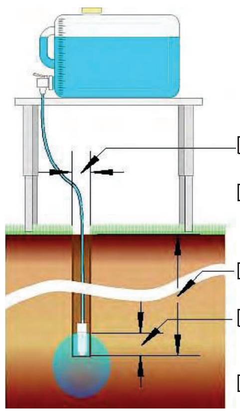
Ksat Method: Earth Manual

Steady Flow Rate achieved when Water Consumption Rate changes less than
+/- 10 % for 3 consecutive readings

Steady Flow Rate:	16.500 ml/min
Tmp Adj Flow Rate:	16.501 ml/min
Percolation Rate:	8.044 min/cm
Ksat:	3.69E-05 cm / sec

Site Details:

Notes:



Site GPS Position

Longitude: 121.2495 degrees West
Latitude: 38.12878 degrees North

5.1 inches — Hole Diameter

60.1 º F — Water Temperature

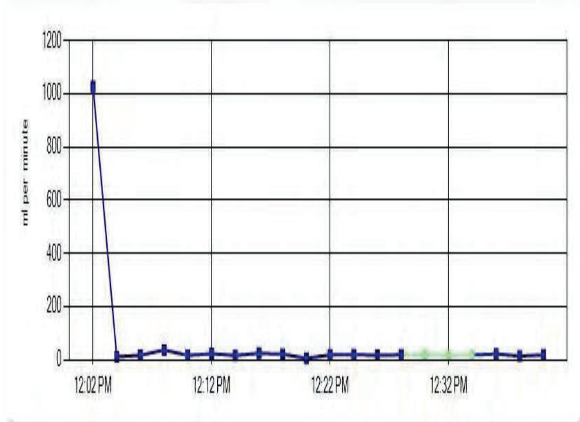
59.8 inches — Hole Depth

72.8 inches — Water Height in Hole

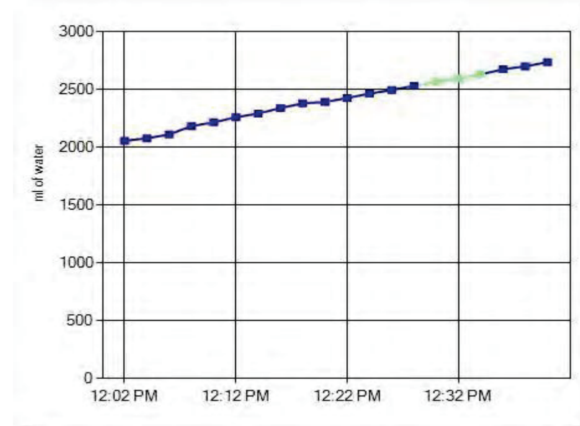
Water Table Depth

Soil Texture Structure Category:
Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.

Water Consumption Rate



Total Water Consumed



Time	Reservoir Water Level (ml)	Elapsed Time Interval (minutes)	Interval Water Consumed (ml)	Total Water Consumed (ml)	Water Consumption Rate (ml / min)	Ignore this Reading?
5/23/2019 12:00:48 P	8363.8	0				
5/23/2019 12:02:48 P	6309.4	2	2054.4	2054.4	1027.2	
5/23/2019 12:04:48 P	6288	2	21.4	2075.8	10.7	
5/23/2019 12:06:48 P	6254.8	2	33.2	2109	16.6	
5/23/2019 12:08:48 P	6182.4	2	72.4	2181.4	36.2	
5/23/2019 12:10:48 P	6150.4	2	32	2213.4	16	
5/23/2019 12:12:48 P	6105.6	2	44.8	2258.2	22.4	
5/23/2019 12:14:48 P	6074	2	31.6	2289.8	15.8	
5/23/2019 12:16:48 P	6026.4	2	47.6	2337.4	23.8	
5/23/2019 12:18:49 P	5985.2	2	41.2	2378.6	20.43	
5/23/2019 12:20:49 P	5974.6	2	10.6	2389.2	5.3	
5/23/2019 12:22:49 P	5939.2	2	35.4	2424.6	17.7	
5/23/2019 12:24:49 P	5902	2	37.2	2461.8	18.6	
5/23/2019 12:26:49 P	5869.6	2	32.4	2494.2	16.2	
5/23/2019 12:28:49 P	5833.8	2	35.8	2530	17.9	
5/23/2019 12:30:49 P	5800	2	33.8	2563.8	16.9	
5/23/2019 12:32:49 P	5768.8	2	31.2	2595	15.6	
5/23/2019 12:34:49 P	5734.8	2	34	2629	17	
5/23/2019 12:36:49 P	5691.6	2	43.2	2672.2	21.6	
5/23/2019 12:38:49 P	5665.4	2	26.2	2698.4	13.1	
5/23/2019 12:40:49 P	5629.8	2	35.6	2734	17.8	

APPENDIX D

GBA INFORMATION SHEET

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733

e-mail: info@geoprofessional.org   www.geoprofessional.org

H3

LEU Industrial and Guild Substations
Geotechnical Investigation Report

Preliminary Subsurface Information for the Electric Industrial Substation Expansion Lodi, California

THERE IS NO EXPRESS OR IMPLIED GUARANTEE AS TO THE ACCURACY OR COMPLETENESS OF THE INFORMATION AND DATA CONTAINED HEREIN, NOR OF THE INTERPRETATION THEREOF BY BURNS & McDONNELL ENGINEERING COMPANY OR ANY OF THEIR REPRESENTATIVES.

THE SUBSURFACE INFORMATION AND DATA CONTAINED HEREIN DO NOT FORM A PART OF ANY CONTRACT DOCUMENT ISSUED BY BURNS & McDONNELL.

Northern California Power Agency
Lodi, California
Project No. 118664
January 2020

Page No.

1.0 GENERAL

This subsurface information document consists of a report titled *Preliminary Geotechnical Report, Lodi Electric Industrial Substation Expansion*, dated December 27, 2019. This *Preliminary Geotechnical Report* was prepared by Kleinfelder, Inc. (Kleinfelder) of Pleasanton, CA.

Neither drilling nor laboratory testing was performed by Kleinfelder in the preparation of the *Preliminary Geotechnical Report*. The *Preliminary Geotechnical Report* was developed by Kleinfelder from other site investigations completed by Kleinfelder in the vicinity of this project site. The *Preliminary Geotechnical Report*, as prepared by Kleinfelder, is included in Appendix A of this document.

2.0 LIMITATIONS

2.1 Document Use

The information presented in this document has been prepared for the use of Burns & McDonnell. No other warranty, express or implied, is made as to the information included in this document. In the event that conclusions and recommendations based on data contained in this document are made by others, such conclusions and recommendations are the responsibility of others.

The information gathered and presented in this document was not obtained for an environmental audit nor to evaluate the potential for hazardous materials at the Site. The equipment, techniques, and personnel used to perform geoenvironmental exploration differ substantially from those applied in soil and foundation engineering.

This document is not intended to be utilized as a Geotechnical Baseline Report.

2.2 Variations

The subsurface information submitted in this document is based upon information obtained from site investigations completed in the vicinity of this Site. This document does not reflect variations which may occur, the nature and extent of which may not become evident until construction is performed. If during construction, soil, rock, and/or groundwater conditions appear to be different from those described herein, Burns & McDonnell should be advised so that recommendations made may be evaluated and modified, if necessary. Fluctuations or changes in water levels and groundwater conditions can be influenced by sources outside the site investigated, by seasonal rainfall, and by changes in drainage conditions in and around the Site.

APPENDIX A – REPORT BY KLEINFELDER

Preliminary Geotechnical Report, Lodi Electric Industrial Substation Expansion
Lodi, California
Report Prepared for Burns & McDonnell
Report Prepared by Kleinfelder, Inc.
Report Dated 12/27/2019

PRELIMINARY GEOTECHNICAL REPORT
LODI ELECTRIC INDUSTRIAL SUBSTATION EXPANSION
1215 EAST THURMAN STREET
LODI, CALIFORNIA

PROJECT NO. 20202783.001A

DECEMBER 27, 2019

December 27, 2019
Project No. 20202783.001A

Michael D. Washburn, Senior Electrical Engineer
Burns & McDonnell
9400 Ward Parkway
Kansas City, MO 64114
Via Email: mdwashburn@burnsmcd.com

SUBJECT: **Preliminary Geotechnical Report**

PROJECT: **Lodi Electric Industrial Substation Expansion**
1215 East Thurman Street
Lodi, California

Dear Mr. Washburn:

The attached report presents Kleinfelder's preliminary geotechnical recommendations for the Lodi Electric Industrial Substation Expansion located in Lodi, California. The report describes the study, findings, conclusions, and recommendations for use in project planning, preliminary design and preparation of preliminary construction specifications. Kleinfelder's services are authorized by our proposal dated October 14, 2019 and were performed in general accordance with the terms of our Master Services Agreement No. 4400007810.

The primary geotechnical concern at this site is shallow foundation support and potential caving of drilled pier excavations due to the loose to medium dense sand soils that are anticipated to be in the subsurface. Based on historical information and Kleinfelder's experience in the area, it is our professional opinion that the subject site is geotechnically suitable for construction of the proposed improvements using conventional grading and shallow and deep foundation systems. Preliminary recommendations for shallow slab, spread footing, and drilled pier foundations are provided in this report. The preliminary recommendations presented herein may be incorporated into project planning, project design, and preparation of construction specifications.

Kleinfelder should review the project plans and specifications when complete to assess if the preliminary recommendations provided herein are consistent with our assumptions and limited understanding of the project. In addition, a final Geotechnical Investigation Report should be prepared that includes nearby subsurface soil data, as permitted, or borings and appropriate testing shall be performed to support the preparation of final plans and specifications for construction.

Kleinfelder appreciates the opportunity to provide geotechnical engineering services to Burns & McDonnell during the planning and preliminary design phase of this project. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully Submitted,

KLEINFELDER, INC.

Alvin Lin
Professional

Stephen P. Plauson, PE, GE
Principal Geotechnical Engineer

cc: Yvonne Barnard (ybarnard@kleinfelder.com)
Kris Johnson (kjjohnson@kleinfelder.com)
Liana Serrano (lserrano@kleinfelder.com)

TABLE OF CONTENTS

SECTION **PAGE**

FIGURES

APPENDICES

1 INTRODUCTION

This report presents the results of a geotechnical engineering data study for the Lodi Electric Industrial Substation Expansion, located at 1215 East Thurman Street in Lodi, California. A site plan and vicinity map are shown on Figure 1. Kleinfelder was retained by Burns & McDonnell to provide geotechnical engineering services for the project. The purpose of this report is to provide preliminary geotechnical engineering recommendations to aid in preliminary project design and preparation of preliminary construction specifications based on Kleinfelder's experience in the area.

1.1 PROPOSED CONSTRUCTION

Project understanding is based on email and telephone correspondence with the project team through September 17, 2019. We understand that Lodi Electric plans to expand the existing Industrial Substation. At this time, foundation loading and dimensions for the aforementioned structures has not been provided. Kleinfelder should review the project plans and specifications when complete to assess if the preliminary recommendations provided herein are consistent with our assumptions and limited understanding of the project. In addition, a final Geotechnical Investigation Report should be prepared that includes nearby subsurface soil data, as permitted, or borings and appropriate testing shall be performed to support the preparation of final plans and specifications for construction.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this data study was to develop geotechnical conclusions and recommendations for use in preliminary project design and specification development. To accomplish these purposes, Kleinfelder's scope of services involves preparing this preliminary report including the following:

- A description of the proposed project including a site vicinity map and site plan.
- General descriptions of the local and regional geology, including a geologic map.
- 2016 California Building Code seismic design criteria.
- Recommendations for site preparation and earthwork.
- Discussion of general earthwork concerns including rock excavation, reuse of onsite soil for engineered fill, and wet weather grading recommendations.
- Recommendations to aid in the design of site drainage.

- General recommendations for concrete slab and/or spread footing foundations to support substation structures, including bearing capacity, lateral resistance, and settlement estimates.
- An axial capacity analysis for a single drilled pier foundation of a unit diameter based on one possible soil profile across the site.
- Recommendations for lateral capacity of deep foundations including one subsurface profile for use in L-pile analysis.
- Recommendations for drilled pier construction, including recommended drilling methods and concrete placement guidelines.
- Comments on the corrosion potential of foundation soil.

2 PREVIOUS STUDIES

2.1 PREVIOUS STUDIES

Kleinfelder has performed multiple investigations in the vicinity of the project site. These nearby investigations were reviewed, and relevant data were used to characterize the subsurface conditions in the vicinity of the project site and to develop preliminary recommendations.

3 GEOLOGIC CONDITIONS

3.1 AREA AND SITE GEOLOGY

The site is located along the central section of the Great Valley geomorphic province 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 mountain ranges were initially formed by regional 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. The deepest and oldest of the sediments that fill the structural trough are marine sediments deposited before the uplift of the Coast Ranges. A mix of marine and continental deposits formed over these older units as seas advanced and retreated in the Sacramento and San Joaquin Valleys. The upper and youngest sediments in the basin are continental deposits consisting of alluvial fan deposits and flood-basin, lake, and marsh deposits.

According to geologic mapping by Marchand and Bartow (1979) and Dawson (2009), the substation area is underlain by Pleistocene aged terrace and alluvial fan deposits of the Upper Modesto formation (see Figure 2). In the project area, these soils generally consist of silts, sands, and gravels with minor clays, which are relatively comparable to the mapped deposits. Regional groundwater levels in the area are greater than 70 feet deep based on DWR well records near the site.

3.2 LOCAL AND REGIONAL FAULTING

The site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site. The nearest zoned faults to the project site are the Greenville fault (located about 32 miles to the southwest), Calaveras fault (located about 44 miles to the southwest), Hayward fault (located about 52 miles to the southwest), and San Andreas fault zone (located about 71 miles to the southwest).

4 SITE CONDITIONS

4.1 SITE AND SURFACE DESCRIPTION

The existing Lodi Electric Industrial Substation is located at 1215 East Thurman Road in Lodi, California. The area of expansion is located within an undeveloped area, directly east of the existing substation at the aforementioned address. The undeveloped area is bordered by developed commercial property on all sides, with East Thurman Road to its south, and rail lines with East Lodi Avenue beyond to the north. The expansion area is relatively level and was observed to be covered with dried vegetation by Kleinfelder staff visiting an adjacent property in May of 2019.

4.2 SUBSURFACE CONDITIONS

Based on previous borings in the vicinity of the subject site and local geology, we anticipate that the subsurface soils include interbedded layers of silty sand, poorly graded sand, and clayey sands. The near surface soils are generally found to be relatively loose in the upper 5 feet and typically increase in relative density with subsequent depth. We also anticipate that there is a layer of disked topsoil in the upper 6 to 12 inches of the site based from review of aerial images from Google Earth, our experience with adjacent properties, and knowledge with typical disking of sites to maintain weed control.

4.3 GROUNDWATER

According to regional well record data published by the State Water Resources Control Board (https://www.waterboards.ca.gov/), regional groundwater levels are generally greater than 70 feet below the ground surface. Regional groundwater was not encountered during our explorations.

It is possible that groundwater conditions at the site could change due to variations in rainfall and runoff, regional groundwater withdrawal or recharge, construction activities, or other factors not apparent at the time the study was performed.

4.4 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions at the site are based on the conditions encountered in the borings drilled for this project. The conclusions and recommendations that follow are based on those interpretations. If soil or groundwater conditions exposed during construction vary from those presented in this report, Kleinfelder should be notified to evaluate whether our conclusions or recommendations should be modified.

5 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, the proposed construction is feasible provided the recommendations presented in this report are incorporated into the project design and construction. The following sections discuss conclusions and recommendations with respect to geologic and seismic hazards, California Building Code (CBC) design considerations, site preparation and grading, and foundation design.

5.1 2016 CBC SEISMIC DESIGN PARAMETERS

5.1.1 Site Class

Based on information obtained from the investigation, published geologic literature and maps, and on our interpretation of the 2016 California Building Code (CBC) criteria, it is our opinion that the project site may be classified as Site Class D, Stiff Soil, according to Section 1613.3.2 of 2016 CBC and Table 20.3-1 of American Society of Civil Engineers (ASCE) 7-10 (2010). Site Class D is defined as a soil profile consisting of stiff soil profile with a shear wave velocity between 600 feet per second and 1,200 feet second, standard penetration test (SPT) blow counts (N-value) between 15 blows per foot and 50 blows per foot, or undrained shear strength between 1,000 pounds per square foot and 2,000 pounds per square foot in the top 100 feet.

5.1.2 Seismic Design Parameters

Approximate coordinates for the site are noted below.

- Latitude: 38.129283331 °
- Longitude: -121.25073160 °

For a 2016 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1), associated soil amplification factors (F_a and F_v), and mapped peak ground acceleration (PGA) are presented in Table 5-1. Corresponding site modified (S_{MS} and S_{M1}) and design (S_{DS} and S_{D1}) spectral accelerations, PGA modification coefficient (F_{PGA}), PGA_M, risk coefficients (C_{RS} and C_{R1}), and long-period transition period (T_L) are also presented in Table 5-1. Presented values were estimated using Section 1613.3 of the 2016 California Building Code (CBC), Chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps (https://seismicmaps.org/).

Table 5-1
Ground Motion Parameters Based on 2016 CBC

Parameter	Value	Reference
S_S	0.725g	2016 CBC Section 1613.3.1
S_1	0.295g	2016 CBC Section 1613.3.1
Site Class	D	2016 CBC Section 1613.3.2
F_a	1.22	2016 CBC Table 1613.3.3(1)
F_v	1.811	2016 CBC Table 1613.3.3(2)
PGA	0.249g	ASCE 7-10 Figure 22-7
S_{MS}	0.884g	2016 CBC Section 1613.3.3
S_{M1}	0.534g	2016 CBC Section 1613.3.3
S_{DS}	0.590g	2016 CBC Section 1613.4.4
S_{D1}	0.356g	2016 CBC Section 1613.4.4
F_{PGA}	1.301	ASCE 7-10 Table 11.8-1
PGA_M	0.325g	ASCE 7-10 Section 11.8.3
C_{RS}	1.1	ASCE 7-10 Figure 22-17
C_{R1}	1.142	ASCE 7-10 Figure 22-18
T_L	12 seconds	ASCE 7-10 Figure 22-12

5.2 LIQUEFACTION

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table but can also occur in non-plastic to low-plasticity, finer-grained soils. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or "cyclic mobility," increased lateral earth pressures on retaining walls, liquefaction settlement, and lateral spreading or "flow failures" in slopes.

Based on the experience in the area and historical depth to groundwater at the site, the potential for liquefaction is considered negligible.

5.3 EXPANSIVE SOILS

Based on experience and historical information in the area, we do not anticipate the surficial soils will shrink or swell significantly as a result of soil moisture content changes.

5.4 SITE PREPARATION

5.4.1 General

Considering site grades are presently well established, site grading is anticipated to be minimal, minus the grading for the proposed pond. General recommendations for site preparation and earthwork construction are presented in the following sections of this report. All earthwork, including excavation, backfill and preparation of subgrade, should be performed in accordance with the geotechnical recommendations presented in this report and applicable portions of the grading code of local regulatory agencies. The grading contractor is responsible to notify governmental agencies, as required, and the geotechnical engineer at the start of site cleanup, the initiation of grading and any time that grading operations are resumed after an interruption. All earthwork should be performed under the observation and testing of a Kleinfelder representative. All references to compaction, maximum density and optimum moisture content are based on ASTM D1557, unless otherwise noted.

5.4.2 Stripping and Grubbing

Any miscellaneous surface or encountered subsurface obstructions, vegetation, debris, or other deleterious materials should be removed from the project area prior to any site grading. Based on experience in the area, the site surface may be loose and contain organics of seasonal vegetation due to previous disking for weed control. The depth of stripping at the time of construction should be enough to remove the visible organics. The stripped materials should not be incorporated into any engineered fill unless they can be thoroughly blended to achieve an organic content less 3 percent by weight and no visible organic matter.

5.4.3 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

Initial site grading should include a reasonable search to locate soil disturbed by previous activity and abandoned underground structures or existing utilities that may exist within the areas of construction. Any loose or disturbed soils, void spaces that may be encountered should be over-excavated to expose firm and relatively unyielding native soil, as approved by a representative of Kleinfelder.

Unless approved otherwise by an on-site representative of Kleinfelder during grading, undocumented fills at the locations of any future grading or shallow foundations should be over-excavated and replaced with engineered fill as recommended below in the “Engineered Fill-Placement and Compaction Criteria” section of this report.

5.4.4 Scarification and Compaction

In areas requiring placement of fill, it is recommended the fill be placed and compacted as engineered fill. Following site stripping and any required grubbing and/or over-excavation, it is recommended areas to receive engineered fill be scarified to a depth of 8 inches, uniformly moisture conditioned to at least the optimum moisture content for sandy soils (SP, SM, SC) or at least 3 percent above the optimum moisture content for clayey soils (CL, CH) and compacted to at least 90 percent relative compaction for sandy soils or between 88 and 92 percent relative compaction for clayey soils, as determined by ASTM D1557.

5.5 ENGINEERED FILL

5.5.1 Onsite Materials

The on-site soil appears suitable for use as engineered fill. All engineered fill should be free of debris, visible organics, or other deleterious materials, and have a maximum particle size less than 3 inches in maximum dimension. Where imported material is brought in, it is recommended that it be granular in nature and conform to the minimum criteria discussed in Table 5-2.

5.5.2 Non-Expansive Engineered Fill Requirements

Specific requirements for engineered fill as well as applicable test procedures to verify material suitability are provided below:

Table 5-2
Engineered Fill Requirements

Fill Requirement		Test Procedures	
		ASTM	Caltrans
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D6913	202
¾ inch	70-100	D6913	202
No. 200	20-50	D6913	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D4318	204
Organic Content			
No visible organics		---	---
Expansion Potential			---
20 or less		D4829	---
Soluble Sulfates			
Less than 2,000 ppm		---	417
Soluble Chloride			
Less than 300 ppm		---	422
Resistivity			
Greater than 2,000 ohm-cm		---	643

Materials to be used for engineered fill should be sampled and tested by Kleinfelder prior to being transported to the site. Highly pervious materials such as clean crushed stone or pea gravel are not recommended for use in engineered fill because they can permit transmission of water into the underlying materials. We recommend representative samples of imported materials proposed for use as engineered fill be submitted to Kleinfelder for testing and approval at least one week prior to the start of grading and import of this material.

In addition, we recommend that a laboratory corrosion test series (pH, resistivity, redox, sulfides, chlorides, and sulfates) be performed on all proposed import materials.

5.5.3 Placement and Compaction Criteria

Non-expansive soils that meet the criteria outlined in Table 5-2 that are to be used for engineered fill should be uniformly moisture conditioned to at least the optimum moisture content, placed in horizontal lifts less than about 8 inches in loose thickness, and compacted to at least 90 percent relative compaction, as determined by ASTM D1557. Onsite clayey soils to be used for general fill where engineered fill is not required should be uniformly moisture conditioned to at least 4 percent over the optimum moisture content, placed in horizontal lifts no more than about 8 inches in loose thickness, and compacted to between 88 and 92 percent relative compaction, as determined by ASTM D1557.

Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or moisture content, or if soil conditions are not stable. Disking or blending may be required to uniformly moisture condition soils used for engineered fill. Ponding or jetting compaction methods should not be allowed.

All site preparation and fill placement should be observed by Kleinfelder. It is important that during the stripping and scarification processes, a representative of Kleinfelder be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during the (future final) geotechnical site exploration.

5.6 WET WEATHER CONSIDERATIONS

Should construction be performed during or subsequently after 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 geogrid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork and construction operations.

5.7 SITE DRAINAGE

Final site grading should provide surface drainage away from all structures and areas to be traversed by vehicles and maintenance equipment. In general, we recommend consideration be given to providing at least 2 percent slope away from structure foundations or access ways.

5.8 TEMPORARY EXCAVATIONS

5.8.1 General

All excavations should comply with applicable local, state, and federal safety regulations including the current Occupational Safety & Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the Contractor, who is 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.

5.8.2 Excavation and Slopes

Excavated slope height, slope inclination, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties.

Underground utilities should be located above a 1H:1V (horizontal to vertical) plane projected down and out from the bottoms of new footings to avoid undermining the footings during the excavation of the utility trench.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should be kept sufficiently away from the top of any excavation to prevent any unanticipated surcharging. Alternatively, excavation slopes and shoring systems can be designed to accommodate surcharge loadings, if necessary. Shoring, bracing, or underpinning required for the project (if any), should be designed by a professional engineer registered in the State of California.

5.9 TRENCH BACKFILL

All trench backfill should be placed and compacted in accordance with recommendations provided for engineered fill (see Section 5.5). Mechanical compaction is recommended. Ponding or jetting should not be used as a sole means of soil compaction.

5.10 SHALLOW FOUNDATIONS

This section provides general preliminary recommendations for shallow foundations. Kleinfelder should review the design to ensure compliance with the intent of the preliminary geotechnical conclusions and recommendations provided in this report. In addition, a final Geotechnical Investigation Report should be prepared that includes nearby subsurface soil data, as permitted, or borings and appropriate testing shall be performed to support the preparation of final plans and specifications for construction.

Foundations should satisfy two independent criteria with respect to foundation soils. First, the foundation should have an adequate safety factor against bearing failure with respect to the shear strength of the foundation soils. Second, the vertical movements of the foundation due to settlement (both immediate elastic settlement and consolidation settlement) should be within tolerable limits for the structure. Depending on the settlement tolerance of planned structures, design loading, and foundation dimensions, the general recommendations presented in this report may be subject to modification. If future project needs require additional foundation capacity, Kleinfelder should be contracted to evaluate this potential for specific foundation designs.

Structures may be supported on conventional, shallow, reinforced concrete mat foundations or spread footings, provided the site structures can tolerate the anticipated settlement.

5.10.1 Spread Footings

5.10.1.1 Allowable Bearing Pressure

Shallow spread footings constructed of reinforced concrete may be founded on approved undisturbed native soil and/or engineered fill. The footings should be founded at least 18 inches below lowest adjacent finished grade on subgrade soils that have been prepared in accordance with the recommendations provided in this report. Continuous and isolated rectangular footings should have a minimum width of 12 inches.

For foundation subgrade prepared in accordance with the recommendations provided in this report, spread and strip footings may be designed for a net allowable bearing pressure of up to 2,000 pounds per square foot (psf) due to dead plus live loads. The weight of the foundation that extends below grade may be neglected when computing dead loads. The allowable bearing pressure includes a safety factor of at least 3 with respect shear failure of the foundation soils and may be increased by one-third for transient loading due to wind or seismic forces.

To maintain the desired support, foundations adjacent to utility trenches or other existing foundations should be deepened so that their bearing surfaces are below an imaginary plane having an inclination of 1 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent foundations or utility trenches.

5.10.1.2 Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.39 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 360 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches should be neglected unless the area in front of the footing is protected from disturbance by concrete or pavement. The allowable friction coefficient and passive resistance may be used concurrently.

5.10.1.3 Settlement

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Foundation dimensions and loads have not been provided for the proposed structures, we estimate maximum total settlement of foundations designed and constructed in accordance with the preceding recommendations of up to about ½ inch or less. Differential settlement between similarly loaded, adjacent footings are estimated to be about half the total settlement. The majority of foundation settlement is expected to occur rapidly and should be essentially complete shorty after initial application of the loads.

5.10.1.4 Shallow Foundation Construction Considerations

Prior to placing steel or concrete, foundation excavations should be cleaned of any debris, disturbed soil or water. All foundation excavations should be observed by a representative of Kleinfelder just prior to placing fill and/or steel or concrete. The purpose of these observations is to check that the bearing soils actually encountered in the foundation excavations are similar to those assumed in analysis and to verify the recommendations contained herein are implemented during construction.

5.10.2 Mat Foundations

Preliminary recommendations for design and construction of small mat slab foundations up to about 25 feet wide are presented below. Kleinfelder should be consulted to provide supplementary mat foundation recommendations if larger mat slab foundations are planned in the future.

5.10.2.1 Allowable Bearing Pressure

For subgrades prepared as recommended in this report, reinforced concrete mat foundations may be designed for a net allowable bearing pressure of 2,000 psf. If higher allowable bearing capacity for mat foundations is required, Kleinfelder should be consulted to provide supplemental engineering and construction recommendations on a case-by-case basis. The allowable bearing pressure applies to dead plus live loads, includes a safety factor of at least 3 with respect to shear failure of the foundation soils, and may be increased by one-third for short-term loading due to wind or seismic forces.

5.10.2.2 Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.39 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 360 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches should be neglected unless the area in front of the foundation is protected from disturbance by concrete or pavement. The friction coefficient and passive resistance may be used concurrently.

5.10.2.3 Subgrade Modulus

For preliminary design purposes, a modulus of subgrade reaction, K_{v1}, of 150 pounds per square inch per inch of deflection (for a 1 square-foot bearing plate) may be used for design of mat slabs. The modulus should be adjusted for the actual slab size using appropriate formulas or software.

5.10.2.4 Mat Slab Settlement

For foundations with design pressures equal to or less than the net allowable pressure provided above, and under static loading conditions, total post-construction foundation settlement is expected to be less than about ½ inch at the center of the mat foundations. Post-construction differential settlement of individual foundation elements is expected to be about one-half the total settlement.

These settlement estimates are based on the assumption that the foundation subgrade is properly prepared, and the foundations are designed and constructed in accordance with the recommendations presented in this report.

5.10.2.5 Mat Foundation Construction Considerations

Underground utilities that are 4 feet deep or shallower and that run parallel to shallow mat foundations generally should be located no closer than 2 feet horizontally away from the perimeter edges of the slab. Deeper utilities should be located above a 1H:1V (horizontal to vertical) slope projected downward from the bottom edges of the slab. Utility plans should be reviewed by Kleinfelder prior to trenching to evaluate conformance with this requirement.

Beneath exterior cast-in-place concrete mat foundations, we recommend the design include a base course of well-graded crushed aggregate base at least 6 inches thick. Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate base. Under slabs that will be subject to vehicle loading, the aggregate base course thickness should be increased to a minimum of 6 inches. The base course should be compacted to at least 95 percent relative compaction at optimum moisture content. Thickened slab edges embedded to at least 18 inches below grade need not be underlain by the gravel base course.

5.11 DRILLED PIER FOUNDATIONS

Preliminary recommendations for design and construction of drilled pier foundations are presented in the following sections of this report. Kleinfelder should review the design to ensure compliance with the intent of the preliminary geotechnical conclusions and recommendations provided in this report. In addition, a final Geotechnical Investigation Report should be prepared that includes nearby subsurface soil data, as permitted, or borings and appropriate testing shall be performed to support the preparation of final plans and specifications for construction.

5.11.1 Axial Capacity

Axial pile capacity was developed based on Federal Highway Administration methods using the commercial computer software SHAFT, version 2017, produced by Ensoft, Inc. Static soil strength parameters are based on strength and soil properties measured during the field and laboratory testing phases of this investigation.

Axial loads on drilled piers should be supported by the frictional capacity of the pier. End bearing is not considered in the axial capacity due to strain incompatibility issues between skin friction and end bearing, settlement issues, and the potential for loose materials to exist at the bottoms of the pier holes during construction that cannot be effectively cleaned out. If additional axial capacity is required beyond what is provided in this report, Kleinfelder should be consulted to provide a portion of end bearing capacity and additional construction recommendations.

A curve illustrating the ultimate axial compressive capacity of a unit (1-foot) diameter straight-sided drilled pier installed from the existing grade under static conditions is shown on Figure 3a.

Corresponding tabulated values are presented on Figure 3b. Capacities for drilled piers with diameters other than 1 foot may be obtained by multiplying the capacity for the 1-foot diameter pier by the actual pier diameter (in feet). For evaluation of allowable axial capacity under static conditions, we recommend a factor of safety of 3 be applied to the ultimate capacity (per the General Order 95 code). Note that the weight of the foundation need not be considered for evaluation of allowable axial capacity.

Ultimate tensile capacity may be obtained by multiplying the compressive capacity by a factor of 0.8 and adding the weight of the foundation. For allowable tension capacity under transient flood, wind or seismic conditions, a safety factor of at least 1.5 should be used. For allowable sustained tension, a safety factor of 3 should be used.

5.11.1.1 Estimated Settlement

Based on the methods outlined by FHWA Drilled Shaft Manual, Brown et al. (2010), total static settlement of each drilled pier should be on the order of 0.1 percent of the pier diameter for a drilled pier designed and constructed in accordance with the recommendations presented in this report. This value includes elastic compression of the pile under design loads. The majority of the settlement should occur during and shortly after application of the structure loads. We suggest allowing for about ¼ inch of settlement to accommodate potential long-term settlement, construction issues, and some soil variability across the site.

5.11.1.2 Axial Capacity Group Effects

The axial capacity of piers developed in accordance with the recommendations provided above applies to single, isolated piers. Consideration of group effects on axial capacity of drilled piers is usually not necessary for piers with center-to-center spacings of at least 3 effective diameters. For closer spacings the capacity of individual piers will be reduced. For these cases Kleinfelder should be consulted to evaluate axial capacity on a case-by-case basis. Note that group effects should also be considered where new foundations are constructed immediately adjacent to existing foundations.

5.11.2 Lateral Response

5.11.2.1 LPILE Analysis Soil Parameters

Lateral capacity of drilled piers may be developed through analysis of pier response due to a range of design loads. Table 5-3 contains recommended input soil parameters for lateral response analysis of deep foundations using the LPILE computer program (by Ensoft, Inc., Version 2018. Program default values may be used for strain factor (E_{50}) and horizontal subgrade reaction (K).

Table 5-3
LPILE Geotechnical Parameters
Static Conditions

Depth (feet)	Model P-Y Curve	Effective Unit Weight (lb/ft^3)	Cohesion c (psf)	Internal Friction Angle, Φ (degrees)
0 to 30	Sand (Reese)	115	-	30

LPILE analyses determinations could not be performed at this time, as the loading of individual piles have not yet been established by the Burns & McDonnell. When loading is available, Kleinfelder can provide Lpile analysis for an additional fee.

5.11.3 Drilled Pier Construction Considerations

Successful completion of drilled pier foundations requires good construction procedures. Drilled pier excavations should be constructed by a skilled operator using techniques that allow the excavations to be completed, the reinforcing steel placed, and the concrete poured in a continuous manner to reduce the time that excavations remain open. Steel reinforcement and concrete should be placed on the same day of completion of each pier excavation. Additionally, drilled pier excavations should be scheduled to allow concrete in each pile to set over night before drilling adjacent holes that are closer than 4 diameters center-to-center.

The following considerations should be implemented during construction of drilled shaft foundations. We recommend the contractor follow the procedures for drilled pier construction contained in the Federal Highway Administration (FHWA) manual on drilled shaft construction (Brown et al., 2010).

Consistent with Chapter 17 of the 2016 CBC, drilled pier excavations should be inspected and approved by the geotechnical engineer prior to installation of reinforcement. The depths of all pier excavations should be checked immediately prior to concrete placement to verify excessive sloughing and/or caving has not reduced the required hole depth. This may be done with a weighted tape measure or similar measuring device.

As described above, loose sandy soils may be encountered during drilled pier construction. In addition, perched groundwater depending on local rainfall and runoff patterns may also be present at the time of construction. The contractor should be prepared to handle caving sandy soil and possibly of perched groundwater conditions during construction of drilled piers at the site.

The depth to regional groundwater is on the order of 70 feet bgs, therefore, it is unlikely that drilled shafts will encounter regional groundwater. If drilled shaft excavations extend below groundwater levels, the excavations should be cleaned such that less than about 1 inch of loose soil remains at the bottom of the drilled hole. Since the piers should be designed to derive their support in skin friction along the sides of the shafts, consideration could be given to over-drilling the shafts to accommodate any sloughing that may occur between drilling and concrete placement. It is recommended that a representative from Kleinfelder observe each drilled shaft excavation to verify soil and excavation conditions prior to placing steel reinforcement or concrete.

Steel reinforcement and concrete should be placed on the same day the drilled hole is completed to reduce the potential for caving and reduce the quantity of suspended soil particles that may settle to the bottom of the hole during wet-method construction. Excavation depths should be checked several times before concrete placement to ensure excessive sedimentation has not occurred. Concrete used for pier construction should be discharged vertically into the drilled hole to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during shaft construction. 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.

In order to develop the design skin friction values provided in the axial capacity figures, concrete used for drilled pier 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 pile.

If water or drilling fluids are present during concrete placement, concrete should be placed into the hole using tremie methods. Tremie concrete placement should be performed in strict accordance with ACI 304R. The tremie pipe should be rigid and remain below the surface of the in-place concrete at all times to maintain a seal between the water or slurry and 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 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 thick but will depend on the pile diameter, amount of water seepage, and construction workmanship.

Loose sandy soils will likely be encountered during drilled pier construction. Use of slurry drilling methods will likely be needed to reduce the potential for caving in the drilled pier excavations. Use of slurry drilling methods normally requires experienced construction personnel to batch and mix the slurry, test the slurry for proper mixing, hydration, viscosity and other important properties, and to monitor slurry performance during drilling. If slurry drilling methods are used, we recommend use of a polymer slurry that meets Caltrans requirements for drilled shaft construction or bentonite-based slurry, mixed and used in accordance with the guidelines in the FHWA Drilled Shaft Manual (Brown et al., 2010). This guideline recommends bentonite slurry mixtures not be left in the hole for more than about 4 hours in order to avoid potential side friction losses that may be caused by excessive thickness of bentonite filter cake on the hole wall.

If caving conditions are encountered in a drilled pier excavation and there are no overhead clearance issues, temporary casing could be used to help mitigate this condition. If temporary steel casing is used, it should be removed from the hole as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete during casing withdrawal and concrete placement operations. Casing should not be withdrawn until sufficient quantities of concrete have been placed into the excavation to balance the groundwater head outside the casing. Continuous vibration of the casing or other methods may be required to reduce the potential for voids occurring within the concrete mass during casing withdrawal. Corrugated metal pipe should not be used as casing. In no case should casing material be left in the excavation after concrete has been placed without the approval of the project structural and geotechnical engineers. Concrete should be in direct contact with the surrounding soil or the design parameters and recommendations in the geotechnical report are not valid.

5.12 SOIL CORROSION

Based on historical information and past experience in the area, Kleinfelder anticipates that the potential for the soils at the site to be corrosive to concrete elements and buried ferrous metal piping, cast iron pipes, or other objects made of these materials is negligible. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

6 ADDITIONAL SERVICES

6.1 PREPARATION OF FINAL GEOTECHNICAL INVESTIGATION REPORT

The preliminary recommendations herein are intended to support planning and preliminary design of the new expansion planned at the Lodi Electric Industrial Substation in Lodi, California. Kleinfelder should conduct a general review of the preliminary plans and specifications to evaluate that the earthwork and foundation recommendations presented in this report have been properly interpreted and implemented during design. In addition, a final Geotechnical Investigation Report should be prepared that includes nearby subsurface soil data, as permitted, or borings and appropriate testing shall be performed to support the preparation of final plans and specifications for construction.

6.2 PLANS AND SPECIFICATIONS REVIEW

Kleinfelder should conduct a general review of the final plans and specifications to evaluate that the earthwork and foundation recommendations have been properly interpreted and implemented during design. In the event Kleinfelder is not retained to perform this recommended review, no responsibility for misinterpretation of the recommendations by Kleinfelder is accepted.

6.3 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that all earthwork and foundation construction be monitored by a representative from Kleinfelder, including site preparation, placement of all engineered fill and trench backfill, construction of slab and all foundation excavations. The purpose of these services is to observe the soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

7 LIMITATIONS

This report presents information for planning and preliminary design of the new expansion, planned at the Lodi Electric Industrial Substation in Lodi, California. Preliminary recommendations contained in this report are based on historical information, experience in the area, geologic interpretation based on published articles and geotechnical data, and our present knowledge of the proposed construction.

It is possible that soil conditions could vary beyond the points explored. If the scope of the proposed construction, including the proposed location, changes from that described in this report, we should be notified immediately in order that a review may be made, and any supplemental recommendations provided.

We have prepared this report in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty expressed or implied is made.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on-site and off-site) 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.

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FIGURES

LEGEND

PROPOSED EXPANSION

NOTE:
BASE MAPPING AND VICINITY MAP CREATED FROM LAYERS COMPILED BY ESRI PRODUCTS AND 2019 MICROSOFT CORPORATION.
COORDINATE SYSTEM: NAD 1983 2011 STATEPLANE CALIFORNIA III FIPS 0403

VICINITY MAP

NOT TO SCALE

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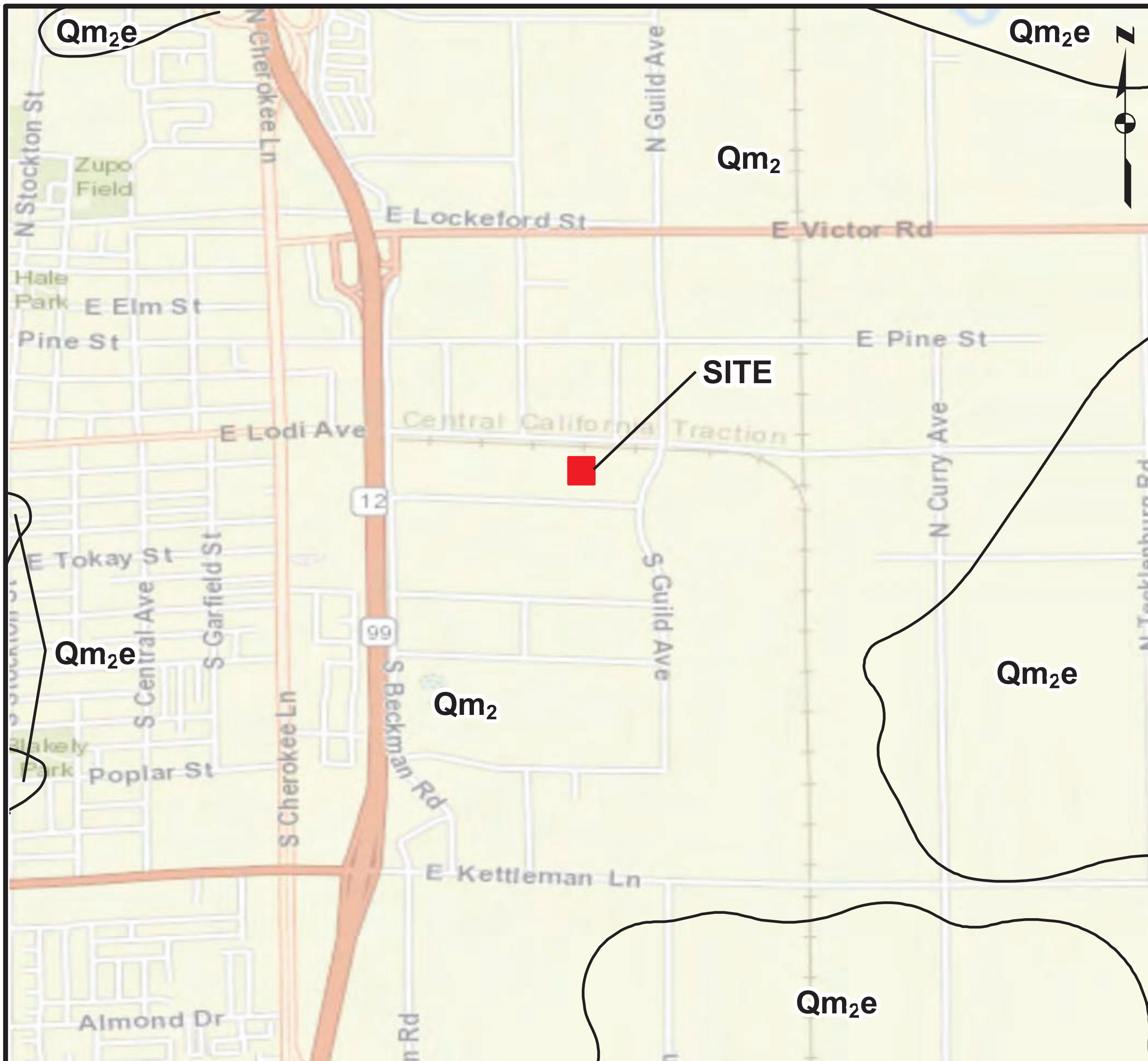
0 200 400

1 " = 200 SCALE IN FEET

KLEINFELDER Bright People. Right Solutions.	PROJECT NO. 20202783.001A	SITE PLAN AND VICINITY MAP	FIGURE
	DRAWN BY: JSala	Lodi Electric Industrial Substation Expansion	1
	CHECKED BY: A. Lin	1215 Thurman Street	
	DATE: 12-09-2019	Lodi, California	



0 1,000 2,000 4,000 Feet

Service Layer Credits: Sources: Esri, HERE, DeLorme, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), MapmyIndia, NGCC, © OpenStreetMap contributors, and the GIS User Community

LEGEND

GEOLOGY

Geologic contact

Qm$_2$ - Riverbank formation, upper member, undivided

Qm$_2$e - Riverbank formation, upper member, eolian dune sand deposits

Source:
Marchand, D.E., and Barstow, J.A., 1979, “Preliminary geologic map of Cenezoic deposits of the Bellota quadrangle, California: U.S. Geological Survey, Open-File Report OF 79-664”, scale 1:62,500

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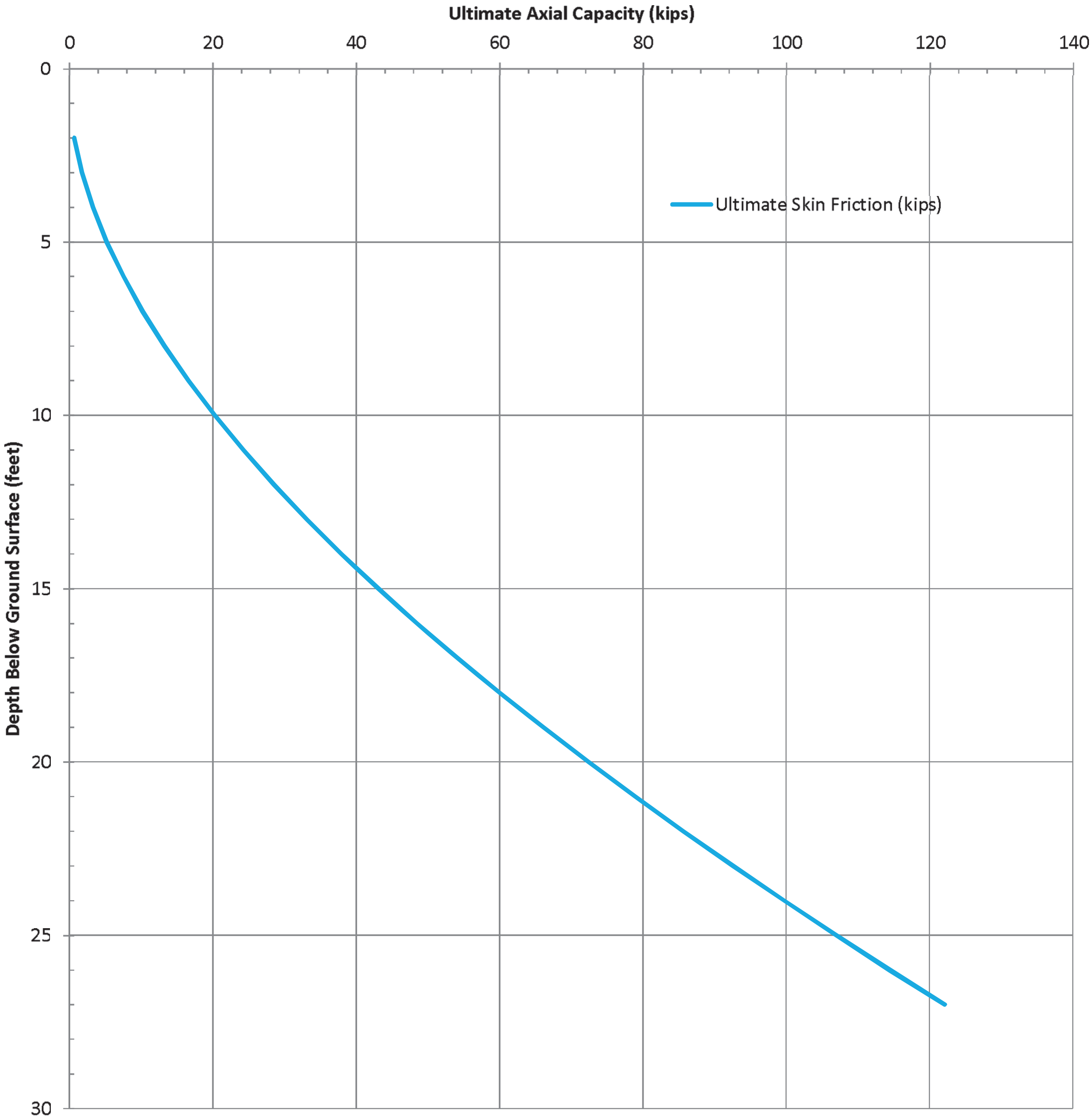
PROJECT NO	20202783
DRAWN:	12/26/2019
DRAWN BY:	JP
CHECKED BY	AL
FILE NAME:	LodiElec_Geo.MXD

GEOLOGY MAP

LODI ELECTRIC INDUSTRIAL SUBSTATION EXPANSION
1215 THURMAN STREET
LODI, CALIFORNIA

FIGURE

2



Notes:

1. Axial capacities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the pier (in feet).
2. Ultimate tensile capacity may be obtained by multiplying the ultimate compressive capacity by a factor of 0.8.
3. The curve represents ultimate axial capacity of a straight-sided drilled pier. See text discussion for factor of safety and group effects.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20202783.001A DRAWN BY: AL CHECKED BY: DATE: 12/10/2019 REVISED:	ULTIMATE AXIAL CAPACITY TABLE UNIT DIAMETER (1-FOOT) DRILLED PIER STATIC CONDITION LODI ELECTRIC INDUSTRIAL SUBSTATION EXPANSION 1215 EAST THURMAN STREET LODI, CA	FIGURE 3A

Depth (ft)	Ultimate Axial Capacity (Kips)	Depth (ft)	Ultimate Axial Capacity (Kips)
2	0.7	[illegible]	48.5
3	1.7	16	54.1
4	3.3	17	60.0
5	5.2	18	66.1
6	7.5	19	72.4
7	10.2	20	78.9
8	13.2	21	85.7
9	16.6	22	92.6
10	20.3	23	99.7
11	24.3	24	107.0
12	28.6	25	114.5
13	33.1	26	122.1
14	38.0	27	

Notes:

1. Axial capcities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the pier (in feet).
2. Ultimate tensile capacity may be obtained by multiplying the ultimate compressive capacity by a factor of 0.8.
3. The curve represents ultimate axial capacity of a straight-sided drilled pier. See text discussion for factor of safety and group effects.

KLEINFELDER
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PROJECT NO.:	20202783.001A	ULTIMATE AXIAL CAPACITY TABLE UNIT DIAMETER (1-FOOT) DRILLED PIER STATIC CONDITION	FIGURE
DRAWN BY:	AL		
CHECKED BY:		LODI ELECTRIC INDUSTRIAL SUBSTATION EXPANSION	3B
DATE:	12/10/2019	1215 EAST THURMAN STREET	
REVISED:		LODI, CA	

APPENDIX A

GBA INFORMATION SHEET

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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