GEOTECHNICAL INVESTIGATION REPORT
NEW PG&E SANTA TERESA SUBSTATION
6402 SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

PROJECT NO. 20163632.001A

JANUARY 26, 2016

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January 26, 2016
Project No. 20163632.001A

Pacific Gas and Electric Company
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Attention: Alex Chung (A2CN@pge.com)
Joseph Sun, PhD, PE, GE (JIS4@pge.com)

SUBJECT: 2016_0126_Santa Teresa Substation_Geotechnical Investigation_PLE16R33661
Geotechnical Investigation Report
New PG&E Santa Teresa Substation
6402 Santa Teresa Boulevard
San Jose, California

Dear Mr. Chung:

The attached report presents the results of Kleinfelder's geotechnical investigation for the planned new PG&E Santa Teresa Substation in San Jose, California. The attached report describes the study, findings, conclusions, and recommendations for use in project design and construction. Kleinfelder's services are authorized by Contract Work Authorization No. 2501280865 dated October 21, 2015 and were performed in accordance with the terms of our Master Services Agreement No. 4400007810.

Ground shaking due to regional earthquake activity is anticipated during the life of the project and should be considered in project design. Recommendations for design of foundations, site grading, and other geotechnical considerations are presented in this report. The recommendations presented in this report should be incorporated into project design and construction.

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

Respectfully Submitted,
KLEINFELDER, INC.

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TABLE OF CONTENTS

SECTION **PAGE**

1 INTRODUCTION 1
1.1 PROPOSED CONSTRUCTION 1
1.2 PURPOSE AND SCOPE OF SERVICES 1

2 FIELD EXPLORATION AND LABORATORY TESTING 3
2.1 FIELD EXPLORATION 3
2.1.1 Cone Penetration Tests 3
2.1.2 Sampling Procedures 4
2.2 LABORATORY TESTING 4

3 GEOLOGIC CONDITIONS 5
3.1 SITE GEOLOGY 5
3.2 LOCAL AND REGIONAL FAULTING 5

4 SITE CONDITIONS 6
4.1 SITE DESCRIPTION 6
4.2 Subsurface conditions 6
4.3 GROUNDWATER 6
4.4 VARIATIONS IN SUBSURFACE CONDITIONS 7

5 CONCLUSIONS AND RECOMMENDATIONS 8
5.1 LIQUEFACTION 8
5.2 EXPANSIVE SOILS 9
5.3 2013 CBC SEISMIC DESIGN PARAMETERS 9
5.4 SITE PREPARATION 10
5.4.1 General 10
5.4.2 Stripping and Grubbing 10
5.4.3 Disturbed Soil, Undocumented Fill and Subsurface Obstructions 11
5.4.4 Scarification and Compaction 11
5.5 ENGINEERED FILL 11
5.5.1 Onsite Materials 11
5.5.2 Imported and Non-Expansive Fill Requirements 12
5.5.3 Placement and Compaction Criteria 13
5.6 WET WEATHER CONSIDERATIONS 13
5.7 SITE DRAINAGE 13
5.8 TEMPORARY EXCAVATIONS 13
5.8.1 General 13
5.8.2 Excavation and Slopes 14
5.9 TRENCH BACKFILL 14
5.10 SHALLOW FOUNDATION DESIGN 14
5.10.1.4 Spread Footing Construction Considerations 16
5.11 DRILLED PIER FOUNDATIONS 18
5.11.1 Axial Capacity 18
5.11.1.1 Estimated Settlement 19
5.11.2 Lateral Response 19
5.11.2.1 LPILE Analysis Soil Parameters 19
5.11.2.2 Canedo "Q" Value 19

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5.11.2.3 Lateral Load Resistance 20
5.11.2.4 Lateral Response Group Effects 20
5.11.3 Drilled Pier Construction Considerations 21
5.11.4 Temporary Casing 23
5.12 SOIL CORROSION 23

6 ADDITIONAL SERVICES 25
6.1 PLANS AND SPECIFICATIONS REVIEW 25
6.2 CONSTRUCTION OBSERVATION AND TESTING 25

7 LIMITATIONS 26

8 REFERENCES 27

FIGURES

Figure 1 Site Vicinity Map
Figure 2 Site Plan
Figures 3a and 3b Ultimate Axial Capacity, Unit Diameter (1-Foot) Drilled Pier

APPENDIX A – FIELD EXPLORATION - CPT LOGS

APPENDIX B – LABORATORY TEST RESULTS

Figure B-1 Plasticity Chart
Corrosivity Testing Results

APPENDIX C – LPILE ANALYSIS RESULTS, 3.5- FOOT DIAMETER

APPENDIX D – GBA INFORMATION SHEET

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1 INTRODUCTION

This report presents the results of our geotechnical investigation conducted for the planned new PG&E Santa Teresa Substation to be located at 6402 Santa Teresa Boulevard in San Jose, California. A site vicinity map is shown on Figure 1.

This report includes conclusions and recommendations related to the geotechnical aspects of project design and construction. 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.

1.1 PROPOSED CONSTRUCTION

We understand PG&E plans to build a new substation that will include a control building, circuit breakers, switchgear, and transformers, dead end structures, and other miscellaneous electrical equipment. Additional details provided by PG&E are noted below:

- Control building likely to be founded on a shallow foundation. Weight approximately 100 kips.
- Transformer bank likely to be founded on a shallow mat slab foundation. Weight approximately 150 kips.
- Dead-end structures:
    - The dead-end structures are anticipated be founded on 3.5-foot diameter, 8.5-foot deep drilled piers.
    - Ground-line moment = 53 foot-kips; ground-line shear = 2.5 kips; axial load 1.5 kips.

Onsite grading is anticipated to consist of minor cuts and fills up to 4 feet to remove any stockpiles, existing structures, and for site drainage toward the north. No retaining walls are anticipated.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore and evaluate subsurface conditions at the site and develop geotechnical recommendations for project design, specification development, and construction. Kleinfelder's understanding of the project is based on the Geotechnical Investigation

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Request dated November 24, 2015 provided by Mr. Alex Chung of PG&E. Our scope of work includes the following:

- Review existing publically available geotechnical information in the vicinity of the project site.
- Field exploration including three Cone Penetration Tests (CPTs) to depths of up to 45 feet to explore subsurface conditions.
- Laboratory testing of samples retrieved from the upper 5 feet to evaluate relevant geotechnical engineering parameters of the subsurface soils including corrosion potential.
- Analyses of the field and laboratory data to develop conclusions and recommendations to guide the geotechnical design and construction of the project.
- Preparation of this report.

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

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2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

2.1.1 Cone Penetration Tests

Prior to subsurface exploration, site-specific health and safety plan was prepared for the field exploration activities. This plan was discussed with the filed crew prior to the start of field exploration work. The area of explorations was marked and Underground Service Alert (USA) was contacted to provide utility clearance in the public right-of-way. In addition, the upper 5 feet of the exploration locations were cleared using hand auger methods to confirm the absence of buried utilities.

The locations of the explorations are shown on Figure 2, Site Plan. Explorations were located in the field by measuring from existing landmarks. Horizontal coordinates and elevations of the explorations were not surveyed.

On December 28, 2015 three CPT soundings (CPT-01, CPT-02, and CPT-03) were advanced to depths of about 45 feet below the ground surface. The CPT soundings were performed by Middle Earth Geo Testing, Inc. of Hayward, California using a 25-ton, truck-mounted, International Paystar 5000 CPT rig.

The CPT soundings were performed in general accordance with ASTM D5778 using an electronic cone penetrometer. A set of hydraulic rams were used to continuously push the cone and rods into the soil at a rate of approximately 2 centimeters per second while the cone tip resistance (Qt) and sleeve friction resistance (Fs) were recorded in 2-centimeter increments.

The cone penetration assembly used consists of a conical tip and a cylindrical friction sleeve. The conical tip has a 60-degree apex angle and approximately 3.6 centimeter diameter. The cylindrical friction sleeve has a surface area of 10 square centimeters. The CPT data (cone tip resistance, sleeve friction resistance, friction ratio, and equivalent Standard Penetration Test blow counts) versus penetration depth below the existing ground surface are presented on the CPT logs in Appendix A.

The stratigraphic interpretation of the CPT data was performed based on relationships between cone tip resistance and sleeve friction resistance versus penetration depth. The friction ratio, which is sleeve friction resistance divided by cone tip resistance, is a calculated parameter which

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is used to infer soil behavior type. Cohesive soils (clays) generally have high friction ratios, low cone tip resistance values, and generate large excess pore water pressures. Cohesionless soils (sands) generally have lower friction ratios, high cone tip resistance values, and generate small excess pore water pressures. The interpretation of soil behavior type from the cone data was carried out based on Robertson and Cabal (2012) and Robertson (2009). It should be noted that it is not always possible to clearly identify a soil type based on cone tip resistance and sleeve friction resistance. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.

After the CPTs were completed they were backfilled with neat cement grout.

2.1.2 Sampling Procedures

Near-surface bulk samples were obtained from hand-auger cuttings in the upper 5 feet of the CPT explorations. Soil samples were packaged and sealed in the field to reduce moisture loss. Following the field exploration, the samples were returned to our laboratory for further examination and testing.

2.2 LABORATORY TESTING

Kleinfelder performed laboratory tests on samples retrieved from the upper five feet to evaluate their physical and engineering characteristics. The following laboratory tests were performed:

- Atterberg Limits (ASTM D4318)
- Corrosion - Soluble Sulfate Content (ASTM D4327)
- Corrosion - Soluble Chloride Content (ASTM D4327)
- Corrosion - pH (ASTM D4972)
- Corrosion - Minimum Resistivity (ASTM G57)
- Corrosion - Redox (ASTM D1498)
- Corrosion - Sulfide (ASTM D4658)

The results of the laboratory tests and the test data are included in Appendix B.

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3 GEOLOGIC CONDITIONS

3.1 SITE GEOLOGY

The site is located within Santa Clara Valley, which lies between the Santa Cruz Mountains to the southwest and the Diablo Range to the northeast. The Santa Clara Valley extends about 70 miles to the southeast from San Francisco and is filled with over 1,500 feet of Quaternary and late Tertiary age alluvial sediments deposited by the many creeks that flow from the bordering hills and mountains.

The project site has been mapped by the California Geological Survey as part of their Seismic Hazard Zonation Program in a Seismic Hazard Zonation Report for the Santa Teresa Hills Quadrangle (CGS, 2003). The report includes borehole log information relating to subsurface geology and engineering characteristics compiled from unpublished consultants' geotechnical and environmental reports and filed with the California Department of Transportation and the City of San Jose Department of Public Works. Based geologic mapping within the CGS (2003) report, the planned substation is underlain by Holocene Age Alluvial Fan Levee deposits which are generally comprised of interbedded clay, silt, and silty sand, and are considered to have a low liquefaction potential when groundwater is deeper than 30 feet. In this area, the Alluvial Fan Levee deposits are associated with the Coyote Creek, located about a mile north of the site (see Figure 1).

3.2 LOCAL AND REGIONAL FAULTING

In the City of San Jose, faults are mapped or zoned by three agencies where site-specific studies addressing the potential for surface fault rupture are required; the City of San Jose (City), Santa Clara County (County), and as Alquist-Priolo Fault Zones (AP Zones) by the State of California (State). The substation is not located within a State, County, or City designated fault zone, and no known active faults traverse the alignment. The nearest faults to the project site are the Coyote Creek fault zone (located about 2 miles to the east), the Hayward Fault zone (located about 4 miles to the northeast), and the Calaveras fault zone (located about 8 miles to the east). Other faults that may contribute to ground shaking at the site include the Sargent fault zone (located about 10 miles to the southwest) and the San Andreas fault zone (located about 11 miles to the southwest).

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4 SITE CONDITIONS

4.1 SITE DESCRIPTION

The project site is located in a mixed industrial, commercial, and residential area in southern San Jose. The PG&E Santa Teresa Substation is planned to be located behind the existing PG&E Edenvale Service Center (see Figure 1) in a relatively flat lot that is currently used as a PG&E training facility and storage yard. The ground surface includes bare earth, gravel paving, and asphalt-concrete paving. Existing structures within the footprint of the planned substation include wood poles used for training, and a three-sided masonry block storage structure. The site is bordered to the east by the Santa Teresa light rail station, to the north by Highway 85, to the west by Miyuki Drive, and to the south by the PG&E Edenvale Service Center.

4.2 SUBSURFACE CONDITIONS

The following description provides a general summary of the subsurface conditions encountered during this study. Soil descriptions for the upper 5 feet of the subsurface profile are based on observation of the hand auger cuttings. Soil descriptions below 5 feet are based on inferred soil behavior type, as discussed in Section 2, above. For more thorough representations of the actual conditions encountered at specific locations, refer to the hand auger and CPT logs in Appendix A. Note that due to the hand augering process, the upper 5 feet of the CPT log should be disregarded.

Our field exploration generally encountered up to 1 foot of gravel and gravelly lean clay fill, underlain by moist, dark reddish brown to dark brown, hard, lean clay within the hand augered upper 5 feet. The soil behavior types inferred by the CPTs consist of firm to hard lean clay with varying amounts of sand and silt from about 5 feet to the depth explored (45 feet), except for a layer of silty sand and sandy silt encountered between depths of about 22 to 27 feet. The subsurface conditions encountered in our CPTs are in general agreement with the mapped geology and borehole log data available from the CGS (2003).

4.3 GROUNDWATER

Groundwater was not encountered within the CPTs performed for our field exploration, based on pore water dissipation measurements. According to publicly available well data published online by the State Water Resources Control Board (www.water.ca.gov) for two nearby wells, one

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located 700 feet west and the other 1,200 feet southwest from the site, previous groundwater levels trend from a high of about 30 feet in 2012 to about 70 feet in late 2015. This is in general agreement with groundwater level mapping by the CGS (2003) at the project site at depths greater than 30 feet below the ground surface.

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

4.4 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions at the site are based on the conditions encountered in the CPTs performed for this project, with confirmation from publically available borehole data from the CGS (2003). 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.

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5 CONCLUSIONS AND RECOMMENDATIONS

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

5.1 LIQUEFACTION

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

In the past decade, several concentrated efforts have been undertaken to establish a uniform guideline for field-based simplified liquefaction analyses. Youd et al. (2001) published general guidelines for liquefaction analyses, which presented the consensus of a task committee. However, subsequent earthquakes provided additional data to researchers, especially for low plasticity clays and silts, which resulted in significant modifications to liquefaction evaluation methods, especially for soils with higher fines contents. Liquefaction triggering analyses were performed using the methods proposed by Youd et al. (2001) and Idriss and Boulanger (2008) using the information obtained from CPT-01 advanced as part of this study. In order to perform liquefaction analysis, estimates of earthquake magnitude and peak ground acceleration (PGA_M) are needed. Using the U.S. Geological Survey (USGS) interactive deaggregation website, the modal earthquake magnitude M_W = 6.6 was estimated and used in the analysis. Peak ground acceleration (PGA_M) value for our analyses was calculated based on Equation 11.8-1 in Section 11.8.3 of the American Society of Civil Engineers (ASCE) 7-10 for the Risk-Targeted Maximum Considered Earthquake (MCE_R). The PGA_M value was calculated using the US Seismic Design Maps application assuming a Site Class D. The calculated PGA_M value is 0.584g for the MCE_R.

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The evaluation of liquefaction in response to an earthquake is based on a comparison of a soil's resistance to liquefaction and the cyclic load or demand placed on the soil by the earthquake. A safety factor against liquefaction is commonly defined as the ratio of the cyclic shear stress required to cause liquefaction (cyclic resistance ratio, or CRR) to the equivalent cyclic shear stress induced by the earthquake (cyclic stress ratio, or CSR). Per CGS Special Publication 117A (CGS, 2008), if the calculated safety factor against liquefaction (i.e., the ratio CRR/CSR) is less than about 1.3 the soil is considered to be liquefiable for design purposes. Liquefaction-induced settlements were estimated using Youd et al. (2001) method and the Idriss and Boulanger (2008) method.

Based on publically available well data, design groundwater level for liquefaction analysis was selected at 30 feet below existing grade.

Based on the depth to design groundwater and the soil conditions encountered in our field investigation, the potential for liquefaction-related settlement is considered negligible.

5.2 EXPANSIVE SOILS

The surficial soils have a low expansion potential based on the results of an Atterberg limits test (Liquid Limit of 29 and Plasticity Index of 11) performed on a near-surface sample of lean clay taken from the upper 5 feet of CPT-01. Expansive soils are not anticipated to be a concern for the planned foundations.

5.3 2013 CBC SEISMIC DESIGN PARAMETERS

For a 2013 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1), associated soil amplification factors (F_a and F_v), and mapped peak ground acceleration (PGA) are presented in Table 5-1. Corresponding site modified (S_{MS} and S_{M1}) and design (S_{DS} and S_{D1}) spectral accelerations, PGA modification coefficient (F_{PGA}), PGA_M, risk coefficients (C_{RS} and C_{R1}), and long-period transition period (T_L) are also presented in Table 5-1. Presented values were estimated using Section 1613.3 of the 2013 California Building Code (CBC), chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps.

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Table 5-1
Ground Motion Parameters Based on 2013 CBC

Parameter	Value	Reference
S_S	1.536g	2013 CBC Section 1613.3.1
S_1	0.600g	2013 CBC Section 1613.3.1
Site Class	D	2013 CBC Section 1613.3.2
F_a	1.000	2013 CBC Table 1613.3.3(1)
F_v	1.500	2013 CBC Table 1613.3.3(2)
PGA	0.584	ASCE 7-10 Figure 22-7
S_{MS}	1.536g	2013 CBC Section 1613.3.3
S_{M1}	0.900g	2013 CBC Section 1613.3.3
S_{DS}	1.024g	2013 CBC Section 1613.4.4
S_{D1}	0.600g	2013 CBC Section 1613.4.4
F_{PGA}	1.000	ASCE 7-10 Table 11.8-1
PGA_M	0.584g	ASCE 7-10 Section 11.8.3
C_{RS}	1.111	ASCE 7-10 Figure 22-17
C_{R1}	1.065	ASCE 7-10 Figure 22-18
T_L	12 seconds	ASCE 7-10 Figure 22-12

5.4 SITE PREPARATION

5.4.1 General

It is anticipated that site grading can be performed with conventional grading equipment and techniques. Site grading should be performed with drainage toward the north. General recommendations for site preparation and earthwork construction are presented in the following sections of this report. All references to compaction, maximum density and optimum moisture content are based on ASTM D1557, unless otherwise noted.

5.4.2 Stripping and Grubbing

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

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power poles and masonry block storage structures should be removed and the subsurface excavated to expose firm soil.

5.4.3 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

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

5.4.4 Scarification and Compaction

In areas requiring placement of fill, it is recommended the fill be placed and compacted as engineered fill. Following site stripping and any required grubbing and/or over-excavation, it is recommended areas to receive engineered fill be scarified to a depth of 8 inches, uniformly moisture conditioned to at least 2 percent above the optimum moisture content and compacted to at least 90 percent relative compaction.

If undocumented fill is encountered, the fill should be over-excavated and replaced as engineered fill compacted as recommended below in the “Engineered Fill- Compaction Criteria” section of this report.

5.5 ENGINEERED FILL

5.5.1 Onsite Materials

The excavated on-site soil is anticipated to consist of lean clay. The on-site soil is suitable for use as engineered fill, provided it is free of debris, significant organics or other deleterious materials, and has a maximum particle size less than 3 inches in maximum dimension. Where imported material is brought in for engineered and “non-expansive” fill, it is recommended that it be granular in nature and conform to the minimum criteria discussed in Section 5.5.2.

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5.5.2 Imported and Non-Expansive Fill Requirements

In addition to the above requirements, specific requirements for imported engineered fill and non-expansive engineered fill as well as applicable test procedures to verify material suitability are provided in Table 5-2.

Table 5-2
Imported and Non-Expansive Engineered Fill Requirements

Fill Requirement		**Test Procedures**	
Gradation		**ASTM[1]**	**Caltrans[2]**
Sieve Size	**Percent Passing**		
3 inch	100	D422	202
¾ inch	70-100	D422	202
No. 200	20-50	D422	202
Liquid Limit	Plasticity Index		
<30	<12	D4318	204
Organic Content			
No visible organics		---	---
Expansion Potential			---
20 or less		D4829	---
Soluble Sulfates			
Less than 2,000 ppm		---	417
Soluble Chloride			
Less than 300 ppm		---	422
Resistivity			
Greater than 2,000 ohm-cm		---	643

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

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

In addition, we recommend that a laboratory corrosion test series (pH, resistivity, redox, sulfides, chlorides, and sulfates) be performed on all proposed import materials. The corrosivity of proposed import materials should be evaluated and should be no more corrosive than the on-site soils as indicated by the laboratory results presented in this report.

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5.5.3 Placement and Compaction Criteria

Imported non-expansive soils that meet the criteria outlined in Table 5-2 that are to be used for engineered fill should be uniformly moisture conditioned to at least the optimum moisture content, placed in horizontal lifts less than about 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. Onsite clayey soils to be used for general fill should be uniformly moisture conditioned to at least 2 percent over the optimum moisture content, placed in horizontal lifts less than about 8 inches in loose thickness, and compacted to between at least 90 percent relative compaction.

5.6 WET WEATHER CONSIDERATIONS

Should construction be performed during or subsequent to wet weather, near-surface site soils may be significantly above the optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact site soils to the recommended compaction criteria. Disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile fabric or geogrid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork and construction operations.

5.7 SITE DRAINAGE

Final site grading should provide surface drainage away from all structures and areas to be traversed by vehicles and maintenance equipment. In general, we recommend consideration be given to providing at least 1 to 2 percent slope away from structure foundations or access ways.

5.8 TEMPORARY EXCAVATIONS

5.8.1 General

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

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5.8.2 Excavation and Slopes

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

5.9 TRENCH BACKFILL

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

5.10 SHALLOW FOUNDATION DESIGN

Foundations should satisfy two independent criteria with respect to foundation soils. First, the foundation should have an adequate safety factor against bearing failure with respect to the shear strength of the foundation soils. Second, the vertical movements of the foundation due to settlement (both immediate elastic settlement and consolidation settlement) and/or heave should be within tolerable limits for the structure.

Lightly-loaded structures may be supported on conventional, shallow, reinforced concrete mat foundations or continuous footings, provided the site structures can tolerate the anticipated settlement. Recommendations for foundation design are presented below.

5.10.1 Spread Footings

5.10.1.1 Allowable Bearing Pressure

Shallow spread footings constructed of reinforced concrete may be founded on approved undisturbed native soil and/or engineered fill. The footings should be founded at least 18 inches below lowest adjacent finished grade on subgrade soils that have been prepared in accordance with the recommendations provided in this report. Continuous footings should have a minimum width of 12 inches, and isolated rectangular footings should have a minimum width of 24 inches.

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Depending on the settlement tolerances of the planned structures, spread footings may be designed for a net allowable bearing pressure of up to 2,000 pounds per square foot (psf) due to dead plus live loads.

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

To maintain the desired support, foundations adjacent to utility trenches or other existing foundations should be deepened so that their bearing surfaces are below an imaginary plane having an inclination of 2 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent foundations or utility trenches.

5.10.1.2 Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.30 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 300 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches of soil should be neglected unless the area in front of the footing is protected from disturbance by concrete or pavement. The allowable friction coefficient and passive resistance may be used concurrently.

5.10.1.3 Settlement and Heave

Total settlement and/or heave of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on anticipated foundation dimensions and loads, we estimate maximum total settlement of foundations designed and constructed in accordance with the preceding recommendations to be on the order of 1 inch or less. Differential settlement between similarly loaded, adjacent footings is estimated to be about half the total settlement. Ground heave due to expansive soil effects is not anticipated at the project site.

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5.10.1.4 Spread Footing Construction Considerations

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

5.10.2 Mat Foundations

5.10.2.1 Subgrade Preparation

We recommend that the bottoms of excavations to receive mat slabs be scarified to a depth of at least 8 inches, uniformly moisture conditioned to at least 2 percent above the optimum moisture content, and compacted to at least 90 percent relative compaction. These excavations should be covered as soon as possible and/or be wetted periodically so the soils are not allowed to dry out prior to concrete placement.

Beneath cast-in-place concrete mat foundations, we recommend the design include a base course of well-graded crushed aggregate (such as Caltrans Class 2 Aggregate Base) at least 4 inches thick. Under slabs that will be subjected to vehicle loading, the aggregate base course thickness should be increased to a minimum of 6 inches. The base course should be compacted to at least 95 percent relative compaction at a moisture content slightly above optimum. Thickened slab edges should engage the building pad soil and should not be underlain by the gravel base course.

5.10.2.2 Allowable Bearing Pressure

For subgrades prepared as recommended in this report, reinforced concrete mat foundations may be designed for a net allowable bearing pressure of up to 1,500 psf. The allowable bearing pressure applies to dead plus live loads, includes a safety factor of at least 3 with respect to shear failure of the foundation soils, and may be increased by one-third for short-term loading due to wind or seismic forces.

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5.10.2.3 Lateral Load Resistance

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.30 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 300 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches should be neglected unless the area in front of the foundation is protected from disturbance by concrete or pavement. The friction coefficient and passive resistance may be used concurrently.

5.10.2.4 Subgrade Modulus

An allowable modulus of subgrade reaction, K_{v1}, of 75 pounds per square inch per inch of deflection (for a 1 square-foot bearing plate) may be used for design in the on-site lean clay soils. The modulus should be adjusted for the actual slab size using appropriate formulas or software.

5.10.2.5 Settlement

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. For smaller mat slabs supporting electrical equipment (generally about 8 feet wide), total settlement is anticipated to be less than about ½ inch. For the larger transformer mat slabs, we estimate maximum total settlement of the foundations with dimensions up to 15 by 30 feet that are designed and constructed in accordance with the preceding recommendations to be on the order of 1 inch or less. Differential settlement between the center and corners of the mat is estimated to be about ⅔ of the total settlement.

5.10.2.6 Mat Foundation Construction Considerations

Underground utilities that are 4 feet deep or shallower and that run parallel to shallow mat foundations generally should be located no closer than 2 feet horizontally away from the perimeter edges of the slab. Deeper utilities should be located above a 1H:1V (horizontal to vertical) slope projected downward from the bottom edges of the slab. Utility plans should be reviewed by Kleinfelder prior to trenching to evaluate conformance with this requirement. Turned down or thickened edges at least 12 inches deep should be used to provide perimeter confinement and reduce the potential for water infiltration beneath the slabs.

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5.11 DRILLED PIER FOUNDATIONS

It is our understanding that the dead end structures and other possible additional structures will be founded on reinforced concrete drilled pier foundations. Recommendations for design and construction of drilled pier foundations are presented in the following sections of this report.

5.11.1 Axial Capacity

Axial loads on drilled piers should be supported by skin friction. End bearing is not considered in the axial capacity due to strain incompatibility issues between skin friction and end bearing, settlement issues, and the potential for loose materials to exist at the bottoms of the pier holes during construction that cannot be effectively cleaned out. The downward vertical movement associated with mobilizing the full end-bearing resistance of drilled piers is frequently beyond structural tolerances and normally well beyond the movement required to engage side friction.

A curve illustrating the ultimate axial compressive capacity of a unit (1-foot) diameter straight-sided drilled pier installed from existing grade is shown on Figures 3a and 3b. Axial capacity was computed using Federal Highway Administration (FHWA) procedures for design of drilled pier foundations (Brown et al., 2010). Skin friction capacity in the upper 2 feet of soil was neglected in our analyses. Ultimate tensile capacity may be obtained by multiplying the compressive capacity by a factor of 0.8 and adding the weight of the foundation. For evaluation of allowable axial capacity under static conditions, we recommend a factor of safety of 3 be applied to the ultimate capacity (per the General Order 95 code). A one-third increase in the allowable capacity may be used for consideration of transient loads such as wind or seismic.

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 3a and 3b. The curve is applicable for drilled pier foundations up to 7 feet in diameter that are spaced at least 3 diameters apart. For closer spacings, group effects may govern and the group capacity should be evaluated.

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5.11.1.1 Estimated Settlement

Based on the methods outlined by Brown et al. (2010), total static settlement of each drilled pier should be on the order of 0.1 percent of the pier diameter for a drilled pier designed and constructed in accordance with the recommendations presented in this report. This value includes elastic compression of the pile under design loads. The majority of the settlement should occur during and shortly after application of the structure loads.

5.11.2 Lateral Response

5.11.2.1 LPILE Analysis Soil Parameters

Lateral capacity of deep foundations may be developed through analysis of pier or pile response due to a range of design loads. Table 5-3 contains recommended input soil parameters for lateral response analysis of deep foundations using the LPILE computer program (by Ensoft, Inc., Version 2013). Program default values may be used for strain factor (E_{50}) and horizontal subgrade reaction (K).

Table 5-3
LPILE Geotechnical Parameters

Depth (feet)	Model P-Y Curve	Effective Unit Weight (lb./ft^3)	Cohesion, c (psf)	Internal Friction Angle, Φ (degrees)
0 to 7	Stiff Clay without Free Water	120	1,100	--
7 to 27	Stiff Clay without Free Water	115	1,800	--
27 to 30	Stiff Clay without Free Water	115	1,500	--
30 to 45	Stiff Clay without Free Water	53	1,500	--

LPILE analyses were performed for the various drilled pier dead end foundations using the LPILE version 2013 computer program (by Ensoft, Inc.). The results were also used to evaluate the Canedo Q value for the site, as discussed below. The results of our LPILE analyses are presented in Appendix C.

5.11.2.2 Canedo "Q" Value

PG&E provided loading information (Shear = 2.5 kips, Vertical = 1.5 kips, and Moment 53 ft.-kips) and dimensions (diameter = 42 inches and depth = 8.5 feet) for the planned dead end structures at the project site for use in our LPILE analysis and Canedo Q calculations for drilled piers.

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Kleinfelder evaluated the Canedo "Q" value for design of drilled pier foundations using the PG&E "Steel Structures, Equipment Anchorages and Foundations" substation design criteria dated February, 2013. To determine the Canedo "Q" value for pier design, an LPILE analysis was conducted to evaluate the required embedment depth for the pile loading and diameter case presented above.

Based on our analysis, the computed pile head deflection for planned depth of 8.5 feet is less than 2 percent of the pier diameter, and the computed pier head rotation is less than 0.5 degree. Therefore, the performance of the proposed drilled piers meet the current performance criteria established by PG&E under the given loads.

The recommended Canedo Q value to be used in design is 950 psf/ft for the case described above. A minimum embedment depth of 7 feet is required for a 42 inch diameter drilled pier for the dead-end structure to stay within pier head rotation limits. The results of the LPILE analysis are presented in Appendix C.

5.11.2.3 Lateral Load Resistance

For smaller piers such as those used for fences, when designing drilled piers using CBC Section 1807.3.2.1, lateral loads may be resisted by passive pressure acting against the pier sides and using twice the diameter of the pier, provided they are spaced at least 5 diameters apart (center to center) and a ½ inch deflection at the ground line is tolerable. We recommend an allowable passive pressure value of 160 psf per foot of depth (to be applied over a 2 pier diameter width) for CBC design. The lateral soil bearing pressure should not exceed 1,920 psf.

5.11.2.4 Lateral Response Group Effects

Where drilled piers are spaced at least 8 diameters center-to-center perpendicular to the direction of load, the piers may be assumed to act as individual elements and no additional group action lateral resistance reduction factor is needed. Lateral resistance of each individual pier within a group should be reduced by the factors provided in Table 5-4 below to account for group action effects where the spacing is less than 8 pier diameters. We anticipate piers will be spaced at least 3 diameters center-to-center.

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As a general guide, Table 5-4 below may be used to estimate group efficiency factors for a range of pier group configurations and spacings. For group configurations with center-to-center spacings closer than 3 diameters, Kleinfelder should be consulted to evaluate group efficiency on a case-by case basis.

Table 5-4
Recommended Pier Group Efficiency Factors

Groups Having 2 Rows or More Aligned in The Direction of Load				
Center-to-Center Pier Spacing (diameters)	**Front Row, Outer**	**Front Row, Inner**	**Trailing Rows, Outer**	**Trailing Rows, Inner**
3	0.7	0.5	0.4	0.3
4	0.8	0.6	0.5	0.4
5	0.9	0.7	0.6	0.5
6	1.0	0.8	0.7	0.6
7	1.0	0.9	0.9	0.8
8	1.0	1.0	1.0	1.0

Groups Having 1 Row Aligned in The Direction of Load		
Center-to-Center Pier Spacing (diameters)	**Front Pier**	**Trailing Pier**
3	1.0	0.6
4	1.0	0.7
5	1.0	0.8
6	1.0	0.9
7	1.0	1.0
8	1.0	1.0

Group effects should be also be considered where new foundations are constructed adjacent to existing foundations. Once the pier group configurations and loads are established, Kleinfelder would be pleased to review the design and comment on group effects.

5.11.3 Drilled Pier Construction Considerations

The onsite clay soils appear to be favorable for drilled pier construction. However, interbedded silty soils may exist that could be prone to some caving. Caving of the surficial gravels around the pier holes is likely and should be mitigated during construction.

Consistent with the 2013 CBC, drilled pier excavations should be inspected and approved by the geotechnical engineer prior to installation of reinforcement. The depths of all pier excavations should be checked immediately prior to concrete placement to verify excessive sloughing and/or caving has not reduced the required hole depth. This may be done with a weighted tape measure or similar measuring device.

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

Concrete used for drilled pier construction should be discharged vertically into the drilled holes to reduce aggregate segregation. Under no circumstances during pier construction should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation. Sufficient space should be provided in the pier reinforcement cage during fabrication to allow the insertion of a pump hose or tremie tube for concrete placement. The pier reinforcement cage should be installed and the concrete pumped immediately after drilling is completed.

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

If slurry drilling methods are used for drilled pier construction, concrete should be placed into the hole using tremie methods. Tremie concrete placement should be performed in accordance with American Concrete Institute (ACI) 304R. The tremie pipe should be rigid and remain several feet below the surface of the in-place concrete at all times to maintain a seal between the water or slurry and the fresh concrete. The upper concrete seal layer will likely become contaminated with excess water and/or soil as the concrete is placed and should be removed to expose uncontaminated concrete during or immediately following completion of concrete placement. It has been our experience that the concrete seal layer may be on the order of 3 to 5 feet thick but will depend on the pile diameter, amount of water seepage, and construction workmanship.

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

5.11.4 Temporary Casing

If temporary straight-sided steel casing is used for conventional drilled pier construction, 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.

5.12 SOIL CORROSION

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

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

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Table 5-5
Chemistry Laboratory Test Results

Boring and Depth	Material	Resistivity, ohm-cm	pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
					Chloride	Sulfide	Sulfate
CPT-02 at 3 ft. and 5 ft.	Lean Clay	3,100	7.44	420	N.D.*	N.D.*	22.0

*N.D. - None Detected

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

Based on the “10-point” method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the soils at the site are not anticipated to be corrosive to buried ferrous metal piping, cast iron pipes, or other objects made of these materials. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures, if required.

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication “Guide to Durable Concrete” (ACI 201.2R-08) provides guidelines for this assessment. The sample had a sulfate concentration of 22.0 parts per million (ppm). The results of sulfate test indicate the potential for deterioration of concrete is mild, no special requirements should be necessary for the concrete mix.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater. Chloride tests did not detect the presence of chloride in the sample. The project structural engineer should review this data to determine if remedial measures are necessary for the concrete reinforcing steel.

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6 ADDITIONAL SERVICES

6.1 PLANS AND SPECIFICATIONS REVIEW

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

6.2 CONSTRUCTION OBSERVATION AND TESTING

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

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7 LIMITATIONS

This report presents information for planning, permitting, design, and construction of the switchgears, transformers, circuit breakers, dead end structures, and control building at the new Santa Teresa Substation in San Jose, California. Recommendations contained in this report are based on materials encountered in the field explorations performed for this investigation (CPT-01, CPT-02, and CPT-03), geologic interpretation based on published articles and geotechnical data, and our present knowledge of the proposed construction.

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

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

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on-site and off-site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

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FIGURES

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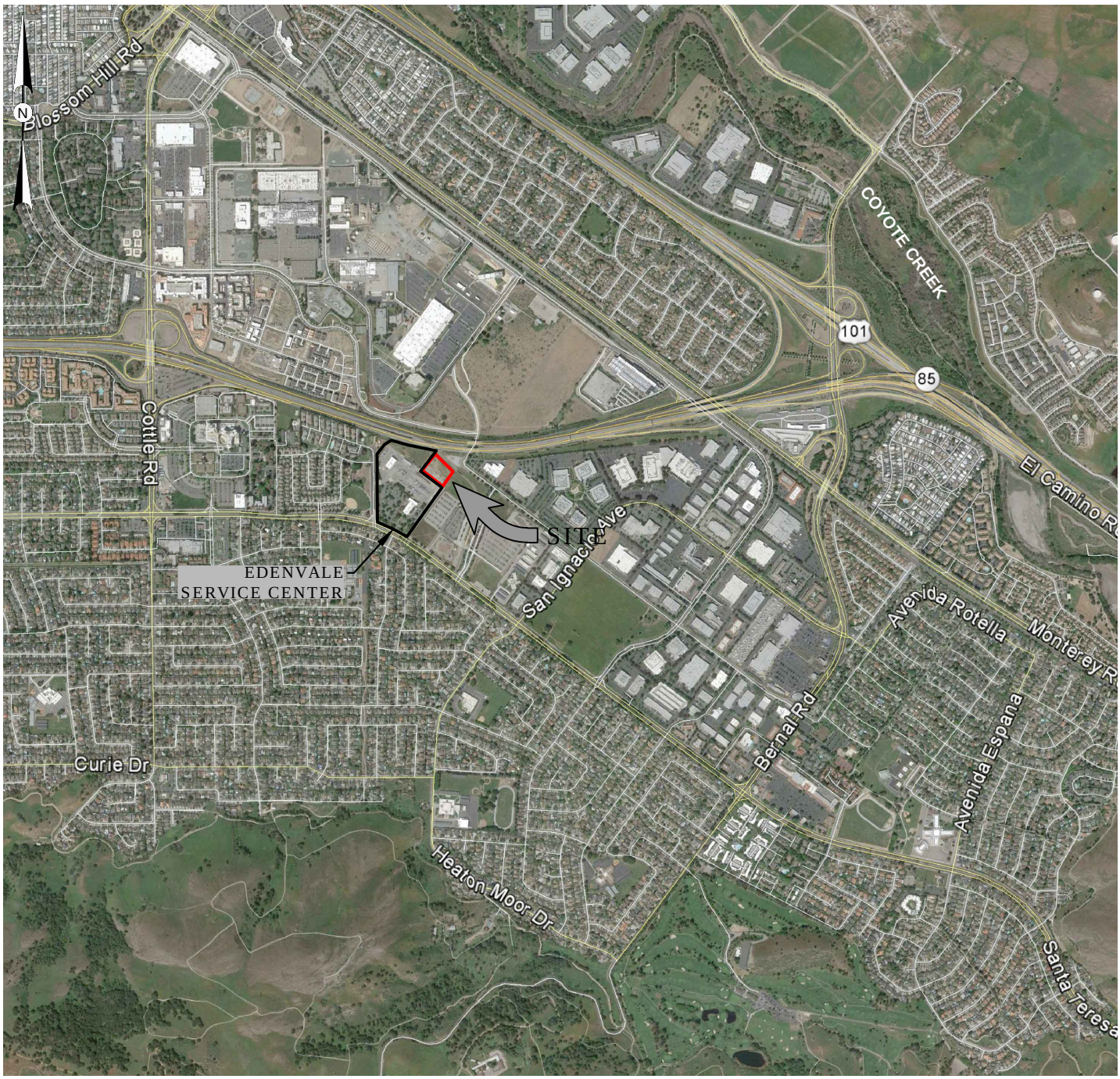

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CHECKED BY:	LS
DATE:	01-19-2016
REVISED:	

SITE VICINITY MAP

PG&E SANTA TERESA SUBSTATION
6402 SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

FIGURE

1

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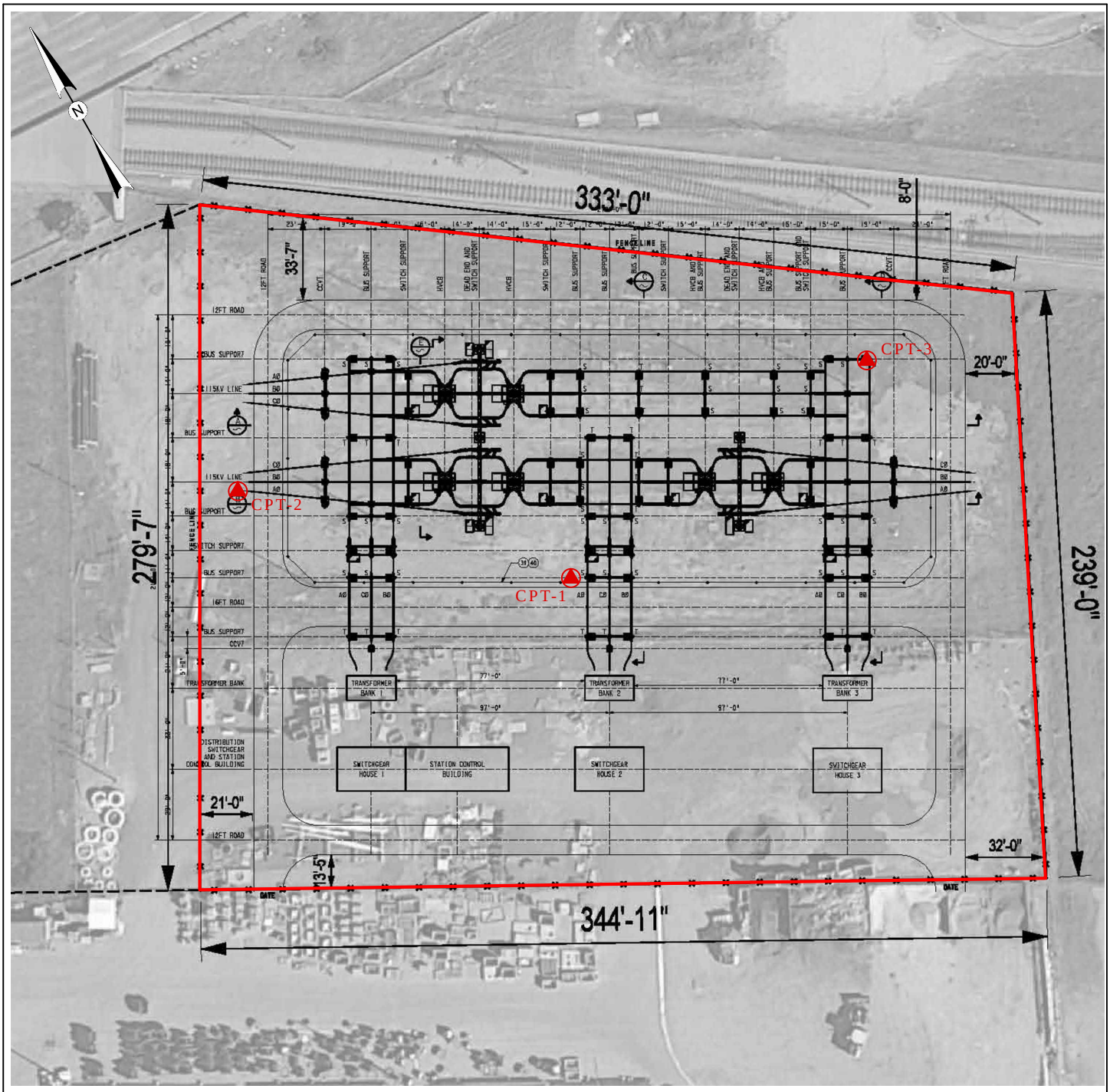

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PG&E Santa Teresa Substation General
Arrangement Outdoors, sheet No. 4106123, no date.

LEGEND

SITE FENCE

CPT-1 CONE PENETRATION TEST (By Kleinfelder, 12/2015)

NOTE: Locations are approximate.

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SITE PLAN

PG&E SANTA TERESA SUBSTATION
6402 SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

FIGURE

2

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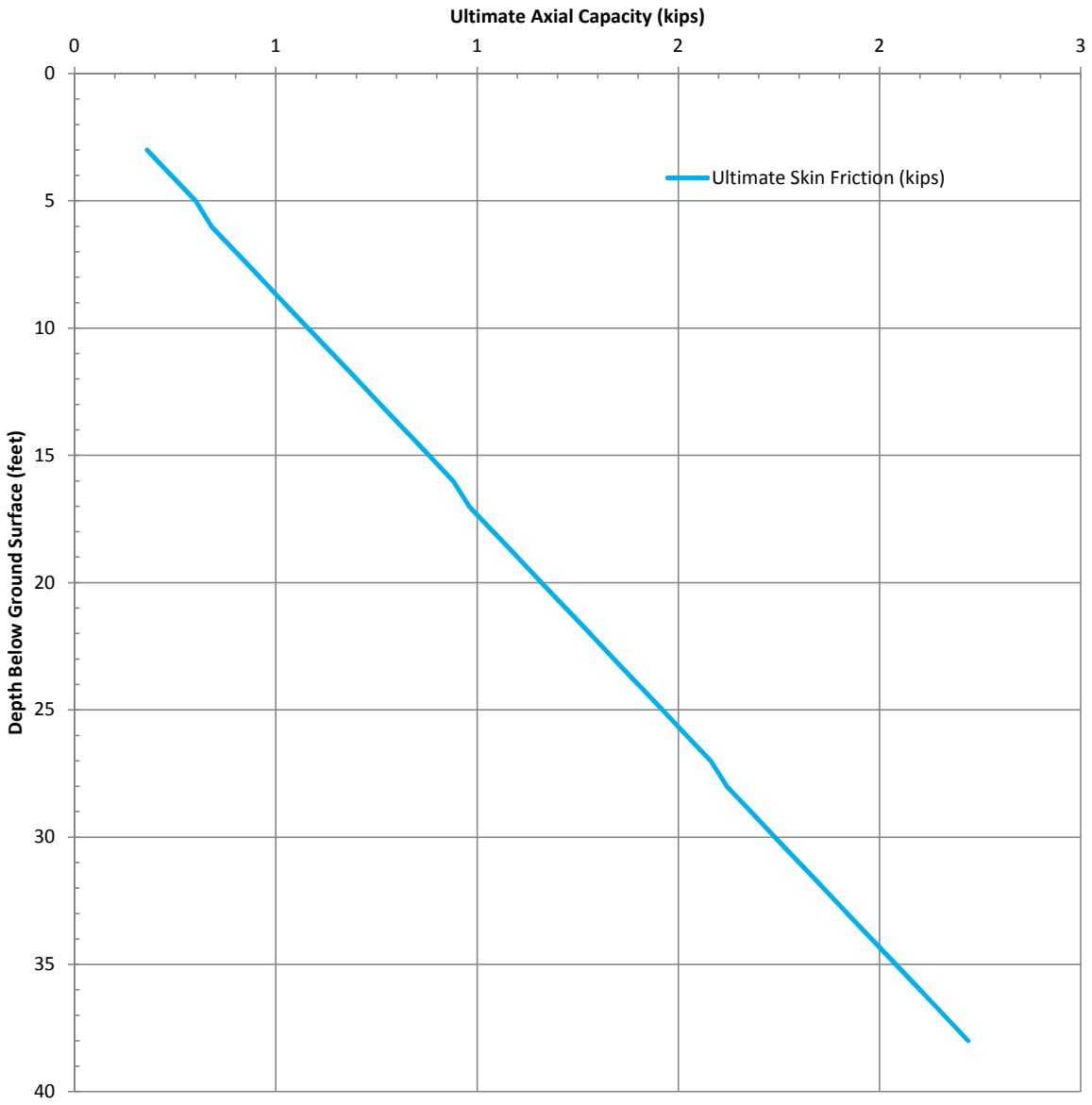

Notes:

1. Axial capacities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the 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.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20163632 DRAWN BY: BP CHECKED BY: 0 DATE: 1/19/2015 REVISED: 0	ULTIMATE AXIAL CAPACITY UNIT DIAMETER (1-FOOT) DRILLED PIER PG&E SANTA TERESA SUBSTATION 6402 SANTA TERESA BOULEVARD SAN JOSE, CALIFORNIA	FIGURE 3a

Depth (ft)	Ultimate Axial Capacity (kips)	Depth (ft)	Ulitimate Axial Capacity (kips)
3	0.18	23	1.34
4	0.24	24	1.4
5	0.3	25	1.46
6	0.34	26	1.52
7	0.4	27	1.58
8	0.46	28	1.62
9	0.52	29	1.68
10	0.58	30	1.74
11	0.64	31	1.8
12	0.7	32	1.86
13	0.76	33	1.92
14	0.82	34	1.98
15	0.88	35	2.04
16	0.94	36	2.1
17	0.98	37	2.16
18	1.04	38	2.22
19	1.1	39	2.26
20	1.16	40	2.32
21	1.22	41	2.38
22	1.28	42	2.44

Notes:

1. Axial capcities of drilled piers with diameters other than one foot may be obtained by multiplying the unit capacity by the diameter of the 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 pile. See text discussion for factor of safety and group effects.

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PROJECT NO.:	20163632	ULTIMATE AXIAL CAPACITY UNIT DIAMETER (1-FOOT) DRILLED PIER	FIGURE
DRAWN BY:	BP		
CHECKED BY:	0	PG&E SANTA TERESA SUBSTATION 6402 SANTA TERESA BOULEVARD SAN JOSE, CALIFORNIA	3b
DATE:	1/19/2015		
REVISED:	0		

APPENDIX A

FIELD EXPLORATION - CPT LOGS

Table A-1
Log of Hand Auger, CPT-1

Depth (feet)	Subsurface Description
0 to 5	Lean Clay (CL), dark reddish brown, moist, hard, low plasticity, trace fine sand, with grass and roots to about 1 inch. Changes to dark brown about 4 feet.

Table A-2
Log of Hand Auger, CPT-2

Depth (feet)	Subsurface Description
0 to ½	Poorly Graded Gravel (GP), Gray, moist, max diameter about ¾ inch (FILL)
½ to 1	Gravelly Lean Clay (CL), dark reddish brown, moist, hard, low plasticity (FILL)
1 to 5	Lean Clay (CL), dark reddish brown, moist, hard, low plasticity, trace fine sand

Table A-3
Log of Hand Auger, CPT-3

Depth (feet)	Subsurface Description
0 to ¼	Poorly Graded Gravel (GP), Gray, moist, max diameter about ¾ inch (FILL)
¼ to ½	Gravelly Lean Clay (CL), dark reddish brown, moist, hard, low plasticity (FILL)
½ to 5	Lean Clay (CL), dark reddish brown, moist, hard, low plasticity, trace fine sand Changes to brown at about 3.5 feet.

Note that due to the hand augering process, the upper 5 feet of the CPT log should be disregarded.

Kleinfelder Inc

Project	6402 Santa Teresa	Operator	CB	Filename	SDF(367).cpt
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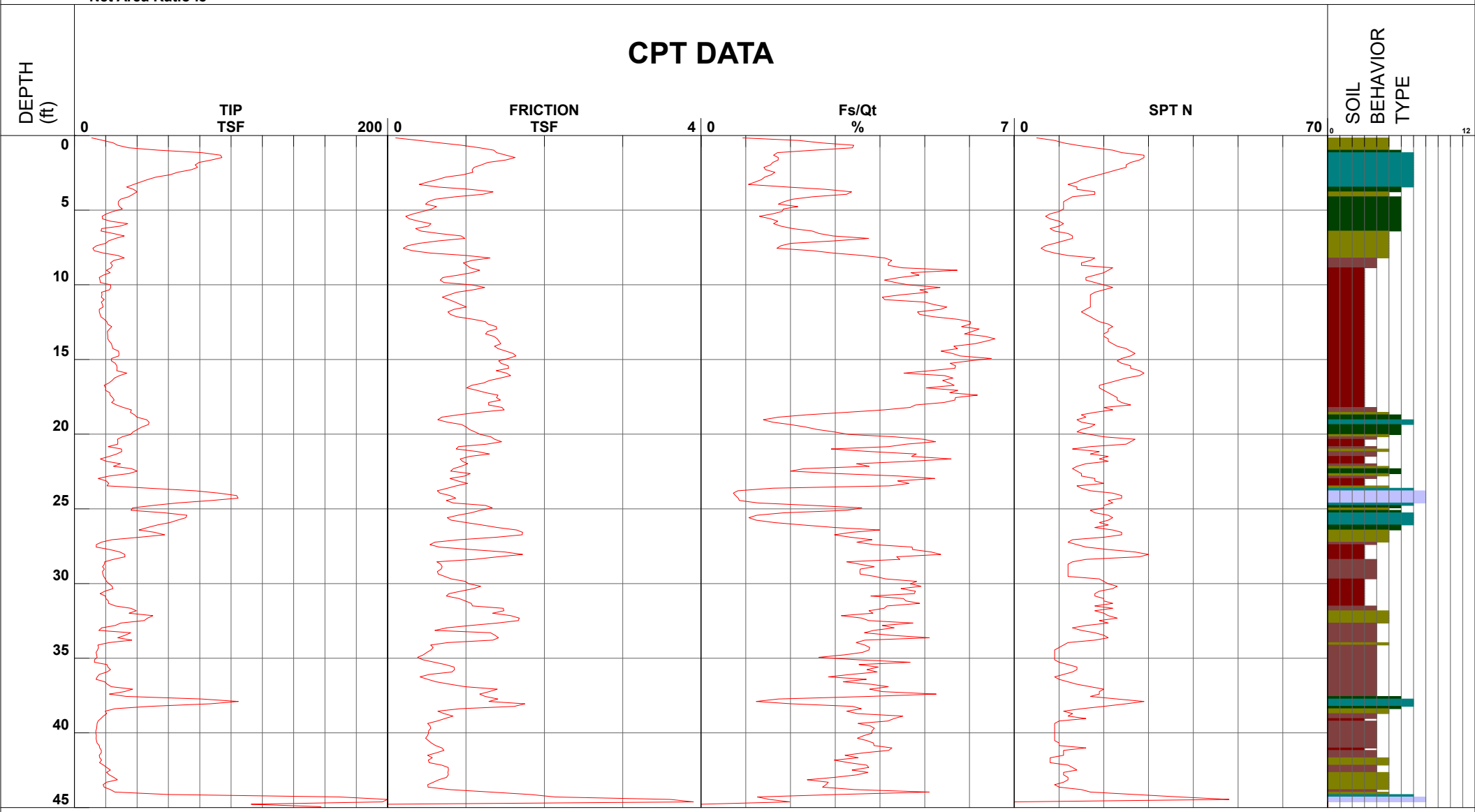
EST GW Depth During Test		>45.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

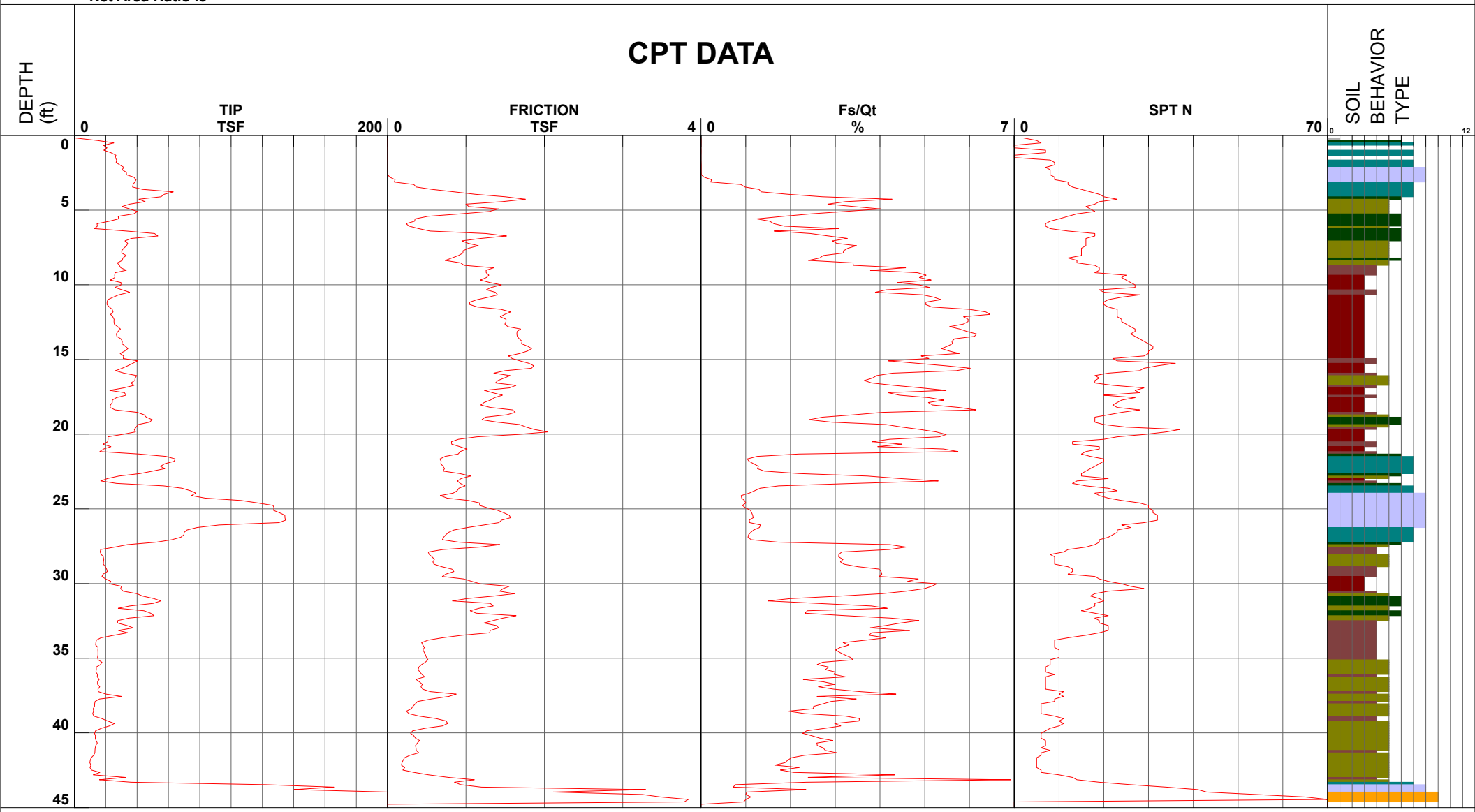
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Project	6402 Santa Teresa	Operator	CB	Filename	SDF(365).cpt
Job Number		Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	12/28/2015 2:33:32 PM	Maximum Depth	44.95 ft
EST GW Depth During Test		>45.00 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

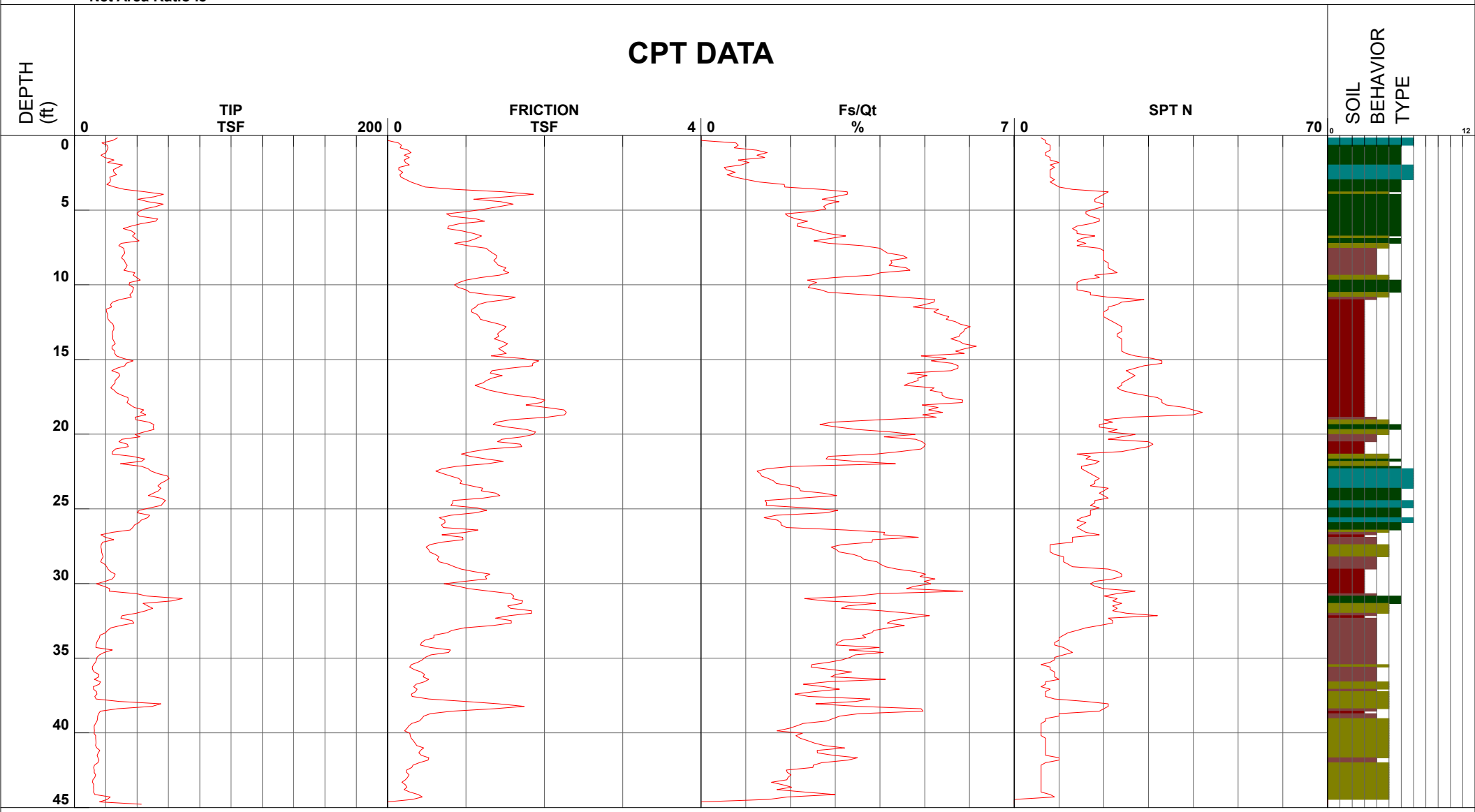
S*Soil behavior type and SPT based on data from UBC-1983

Kleinfelder Inc

Project	6402 Santa Teresa	Operator	CB	Filename	SDF(366).cpt
Job Number		Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	12/28/2015 3:11:06 PM	Maximum Depth	44.78 ft
EST GW Depth During Test		>45.00 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

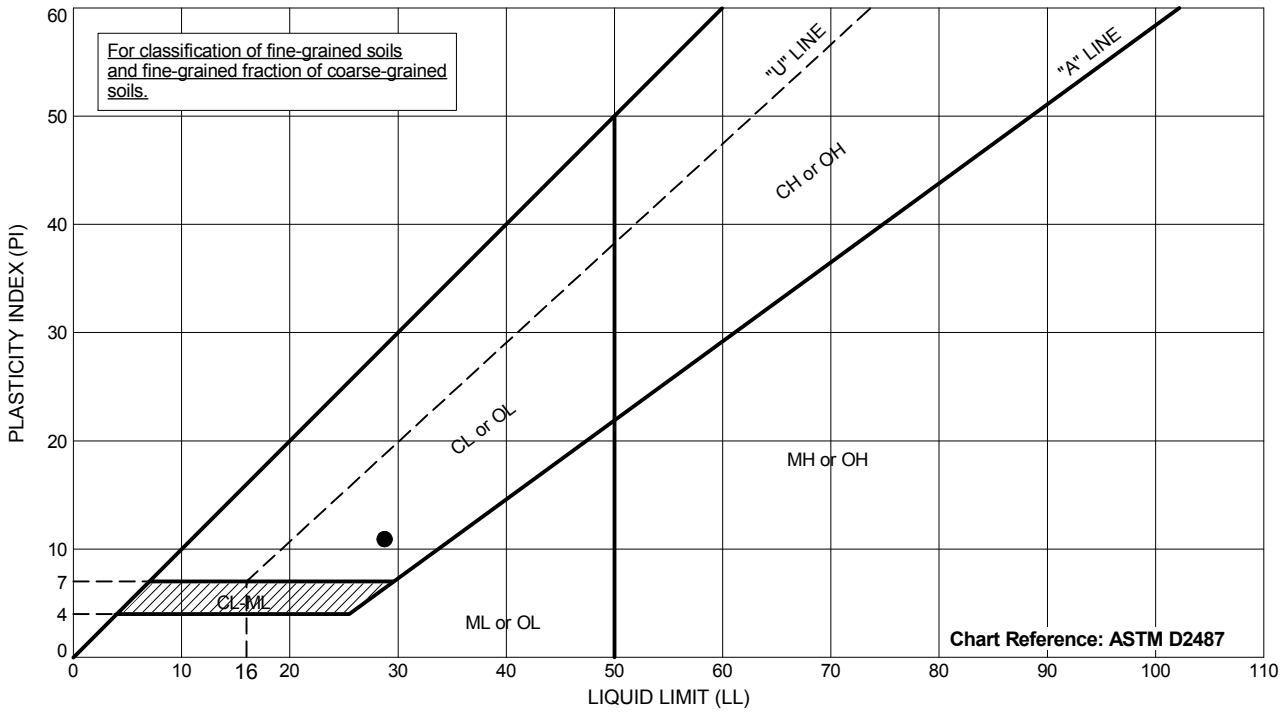
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX B

LABORATORY TEST RESULTS

PLOTTED: 01/19/2016 04:41 PM BY: jsala



	Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
●	CPT-1	1	DARK BROWN LEAN CLAY (CL)	NM	29	18	11

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

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	20163632
DRAWN BY:	CP
CHECKED BY:	CP
DATE:	1/8/2016
REVISED:	-

ATTERBERG LIMITS

PG&E SANTA TERESA
6402 SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

B-1

gINT FILE: PROJECTWISE: 20163632 - Pg&e Santa Teresa.gpj
gINT TEMPLATE: PROJECTWISE: KLF_STANDARD_GINT_LIBRARY_2015.GLB [KLF_ATTERBERG (ASTM)]

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.: 20163632.001A
Client's Project Name: PG&E - Santa Teresa
Date Sampled: 5-Jan-2016
Date Received: 7-Jan-2016
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 15-Jan-2016

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)*
1601075-001	CP-2, 2 @ 3' & 5'	+420	7.44	16,000	3,100	N.D.	N.D.	22

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	13-Jan-2016	13-Jan-2016	13-Jan-2016	13-Jan-2016	12-Jan-2016	14-Jan-2016	14-Jan-2016

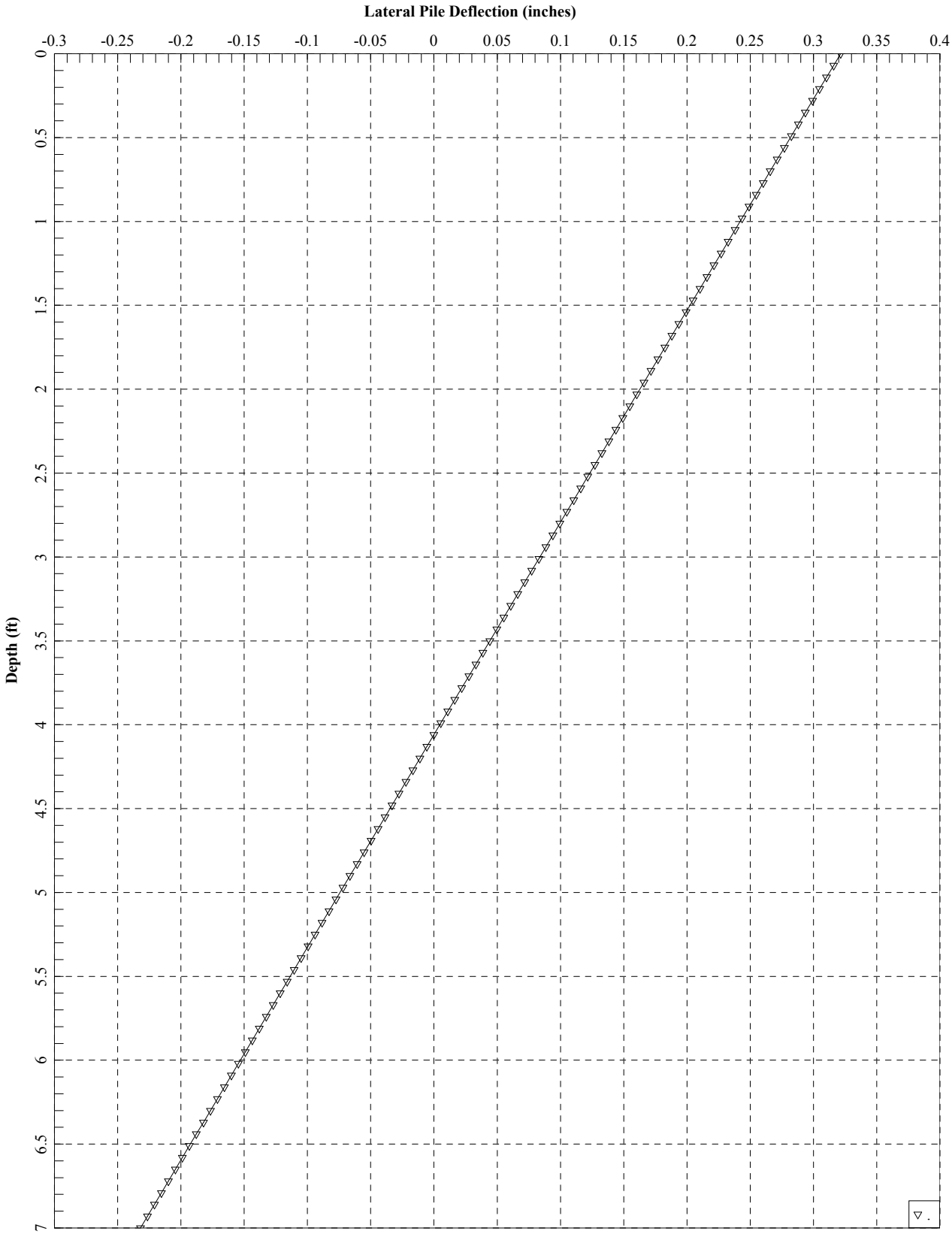
Cheryl McMillen
Laboratory Director

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

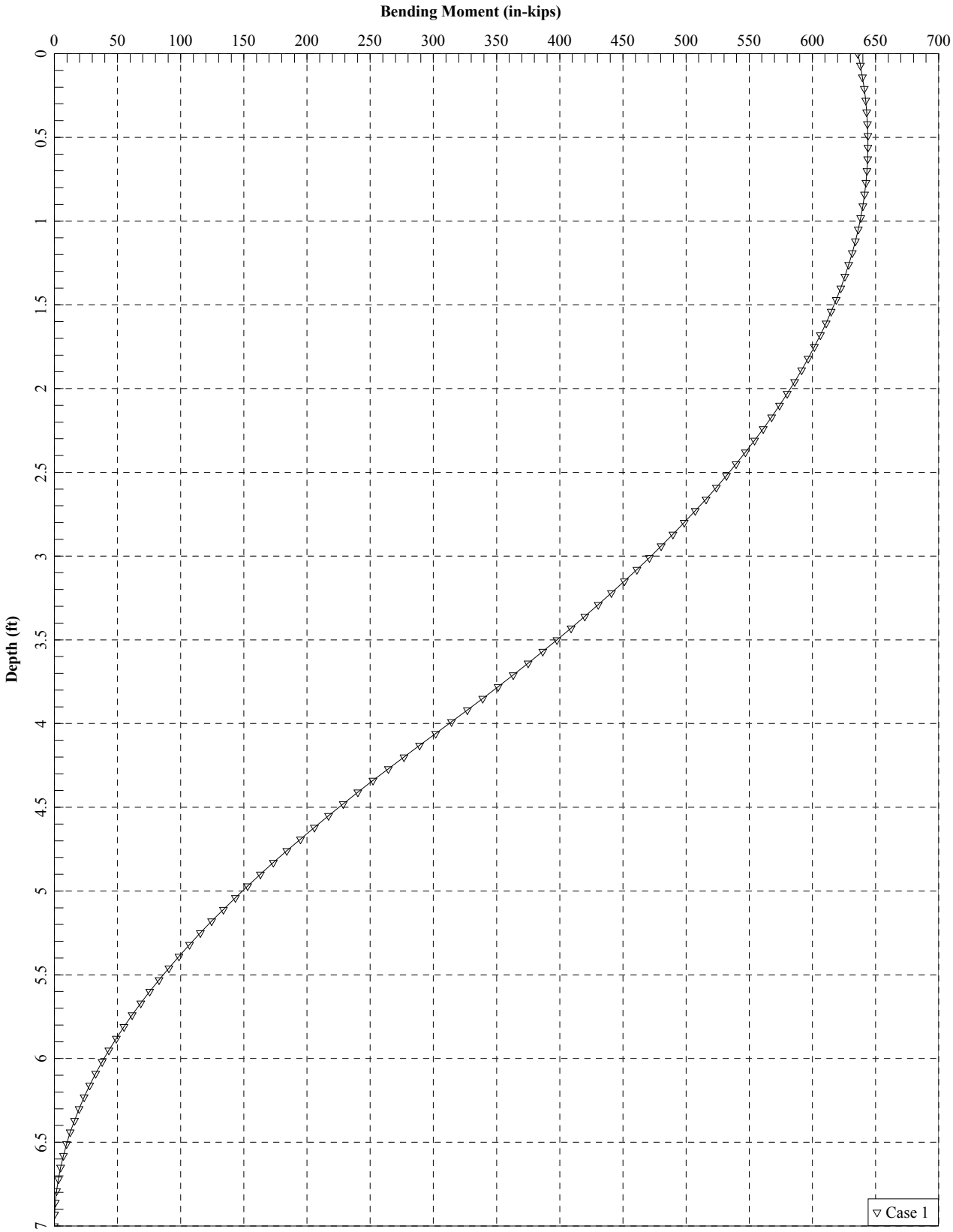
Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

APPENDIX C

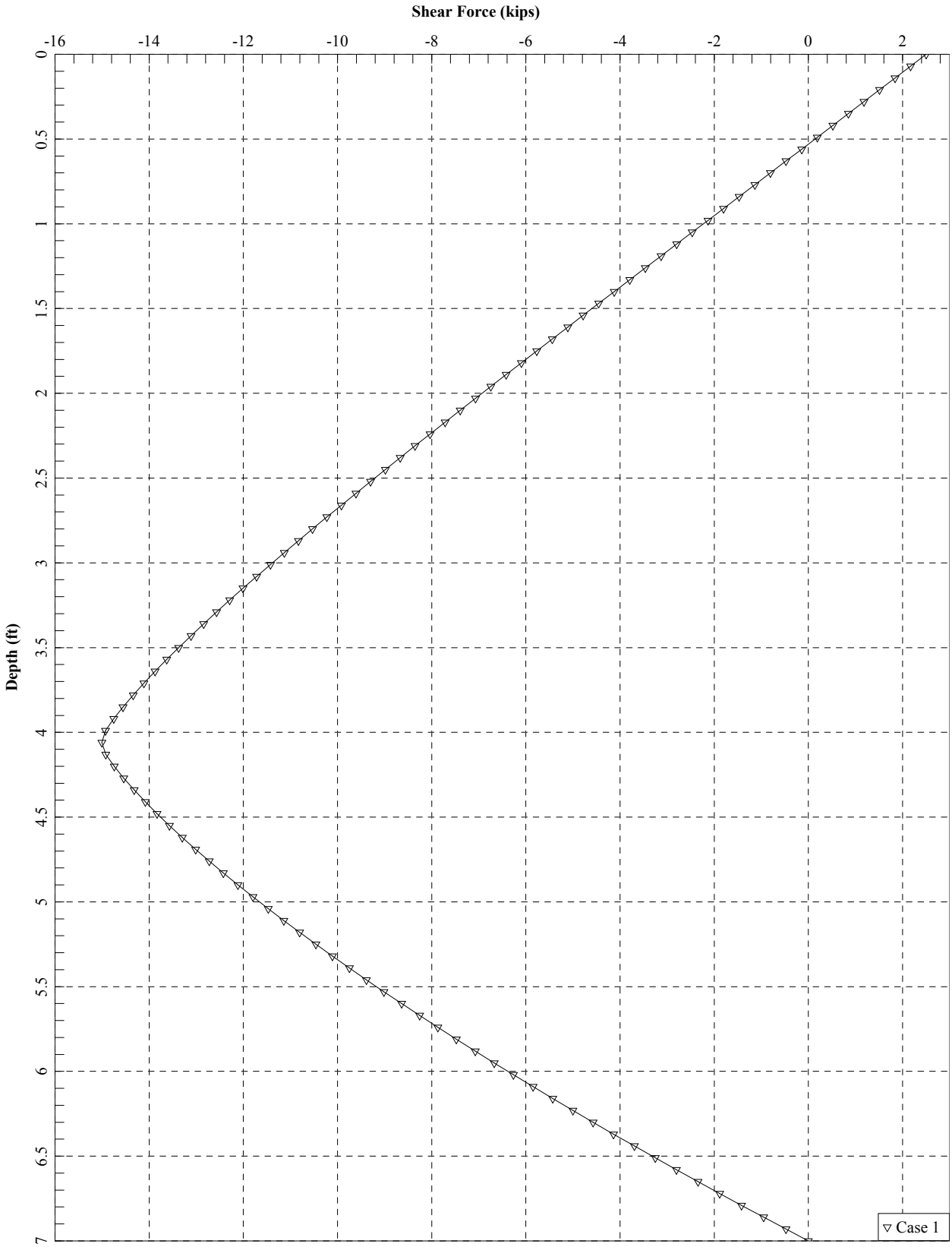
LPILE ANALYSIS RESULTS



3.5-Foot Diameter



3.5-Foot Diameter



3.5-Foot Diameter

==========

LPile Plus for Windows, Version 2013-07.007

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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is forbidden by the software license agreement.

Files Used for Analysis

Path to file locations: B:\2016\PG&E\PG&E Geo\Santa Teresa Substation\LPile\
Name of input data file: PG&E Santa Teresa Substation.lp7d
Name of output report file: PG&E Santa Teresa Substation.lp7o
Name of plot output file: PG&E Santa Teresa Substation.lp7p
Name of runtime messeage file: PG&E Santa Teresa Substation.lp7r

---

Date and Time of Analysis

Date: January 19, 2016 Time: 17:03:03

Problem Title

Project Name: PG&E Santa Teresa Substation

Job Number: 20163632.001A

Client: PG&E

Engineer: B.Price

Description:

Program Options and Settings

Engineering Units of Input Data and Computations:
- Engineering units are US Customary Units (pounds, feet, inches)

```
Analysis Control Options:
- Maximum number of iterations allowed                  =          500
- Deflection tolerance for convergence                  =   1.0000E-05 in
- Maximum allowable deflection                          =     100.0000 in
- Number of pile increments                             =          100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

Computational Options:
 - Use unfactored loads in computations (conventional analysis)
 - Compute pile response under loading and nonlinear bending properties of pile
   (only if nonlinear pile properties are input)
 - Use of p-y modification factors for p-y curves not selected
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - No p-y curves to be computed and reported for user-specified depths
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1


--------------------------------------------------------------------------------
                    Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Total number of pile sections                           =          1

Total length of pile                                    =       7.00 ft

Depth of ground surface below top of pile               =       0.00 ft

Pile diameter values used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile.

Point         Depth              Pile
                X              Diameter
                ft               in
-----       ---------        -----------
  1           0.00000         42.0000000
  2          7.000000         42.0000000


Input Structural Properties:
----------------------------

Pile Section No. 1:

   Section Type                                         =    Elastic Pile
   Cross-sectional Shape                                =        Circular
   Section Length                                       =      7.00000 ft
   Top Width                                            =     42.00000 in
   Bottom Width                                         =     42.00000 in
   Top Area                                             =   1385.44236 Sq. in
   Bottom Area                                          =   1385.44236 Sq. in
   Moment of Inertia at Top                             =      152745. in^4
   Moment of Inertia at Bottom                          =      152745. in^4
   Elastic Modulus                                      =     3600000. lbs/in^2


--------------------------------------------------------------------------------
                      Ground Slope and Pile Batter Angles
--------------------------------------------------------------------------------
```

```
Ground Slope Angle                                    =        0.000 degrees
                                                      =        0.000 radians

Pile Batter Angle                                     =        0.000 degrees
                                                      =        0.000 radians


--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 4 layers

Layer 1 is stiff clay without free water

   Distance from top of pile to top of layer          =       0.0000 ft
   Distance from top of pile to bottom of layer       =      7.00000 ft
   Effective unit weight at top of layer              =    120.00000 pcf
   Effective unit weight at bottom of layer           =    120.00000 pcf
   Undrained cohesion at top of layer                 =   1100.00000 psf
   Undrained cohesion at bottom of layer              =   1100.00000 psf
   Epsilon-50 at top of layer                         =       0.0000
   Epsilon-50 at bottom of layer                      =       0.0000

   NOTE: Internal default values for Epsilon-50 will be computed for this soil layer.

Layer 2 is stiff clay without free water

   Distance from top of pile to top of layer          =      7.00000 ft
   Distance from top of pile to bottom of layer       =     27.00000 ft
   Effective unit weight at top of layer              =    115.00000 pcf
   Effective unit weight at bottom of layer           =    115.00000 pcf
   Undrained cohesion at top of layer                 =   1800.00000 psf
   Undrained cohesion at bottom of layer              =   1800.00000 psf
   Epsilon-50 at top of layer                         =       0.0000
   Epsilon-50 at bottom of layer                      =       0.0000

   NOTE: Internal default values for Epsilon-50 will be computed for this soil layer.

Layer 3 is stiff clay without free water

   Distance from top of pile to top of layer          =     27.00000 ft
   Distance from top of pile to bottom of layer       =     30.00000 ft
   Effective unit weight at top of layer              =    115.00000 pcf
   Effective unit weight at bottom of layer           =    115.00000 pcf
   Undrained cohesion at top of layer                 =   1500.00000 psf
   Undrained cohesion at bottom of layer              =   1500.00000 psf
   Epsilon-50 at top of layer                         =       0.0000
   Epsilon-50 at bottom of layer                      =       0.0000

   NOTE: Internal default values for Epsilon-50 will be computed for this soil layer.

Layer 4 is stiff clay with water-induced erosion

   Distance from top of pile to top of layer          =     30.00000 ft
   Distance from top of pile to bottom of layer       =     45.00000 ft
   Effective unit weight at top of layer              =     52.60000 pcf
   Effective unit weight at bottom of layer           =     52.60000 pcf
   Undrained cohesion at top of layer                 =   1500.00000 psf
   Undrained cohesion at bottom of layer              =   1500.00000 psf
   Epsilon-50 at top of layer                         =       0.0000
   Epsilon-50 at bottom of layer                      =       0.0000
   Subgrade k at top of layer                         =       0.0000 pci
   Subgrade k at bottom of layer                      =       0.0000 pci

   NOTE: Internal default values for Epsilon-50 will be computed for this soil layer.

   NOTE: Internal default values for subgrade k will be computed for this soil layer.
```

(Depth of lowest soil layer extends 38.00 ft below pile tip)

Summary of Soil Properties

Layer Num.	Layer Soil Type (p-y Curve Criteria)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Strain Factor Epsilon 50	kpy pci
1	Stiff Clay w/o Free Water	0.00	120.000	1100.000	default	--
		7.000	120.000	1100.000	default	--
2	Stiff Clay w/o Free Water	7.000	115.000	1800.000	default	--
		27.000	115.000	1800.000	default	--
3	Stiff Clay w/o Free Water	27.000	115.000	1500.000	default	--
		30.000	115.000	1500.000	default	--
4	Stiff Clay with Free Water	30.000	52.600	1500.000	default	default
		45.000	52.600	1500.000	default	default

Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
1	1	V = 2500.00000 lbs	M = 636000. in-lbs	0.0000000	No

V = perpendicular shear force applied to pile head
M = bending moment applied to pile head
y = lateral deflection relative to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applie to pile head
Axial thrust is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

PG&E Santa Teresa Substation.lp7o
Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 2500.0 lbs
Applied moment at pile head = 636000.0 in-lbs
Axial thrust load on pile head = 0.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/in	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3215	636000.	2500.0000	-0.006626	87.4398	5.499E+11	-391.3734	511.3315	0.000
0.07000	0.3159	637962.	2170.9988	-0.006625	87.7096	5.499E+11	-391.9592	1042.2370	0.000
0.140	0.3103	639647.	1841.5266	-0.006624	87.9413	5.499E+11	-392.5023	1062.3944	0.000
0.210	0.3048	641056.	1511.6150	-0.006623	88.1349	5.499E+11	-393.0015	1083.1642	0.000
0.280	0.2992	642187.	1181.3030	-0.006622	88.2904	5.499E+11	-393.4557	1104.5770	0.000
0.350	0.2937	643040.	850.6288	-0.006621	88.4078	5.499E+11	-393.8637	1126.6656	0.000
0.420	0.2881	643616.	519.6320	-0.006620	88.4869	5.499E+11	-394.2242	1149.4653	0.000
0.490	0.2825	643913.	188.3528	-0.006619	88.5278	5.499E+11	-394.5357	1173.0138	0.000
0.560	0.2770	643932.	-143.1670	-0.006618	88.5304	5.499E+11	-394.7971	1197.3518	0.000
0.630	0.2714	643673.	-474.8845	-0.006617	88.4947	5.499E+11	-395.0066	1222.5228	0.000
0.700	0.2659	643134.	-806.7556	-0.006616	88.4207	5.499E+11	-395.1628	1248.5740	0.000
0.770	0.2603	642317.	-1138.7348	-0.006615	88.3084	5.499E+11	-395.2639	1275.5561	0.000
0.840	0.2547	641221.	-1470.7751	-0.006614	88.1577	5.499E+11	-395.3083	1303.5240	0.000
0.910	0.2492	639846.	-1802.8281	-0.006613	87.9687	5.499E+11	-395.2941	1332.5371	0.000
0.980	0.2436	638193.	-2134.8438	-0.006612	87.7413	5.499E+11	-395.2193	1362.6600	0.000
1.050	0.2381	636260.	-2466.7703	-0.006611	87.4756	5.499E+11	-395.0819	1393.9625	0.000
1.120	0.2325	634048.	-2798.5540	-0.006610	87.1715	5.499E+11	-394.8795	1426.5210	0.000
1.190	0.2270	631558.	-3130.1396	-0.006609	86.8292	5.499E+11	-394.6100	1460.4186	0.000
1.260	0.2214	628790.	-3461.4695	-0.006608	86.4486	5.499E+11	-394.2707	1495.7462	0.000
1.330	0.2159	625743.	-3792.4839	-0.006607	86.0297	5.499E+11	-393.8590	1532.6033	0.000
1.400	0.2103	622418.	-4123.1211	-0.006606	85.5726	5.499E+11	-393.3721	1571.0993	0.000
1.470	0.2048	618816.	-4453.3163	-0.006605	85.0773	5.499E+11	-392.8069	1611.3545	0.000
1.540	0.1992	614937.	-4783.0025	-0.006604	84.5440	5.499E+11	-392.1602	1653.5016	0.000
1.610	0.1937	610781.	-5112.1096	-0.006603	83.9726	5.499E+11	-391.4283	1697.6876	0.000
1.680	0.1881	606349.	-5440.5647	-0.006603	83.3632	5.499E+11	-390.6077	1744.0754	0.000
1.750	0.1826	601641.	-5768.2914	-0.006602	82.7160	5.499E+11	-389.6940	1792.8463	0.000
1.820	0.1770	596658.	-6095.2099	-0.006601	82.0309	5.499E+11	-388.6830	1844.2028	0.000
1.890	0.1715	591401.	-6421.2361	-0.006600	81.3081	5.499E+11	-387.5699	1898.3716	0.000
1.960	0.1660	585870.	-6746.2822	-0.006599	80.5478	5.499E+11	-386.3495	1955.6076	0.000
2.030	0.1604	580067.	-7070.2557	-0.006598	79.7499	5.499E+11	-385.0160	2016.1984	0.000
2.100	0.1549	573992.	-7393.0590	-0.006597	78.9147	5.499E+11	-383.5634	2080.4698	0.000
2.170	0.1493	567647.	-7714.5892	-0.006596	78.0423	5.499E+11	-381.9848	2148.7927	0.000
2.240	0.1438	561032.	-8034.7373	-0.006595	77.1329	5.499E+11	-380.2727	2221.5908	0.000
2.310	0.1382	554148.	-8353.3877	-0.006595	76.1865	5.499E+11	-378.4189	2299.3510	0.000
2.380	0.1327	546998.	-8670.4177	-0.006594	75.2035	5.499E+11	-376.4143	2382.6359	0.000
2.450	0.1272	539582.	-8985.6961	-0.006593	74.1839	5.499E+11	-374.2486	2472.0988	0.000
2.520	0.1216	531902.	-9299.0830	-0.006592	73.1280	5.499E+11	-371.9106	2568.5036	0.000
2.590	0.1161	523959.	-9610.4281	-0.006591	72.0361	5.499E+11	-369.3873	2672.7496	0.000
2.660	0.1106	515756.	-9919.5697	-0.006590	70.9083	5.499E+11	-366.6643	2785.9035	0.000
2.730	0.1050	507295.	-10226.	-0.006590	69.7449	5.499E+11	-363.7251	2909.2403	0.000
2.800	0.0995	498576.	-10531.	-0.006589	68.5463	5.499E+11	-360.5510	3044.2990	0.000
2.870	0.0940	489603.	-10832.	-0.006588	67.3126	5.499E+11	-357.1202	3192.9546	0.000
2.940	0.0884	480378.	-11130.	-0.006587	66.0444	5.499E+11	-353.4075	3357.5180	0.000
3.010	0.0829	470904.	-11426.	-0.006587	64.7418	5.499E+11	-349.3833	3540.8722	0.000
3.080	0.0774	461184.	-11717.	-0.006586	63.4054	5.499E+11	-345.0125	3746.6651	0.000
3.150	0.0718	451219.	-12005.	-0.006585	62.0355	5.499E+11	-340.2531	3979.5860	0.000
3.220	0.0663	441015.	-12289.	-0.006585	60.6325	5.499E+11	-335.0541	4245.7741	0.000
3.290	0.0608	430575.	-12568.	-0.006584	59.1971	5.499E+11	-329.3524	4553.4377	0.000
3.360	0.0552	419901.	-12842.	-0.006583	57.7297	5.499E+11	-323.0689	4913.8258	0.000
3.430	0.0497	409000.	-13110.	-0.006583	56.2310	5.499E+11	-316.1020	5342.8127	0.000
3.500	0.0442	397876.	-13372.	-0.006582	54.7017	5.499E+11	-308.3182	5863.6072	0.000
3.570	0.0386	386535.	-13628.	-0.006581	53.1424	5.499E+11	-299.5357	6511.6524	0.000
3.640	0.0331	374982.	-13875.	-0.006581	51.5540	5.499E+11	-289.4976	7344.1438	0.000
3.710	0.0276	363225.	-14113.	-0.006580	49.9376	5.499E+11	-277.8211	8460.2764	0.000
3.780	0.0221	351271.	-14341.	-0.006580	48.2942	5.499E+11	-263.8937	10050.	0.000
3.850	0.0165	339132.	-14555.	-0.006579	46.6252	5.499E+11	-246.6300	12533.	0.000

3.920	0.0110	326818.	-14753.	-0.006579	44.9323	5.499E+11	-223.7630	17081.	0.000
3.990	0.005478	314347.	-14926.	-0.006578	43.2177	5.499E+11	-188.7880	28949.	0.000
4.060	-4.742E-05	301743.	-15003.	-0.006578	41.4848	5.499E+11	6.6989	118655.	0.000
4.130	-0.005573	289143.	-14919.	-0.006577	39.7525	5.499E+11	191.2663	28831.	0.000
4.200	-0.0111	276678.	-14743.	-0.006577	38.0388	5.499E+11	228.2027	17274.	0.000
4.270	-0.0166	264374.	-14541.	-0.006576	36.3472	5.499E+11	253.5529	12814.	0.000
4.340	-0.0221	252249.	-14320.	-0.006576	34.6802	5.499E+11	273.5883	10377.	0.000
4.410	-0.0277	240317.	-14083.	-0.006576	33.0398	5.499E+11	290.4974	8819.0712	0.000
4.480	-0.0332	228590.	-13832.	-0.006575	31.4275	5.499E+11	305.3251	7726.7841	0.000
4.550	-0.0387	217079.	-13570.	-0.006575	29.8448	5.499E+11	318.6582	6913.7831	0.000
4.620	-0.0442	205792.	-13298.	-0.006575	28.2931	5.499E+11	330.8619	6282.3785	0.000
4.690	-0.0498	194739.	-13015.	-0.006574	26.7735	5.499E+11	342.1800	5776.2083	0.000
4.760	-0.0553	183927.	-12723.	-0.006574	25.2870	5.499E+11	352.7838	5360.3372	0.000
4.830	-0.0608	173364.	-12422.	-0.006574	23.8348	5.499E+11	362.7987	5011.8883	0.000
4.900	-0.0663	163057.	-12114.	-0.006574	22.4178	5.499E+11	372.3194	4715.2166	0.000
4.970	-0.0718	153013.	-11797.	-0.006573	21.0369	5.499E+11	381.4193	4459.2384	0.000
5.040	-0.0774	143238.	-11473.	-0.006573	19.6930	5.499E+11	390.1565	4235.8688	0.000
5.110	-0.0829	133738.	-11142.	-0.006573	18.3869	5.499E+11	398.5777	4039.0635	0.000
5.180	-0.0884	124520.	-10804.	-0.006573	17.1195	5.499E+11	406.7212	3864.2078	0.000
5.250	-0.0939	115588.	-10459.	-0.006572	15.8916	5.499E+11	414.6187	3707.7136	0.000
5.320	-0.0995	106950.	-10107.	-0.006572	14.7039	5.499E+11	422.2969	3566.7468	0.000
5.390	-0.1050	98609.	-9749.2148	-0.006572	13.5571	5.499E+11	429.7783	3439.0373	0.000
5.460	-0.1105	90571.	-9385.1333	-0.006572	12.4520	5.499E+11	437.0823	3322.7436	0.000
5.530	-0.1160	82842.	-9014.9841	-0.006572	11.3894	5.499E+11	444.2254	3216.3560	0.000
5.600	-0.1215	75426.	-8638.8962	-0.006572	10.3698	5.499E+11	451.2221	3118.6239	0.000
5.670	-0.1271	68328.	-8256.9873	-0.006572	9.3940	5.499E+11	458.0850	3028.5020	0.000
5.740	-0.1326	61554.	-7869.3650	-0.006572	8.4627	5.499E+11	464.8252	2945.1097	0.000
5.810	-0.1381	55108.	-7476.1283	-0.006571	7.5764	5.499E+11	471.4525	2867.6993	0.000
5.880	-0.1436	48994.	-7077.3685	-0.006571	6.7359	5.499E+11	477.9757	2795.6318	0.000
5.950	-0.1491	43218.	-6673.1696	-0.006571	5.9417	5.499E+11	484.4026	2728.3575	0.000
6.020	-0.1547	37783.	-6263.6097	-0.006571	5.1946	5.499E+11	490.7400	2665.4010	0.000
6.090	-0.1602	32695.	-5848.7612	-0.006571	4.4950	5.499E+11	496.9944	2606.3485	0.000
6.160	-0.1657	27957.	-5428.6916	-0.006571	3.8437	5.499E+11	503.1713	2550.8385	0.000
6.230	-0.1712	23575.	-5003.4638	-0.006571	3.2411	5.499E+11	509.2759	2498.5535	0.000
6.300	-0.1767	19551.	-4573.1365	-0.006571	2.6880	5.499E+11	515.3129	2449.2134	0.000
6.370	-0.1823	15892.	-4137.7647	-0.006571	2.1849	5.499E+11	521.2866	2402.5702	0.000
6.440	-0.1878	12600.	-3697.4001	-0.006571	1.7323	5.499E+11	527.2008	2358.4035	0.000
6.510	-0.1933	9680.1330	-3252.0909	-0.006571	1.3309	5.499E+11	533.0591	2316.5165	0.000
6.580	-0.1988	7136.4399	-2801.8829	-0.006571	0.9811	5.499E+11	538.8648	2276.7332	0.000
6.650	-0.2043	4972.9698	-2346.8189	-0.006571	0.6837	5.499E+11	544.6210	2238.8955	0.000
6.720	-0.2099	3193.7843	-1886.9393	-0.006571	0.4391	5.499E+11	550.3303	2202.8611	0.000
6.790	-0.2154	1802.9118	-1422.2825	-0.006571	0.2479	5.499E+11	555.9955	2168.5012	0.000
6.860	-0.2209	804.3496	-952.8846	-0.006571	0.1106	5.499E+11	561.6188	2135.6995	0.000
6.930	-0.2264	202.0657	-478.7795	-0.006571	0.0278	5.499E+11	567.2025	2104.3503	0.000
7.000	-0.2319	0.000	0.000	-0.006571	0.000	5.499E+11	572.7487	1037.1786	0.000

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =      0.3214682 inches
Computed slope at pile head      =     -0.0066258 radians
Maximum bending moment           =        643932. inch-lbs
Maximum shear force              =        -15003. lbs
Depth of maximum bending moment  =      0.5600000 feet below pile head
Depth of maximum shear force     =      4.0600000 feet below pile head
Number of iterations             =             39
Number of zero deflection points =              1
```

Summary of Pile Response(s)

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, lbs, and Load 2 = Moment, in-lbs
Load Type 2: Load 1 = Shear, lbs, and Load 2 = Slope, radians
Load Type 3: Load 1 = Shear, lbs, and Load 2 = Rotational Stiffness, in-lbs/radian
Load Type 4: Load 1 = Top Deflection, inches, and Load 2 = Moment, in-lbs
Load Type 5: Load 1 = Top Deflection, inches, and Load 2 = Slope, radians

Load Case No.	Load Type No.	Pile-head Condition 1 V(lbs) or y(inches)	Pile-head Condition 2 in-lb, rad., or in-lb/rad.	Axial Loading lbs	Pile-head Deflection inches	Maximum Moment in Pile in-lbs	Maximum Shear in Pile lbs	Pile-head Rotation radians
1	1	V = 2500.0000	M = 636000.	0.0000000	0.32146820	643932.	-15003.	-0.00662579

The analysis ended normally.

APPENDIX D

GBA INFORMATION SHEET

Important Information about This

Geotechnical-Engineering Report

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

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

Geotechnical 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 civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

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

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly

problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of 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. Confer with you GBC-Member geotechnical engineer for more information.

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