# Final Geotechnical Study

Sycamore to Penasquitos
230kV Transmission Line
Alternate 5
San Diego County, California

**Prepared for:**

**Attention: Mr. Kevin Mathey, PE**

**Project No.: T-0126-G**

**January 12, 2017**

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January 12, 2017
Project No.: T-0126-G

Attention: **Mr. Kevin Mathey, PE**

Project: SDG&E 230kV Transmission Line (Alternate 5)
Sycamore to Penasquitos
San Diego County, California

Subject: FINAL GEOTECHNICAL STUDY

Dear Mr. Mathey:

This report presents the results of the final geotechnical study for the Sycamore to Penasquitos 230kV Transmission Line project – Alternate 5 Alignment. TGE previously prepared a Supplemental Geotechnical Study presenting parameters for Alternate Alignments 4 & 5 and a Geotechnical Study for the original alignment consisting of 61 new steel poles supporting the transmission line. The purpose of this final study is to provide geotechnical analysis and foundation design parameters at steel pole locations along the Alternate 5 Alignment.

Presented herein are references to our previous Sycamore to Penasquitos 230kV Transmission Line Geotechnical Study, Sycamore to Penasquitos 230kV Transmission Line Supplemental Geotechnical Study and geotechnical studies performed by others associated with the design / construction of Sycamore Substation and TL 6961. The previous studies included borings, seismic lines, test pits, and hollow-stem auger borings. The referenced reports have been reviewed and the information presented was included to provide the appropriate design parameters and recommendations. Previous subsurface explorations used in reference to the proposed pole structures are provided in *Appendix A, Previous Geotechnical Information by TGE and Appendix B, Previous Geotechnical Information by Others.*

It is also recommended that the Design Engineer consider the information contained in ***Tables 1 and 2*** in *Section 6* of this report for the structural design of the steel pole foundations, as well as the development of the project plans and specifications.

It is recommended that the forthcoming project plans and specifications, be reviewed by TRINITY Geotechnical Engineering (TGE) for consistency with our report prior to the bid process in order to avoid possible conflicts, misinterpretations, inadvertent omissions, etc. It should also be noted that the applicability and final evaluation of recommendations presented herein are contingent upon construction phase field monitoring by TGE in light of the widely acknowledged importance of geotechnical consultant continuity through the various planning, design and construction stages of a project.

TGE appreciates the opportunity to provide this geotechnical engineering service for this project and we look forward to continuing our role as your geotechnical engineering consultant.

Respectfully submitted,
**TRINITY Geotechnical Engineering, Inc.**

Jeffrey Magalong, PE
President

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## 1. INTRODUCTION

This report provides the results of the final geotechnical study conducted for the Sycamore to Penasquitos 230kV Transmission Line project – Alternate 5 Alignment which extends from the Sycamore Substation to the Penasquitos Substation. The report has been specifically prepared to provide geotechnical parameters in connection with the foundation design of a total of seven (7) proposed tubular steel pole structures along the alignment. The alignment of the proposed project in relation to nearby streets and landmarks is shown on *Figure No. 1, Vicinity Map*.

This report has been prepared for the exclusive use of the client and their consultants in the design of the proposed project. Contract requirements set forth by the project plans and specifications will supersede any general observations and all recommendations presented in this report.

## 2. SCOPE OF SERVICES

Our scope of services for this project included the following tasks:

- Reviewed readily available background data, including in-house geotechnical data, geologic maps, topographic maps, and literature relevant to the subject project.
- Engineering evaluation of data by others was collected to develop geotechnical design parameters and recommendations for some of the proposed steel pole foundations.
- Preparation of this report including reference maps and graphics, presenting our findings, conclusions and geotechnical recommendations specifically addressing the following items:
    - Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
    - Evaluation of the site geology and geotechnical/geologic hazards.
    - Evaluation of project feasibility and suitability of on-site soils/bedrock for foundation support.
    - Recommendations including geotechnical parameters to be used for foundation design analysis.
    - General construction considerations and preliminary recommendations for the steel pole foundations.

## 3. PROJECT DESCRIPTION

The project site extends from the Sycamore Substation in Poway, CA to the Penasquitos Substation located south of the confluence of Soledad and Carmel Valleys in the coastal region of San Diego, CA.

The CPUC has selected the Sycamore to Penasquitos 230kV Alternate 5 Alignment as the approved alignment for this Transmission Line. This project will consist of a majority of the Transmission Line placed underground and seven (7) steel poles used at select locations to support the line overhead (see *Figure Nos. 2 to 5, Plot Plans*). The alignment runs parallel along major roadways, such as Pomerado Road and Carroll Canyon Road before it traverses northwest along mountainous terrain and terminates at the Penasquitos Substation.

The project includes minor modifications of the existing Sycamore Canyon and Penasquitos Substations to allow for connection of the new 230 kV transmission line.

The proposed alignment traverses an overall gently sloping westward trending topographic plain with local moderately variable terrain. Elevations for the project range from a high of approximately 860 feet above mean sea level (MSL) at the easterly beginning (Sycamore Substation) to a low of approximately 310 feet above MSL at the western terminus (Penasquitos Substation). Detailed geographic and topographic information for the project alignment is presented on *Figure No. 1, Vicinity Map*.

## 4. SUBSURFACE CONDITIONS

A map of the project geology is shown on *Figure No. 6, Regional Geology Map*. Subsurface conditions relative to the seven (7) proposed steel pole structure locations was compiled from previous reports and is presented in detail in *Appendix A, Previous Geotechnical Information by TGE*, and *Appendix B, Previous Geotechnical Information by Others*.

## 5. GEOTECHNICAL/GEOLOGIC HAZARDS

A summary of the geotechnical conditions which may affect the design and construction of the project are provided in the sections below.

### 5.1 Faulting

Based on review of the USGS fault map, no known active faults with the potential for surface fault rupture are known to exist beneath the pole locations. Accordingly, the potential for surface rupture at the sites due to faulting is considered very low during the design life of the proposed structures.

### 5.2 Seismicity / Ground Shaking

Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the structures are designed and constructed in conformance with current building codes and engineering practices (see *Section 6.4, Seismic Design*).

### 5.3 Landslide, Rock Fall & Slope Instability

Review of regional geology maps and literature did not reveal the existence of rockfall, landslides or landslide activity within the pole locations. In addition, we found no obvious visible physiographic features suggesting the existence of a landslide and rockfall along the alignment during our preliminary review. Therefore, the potential for landslide impacting the site is negligible.

Certain pole sites are located or adjacent to steeply sloping terrain. However, these pole sites are founded on formations that are not known to have gross slope instability in its natural state. Therefore, the potential for a slope failure is considered low.

It should be noted that all slopes (natural, cut, fill or otherwise) are subject to downhill "creep" to some degree, as well as possible surficial deterioration and erosion due to normal weathering. This general observation is made in order to emphasize the importance of slope maintenance, and is not intended to suggest a particularly unusual or compelling adverse condition.

### 5.4 Liquefaction and Seismically-induced Settlement

Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays is not adversely affected by ground shaking. Liquefaction is generally known to occur in saturated cohesionless soils at depths shallower than approximately 50 feet. Dynamic settlement due to earthquake shaking can occur in both dry and saturated sands.

The pole sites are underlain by either very dense bedrock or very dense/hard formation. Liquefaction is not anticipated within the poles founded within the bedrock and formational sites due to the very dense nature of the bedrock / formation and the lack of a groundwater table. Therefore, the potential for liquefaction and the associated ground deformation occurring beneath the structural site areas is considered nil.

### 5.5 Flood

Flood Insurance Rate Map (FEMA, Map Number 06073C1363G, 2012) show that the pole sites are not within the hazard Flood Zones; therefore, it is anticipated that flooding should not impact the foundation design of the power poles.

### 5.6 Slope Erosion

In anticipation of minor erosion due to normal weathering, it is understood that SDG&E requires a minimum of 2-feet soil discount to be incorporated into the pole foundation design. TGE did not observe any locations with excessive erosion that may require additional soil discount. In addition, the use of construction Best Management Practices (BMPs) for erosion control should be implemented to avoid significant impacts along the alignment.

### 5.7 Expansive Soil

The soils underlying the pole foundations generally consist of fill and formation that are not anticipated to have expansive soils. The Expansion Index of the on-site surficial materials within the upper 10-feet is anticipated to be in the "Very Low" to "Low" range.

## 6. DESIGN RECOMMENDATIONS

Based on the results of TGE's previous field explorations, data review of previous geotechnical information, and engineering analyses, it is TGE's opinion that the proposed project is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and implemented during construction.

The following sections provide geotechnical design recommendations for the seven (7) proposed steel pole structures, as well as preliminary parameters for maintenance pads and access road earthwork and retention structures.

### 6.1 Deep Foundation Design

Drilled Piers

Design parameters for drilled piers are presented below for the proposed steel pole structures requiring engineered foundations.

Tables 1 and 2 below summarizes the engineering properties and subsurface material profiles anticipated for each of the proposed pole locations and may be utilized in the Moment Foundation Analysis and Design (MFAD) computer program used for pier foundation design. The use of specific material parameters for MFAD 4.0 and 5.1.18 should be selected by the pole foundation engineering designer in collaboration with an MFAD technical representative to confirm applicability and / or software limitations. These properties are estimates which were derived based on the field exploration program, visual observations, laboratory testing, engineering evaluation and analyses, empirical correlations, technical research, and our professional judgment. It should be noted that the estimated parameters are, in part, based on empirical correlations developed by Electric Power Research Institute (EPRI) and are considered to be conservative and do not reflect the actual in-situ strengths since pressure meter testing was not performed as a part of this project. TGE has also assumed that there will not be significant grade change from the existing elevations. If the scope of improvements and/or the assumptions made for the purposes of engineering evaluation change from those currently anticipated, TGE should be contacted for further evaluation.

**Table 1: MFAD Design Parameters**

| Drilled Pier Design Parameter | Material 1[1,2] | Material 2[3] | Material 3[4] |
|---|---|---|---|
| Total Unit Weight, $\gamma$ (pcf) | 120 | 125 | 130 |
| Apparent Cohesion (psf) | 100 | 400 | 800 |
| Internal Friction Angle ($^0$) | 28 | 35 | 40 |
| Deformation Modulus[6], $E_{pmt}$ (ksi) | 1.0 | 2.0 | 10.0 |
| Strength Reduction Factor[7] | 1.0 | 1.0 | 0.9 |
| Passive Pressure Multiplier[8] | 2.2 | 2.8 | 3.0 |
| Allowable Skin Friction (psf)[9] | 500 | 1,200 | 8,000 |
| End Bearing Capacity (psf) | 2,000 | 5,000 | 10,000 |

(1) Soil discount of 2 feet should be applied in Material 1;
(2) Material 1: Fill, soft to very stiff; Alluvium, loose to medium dense;
(3) Material 2: Fill, stiff to very stiff; Alluvium/Formation, medium dense to very dense;
(4) Material 3: Formation/Bedrock;
(5) Deformation modulus representing pressure meter test;
(6) The parameters provided for the Strength Reduction Factor are for use in MFAD version 4.0;
(7) Passive pressure multiplier is a factor representing increased lateral capacity from material arching;
(8) For the uplift design condition these values should be reduced by 30%; however, the unfactored pier weight may be added to the resistance.

**Table 2: MFAD Subsurface Profile**

| Pole Site | Subsurface Exploration | Layer Depth Range (feet; below existing grades) | | |
|---|---|---|---|---|
| | | Material 1 | Material 2 | Material 3 |
| CP3A, CP3B | Geo [1] | - | 0-5 | 5+ |
| P4 | Geo [1] | 0-2 | 2-5 | 5+ |
| CP5 | Geo [1] | - | 0-10 | 10+ |
| P6 | Geo [1], TGE SL [2] | 0-5 | 5-10 | 10+ |
| I-15E CP | TGE SL [2] | - | 0-25 | 25+ |
| I-15W CP | TGE SL [2] | - | 0-25 | 25+ |
| CC CP | TGE Boring [2] | 0-5 | 5-27 | 27+ |

(1) The layer depth range of these steel pole locations was evaluated based on the data obtained from Geocon (2012); see Appendix B

(2) The layer depth range of these steel pole locations was evaluated based on the data obtained from the previous TGE Geotechnical Studies; see Appendix A

## 6.2 Retaining Structures

It is anticipated that a cantilever block retaining wall will be utilized at the Carroll Canyon (CC CP) location and a Mechanically Stabilized Earth (MSE) retaining wall will be utilized

at CP5 location to accommodate maintenance pads. The following sections are preliminary designs recommendations and parameters for the retaining wall.

### 6.2.1 Lateral Earth Pressure

Retaining walls should be designed to resist a triangular distribution of lateral earth pressure plus surcharges from any adjacent loads. The recommended lateral earth pressures for retaining walls free to rotate, with level, 1.5:1 (H:V), and 2:1 (H:V) slope backfills, are 40, 55 and 50 pounds per cubic foot (equivalent fluid pressure), respectively. Simple surface surcharge pressures and point loads should be added to the active pressure contribution from the backfill. The geotechnical engineer should check the lateral magnitude and distribution resulting from surcharge loads.

The recommended earth pressure is calculated assuming that a drainage system will be installed behind the retaining walls, so that external water pressure will not develop.

### 6.2.2 Seismic Lateral Earth Pressure

In addition to the above-mentioned lateral earth pressures, walls more than 6 feet in height should be designed to support a seismic active pressure. The recommended seismic active pressure distribution on the retaining walls is an inverted triangular with the maximum pressure equal to 24H pounds per square foot (psf) where H is the differential wall height in feet.

### 6.2.3 Drainage

Retaining walls should be properly drained. Adequate backfill drainage is essential to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls.

Retaining walls should be appropriately waterproofed. Drainage behind the retaining walls may be provided with a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall. The drain should be placed continuously along the back of the wall and connected to a 4-inch-diameter perforated pipe. The pipe should be sloped at least 2% and surrounded by 3 cubic feet per foot of ¾-inch crushed rock wrapped in suitable non-woven filter fabric (Mirafi 140N or equivalent) or Caltrans Class 2 permeable granular filter materials without filter fabric. The crushed rock should meet the requirements defined in Section 200-1.2 of the latest edition of the Standard Specification for Public Works Construction (Public Works Standards, Inc., 2015). These drains should be connected to an adequate discharge system.

### 6.2.4 Backfill

Any retaining wall backfill material should be non-expansive (E.I. of 20 or less) and free draining. The on-site materials should be tested to verify if the material is

suitable for backfill. In lieu of the on-site materials, import of select fill meeting the expansion index requirement may be used. Wall backfill should be moisture conditioned to about 1 to 3 percent above optimum moisture content, and compacted in 8-inch lifts to at least 90 percent relative compaction (ASTM D-1557, ASTM International, 2009).

### 6.2.5 Slope Stability Analyses

Previous slope stability analyses were performed on pole locations P5 and CC CP. Based on our analyses, the pole sites critical factor of safety exceeds or is marginally in compliance with the minimum 1.5 and 1.1 for permanent static and pseudo-static conditions, respectively. The pole sites are therefore considered to be generally grossly stable. In order to provide additional resistance to slope failure, TGE recommends a deepened keyway be constructed at the toe of the retaining wall; the configuration of this keyway should be established during wall grading operations based on the subsurface conditions encountered.

### 6.2.6 Shallow Foundations

Shallow spread foundations for the retaining wall should be designed using the geotechnical design parameters presented in *Table 3* below. Footings should be designed and reinforced in accordance with the recommendations of the structural engineer and should conform to the 2013 California Building Code Part 2, Volume 2 (California Building Standards Commision, 2013).

**Table 3: Spread Footing Parameters**

| | |
|---|---|
| **Foundation Dimensions** | At least 18 inches below the lowest adjacent grade. |
| **Allowable Bearing Capacity (dead-plus-live load)** | Compacted fill(1): 3,000 pounds per square foot (psf)<br><br>The allowable bearing value may be increased by one-third for transient live loads from wind or seismic. |
| **Estimated Static Settlement (Total/Differential)** | <1-inch total & < ½-inch in 40 feet differential |
| **Allowable Coefficient of Friction** | 0.40 |
| **Allowable Lateral Passive Resistance** | Compacted fill(1): 300 pounds per cubic foot (pcf; EFP)<br><br>Native: 200 pounds per cubic foot (pcf; EFP) |

Note: (1) See Section 6.3, Site Earthwork

The total allowable lateral resistance can be taken as the sum of the friction resistance and passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance. The passive resistance values may be increased by one-third when considering wind or seismic loading.

Estimated settlements will depend on the foundation size and depth, the loads imposed, and the bearing values. For preliminary design purposes, the total settlement for spread footings for the proposed structures is estimated to be on the order of less than 1 inch.

Differential settlements will depend on the foundation size and depth, and the loads imposed. However, based on our knowledge of the project, differential settlements are anticipated to be 0.50 inches or less in 40 feet. In any case, comprehensive settlement analyses will need to be performed when detailed foundation load information is available to evaluate total and differential settlement.

Lateral loads may be resisted by friction and by the passive resistance of the supporting soils. A coefficient of friction of 0.40 may be used between foundations and compacted soil. The passive resistance of compacted fill materials may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot (pcf). A one-third increase in the passive value may be used for wind or seismic loads. The passive resistance of the materials may be combined with the frictional resistance provided the lateral bearing resistance does not exceed two-thirds of the total lateral resistance.

### 6.3 Site Earthwork

The following section is provided in connection with the grading of access roads and pole maintenance pads (Note: In addition to grading / earthwork requirements in SDG&E Specification No. TE-0101, dated May 18, 2007; *SDG&E, 2007*).

Clearing and Grubbing

Prior to grading, the project area should be cleared of all rubble, trash, debris, etc. Any buried organic debris or other unsuitable contaminated material encountered during subsequent excavation and grading work should also be removed.

Excavations for removal of any existing footings, utility lines, tanks, and any other subterranean structures should be processed and backfilled in the following manner:

1. Clear the excavation bottom and sidecuts of all loose and/or disturbed material.

2. Prior to placing backfill, the excavation bottom should be moisture conditioned to within 2 percent of the optimum moisture content and compacted to at least 90 percent of the ASTM D-1557 laboratory test standard.

3. Backfill should be placed, moisture conditioned (i.e., watered and/or aerated as required and thoroughly mixed to a uniform, near optimum moisture content), and

compacted by mechanical means in approximate 6-inch lifts. The degree of compaction obtained should be at least 90 or 95 percent of the ASTM D-1557 laboratory test standard, as applicable.

It is also critical that any surficial subgrade materials disturbed during initial demolition and clearing work be removed and/or recompacted in the course of subsequent site preparation earthwork operations.

Site Grading

In order to create uniform subgrade support conditions for the maintenance pads and access roads, the following earthwork operations are recommended.

- Prior to placing fills, the exposed soils shall be scarified a minimum of 8 inches, moisture conditioned to within 2 percent of optimum moisture content, and recompacted to a minimum of 90 percent relative compaction per ASTM D-1557.

- Fill soils shall consist of low expansive on-site/import soils with an EI of 20 or less and maximum rock size of 6 inches (Note: the upper 12-inches of pads shall not contain rocks greater than 3-inches in maximum dimension). All fills shall be compacted to a minimum 90 percent relative compaction (Note: ASTM D-1557). In addition to the relative compaction requirements, all fills shall be compacted to a firm unyielding condition.

- If materials at the bottom of receiving subgrades and/or any excavations are disturbed during construction activities, these should be removed and recompacted to a minimum 90 percent relative compaction, based on ASTM D-1557.

- Where grading is planned within sloping terrain, suitable keyways and benching should be established by the engineer prior to fill placement. The final slope face shall be densified by over-building with compacted fill and trimming back to shape with appropriate equipment.

- Import soils if required, should be sampled, tested, and approved by TGE prior to arrival on site. Imported and on-site soils shall consist of clean soils with low expansion (EI of 20 or less), free from vegetation, debris, or rocks larger than 6 inches maximum dimension.

TGE understands that fill and cut slopes will be constructed as part of the grading for the maintenance pads and access roads. Slopes constructed at a gradient of 2:1 (horizontal to vertical) are considered to be stable if constructed in accordance with the recommendations in this section. TGE should be contacted to perform a gross slope stability analysis if slopes steeper than a 2:1 are planned.

## 6.4 Seismic Design

Studies and calculations performed by SDG&E conclude that forces resulting from seismic loading are less than forces generated by wind and broken conductor loading on pole structures. Therefore, seismic ground motion does not need to be considered for design of SDG&E transmission structures.

However, seismic loads may need to be considered for any proposed earth retaining structures (i.e., CP5 and CC CP). Seismic design parameters are developed using guidelines outlined in the 2013 CBC, Volume 2, Chapter 16 (Note: 2012 International Building Code) and the JAVA ™ application, Java Ground Motion Parameter Calculator available on the USGS website (http://earthquake.usgs.gov; USGS, 2016 ). The preliminary seismic design parameters for the project sites are presented in Table 4 below.

**Table 4: 2012 IBC Seismic Design Parameters**

| Vault No. | Site Class | $S_{DS}$ (g) | $S_{D1}$ (g) |
|---|---|---|---|
| P5 | D | 0.683 | 0.396 |
| CC CP | D | 0.759 | 0.431 |

## 6.5 Soil Corrosion

The corrosion potential of the on-site materials to steel and buried concrete was evaluated from the previous geotechnical studies. Results are presented in Appendix A and B. General recommendations to address the corrosion potential of the on-site materials are provided below. If additional recommendations are desired, it is recommended that a corrosion specialist be consulted.

### 6.5.1 Reinforced Concrete

Laboratory tests indicate that the potential of sulfate attack on concrete in contact with the structure site soils are “Not Applicable” to marginally “Moderate” based on ACI 318-11, Table 4.2.1 (American Concrete Institute, 2011). It is recommended that Type V cement to be used for all proposed steel pole sites. It is further recommended that at least a 4.0-inch thick concrete cover be maintained over the reinforcing steel where possible for concrete in contact with the soil for non-wet holes and increase to 6.0-inches for wet holes.

The results of chloride content testing at the near-surface soil indicate the potential of chloride attack on concrete structures is low. Reinforcing steel in concrete structures and pipes in contact with soil are not considered to be susceptible to chloride attack; however, TGE recommends that the level of protection should anticipate a chloride content of 200 ppm. The pH-value is marginally near-neutral

and may warrant corrosion consideration. If considered necessary, possible methods of protection that could be used include increased concrete cover, low water-cement ratio, corrosion inhibitor admixture, silica fume admixture, waterproof coating on the concrete exterior.

### 6.5.2 Metallic

Laboratory tests indicate that the soils have very low electrical resistivity, which presents a very high potential for corrosion to buried ferrous metals. This is considered to be the worst case for all poles located within alluvial soils. Therefore, metallic elements should be reviewed by a Corrosion Engineer for the proposed steel pole structures

## 7. CONSTRUCTION CONSIDERATIONS

Engineered Foundations

1. The foundation excavation should be observed by the engineer during excavation to confirm anticipated conditions and verify construction is performed per IFC plans and specifications.
2. Groundwater is not anticipated within the proposed pole locations. However, periodic ground water seepage zones may occur along geologic contacts.
3. Foundation excavation within alluvial materials may require temporary casing.
4. The contractor should anticipate variable drilling conditions within the formational materials. The contractor should also anticipate the need for soft and hard rock drilling techniques to extend the drilled piers to the specified tip elevations. The amount of drilling difficulty experienced by the contractor will vary with the methods used.

Access Roads / Maintenance Pads

1. Seismic refraction surveys were conducted along the proposed project alignment to evaluate the underlying subsurface material conditions. Based on this information the pole sites are underlain by soil to very hard rock. In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogeneous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The seismic P-wave velocity ranges presented in Table 5 are based on TGE's experience with similar materials. The rippability of the mass can be classified on a scale of "Easy" to "Blasting Generally Required", as presented here in Table 5 below. The rippability classifications are also dependent on the excavation equipment used and the skill of the equipment operator. Caterpillar D-9 Dozer ripping with a single shank. The rippability values and classifications are considered approximate and that rock characteristics, such as depth, orientation and fracturing, have an effect in determining the rock rippability.

**Table 5: Rippability Classification**

| Seismic P-wave Velocity | Rippability | Soil/Rock Description |
|---|---|---|
| 0 to 2,000 ft/sec | Easy | Soil, loose to medium dense |
| 2,000 to 3,000 ft/sec | Moderate | Soil, medium dense |
| 3,000 to 4,000 ft/sec | Difficult, Possible Local Blasting | Soil, dense to very dense<br>Rock, very weak to weak |
| 4,000 to 8,000 ft/sec | Very Difficult, Probable Local to General Blasting | Rock, medium strong to strong |
| Greater than 8,000 ft/sec | Blasting Generally Required | Rock, very strong |

*Table 5* above may be used as a basis for preliminary evaluations of excavatability when utilizing the Seismic Refraction Survey Data.

## 8. LIMITATIONS

The recommendations and opinions expressed in this report are based on TGE's review of background documents and on information developed during this study. More detailed limitations of the geotechnical engineering report are presented in the ASFE's information bulletin in *Appendix C*.

Due to the limited nature of our study, conditions not observed and described in this report may be present at the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during substation expansion construction operations.

Site conditions, including ground-water level, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which TGE has no control.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. TGE should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

TGE has endeavored to perform this assessment using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience

in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this assessment.

## 9. REFERENCES

1. ASTM International, ASTM D1557-09, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft$^3$), 2009;

2. American Concrete Institute, Building Code Requirements for Structural Concrete (ACI 318-11), 2011;

3. California Building Standards Commission, 2013, California Building Code, Title 24, Part 2, Vol. 2 (http://www.bsc.ca.gov/Codes.aspx);

4. San Diego Gas & Electric Company, Construction Specification Site Preparation and Access Roads – Specification No. TE-0101, May 18, 2007;

5. FEMA, Flood Insurance Rate Map, San Diego County, CA, Map Number 06073C1363G, Panel 1363, May 16, 2012;

6. Public Works Standards, Inc., The Greenbook – Standard Specifications for Public Works Construction, BNI Publications, Inc., 2015;

7. USGS Seismic Design Maps (https://earthquake.usgs.gov/designmaps/us/application.php), 2016.

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# FIGURES


References: USGS 2015 Topographic Map (Del Mar, El Cajon, La Jolla, La Jolla OE W, CA - 7.5 minute quadrangle)

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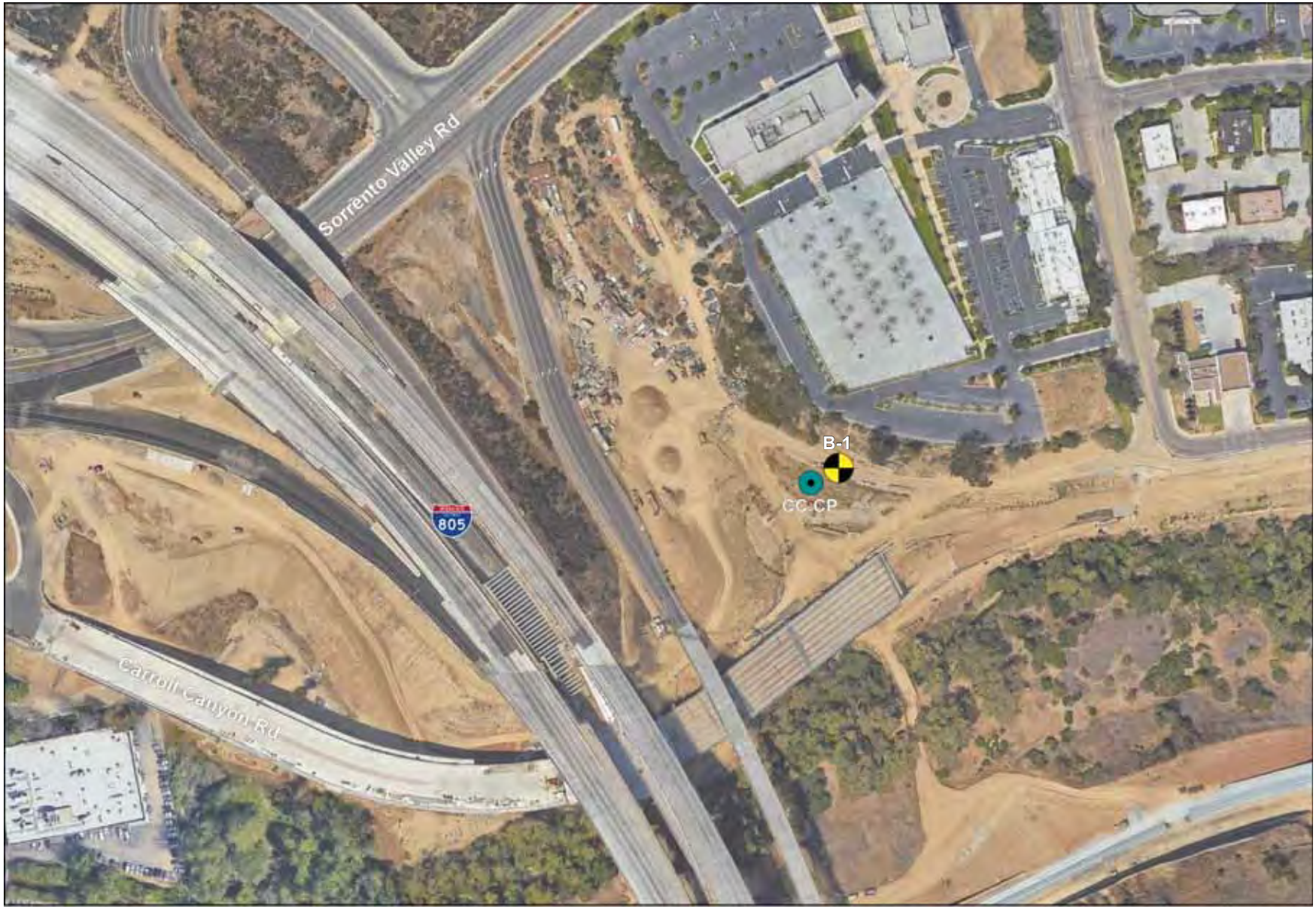
## Legend

- Proposed Alternate 5 Alignment
- Proposed Pole Locations

# Vicinity Map

**SDG&E 230 kV Transmission Line**
**Sycamore to Penasquitos**

| TGE Project No.: T-0126-G | Figure No.: 1 |
|---|---|




References: Google Earth

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**Legend**

- Approximate Boring Location (TGE, 7/27/2016)
- Proposed Pole Location

# Plot Plan

**SDG&E 230kV Transmission Line**
**Sycamore to Penasquitos**

| TGE Project No.: T-0126-G | Figure No.: 2 |
|---|---|


Reference: Google Earth

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**Legend**

- Approximate Seismic Line Location (TGE, 7/27/2015)
- Proposed Pole Location

# Plot Plan

**SDG&E 230kV Transmission Line**
**Sycamore to Penasquitos**

| TGE Project No.: T-0126-G | Figure No.: 3 |
|---|---|


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GEOTECHNICAL ENGINEERING, INC.

**Legend**

- Approximate Seismic Line Location (TGE, 4/15/2015)
- Approximate Trench Location (Geocon, 9/12/2012)
- Proposed Pole Location

# Plot Plan

**SDG&E 230kV Transmission Line**
**Sycamore to Penasquitos**

| TGE Project No.: T-0126-G | Figure No.: 4 |
|---|---|


TRINITY
GEOTECHNICAL ENGINEERING, INC.

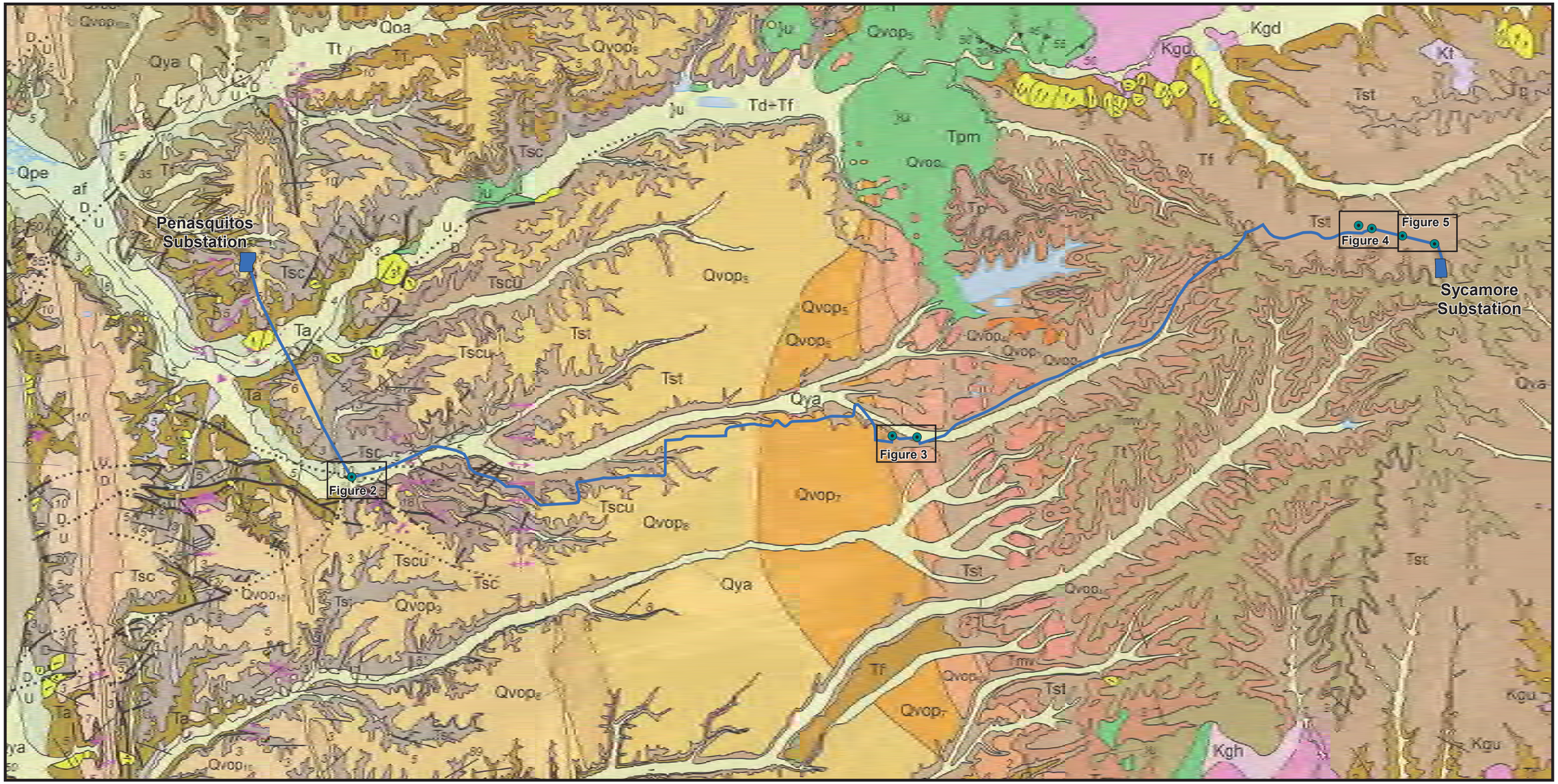
**Legend**

- Approximate Boring Location (Geocon, 9/12/2012)
- Proposed Pole Location

# Plot Plan

**SDG&E 230kV Transmission Line**
**Sycamore to Penasquitos**

| TGE Project No.: T-0126-G | Figure No.: 5 |
|---|---|




## Legend

- Proposed Alternate 5 Alignment
- Proposed Pole Locations

- Qya - Young Alluvial Flood-Plain Deposits
- Qop - Old Paralic Deposits
- Qvop - Very Old Paralic Deposits
- Tsc - Scripps Formation (Tscu - upper unit)
- Tst - Stadium Conglomerate
- Tf - Friars Formation (non-marine claystone and sandstone)
- Tmv - Mission Valley Formation (marine sandstone with cobble conglomerate lenses)

TRINITY GEOTECHNICAL ENGINEERING, INC.

# Regional Geology Map

**SDG&E 230 kV Transmission Line**
**Sycamore to Penasquitos**

| TGE Project No.: T-0126-G | Figure No.: 6 |
|---|---|

TRINITY
GEOTECHNICAL ENGINEERING, INC.

# APPENDIX A

## Previous Geotechnical Information By TGE

# Supplemental Geotechnical Study

Sycamore to Penasquitos
230kV Transmission Line
Alternates 4 and 5
San Diego County, California

**Prepared for:**

BURNS McDONNELL

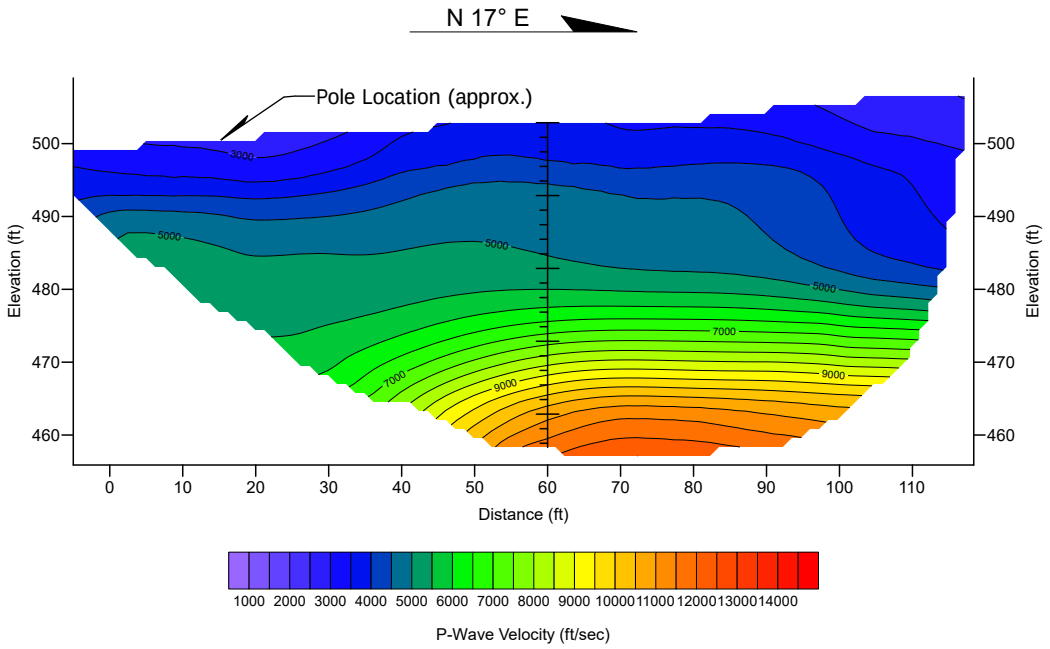
**Attention: Mr. Kevin Mathey, PE**

**Project No.: T-0126-G**

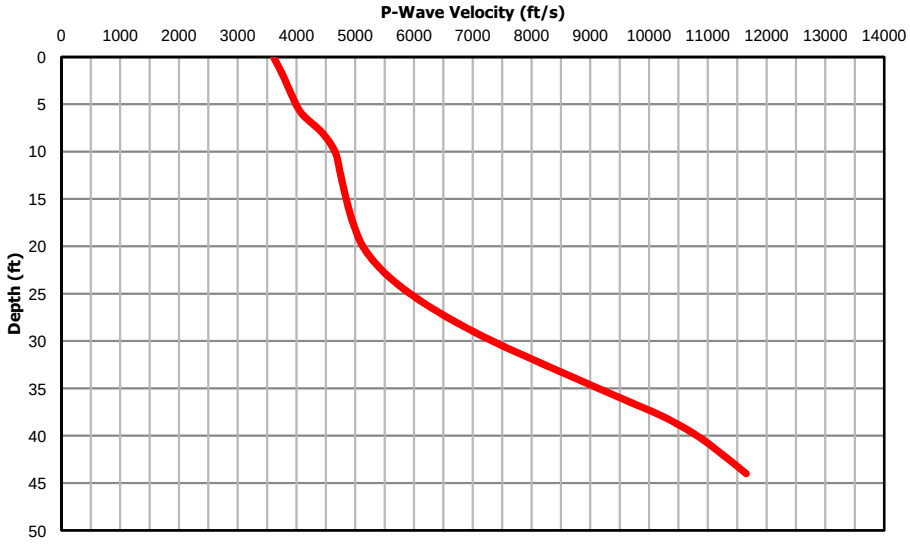
**July 27, 2016**

**TRINITY Geotechnical Engineering, Inc.**
**13230 Evening Creek Drive, Suite 206**
**San Diego, CA 92128**

## Seismic Refraction Profile




## Pole Site $V_p$ Profile




TRINITY GEOTECHNICAL ENGINEERING, INC.

**SL-2 (I-15W CP)**

**SDG&E 230 kV Transmission Line**
**Sycamore to Penasquitos**

**TGE Project No.:** T-0126-G | **Figure No.:** A-2

# Boring Log B-1



**Date Drilled:** 3/30/16 | **Logged By:** CC

**Exploratory Equipment:** CME 75 | **Approximate Surface Elevation:** 152 feet above MSL

**Driving Weight:** 140 lb auto hammer | **Total Depth of Boring:** 45.0 feet

**Drilling Method:** HSA (8.5"/4.25") | **Groundwater Elevation During Drilling:** Not encountered

| Depth in Feet | Driven Sample | Blow Counts | N value | Material Description | USCS | Color | Consistency | Moisture | Time (Min:Sec) | Dry Density (pcf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | **Fill -** Sandy SILT, abundance of angular and rounded gravel | ML | brown | soft | dry to moist | | |
| | | 9 13 19 | 19 | | | | very stiff | | | 112.1 |
| 5 | | 8 20 20 | 24 | Bits of asphalt, minor gravel | | | | | | 101.9 |
| | | 7 10 18 | 17 | Clayey SILT | CL/ML | yellow brown/ mottled gray | | | | 98.4 |
| 10 | | 11 14 15 | 17 | No return | | | | | | |
| | | 8 13 18 | 19 | | | | | moist to very moist | | 107.4 |
| 15 | | 7 10 15 | 15 | | | | | | | 97.9 |
| | | 5 8 14 | 13 | Silty CLAY, some clasts with orange $Fe_2O_3$ staining | | gray | stiff | | | 101.7 |
| 20 | | 6 8 12 | 12 | | | | | | | 120.7 |
| 25 | | 7 18 30 | 29 | Silty SAND (fine to medium grained) with silty clasts (very fine to fine grained) | SM/SC | orange-brown/ gray clasts | medium dense | | | 98.2 |
| | | 19 50/6" | 50+ | **Ardath Shale (Ta) -** Claystone, micaceous, some platy jointing, some orange $Fe_2O_3$ staining | | gray | very dense | | | 104.6 |
| 35 | | 18 50/6" | 50+ | | | | | | | 101.7 |
| 40 | | 22 41 50/4" | 50+ | | | | | | | 103.5 |
| 45 | | 26 50/3" | 50+ | | | | | | | 104.8 |

Moisture Content (% Dry Weight): 10 20 30 40 50

**End of boring at 45.0 feet**

Note: 1. Groundwater not encountered

TRINITY GEOTECHNICAL ENGINEERING, INC.

**B-1 (CC-CP)**

**SDG&E 230 kV Transmission Line**
**Sycamore to Penasquitos**

**TGE Project No.:** T-0126-G | **Figure No.:** B-3

# Laboratory Test Results

## In-Situ Moisture Content and Dry Density

The in-situ moisture content and dry density of the soils were determined in accordance with ASTM D-2216 and ASTM D-2937 laboratory test methods, respectively. The moisture content method involves obtaining the moist weight of the sample and then drying the sample to obtain its dry weight. The moisture content is then calculated by taking the difference between the wet and dry weights, dividing it by the dry weight of the sample, and expressing the result as a percentage. Dry density is calculated by dividing the dry weight by the total volume expressed in pounds per cubic foot (Note: test performed on relatively undisturbed samples only). The results of the in-situ moisture content and dry density tests are presented in the table below and in *Appendix A, Exploratory Boring Log*:

Table 1: Moisture Content and Dry Density Test Results (ASTM D-2216 & D-2937)

| Location | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|
| B-1 @ 3’ | 13.3 | 112.1 |
| B-1 @ 5’ | 13.2 | 101.9 |
| B-1 @ 8’ | 20.4 | 98.4 |
| B-1 @ 13’ | 20.8 | 107.4 |
| B-1 @ 15’ | 20.9 | 97.9 |
| B-1 @ 18’ | 23.7 | 101.7 |
| B-1 @ 20’ | 15.4 | 120.7 |
| B-1 @ 25’ | 26.4 | 98.2 |
| B-1 @ 30’ | 22.9 | 104.6 |
| B-1 @ 35’ | 22.5 | 101.7 |
| B-1 @ 40’ | 24.3 | 103.5 |
| B-1 @ 45’ | 21.2 | 104.8 |

## Particle Size Analyses

In accordance with ASTM D-422, quantitative determinations of the distribution of coarse-grained particle sizes in selected samples were made. Mechanically actuated sieves were utilized for separating the various classes of coarse-grained (gravel and sand) particles. For soil samples containing fine-grained particle sizes, additional testing was conducted in accordance with ASTM D-1140 to determine the fines content (i.e., soil passing a No. 200 Sieve). The sieve analysis test results are provided in the table below:

Table 2: Sieve Analysis Test Results (ASTM D-422 & D-1140)

| Sieve Size | B-1 @ 3-5' Percent Passing |
|---|---|
| 2 in | 100 |
| 1 in | 100 |
| ¾ in | 100 |
| ½ in | 97 |
| 3/8 in | 93 |
| ¼ in | 90 |
| #4 | 87 |
| #8 | 80 |
| #10 | 79 |
| #16 | 75 |
| #30 | 70 |
| #40 | 67 |
| #50 | 64 |
| #100 | 59 |
| #200 | 54 |
| **Classification** | ML |

## Direct Shears

Direct shear tests were performed on relatively undisturbed soil samples in accordance with ASTM D-3080 to evaluate the shear strength characteristics of the in-situ materials. The test method consists of placing the soil sample in the direct shear device, applying a series of normal stresses, and then shearing the sample at a constant rate of shearing deformation. The shearing force and horizontal displacements are measured and recorded as the soil specimen is sheared. The shearing is continued well beyond the point of maximum stress until the stress reaches a constant or residual value. Final test results are presented in the table below:

Table 3: Direct Shear Test Results (ASTM D-3080)

| Location | Apparent Cohesion (psf) | Friction Angle (degrees) |
|---|---|---|
| B-1 @ 8' | 400 | 31 |
| B-1 @ 15' | 60 | 39 |
| B-1 @ 25' | 80 | 40 |

## Corrosion Tests

Chemical analytical tests were performed on a bulk soil sample collected during the field exploration program to evaluate the corrosion potential of the on-site materials. These tests were performed in accordance with California Test Method Nos. 417 (sulfate), 422 (chloride), and 532/643 (pH and resistivity). The results of the tests are summarized below:

Table 4: Corrosion Test Results (CTM Nos. 417, 422, 532 & 643)

| Location | Depth (feet) | pH | Resistivity (ohm-cm) | Chloride Content (ppm) | Sulfate Content (ppm) |
|---|---|---|---|---|---|
| B-1 | 3-5 | 8.8 | 500 | 150 | 310 |

# Geotechnical Study

Sycamore to Penasquitos
230kV Transmission Line
San Diego County, California

**Prepared for:**

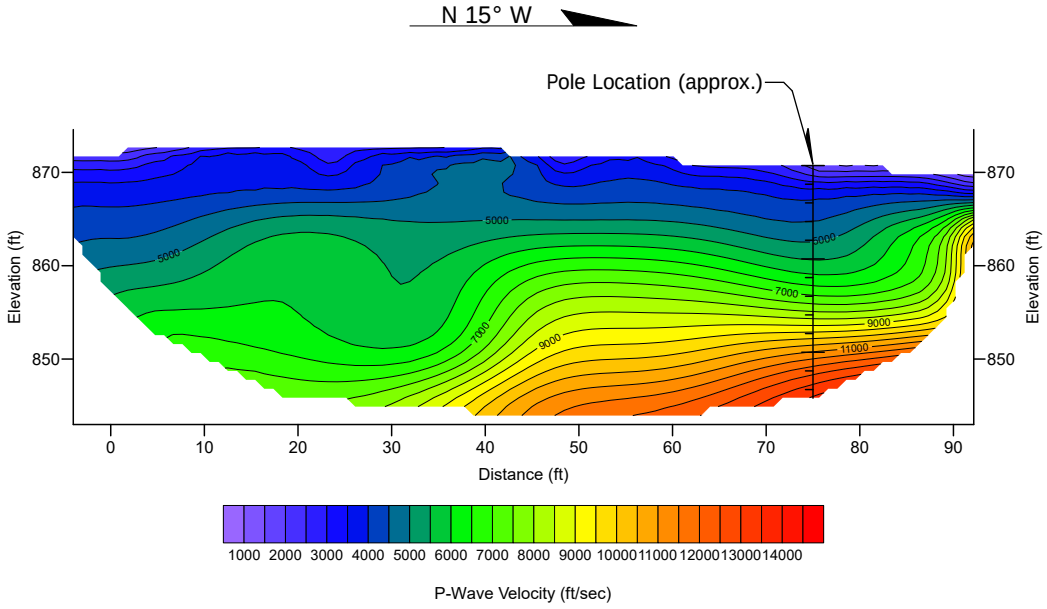
**Attention: Mr. Kevin Mathey, PE**

**Project No.: T-0126-G**

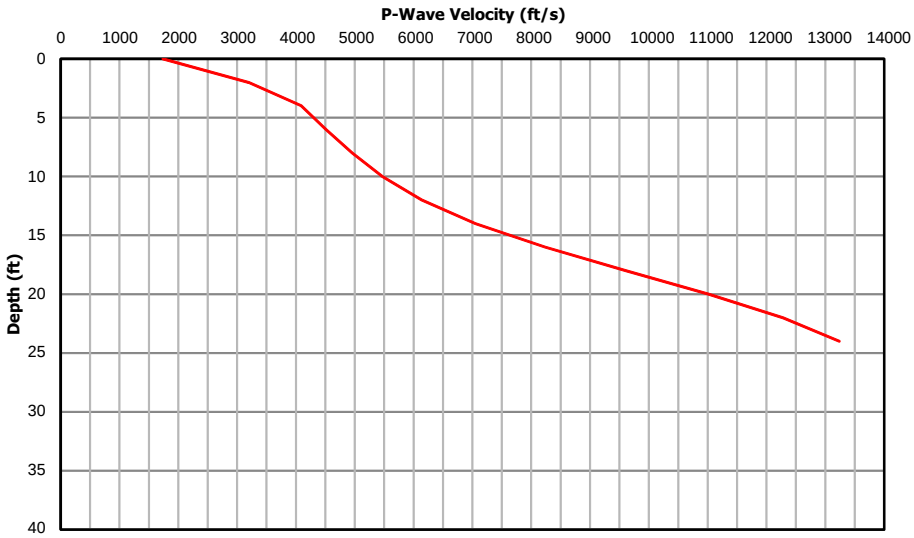
**April 15, 2015**

**TRINITY Geotechnical Engineering, Inc.**
**13230 Evening Creek Drive, Suite 206**
**San Diego, CA 92128**

## Seismic Refraction Profile




## Pole Site $V_p$ Profile





**Trinity Geotechnical Engineering, Inc.**
13230 Evening Creek Drive
Suite 206
San Diego, CA 92128
Phone: 858.486.2888


# SL-1 (P-6)

**SDG&E 230kV Transmission Line**
**Sycamore to Penasquitos**

| Project No.: T-0126-G | Figure No.: A-1 |
|---|---|

TRINITY
GEOTECHNICAL ENGINEERING, INC.

# APPENDIX B

## Previous Geotechnical Information By Others

# GEOTECHNICAL INVESTIGATION

## SDG&E TL6961
## POLE FOUNDATIONS
## M.S.A. 6160015454
## SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

SAN DIEGO GAS AND ELECTRIC COMPANY
SAN DIEGO, CALIFORNIA

SEPTEMBER 12, 2012
PROJECT NO. G1115-32-39

Project No. G1115-32-39
September 12, 2012

San Diego Gas and Electric Company
Civil/Structural Engineering
8316 Century Park Court
San Diego, California 92123

Attention: Mr. Tyler Lonsdale

Subject: GEOTECHNICAL INVESTIGATION
SDG&E TL6961 POLE FOUNDATIONS
M.S.A. 6160015454
SAN DIEGO, CALIFORNIA

Dear Mr. Lonsdale:

In accordance with your authorization of our Proposal No. LG-12131 dated May 9, 2012, we herein submit the results of our geotechnical investigation for the power pole foundations proposed along the subject transmission line. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed improvements. The pole locations are suitable for foundations provided the recommendations of this report are followed.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

Troy K. Reist
Troy K. Reist
CEG 2408

Joseph J. Vettel
GE 2401

PROFESSIONAL GEOLOGIST
TROY K. REIST
No. 2408
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JOSEPH J. VETTEL
No. 2401
EXP. 09/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

TKR:JJV:dmc

(2) Addressee

PROJECT NO. 05545-22-30

# BORING B 1

ELEV. (MSL.) _____ DATE COMPLETED 5/26/98

EQUIPMENT 30" BUCKET AUGER

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | |
| 2 | B1-1 | | | SM | **POMERADO CONGLOMERATE** Very dense, damp, light brown, Silty, fine to medium SANDSTONE C= 300 Φ=33° | 3/7" | 100.0 | 21.0 |
| 4 | B1-2 | | | SM | Very dense, damp, brown, Silty, fine to coarse SAND matrix with 30% gravel and cobbles to 6 inches | | | |
| 6 | | | | | | | | |
| 8 | | | | | | | | |
| 10 | B1-3 | | | SM | Very dense, damp, light brown, Silty, fine to coarse medium SANDSTONE C=850 Φ=35° | 8 | 118.8 | 11.7 |
| 12 | | | | | | | | |
| 14 | | | | | Very dense, brown, Silty, fine to coarse SAND matrix with 30% gravel and cobbles to 6 inches | | | |
| 16 | | | | | | | | |
| 18 | B1-4 | | | SM | | | | |
| 20 | | | | | | | | |
| 22 | | | | | | | | |
| 24 | | | | | -Boulder (24") at 23 feet to 25 feet | | | |
| 26 | B1-5 | | | | **STADIUM CONGLOMERATE** | | | |
| 28 | | | | SC | Very dense, damp, brown, Clayey, fine to coarse SAND matrix with 40% gravel and cobbles to 8" | | | |
| 30 | | | | | BORING TERMINATED AT 30 FEET | | | |

**Figure A-2 Log of Boring B 1, page 1 of 1**

SPPS

SAMPLE SYMBOLS: ☐ ... SAMPLING UNSUCCESSFUL; ▨ ... DISTURBED OR BAG SAMPLE; ◧ ... STANDARD PENETRATION TEST; ◪ ... CHUNK SAMPLE; ■ ... DRIVE SAMPLE (UNDISTURBED); ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06355-42-02

# BORING LB 4

ELEV. (MSL.) 852 DATE COMPLETED 2/8/02

EQUIPMENT SOIL MEC. R-208 30"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | GC | **TOPSOIL** Loose, medium dense, dry, medium to dark yellow-brown, very Clayey, fine to medium GRAVEL | | | |
| 2 | | | | GM | | | | |
| 4 | | | | SM-GM | **STADIUM CONGLOMERATE** Very dense, humid, light brown, Sandy, medium to coarse CONGLOMERATE; massive, with 40 to 50% cobble, 1" to 3" diameter | | | |
| 6 | | | | SM | Very dense, damp, light brown-tan, very Gravelly SANDSTONE ~20 to 30% cobble, 2" to 5" diameter | | | |
| 8 | | | | | Dense, damp, light brown-tan, Silty, fine to medium SANDSTONE | | | |
| 10 | | | | | Very dense, damp, medium olive-brown to yellow-brown, Sandy, massive, coarse CONGLOMERATE ~40 to 50% 3" to 8" diameter cobble, some clay | | | |
| 12 | LB4-1 | | | | | | | |
| 14 | | | | | | | | |
| 16 | | | | | | | | |
| 18 | | | | | | | | |
| 20 | | | | | | | | |
| 22 | | | | | | | | |
| 24 | | | | | | | | |
| 26 | | | | | | | | |
| 28 | | | | | -Becomes more dense, with ~50% 3" to 8" diameter cobble | | | |

max 126.8 @10.6
Remolded shear 114.3 @ 10.3
C = 561 Φ = 36°

Sulfate = 0.007 %wt

**Figure A-9, Log of Boring LB 4**

SYCES

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ⧩ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06355-42-02

## BORING LB 4

ELEV. (MSL.) 852 DATE COMPLETED 2/8/02

EQUIPMENT SOIL MEC. R-208 30"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 30 | | | | | Sandstone layer - approximately horizontal 8" to 10" thick | | | |
| 32 | | | | | Very dense, damp, medium brown, Sandy, coarse CONGLOMERATE<br>40 to 50% cobble 3" to 8", some clay | | | |
| 34 | | | | GM | | | | |
| 36 | | | | | | | | |
| 38 | | | | | 8" to 10" thick approximately horizontal sandstone layer | | | |
| 40 | | | | | | | | |
| 42 | | | | | | | | |
| 44 | | | | | | | | |
| 46 | | | | | BORING TERMINATED AT 47 FEET | | | |

**Figure A-10, Log of Boring LB 4**

SYCES

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

▨ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▽ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

## TABLE B-IV
## SUMMARY OF LABORATORY SOLUBLE SULFATE TEST RESULTS

| Sample No. | Sulfate (% SO4) | Sulfate Exposure | Soil Type or Formation |
|---|---|---|---|
| LB1-2 | .002 | Negligible | Stadium Conglomerate |
| LB4-1 | .007 | Negligible | Stadium Conglomerate |
| LB5-1 | .006 | Negligible | Pomerado Conglomerate |
| T78-1 | .005 | Negligible | Colluvium |
| T88-1 | .009 | Negligible | Alluvium |
| T103-1 | .014 | Negligible | Pomerado Conglomerate |

## TABLE B-V
## SUMMARY OF LABORATORY TEST RESULTS FOR Ph AND RESISTIVITY

| Sample No. | pH | Resistivity | Soil Type or Formation |
|---|---|---|---|
| T103-1 | 4.13 | 506.9 | Pomerado Conglomerate (Tank Site) |

## TABLE B-VI
## SUMMARY OF LABORATORY TEST RESULTS FOR SAND EQUIVALENT

| Sample No. | Sand Equivalent (SE) | Soil Type or Formation |
|---|---|---|
| LB7-1 | 17 | Pomerado Conglomerate |
| LB8-1 | 13 | Pomerado Conglomerate |
| LB9-1 | 15 | Stadium Conglomerate |
| LB10-1 | 18 | Stadium Conglomerate |

## TABLE B-VII
## SUMMARY OF LABORATORY TEST RESULTS FOR SPECIFIC GRAVITY

| Sample No. | Specific Gravity | Soil Type or Formation |
|---|---|---|
| LB7-1 | 2.533 | Pomerado Conglomerate |
| LB8-1 | 2.626 | Pomerado Conglomerate |
| LB9-1 | 2.597 | Stadium Conglomerate |
| LB10-1 | 2.634 | Stadium Conglomerate |

PROJECT NO. 06160-42-03

## TRENCH T 28

ELEV. (MSL.) 875 DATE COMPLETED 7/23/99

EQUIPMENT JD 555 TRACKHOE 24"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | |
| | | | | CL-SC | **TOPSOIL**<br>Stiff, damp, reddish-brown, Gravelly, Sandy CLAY to Clayey SAND | | | |
| 2 | | | | | | | | |
| | | | | SM | **STADIUM CONGLOMERATE**<br>Very dense, damp, light brown-tan, Silty, fine SANDSTONE<br>-With some clay and clay-lined seams | | | |
| 4 | | | | | | | | |
| 6 | | | | | | | | |
| | | | | GM | Very dense, damp, light brown, Sandy, coarse CONGLOMERATE; with some silt | | | |
| 8 | | | | | TRENCH TERMINATED AT 8 FEET | | | |

**Figure A-35, Log of Trench T 28**

MONTE

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▨ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▽ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06160-42-03

# TRENCH T 29

ELEV. (MSL.) 830 DATE COMPLETED 7/23/99

EQUIPMENT JD 555 TRACKHOE 24"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | |
| | | | | | **TOPSOIL**<br>Loose, dry, dark brown, very Gravelly-Silty SAND | | | |
| 2 | | | | GM | **STADIUM CONGLOMERATE**<br>Very dense, humid, light tan-white, cemented, Sandy, coarse CONGLOMERATE; with some silt<br>-Marginal to nonrippable | | | |
| 4 | | | | | | | | |
| | | | | | TRENCH TERMINATED AT 5 FEET | | | |

**Figure A-36, Log of Trench T 29**

MONTE

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

▨ ... DISTURBED OR BAG SAMPLE ◺ ... CHUNK SAMPLE ▽ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06160-42-03

## TRENCH T 30

ELEV. (MSL.) 835 DATE COMPLETED 7/23/99

EQUIPMENT JD 555 TRACKHOE 24"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | GM | **TOPSOIL**<br>Loose, dry, dark red-brown, Sandy, coarse GRAVEL; with some silt | | | |
| 2 | | | | GM | **STADIUM CONGLOMERATE**<br>Very dense, damp, light brown to tan, Sandy, coarse CONGLOMERATE; with some silt<br>-Massive, with imbricated clasts and moderate cemented | | | |
| 4 | | | | | | | | |
| 6 | | | | | | | | |
| | | | | | TRENCH TERMINATED AT 7 FEET | | | |

**Figure A-37, Log of Trench T 30**

MONTE

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ⧗ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06160-42-03

## BORING B 2

ELEV. (MSL.) 870 DATE COMPLETED 7/27/99

EQUIPMENT ROTARY BUCKET 30"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **STADIUM CONGLOMERATE**<br>Very dense, humid, light brown to tan, Sandy, coarse CONGLOMERATE; with some silt | | | |
| 2 | | | | | | | | |
| 4 | | | | | | | | |
| 6 | | | | GM | | | | |
| 8 | | | | | | | | |
| 10 | | | | | | | | |
| 12 | | | | | | | | |
| 14 | B2-1 | | | SM | Dense, damp, light tan to gray, Silty, fine SANDSTONE<br>-N 60 E, 10 S BEDDING | 5 | 118.8 | 10.7 |
| 16 | B2-2 | | | | max 120.2 e 12.8<br>Remolded shear 107.4 e 13.7 = 90%<br>C = 814 Φ = 27° | | | |
| 18 | | | | | EI = 9.1% 23.0% 113.2 EI=2 | | | |
| | | | | | -Horizontal contact | | | |
| 20 | | | | | Very dense, damp, light brown, Sandy, coarse CONGLOMERATE; with some silt | | | |
| 22 | | | | GM | | | | |
| 24 | | | | | | | | |
| 26 | | | | | | | | |
| 28 | | | | | Very dense, moist, medium olive-brown, Sandy, coarse CONGLOMERATE<br>-With some clay | | | |

**Figure A-4, Log of Boring B 2**

MONTE

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06160-42-03

## BORING B 2

ELEV. (MSL.) 870 DATE COMPLETED 7/27/99

EQUIPMENT ROTARY BUCKET 30"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 30 | B2-3 | | | GM | | | | |
| 32 | | | | | | | | |
| 34 | | | | | | | | |
| 36 | | | | | Very dense, damp, light brown, Sandy, coarse CONGLOMERATE; with some silt | | | |
| 38 | | | | | | | | |
| 40 | | | | | | | | |
| 42 | | | | GM | -Cemented zone 2 feet thick at 42 feet; with strong calcium carbonate cementation | | | |
| 44 | | | | | | | | |
| 46 | | | | | -8" sandstone layer at 47 feet | | | |
| 48 | | | | | | | | |
| 50 | B2-4 | | | SM | Dense, damp, light gray-tan, Silty, fine SANDSTONE | 10 | 118.4 | 14.6 |
| 52 | | | | | | | | |
| 54 | B2-5 | | | GM | Very dense, damp to moist, medium brown, Sandy, coarse CONGLOMERATE<br>-With some clay and silt<br>-12" cemented layer | | | |
| 56 | | | | | | | | |
| 58 | | | | | | | | |

**Figure A-5, Log of Boring B 2**

MONTE

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

▨ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06160-42-03

## BORING B 2

ELEV. (MSL.) 870 DATE COMPLETED 7/27/99

EQUIPMENT ROTARY BUCKET 30"

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 60 | | | | | Very dense, damp, medium brown, Sandy, coarse CONGLOMERATE<br>-With interbedded thin sandstone bed, with some clay | | | |
| 62 | | | | | | | | |
| 64 | | | | | | | | |
| 66 | | | | GM | | | | |
| 68 | | | | | | | | |
| 70 | B2-6 | | | | 12"-24" SANDSTONE layer | 30/11" | 117.0 | 12.9 |
| 72 | | | | | | | | |
| 74 | | | | SM | Dense, damp, light gray to tan, Silty, fine SANDSTONE | | | |
| 76 | | | | | | | | |
| 78 | | | | | -Horizontal contact | | | |
| 80 | | | | | Very dense, damp, medium brown, Sandy, coarse CONGLOMERATE<br>-With silt and clay | | | |
| 82 | | | | | | | | |
| 84 | | | | | | | | |
| 86 | | | | | -Clasts up to 12" diameter | | | |
| 88 | B2-7 | | | GM | | | | |

**Figure A-6, Log of Boring B 2**

MONTE

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ⧩ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

TRINITY
GEOTECHNICAL ENGINEERING, INC.

# APPENDIX C

## ASFE Important Information About Your Geotechnical Engineering Report

# Important Information About Your

# Geotechnical Engineering Report

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

*The following information is provided to help you manage your risks.*

## 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 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 your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the 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.

## A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of 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,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- 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 study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the 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 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 construction recommendations included in your report. *Those 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 recommendations if that engineer does not perform construction observation.*

## A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower 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. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing 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 Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors 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 contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors 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 contractors do not 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.

## Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental 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 geoenvironmental 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, a number of 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 ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

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