GEOTECHNICAL INVESTIGATION REPORT
PACIFIC GAS AND ELECTRIC COMPANY
FULTON-FITCH TSP REPLACEMENT PROJECT
SONOMA COUNTY, CALIFORNIA

PROJECT NO. 20190527.001A

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Attention: Henry Ho, PE
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SUBJECT: Geotechnical Investigation Report

PROJECT: PG&E Fulton-Fitch TSP Replacement Project
Sonoma County, California

Dear Mr. Ho:

This report presents the results of Kleinfelder's geotechnical investigation for the proposed Fulton-Fitch TSP Replacement Project in Santa Rosa, California. The purpose of our investigation was to explore and evaluate the geologic and subsurface conditions along the proposed replacement alignment in order to develop geotechnical engineering recommendations for project design, specification development, and construction.

Based upon the results of our field exploration and laboratory testing programs, it is our professional opinion that the proposed tubular steel poles can be supported on reinforced concrete drilled pier foundations. The soil conditions encountered in the borings drilled for this investigation vary somewhat in strength, density, and in engineering characteristics along the alignment. Based on the results of our field investigation and laboratory testing, we have grouped the alignment into three reaches:

- Reach 1 (South Reach) – Poles 7_A/B through Pole 13
- Reach 2 (Central Reach) – Pole 14 through Pole 22
- Reach 3 (North Reach) – Pole 23

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Kleinfelder appreciates the opportunity to provide geotechnical engineering services to PG&E. If there are any questions concerning the information presented in this report, please contact us.

Sincerely,

KLEINFELDER, INC.

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1. INTRODUCTION

1.1 GENERAL

This report presents the results of the geotechnical investigation conducted for the proposed tubular steel pole (TSP) replacements in a line segment starting near Fulton substation and continuing north to Faught Road near Shiloh Ranch Regional Park. A site location map and site plan showing the exploration locations are shown on Figures 1 and 2, respectively. Larger scale site plans showing the proposed TSP replacement locations are provided on Figures 3 through 6.

Conclusions and recommendations presented in this report are based on the subsurface conditions encountered at the locations of our explorations. Recommendations presented herein should not be extrapolated to other areas or used for other projects without our prior review.

1.2 PROJECT DESCRIPTION

Our understanding of the project is based on correspondence with PG&E, a conference call on April 5, 2018 with PG&E, and a review of CPUC records. We understand that 21 TSPs will be replaced along an approximately 9,000-foot long segment of the southern "Fulton Shiloh Segment", which includes the line segment between the Fulton Substation and Faught Road near Shiloh Ranch Regional Park. The poles slated for replacement are Poles 7_A/B through Pole 23, as shown on Figures 3 through 6.

1.3 SCOPE OF SERVICES.

The purpose of this investigation was to explore and evaluate subsurface conditions at the site and develop conclusions and recommendations to guide geotechnical aspects of project design, specification development, and construction. Our scope of work includes the following:

- Field exploration including drilling five soil borings to depths of approximately 44 to 61½ feet to explore subsurface conditions and to obtain samples for laboratory testing.
- Laboratory testing to evaluate pertinent geotechnical engineering parameters.
- Analyses of the field and laboratory data to develop conclusions and recommendations for design and construction of the replacement TSP foundations.
- Preparation of this report.

2. FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration program was conducted from July 16, 2018 to July 20, 2018 and included the drilling of five borings, as described below.

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

2.1.1 Exploratory Borings

Beginning on July 16, 2018, five borings, Boring KB-1 through Boring KB-5, were drilled sequentially to depths ranging from approximately 44 to 61½ feet below the existing ground surface. The borings were cleared to a depth of 5 feet using hand auger methods to confirm the absence of utilities or other buried obstructions. All five borings were drilled by Taber Drilling of West Sacramento, California. All five borings were drilled using a CME-55 track drill rig using a 6-inch solid-flight auger, switching to mud rotary drilling with a 4.5-inch bit upon encountering groundwater or reaching a depth of 20 feet. The approximate boring locations are shown on Figure 2, and on Figures 3, 4, and 6. Horizontal coordinates and elevations of the boring were not surveyed. Latitude, longitude and elevation shown on the boring logs were estimated using Google Earth.

A Kleinfelder professional maintained logs of the borings, visually classified the soils encountered according to the Unified Soil Classification System (presented on Figure A-1 in Appendix A), and obtained samples of the subsurface materials. Soil classifications made in the field from samples and auger cuttings were made in general 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. Correction factors for sampler size were applied to the raw sampler blow counts to estimate the sample apparent density noted on the boring logs.

Keys to the soil descriptions and symbols used on the boring log are presented on Figures A-1 and A-2 in Appendix A. The boring logs are presented on Figures A-3 through A-7.

After the borings were completed, they were backfilled with cement grout per Sonoma County standards in accordance with the conditions of our drilling permit. Drilling spoils were contained in 55-gallon drums and staged at the Kleinfelder Santa Rosa office for subsequent testing, and eventual disposal after PG&E provided approval for disposal as non-hazardous soil.

2.1.2 Sampling Procedures

Bulk soil samples were collected from each boring within the upper 5 feet during hand-augering. Driven samples were then collected at depth intervals ranging from approximately 2.5 to 5 feet. Samples were collected from the boring 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 into undisturbed soil, or less when practical refusal was encountered. The samplers were driven using a 140-pound automatic hammer free-falling a vertical distance of 30 inches. Blow counts were recorded at 6-inch intervals for each sample attempt and are reported on the boring logs.

The SPT sampler was used without liners, although the sampler had space for them. 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 boring were packaged and sealed in the field to reduce moisture loss and disturbance. Following drilling, the samples were delivered to our laboratory for further examination and testing.

2.2 LABORATORY TESTING

Kleinfelder performed laboratory tests on selected samples recovered from the boring to evaluate their physical and engineering characteristics. The following laboratory tests were performed:

Geotechnical Testing

- Moisture Content (ASTM D2216)
- Unit Weight (ASTM D2937)
- Grain-size analyses (ASTM D422)

- Atterberg Limit testing (ASTM D4318)
- Unconsolidated-Undrained Triaxial Compression testing (ASTM D2850)

Corrosivity Testing

- Redox (ASTM D1498)
- pH (ASTM D4972)
- Resistivity, As Received (ASTM G57)
- Resistivity, 100% saturation (ASTM G57)
- Sulfide, 100% saturation (ASTM D4658M)
- Soluble Chloride and Sulfate Content (ASTM D4327)

The geotechnical laboratory results are presented in Appendix B and on the boring logs. The corrosivity testing results are presented in Appendix C and in Section 5.7 of this report.

3. GEOLOGIC CONDITIONS

3.1 REGIONAL GEOLOGY

The alignment is located along the east margin of the northern Santa Rosa Valley, in Sonoma County, California, within the Coast Range Geomorphic Province of Northern California. This province is generally characterized by northwest-trending mountain ranges and intervening valleys, which are a reflection of the dominant northwest structural trend of the bedrock in the region. The basement rock in the northern portion of this province consists of the Great Valley Complex, a Jurassic (approximately 145 to 175 million years old) volcanic ophiolite sequence with associated Lower Cretaceous to Upper Jurassic (approximately 100 to 160 million years old) sedimentary rocks, and the Franciscan Complex, a subduction complex of diverse groups of igneous, sedimentary, and metamorphic rocks of Cretaceous to Upper Jurassic age (65 to 160 million years old). The Great Valley Complex was tectonically juxtaposed with the Franciscan Complex (most likely during subduction accretion of the Franciscan Complex), and these ancient fault boundaries are truncated by a modern right-lateral fault system that includes the San Andreas, Hayward-Rodgers Creek, and Maacama faults. Located approximately 19.8 miles southwest of the site, the San Andreas fault defines the westernmost boundary of the local bedrock. In the site vicinity, the Great Valley Sequence and Franciscan Complex are unconformably overlain by Tertiary age (approximately 2.6 to 65 million years old) continental and marine sedimentary and volcanic rocks. These Tertiary age rocks are locally overlain by younger Quaternary (approximately 2.6 million years old to present day) alluvial, colluvial and landslide deposits.

3.2 SITE GEOLOGY

The geology along the alignment has been mapped by Witter et al. (2006), and Delattre (2011), among others. Witter et al. (2006) indicate the majority of the alignment is underlain by Holocene age (approximately 11,700 years old to present day) alluvial fan deposits, consisting of sand, gravel, silt, and minor clay. The active Mark West Creek channel has been mapped as being underlain by historical stream channel deposits, consisting of sand, gravel, and cobbles, with minor silt and clay. The low hills within the Regional Park at the north end of the alignment are shown to be underlain by Pre-Quaternary deposits or bedrock. Witter et al. (2006) indicate the Holocene alluvial fan deposits have moderate liquefaction susceptibility, while the historic stream channel deposits have very high liquefaction susceptibility, and the bedrock has very low liquefaction susceptibility.

Delattre (2011) indicates the low hills within the Regional Park are underlain by Plio-Pleistocene age (approximately 11,700 to 5.3 million years old) fluvial deposits, comprised of weekly consolidated gravel, tuffaceous sand, silt, clay and reworked tuff. The majority of the remaining alignment (along the valley floor) is mapped by Delattre (2011) as being underlain by Holocene age alluvial fan deposits, comprised of gravel, sand, silt, and minor clay. The unit is further divided by relative age; the northwest-southeast-bearing contact between the sub-units is located in the vicinity of Pole 15, where Delattre (2011) indicates the deposits north of the contact are older than those to the south. The Mark West Creek channel is shown by Delattre (2011) to be underlain by Holocene stream channel deposits comprised of loose sand, silt and gravel.

In addition, Delattre (2011) identifies a landslide feature approximately 50 feet north of the northern endpoint of the alignment. The feature has been queried, indicating its existence is questionable. Landslide features are also identified by Delattre (2011) approximately 200 feet east and 300 feet northeast of this northern endpoint.

3.3 LOCAL AND REGIONAL FAULTING

The northern end of the alignment is located within the Hayward-Rodgers Creek Earthquake Fault Zone as defined by the California Geological Survey (CGS, 2018) in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972. According to the CGS (2018), the fault is located approximately 200 feet northeast of the alignment endpoint. The Hayward-Rodgers Creek fault is capable of producing a maximum earthquake magnitude event of M7.3. Moderate to major earthquakes generated on this fault, and others in the site vicinity can be expected to cause strong ground shaking at the site.

The proximities and seismic parameters of significant faults in the vicinity of the alignment are listed in Table 3.1. For faults with multiple segmentation scenarios we have only listed parameters for the scenario rupturing the most segments (i.e., the most severe scenario). The locations of the faults and associated parameters presented on Table 3.1 are based on Petersen et al. (2008). The maximum earthquake magnitudes presented in this table are based on the moment magnitude scale developed by Kanamori (1977). Felzer (2008) details calculations of California seismicity rates including correction for magnitude rounding and error, Gutenberg-Richter b value and seismicity rates.

TABLE 3.1
Significant Faults

Fault Name	Closest Distance to Site* (mi)	Magnitude of Characteristic Earthquake**	Slip Rate (millimeters/year)
Hayward-Rodgers Creek-SH+NH+RC	<0.1 (200 feet)	7.3	9
Maacama-Garberville	5.7	7.4	9
Collayomi	19.0	6.7	0.6
San Andreas-SAS+SAP+SAN+SAO	19.8	8.1	17-24
West Napa	22.3	6.7	1
Hunting Creek-Berryessa	25.5	7.1	6

* Closest distance to the potential rupture.
** *Moment magnitude*: An estimate of an earthquake's magnitude based on the seismic moment (measure of an earthquake's size utilizing rock rigidity, amount of slip, and area of rupture).

According to Petersen et al. (2008), characterizations of the Hayward-Rodgers Creek and the San Andreas faults are based on the following fault rupture segments and fault rupture scenarios:

- The Hayward-Rodgers Creek fault has been characterized by three segments and six rupture scenarios plus a floating earthquake. The three segments are the Rodgers Creek fault (RC), the Hayward North (HN), and the Hayward South (HS).
- The San Andreas fault has been characterized by four segments and nine rupture scenarios, plus a floating earthquake. The four segments are Santa Cruz Mountains (SAS), Peninsula (SAP), North Coast (SAN), and Offshore (SAO).

A number of large earthquakes have occurred within this region in the historic past. Some of the significant nearby events include two 1969 Santa Rosa earthquakes (M5.6, 5.7), the 2000 Yountville earthquake (M5.2), the 1869 Ukiah earthquake (M5.6), the 1906 San Francisco earthquake (M8+), and the 2014 South Napa earthquake (M6.0). Future seismic events in this region can be expected to produce strong seismic ground shaking along the project alignment. The intensity of future shaking will depend on the distance from the alignment to the earthquake focus, magnitude of the earthquake, and the response of the underlying soil and bedrock.

4. SITE CONDITIONS

4.1 SITE DESCRIPTION

The project vicinity is illustrated on Figures 1 and 2. The terrain through which the transmission line passes is generally flat to gently rolling, primarily alongside surface streets. The transmission line crosses mainly residential areas between Pole 8 and Pole 19 and mainly agricultural and undeveloped lands between Pole 19 and Pole 23. Surface vegetation along the alignment includes various crops, annual grasses, various shrubs and trees, and a forested Sonoma County Regional Park at the Pole 23 location.

4.2 SUBSURFACE CONDITIONS

The following description provides a general summary of the subsurface conditions encountered during this study. For more detailed descriptions of the actual conditions encountered at specific boring locations, refer to the boring logs provided in Appendix A.

4.2.1 Alignment Reaches Based on Encountered Subsurface Conditions

As stated in Section 3.2, the proposed TSP foundations are located within soil mapped as Holocene age alluvial fan deposits, with the exception of Pole 23, which is in an area mapped as Pre-Quaternary deposits or bedrock. Based on conditions encountered during our exploration, there appears to be a distinct transition with respect to geotechnical characteristics of the alluvial fan deposits somewhere in between Boring KB-2 (near Pole 12) and Boring KB-5 (near Pole 15). For geotechnical considerations and presentation of recommendations, the alignment has been divided into three reaches with similar subsurface conditions. Below is a summary of the three reaches, the TSPs that will be constructed, and the associated borings.

Table 4.1
Geotechnical Reaches and Associated TSPs and Boring

Reach	TSP	Relevant Borings
South	7_A/B	KB-1 through KB-2
	8	
	9	
	10	
	11	
	12	
	13	
Central	14	KB-5 and KB-3
	15	
	16	
	17	
	18	
	19	
	20	
	21	
	22	
North	23	KB-4

4.2.2 South Reach – Poles 7_A/B through Pole 13 (Borings KB-1 and KB-2)

Borings KB-1 and KB-2 were drilled to depths of approximately 43½ feet and 50 ½ feet, respectively. Medium stiff to hard lean clay and loose to medium dense clayey sand layers were encountered within the upper 35 to 30 feet of each boring. Below those depths, the density of the coarse-grained soils increased to dense to very dense, and the lean and fat clay encountered was a similar consistency as the upper fine-grained soils encountered in those borings. Boring KB-2 was drilled near Mark West Creek, which based on geologic maps consists of recent alluvial deposits within the creek channel. Based on our knowledge of the area, review of geologic and topography maps, we expect that subsurface conditions near Pole 13 will be similar to those encountered in Boring KB-2.

4.2.3 Central Reach – Pole 14 through Pole 22 (Borings KB-5 and KB-3)

Boring KB-5 was drilled near Pole 15 to a depth of approximately 61 ½ feet, and Boring KB-3, drilled near Pole 21, was drilled to a depth of approximately 61 feet below existing grade. In comparison to Borings KB-1 and KB-2, the Central Reach borings encountered predominantly very stiff to hard lean and fat clay with varying amounts of sand. Additionally, no sand layer was encountered within the upper 50 feet of Boring KB-5, and an approximate 2½-foot-thick very dense clayey sand layer was encountered within Boring KB-3 at approximately 21 feet deep. Very dense clayey sand was encountered near the bottom of each boring, below than 50 feet deep.

4.2.4 North Reach – Pole 23 (Boring KB-4)

This pole location is elevated from nearby Faught Road within the base of a hillside that is mapped as pre-quaternary deposits or bedrock (Glen Ellen Formation). Glen Ellen bedrock was encountered within Boring KB-4, is very weak, and can be described as a soil, which is how the bedrock was classified on the boring logs and within this section. Completely weathered bedrock was encountered at the surface to approximately 5 feet deep. Below 5 feet to the bottom of the boring, highly weathered bedrock was encountered. The upper five feet was classified as stiff to very stiff sandy lean clay. Below five feet, dense to very dense clayey sand was encountered to approximately 9½ feet. From 9½ feet to approximately 28½ feet hard lean clay and hard sandy fat clay was encountered. From approximately 28½ feet to the bottom of the boring at 56½ feet, very dense poorly graded sand with clay, and medium dense to very dense clayey sand was encountered.

4.3 GROUNDWATER

The borings were drilled using auger drilling methods until groundwater was encountered or until auger methods became impractical. After groundwater was encountered, the augered borings were completed using mud-rotary drilling methods, and the measured depth to water was recorded on the boring logs. Some of the borings were drilled using mud-rotary methods, which precluded groundwater measurements during drilling. Below is the groundwater level measured within each boring.

TABLE 4.2
Groundwater Measurements

Boring	Depth to Groundwater (feet)
KB-1	11½
KB-2	17½
KB-3	NE
KB-4	NE
KB-5	19

NE = Not encountered within upper 20 feet. Mud rotary drilling began at 20 feet.

A discussion of groundwater conditions along the project alignment is provided in Section 5.3.

4.4 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions along the alignment 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 RECOMENDATIONS

5.1 GENERAL

Based upon the results of our field exploration and laboratory testing programs, it is our opinion that the proposed tubular steel poles can be supported on reinforced concrete drilled pier foundations. Groundwater is expected to be encountered in the majority of the drilled shaft excavations and caving sandy soils may be encountered during construction of drilled pier foundations along most of the proposed alignment. Specific recommendations to reduce potential adverse effects of shallow groundwater, as well as general recommendations regarding the geotechnical aspects of project design and construction, are presented below.

5.2 SEISMIC DESIGN CRITERIA

Seismic design information based upon the 2016 CBC, which utilizes the ASCE 7-10, is presented in Table 5.1. The Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1), mapped peak ground acceleration (PGA), and mapped long-period transition period (T_L) were estimated based on Section 1613 of the CBC and Chapter 22 of the ASCE 7-10 using the United States Geological Survey (USGS) U.S. seismic design maps. The mapped acceleration values, associated soil amplification factors (F_a and F_v), and corresponding site modified (S_{MS} and S_{M1}) and design spectral accelerations (S_{DS} and S_{D1}), based on CBC, are presented in Tables 5.1 and 5.2. Considering the soil and rock conditions encountered at the site, and after a review of geologic publications, we recommend Site Class D for the South and Central Reaches and a Site Class C for the North Reach for this project. The Seismic Design Category is estimated to be E for all reaches.

To provide the ground motion parameters associated with the 2016 CBC, an online tool (https://earthquake.usgs.gov/designmaps/us/application.php?) was used, which was developed by the USGS based on the Seismic Design Maps in the 2015 IBC. Estimated values of PGA are based on mapped values of Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Accelerations (Figure 22-7, ASCE 7-10). The resulting 2016 CBC seismic design factors (for a risk factor of I, II, or III) are presented below in Tables 5.1 and 5.2.

Table 5.1: Ground Motion Parameters Based on 2016 CBC – South and Central Reach

Parameter	Value	Reference
S_S	2.429g	2016 CBC Section 1613.3.1
S_1	1.009g	2016 CBC Section 1613.3.1
Site Class	D	2016 CBC Section 1613.3.2
Seismic Design Category	E	2016 CBC Tables 1613.3.5 (1) and (2)
F_a	1.0	2016 CBC Table 1613.3.3(1)
F_v	1.5	2016 CBC Table 1613.3.3(2)
S_{MS}	2.429g	2016 CBC Section 1613.3.3
S_{M1}	1.514g	2016 CBC Section 1613.3.3
S_{DS}	1.619g	2016 CBC Section 1613.4.4
S_{D1}	1.009g	2016 CBC Section 1613.4.4
PGA	0.937g	ASCE 7-10 Figure 22-7
F_{PGA}	1.000	ASCE 7-10 Table 11.8-1
PGA_M	0.937g	ASCE 7-10 Section 11.8.3
C_{RS}	0.942	ASCE 7-10 Figure 22-17
C_{R1}	0.923	ASCE 7-10 Figure 22-18
T_L	8 seconds	ASCE 7-10 Figure 22-12

Table 5.2: Ground Motion Parameters Based on 2016 CBC – North Reach

Parameter	Value	Reference
S_S	2.442g	2016 CBC Section 1613.3.1
S_1	1.014g	2016 CBC Section 1613.3.1
Site Class	C	2016 CBC Section 1613.3.2
Seismic Design Category	E	2016 CBC Tables 1613.3.5 (1) and (2)
F_a	1.0	2016 CBC Table 1613.3.3(1)
F_v	1.3	2016 CBC Table 1613.3.3(2)
S_{MS}	2.442g	2016 CBC Section 1613.3.3
S_{M1}	1.318g	2016 CBC Section 1613.3.3
S_{DS}	1.628g	2016 CBC Section 1613.4.4
S_{D1}	0.879g	2016 CBC Section 1613.4.4
PGA	0.943g	ASCE 7-10 Figure 22-7
F_{PGA}	1.000	ASCE 7-10 Table 11.8-1
PGA_M	0.943g	ASCE 7-10 Section 11.8.3

Parameter	Value	Reference
C_{RS}	0.942	ASCE 7-10 Figure 22-17
C_{R1}	0.922	ASCE 7-10 Figure 22-18
T_L	8 seconds	ASCE 7-10 Figure 22-12

5.3 DESIGN GROUNDWATER CONDITIONS

Recommended design groundwater conditions are based on the findings from the exploratory borings drilled for this study, and a review of available California Department of Water Resources data. Table 5.3 presents recommended design groundwater levels for use in pole foundation design and construction planning.

Table 5.3
Recommended High Groundwater Levels for Design

Reach	Depth Below Ground Surface (feet)
South	10
Central	10
North	25

Actual groundwater levels at any given location will vary with seasonal variations in rainfall and runoff, adjacent canal or river stage, irrigation practices, and other factors not apparent at the time of our field investigation. A site-specific hydrogeologic evaluation for this project to evaluate specific seasonal fluctuations is beyond the scope of this study.

5.4 SOIL LIQUEFACTION

5.4.1 General

Soil liquefaction is a condition in which saturated, granular and low-plasticity cohesive soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stresses induced by earthquakes. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded and fine-grained sand deposits. Based on recent observations and study, under certain conditions "liquefaction," or cyclic strain softening, can occur in low-plasticity silts and clays (Seed et al., 2003; Bray and Sancio, 2006; Boulanger and Idriss, 2006). If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo excessive settlements, lateral deformations and additional structural loads due to down drag.

5.4.2 Susceptibility Assessment

Liquefaction susceptibility of the soils encountered within Borings KB-1 through KB-5 were evaluated using methodologies proposed by Youd et al. (2001), Seed et al (2003), Idriss & Boulanger (2008), Tokimatsu and Seed (1987), and Cetin et al. (2009). Below is an assessment of the liquefaction susceptibility of soils within each of the three reaches for this project.

5.4.2.1 *South Reach – Borings KB-1 and KB-2*

Prior to laboratory testing, some of the clayey sand layers within Borings KB-1 and KB-2 were identified as potentially liquefiable. Atterberg limits testing, and percent passing the No. 200 sieve testing was performed on those suspect soils. The results of that testing program suggest that the suspect layers have a low liquefaction potential based on Liquid Limits ranging from 31 to 33, Plasticity Indexes ranging from 9 to 16, and percent passing the No. 200 sieve results ranging from 40 to 49 percent. Laboratory testing to check for liquefaction potential was not completed on samples that were observed to have a tight clay matrix because based on visual inspection, the soil had a low liquefaction potential. Based on our review of the laboratory test results and our visual classifications, we consider the potential for liquefaction along the South Reach to be low.

5.4.2.2 *Central Reach – Borings KB-3 and KB-5*

Based on the apparent density of granular soils in Borings KB-3 and KB-5 the plasticity characteristics of fine-grained soils in these borings, we consider the liquefaction potential along the Central Reach to be low.

5.4.2.3 *North Reach – Boring KB-4*

At Boring KB-4, which represents the North Reach, the shallow Glen Ellen bedrock is considered to have a low potential for liquefaction.

5.5 EXPANSIVE SOIL

Based on a review of maps published by the U.S. Department of Agriculture and Natural Conservation Resource Service, expansive soils with a high shrink/swell potential exist near poles 16, 17 and 18. This soil unit is identified as Clearlake Clay (CeA). The near surface soil encountered in in the borings drilled for this study consist of lean clay with varying amounts of sand with a low to moderate expansion potential. However, it is possible that near surface fat clay with a high shrink/swell and expansion potential may be encountered at some pole locations throughout the alignment, with an increased likelihood near poles 16 through 18. Highly expansive clay soils have the potential to undergo volume change due to seasonal changes in moisture content. Below are conditions that apply to TSP foundation design in soils susceptible to potential for swell and shrinkage of the near surface soils.

Swell Potential

Heaving or swelling of near-surface soils can occur if the water content of the expansive soil increases following an extended period of dry weather. This swelling can induce an upward drag force on a shaft or pile foundation. In our opinion, any drag force effect due to swelling soils will be relatively small for the anticipated depths of the TSP foundations.

Shrinkage Potential

During periods of extended dry weather, near-surface expansive soils can become desiccated and shrink as the water content drops seasonally. This soil shrinkage around drilled pier or pile foundations can reduce the contact between the foundations and the surrounding soil. This can result in a reduction of axial and lateral capacity within the upper 2 to 3 feet of a shaft or pile foundation. This potential soil shrinkage has been taken into consideration during the development of foundation recommendations discussed in Section 5.6.

5.6 DRILLED SHAFT FOUNDATIONS

Based on conversations with PG&E, we understand that the minimum diameter for the TSP drilled piers will be 6-feet. Below is a summary of each planned TSP replacement and the maximum lateral unfactored loading conditions provided by PG&E.

Table 5.4
TSP Pole Type and Loading Conditions

<table>
<tr><th>Reach</th><th>TSP</th><th>TSP Pole Type</th><th>Unfactored Governing Lateral Loading Conditions[1]</th><th>Relevant Borings</th></tr>
<tr><td rowspan="7">South</td><td>7_A/B</td><td>Angle</td><td>V = 40.06 kips, M =3,999 ft-kips, A = 45.25 kips</td><td rowspan="7">KB-1 through KB-2</td></tr>
<tr><td>8</td><td>Angle</td><td>V = 50.01 kips, M =4,734 ft-kips, A = 44.24 kips</td></tr>
<tr><td>9</td><td rowspan="2">Tangent</td><td rowspan="2">V = 28.33 kips, M = 2,938 ft-kips, A =45.43 kips</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td><td>Running Angle</td><td>V = 28.37 kips, M = 2,864 ft-kips, A =45.68 kips</td></tr>
<tr><td>12</td><td>Tangent</td><td>V = 28.33 kips, M = 2,938 ft-kips, A =45.43 kips</td></tr>
<tr><td>13</td><td>Angle</td><td>V = 30.27 kips, M = 3,120 ft-kips, A = 39.28 kips</td></tr>
<tr><td rowspan="9">Central</td><td>14</td><td>Running Angle</td><td>V = 28.37 kips, M = 2,864 ft-kips, A =45.68 kips</td><td rowspan="9">KB-3 and KB-5</td></tr>
<tr><td>15</td><td rowspan="6">Tangent</td><td rowspan="6">V = 28.33 kips, M = 2,938 ft-kips, A =45.43 kips</td></tr>
<tr><td>16</td></tr>
<tr><td>17</td></tr>
<tr><td>18</td></tr>
<tr><td>19</td></tr>
<tr><td>20</td></tr>
<tr><td>21</td><td>Angle</td><td>V = 57.96 kips, M = 5,475 ft-kips, A = 44.26 kips</td></tr>
<tr><td>22</td><td>Running Angle</td><td>V = 28.37 kips, M = 2,864 ft-kips, A =45.68 kips</td></tr>
<tr><td>North</td><td>23</td><td>Angle</td><td>V = 30.45 kips, M = 2,621 ft-kips, A = 47.28 kips</td><td>KB-4</td></tr>
</table>

[1] V = Shear reaction at pier head, M = Moment reaction at pier head, A = Downward axial loading

5.6.1 Axial Capacity

Axial loads imposed by the poles should be supported by the frictional capacity of the drilled pier foundation. End bearing was not considered in the axial capacity due to the potential for loose materials to exist at the bottoms of the pier holes during construction that cannot be effectively cleaned out. If axial capacity becomes a governing load condition for pier design, we should be consulted to provide additional design and construction recommendations to allow for inclusion of a portion of end bearing capacity.

Two curves illustrating the ultimate axial compressive capacity of a unit (1-foot) diameter straight-sided drilled pier installed from the existing ground surface are shown on Figures 7 (South Reach) and 8 (Central and North Reach).

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). The weight of the foundation is not included in the ultimate resistance shown on Figures 7 and 8.

Axial capacity was computed using Federal Highway Administration (FHWA) procedures for design of drilled pier foundations (Brown et al., 2010). 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 (GO 95) code. The ultimate uplift capacity may be estimated as 80 percent of the ultimate compressive axial capacity as indicated on Figures 7 and 8. A one-third increase in the allowable capacity may be used for consideration of transient loads such as wind or seismic.

5.6.2 Estimated Settlement

Based on the methods outlined by Brown et al. (2010), we expect total static settlement of each drilled pier to be on the order of 0.2 percent of the pier diameter for a drilled pier designed and constructed in accordance with the recommendations presented in this report. We expect most of the settlement to occur during and shortly after application of the structure loads.

5.6.3 Lateral Response

Lateral response of the piers normally controls the design length of drilled piers for transmission line poles. We understand current PG&E design criteria for transmission line foundations will be used to determine required drilled pier foundation lengths. Resistance to lateral loads will be provided by passive resistance of the soil against the pier foundations and by the bending stiffness of the piers. PG&E provided loading conditions for each angle pole, the running angle poles, and tangent poles. Tables 5.5 through 5.10 contain recommended input soil parameters for each angle pole, and the South and Central tangent and running angle poles for lateral analysis of drilled pier foundations using the LPILE computer program (by Ensoft, Inc., Version 2018).

Table 5.5
Recommended LPILE Geotechnical Parameters
Poles 7A, 7B, and 8
(Profile Based on Boring KB-1)

Depth (feet)	P-Y Curve Soil Model	$\gamma_{effective}$ (pcf)	C (psf)	ϕ (degree)	k (pci)	ε_{50}
0 to 2	Soft Clay (Matlock)	130	200	-	-	*
2 – 10	Stiff Clay w/o Free Water (Reese)	130	1,300	-	-	*
10 – 13.5	Sand (Reese)	53	-	32	*	-
13.5 – 18.5	Stiff Clay w/o Free Water (Reese)	63	1,300	-	-	*
18.5 – 23	Sand (Reese)	53	-	32	*	-
23 – 28	Stiff Clay w/o Free Water (Reese)	48	600	-	-	*
28 – 36	Sand (Reese)	60	-	33	*	-
36 – 55	Sand (Reese)	62	-	38	*	-

* = Use software default value

Table 5.6
Recommended LPILE Geotechnical Parameters
Pole 13

(Profile Based on Boring KB-2)

Depth (feet)	P-Y Curve Soil Model	$\gamma_{effective}$ (pcf)	C (psf)	ϕ (degree)	k (pci)	ε_{50}
0 to 2	Soft Clay (Matlock)	105	200	-	-	*
2 – 7	Stiff Clay w/o Free Water (Reese)	105	2,000	-	-	*
7 – 10	Sand (Reese)	115	-	32	*	-
10 – 16.5	Stiff Clay w/o Free Water (Reese)	53	-	32	*	-
16.5 – 33.5	Sand (Reese)	63	-	37	*	-
33.5 – 44	Stiff Clay w/o Free Water (Reese)	65	3,000	-	-	*
44 – 50	Sand (Reese)	63	-	38	*	-

* = Use software default value

Table 5.7
Recommended LPILE Geotechnical Parameters
Pole 21

(Profile Based on Boring KB-3)

Depth (feet)	P-Y Curve Soil Model	$\gamma_{effective}$ (pcf)	C (psf)	ϕ (degree)	k (pci)	ε_{50}
0 to 2	Soft Clay (Matlock)	130	200	-	-	*
2 – 10	Stiff Clay w/o Free Water (Reese)	130	3,000	-	-	*
10 – 33	Stiff Clay w/o Free Water (Reese)	68	3,000	-	-	*
33 – 51	Sand (Reese)	68	3,500	-	-	*

* = Use software default value

Table 5.8
Recommended LPILE Geotechnical Parameters
Pole 23

(Profile Based on Boring KB-4)

Depth (feet)	P-Y Curve Soil Model	$\gamma_{effective}$ (pcf)	C (psf)	ϕ (degree)	k (pci)	ε_{50}
0 to 2	Soft Clay (Matlock)	96	200	-	-	*
2 – 5	Stiff Clay w/o Free Water (Reese)	96	3,000	-	-	*
5 – 10	Stiff Clay w/o Free Water (Reese)	96	4,000	-	-	*
10 – 18.5	Stiff Clay w/o Free Water (Reese)	103	4,000	-	-	*
18.5 – 25	Stiff Clay w/o Free Water (Reese)	115	4,000	-	-	*
25 – 33	Stiff Clay w/o Free Water (Reese)	55	4,000	-	-	*
33 – 48	Sand (Reese)	55	-	40	*	-
48 – 56	Sand (Reese)	55	-	38	*	-

* = Use software default value

Table 5.9
Recommended LPILE Geotechnical Parameters
Tangent and Running Angle Poles, South Reach (Boring KB-1)

Depth (feet)	P-Y Curve Soil Model	$\gamma_{effective}$ (pcf)	C (psf)	ϕ (degree)	k (pci)	ε_{50}
0 to 2	Soft Clay (Matlock)	130	200	-	-	*
2 – 10	Stiff Clay w/o Free Water (Reese)	130	1,300	-	-	*
10 – 13.5	Sand (Reese)	53	-	32	*	-
13.5 – 18.5	Stiff Clay w/o Free Water (Reese)	63	1,300	-	-	*
18.5 – 23	Sand (Reese)	53	-	32	*	-
23 – 28	Stiff Clay w/o Free Water (Reese)	48	600	-	-	*
28 – 36	Sand (Reese)	60	-	33	*	-
36 – 55	Sand (Reese)	62	-	38	*	-

* = Use software default value

Table 5.10
Recommended LPILE Geotechnical Parameters
Tangent and Running Angle Poles, Central Reach (Boring KB-5)

Depth (feet)	P-Y Curve Soil Model	$\gamma_{effective}$ (pcf)	C (psf)	ϕ (degree)	k (pci)	ε_{50}
0 to 2	Soft Clay (Matlock)	125	200	-	-	*
2 – 7	Stiff Clay w/o Free Water (Reese)	125	2,000	-	-	*
7 – 10	Stiff Clay w/o Free Water (Reese)	125	1,300	-	-	*
10 – 14	Stiff Clay w/o Free Water (Reese)	63	1,300	-	-	*
14 – 18	Stiff Clay w/o Free Water (Reese)	65	2,000	-	-	*
18 – 23	Stiff Clay w/o Free Water (Reese)	63	1,300	-	-	*
23 – 34	Stiff Clay w/o Free Water (Reese)	70	3,400	-	-	*
34 – 40	Stiff Clay w/o Free Water (Reese)	63	1,500	-	-	*
40 – 61	Stiff Clay w/o Free Water (Reese)	70	3,400	-	-	*

* = Use software default value

Per PG&E design standards, the total pier top rotation under the applied loads should be within 1/2 degree of vertical, and the pier head deflection should be less than 2 percent of the pier diameter.

Using the soil parameters described above and load information provided by the designer, Kleinfelder performed lateral response analyses for several cases of drilled pier foundations for different soil profile cases to verify adequate drilled pier penetration to meet current PG&E pier head deflection and rotation criteria of 2 percent of the pier diameter and ½ degree, respectively. The results of these analyses are presented on Figures 9 through 15.

5.7 CONSTRUCTION CONSIDERATIONS – DRILLED PIER FOUNDATIONS

5.7.1 General

Successful completion of drilled pier foundations requires careful 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. Drilled excavations should not remain open overnight. For this project, potentially caving soil conditions exist in some areas along the alignment. The following considerations should be implemented during construction of drilled shaft foundations.

5.7.2 Caving/Water Intrusion

In most areas of the alignment, groundwater levels could be high enough to cause caving and/or water intrusion into drilled shaft excavations, especially where cohesionless soils are present. We recommend that the contractor be prepared to deal with shallow groundwater and potentially caving conditions during construction.

5.7.3 Temporary Casing

If temporary, straight-sided steel casing is used, we recommend its removal 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. Casing should not be left in the ground except by permission of the project geotechnical and structural engineers. Corrugated metal pipe (CMP) casing should not be used under any circumstances.

5.7.4 Bottom Preparation

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 pier excavation to verify soil and excavation conditions prior to placing steel reinforcement or concrete.

5.7.5 Steel and Concrete Placement

It is recommended that steel reinforcement and concrete be placed on the same day of completion of each drilled shaft excavation 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 (slurry) 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.

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 thickness of the contaminated concrete seal layer will depend on the shaft diameter and construction method, but it can approach the shaft diameter.

It is recommended that concrete used for tremie construction have a slump of 6 to 8 inches. The concrete mix should be designed with an appropriate water/cement ratio for the design strength and use water reducing/plasticizing admixtures to achieve the recommended slump. Adding water to a conventional mix to achieve the recommended slump should not be allowed. Vibration of concrete under water during placement is generally not recommended as it may result in contamination of the concrete or cause aggregate settlement within the shaft. A relatively fluid and properly designed concrete mix helps to avoid segregation, rock pockets, and poor adherence of the concrete to the reinforcing steel. Careful vibration of the tops of the shafts following removal of the seal layer is recommended to consolidate the concrete around anchor bolt assemblies.

5.8 SOIL CORROSION

Two composite specimens of multiple near-surface samples encountered within Borings KB-1 through KB-5 were subjected to chemical analysis for the purpose of corrosion assessment. Cerco Analytical of Concord, California performed the tests under subcontract to Kleinfelder. The test results are presented in Appendix C and below in Table 5.11, Summary of Corrosion Test Results.

Table 5.11
Summary of Corrosion Test Results

Boring No.	Depth (ft.)	pH	Minimum Resistivity, As Received (ohms-cm)	Minimum Resistivity, 100% Saturated (ohms-cm)	Water Soluble Chlorides (ppm)	Water Soluble Sulfates (mg/kg)
KB-1	5.5	6.75	790	1,100	ND	26
KB-1	15					
KB-2	5.5					
KB-3	5.5	7.17	2,400	980	36	48
KB-3	16					
KB-4	10.5					
KB-4	15.5					
KB-5	5.5					
KB-5	8					

The reported resistivity results in a saturated condition indicate that the soil tested is considered to be highly to extremely corrosive to buried, unprotected metal objects (Roberge, 2006).

According to ACI 318, a water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete. Sulfate concentrations less than 0.10 percent by mass of soil (1000 parts per million [ppm]) is considered non-applicable. According to ACI, the minimum compressive strength (f'c) for concrete should be 2,500 psi with no maximum water cement ratio.

The above corrosivity results are an indicator of potential soil corrosivity for the sample tested. Other soils found on the site may be more, less, or of a similar corrosive nature. 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.

6. LIMITATIONS

This report presents information for planning, permitting, design, and construction of the Fulton-Fitch TSP Replacement Project in Sonoma County, California. This report should not be used to define site conditions for contractual purposes, and Kleinfelder will accept no liability for changed conditions claims based on this report.

Recommendations contained in this report are based on conditions encountered in our exploratory borings, evaluation of existing geotechnical data, 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 between or beyond the points explored. If the scope of the proposed construction, including the proposed alignment location, changes from that described in this report, we should be notified immediately to review the information and possibly provide supplemental recommendations.

This report has been prepared in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty is expressed or implied.

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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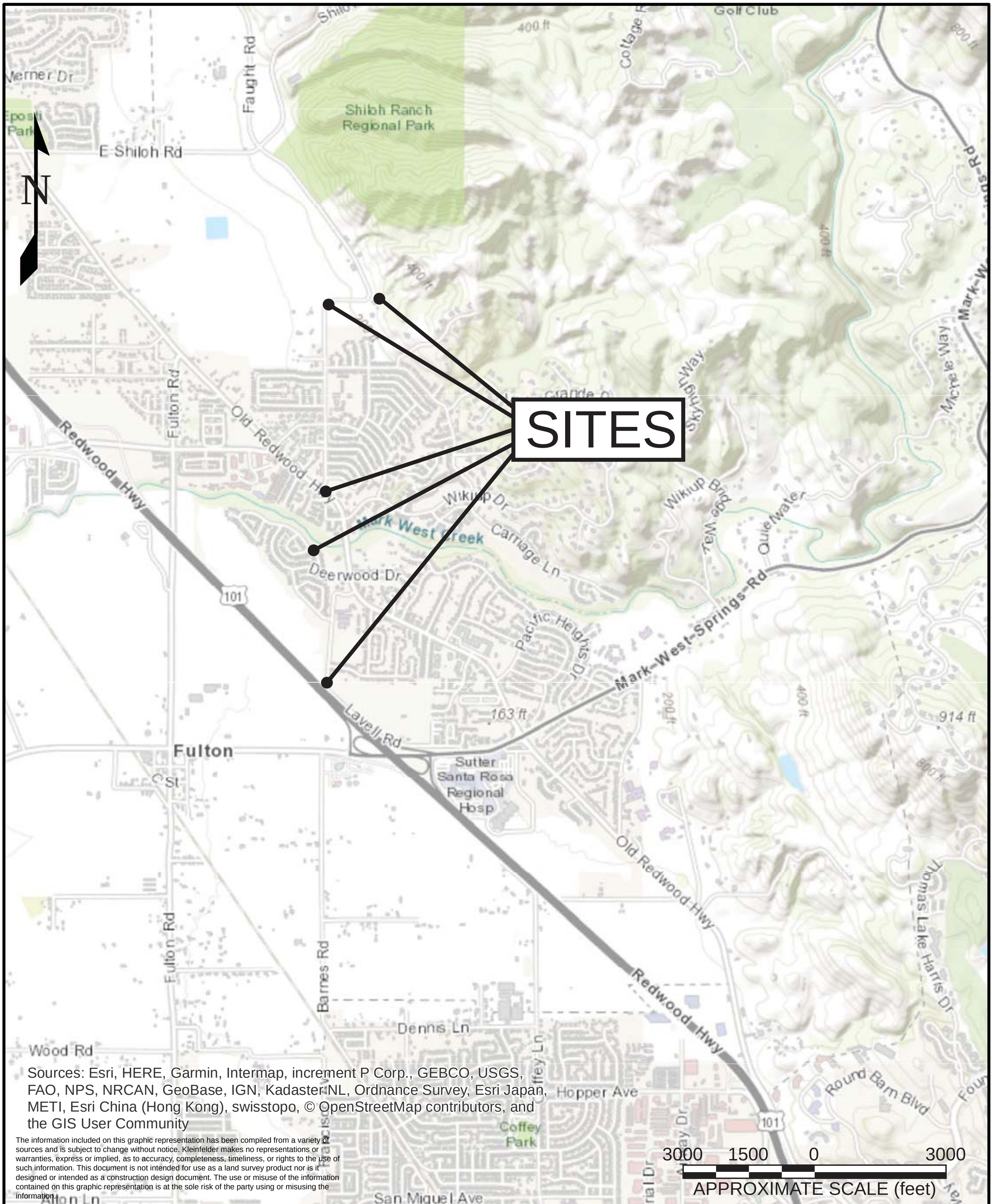
Figures



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

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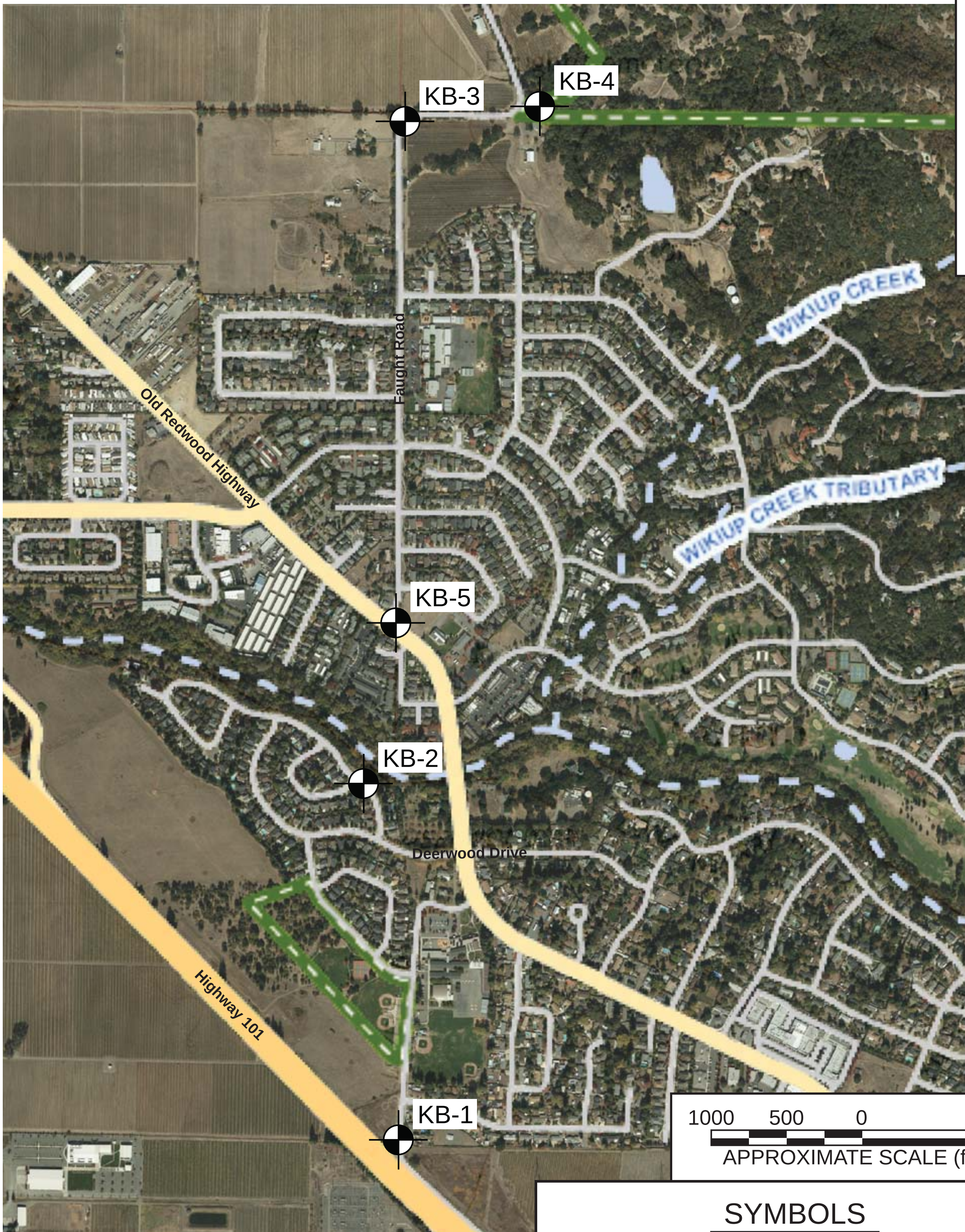
FILE NAME	Figure 1 Site Location.ai

SITE LOCATION

FULTON FITCH TSP REPLACEMENT
GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

FIGURE

1



N

1000	500	0	1000

APPROXIMATE SCALE (feet)

SYMBOLS

KB-1 Approximate Boring Location (Kleinfelder, 2018)

Source: City of Santa Rosa Public GIS Viewer

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SITE PLAN

PG&E FULTON FITCH TSP REPLACEMENT
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FIGURE 2

150 75 0 150

APPROXIMATE SCALE (feet)

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SITE PLAN

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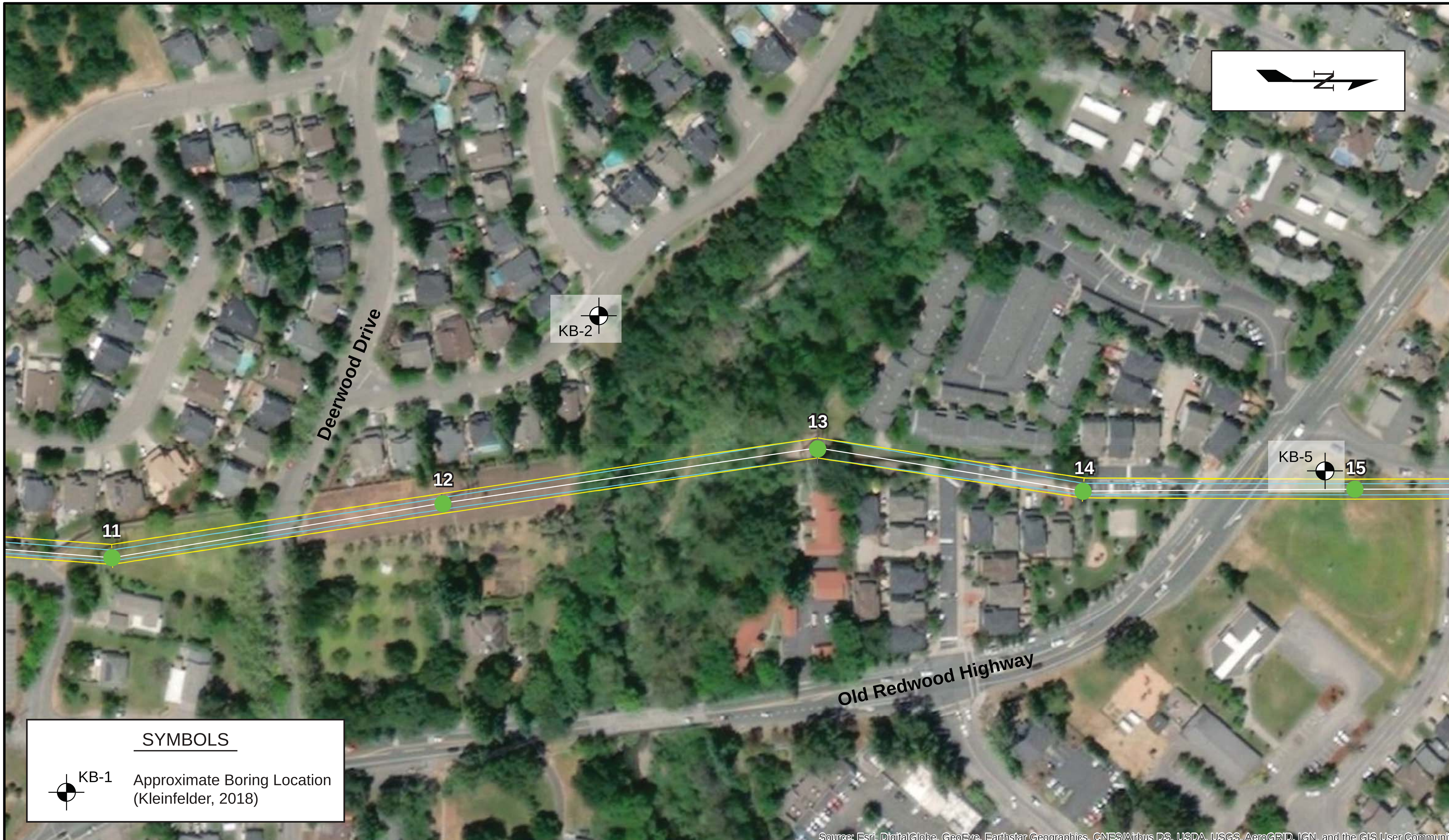
GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

FIGURE

3



150 75 0 150
APPROXIMATE SCALE (feet)

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SITE PLAN

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GEOTECHNICAL INVESTIGATION
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FIGURE
4

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150 75 0 150

APPROXIMATE SCALE (feet)

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SITE PLAN

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GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

FIGURE

5

150 75 0 150
APPROXIMATE SCALE (feet)

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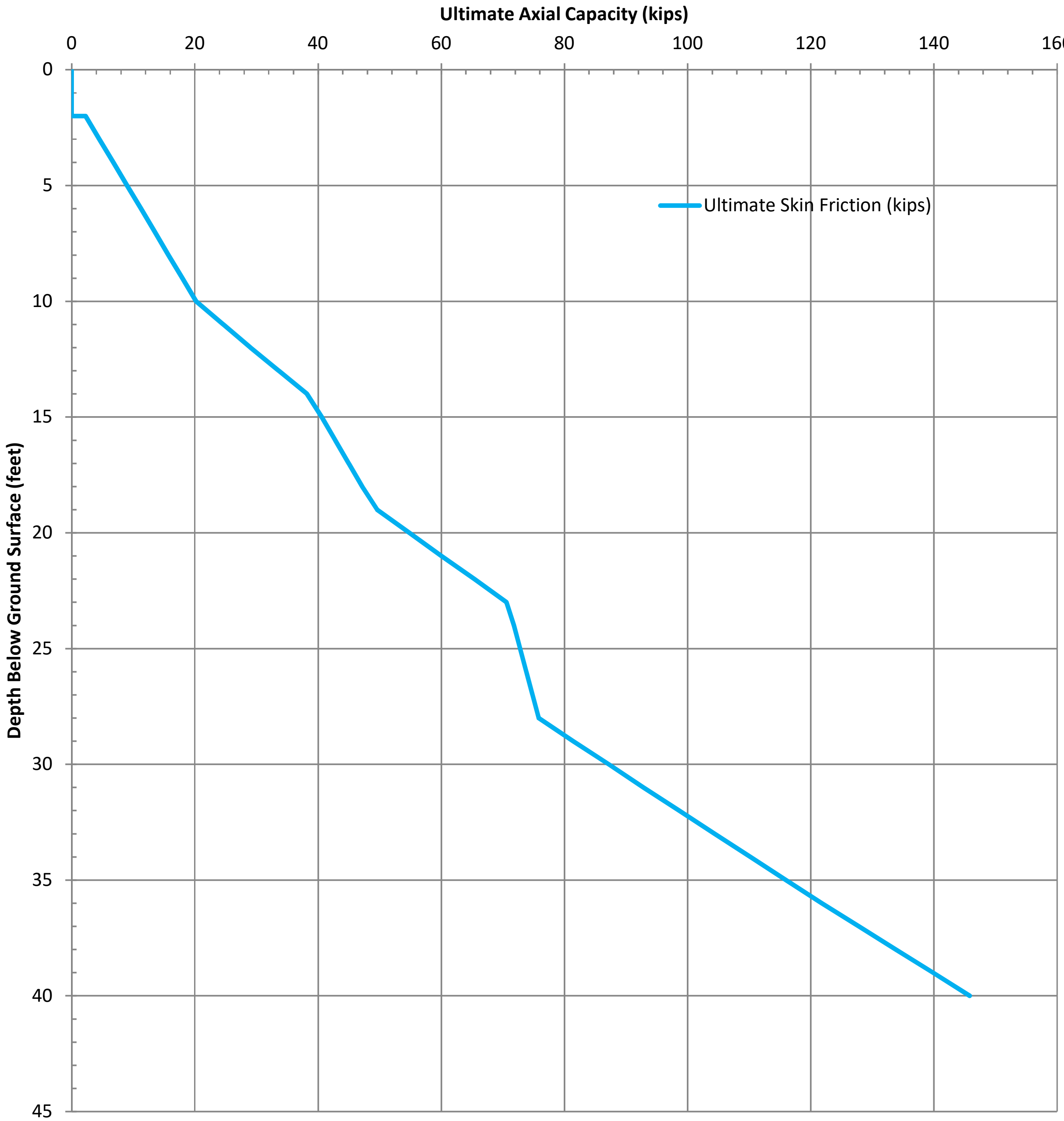
PROJECT NO. 20190527	SITE PLAN		FIGURE
DRAWN: AUG 2018			6
DRAWN BY: SDC			
CHECKED BY: MJP	PG&E FULTON FITCH TSP REPLACEMENT GEOTECHNICAL INVESTIGATION SANTA ROSA, CALIFORNIA		
FILE NAME: Figure 2 Site Plan.ai			



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 pile (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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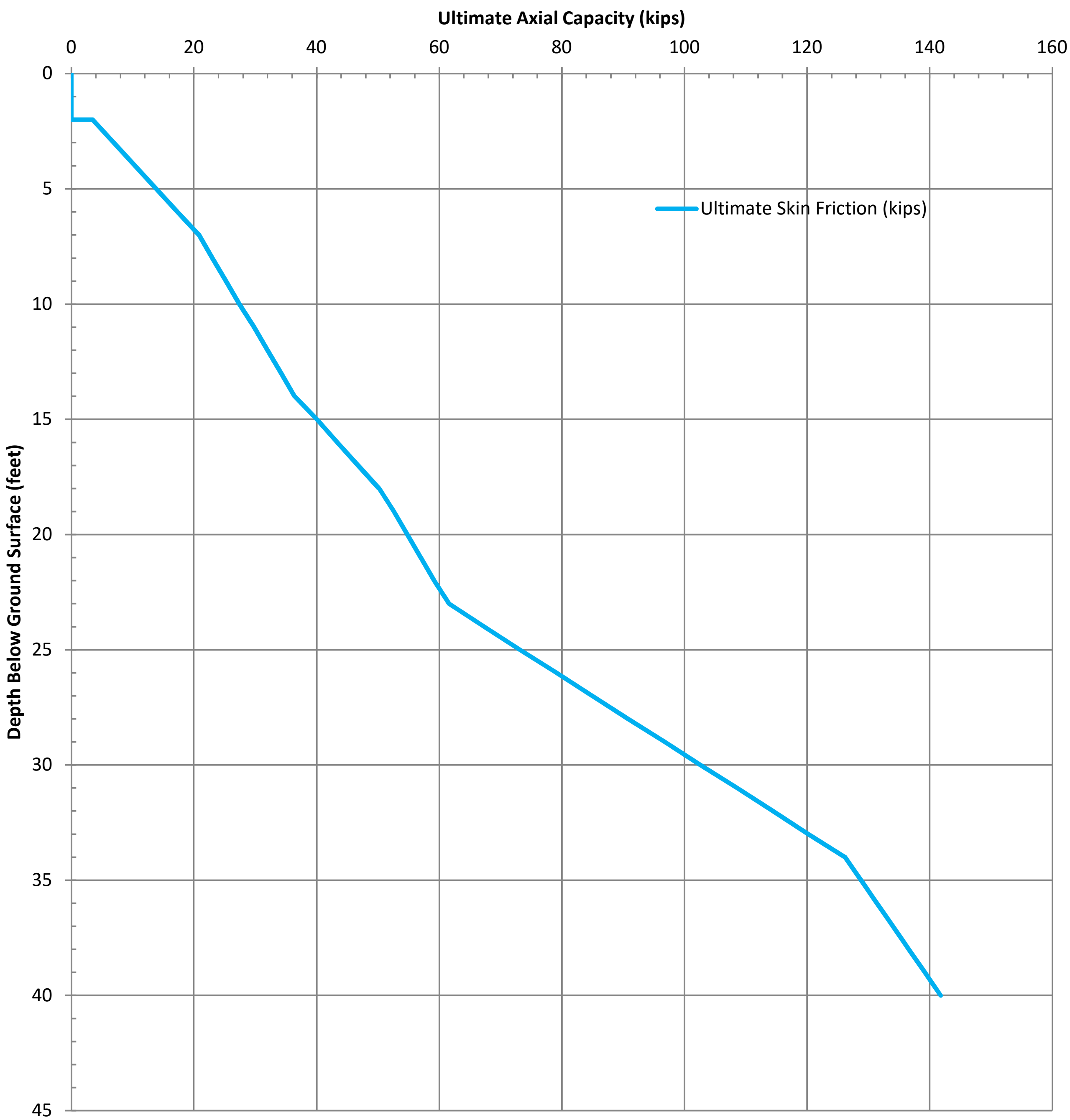
PROJECT NO.: 20190527.001A DRAWN BY: JA CHECKED BY: MP DATE: 9/5/2018 REVISED:	SOUTH REACH POLE 7_A/B THROUGH POLE 13 ULTIMATE AXIAL CAPACITY PLOT UNIT DIAMETER (1-FOOT) DRILLED PIER FULTON FITCH TSP REPLACEMENT GEOTECHNICAL INVESTIGATION SANTA ROSA, CALIFORNIA	FIGURE 7



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 pile (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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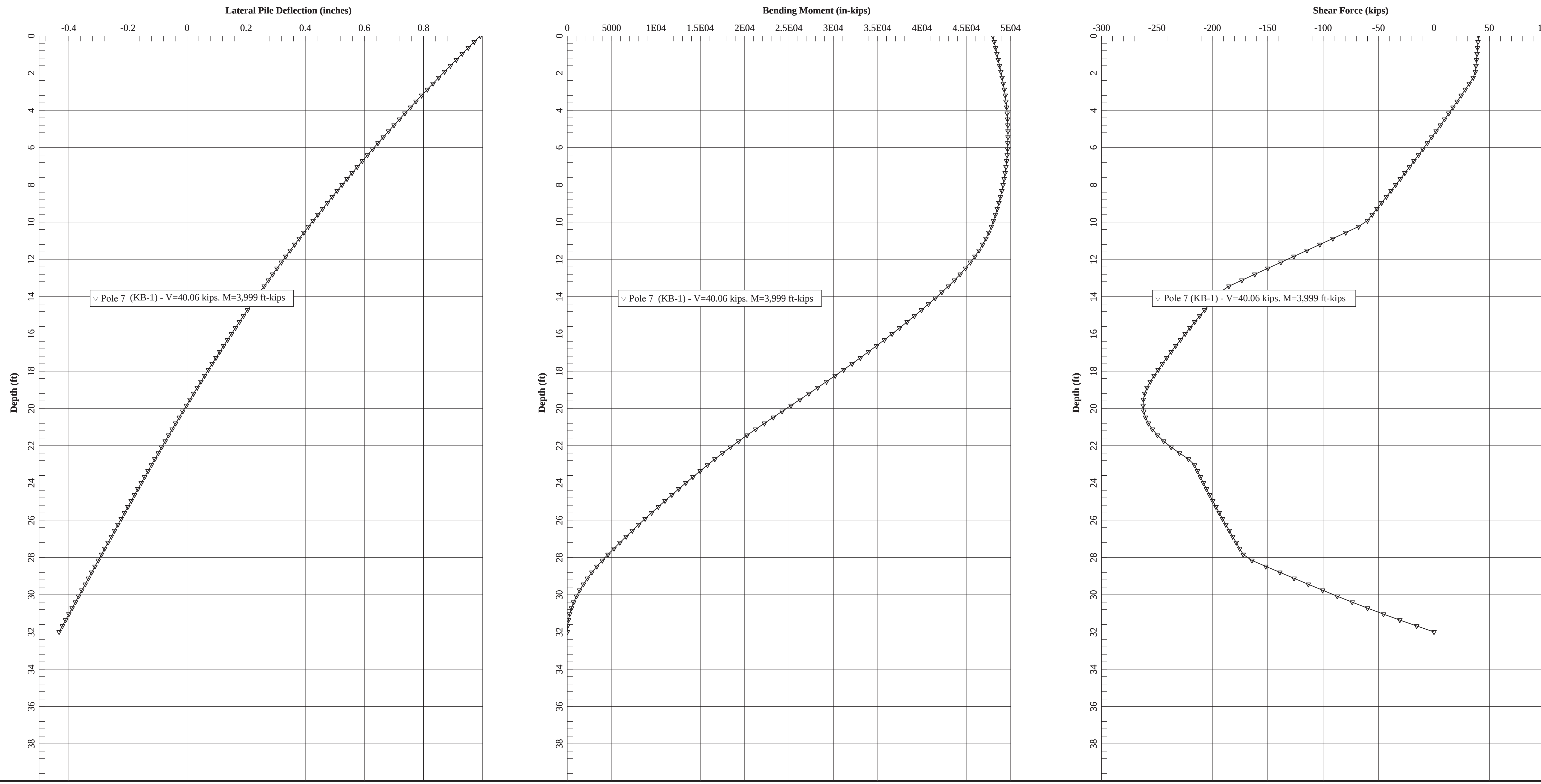
PROJECT NO.: 20190527.001A		CENTRAL AND NORTH REACH POLE 14 THROUGH POLE 23 ULTIMATE AXIAL CAPACITY PLOT UNIT DIAMETER (1-FOOT) DRILLED PIER	FIGURE
DRAWN BY: JA			8
CHECKED BY: MP		FULTON FITCH TSP REPLACEMENT GEOTECHNICAL INVESTIGATION SANTA ROSA, CALIFORNIA	
DATE: 9/5/2018			
REVISED:			



Santa Rosa

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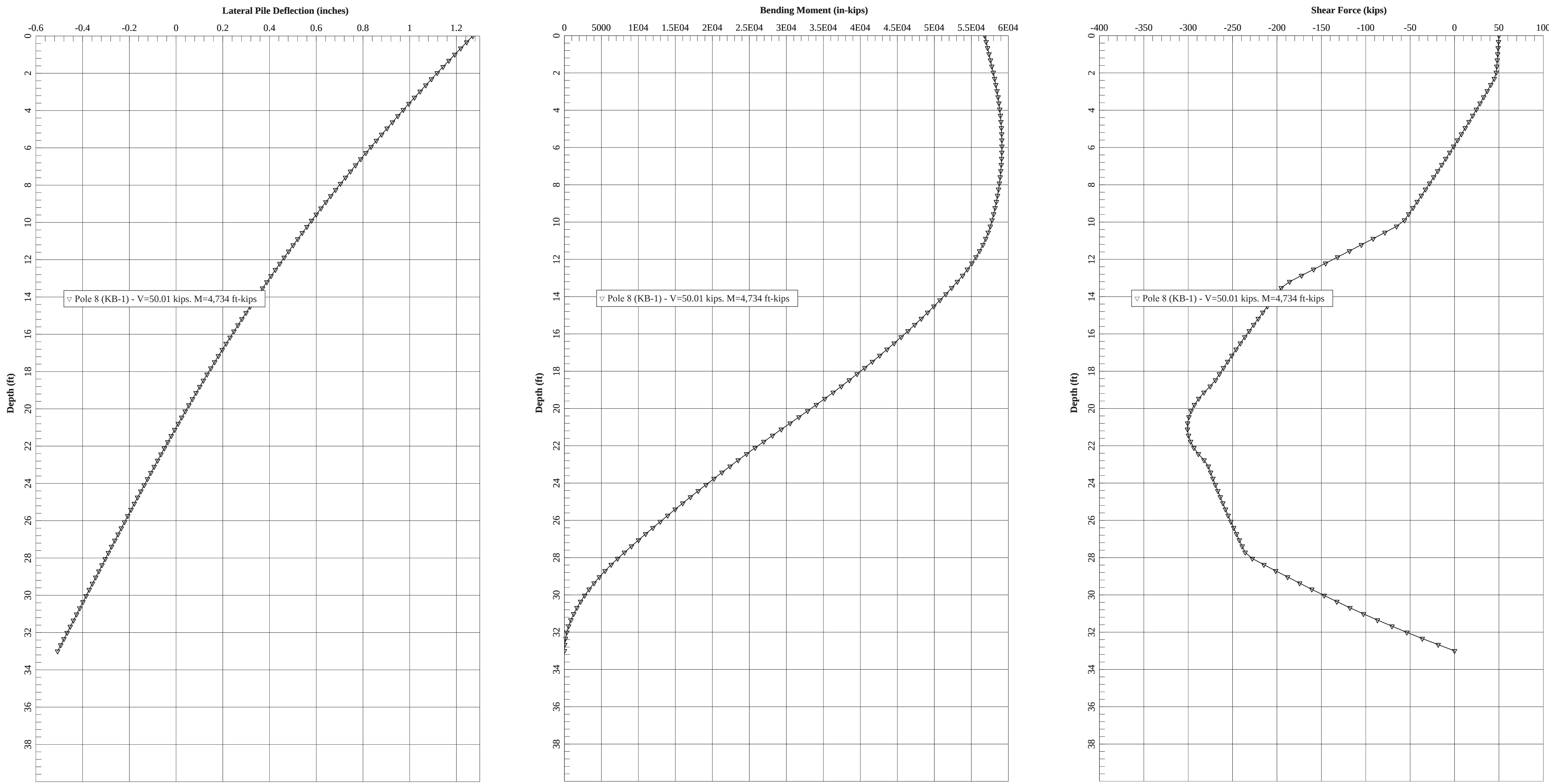
PROJECT NO.	20190527.001A
DRAWN	AUG 2018
DRAWN BY	SDC
CHECKED BY	MJP
FILE NAME	Figure 4 .ai

POLE 7
LATERAL PILE RESPONSE
72-INCH DIAMETER DRILLED PIER

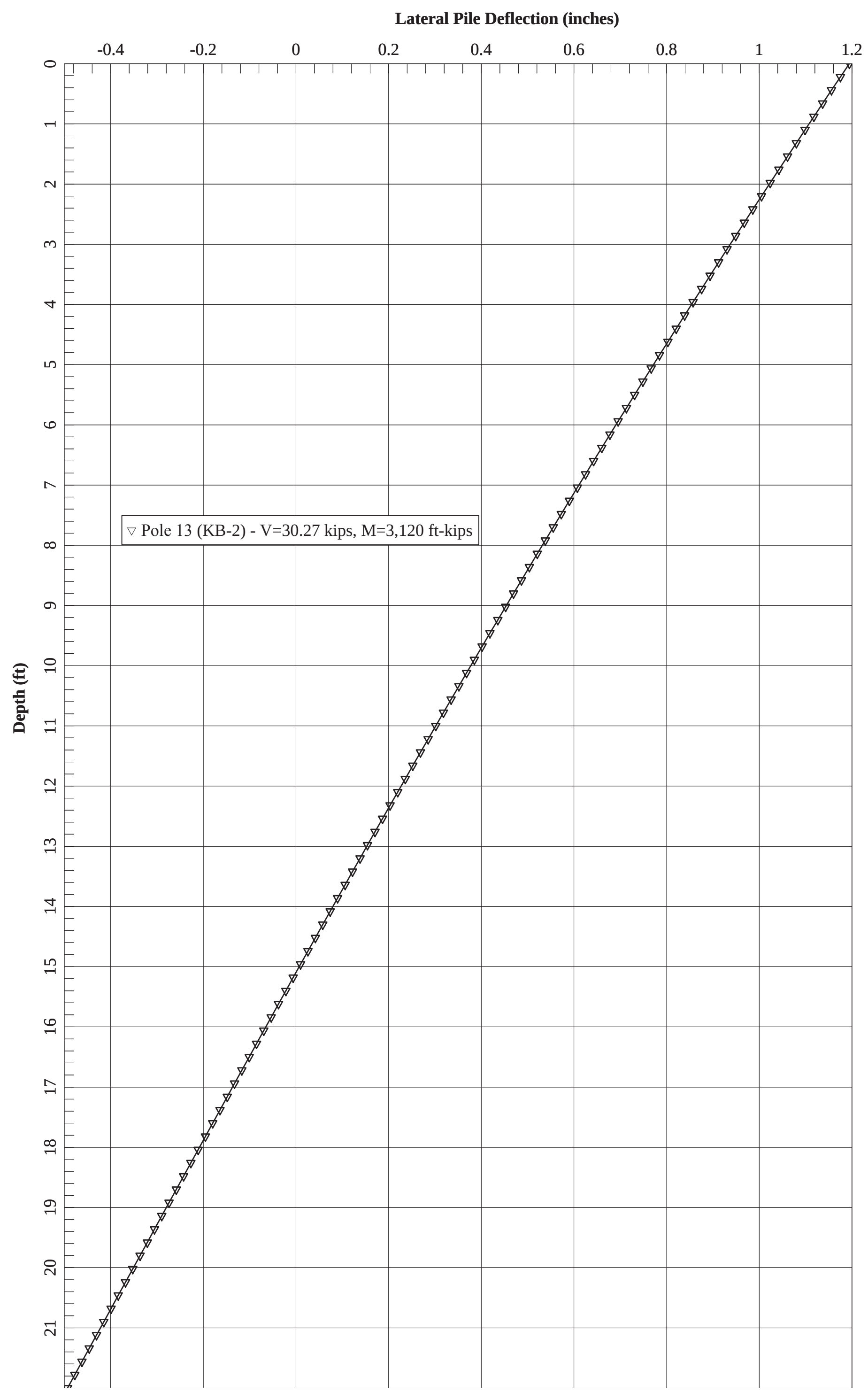
PG&E FULTON FITCH TSP REPLACEMENT
GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

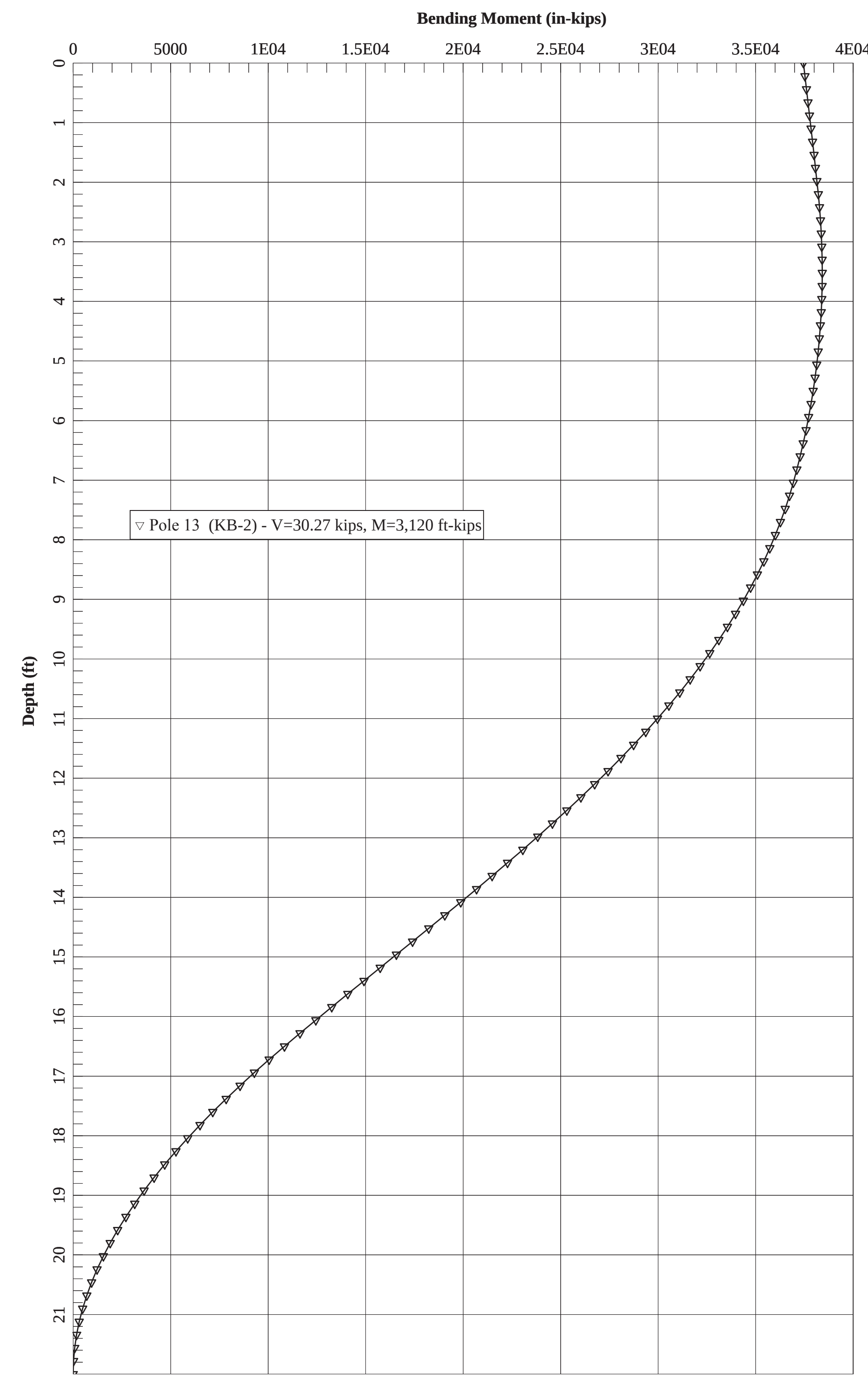
FIGURE
9

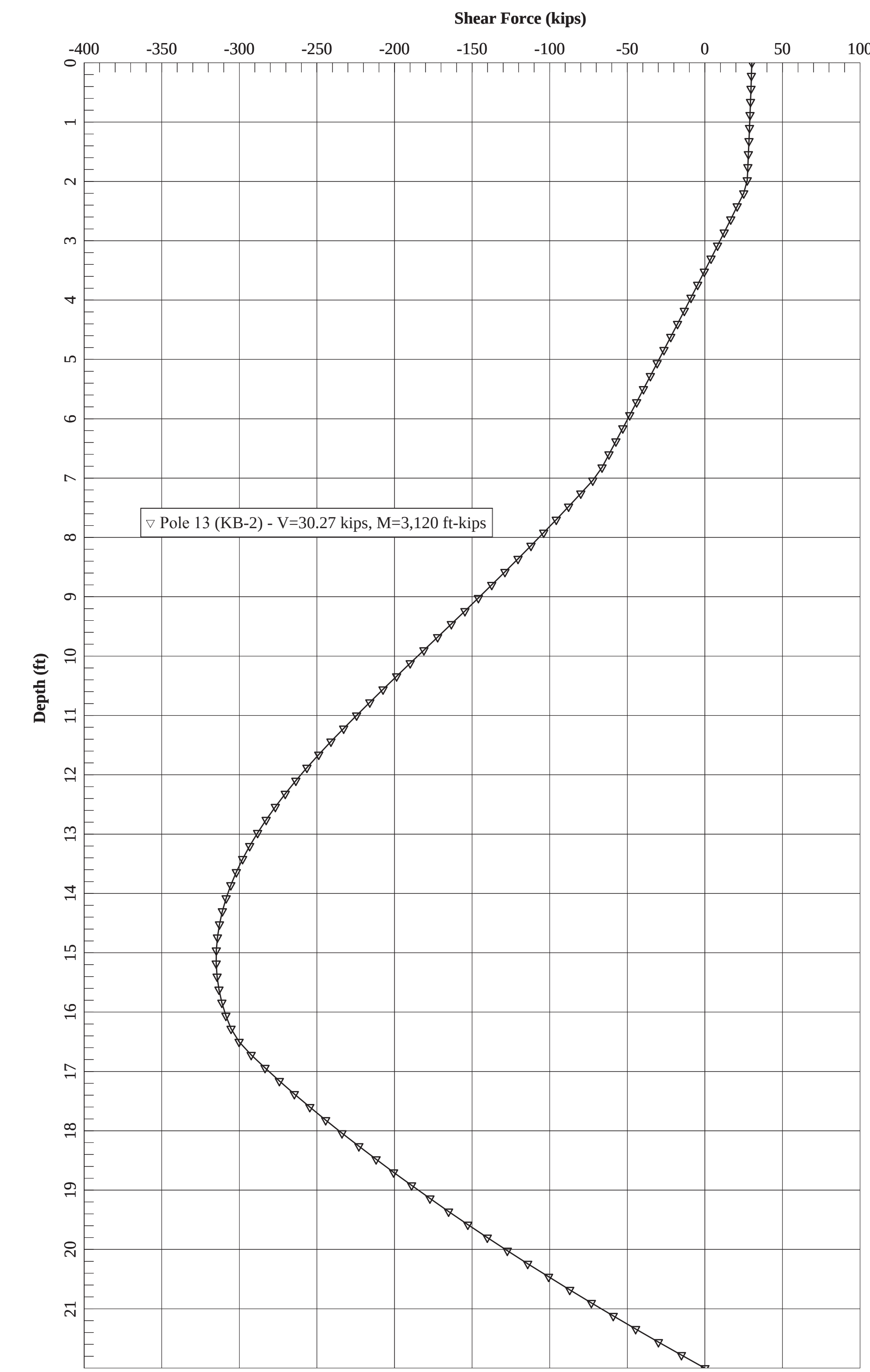


KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20190527.001A	POLE 8 LATERAL PILE RESPONSE 72-INCH DIAMETER DRILLED PIER	FIGURE
	DRAWN AUG 2018		
	DRAWN BY SDC		10
	CHECKED BY MJP	PG&E FULTON FITCH TSP REPLACEMENT GEOTECHNICAL INVESTIGATION SANTA ROSA, CALIFORNIA	
	FILE NAME Figure 4 .ai		

Santa Rosa







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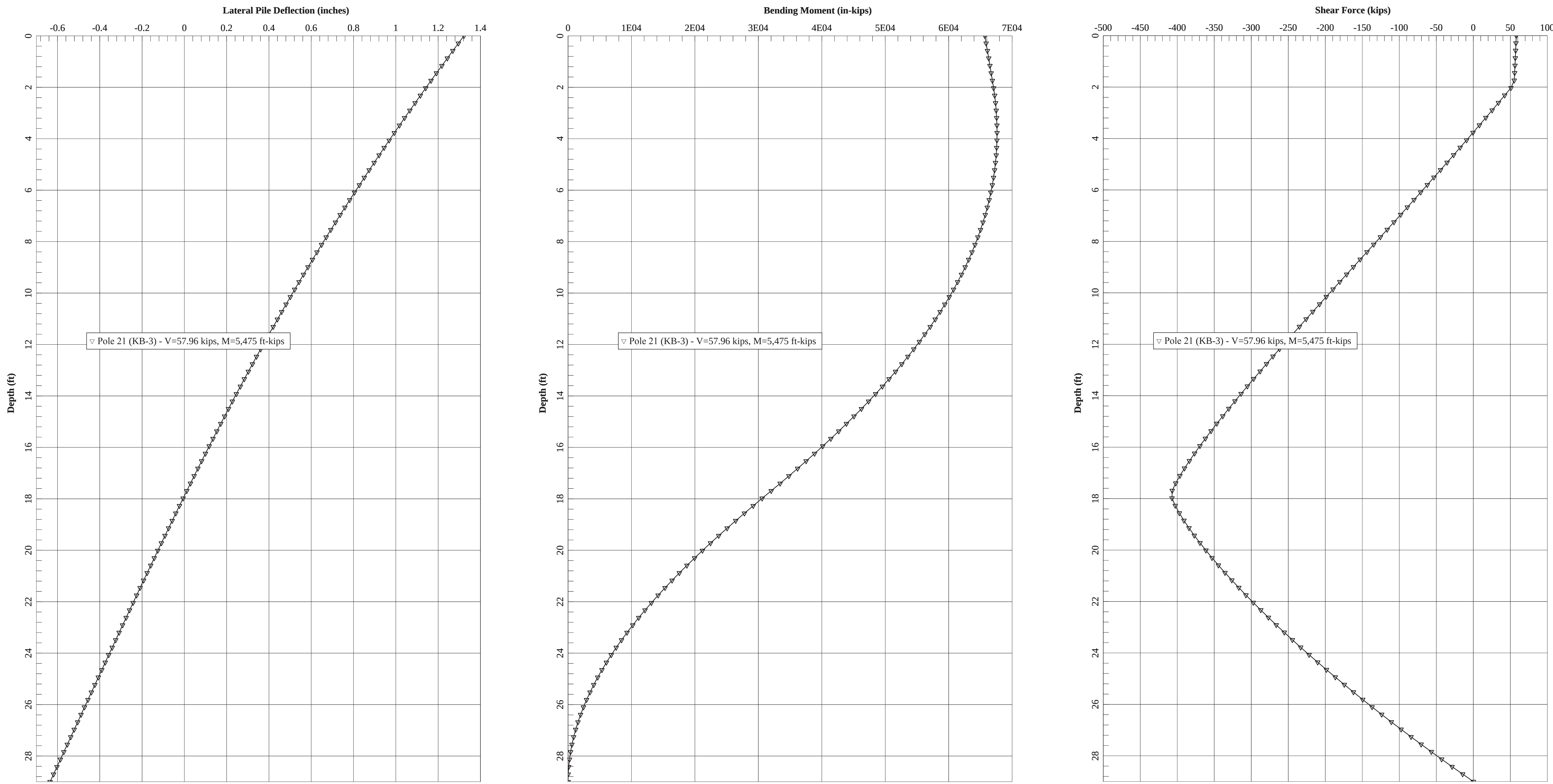
FILE NAME	Figure 4 .ai

POLE 13
LATERAL PILE RESPONSE
72-INCH DIAMETER DRILLED PIER

PG&E FULTON FITCH TSP REPLACEMENT
GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

FIGURE
11



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DRAWN	AUG 2018
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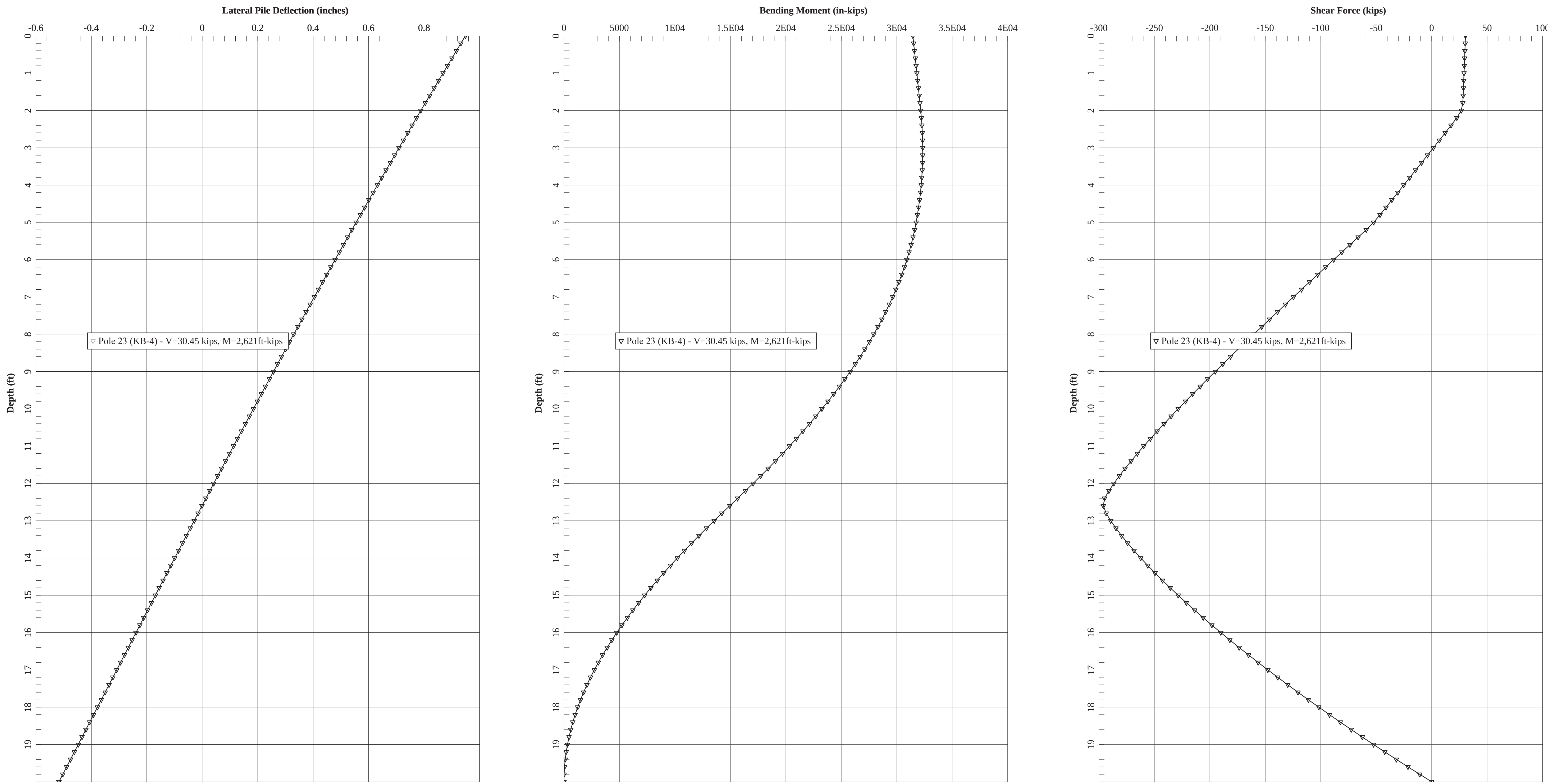
FILE NAME	Figure 4 .ai

POLE 21
LATERAL PILE RESPONSE
72-INCH DIAMETER DRILLED PIER

PG&E FULTON FITCH TSP REPLACEMENT
GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

FIGURE
12

Santa Rosa



Santa Rosa

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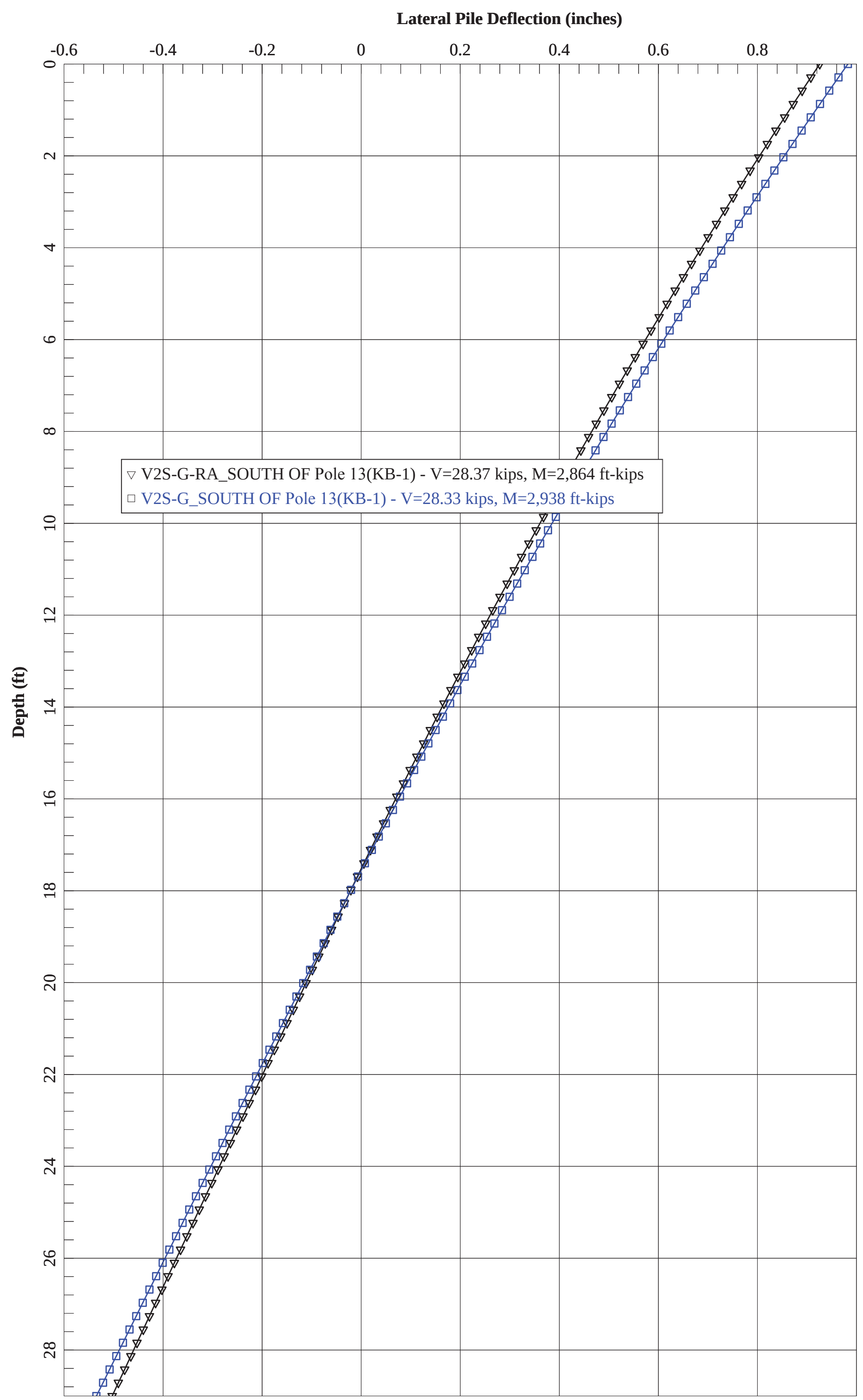
FILE NAME	Figure 4 .ai

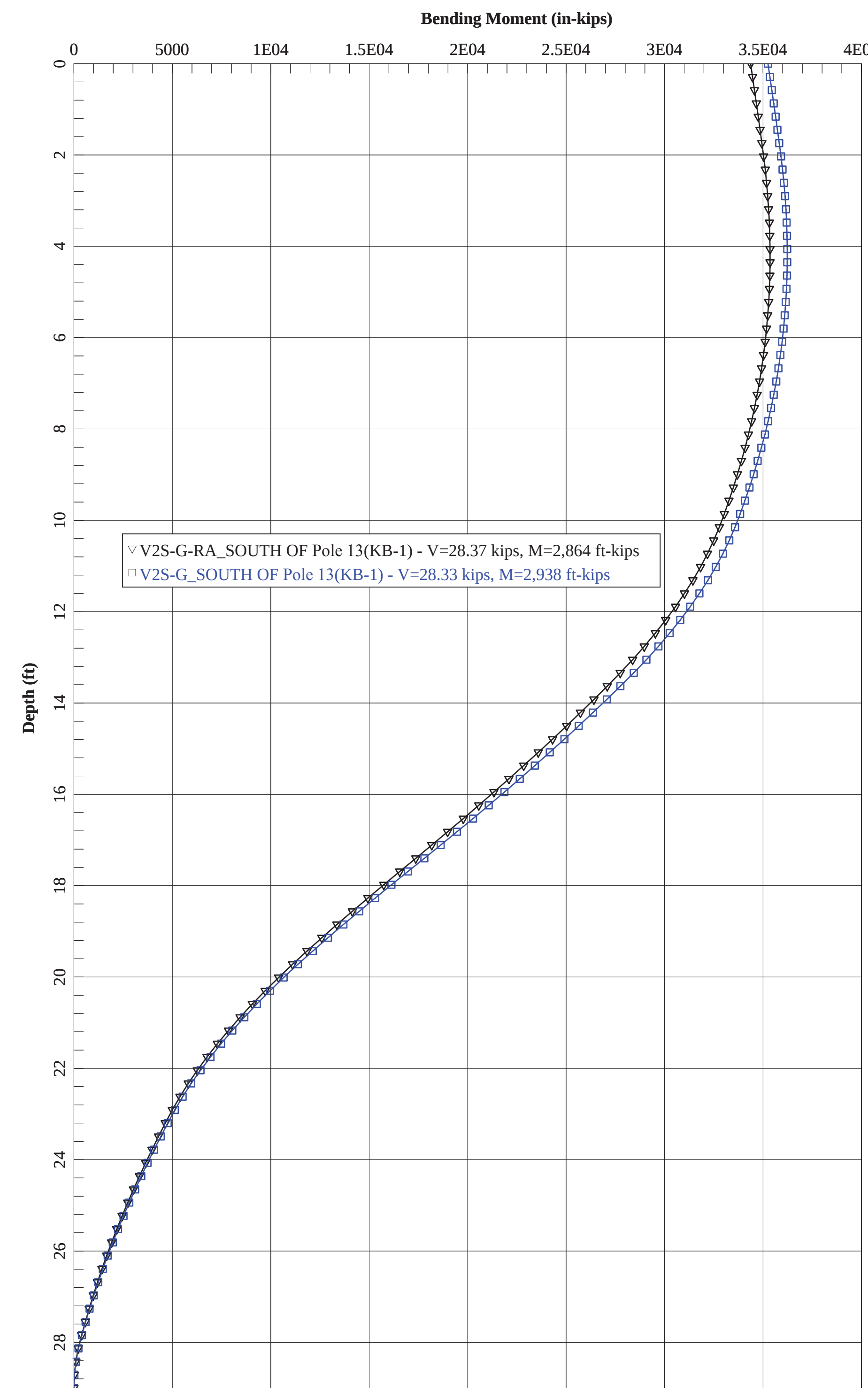
POLE 23
LATERAL PILE RESPONSE
72-INCH DIAMETER DRILLED PIER

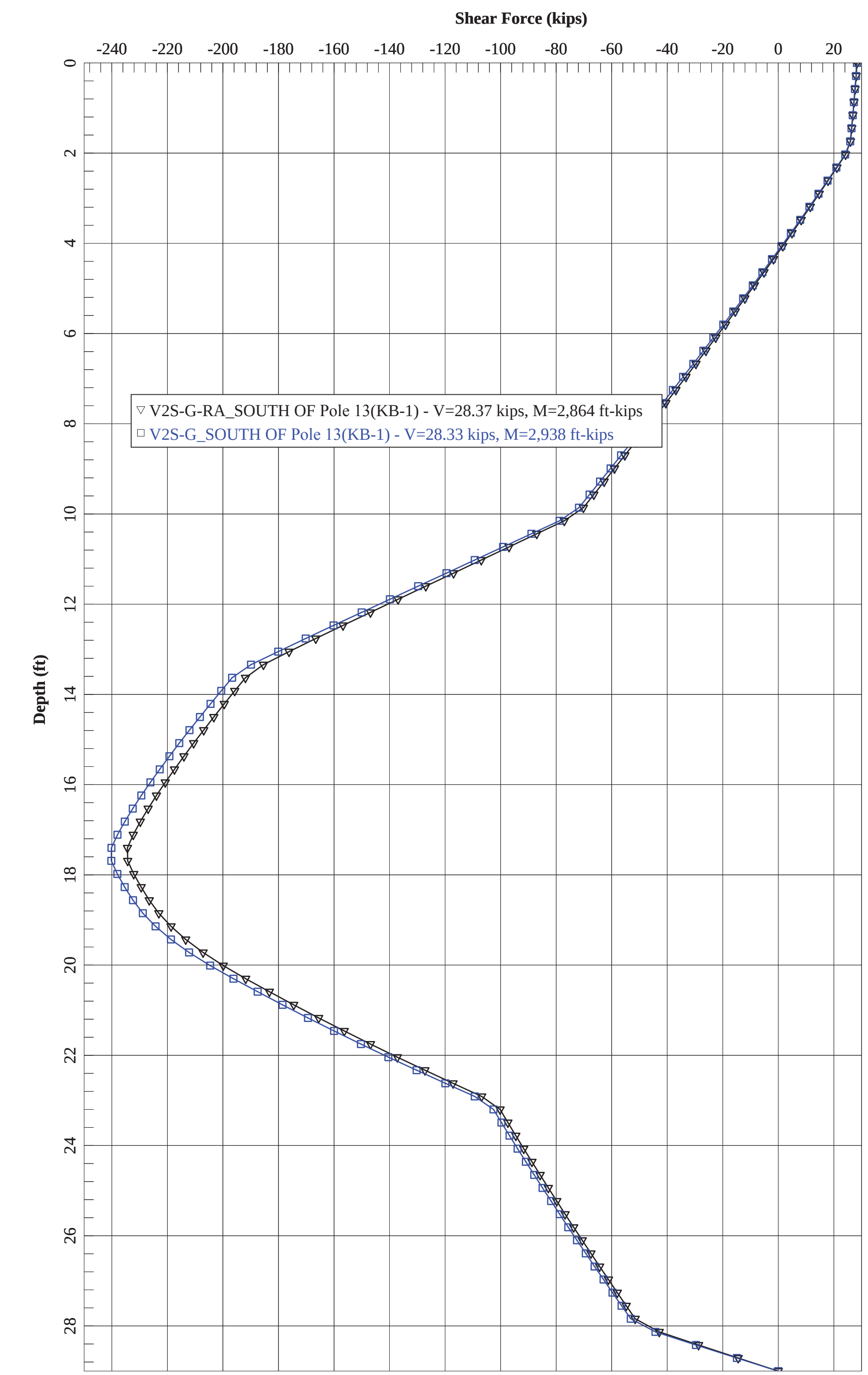
PG&E FULTON FITCH TSP REPLACEMENT
GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

FIGURE

13







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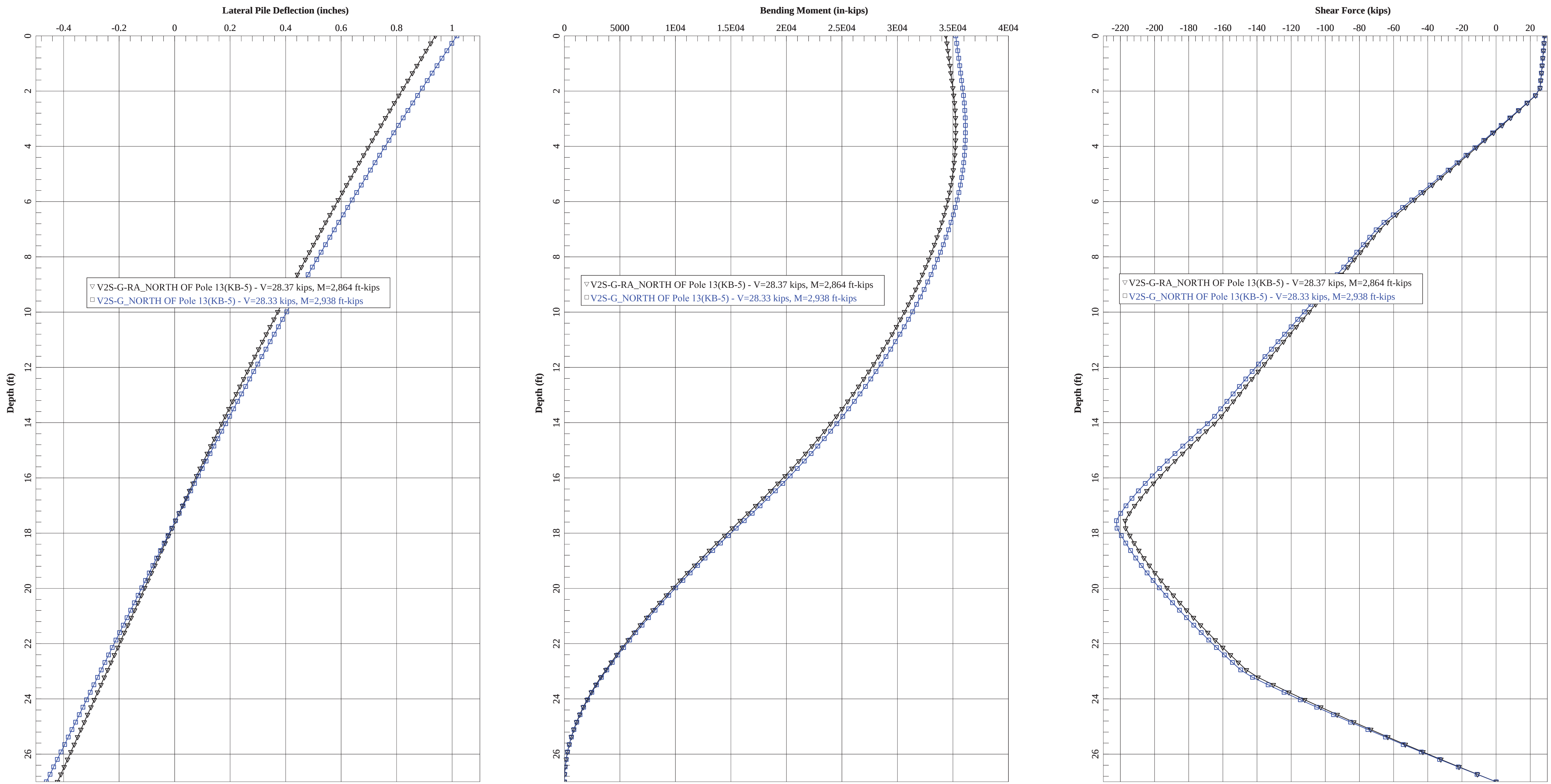
CHECKED BY	MJP
FILE NAME	Figure 4 .ai

V2S-G AND V2S-G-RA: SOUTH OF POLE 13
LATERAL PILE RESPONSE
72-INCH DIAMETER DRILLED PIER

PG&E FULTON FITCH TSP REPLACEMENT
GEOTECHNICAL INVESTIGATION
SANTA ROSA, CALIFORNIA

FIGURE

14



PROJECT NO. 20190527.001A		V2S-G AND V2S-G-RA: NORTH OF POLE 13 LATERAL PILE RESPONSE 72-INCH DIAMETER DRILLED PIER	FIGURE
DRAWN	AUG 2018		15
DRAWN BY	SDC		
CHECKED BY	MJP	PG&E FULTON FITCH TSP REPLACEMENT GEOTECHNICAL INVESTIGATION SANTA ROSA, CALIFORNIA	
FILE NAME	Figure 4 .ai		

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Appendix A

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- 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)
- HQ CORE SAMPLE (2.500 in. (63.5 mm.) core diameter)
- SHELBY TUBE SAMPLER
- PUSH TYPE SAMPLER
- SONIC CONTINUOUS SAMPLER
- HAND AUGER
- AUGER CUTTINGS

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)

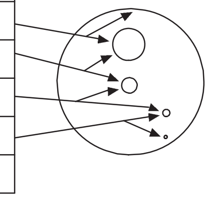
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu≥4 and 1≤Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/ or 1>Cc>3	**GP**	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

PLOTTED: 08/30/2018 09:04 AM BY: SDCain
OFFICE FILTER: SANTA ROSA
PROJECT NUMBER: 20190527.001A
gINT FILE: Klf_gint_master_2017
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [LEGEND 1 (GRAPHICS KEY) USCS_NOR_CAL]

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.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20190527 DRAWN BY: CHECKED BY: DATE: REVISED: -	SOIL DESCRIPTION KEY PG&E Fulton-Fitch TSP Replacements Santa Rosa, California	FIGURE A-2

PLOTTED: 08/30/2018 09:05 AM BY: SDCain
OFFICE FILTER: SANTA ROSA
PROJECT NUMBER: 20190527.001A
gINT FILE: Klf_gint_master_2017
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [LEGEND 2 (SOIL DESC KEY)]

Date Begin - End:	7/16/2018	**Drilling Company:**	Taber Drilling
Logged By:	S. Cain	**Drill Crew:**	Chad, Trevor, Shawn, Lawrence
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

BORING LOG KB-1

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.49993° N
Longitude: -122.76026° E
Approximate Ground Surface Elevation (ft.): 144
Surface Condition: AC

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	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
		AC: 5"											Hand Auger to 5'
		AB: 5"											
		Lean CLAY with Sand (CL): dark brown, dry to moist, medium to coarse grained sand (fill)											
140		**Sandy Lean CLAY (CL)**: olive brown, moist, medium grained sand (fill/reworked native?)											
	5	**Lean CLAY (CL)**: mottled dark brown and olive brown, moist, stiff to very stiff, some fine to medium grained sand (alluvium)		BC=4 7 9 PP=3.0	94%		30.4	92.7					Switch to 6" Solid Flight Auger at 5'
135		mottled gray and olive brown, stiff, increasing sand content		BC=5 6 8 PP=2.5	78%		29.6	107.6					TXUU: c = 1.32 ksf
	10	**Clayey SAND (SC)**: olive, moist to wet, loose, fine to medium grained sand (groundwater ∇ at ~11.3')		BC=2 3 3	100%	SC	35.6	83.5		40	33	16	
130		**Lean CLAY (CL)**: olive, moist to wet, medium stiff											
	15			BC=5 4 4	78%								Switch to 4.5" Mud Rotary at 15'
125		**Clayey SAND (SC)**: olive to grayish olive, wet, loose, fine to medium grained sand											
	20			BC=2 3 4	94%	SC	31.6	91.7		48	33	14	
120		**Sandy Fat CLAY (CH)**: gray to gray brown, moist, soft, fine to medium grained sand, oxidation staining											
	25			BC=2 3 5 PP=1	100%		44.7	76.0					TXUU: c = 0.63 ksf
115		**Clayey SAND (SC)**: gray to olive gray, moist, medium dense, fine to medium grained sand											
	30			BC=6 9 9	61%	SC	32.8	92.0		49	32	12	
110		**Lean CLAY (CL)**: gray, moist, stiff, trace subrounded gravel to 0.25", black seams											

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PROJECT NO.:	20190527
DRAWN BY:	SDC
CHECKED BY:	MJP
DATE:	
REVISED:	-

BORING LOG KB-1

PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

FIGURE
A-3

Date Begin - End:	7/16/2018	Drilling Company:	Taber Drilling	**BORING LOG KB-1**
Logged By:	S. Cain	Drill Crew:	Chad, Trevor, Shawn, Lawrence	
Hor.-Vert. Datum:	WGS84	Drilling Equipment:	CME-55 Track Rig	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hand Auger, Solid Flight Auger, Mud Rotary	
Weather:	Clear, Sunny	Exploration Diameter:	6/4.5 in. O.D.	

FIELD EXPLORATION — Latitude: 38.49993° N; Longitude: -122.76026° E; Approximate Ground Surface Elevation (ft.): 144; Surface Condition: AC

LABORATORY RESULTS

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	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
			Poorly Graded SAND with Clay and Gravel (SP-SC): olive gray to olive, moist, very dense, some scattered orange coloring, fine to coarse grained sand, subrounded to subangular gravel to 0.5", 2.5" gravel in shoe		BC=18 28 37 PP=4.5+	83%								Drill rig chattering at 36.5'
105														
	40		wet, reduced fines content, 2" rock fragment		BC=10 28 26	28%								Continued drill rig chatter
														Increased drilling resistance
100			1.5" rock fragment		BC=50/2"	100%								
	45													
95	50													
90	55													
85	60													
80	65													
75														

The boring was terminated at approximately 43.5 ft. below ground surface. The boring was backfilled with cement grout with a rapid set concrete surface patch on July 16, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 11.5 ft. below ground surface during drilling.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder using Google Earth.

BORING LOG KB-2

Date Begin - End:	7/17/2018	**Drilling Company:**	Taber Drilling
Logged By:	S. Cain	**Drill Crew:**	Chad, Trevor, Lawrence
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION

Latitude: 38.50628° N
Longitude: -122.76122° E
Approximate Ground Surface Elevation (ft.): 149
Surface Condition: AC

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	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
		AC: 3"											Hand Auger to 5'
		AB: 15"											
		Sandy Lean CLAY (CL): brown to dark brown, dry to moist, fine to coarse grained sand											
		fine to medium grained sand											
145													
	5	moist, very stiff to hard, rootlets encountered		BC=5 4 4 PP=4	67%		19.2	86.7					Switch to 6" Solid Flight Auger at 5' TXUU: c = 2.01 ksf
		Clayey SAND (SC): brown, moist, loose, fine to medium grained sand (alluvium)		BC=4 4 4 PP=4.5	83%								
140	10			BC=3 4 4 PP=3.5	83%	SC				41	31	9	
135	15	mottled gray and brown, decreasing fines content		BC=2 3 5	72%								
		Poorly Graded SAND with Clay (SP-SC): orange brown, wet, medium dense, fine to coarse grained sand, trace subrounded gravel to 0.5"		BC=4 7 10	72%								
130		**Poorly Graded SAND with Gravel (SP)**: brown to gray brown, wet, dense, medium to coarse grained sand, subangular to subrounded gravel to 1", 1.5" quartz fragment											Switch to 4.5" Mud Rotary at 18'
	20			BC=11 24 26	44%								Drill chattering from 19', loss of circulation Increased circulation loss, continued chattering/grinding
125		**Poorly Graded GRAVEL with Silt and Sand (GP-GM)**: brown to gray brown, wet, dense, medium to coarse grained sand, subangular to subrounded gravel to 1"											
	25			BC=12 13 17	39%	GP-GM			50	11			Continued but lessening drill chatter and circulation loss
120	30			BC=15 18 17	72%								No chatter at 30'
		Clayey SAND (SC): olive to olive brown, wet, dense, fine to medium grained sand, black speckling											
115													

LABORATORY RESULTS

Date Begin - End:	7/17/2018	**Drilling Company:**	Taber Drilling
Logged By:	S. Cain	**Drill Crew:**	Chad, Trevor, Lawrence
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

BORING LOG KB-2

FIELD EXPLORATION

Latitude: 38.50628° N
Longitude: -122.76122° E
Approximate Ground Surface Elevation (ft.): 149
Surface Condition: AC

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	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): olive brown, wet, medium stiff, fine to medium grained sand, black carbon speckling		BC=4 6 7	22%								
110			**Fat CLAY (CH)**: olive to grayish olive, moist, hard, trace medium to coarse grained sand, trace subrounded gravel to 0.5"											
	40				BC=8 11 14	94%								
105			**Poorly Graded GRAVEL with Clay and Sand (GP-GC)**: olive gray, wet, very dense, medium to coarse grained sand, subangular to subrounded gravel to 2"											Intermittent chattering/grinding from 43'
	45				BC=20 16 39	6%								
														Increasing chatter at 47'
100														
	50				BC=29 50/2"	25%								Heavy drill rig chatter at 49.5'
95	55													
90	60													
85	65													
80														

LABORATORY RESULTS

The boring was terminated at approximately 50.5 ft. below ground surface. The boring was backfilled with cement grout with a rapid set concrete surface patch on July 17, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 17.5 ft. below ground surface during drilling.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder using Google Earth.

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PROJECT NO.: 20190527	BORING LOG KB-2	FIGURE
DRAWN BY: SDC		
CHECKED BY: MJP	PG&E Fulton-Fitch TSP Replacements Santa Rosa, California	A-4
DATE:		
REVISED: -		PAGE: 2 of 2

BORING LOG KB-3

Date Begin - End:	7/18/2018	**Drilling Company:**	Taber Drilling		
Logged By:	S. Cain	**Drill Crew:**	Chad, Trevor, Lawrence		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary		
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.		

FIELD EXPLORATION

Latitude: 38.51809° N
Longitude: -122.76050° E
Approximate Ground Surface Elevation (ft.): 174
Surface Condition: AC

LABORATORY RESULTS

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	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		AC: 5"											Hand Auger to 5'
			AC: 7"											
170			**Lean CLAY with Sand (CL)**: brown to dark brown, moist, fine to coarse sand (reworked native)											
	5		**Sandy Lean CLAY (CL)**: dark brown, dry to moist, very stiff, fine to coarse grained sand, trace subangular to subrounded gravel to 0.75" (alluvium)		BC=6 9 13 PP=3.5	100%								Switch to 6" Solid Flight Auger at 5'
			hard		BC=7 14 20 PP=4.5+	72%								
165	10		sand lense (3")		BC=14 17 13 PP=4.5+	100%								
			Sandy Lean CLAY with Gravel (CL): brown to dark brown, dry to moist, hard, fine to coarse grained sand, subangular to rounded gravel to 0.5"											
160	15				BC=17 17 13 PP=4.5+	100%		19.5	109.0					
155	20				BC=23 43 30 PP=4.5+	78%								Switch to 4.5" Mud Rotary at 20'
			Clayey SAND with Gravel (SC): brown, dry to moist, very dense, fine to coarse grained sand, subangular to subrounded gravel to 2"											
150			**Sandy Lean CLAY with Gravel (CL)**: grayish olive, moist to wet, hard, fine to medium grained sand, subangular gravel to 2"											
	25				BC=10 11 11 PP=4.5+	67%								
145														Drill rig chattering from 29'
	30		fine to coarse grained sand		BC=10 15 9 PP=4.5	72%		15.1	111.2					
140														Intermittent chattering from 33-35'

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PROJECT NO.: 20190527	BORING LOG KB-3		FIGURE
DRAWN BY: SDC			
CHECKED BY: MJP	PG&E Fulton-Fitch TSP Replacements Santa Rosa, California		A-5
DATE:			
REVISED: -			

Date Begin - End:	7/18/2018	**Drilling Company:**	Taber Drilling		**BORING LOG KB-3**
Logged By:	S. Cain	**Drill Crew:**	Chad, Trevor, Lawrence		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary		
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.51809° N
Longitude: -122.76050° E
Approximate Ground Surface Elevation (ft.): 174
Surface Condition: AC

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	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 with Gravel (CL): grayish olive, moist to wet, hard, fine to medium grained sand, subangular gravel to 2" 3" gravel encountered	BC=12 15 22 PP=4.5+	72%								
135	40	moist, mostly coarse sand, decreasing sand and gravel content	BC=12 17 23 PP=4.5+	100%								
		Lean CLAY with Sand (CL): grayish olive, moist, hard, fine sand										
130	45	trace subangular gravel to 0.5"	BC=21 24 50/5" PP=4.5+	88%								
125		increase in sand content										
	50		BC=12 22 50/5" PP=4.5+	100%								
		Clayey SAND with Gravel (SC): grayish brown, moist, very dense, fine to coarse grained sand, mostly coarse, subangular to rounded gravel to 1.5"										Drill rig chattering from 51.5', increased drilling resistance
120	55	medium dense, increasing fines content	BC=12 13 15	56%								
												Reduced chatter at 57.5'
115	60	very dense	BC=34 50/4"	40%								
110	65											
105												

The boring was terminated at approximately 61 ft. below ground surface. The boring was backfilled with cement grout with a rapid set concrete surface patch on July 18, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater not measured

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder using Google Earth.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20190527 DRAWN BY: SDC CHECKED BY: MJP DATE: REVISED: -	BORING LOG KB-3 PG&E Fulton-Fitch TSP Replacements Santa Rosa, California	FIGURE A-5

BORING LOG KB-4

Date Begin - End:	7/19/2018	**Drilling Company:**	Taber Drilling		
Logged By:	S. Cain	**Drill Crew:**	Chad, Trevor, Lawrence		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary		
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.		

FIELD EXPLORATION: Latitude: 38.51846° N; Longitude: -122.75695° E; Approximate Ground Surface Elevation (ft.): 250; Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	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): light brown to brown, dry, fine to coarse grained sand, trace angular fine gravel (completely weathered Glen Ellen Bedrock)										Hand Auger to 5'
		brown, dry to moist, trace medium grained sand, some structure evident										
245	5	**Clayey SAND (SC)**: light yellowish brown, dry to moist, dense, fine grained sand, trace silt, some structure evident (highly weathered Glen Ellen Bedrock)	BC=30 41 44 PP=4.5+	100%		17.2	81.8					Switch to 6" Solid Flight Auger at 5'
		brown, trace medium grained sand	BC=12 30 28 PP=4.5+	67%								
240	10	**Lean CLAY (CL)**: olive brown, dry to moist, hard, (highly weathered Glen Ellen Bedrock)	BC=9 22 33 PP=4.5+	83%		21.1	84.6					TXUU: c = 9.27 ksf Increased drilling resistance from 11.5'
235	15	brown to light grayish brown, moist, trace silt, some fine to medium grained sand, oxidation staining/seams, rootlets	BC=20 44 50/5" PP=4.5+	65%								
		Sandy Fat CLAY (CH): brown to gray brown, moist, hard, trace fine to medium grained sand (highly weathered Glen Ellen Bedrock)										
230	20	light grayish brown, black and orange oxidation staining/seams	BC=31 49 39 PP=4.5+	56%		25.0	92.0					Switch to 4.5" Mud Rotary at 20'
225	25	olive brown, gray streaks, black speckling	BC=12 20 23 PP=4.5+	94%		28.6	89.3					TXUU: c = 5.88 ksf
		Sandy Lean CLAY (CL): dark gray blue, moist, hard, fine to medium grained sand, occasional oxidation staining, friable (highly weathered Glen Ellen Bedrock)										
220	30		BC=13 27 39 PP=4.5+	100%		34.7	87.1					
												Drill rig chatter/grinding from 33'

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PROJECT NO.: 20190527	BORING LOG KB-4		FIGURE
DRAWN BY: SDC			A-6
CHECKED BY: MJP	PG&E Fulton-Fitch TSP Replacements Santa Rosa, California		
DATE:			
REVISED: -			

BORING LOG KB-4

Date Begin - End:	7/19/2018	**Drilling Company:**	Taber Drilling		
Logged By:	S. Cain	**Drill Crew:**	Chad, Trevor, Lawrence		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary		
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.		

FIELD EXPLORATION — Latitude: 38.51846° N; Longitude: -122.75695° E; Approximate Ground Surface Elevation (ft.): 250; Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	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
			Poorly Graded SAND with Clay (SP-SC): dark gray, wet, very dense, fine to coarse grained sand, subangular to subrounded gravel and rock fragments to 1.5" (highly weathered Glen Ellen Bedrock)		BC=36 50/6"	100%								
														Heavy chattering from 38'
210	40				BC=50/1"	100%								Circulation loss from 40'
														Heavy chatter continued to 45', consistent circulation loss
205	45				BC=22 27 32	28%								Intermittent chatter from 45'
			Clayey SAND (SC): bluish gray, moist to wet, medium dense, fine to medium grained sand, trace coarse grained sand and subangular to subrounded gravel to 1"											
200	50				BC=15 9 12	56%								
														Refill mud tub at 51.5'
			increased clay content (possible Sandy CLAY)											Increased circulation loss at 54'
195	55				BC=10 17 42 PP=4.5+	83%								Refill mud tub at 54.5'
190	60													
185	65													

The boring was terminated at approximately 56.5 ft. below ground surface. The boring was backfilled with cement grout with a surface patch of auger cuttings on July 19, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater not measured

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder using Google Earth.

Date Begin - End:	7/20/2018	**Drilling Company:**	Taber Drilling
Logged By:	S. Cain	**Drill Crew:**	Chad, Lawrence, Adam
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

BORING LOG KB-5

FIELD EXPLORATION

Latitude: 38.50923° N
Longitude: -122.76038° E
Approximate Ground Surface Elevation (ft.): 157
Surface Condition: Grass

LABORATORY RESULTS

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	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
		Clayey SAND (SC): brown, dry, fine to coarse grained sand (mostly fine to medium)										Hand Auger to 5'
155		**Sandy Lean CLAY (CL)**: brown, dry, fine to coarse grained sand subangular to subrounded gravel to 1.5"										
	5	moist	BC=9 12 15 PP=4.5+	72%		21.0	105.0					Switch to 6" Solid Flight Auger at 5'
150		**Sandy Fat CLAY (CH)**: brown, moist, stiff, fine to coarse grained sand, subangular to subrounded gravel to 0.5", rootlets olive brown, mostly fine grained sand	BC=4 5 6 PP=2	100%								
	10	medium stiff	BC=4 6 6 PP=1	94%								
145												
	15	angular coarse grained sand, rootlets in top liner	BC=12 15 16 PP=4.5+	100%		19.2	107.3					
		Fat CLAY with Sand (CH): grayish brown to dark grayish brown, moist, hard, fine to coarse grained sand										
140												
	∇											
	20	medium stiff, increased sand content, occasional black carbon speckling	BC=6 8 11 PP=2.0	100%		19.6	104.7					Switch to 4.5" Mud Rotary at 20' TXUU: c = 1.34 ksf
135												
		Sandy Lean CLAY (CL): olive to olive brown, moist, hard, fine to coarse grained sand										
	25		BC=10 14 20 PP=4.5+	83%								
130												
		sand lense, fine to coarse grained sand, subangular to subrounded gravel to 2"										Light drill rig chatter at 28'
	30	very stiff	BC=15 10 17 PP=3.0	89%		14.2	115.8					TXUU: c = 3.43 ksf
125												

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PROJECT NO.:	20190527
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CHECKED BY:	MJP
DATE:	
REVISED:	-

BORING LOG KB-5

PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

FIGURE
A-7

Date Begin - End:	7/20/2018	**Drilling Company:**	Taber Drilling		**BORING LOG KB-5**
Logged By:	S. Cain	**Drill Crew:**	Chad, Lawrence, Adam		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	CME-55 Track Rig	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hand Auger, Solid Flight Auger, Mud Rotary		
Weather:	Clear, Sunny	**Exploration Diameter:**	6/4.5 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.50923° N
Longitude: -122.76038° E
Approximate Ground Surface Elevation (ft.): 157
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	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): olive to olive brown, moist, hard, fine to coarse grained sand		BC=10 7 8	%								
120														Coarse sand in fluid returns
			light olive brown, very stiff to hard, mostly coarse grained sand											
	40		**Fat CLAY with Sand (CH)**: light olive brown to grayish brown, moist, very stiff to hard, fine to coarse grained sand, rock fragments to 1.5"		BC=12 11 13 PP=4.0	100%								
115														Drill rig chattering at 43'
	45		sand lense (6")		BC=30 33 22	83%								
110														
			Lean CLAY with Sand (CL): olive and orange brown, moist, stiff, fine to coarse grained sand (mostly fine grained sand), trace subrounded gravel to 0.25"											
	50				BC=9 9 12 PP=2.0	100%								
105														
			Clayey SAND (SC): brown to grayish brown, moist to wet, very dense, fine to coarse grained sand (mostly coarse grained sand), trace subangular to subrounded gravel to 0.5"											
	55				BC=20 33 40	78%								
100														
			Lean CLAY (CL): olive brown, moist to wet, very stiff, trace fine to coarse grained sand											
	60				BC=12 17 19	17%								
95														
	65													
90														

The boring was terminated at approximately 61.5 ft. below ground surface. The boring was backfilled with cement grout with a surface patch of auger cuttings on July 20, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 19 ft. below ground surface during drilling.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder using Google Earth.

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PROJECT NO.:	20190527	BORING LOG KB-5	FIGURE
DRAWN BY:	SDC		A-7
CHECKED BY:	MJP	PG&E Fulton-Fitch TSP Replacements Santa Rosa, California	
DATE:			
REVISED:	-		PAGE: 2 of 2

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Appendix B

Exploration ID	Depth (ft.)	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	
KB-1	6.0		30.4	92.7							
KB-1	9.0		29.6	107.6							TXUU: c = 1.32 ksf
KB-1	11.0	CLAYEY SAND (SC)	35.6	83.5			40	33	17	16	
KB-1	21.0	CLAYEY SAND (SC)	31.6	91.7			48	33	19	14	
KB-1	25.5		44.7	76.0							TXUU: c = 0.63 ksf
KB-1	31.0	CLAYEY SAND (SC)	32.8	92.0			49	32	20	12	
KB-2	6.0		19.2	86.7							TXUU: c = 2.01 ksf
KB-2	11.0	CLAYEY SAND (SC)					41	31	22	9	
KB-2	25.0	POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM)			80	50	11				
KB-3	16.0		19.5	109.0							
KB-3	31.0		15.1	111.2							
KB-4	6.0		17.2	81.8							
KB-4	11.0		21.1	84.6							TXUU: c = 9.27 ksf
KB-4	21.0		25.0	92.0							
KB-4	26.0		28.6	89.3							TXUU: c = 5.88 ksf
KB-4	31.0		34.7	87.1							
KB-5	6.0		21.0	105.0							
KB-5	15.5		19.2	107.3							
KB-5	21.0		19.6	104.7							TXUU: c = 1.34 ksf
KB-5	31.0		14.2	115.8							TXUU: c = 3.43 ksf

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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PROJECT NO.: 20190527
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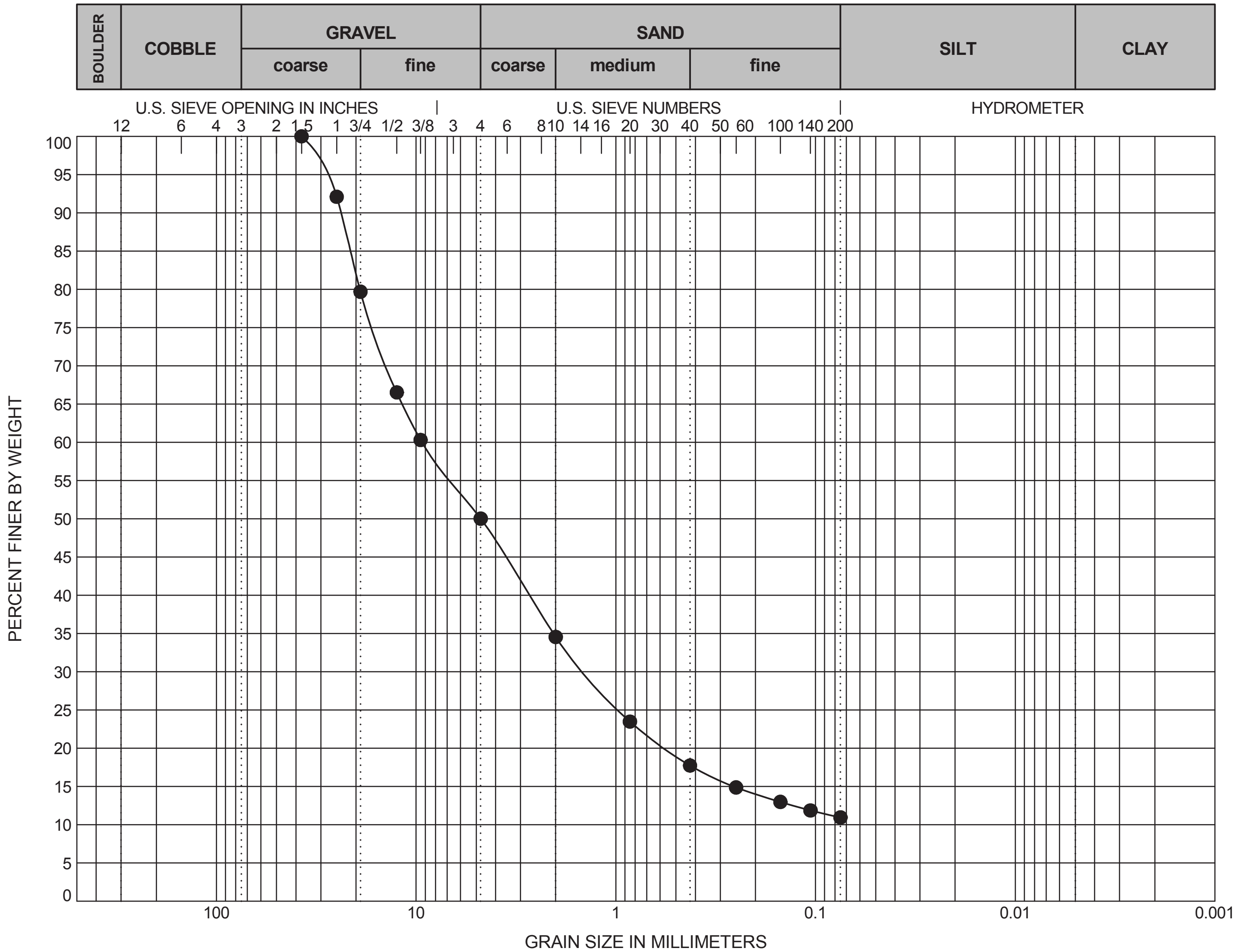
DATE:
REVISED: -

LABORATORY TEST RESULT SUMMARY

PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

FIGURE
B-1



Exploration ID	Depth (ft.)	Sample Description	LL	PL	PI
● KB-2	25	POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM)	NM	NM	NM

Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
● KB-2	25	37.5	9.309	1.407	NM	4.07	178.13	80	50	11	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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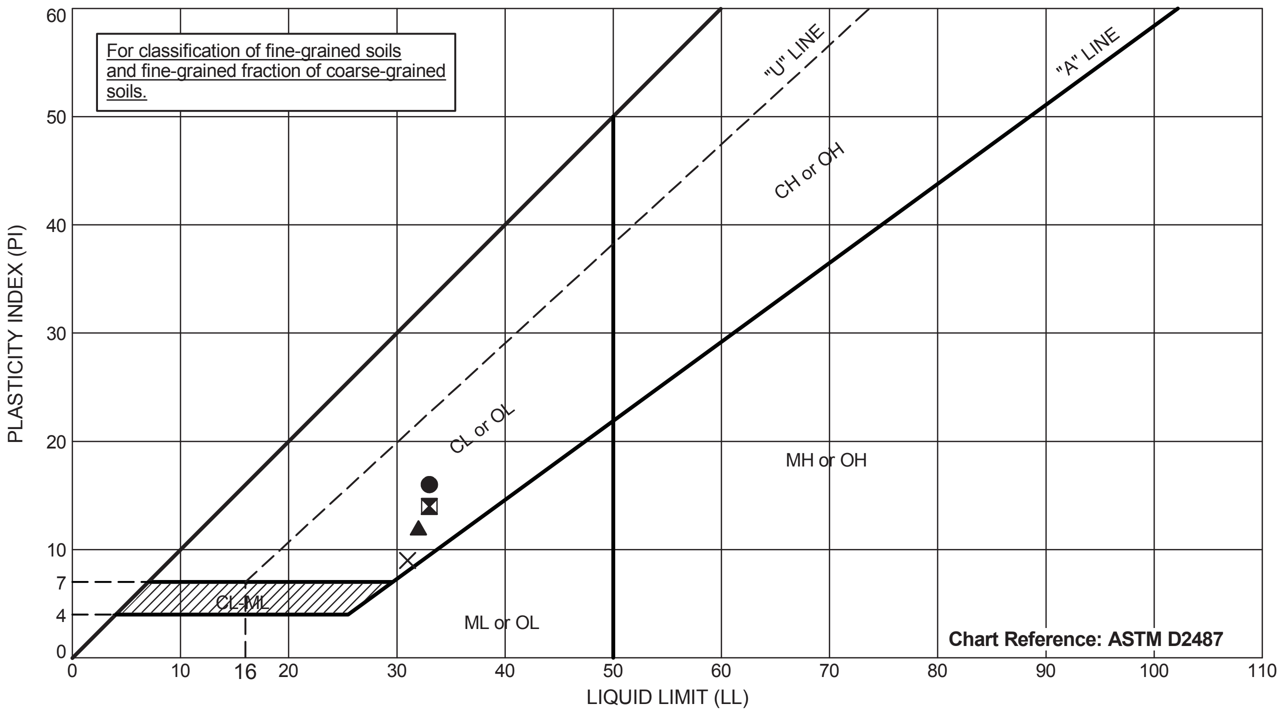
PROJECT NO.:	20190527
DRAWN BY:	SDC
CHECKED BY:	MJP
DATE:	
REVISED:	-

SIEVE ANALYSIS

PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

FIGURE
B-2



	Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
●	KB-1	11	CLAYEY SAND (SC)	40	33	17	16
⊠	KB-1	21	CLAYEY SAND (SC)	48	33	19	14
▲	KB-1	31	CLAYEY SAND (SC)	49	32	20	12
×	KB-2	11	CLAYEY SAND (SC)	41	31	22	9

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

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PROJECT NO.:	20190527
DRAWN BY:	SDC
CHECKED BY:	MJP
DATE:	
REVISED:	-

ATTERBERG LIMITS

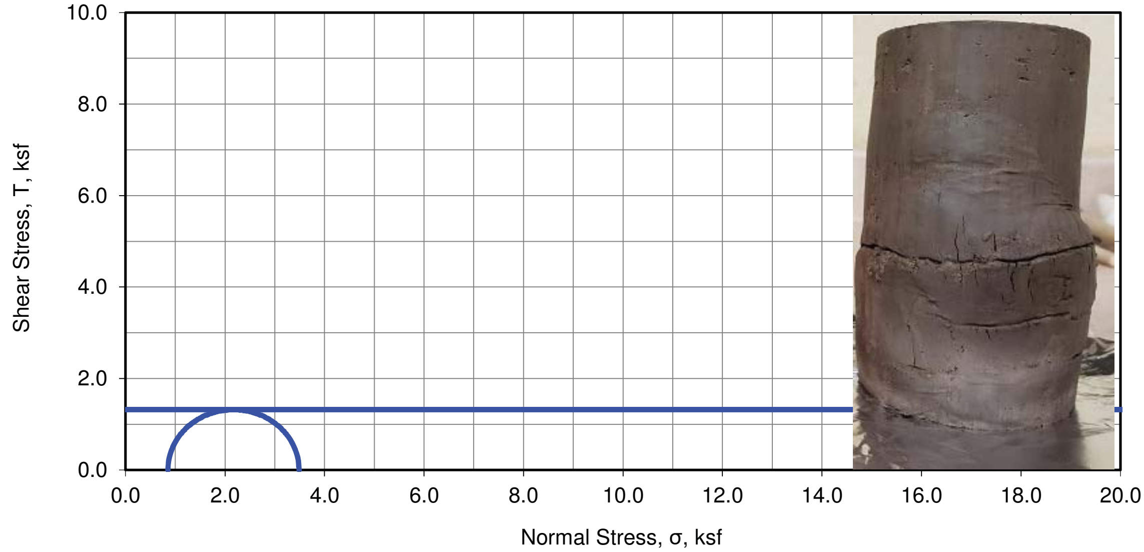
PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

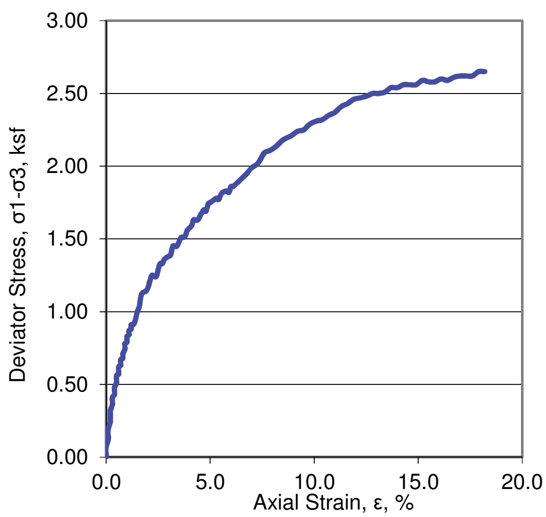
FIGURE

B-3

Total		
c =	1.32	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	4.37
	Water Content, %	ω_O	29.6
	Dry Density, lbs/ft^3	γd_o	107.6
	Saturation, %	S_O	146
	Void Ratio	e_O	0.537
Minor Principal Stress, ksf		σ_3	0.85
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	2.65
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	14.83
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.56
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.83

Description of Specimen: Dark Yellowish Brown Lean Clay (CL)

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:	KB-1
Sample:	2B
Depth, ft:	9.0
Test Date:	8/1/18

Remarks: nm= not measured, na = not applicable

KLEINFELDER
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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20190527.001A
Date:	8/3/18
Entry By:	CP
Checked By:	CP
File Name:	HL11486

TRIAXIAL COMPRESSION TEST (UU)

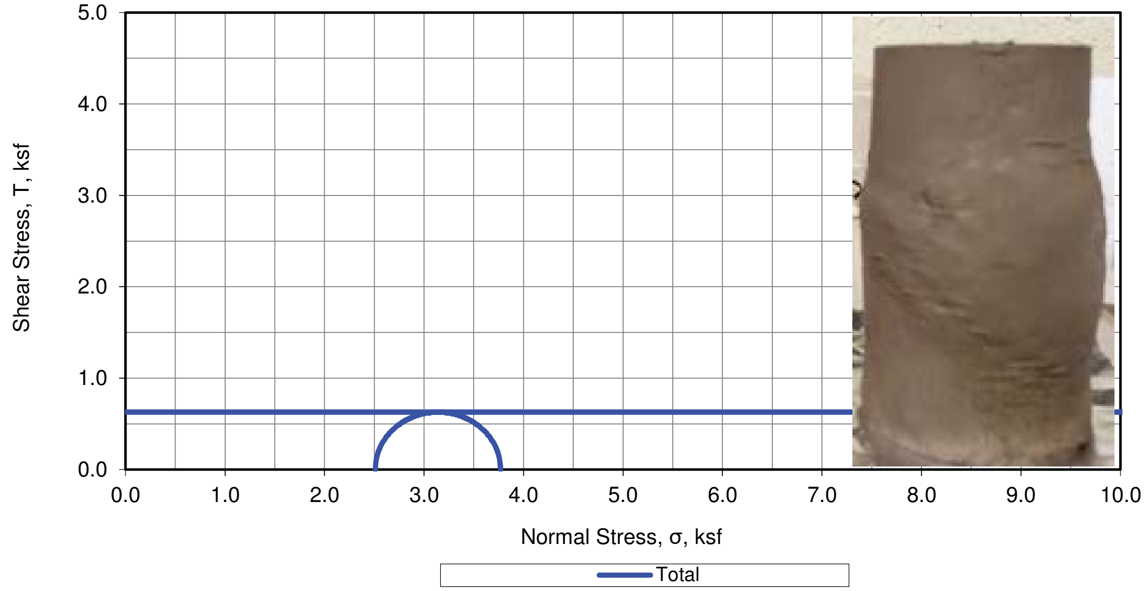
PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

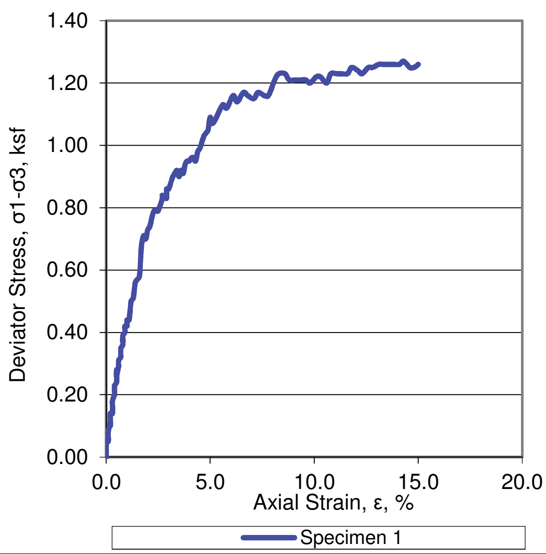
FIGURE

B-4

Total	
c = 0.63	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.39
	Height, in	H_O	5.36
	Water Content, %	ω_O	44.7
	Dry Density, lbs/ft^3	γ_{d_o}	76.0
	Saturation, %	S_O	101
	Void Ratio	e_O	1.176
Minor Principal Stress, ksf		σ_3	2.51
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	1.27
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	1.26
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.33

Description of Specimen: Dark Grayish Brown Sandy Fat Clay (CH)

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:	KB-1
Sample:	6B
Depth, ft:	25.5
Test Date:	8/1/18

Remarks: nm= not measured, na = not applicable

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Project No.:	20190527.001A
Date:	8/3/18
Entry By:	CP
Checked By:	CP
File Name:	HL11486

TRIAXIAL COMPRESSION TEST (UU)

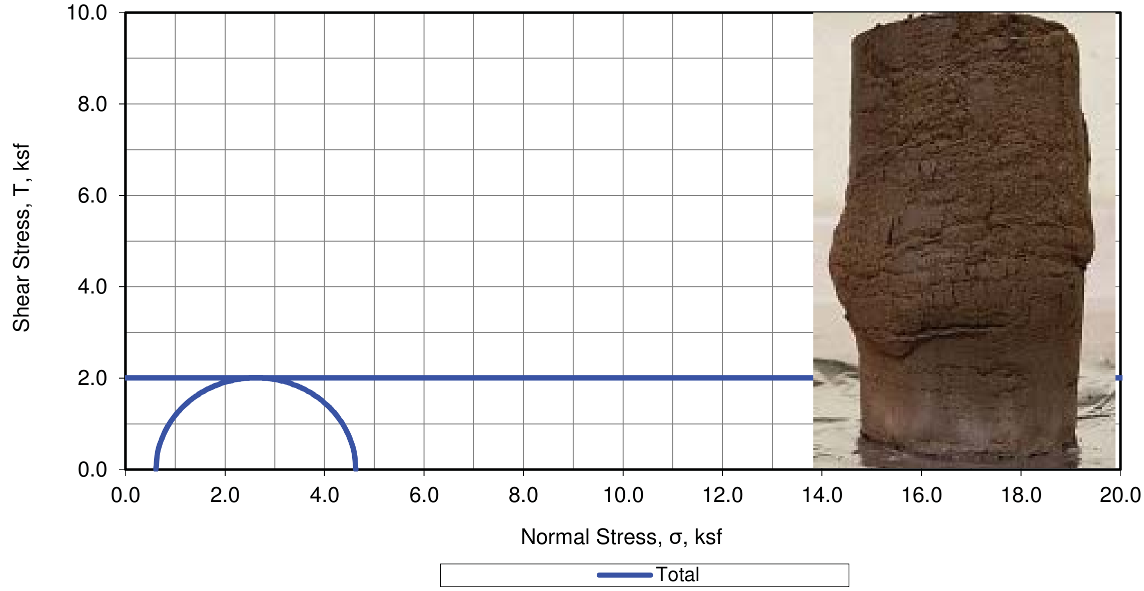
PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

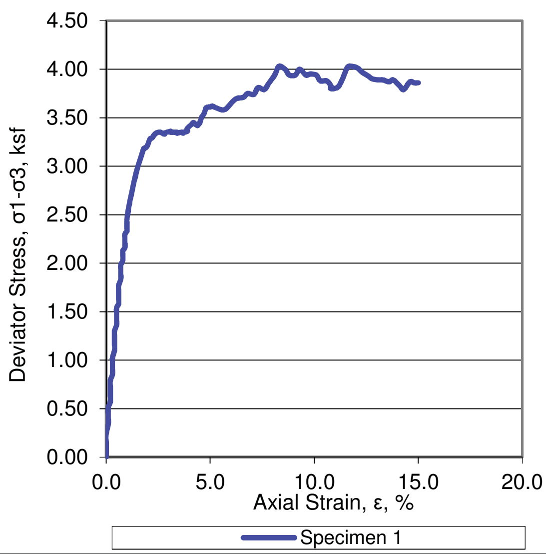
FIGURE

B-5

Total	
c = 2.01	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.39
	Height, in	H_O	5.67
	Water Content, %	ω_O	19.2
	Dry Density, lbs/ft^3	γd_o	86.7
	Saturation, %	S_O	56
	Void Ratio	e_O	0.907
Minor Principal Stress, ksf		σ_3	0.60
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	4.03
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	8.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	3.86
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	8.33

Description of Specimen: Dark Yellowish Brown Sandy Lean Clay (CL)

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:	KB-2
Sample:	2C
Depth, ft:	6.0
Test Date:	8/2/18

Remarks: nm= not measured, na = not applicable

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Project No.:	20190527.001A
Date:	8/3/18
Entry By:	CP
Checked By:	CP
File Name:	HL11486

TRIAXIAL COMPRESSION TEST (UU)

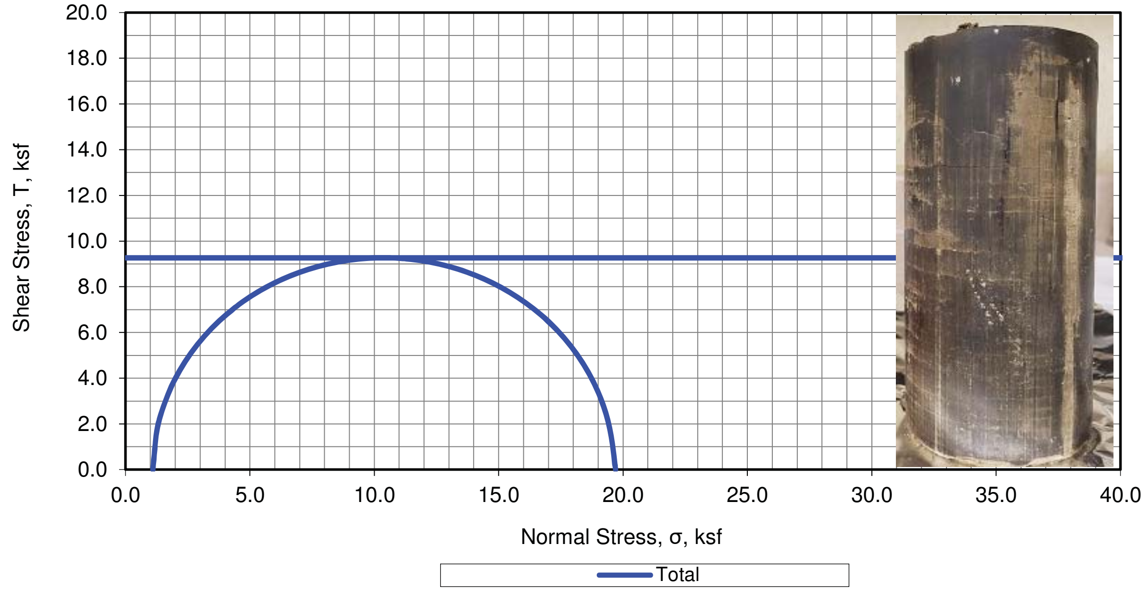
PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

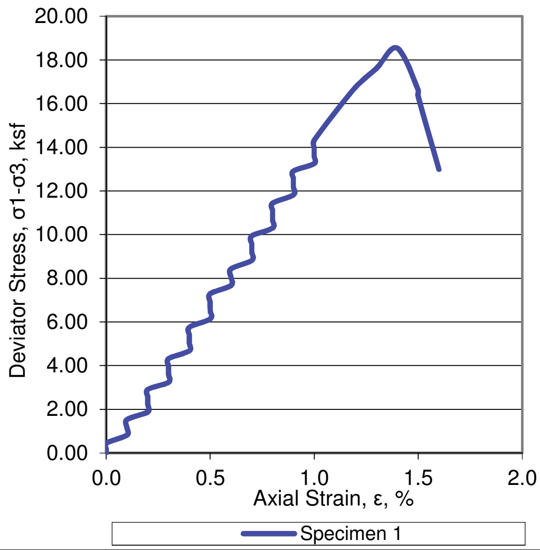
FIGURE

B-6

Total	
c = 9.27	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.30
	Water Content, %	ω_O	21.1
	Dry Density, lbs/ft^3	γd_o	84.6
	Saturation, %	S_O	59
	Void Ratio	e_O	0.955
Minor Principal Stress, ksf		σ_3	1.09
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	18.54
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	1.45
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	12.99
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	1.45

Description of Specimen: Olive Brown Lean Clay (CL)

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:	KB-4
Sample:	4C
Depth, ft:	11.0
Test Date:	8/2/18

Remarks: nm= not measured, na = not applicable

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Project No.:	20190527.001A
Date:	8/3/18
Entry By:	CP
Checked By:	CP
File Name:	HL11486

TRIAXIAL COMPRESSION TEST (UU)

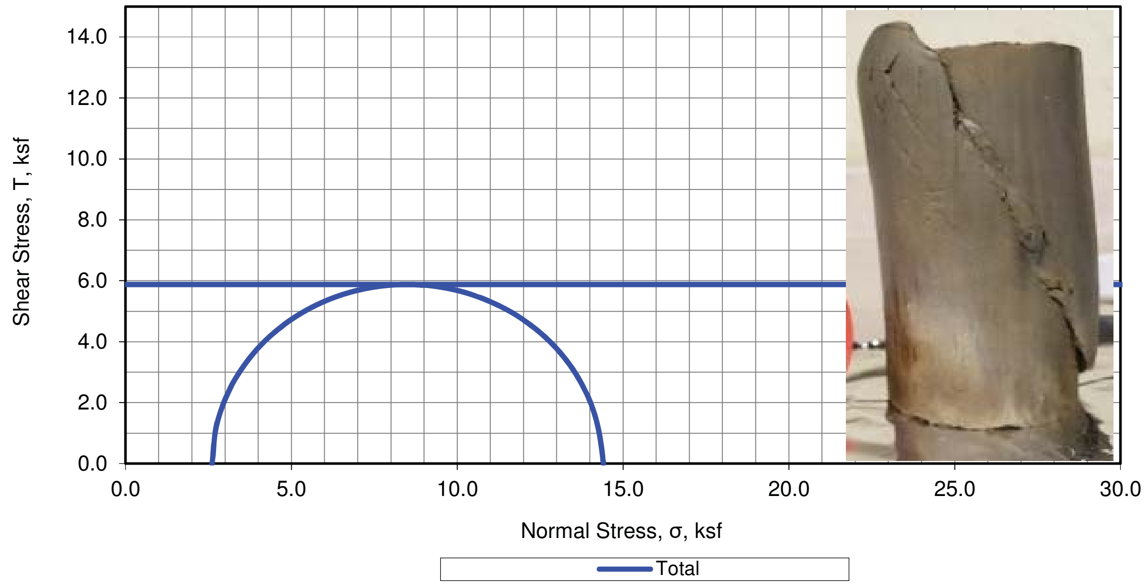
PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

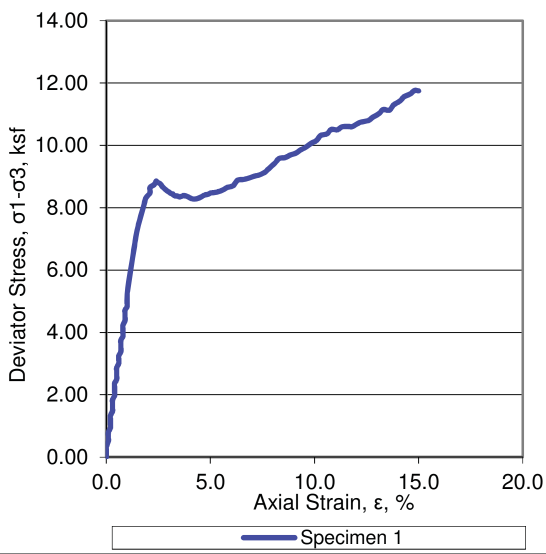
FIGURE

B-7

Total		
c =	5.88	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.47
	Water Content, %	ω_O	28.6
	Dry Density, lbs/ft^3	γ_{d_o}	89.3
	Saturation, %	S_O	89
	Void Ratio	e_O	0.852
Minor Principal Stress, ksf		σ_3	2.61
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	11.76
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	14.82
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	11.75
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.82

Description of Specimen: Olive Brown Sandy Fat Clay (CH)

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:	KB-4
Sample:	7C
Depth, ft:	26.0
Test Date:	8/2/18

Remarks: nm= not measured, na = not applicable

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Date:	8/3/18
Entry By:	CP
Checked By:	CP
File Name:	HL11486

TRIAXIAL COMPRESSION TEST (UU)

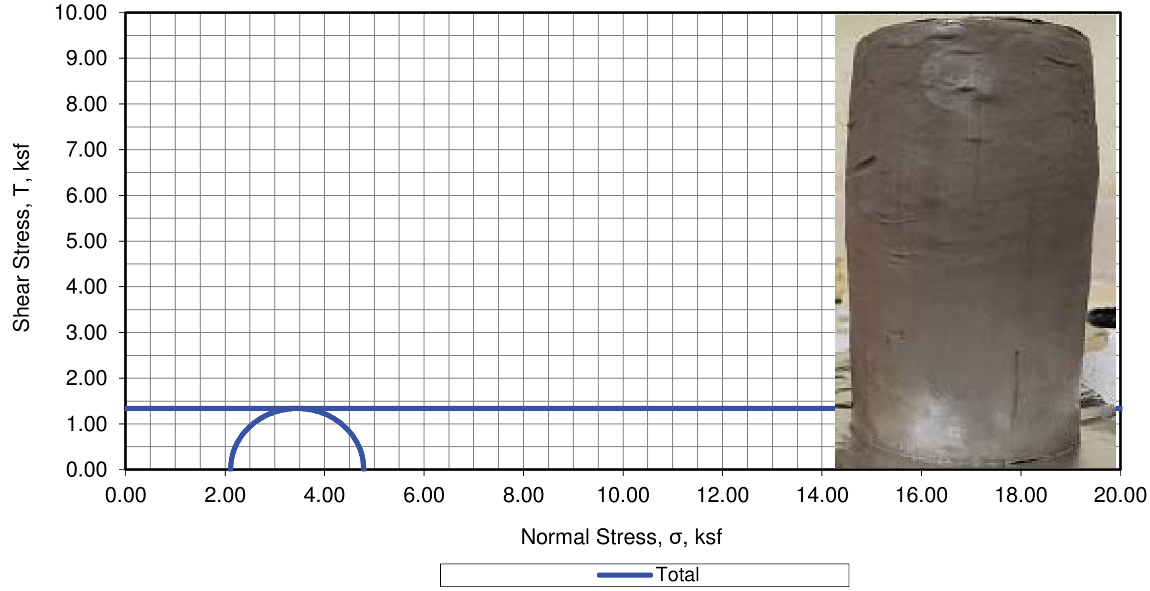
PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

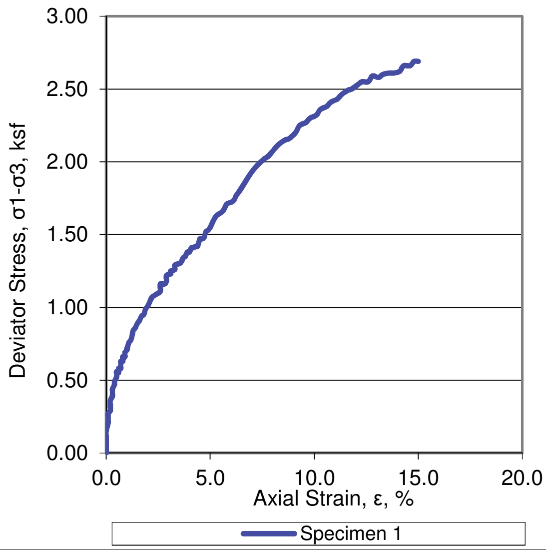
FIGURE

B-8

Total		
c =	1.34	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.39
	Height, in	H_O	5.35
	Water Content, %	ω_O	19.6
	Dry Density, lbs/ft^3	γd_o	104.7
	Saturation, %	S_O	90
	Void Ratio	e_O	0.579
Minor Principal Stress, ksf		σ_3	2.10
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	2.69
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.83
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.69
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.83

Description of Specimen: Sandy Fat Clay (CH)

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:	KB-5
Sample:	6C
Depth, ft:	21.0
Test Date:	8/2/18

Remarks: nm= not measured, na = not applicable

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Project No.:	20190527.001A
Date:	8/3/18
Entry By:	CP
Checked By:	CP
File Name:	HL11486

TRIAXIAL COMPRESSION TEST (UU)

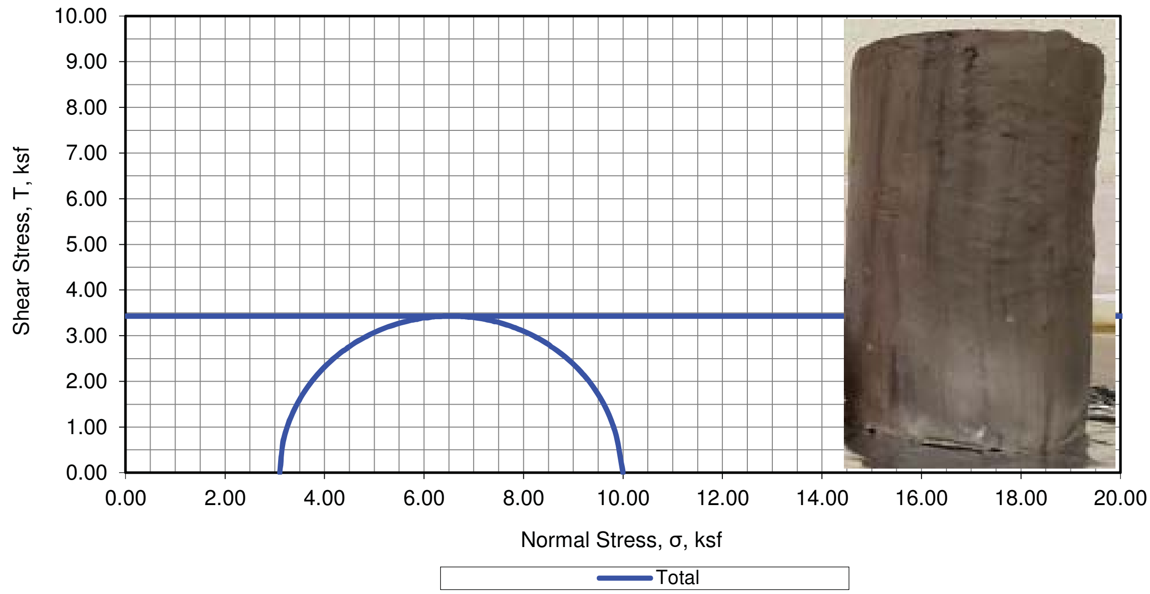
PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

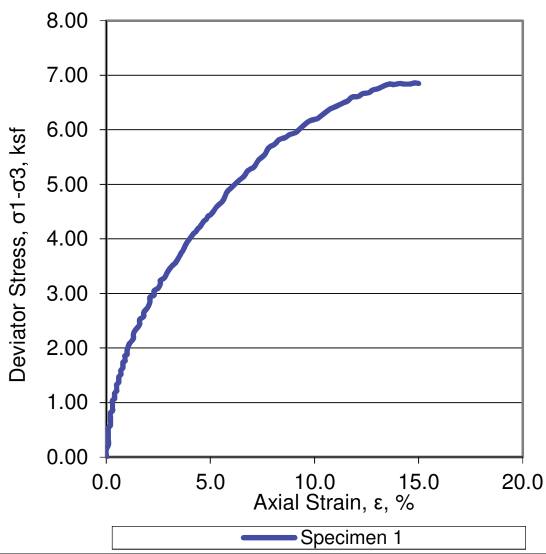
FIGURE

B-9

Total	
c = 3.43	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
Initial	Height, in	H_O	5.05
Initial	Water Content, %	ω_O	14.2
Initial	Dry Density, lbs/ft^3	γd_o	115.8
Initial	Saturation, %	S_O	88
Initial	Void Ratio	e_O	0.428
Minor Principal Stress, ksf		σ_3	3.10
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	6.86
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.82
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	6.85
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.82

Description of Specimen: Sandy Lean Clay (CL)

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:	KB-5
Sample:	8C
Depth, ft:	31.0
Test Date:	8/2/18

Remarks: nm= not measured, na = not applicable

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Project No.:	20190527.001A
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Entry By:	CP
Checked By:	CP
File Name:	HL11486

TRIAXIAL COMPRESSION TEST (UU)

PG&E Fulton-Fitch TSP Replacements
Santa Rosa, California

FIGURE
B-10

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Appendix C

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.: 20190527.001A
Client's Project Name: PG&E Fulton Fitch TSP Replacement
Date Sampled: 07/16-20/18
Date Received: 6-Aug-18
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 22-Aug-2018

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)*
1808022-001	KB-1, 1B @ 5.5'							
1808022-002	KB-1, SPT-4 @ 15'	+410	6.75	790	1,100	N.D.	N.D.	26
1808022-003	KB-2, 2B @ 5.5'							
1808022-004	KB-3, 2B @ 5.5'							
1808022-005	KB-3, 5C @ 16'							
1808022-006	KB-4, 4B @ 10.5'							
1808022-007	KB-4, 5B @ 15.5'	+340	7.17	2,400	980	N.D.	36	48
1808022-008	KB-5, 2B @ 5.5'							
1808022-009	KB-5, 3B @ 8'							

	Redox	pH	Resistivity (As Received)	Resistivity (100% Saturation)	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
	14-Aug-2018	14-Aug-2018	16-Aug-2018	16-Aug-2018	22-Aug-2018	19-Jul-2018	19-Jul-2018

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

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Appendix D

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. *Th se 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 fi st 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 fa tors when designing the study behind this report and developing the confi mation-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, configur tion, 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 uilding, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configur tion, 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 fl ods, 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 modifi d 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 signifi antly – 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 fin sh, 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 confi mation-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 fi alize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confi ms 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 specifi ations,
- review pertinent elements of other design professionals' plans and specifi ations, 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 roject requirements, including options selected from the report, *only* from the design drawings and specifi ations. 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 fi ancial 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 signifi antly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental fi dings, 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 deficie cies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infi tration.*** 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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